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5-1 Roadway and Structure Geometrics and Drainage

Roadway and structure pavement drainage should be considered early in the design stage while the roadway geometry is still being developed. The hydraulic capacity of gutters and inlets is determined by the longitudinal slope and superelevation of the pavement and minor changes to roadway or structure geometrics are more easily accomplished early in the design phase. The imperviousness of the roadway pavement will result in significant runoff from any rainfall event. To ensure safety to the traveling public, careful consideration must be given to removing the runoff from the roadway through structure pavement drainage facilities.

A roadway with a gutter section should normally be placed at a minimum longitudinal slope of 0.3 percent to 0.5 percent to allow for reasonable drainage. The flatter slopes may be used with wider shoulders and the 0.5 percent should be used as a minimum for narrow shoulders. Superelevation and/or widening transitions can create a gutter profile far different from the centerline profile. The designer must carefully examine the geometric profile of the gutter to eliminate the formation of sumps or “birdbaths” created by these transitions. These areas should be identified and eliminated. This generally requires geometric changes stressing the need for early consideration of drainage.

Improperly placed superelevation transitions can cause serious problems especially on bridges. As discussed in Section 5-4, inlets or other means must pick up gutter flow before the flow crosses over to the other side of the pavement. The collection of crossover flow on bridges is very complex as effective drain inlets are difficult to place within structure reinforcement. Bridges over waterways and wetlands pose water quality issues as well and drop drains may not be allowed. Also, bridge drain downspouts have a history of plugging problems and are an objectionable aesthetic impact on the structure.

Eliminating inlets on bridges can usually be accomplished by considering drainage early in the design phase. Superelevation transitions, zero gradients, and sag vertical curves should be avoided on bridges. Modern bridges generally use watertight expansion joints so that all surface water can be run off of the structure and collected in inlets placed at the bridge ends.

Drainage design at bridge ends requires a great deal of coordination between the Region designer, Bridge designer, and the Head Quarters (HQ) Hydraulics Office. In many areas, the drainage plan may include the bridge. The designer is responsible for drainage upstream and downstream of the bridge. HQ is responsible for bridge drainage and coordination necessary at the downstream end of the bridge.

Multi-lane highways create unique drainage situations. The number of lanes draining in one direction should be considered during the design phase. The Geometric Cross Section chapter in the WSDOT *Design Manual* is a good reference when designing drainage for multi-lane highways. The Region Hydraulics Engineer and HQ Hydraulics Office is also available to provide design guidance. See section 5-6 for discussion on hydroplaning and hydrodynamic drag and how to reduce its potential.

5-2 Computing Runoff for Highway Pavements

The runoff for highway pavements is computed by the rational method (see Chapter 2). This method is easy to use for pavement drainage design because the time of concentration is generally taken as 5 minutes. The recurrence interval for typical pavement drainage design is the 10-year rainfall event. When roadways are placed in a sump, the recurrence interval shall change to the 50-year rainfall event.

5-3 Rural Highway Drainage

When rural highways are built on a fill, roadway drainage is usually allowed to flow, uncollected, to the sides of the roadway and over the side of the fill slope. Usually, this sheet flow of highway drainage does not present any problem to adjacent property owners nor is it a threat to the highway fill. This type of drainage is allowed for fills up to 25 ft (7.5 m). A curb should be used in highly erosive soils when the fill is high enough to justify the use of a guardrail.

Fill heights greater than 25 ft (7.5 m) may present an erosion threat to the embankment especially where the roadway forms a sag vertical curve. This problem is usually present immediately after construction and before vegetation is established. In these situations, it may be prudent to construct a curb and gutter to collect the sheet flow from the pavement and discharge this flow through a runoff treatment or flow control Best Management Practice (BMP). The treated and controlled runoff can then be discharged into an established stream or a low spot in the surrounding terrain. Selection of an appropriate BMP is dependent on the characteristics of the project site. Designers should reference the *Highway Runoff Manual* for selection and design criteria of BMP usage. Some designers prefer to use channels flowing down an embankment to carry away concentrated stormwater. If these channels are lined with rock spalls, over filter material or geotextile, they will provide good service for many years. Paved channels, on the other hand, are very vulnerable to damage. The edges of the pavement have been found to break off easily, especially if the capacity of the channel is frequently exceeded or seepage is able to get under the pavement. The HQ Hydraulics Office does not recommend paved channels unless they have a very short length and have adequate soils supporting the sides of the channel.

As noted above, curbs are often used before vegetation is established to prevent erosion. Once sufficient vegetation is present to resist erosion and treat runoff, consideration should be given to eliminating the curb in future overlay contracts. However, since most approach slabs include curb, consideration must be given to dispersing the concentrated flow at the bridge ends before removing curb. Possible solutions include; discharging runoff to an inlet, maintain curbing until runoff can be properly dispersed or utilizing a fabric or filter blanket.

The down stream end of bridges need special attention, as further described in this paragraph. If a storm drain inlet system is not provided, a channel should be provided at the end of any significant barrier, which collects and concentrates stormwater away from the bridge. Bridges with approach slabs generally have an extruded curb beginning at the bridge end and terminating just past the approach slab. The concentrated flow shall be directed into a rock-lined ditch by creating a small depression and shaping an asphalt chute in the edge of the shoulder apron. Inlets should be located a minimum of 10 feet downstream from an approach slab to avoid approach slab settlement, see Standard Plan A-4.

Bridges without approach slabs and curbing pose yet another set of problems. The concentrated flow runs off the bridge slab and flows off the fill slope, or drains behind the wing walls. Care must be taken to assure the flow is directed into the ditch, and not allowed to erode material away from the bridge end.

A ditch running parallel to the roadway generally drains rural highways in a cut section. These ditches are designed and sized in accordance with the criteria shown in Chapters 4. If the ditch slopes are very steep, they may be fitted with a series of check dams made of rock spalls and keyed into the sides of the ditch. Check dams will reduce flow velocities, prevent erosion of the soil, and may help to trap sediment from upstream sources. Check dams as well as other erosion and sediment control BMP's are covered in the *Highway Runoff Manual*.

5-3.1 Slotted Drains and Trench Systems

Historically, slotted drains have been used with varying degrees of success. In fact, the situations, which warrant the use of slotted drain inlets, can actually hinder their performance. Slotted drain inlets are usually placed in areas of minimal horizontal slope and superelevation. Since the invert of the drain is parallel to the pavement, siltation can occur due to low flow velocities. Trench drains are incapable of handling velocities above 5 cfs. Slotted drains should be capable of H-25 loading, for installation in heavy traffic locations. Designers should contact Region or HQ Hydraulics for design assistance.

