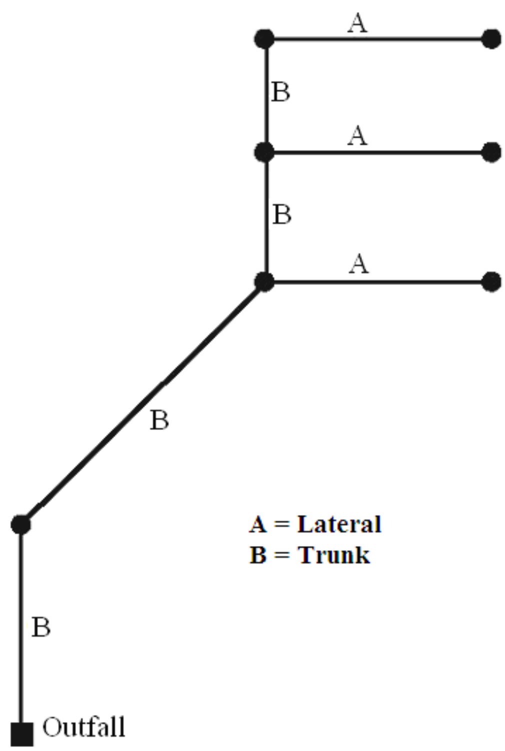
6-1 Introduction

A storm drain (storm sewer) is a network of pipes that conveys surface drainage from a surface inlet or through a manhole, to an outfall. Storm drains are defined as closed pipe networks connecting two or more inlets, see Figure 6-1.1. Storm drain networks typically consist of lateral(s) that discharge into a trunk line. The trunk line then receives the discharge and conveys it to an outfall.



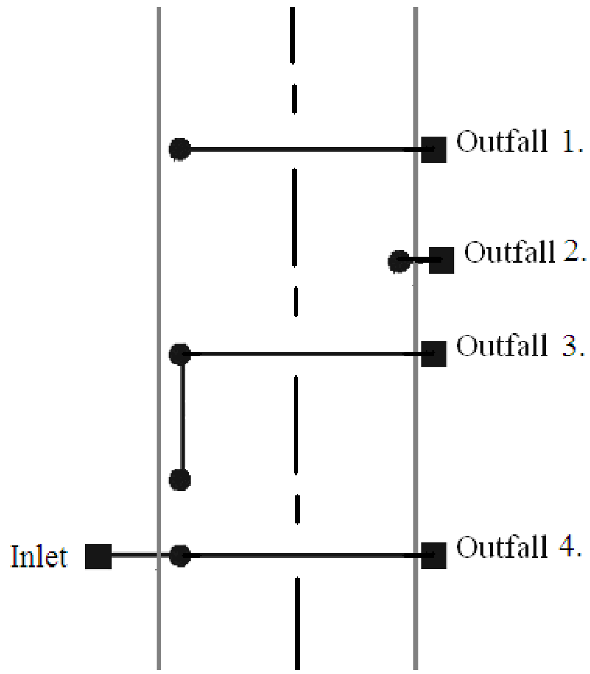
Storm Drain Structure
Figure 6-1.1

While configurations like the one shown in Figure 6-1.1 are typical, there are also other configurations that do not meet the storm drain definition as shown in Figure 6-1.2. In cases where there is only one inlet and no more than two pipes, this should be classified as a culvert on the plan sheets and designed as follows:

1. Storm drain – that does not require pressure testing.

2. Lateral – that does not require pressure testing.

3. Storm drain – that does require pressure testing.
4. Storm drain – that does not require pressure testing.



Storm Drain Configurations
Figure 6-1.2

All storm drain designs will be based on an engineering analysis which takes into consideration runoff rates, pipe flow capacity, hydraulic grade line, soil characteristics, pipe strength, potential construction problems, and potential runoff treatment issues. The majority of time spent on a storm drain design is calculating runoff from an area and designing a pipe to carry the flow. A storm drain design may be performed by hand calculations or by one of several available computer programs and spreadsheets. In addition to storm drain design guidance, this chapter also contains information on drywells (Section 6-7), pipe materials used for storm drains (Section 6-8), and designing for subsurface drainage (Section 6-9).

6-2 Design Criteria

Along with determining the required pipe sizes for flow conveyance and the hydraulic grade line, storm drain system design should consider the following guidelines:

1. **Soil Conditions** – Soil with adequate bearing capacity must be present to interact with the pipes and support the load imparted by them. Surface and subsurface drainage must be provided to assure stable soil conditions.

Soil resistivity and pH must also be known so the proper pipe material can be specified. See Section 8-5 for further guidance.

2. **Inlet Spacing and Capacity** – Design guidelines for inlet spacing and capacity are detailed in Chapter 5, Drainage of Highway Pavements. For minimum clearance between culverts and utilities, designers should consult the Region Utilities Office for guidance.

3. **Junction Spacing** – Junctions (catch basins, grate inlets and manholes) should be placed at all breaks in grade and horizontal alignment. Pipe runs between junctions should not exceed 300 feet (100 meters) for pipes smaller than 48 inches (1,200 millimeters) in diameter and 500 feet (150 meters) for pipes 48 inches (1,200 millimeters) or larger in diameter. When grades are flat, pipes are small or there could be debris issues; designers should consider reducing the spacing. Region Maintenance should be consulted for final approval on maximum spacing.

