
Storm Sewer Sizing

A. Introduction

The purpose of this section is to outline the basic hydraulic principles in order to determine the storm sewer size. The elements covered include basic flow formulas (Bernoulli Equation and Manning Equation), hydraulic losses, and hydraulic design of storm sewers. Information in this section was derived from FHWA's HEC-22 except where noted.

B. Definitions

Energy Grade Line: The energy grade line represents the total energy along a channel or conduit carrying water. For a fluid flowing without any losses due to friction (major losses) or components (minor losses) the energy grade line would be at a constant level. In practice, the energy grade line decreases along the flow due to these losses.

Hydraulic Grade Line: The hydraulic grade line equals the total head available to the fluid, minus the velocity head. Under open channel flow, the hydraulic grade line is at the water surface. Under pressure flow, the hydraulic grade line represents the level to which water would rise in piezometric pipes (or in manholes and intakes).

Pressure Head: Pressure head is the height of a column of water that would exert a unit pressure equal to the pressure of the water.

Velocity Head: Velocity head is a quantity proportional to the kinetic energy flowing water expressed as a height or head of water.

C. Hydraulics of Storm Sewers

1. **Flow Assumptions:** The design procedures presented here assume that flow within each storm sewer segment is steady and uniform. This means that the discharge and flow depth in each segment are assumed to be constant with respect to time and distance. Also, since storm sewers typically have a uniform pipe size within a segment, the average velocity throughout each segment is considered to be constant.

In actual storm sewer systems, the flow at each inlet is variable, and flow conditions are not truly steady or uniform. However, since the usual hydrologic methods employed in storm drain design are based upon computed peak discharges at the beginning of each run, it is a conservative practice to design using the steady uniform flow assumption.

2. **Open Channel vs. Pressure Flow:** Two design philosophies exist for sizing storm sewers under the steady uniform flow assumption. The first is referred to as open channel or gravity flow design. To maintain open channel flow, the segment must be sized so that the water surface within the conduit remains open to atmospheric pressure. For open channel flow, flow energy is derived from the flow velocity (kinetic energy), depth (pressure), and elevation (potential energy). If the water surface throughout the conduit is to be maintained at atmospheric pressure, the flow depth must be less than the height of the conduit.

Pressure flow design requires that the flow in the conduit be at a pressure greater than atmospheric. Under this condition, there is no exposed flow surface within the conduit. In pressure flow, flow energy is again derived from the flow velocity, depth, and elevation. The significant difference here is that the pressure head will be above the top of the conduit, and will not equal the depth of flow in the conduit. In this case, the pressure head rises to a level represented by the hydraulic grade line.

The question of whether open channel or pressure flow should control design has been debated. For a given flow rate, a design based on open channel flow requires a larger storm sewer than those sized based on pressure flow. While it may be more expensive to construct storm drainage systems designed for open channel flow, this design procedure provides a margin of safety by providing additional headroom in the conduit to accommodate an increase in flow above the design discharge. This factor of safety is often desirable since the methods of runoff estimation are not exact, and once placed, storm drains are difficult and expensive to replace.

Under ordinary conditions, it is recommended that storm drains be sized based on a gravity flow criteria at flow full or near full. Designing for full flow is a conservative assumption since the peak flow capacity actually occurs at 93% of the full flow depth. When allowed by the Jurisdiction, pressure flow may be used. When pressure flow is allowed, special emphasis should be placed on the proper design of the joints so that they are able to withstand the pressure flow on a regular basis.

- 3. Hydraulic Capacity:** A storm sewer's size, shape, slope, and friction resistance control its hydraulic capacity. These properties are all accounted for with the Manning Equation given as:

$$V = \frac{Q}{A} = \frac{1.486}{n} r^{2/3} s^{1/2} \quad \text{Equation 2D-2.01}$$

where:

V	= Average velocity, ft/s
Q	= Discharge, cfs
A	= Cross-sectional area of flow, ft ²
n	= Manning's roughness coefficient
r	= hydraulic radius, ft
	= A/p (note: for circular pipes flowing full, r=D/4)
p	= wetted perimeter, ft
s	= slope of hydraulic grade line (pipe/channel slope), ft/ft

Table 2D-2.01: Manning Coefficients for Common Storm Sewer Materials

Type of Pipe	Manning's n
Concrete pipe	0.013
PVC pipe (smooth wall)	0.010
HDPE or Polypropylene (corrugated exterior, smooth interior - dual or triple wall)	0.012
HDPE or Polypropylene (corrugated exterior and interior - single wall)	0.020
CMP (2-2/3" x 1/2" corrugations)	0.024
CMP (3" x 1" corrugations)	0.027
CMP (5"x1" corrugations)	0.025
Structural Plate	0.032

Note: for additional manning coefficients, see the pipe manufacturer's information.

D. Conservation of Energy

- Bernoulli Equation:** The law of conservation of energy, as expressed by the Bernoulli Equation, is the basic principle most often used in hydraulics. This equation may be applied to any conduit with a constant discharge. Friction flow formulas such as the Manning's Equation have been developed to express the rate of energy dissipation as it applies to the Bernoulli Equation. The theorem states that the energy head at any cross-section must equal that in any other downstream section plus the intervening losses.