A number of trench drain systems are available including pre-formed systems, as well as slotted channels that may be attached to metal or polyethylene pipe. The pre-formed systems are constructed of various materials and have a cross section that minimizes siltation. These systems are usually encased in a concrete-backfilled trench that provided the support of the frame. Grates vary depending on application, are produced in a variety of load ratings and may be constructed of ductile iron, stainless or galvanized steel, resin composites or fiberglass.

Other systems consist of slotted channels, usually constructed of metal and may have a minimal slope built in to help minimize the siltation problem. The slotted channel is placed in the pavement, but with the built in slope, the host pipe may be sloped slightly to improve flow. The channels can be attached to metal or polyethylene pipe and come in various widths and lengths. HQ Hydraulics has more information on all these systems and is available to assist in their design.

5-3.2 Drop Inlets

The use of the drop inlet (Standard Plans B-45.20 thru B-50.20) is intended for mountainous areas or portions of highways that have very long continuous grades. Normal wheel loads can safely pass over the grate and it is not classified as an obstruction. They have a high hydraulic capacity and are most often used in medians. The outlet pipe usually controls the discharge rather than the grate itself. They are also quite effective in passing debris that would normally plug a standard grate.

When the inlet is located in the clear zone, the designer should place the inlet as close to parallel in the direction of traffic as possible. Placing the inlet at an angle may cause an errant vehicle to overturn.

5-4 Gutter Flow

When stormwater is collected and carried along the roadside in a gutter, the allowable top width of the flow prism (Z_d) is dependant on the Road Classification as noted in Figure 5-4.1.

Road Classification		Design Frequency	Design Spread ¹ (Z_d)
High Volume or Divided or Bidirectional	< 45 mph (70 km/hr)	10-year	Shoulder+2 ft (0.67 m)
	> 45 mph (70 km/hr)	10-year	Shoulder
	Sag Pt.	50-year	Shoulder+2 ft (0.67 m)
Collector & Local Streets	< 45 mph (70 km/hr)	10-year	½ Driving Lane
	> 45 mph (70 km/hr)	10-year	Shoulder
	Sag Pt.	50-year	½ Driving Lane

1. The actual driving lane, shall not be less than 10'.

Design Frequency and Spread

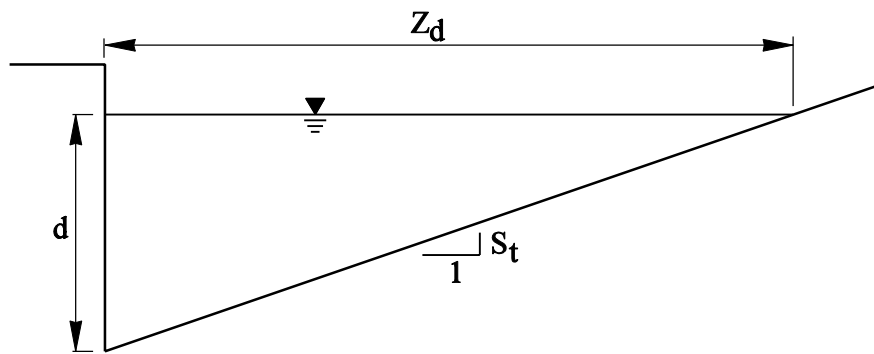
Figure 5-4.1

In urban situations, with much lower speeds than noted in Figure 5-4.1, it may not be feasible to use the design spread recommended in this manual. In this situation, designers should first consider innovative solutions such as: increasing the slope of the gutter (from 2% to 5% for example), depressing the inlet, or using a combination curb opening and grate inlet. If it is still not possible to meet the design spread in Figure 5-4.1, the designer should consider the safety of the intersection, how hydroplaning could affect a driver at this location, and how quickly ponding from the rainfall event will shed off the roadway. The designer should work with the Region Hydraulic Engineer to develop a solution that best suits the project location and keeps the roadway safe. If after considering all possible scenarios, it is determined that the spread of runoff is not safe at this location then more drastic measures such as revising the project scope or seeking more funding may be necessary.

In addition to the requirements above, areas where a superelevation transition causes a crossover of gutter flow, the amount of flow calculated at the point of zero superelevation shall be limited to 0.10 cfs (0.003 m³/s). The designer will find, by the time the roadway approaches the zero point, the Z_d will become very wide. The flow width criteria will be exceeded at the crossover point even when the flow is less than 0.10 cfs (0.003 m³/s). The depth of flow should not exceed 0.12 ft (35 mm) at the edge of a traveled lane for non-interstate roads.

The equation for calculating the gutter flow capacity is a modified version of Manning's equation. It is based on a roughness value of 0.015, which assumes a rough, concrete or asphalt pavement gutter. Equation 5-1 and 5-2 assumes a uniform gutter section as shown in Figure 5-4.2. If the gutter section is different, designers should consult the Regional Hydraulics Engineer and *Hydraulic Engineering Circular No. 22*, Chapter 4 (see web link below) for further guidance. Generally, the discharge, longitudinal slope, and superelevation are known and used to determine the depth of flow and the top width as shown in equations 5-1 and 5-2.

(¹<http://www.fhwa.dot.gov/bridge/hydpub.htm>)



Typical Gutter Section
Figure 5-4.2

$$d = \left[\frac{\Delta Q S_t}{37(S_L)^{0.5}} \right]^{\frac{3}{8}} \quad (5-1)$$

$$Z_d = \frac{d}{S_t} \quad (5-2)$$

- Where:
- d = Depth of flow at the face of the curb (ft)
 - ΔQ = Gutter discharge (cfs)
 - S_L = Longitudinal slope of the gutter (ft/ft)
 - S_t = Transverse slope or superelevation (ft/ft)
 - Z_d = Top width of the flow prism (ft)

5-5 Catch Basins and Grate Inlets

There are many variables involved in determining the hydraulic capacity of an inlet. The depth of flow next to the curb is the major factor in the interception capacity of an inlet. Slight variations in slope or superelevation can have a large effect on flow patterns. The placement of an inlet can also result in dramatic changes in its hydraulic capacity. Drainage structures should never be placed directly in the wheel path. Many are traffic rated and have lock down grates, but the constant pounding of traffic causes unnecessary stress and wear on the structure, frame and grate. Care should be taken to place the inlets next to the face of curb and at the proper elevation relative to the pavement. The structure offset shown in the plans should be to the center of grate, not to the center of structure, to ensure the grate is located along the face of curb.

Generally, median barrier scuppers are not recommended for passing runoff from one side of the barrier to a drainage structure on the other side. Instead Inlets placed on each side of the median barrier should be installed as shown in Standard Plan B-95.20-00, allowing runoff to pass between the structures via a drainpipe.