4. **Future Expansion** – If it is anticipated that a storm drain system may be expanded in the future, provision for the expansion shall be incorporated into the current design. Additionally, prior to expanding an existing system, the existing system should be inspected for structural integrity and hydraulic capacity.

5. **Velocity** – The design velocity for storm drains should be between 3 to 10 feet per second. This velocity is calculated using Manning's Equation (6-1), under full flow condition even if the pipe is only flowing partially full with the design storm. The minimum slope required to achieve these velocities is summarized in the Figure 6-2.

Pipe Diameter (inches)	Minimum Slope (ft/ft)	
n=0.013	2.5 fps	3 fps
12	0.003	0.0044
15	0.0023	0.0032
18	0.0018	0.0025
24	0.0012	0.0017

Minimum Storm Drain Slopes
Figure 6-2

When flows drop below 3 feet per second (1.0 meter per second), pipes can clog due to siltation. Flows can be designed to as low as 2.5 feet per second with justification in the hydraulic report however, lower velocities require prior approval. As the flow approaches (and exceeds) 10 feet per second, higher energy losses are produced in the storm drain system that can cause abrasion in the pipes. For velocities approaching or exceeding 10 feet per second, designers should consult the Section 8-6 for abrasion design guidance.

6. **Grades at Junctions** – Pipe crowns, of differing diameter, branch or trunk lines should be at the same elevation when entering and exiting junctions. For pipes of the same diameter where a lateral is placed so the flow is directed against the main flow through the manhole or catch basin, the lateral invert must be raised to match the crown of the inlet pipe. Matching the crown elevation of the pipes, will prevent backflow in the smaller pipe. (A crown is defined as the highest point of the internal surface of the transverse cross section of a pipe.)

7. **Minimum Pipe Diameter** – The minimum pipe diameter shall be 12 inches (300 millimeters), except that single laterals less than 50 feet (15 meters) long may be 8 inches (200 millimeters) in diameter (some manufacturers are unable to add protective treatment for 8 inch storm drain pipe).

8. **Structure Constraints** – During the storm drain layout design, designers should also consider the physical constraints of the structure. Specifically:
   - **Diameter** – Designers should verify the maximum allowable pipe diameter into a drainage structure prior to design. Some standard plans for drainage structures have pipe allowances clearly stated in tables for various pipe materials.
   - **Angle** – Before finalizing the storm drain layout, designers should verify the layout is constructible with respect to the angle between pipes entering or exiting a junction. In order to maintain the structural integrity of a junction there are minimum clearance requirements that must be met depending on the pipe diameter. Designers can verify the minimum pipe angle with the Pipe Angle Calculation Worksheet located on the HQ Hydraulics web page at: www.wsdot.wa.gov/Design/Hydraulics/ProgramDownloads.htm.

9. **Pipe Material** – Storm drains should be designed to include all Schedule A pipe options, unless specific site constraints limit options. See Section 6-8 for further discussion.

10. **Increase in Profile Grade** – In cases where the roadway or ground profile grades increase downstream along a storm drain, a smaller diameter pipe may be sufficient to carry the flow at the steeper grade. However, due to maintenance concerns, the Washington State Department of Transportation (WSDOT) design practices do not allow pipe diameters to decrease in downstream runs.

Consideration could be given in such cases to running the entire length of pipe at a grade steep enough to allow use of the smaller diameter pipe. Although this will necessitate deeper trenches, the trenches will be narrower for the smaller pipe and therefore the excavation may not substantially increase. A cost analysis is required to determine whether the savings in pipe costs will offset the cost of any extra structure excavation.

11. **Outfalls** – An outfall can be any structure (man-made or natural) where stormwater from WSDOT highways is conveyed off of the right of way (ROW.) Outfalls must conform to the requirements of all federal, state, and local regulations and be documented as described in Appendix 1-3 of this manual.

    Additional considerations for outfalls include energy dissapators and tidal gates. Energy dissipators prevent erosion at the storm drain outfall, for design guidance see Section 3-4.7 of this manual. Installation of tide gates may be necessary when the outfall is in a tidal area, consult the Region Hydraulics Engineer for further guidance.

12. **Location** – Medians usually offer the most desirable storm drain location. In the absence of medians, a location beyond the edge of pavement on state right of way or on easements is preferable. It is generally recommended when a storm drain is placed beyond the edge of the pavement that a one-trunk system, with connecting laterals, be used instead of running two separate trunk lines down each side of the road.

13. **Confined Space and Structures** – Per WAC 296, any structure (catch basin, manhole, grate inlet, or underground detention vault) more than 4 feet in depth is considered a confined space. As such, any structure exceeding 4 feet in depth that could be accessed by personnel must be equipped with a ladder. To determine if personnel will access the structure or if a vactor hose will be used for maintenance, consult the local maintenance office. Structures over 15 feet in depth should be avoided due to the limitations of WSDOT vactor trucks. Any design requiring a structure deeper than 15 feet must consult the Region Hydraulics Office for design approval. Underground detention vaults should only be considered as a last resort due to the overall expense of maintenance. Designers should consult the Region Maintenance Office and Region Hydraulic Engineer before including a vault in any design.