Bernoulli's equation, where the total energy at Section 1 is equal to the energy at Section 2 plus the intervening head loss, is summarized in two versions below:

For open (non-pressure) conduit flow:

$$\frac{V_1^2}{2g} + Y_1 + Z_1 = \frac{V_2^2}{2g} + Y_2 + Z_2 + h_f \tag{Equation 2D-2.02}$$

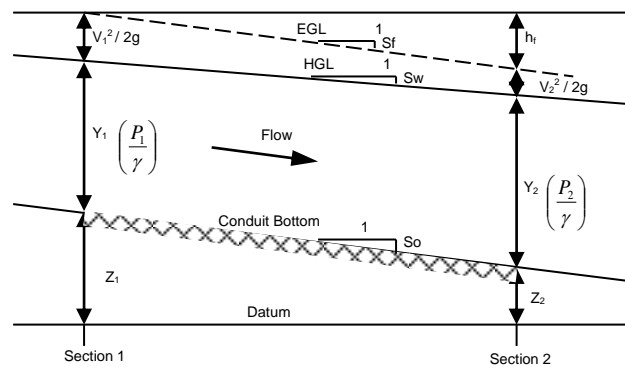
For pressure conduit flow

$$\frac{V_1^2}{2g} + \frac{P_1}{\gamma} + Z_1 = \frac{V_2^2}{2g} + \frac{P_2}{\gamma} + Z_2 + h_f \tag{Equation 2D-2.03}$$

where:

- EGL = Energy grade line
- HGL = Hydraulic grade line
- Y = Water depth, ft
- $V^2/2g$ = Energy head, ft
- V = Average velocity, fps
- S_f = Slope of EGL
- S_w = Slope of HGL
- g = acceleration of gravity (32.2 fps)
- P/γ = Pressure head, ft
- P = Pressure at given location (lb/ft²)
- γ = Specific weight of water (62.2 lb/ft³)
- Z = Elevation relative to some datum
- S_0 = Slope of bottom, ft/ft
- h_f = Head loss, ft

Figure 2D-2.01: Terms Used in the Energy Equation



Source: FHWA, HEC-22

E. Hydraulic Losses

Storm sewers should be designed to convey the minor storm runoff peaks without surcharging the sewer. In situations where surcharging is a concern, the hydraulic grade line may be calculated by accounting for pipe friction losses and pipe form losses. Total hydraulic losses will include friction, expansion, contraction, bend, and junction losses. The methods for estimating these losses are presented herein.

- 1. Pipe Friction Losses:** The major head loss in a storm drainage system is due to pipe friction. Energy losses from pipe friction may be determined by rewriting the Manning's equation with terms as previously defined:

$$S_f = 0.453 \frac{Q^2 n^2}{A^2 R^{4/3}} \quad \text{Equation 2D-2.04}$$

The friction head lost through a segment is simply the hydraulic gradient multiplied by the length of the run:

$$H_f = S_f L \quad \text{Equation 2D-2.05}$$

where:

H_f = Friction head loss, ft
 S_f = Friction slope, ft/ft
 L = Length of outflow pipe, ft

- 2. Exit Losses:** The exit loss from a storm sewer outlet is a function of the change in velocity at the outlet of the pipe. For a sudden expansion such as at an endwall, the exit loss is:

$$H_o = 1.0 \left[\left(\frac{V_o^2}{2g} \right) - \left(\frac{V_d^2}{2g} \right) \right] \quad \text{Equation 2D-2.06}$$

where:

V_o = Average outlet velocity, ft/s
 V_d = Channel velocity downstream of outlet in direction of the pipe flow, ft/s
 g = Acceleration due to gravity, 32.2 ft/s

Note that when $V_d = 0$, as in a reservoir, the exit loss is one velocity head. For part full flow where the pipe outlets in a channel with water moving in the same direction as the outlet water, the exit loss may be reduced to virtually zero.

- 3. Bend Losses:** The bend loss coefficient (H_b) for storm sewer design (for bends in the pipe run, not in a structure) can be estimated using the following formula:

$$H_b = K_b \frac{V^2}{2g} \quad \text{Equation 2D-2.07}$$

where:

K_b = Bend loss coefficient (refer to Table 2D-2.02)

Table 2D-2.02: Bend Loss Coefficients

Bend Radius / Pipe Dia. (R/d)	Degree of Bend		
	22.5°	45°	90°
1	0.12	0.17	0.23
2	0.07	0.10	0.13
4 or larger	0.04	0.06	0.08

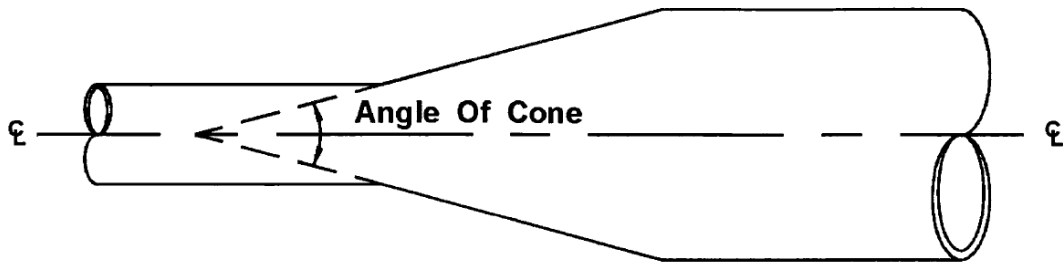
Source: Brater, King, et al.