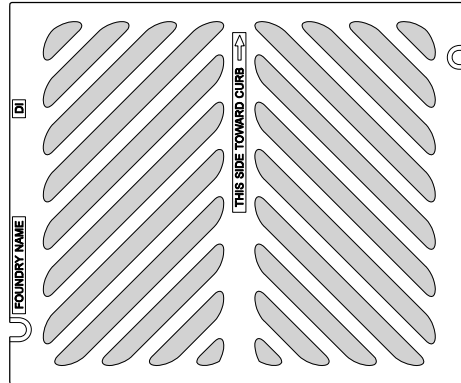
Debris floating in the gutter has a tendency to collect at the inlets, plugging part or the entire grate opening. Inlets placed on a continuous grade are calculated using the full width of the grate with no allowance needed for debris. Inlets placed in a sump are further discussed in section 5-5.3. Areas with deciduous trees and large pedestrian populations are more prone to debris plugging. Bark from logging operations and agricultural areas are also known to cause debris problems.

This section has been divided into three areas: inlets on a continuous grade, side flow

interception and sag analysis. Properties of Grate Inlets available in the WSDOT *Standard Plans* are summarized in Figure 5-5.6 and further discussed below.

Herringbone Pattern or Standard Plan B-30.10

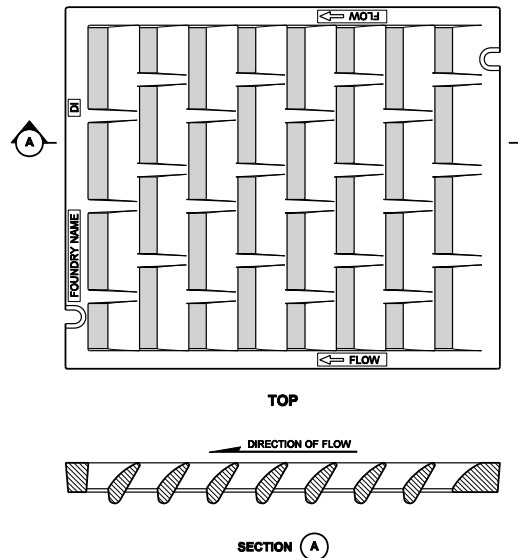
The HQ Hydraulics Office no longer recommends using herringbone grates. Historically, use of the vaned grate was limited due to cost considerations. The cost difference now is minimal, the vaned grate is bicycle safe, and as described further in this section is hydraulically superior under most conditions. Installation of the vaned grate is critical as the grate is directional. If installed backwards the interception capacity is severely limited. Figure 5-5 includes the herringbone information for existing conditions only, it is not intended for new construction.



Herringbone Pattern
Figure 5-5.1

Vaned Grate or Standard Plan B-30.30 or 30.40

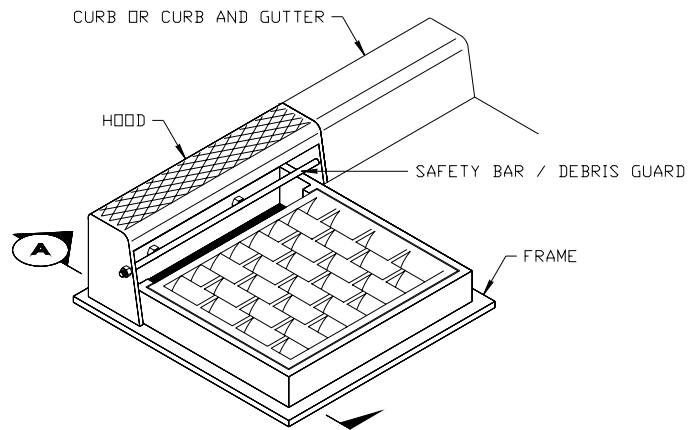
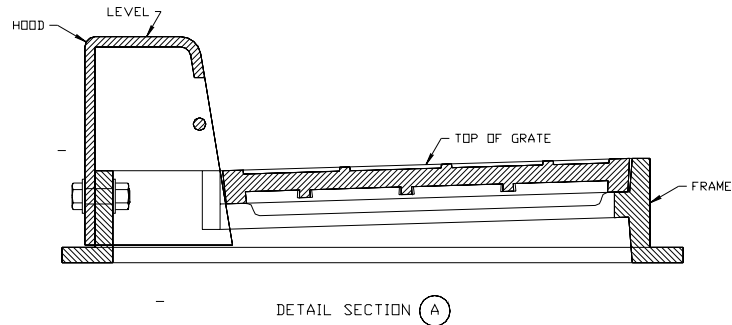
At low velocities the vaned grate and herringbone grate are equally efficient. At higher velocities, greater than 5 ft/s (1.5 m/s), a portion of the flow tends to skip over the herringbone whereas the vaned grate will capture a greater portion of this flow. The vaned grate also has a higher capacity for passing debris and should be used for high debris areas.



Vaned Grate
Figure 5-5.2

Combinations Inlets or Standard Plan B-25.20

The combination inlet is a vaned grate on a catch basin with a hooded curb cut area. Its vaned grate is very debris efficient, and if the grate does become clogged, the overflow goes into the hooded opening. These inlets are extremely useful for sag condition installations although, they can also be effective on continuous grades. The interception capacity of a combination inlet is only slightly greater than with a grate alone. Therefore the capacity is computed neglecting the curb opening and designers should follow the same analysis as for a vaned grate alone (see standard plan B-30.30 or 2c). See section 5-5.3 for design guidance in a sag condition.



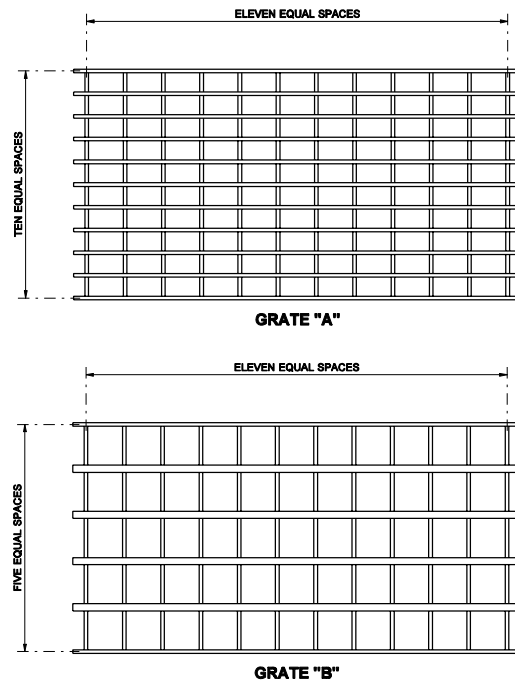
**Section and Isometric View
Combination Inlet
Frame, Hood, and Vaned Grate
Figure 5-5.3**