6-3 Data for Hydraulics Report

The design of a storm drain system requires that data be collected and documented in an organized fashion. Hydraulics reports should <u>include all</u> related calculations (whether performed by hand or computer). See Appendix 1-3 of this manual for guidelines on what information should be submitted and recommendations on how it should be organized.

6-4 Storm Drain Design – Handheld Calculator Method

Storm drain design is accomplished in two parts: determine the pipe capacity and then evaluate the HGL. The steps outlined in this section provide the design guidance to determine the pipe capacity. In this section the pipes are designed under full flow conditions to verify the velocity requirements are met. For the HGL evaluated in Section 6-6, the actual surface water elevation in the pipe will be used to verify the system operates under gravity flow conditions.

6-4.1 General

Storm drain design can be accomplished with a handheld calculator using the Rational Method and Figure 6-4.1 to show calculations. Figure 6-4.1 has five divisions: Location, Discharge, Drain Design, Drain Profile, and Remarks. These divisions are further expanded in the subsections below.

6-4.2 Location

The Location section gives all the layout information of the drain.

Column 1 gives a general location reference for the individual drain lines, normally by the name of a street or a survey line.

Columns 2 and **3** show the stationing and offset of the inlets, catch basins, or manholes either along a roadway survey line or along a drain line.

6-4.3 Discharge

The Discharge section presents the runoff information and total flow into the drain.

Column 4 is used to designate the drainage areas that contribute to particular point in the drain system. The drainage areas should be numbered or lettered according to some reference system on the drainage area maps. The type of ground cover (pavement, median, etc.) may be indicated. Since drainage areas must be subdivided according to soil and ground cover types, a drainage area may have several different parts.

Column 5 shows the area of the individual drainage areas listed in Column 4 in acres (hectares).

Column 6 shows the Rational method runoff coefficient (see Chapter 2). Each individual drainage area must have a corresponding runoff coefficient.

Column 7 is the product of Columns 5 and 6. Column 7 is also the effective impervious area for the subsection.

Storm Drain Design Calculations

Calculated By			Date		Checked By							Date		SR		Project					Sheet #	Of	Sheet
Location			Discharge										Drain Design				Drain Profile						
Drain Located On	From Station	To Station	Source of Drainage	Drainage Area (A)	Runoff Coeff. (C)	CA	Σ CA	Tc to Col.2 Station	Rainfall Intensity in/hr	Runoff cfs	Con. Inflow cfs	Total Flow cfs	Pipe Diameter ft	Pipe Slope ft/ft	Flow Velocity ft/s	Pipe Capacity cfs	Length ft	Elev. Change ft	Upstr. Ground Elev.	Downstr. Ground Elev.	Upstr. Invert Elev.	Downstr. Invert Elev.	Remarks
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24

Storm Drain Design Calculations
Figure 6-4.1

Column 8, the summation of CA, is the accumulation of all the effective impervious areas contributing runoff to the point in the system designated in Column 2. All the individual areas in Column 7 contributing to a point in Column 2 are summed. This would include runoff from upstream inlets that contribute to the pipe capacity.

C**olumn 9** shows the time of concentration to the structure indicated in Column 2. Section 2-5.3 of this manual details how to calculate the time of concentration. Generally the time chosen here would be the longest time required for water to travel from the most hydraulically remote part of the storm drain system to this point. This would include flow over the drainage basin and flow through the storm drain pipes. The time of concentration should be expressed to the nearest minute and as discussed in Chapter 2 is never less than 5 minutes.

When the runoff from a drainage area enters a storm drain and the time of concentration (Tc) of the new area is shorter than the accumulated Tc of the flow in the drain line, the added runoff should be calculated using both values for Tc. First the runoff from the new area is calculated for the shorter Tc. Next the combined flow is determined by calculating the runoff from the new area using the longer Tc and adding it to the flow already in the pipe. The Tc that produces the larger of the two flows is the one that should be used for downstream calculations for the storm drain line.

The easiest method for determining the Tc of the flow already in the system (upstream of the structure in Column 2) is to add the Tc from Column 9 of the previous run of pipe (this value should be on the row above the row that is currently being filled in) to the time it took the flow to travel through the previous run of pipe. To determine the time of flow (or more correctly, the travel time) in a pipe, the velocity of flow in the pipe and the length of the pipe must be calculated. Velocity is computed using Manning's Equation and is found in Column 16 of the previous run of pipe. The length used is the value entered in Column 18 for the previous run of pipe. Obviously, this calculation is not performed for the very first (most upstream) run of pipe in a storm drain system.

$$T_1 = \frac{L}{60V}$$

Where:

T_1 = time of concentration of flow in pipe in minutes
L = length of pipe in feet (meters) Column 18
V = velocity in ft/s (m/s) Column 16 of the previous run of pipe

The designer should note that this calculation assumes that the pipe is flowing full. It is accurate for pipes flowing slightly less than half full up to completely full. It will be slightly conservative for T_c calculations when the pipe is flowing significantly less than half full.

Column 10 shows the rainfall intensity corresponding to the time indicated in Column 9 and the location of the project.

The intensity is in inches per hour to the nearest hundredth for English units (millimeters per hour to the nearest tenth). The rainfall intensity used is a 25-year recurrence interval for storm drain laterals and trunks and the 10-year recurrence interval for laterals without trunks. See Chapter 2 for a complete description of how this intensity can be determined. Projects in eastern Washington should also consult Chapter 4 of the *Highway Runoff Manual* for further design guidance.