- 4. Transition Losses:** A transition is a location where a conduit or channel changes size. Transitions include expansions, contractions, or both. In small storm sewers, transitions should occur within manhole or intake structures. However, in larger storm sewers, or when a specific need arises, expansions may occur within pipe runs. Contractions must always occur within a structure and never within the pipeline, regardless of pipe size.

Energy losses due to expansions in non-pressure flow can be expressed in terms of the kinetic energy at the two ends:

$$H_e = K_e \left[\left(\frac{V_2^2}{2g} \right) - \left(\frac{V_1^2}{2g} \right) \right] \quad \text{Equation 2D-2.08}$$

The head loss coefficient for a pipe expansion, K_e , is dependent upon the rate of change in diameter (angle of cone) as shown in Table 2D-2.03.

Figure 2D-2.02: Angle of Cone for Pipe Diameter Changes

Source: FHWA, HEC-22

Table 2D-2.03: Typical Values of K_e for Gradual Enlargement of Pipes in Non-pressure Flow

D_2 / D_1	Angle of Cone						
	10°	20°	45°	60°	90°	120°	180°
1.5	0.17	0.40	1.06	1.21	1.14	1.07	1.00
3	0.17	0.40	0.86	1.02	1.06	1.04	1.00

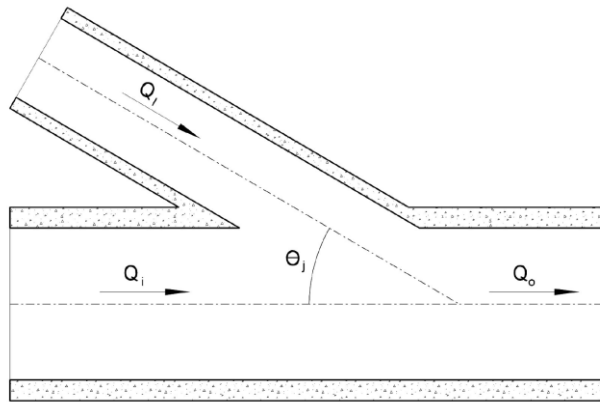
5. **Junction Losses:** A pipe junction is the connection of a lateral pipe to a larger trunk pipe without the use of a manhole or other structure. The minor loss equation for a pipe junction is a form of the momentum equation as follows:

$$H_j = \left\{ \frac{[(Q_o V_o) - (Q_i V_i) - (Q_l V_l \cos \theta_j)]}{[0.5g(A_o + A_i)]} \right\} + \frac{V_i^2}{2g} - \frac{V_o^2}{2g} \quad \text{Equation 2D-2.09}$$

where:

- H_j = Junction loss, ft
 Q_o, Q_i, Q_l = Outlet, inlet, and lateral flows respectively, ft³/s
 V_o, V_i, V_l = Outlet, inlet, and lateral velocities, respectively, ft³/s
 A_o, A_i = Outlet and inlet cross-sectional area, ft²
 θ = Angle between the inflow trunk pipe and inflow lateral pipe, degrees

Figure 2D-2.03: Interior Angle Definition for Pipe Junctions



6. **Structure Losses:** A complex situation exists where a manhole or intake exists at the junction between inflow and outflow pipes. The following method provides approximate results and estimates losses across a structure by multiplying the velocity head of the outflow pipe by a coefficient as represented in Equation 2D-2.10. Table 2D-2.04 tabulates typical coefficients (K_{ah}) applicable for use in this method. Refer to HEC-22 for a detailed explanation of analyzing structure losses.

$$H_{ah} = K_{ah} \left(\frac{V_{oi}^2}{2g} \right) \quad \text{Equation 2D-2.10}$$

This approximate method estimates the necessary elevation drop across a structure required to offset energy losses through the structure. This drop is then used to establish the appropriate pipe invert elevations.

Table 2D-2.04: Head Loss Coefficients through Structures

Pipe Angled Through	K_s
90°	1.00
60°	0.85
45°	0.75
22.5°	0.45
Straight Run	0.15

7. **Structure Drop:** Where pipe size increases in a structure, the invert of the smaller sewer must be raised to maintain the same energy gradient. An approximate method of doing this is to place the 0.8 depth point of both sewers at the same elevation. When there is a change in alignment between storm sewers of 45 degrees or greater, the suggested minimum manhole drop is 0.10 foot.

F. References

Brater, King, et al. *Handbook of Hydraulics*. Seventh Ed. 1996.

U.S. Department of Transportation. *Urban Drainage Design Manual*. Hydraulic Engineering Circular No. 22. Third Ed. 2009.