Grate Inlets Type 1 or 2 or Standard Plan B-35.20 or 35.40

Both types 1 or 2 Grate Inlets have large openings that can compensate for debris problems, however, there are limitations in their usage. A type 1 Grate Inlet is a non-reinforced, cast-in-place concrete inlet, which cannot support traffic loads. Type 2 Grate Inlets are pre-cast and can withstand traffic loading. These inlets are installed with a Grate A, Grate B (see Figure 5-5.4) or a Frame and Vaned Grate (see the next paragraph and Figure 5-5.5 for more information on Frame and Vaned Grates). Due to structural failure of both Grates A or B, neither of these grates can be installed in heavy traffic areas where wheel loads will pass directly over. Grate B has very large openings and is useful in ditches or non-paved median locations, in areas where there

Drainage of Highway Pavements

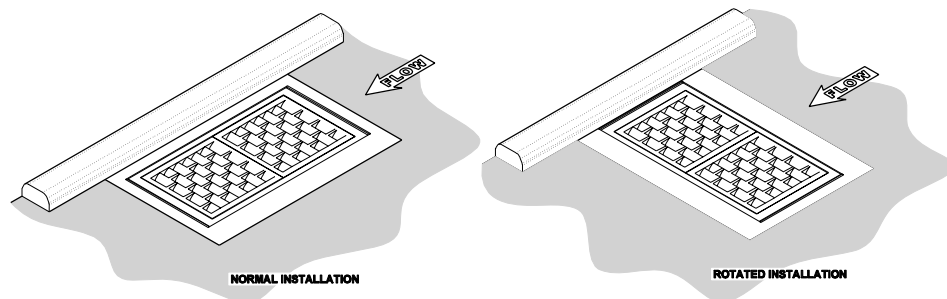
is no pedestrian or bicycle traffic. Grate A can be used anywhere Grate B is used as well as at the curb line of a wide interstate shoulder. Grate A may occasionally be hit by low-speed traffic or parked on but it can not withstand repeated interstate loading or turning vehicles.



Grate A and B
Figure 5-5.4

Frame and Vaned Grates or Standard Plan B-40.40

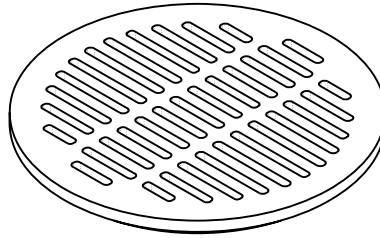
Standard Plan B-40.40 has been tested in H-25 loading and was determined compatible with heavy traffic installations. This frame and double vaned grate should be installed in a Unit H on top of a Grate Inlet Type 2. The frame and vaned grates may be used in either new construction or retrofit situations.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



Frame and Vaned Grates
For Installation on Grate Inlet
Figure 5-5.5

Circular Grate or Standard Plan B-30.80

Circular grates are intended for use with drywells, see standard Plans B-20.20 and B-20.60 for details. Install with circular frames (rings) as detailed in Standard Plan B-30.70.



Circular Grate

Figure 5-5.6

Quarry Spall Placement Around Inlets

Quarry Spalls shall not be placed around inlets. This creates a safety hazard for the maintenance personnel who need good footing to lift the heavy lids. If quarry spall check dams are desired for erosion control, locate them a minimum of 10 feet away from the grate inlet.

Standard Plan	Description	Continuous Grade ¹	Sump Condition ² Perimeter Flows as Weir	
		Grate Width	Width	Length
<u>B-30.10</u> ³	Metal Frame and Grate for Catch Basin and Inlet (Herringbone Pattern)	1.67 ft (0.50 m)	0.69 ft (0.21 m)	0.78 ft (0.24 m)
<u>B-30.30 or 30.40</u> ⁸	Vaned Grate for Catch Basin and Inlet	1.67 ft (0.50 m)	1.31 ft (0.40 m)	1.25 ft (0.38 m)
<u>B-25.20</u> ²	Combination Inlet	1.67 ft (0.50 m)	1.31 ft (0.40 m)	1.25 ft (0.38 m)
<u>B-30.30</u>	Grate Inlet Type 1 (Grate A or B ⁴)	2.01 ft (0.62 m) <u>3.89 ft</u> ⁷ (1.20 m)	1.67 ft (0.50 m) <u>3.52 ft</u> (1.07 m)	3.52 ft (1.07 m) <u>1.67 ft</u> (0.50 m)
<u>B-30.80</u>	Circular Grate ⁹	1.52 ft (0.47 m)	2.55 ft ¹⁰ (0.79 m)	
<u>B-40.40</u>	Frame and Vaned Grates for Grate Inlet Type 2	1.75 ft ⁵ (0.52m) <u>3.52 ft</u> ⁶ (1.05 m)	1.29 ft (0.40 m) 2.58 ft ⁶ (0.80 m)	3.34 ft (1.0 m) 1.67 ft (0.50 m)

1. Inlet widths on a continuous grade are not reduced for bar area or for debris accumulation.
2. The perimeters and areas in this portion of the table have already been reduced for bar area. These values should be cut in half when used in a sag location as described in Section 5-5.2, except for the Combination Inlet B-25.20.
3. Shown for informational purposes only. See Section 5-5.
4. Type B grate shall not be used in areas of pedestrian or vehicular traffic. See section 5-5 for further discussion.
5. Normal Installation, see Figure 5-5.5 or *Standard Plans*.
6. Rotated Installation see Figure 5-5.5 or *Standard Plans*.
7. Rotated installation, see Figure 5-5.4 or *Standard Plans*.
8. For sag conditions, combinations inlets should use a Bi-Directional Vaned grate as shown in Standard Plan B-30.40.
9. Circular Grates are only intended for use with dry wells.
10. Only the perimeter value has been provided for use with weir equations.