Column 11 shows the amount of runoff to the (nearest tenth of a cubic foot per second) (nearest hundredth of a cubic meter per second) up to the point indicated in Column 2. It is computed as the product of Columns 8 and 10. This is simply applying the rational method to compute runoff from all the drainage area upstream of the pipe being analyzed.

Column 12 shows any flow, other than the runoff calculated in Column 11, to the nearest tenth of a cubic foot per second (nearest hundredth of a cubic meter per second) that is entering the system up to the point indicated in Column 2. It is rare to have flow entering a system other than runoff from the drainage basin but this does occur. For instance, when an underdrain, which is draining groundwater, is connected to the storm drain. The label for this column indicates that these flows are considered constant for the duration of the storm so they are independent of the time of concentration.

This column is also used when the junction is a drywell and a constant rate of flow is leaving the system through infiltration. When this occurs the value listed in Column 12 is negative. See Section 6-7 for a complete discussion of drywells.

Column 13 is the sum of columns 11 and 12 and shows the total flow in cubic feet per second to the nearest tenth (cubic meters per second to the nearest hundredth) to which the pipe must be designed.

6-4.4 Drain Design Section

This section presents the hydraulic parameters and calculations required to design storm drain pipes.

Column 14 shows the pipe diameter in feet (millimeters). This should be a minimum of 8 inches or 0.67 feet (200 millimeters) for any pipe run with a length of 50 feet (15 meters) or less. Pipes runs longer than 50 feet (15 meters) must have a minimum diameter of 12 inches or 1 foot (300 millimeters). Pipe sizes should never decrease in the downstream direction.

The correct pipe size is determined through a trial and error process. The engineer selects a logical pipe size that meets the minimum diameter requirements and a slope that fits the general slope of the ground above the storm drain. The calculations in Column 17 are performed and checked against the value in Column 13. If Column 17 is greater than or equal to Column 13, the pipe size is adequate. If Column 17 is less than Column 13 the pipe does not have enough capacity and must have its diameter or slope increased after which Column 17 must be recalculated and checked against Column 13.

Column 15, the pipe slope, is expressed in feet per foot (meters per meter). This slope is normally determined by the general ground slope but does not have to match the surface ground slope. The designer should be aware of buried utilities and obstructions, which may conflict, with the placement of the storm drain.

Column 16 shows the full flow velocity. It is determined by Manning's Equation, which is shown below. The velocity is calculated for full flow conditions even though the pipe is typically flowing only partially full. Partial flows will be very close to the full flow velocity for depths of flow between 30 percent and 100 percent of the pipe diameter.

$$V = \frac{1.486}{n} R^{2/3}\sqrt{S} = \frac{1.486}{n}\left[\frac{D}{4}\right]^{2/3}\sqrt{S} \quad \text{(English Units)} \qquad (6\text{-}1)$$

$$V = \frac{1}{n} R^{2/3}\sqrt{S} = \frac{1}{n}\left[\frac{D}{4}\right]^{2/3}\sqrt{S} \quad \text{(Metric Units)}$$

Where:

V = velocity in ft/s (m/s)
D = pipe diameter in feet (meters)
S = pipe slope in feet/foot (meters/meter)
n = Manning's roughness coefficient (see Appendix 4-1)

Extremely high velocities should be avoided because of excessive abrasion in the pipe and erosion at the outlet of the system. Drop manholes should be considered for pipe velocities over 10 fps (3.0 meters per second). The engineer should also keep in mind that energy losses at junctions become significant above 6 feet per second (2 meters per second).

The minimum velocity as determined by this equation is 3 feet per second (1 meter per second).

Column 17, the pipe capacity, shows the amount of flow in cubic feet per second (cubic meters per second), which can be taken by the pipe when flowing full. It is computed using the following formula:

$$Q = VA = V\frac{\pi D^2}{4} \qquad (6\text{-}2)$$

Where:

- Q = full flow capacity in cfs (cms)
- V = velocity as determined in Column 16 in ft/s (m/s)
- A = cross sectional area of pipe in feet squared (meters sq)
- D = diameter of pipe in feet (meters)

6-4.5 Drain Profile

Columns 18 through **23**, the drain profile section, includes a description of the profile information for each pipe in the storm drain system. It describes the pipe profile and the ground profile. The ground elevations should be finished elevations, to the hundredth of a foot. The items in this section are generally self-explanatory. The only exception is Column 18, the length shown is the horizontal projection of the pipe, in feet (meters), from the center to center of appurtenances. Generally, profiles should be set to provide a minimum of 2 feet (0.6 meters) of cover over the top of the pipe, see Chapter 8 for further design guidance.

6-4.6 Remarks

Column 24, remarks, is for any information, which might be helpful in reviewing the calculations. This space should note unique features such as drop manholes, long times of concentration, changes in the type of pipe, or changes in design frequency.

6-5 Storm Drain Design – Computer Analysis

With the addition of personal computers to most engineering workstations, storm drain design by handheld calculator has become less prevalent. Storm drain design by computer analysis offers some distinct advantages over calculations performed by hand. Chief among these advantages is the decreased amount of time required to perform the pipe sizing and hydraulic grade line calculations and the reduced chance for calculation errors.