Properties of Grate Inlets

Figure 5-5.6

5-5.1 Capacity of Inlets on a Continuous Grade

The interception capacity of an inlet on a continuous grade depends on the amount of water flowing over the grate, the size and configuration of the grate and the velocity of the flow in the gutter. 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. Research and experience has found that this is a very reasonable estimate of the capacity of the inlet in normal highway applications. This method of calculation is slightly conservative for very flat longitudinal slopes, as side flow interception is ignored, and non-conservative for very steep longitudinal slopes where splash-over often occurs. It is most accurate when velocities are in the range of 3 to 5 ft/s (1.0 to 1.5 m/s) at a 2 or 3 percent longitudinal slope. When longitudinal slopes are less than

2% designers should consult section 5-5.2 for side flow interception. The Region Hydraulics Engineer or HQ Hydraulics Office is available to provide direction when velocities exceed 5 ft/s (1.5 m/s). Additional guidance can be found in the FHWA *HEC No. 22, Section 4-3* at the following web page:

(<http://www.fhwa.dot.gov/bridge/hydrpub.htm>)

When locating and determining the capacity of inlets on a continuous grade, the process described in Example 5-1 should be followed. A Microsoft Excel spreadsheet has been developed that uses the procedure described in this section to calculate roadway runoff and inlet interception for a roadway on a longitudinal slope. The spreadsheets are set up for English or Metric input and output and are located at the web site below.

(<http://www.wsdot.wa.gov/eesc/design/hydraulics/downloads.htm>)

The flow bypassing the first grate inlet shall be calculated and added to the flow intercepted by the second grate located downstream. This carry-over process must continue to the bottom of the grade or the end of the inlet system. The last inlet on a highway system is permitted to bypass 0.10 cfs (0.003 m³/s) for the 10 year MRI storm without making any further provisions. This is because less than 0.1 cfs will not usually cause erosion or hydroplaning problems and can bypass without further consideration. However the designer should still consider the cumulative affects from the bypass flow and the area between the bypass flow and the next inlet or outfall. In areas of lower speeds such as local streets or intersections, a bypass of 0.1 cfs or greater may be an acceptable design. The designer should consider the safety of the location with the higher bypass and consult with the Region Hydraulic Engineer for approval.

The amount of flow bypassing the inlet on a continuous grade is computed as follows:

$$Q_{BP} = Q \left[\frac{(Z_d) - (GW)}{(Z_d)} \right]^{\frac{8}{3}} \quad (5-3)$$

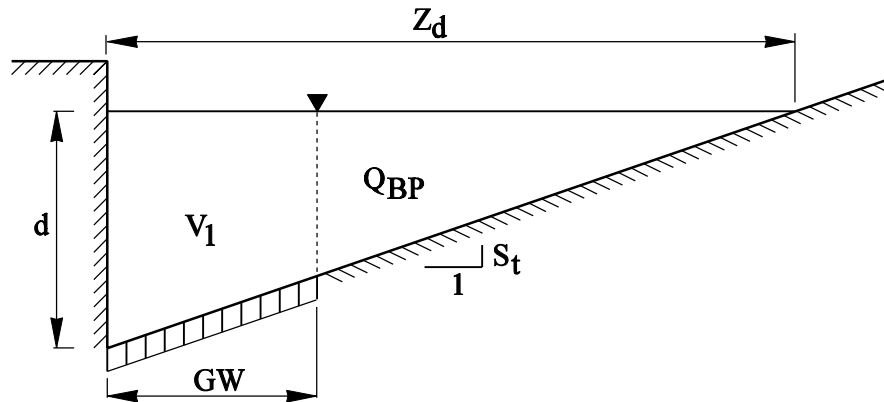
- Where:
- Q_{BP} = Portion of flow outside the width of the grate, cfs
 - Q = Total flow of gutter approaching the inlet, cfs
 - Z_d = Top width of the flow prism, feet
 - GW = Width of the grate inlet perpendicular to the direction of flow in feet

The velocity of the flow directly over the inlet is of interest to the designer and may differ significantly from the average velocity. For an accurate analysis, this velocity should be between 3 and 5 ft/s (1.0 to and 1.8 m/s). When under 3 ft/s (1.0 m/s), the results of the analysis may be conservative and designers should consult section 5-5.2 as well as the Region Hydraulics Engineer. The velocity of flow directly over the inlet is calculated as shown in equation 5-4.

$$V_1 = \frac{Q - Q_{BP}}{(GW)[d - 0.5(GW)(S_t)]} \quad (5-4)$$

- Where:
- V_1 = Velocity over the inlet in ft/s (m/s)
 - S_t = Transverse slope or superelevation in ft/ft (m/m)

d = Depth of flow at the face of the curb ft (m)



Section at Inlet
Figure 5-5.7

5-5.1.1 Example

The project is located in Seattle on a non-interstate roadway. The high point of a vertical curve is at Station 41+00. The width of pavement is 38 ft (11.5 m), with a 5 ft shoulder and three 11 ft lanes. A proposed drainage system consists of grate inlets (see Figure 5-5.4) placed at the following stations:

Station	(S_L) Longitudinal Grade	(S_t) Superelevation
48+50	0.011	0.035
51+50	0.024	0.022
54+50	0.028	0.02
57+50	0.028	0.02
59+00	0.028	0.02

Complete a pavement and drain inlet analysis for this situation using the formulas below:

Solution:

Assume $T_c=5$ min for all inlets
Use 10-year MRI design rainfall

1. Determine the intensity, see Chapter 2 of this manual equation 2-4 and Figure 2-5.4B for m and n values. If the spreadsheet is being used, once the m and n values are input 'I' will automatically be calculated.

$$I = \frac{m}{(T_c)^n} = \frac{5.62}{(5)^{0.530}} = 2.39 \frac{\text{in}}{\text{hr}}$$

2. The designer should then select an inlet from section 5-5 and note the grate width (GW) just below the intensity calculation on the spreadsheet. For this example, grate inlets (see Figure 5-5.4) will be used with a $GW=2.02$. The designer will also insert the GW at particular stations on the spreadsheet when the Z_d exceeds the allowable limit. See number 5 below.
3. Designers should note the superelevation and longitudinal grade given in the table above on the spreadsheet. These values should be updated between each inlet for an accurate analysis.
4. Using the Rational Method, determine the runoff from the flow at the high point

of the vertical curve to the next inlet or ΔQ see section 2-5 for equation details. This is the amount of runoff that could be intercepted by the first inlet. By noting the parameters in steps 1 and 2 along with the width of the paved area and the station where proposed inlets will be located, ΔQ will automatically be calculated on the spreadsheet.

- a. Determine the area of flow from the high point of the vertical curve (Station 41+00) to the first inlet (Station 48+50).

$$A = 38\text{ft} \times ((48 + 50 - 41 + 00)) = 28,500\text{ft}^2$$

(Convert to acres; see Appendix A 1-1 for conversion.)

$$A = \frac{28,500\text{ft}^2}{43560 \frac{\text{ft}^2}{\text{acre}}} = 0.65\text{acres}$$

- b. Determine flow collected from Stations 41+00 to 48+50, see section 2-5 for equation details.

$$\Delta Q = \frac{CIA}{K_c} = \frac{(0.9)(2.39)(0.65)}{1} = 1.41\text{cfs}$$

5. The inlet at Station 48+50 is analyzed next. The depth of flow (d) and width of flow (Z_d) are calculated using the equations 5-1 and 5-2. Verify Z_d is within the allowable limit: the shoulder width (5 ft) plus one-half of the traveled lane (5.50 ft) or 10.5 ft.

$$d = \left[\frac{\Delta QS_t}{37(S_L)^{0.5}} \right]^{\frac{3}{8}} = \left[\frac{1.41 \times 0.035}{37(0.011)^{0.5}} \right]^{\frac{3}{8}} = 0.19\text{ft}$$

$$Z_d = \frac{d}{S_t} = \frac{0.19}{.035} = 5.56\text{ft}$$

Z_d is acceptable since $Z_d = 5.56$ ft which is less than the allowable limit of 10.5 ft. If Z_d is found to be quite small, a zero may be placed in the grate width column on the spreadsheet, which would indicate that no inlet is needed at this location. This may be checked repeatedly until Z_d approaches the allowable limit at which point an inlet should be located by inserting the grate width at that station on the spreadsheet.

6. Q_{BP} is then calculated utilizing equation 5-3, this is the portion of water that is flowing past the inlet and added to the flow for the next inlet (51+50).

$$Q_{BP} = \Delta Q \left[\frac{(Z_d) - (GW)}{Z_d} \right]^{\frac{8}{3}} = 1.41 \left[\frac{(5.56) - (2.05)}{5.56} \right]^{\frac{8}{3}} = 0.41\text{cfs}$$

7. Next, the amount of flow intercepted by the grate is determined.

$$Q_i = \Delta Q - Q_{BP} = 1.41 - 0.41 = 1.0\text{cfs}$$

8. The velocity V_1 is checked to make sure it is acceptable, see equation 5-4 (within 3 to 5 ft/s).

$$V_1 = \frac{\Delta Q - Q_{BP}}{(GW)[d - 0.5(GW)(S_1)]} = \frac{1.41 - 0.41}{(2.05)[0.19 - 0.5(2.05)(0.035)]} = 3.16 \frac{\text{ft}}{\text{s}}$$

The velocity is within limit, so the designer can proceed to the next inlet. If the velocity was not within limit, the designer might want to change the inlet spacing. Since the longitudinal slope is less than 2% from station 41+00 to 48+50, designers could consider side flow analysis with the Region Hydraulics Engineers approval.

9. The designer then proceeds to the next inlet at Station 51+50 and repeats the analysis starting at numbers 2 and 3 as shown below:

$$A = 38\text{ft} \times ((51 + 50 - 48 + 50)) = 11400\text{ft}^2$$

$$A = \frac{11400\text{ft}^2}{43560 \frac{\text{ft}^2}{\text{acre}}} = 0.26\text{acres}$$

$$\Delta Q = \frac{CIA}{K_c} = \frac{(0.9)(2.39)(0.26)}{1} = 0.56\text{cfs}$$

At this point the by pass flow (Q_{BP}) from Station 48+50 needs to be added to delta flow (ΔQ) above to determine the total flow (ΣQ) to the inlet at Station 51+50.

$$\Sigma Q = Q_{BP} + \Delta Q = 0.41 + 0.56 = 0.97 \text{ cfs}$$

The analysis continues back at number 4 using the (ΣQ) above in place of (ΔQ).

This process should be repeated to the end of the inlet system. Designers should be checking that the velocity is between 3-5 ft/s and the Z_D does not exceed the allowable spread.

The last inlet may require an adjustment of spacing (usually a smaller spacing) in order to comply with criteria to limit the last Q_{BP} to 0.10 cfs (0.003 m³/s). Inlets should always have a minimum spacing of 20 ft (7 m) to enable the bypass water to reestablish its flow against the face of curb.

Keep in mind that the deeper a gutter flows, the more efficient the inlet will perform. The 300 ft (90 meter) spacing for inlets shown in Example 5-1 is not a requirement. This distance is a maintenance guideline for situations where catch basins dump directly into a trunk-line. See section 6-2 for further design guidance.

5-5.2 Side Flow Interception

For longitudinal slopes less than 2% and when equation (5-4) yields velocities less than 3 ft/s, designers should consider side flow interception in the capacity analysis for inlets on a continuous grade with the Region Hydraulics Engineers prior approval. Including side flow in this circumstance will produce a more realistic model, likely reducing the total number of inlets. For longitudinal slopes greater than 2%, only a negligible portion of side flow is intercepted and side flow can be ignored. A

spreadsheet has been prepared to aid the designer in locating inlets and is located at the following web site. See FHWA *Hydraulic Engineering Circular #22*, section 4.4.4 for more information.

(<http://www.wsdot.wa.gov/eesc/design/hydraulics/downloads.htm>.)

The analysis is the same as described in section 5-5.1, however the Velocity equation has been modified to include side flow analysis as shown in equation 5-5.

$$V = \left(\frac{K_c}{n} \right) (S_L^{0.5} S_T^{0.67} Z_d^{0.67}) \quad (5-5)$$

Where:

V	=	velocity in triangular channel, ft/s
K _c	=	constant 1.11
N	=	0.015 (Mannings value for concrete pavement)
S _L	=	longitudinal slope

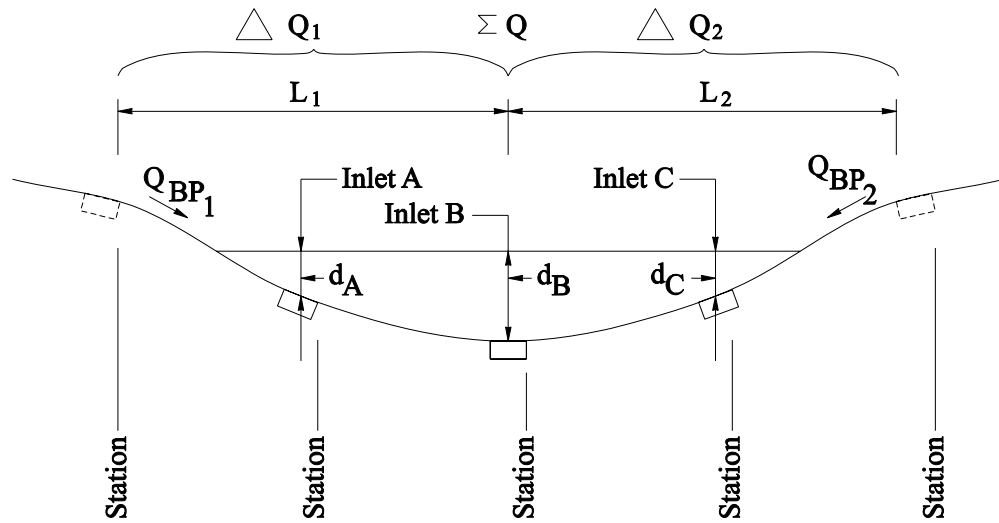
5-5.3 Capacity of Inlets in Sag Locations