Some computer programs will use the Rational method for storm drain design while others will use a hydrograph method such as the SBUH method. Both of these methods are valid for WSDOT storm drain design; however, they will yield different peak runoff values. This is most distinct for drainage basins that have very short times of concentration. As a basin's time of concentration extends beyond 15 minutes the two methods yield more similar answers. This difference in peak runoff values ends up having little effect on storm drain design since runoff from basins with short times of concentration tends to be small and the required pipe size is determined by the minimum allowable pipe size. As flows entering the system increase to the point that minimum pipe sizes are no longer the governing factor, the associated time of concentration becomes greater and the two methods produce similar peak flow rates.

There are several commercially available computer programs for storm drain design. Each of these programs has certain features that make them unique from other programs but the primary calculations are performed the same way. Because of this, nearly any commercially available computer programs that perform storm drain design are acceptable for designing WSDOT storm drains.

The HQ Hydraulics Office has purchased the computer program StormShed 3G for the Ferries Division and each WSDOT region to use whenever designing storm drains. Training material for StormShed 3G has been developed specifically for WSDOT applications and is available on the HQ Hydraulics web page or designers can consult the HQ Hydraulics Office for additional technical assistance. To attain the latest version of StormShed 3G software contact the HQ Hydraulics Office or your Region Hydraulic Engineer. Prior to using StormShed 3G, the distance between catch basins/manholes/inlets in every run of storm drains should be located using a Microsoft® Excel Pavement Drainage spreadsheet. A spreadsheet is available on the HQ Hydraulic web page at: www.wsdot.wa.gov/eesc/design/hydraulics. The spreadsheet lacks the advanced features found in commercially available computer programs but does offer a simple and effective way to locate storm drains.

6-6 Hydraulic Grade Line

The hydraulic grade line (HGL) should be designed so there is a space of air between the top of water and the inside of the pipe. In this condition the flow is operating as gravity flow and the HGL is the water surface elevation traveling through the storm drain system. If the HGL becomes higher than the crown elevation of the pipe the system will start to operate under pressure flow. If the system is operating under pressure flow, the water surface elevation in the catch basin/manhole needs to be calculated to verify the water surface elevation is below the rim (top) elevation. When the water surface elevation exceeds the rim elevation, water will discharge through the inlet and cause severe traffic safety problems. Fortunately, if the storm drain pipes were designed as discussed in the previous sections, then the HGL will only become higher than the catch basin/manhole rim elevation when energy losses become significant or if the cover over a storm drain is low (less than 5 feet).

Regardless of the design conditions, the HGL should always be evaluated especially when energy losses become significant. Possible situations where energy losses can become significant include: high flow velocities through the system (greater than 6.6 ft/s), pipes installed under low cover at very flat gradients, inlet and outlet pipes forming a sharp angle at junctions, and multiple flows entering a junction.

The HGL can only be calculated after the storm drain system has been designed. When computer models are used to determine the storm drain capacity, the model will generally evaluate the HGL. The remainder of this section provides the details for how the analysis is performed.

The HGL is calculated beginning at the most downstream point of the storm drain (outfall) and ending at the most upstream point, which is exactly the opposite direction that was used to design the pipe sizes. To start the analysis, the water surface elevation at the storm drain outfall must be known. Refer to Chapter 3 for an explanation on calculating water surface elevations at the downstream end of a pipe (the tailwater is calculated the same for storm drain outfalls and culverts). Once the tailwater elevation is known, the energy loss (usually called head loss) from friction is calculated for the most downstream run of pipe and the applicable minor losses are calculated for the first junction upstream of the outfall. All of these head losses are added to the water surface elevation at the outfall to obtain the water surface elevation at the first upstream junction (also the HGL at that junction). The head losses are then calculated for the next upstream run of pipe and junction and they are added to the water surface elevation of the first junction to obtain the water surface elevation of the second upstream junction. This process is repeated until the HGL has been computed for each junction. The flow in most storm drainpipes is subcritical; however, if any pipe is flowing supercritical (see Chapter 4 for an explanation of subcritical and supercritical flow) the HGL calculations are restarted at the junction on the upstream end of the pipe flowing supercritical. The HGL calculation process is represented in the following equation:

$$WSEL_{J1} = WSEL_{OUTFALL} + H_{f1} + H_{e1} + H_{ex1} + H_{b1} + H_{m1} \quad (6\text{-}3)$$

$$WSEL_{J2} = WSEL_{J1} + H_{f2} + H_{e2} + H_{ex2} + H_{b2} + H_{m2}$$

...

$$WSEL_{Jn+1} = WSEL_{Jn} + H_{fn+1} + H_{en+1} + H_{exn+1} + H_{bn+1} + H_{mn+1}$$

Where:

- WSEL = water surface elevation at junction noted
- H_f = friction loss in pipe noted (see Section 6-6.1)
- H_e = extrance head loss at junction noted (see Section 6-6.2)
- H_{ex} = exit head loss at junction noted (see Section 6-6.2)
- H_b = bend head loss at junction noted (see Section 6-6.3)
- H_m = multiple flow head loss at junction noted (see Section 6-6.4)

As long as the HGL is lower than the rim elevation of the manhole or catch basin, the design is acceptable. If the HGL is higher than the rim elevation, flow will exit the storm drain and the design is unacceptable. The most common way to lower the HGL below the rim elevation is to lower the pipe inverts for one or more runs of the storm drain or increase the pipe diameter.

6-6.1 Friction Losses in Pipes

Head loss due to friction is a result of the kinetic energy lost as the flow passes through the pipe. The rougher the pipe surface is, the greater the head loss is going to be. Head loss from friction can be calculated with the following equation.

$$H_f = L\left[\frac{2.15Qn}{D^{2.667}}\right]^2 \quad \text{(English Units)} \qquad (6\text{-}4)$$

$$H_f = L\left[\frac{3.19Qn}{D^{2.667}}\right]^2 \quad \text{(Metric Units)}$$

Where:

H_f = head loss due to friction in feet (meters)
L = length of pipe in feet (meters)
Q = flow in pipe in cfs (cms)
n = Manning's roughness coefficient (see Appendix 4-1)
D = diameter of pipe in feet (meters)

6-6.2 Junction Entrance and Exit Losses

When flow enters a junction, it loses all of its velocity. As a result, there is an associated head loss equal to one velocity head. Then when the flow exits the junction and accelerates into the next pipe, there is another head loss equal to approximately half of one velocity head. These two head losses can be represented with the following equations (Metric and English units use the same equations).

$$H_e = \frac{v_2^2}{2g} \qquad (6\text{-}5)$$

$$H_{ex} = 1.0\left(\frac{V^2}{2g} - \frac{V_d^2}{2g}\right) \approx \frac{V^2}{4g}$$

Where:

H_e = head loss from junction entrance in feet (meters)
H_{ex} = head loss from junction exit in feet (meters)
V = flow velocity in pipe in feet per second (m/s)
V_d = channel velocity downstream of outlet in feet per second (m/s)
g = gravitational acceleration constant

6-6.3 Losses From Changes in Direction of Flow

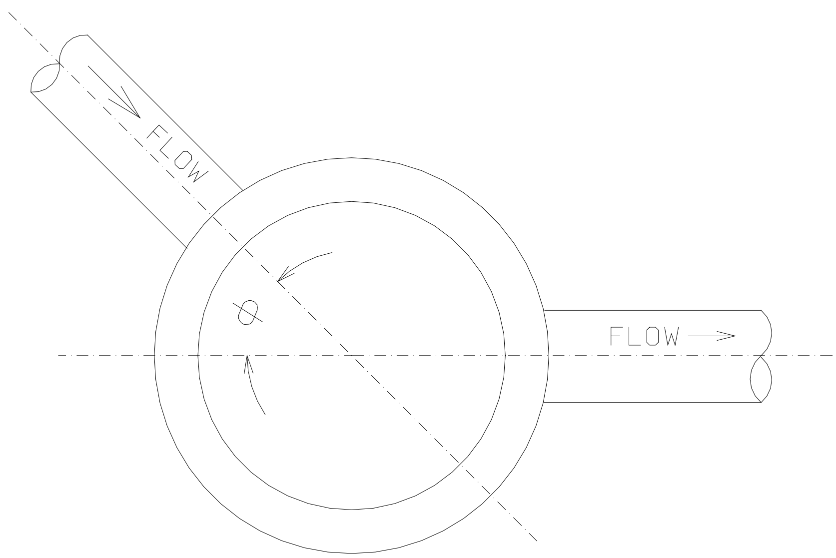
When flow changes direction inside of a junction, there is an associated head loss. The amount of head loss that will occur is dependent on how great the change is. As the angle between the inflow and outflow pipes increase, the amount of head loss increases. This head loss can be calculated with Equation 6-6 (metric and English units use the same equation).

$$H_b = K_c \frac{v^2}{2g} \tag{6-6}$$

Where:

H_b = head loss from change in direction in feet (meters)
K_b = head loss coefficient for change in direction, see below:

K_b	Angle of Change in Degrees
0.00	0
0.19	15
0.35	30
0.47	45
0.56	60
0.64	75
0.70	90 and greater



Changes in Direction of Flow
Figure 6-6.3

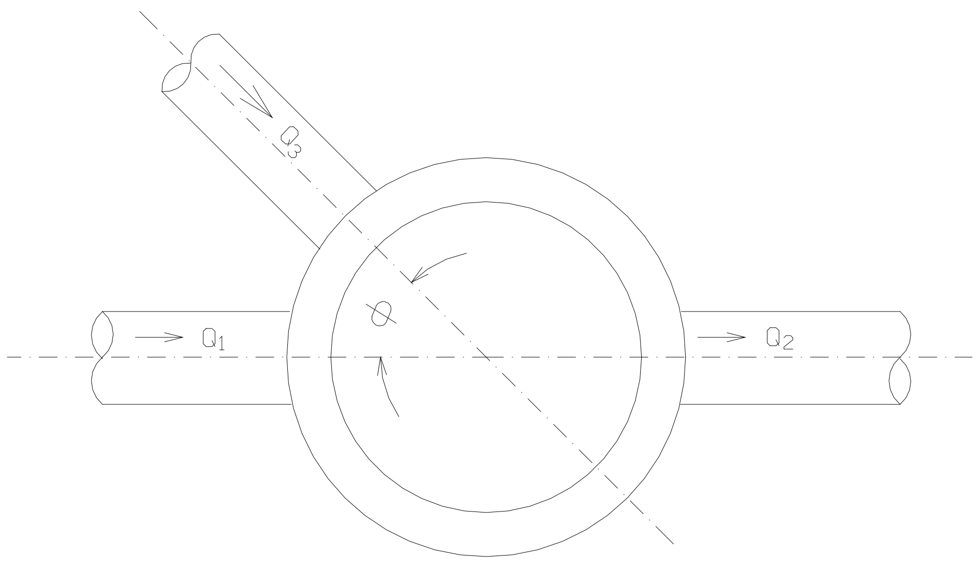
6-6.4 Losses From Multiple Entering Flows