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

Inlets at sag locations can easily become plugged with debris and therefore, it is good engineering practice to provide some type of relief. This relief can be accomplished by locating flanking inlets, on either side of the sag inlet, so they will operate before water exceeds the allowable spread into the travel lane at the sag. This manual recommends flanking inlets be located so the depth of water at the flanking inlet ponds to half the allowable depth at the sag (or 1/2d_B), see Figure 5-4.1. With that said, flanking inlets are only required when the sag is located in a depressed area and water has no outlet except through the system. A curb, traffic barrier, retaining wall, or other obstruction, which prevents the runoff from flowing off of the traveled roadway, generally contains this ponded area. However, if runoff is capable of overtopping the curb and flowing away from the roadway before exceeding the allowable limits noted in Figure 5-4.1, flanking inlets are not required. With this situation there is a low potential for danger to the drivers of the roadway if the inlets do not function as designed. Before flanking inlets are removed in this situation, designers should consider the potential damage of water going over the curb. Designers should use the guidelines provided in this section for locating flanking inlets. If a designer suspects flanking inlet are unnecessary, consult the Region Hydraulics Engineer earlier in the design for approval.

Any section of roadway located in a sag should be designed according to the criteria described below and further detailed in example 5-2. A copy of the sag worksheet is located on the HQ Hydraulic Web page, to aid the designer in sag analysis.

(<http://www.wsdot.wa.gov/eesc/design/hydraulics/downloads.htm>)



Sag Analysis
Figure 5-5.8

Once an inlet has been placed in a sag location, the total actual flow to the inlet can be determined as shown below. Q_{Total} must be less than $Q_{\text{allowable}}$ as described in equation 5-8.

$$Q_{\text{TOTAL}} = Q_{\text{BP1}} + Q_{\text{BP2}} + \Delta Q_1 + \Delta Q_2 \quad (5-6)$$

Where:

- $Q_{\text{BP1\&2}}$ = Bypass flow from the last inlet on either side of a continuous grade calculated using equation 5-3
- $\Delta Q_{1\&2}$ = Runoff that is generated from last inlet on either side of the continuous grades, see Figure 5-5.3.

The effective perimeter of the flanking and sag inlets can be determined using the length and widths for various grates given in Figure 5-5.6. This would be the sum of the three sides of the inlet where flow spills in and where ponding would occur. The three inlets should be assumed to be 50% plugged (except for the Combination Inlet B-25.20, which should be considered 0% plugged), therefore the total available perimeter should be reduced by half in the analysis. This adjustment is in addition to reducing the perimeter to account for the obstruction caused by the bars in the grate. Figure 5-5.6 lists perimeters for various grates with reductions already made for bars.

$$P_n = 0.5[L + 2W] \quad (5-7)$$

Where:

- P = Effective perimeter of the flanking and sag inlet
- L = Length of the inlet from Figure 5-5
- W = Width of the inlet from Figure 5-5

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

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

$$Q_{\text{allowable}} = C_w \times P \times d_{B \text{ allowable}}^{1.5} \quad (5-8)a$$

Where:

- C_w = Weir Coefficient, 3.0 for English (1.66 for Metric)
- P = effective perimeter of the grate in feet
- $d_{B \text{ allowable}}$ = maximum depth of water at the sag inlet in feet

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

$$\Sigma Q = C_w \times P \times [2(0.5d_B)^{1.5} + (d_B)^{1.5}] \quad (5-8)b$$

Where:

- d_B = depth of water at the sag inlet (ft).

In some applications, locating inlets so water ponds to $0.5d_{B \text{ allowable}}$ is too far (generally in cases with long flat slopes). Designers should instead ensure that the spread of surface water does not exceed those noted in Figure 5-4.1 and use the equation below.

$$Q_{\text{allowable}} = C_w P [d_A^{1.5} + d_B^{1.5} + d_C^{1.5}] \quad (5-8)c$$

Where:

- d_N = depth of water at the flanking inlets and the sag (ft).

The actual depth of water over the sag inlet can be found with equation 5-9 below and must be less than $d_{B \text{ allowable}}$ which can be found using equation 5-2. If however, the inlets are or not located at $0.5d_{B \text{ allowable}}$, equation 5-9 will need to be modified to reflect this.

$$d_B = \left[\frac{Q_{\text{TOTAL}}}{(C_{wA} P_A 0.3536 + C_{wB} P_B + C_{wC} P_C 0.3536)} \right]^{\frac{2}{3}} \quad (5-9)$$

Where:

- Q_{Total} = Actual flow into the inlet in cfs, (cms)
- C_w = Weir coefficient, 3.0 (1.66 for metric)
- P_N = Effective grate perimeter, in feet (m), see Figure 5-5.6
- d_B = Actual depth of ponded water at the inlet in feet (m)

After the analysis is completed the designer should verify the allowable depth and flow have not been exceeded. That is verify $Q_{\text{allowable}} > Q_{\text{TOTAL}}$ and $d_{B \text{ allowable}} > d_B$. If the allowable flow and depth are greater than the actual, then the maximum allowable spread will not be exceeded and the design is acceptable. If the actual depth or flow is greater than the allowable, then the runoff will spread beyond the maximum limits and the design is not acceptable. In this case the designer should add flanking inlets or replace the three original inlets with inlets that have larger openings. If additional

flanking inlets are used, they should be placed close to the sag inlet to increase the flow interception and reduce the flow into the sag.

5-5.3.1 Example

For this example, assume there is a roadway with a sag in the profile. Inlet spacing has already been calculated using the 10-year MRI for the continuous grade sections on either side of the sag, see example 5-5.1.1.