When flow enters a junction from more than one pipe there is an associated head loss. The head loss is dependent on the amount of flow in each pipe and the direction flow enters the junction through each pipe. Once the angle is determined, this head loss can be calculated with the following equation (Metric and English units use the same equation).

$$H_m = \frac{Q_2V_2^2 - Q_1V_1^2 - \cos\phi Q_3V_3^2}{2gQ_2} \tag{6-7}$$

Where:

H_m = head loss from multiple flows in feet (meters)



Multiple Flows Entering a Junction
Figure 6-6.4

6-7 Drywells

A drywell is an underground structure that is typically precast with perforations along the structure walls and bottom that allow stormwater runoff to flow directly into the ground. Drywells can be stand alone structures or installed as part of a storm drain system. The primary advantage of drywells is that they reduce flooding by discharging flow into groundwater instead of discharging it to surface waters such as rivers and creeks. Also, when allowed as part of a storm drain system, the drywell reduces the flow which can reduce the size of the pipes in the system. Standard Plan B-20.20 of the WSDOT *Standard Plans for Road, Bridge, and Municipal Construction* depicts a typical drywell. Additional information about the appropriate geotextile (Class A Underground drainage with moderate survivability) to select for the installation of the drywell is located in the *Standard Specifications for Road, Bridge, and Municipal Construction,* Sections 9-33 and 9-03.12(5).

Prior to specifying a drywell in a design, designers should consult the *Highway Runoff Manual* for additional guidance and design criteria. Drywells are considered Underground Injection Control Wells (UICs) and are required to be registered with DOE per WAC 173-218, see Section 4-5.4 of the *Highway Runoff Manual*. Additionally, stormwater must be treated prior to discharging into a drywell using a Best Management Practice described in Chapter 5 of the *Highway Runoff Manual*. Finally, all drywells should be sized following the design criteria outlined in Section 4-5.4.2 of the *Highway Runoff Manual.*

6-8 Pipe Materials for Storm Drains

When designing a storm drain network, the designer should review Section 8-2 (Pipe Materials), as well as the list of acceptable pipe material (Schedule Pipe) in Section 7-04 (Storm Sewers) of the *Standard Specifications*. Storm drain pipe is subject to some use restrictions, which are detailed in Section 8-1.4 (Storm Sewer Pipe) of this manual.

Pipe flow capacity depends on the roughness coefficient, which is a function of pipe material and manufacturing method. Fortunately, most storm drain pipes are 24 inches (600 millimeters) in diameter or less and studies have shown that most common schedule pipe materials of this size range have a similar roughness coefficient. For calculations, the designer should use a roughness coefficient of 0.013 when all schedule pipes 24 inches (600 millimeters) or smaller are acceptable. For larger diameter pipes, the designer should calculate the required pipe size using the largest Manning's Roughness Coefficient for all the acceptable schedule pipe values in Appendix 4-A of this manual. In the event a single pipe alternative has been selected, the designer should design the required pipe size using the applicable Manning's Roughness Coefficient for that material listed in Appendix 4-A.

In estimating the quantity of structure excavation for design purposes at any location where alternate pipes are involved, estimate the quantity of structure excavation on the basis of concrete pipe since it has the largest outside diameter.

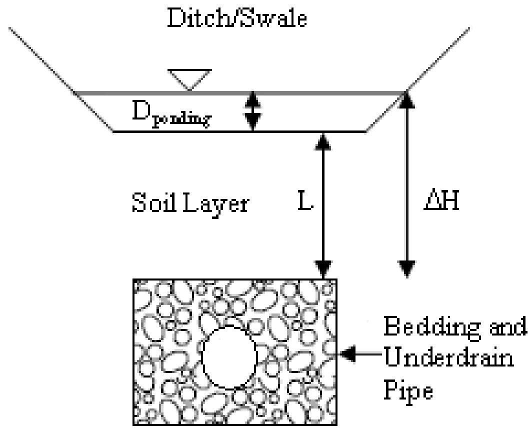
6-9 Subsurface Drainage

Subsurface drainage is provided for control of groundwater encountered at highway locations. Groundwater, as distinguished from capillary water, is free water occurring in a zone of saturation below the ground surface. The subsurface discharge depends on the effective hydraulic head and on the permeability, depth, slope, thickness, and extent of the aquifer.

The solution of subsurface drainage problems often calls for specialized knowledge of geology and the application of soil mechanics. The designer should work directly with the Region Materials Engineer as subsurface conditions are determined and recommendations are made for design in the soil's report.

Subsurface drainage can be intercepted with underdrain pipe that is sized by similar methods used to design storm drain pipes. There are two different methods, recommended in this manual that are used to size underdrains depending on the application.