1. Place an inlet at the low or sag point in the gutter profile, use the gutter profile to determine this location instead of the centerline profile. Whenever possible, a combination inlet should be used at this location to provide continued inlet flow if the grate becomes plugged with debris.
2. The next step is to determine how much runoff will bypass the final inlet on either side of the sag using the 50-year MRI, see step 1 in Example 5-5.1.1. Using the pavement design spreadsheet from example 5-5.1.1, repeat the analysis using the 50 year m and n values. This may create a higher than allowable width of flow at some inlet locations on the continuous grade section previously calculated; however, this is ignored since the flows are calculated only to determine how much flow will bypass the final inlet on the continuous grade and enter the sag during the 50-year MRI. For this example, use 0.1 cfs for a bypass flow from the down stationing side and 0.08 cfs for a bypass flow from the up stationing side (the bypass was found using equation 5-3 similar to 6 of Example 5-5.1.1).
3. The next step is to calculate the runoff, other than bypass flow, that is contributing to the ponding in the sag using the 50 year MRI. This is done as described in step 4 of Example 5-1 and is the runoff that is generated from the pavement between the last inlet on either side of the continuous grades. It is calculated by determining the total pavement area downstream of the continuous grade inlets contributing runoff to the sag and applying the rational method using this area. The rational method is used in the same manner as when runoff is calculated for a continuous grade (see the Example 5-5.1.1 numbers 2 and 3). For this example, use $Q_1 + Q_2 = 0.72$ cfs as the runoff from the pavement in the sag.
4. Once this flow value is calculated, it is added to the two bypass flows to determine the total flow contributing to the sag, using equation 5-6.

$$Q_{\text{TOTAL}} = Q_{\text{BP1}} + Q_{\text{BP2}} + Q_1 + Q_2$$

$$Q_{\text{TOTAL}} = 0.1 + 0.08 + 0.72 = 0.90 \text{ cfs}$$

5. Next, $d_{\text{B allowable}}$ is checked at the sag using equation 5-2. At the lowest point of the sag, in this example, the transverse slope or superelevation at the pavement edge is 0.02 ft/ft. Since the shoulder is 5 feet wide and the traveled lane is 11 feet wide, the allowable width of ponding (Z_d) is 10.5 feet (the shoulder width plus half of the traveled lane). The allowable depth of ponding at the sag is:

$$d_{\text{B allowable}} = S_r \times Z_d = 0.02 \times 10.5 = 0.21 \text{ ft}$$

6. Two additional flanking inlets should be placed on each side of the inlet at the sag this will add relief if the sag inlet becomes clogged or the design spread is exceeded. The flanking inlets can be regular grate inlets and should be located so that the ponded water is $0.5d_{\text{B allowable}} = 0.105$ ft. deep above the flanking inlets and the allowable spread is equal to or less than noted in Figure 5-4.1.

7. Next, use equation 5-9 to determine the effective perimeter of the flanking and sag vaned inlets using the length and widths given in Figure 5-5.6. The three inlets must convey the total flow without causing more than 0.21 ft of ponding at the deepest point.

$$P_n = 0.5[L + 2W]$$

$$P = 0.5 (1.25 + 2 \times 1.31) = 1.94 \text{ ft}$$

8. Next, determine the maximum allowable flow $\Sigma Q_{\text{allowable}}$ into all three inlets when maximum ponding ($d_{B \text{ allowable}}$) occurs. The flow into the lowest inlet is calculated using equation 5-8b with the depth $d_{B \text{ allowable}}$ and the effective perimeter.

$$\Sigma Q_{\text{allowable}} = C_W \times P \times [2(0.5d_B)^{1.5} + (d_B)^{1.5}]$$

$$\Sigma Q_{\text{allowable}} = [3 \times 1.94 \times [2(.5 \times 0.21)^{1.5} + (0.21)^{1.5}]] = 0.95 \text{ cfs}$$

9. The actual depth of water over the sag inlet, d_B should be calculated.

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

$$d_B = \left[\frac{0.90}{3 \times 1.94 \times 0.3536 + 3 \times 1.94 + 3 \times 1.94 \times 0.3536} \right]^{\frac{2}{3}} = 0.20 \text{ ft}$$

10. Finally the actual values are compared to the maximum allowable values as follows:

$$0.95 \text{ cfs} > 0.90 \text{ cfs or}$$

$$Q_{\text{allowable}} > Q_{\text{TOTAL}}$$

∴ Therefore the design is acceptable

$$0.21 \text{ ft} > 0.20 \text{ ft or}$$

$$d_{B \text{ allowable}} > d_B$$

∴ Therefore the design is acceptable

If either the actual depth or flow exceeded the maximum allowable, the design would not be acceptable. In this case the designer would need to repeat the process as described in section 5-5.3 until the design parameters are met. If the design parameters cannot be met due to project constraints, the designer should consult the Region Hydraulics Engineer for further design guidance.

A worksheet of the steps outlined in this example can be found at the following web link (<http://www.wsdot.wa.gov/eesc/design/hydraulics/downloads.htm>). Designers may find it useful to fill out the worksheet for each inlet located at a sag. Worksheets should be submitted with Hydraulics Reports.

5-6 Hydroplaning and Hydrodynamic Drag

As the depth of water flowing over a roadway surfaces increases, the potential for both hydroplaning and hydrodynamic drag increases. Both are discussed in more detail in the subsequent paragraphs below.

Hydrodynamic drag is a term used to describe the force applied to the tire of a vehicle pushing through water as opposed to the tire lifting off the pavement (hydroplaning). The differential force between the tire in the water and the tire out of the water causes the vehicle to "pull" or veer to the side of the water. This usually occurs at speeds less than 50 mph and in water deeper than the depth of the vehicles tire tread. Minimizing water flow depth across lanes and intrusion of flow into lanes will decrease the possibility of hydrodynamic drag.

When rolling tires encounter a film of water on the roadway, the water is channeled through the tire pattern and through the surface roughness of the pavement. Hydroplaning occurs when the drainage capacity of the tire tread pattern and the pavement surface is exceeded and the water begins to build up in front of the tire. As the water builds up, a water wedge is created and this wedge produces a force, which can lift the tire off the pavement surface. This is considered as full dynamic hydroplaning and, since water offers little shear resistance, the tire loses its tractive ability and the driver may lose control of the vehicle.

Hydroplaning is a function of the water depth, roadway geometrics, vehicle speed, tread depth, tire inflation pressure, and conditions of the pavement surface. The following can reduce the hydroplaning potential of a roadway surface:

1. Design the highway geometries to reduce the drainage path lengths of the water flowing over the pavement. This will prevent flow build-up.
2. Increase the pavement surface texture depth by such methods as grooving of Portland cement concrete. An increase of pavement surface texture will increase the drainage capacity at the tire pavement interface.
3. The use of open graded asphaltic pavements has been shown to greatly reduce the hydroplaning potential of the roadway surface. This reduction is due to the ability of the water to be forced through the pavement under the tire. This releases any hydrodynamic pressures that are created and reduces the potential for the tires to hydroplane.
4. The use of drainage structures along the roadway to capture the flow of water over the pavement will reduce the thickness of the film of water and reduce the hydroplaning potential of the roadway surface.