1. When an underdrain is installed for control of seepage in cuts or side hills or the lowering of the groundwater table for proper subgrade drainage, the design method used to size storm drains should be followed. The only difference is that the flow used for the calculations is the predicted infiltration from groundwater into the system instead of flow entering the system from roadway drainage. When subsurface drainage is connected to a storm drain system, the invert of the underdrain pipe shall be placed above the operating water level in the storm drain. This is to prevent flooding of the underdrain system which would defeat its purpose.
2. The second method involves underdrains installed in combination with a BMP or hydraulic feature such as: media filter drains, swales, ditches, and infiltration trenches as shown in Figure 6-9.1. For these applications, underdrain should be sized so water drains from the bedding material substantially faster than water enters the soil layer above. To achieve this, a factor or safety is applied to the inflow as is described on the next page.



Underdrain Installation in Combination with a BMP or Hydraulic Feature
Figure 6-9.1

The following steps should be used to size an underdrain:

1. Determine the runoff volume (Vud) (ft^3) from the basin contributing to the underdrain. The design event used to size the BMP or hydraulic feature should be used to determine the runoff volume.
2. Specify the maximum designed depth of ponding water ($D_{ponding}$) (ft) in the BMP or hydraulic feature above the underdrain (ft). This can be calculated using StormShed or following the design guidance in the *Highway Runoff Manual* for the applicable BMP. For media filter drains, use 12 inches.
3. Determine the cross sectional area (A) (ft^2) of the flow by dividing the runoff volume by the depth of ponding water.

$$A = \frac{V_{ud}}{D_{ponding}} \tag{6-8}$$

4. Determine infiltration rate (rate runoff moves through the soil) using Darcy's Equation or use infiltration rate from lab.

$$q = \frac{K\Delta H}{L} \qquad (6\text{-}9)$$

Where:
- q = flow per cross sectional area (in/hr per unit)
- K = hydraulic conductivity (in/hr)
- ΔH = change in head (ft) at the height of water from ponding depth to top of bedding material
- L = thickness of soil layer (ft)

5. The total flow to the underdrain is based on the rate runoff moves through soil and the basin area contributing to the BMP or hydraulic feature.

$$Q = q \times A \qquad (6\text{-}10)$$

Where:
- Q = total flow to underdrain (cfs)
- q = flow per cross sectional area (in/hr per unit)
- A = cross sectional area of the ditch/swale

6. Determine the design flow Q_{df}, by applying a Factor of Safety (FS) = 2 to Q, so pipe is sized to carry 2 times the total flow.

$$Q_{df} = Q \times 2 \qquad (6\text{-}11)$$

Where:
- Q_{df} = underdrain design flow (cfs)
- Q = flow total flow to underdrain (cfs)

7. Given design flow, determine the pipe diameter. For pipe diameters that exceed 12″, contact either the region or HQ Hydraulics.

$$D = 16\left(\frac{(Q_{df} \times n)}{s^{0.5}}\right)^{3/8} \qquad (6\text{-}12)$$

Where:
- D = underdrain pipe diameter (inches)
- n = Manning's coefficient (use 0.010-0.011 for smooth wall)
- s = slope of pipe (ft/ft)

Sample Problem

An underdrain will be located under a ditch that can intercept runoff from a road that is 1,000 ft by 34 ft. The Materials Lab has determined the ditch has a hydraulic conductivity of 2.9 in/hr. Assume the soil layer will be 2 ft deep and the slope of the underdrain pipe will be set at 0.5 percent. Determine the size of underdrain needed.

1. The runoff volume (Vud) (ft3) was determined to be 2,875 cu ft. The value was determined using the 10-year design event to size the ditch and StormShed 3G.
2. The maximum depth of ponding water (Dponding) (ft) in the ditch was determined to be 4 inches using StormShed.
3. Determine the cross sectional area (A) (ft^2) of the flow.

$$A = \frac{V_{ud}}{D_{ponding}}$$

$$A = \frac{2{,}875 \text{ cu ft}}{0.33 \text{ ft}} = 8{,}712 \text{ sq ft}$$

4. Determine the infiltration rate.

$$q = \frac{K\Delta H}{L}$$

$$q = \frac{2.9 \text{ in/hr} \times 2.33 \text{ ft}}{2 \text{ ft}} = 3.38 \text{ in/hr}$$

5. The total flow to the underdrain is based on the rate runoff moves through soil and basin area contributing to the BMP or hydraulic feature.

$$Q = q \times A$$

$$Q = 3.38 \text{ in/hr} \times 8{,}712 \text{ sq ft} \times 1 \text{ ft/12 in} = 0.68 \text{ cfs}$$

6. Determine design flow Qdf, by applying a Factor of Safety (FS) = 2.

$$Q_{df} = Q \times 2$$

$$Q_{df} = 0.68 \text{ cfs} \times 2 = 1.36 \text{ cfs}$$

7. Given design flow, determine the pipe diameter.

$$D = 16\left(\frac{(Q_{df} \times n)}{s^{0.5}}\right)^{3/8}$$

$$D = 16\left(\frac{(1.36 \times 0.011)}{(0.005)^{0.5}}\right)^{3/8} = 8.94 \text{ in}$$

Upsize pipe diameter to the next available size, which is 10 inches.