
2M-1 General Information for Storm Sewer Design

A. Introduction

Storm sewer facilities collect stormwater runoff and convey it away from structures and through the roadway right-of-way in a manner that adequately drains sites and roadways and minimizes the potential for flooding and erosion to properties. Storm sewer facilities consist of curbs, gutter, intakes, manholes, and storm sewers. The placement and hydraulic capacities of storm sewer facilities should be designed to take into consideration damage to adjacent property and to secure as low a degree of risk of traffic interruption by flooding as is consistent with the importance of the road, the design traffic service requirements, and available funds.

B. Definitions

Hydraulic grade line: The hydraulic grade line is the locus of elevations to which the water would rise in successive piezometer tubes if the tubes were installed along a pipe run.

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. Location of storm sewers

1. Storm sewers in street right-of-way

- a. Storm sewers parallel to the street and in the right-of-way should be placed behind the back of curbs, as close as practical, to fit specific manhole or intake connections.
- b. Storm sewers perpendicular to the street are to connect at each end by intakes or manholes.
- c. Storm sewers in the street right-of-way should be concrete pipe (Class III) to prevent utility cuts through the pipe. This includes storm sewer service stubs equal to or greater than 12 inches in diameter, 10 feet outside of the right-of-way. Class V concrete pipe should be used under railroad tracks (including jack pipe).
- d. If PVC or HDPE pipe is allowed by the Jurisdiction, it should be encased in flowable mortar. The reason for the encasement is to prevent utilities from unknowingly cutting or damaging the plastic pipe. If the flowable mortar encasement is not provided, the backfill envelope around the pipe can be disturbed. Since the pipe depends on the backfill envelope around the pipe for its strength, the pipe can be damaged if the backfill is disturbed. In areas where plastic pipe will not be adjacent to existing or future utilities, the encasement is not needed to maintain the pipe's integrity. An example would be an outfall sewer in a floodplain or a storm sewer outside of street right-of-way.

2. **Public storm sewers outside of street right-of-way but within public easement.** Storm sewers will be placed in a storm sewer public easement. Public storm sewer easements should have a minimum total width of 20 feet or two times the depth of the sewer, whichever is greater, with the storm sewer centered in the easement. Additional width may be required by the Jurisdictional Engineer to ensure proper access for maintenance purposes. When determining the width of the easement, consideration needs to be given to placement of excavated materials for the repair of the pipe.
 - a. Storm sewer outlets (mains) should be concrete pipe, particularly under pavements or where utilities exist or are proposed. Concrete pipe under parking lots and shoulders may be Class II pipe.
 - b. Upon the approval of the Jurisdictional Engineer, plastic pipe and CMP may be used outside of the street right-of-way where the granular backfill will not be disturbed by other utilities or other construction in the area.
 - c. Storm sewer along a side property line should run the length of the property line and outlet past the rear property line to a receiving drainageway.

D. Pipe materials

Pipe material and properties should conform to Table 1. Pipe loading information for storm sewer pipe was removed from this chapter and is currently being revised. It will be re-issued in the future.

Table 1: Typical Stormwater Piping¹

Typical Application	Pipe Material	Standard	Thickness Class (min.)	Pipe Stiffness (min.)	Size Range	Interior	Joints	Max. Depth of Bury ²
Storm sewer	RCP	ASTM C 76	Class III Wall B	N/A	15"-120"	Smooth	Tongue & Groove	7' to 40'
	RCAP	ASTM C 506	Class A III	N/A	Equiv. 15"-120"	Smooth	Tongue & Groove	6' to 14'
	RCEP	ASTM C 507	Class HE III Class VE III	N/A	Equiv. 15"-120"	Smooth	Tongue & Groove	N/A
	RCPP	ASTM C 361	Class C 25	N/A	15"-120"	Smooth	Tongue & Groove	N/A
Culverts	CMP	AASHTO M 36	Iowa DOT RF-32	N/A	18"-120"	Corrugated	Coupling Bands	N/A
	CMAF	AASHTO M 36	Iowa DOT RF-33	N/A	Equiv. 18"-120"	Corrugated	Coupling Bands	N/A
	RCP	ASTM C 76	Class III Wall B	N/A	18"-120"	Smooth	Tongue & Groove	7' to 40'
	Spiral Rib Metal Pipe	ASTM A 760	0.064"	N/A	18"-120"	Corrugated	Coupling Bands	N/A
	Spiral Rib Arch Pipe	ASTM A 760	0.064"	N/A	18"-120"	Corrugated	Coupling Bands	N/A
	Structural Plate	AASHTO M 167	Iowa DOT RF-33	N/A	60" +	Corrugated	Bolted	N/A
	Coated CMP	AASHTO M 274	Iowa DOT RF-32	N/A	18"-120"	Corrugated	Coupling Bands	N/A
	Aluminum Structural Plate	AASHTO M 219	0.100" (min)	N/A	60" +	Corrugated	Bolted	N/A
Subdrains or storm sewers	PVC-solid wall	ASTM D 3034, F 679	SDR 35	46 psi	6"-42"	Smooth	Bell & Spigot	24' to 30'
	Corrugated PVC	ASTM F 679	N/A	46 psi	6"-36"	Smooth	Bell & Spigot	24'
	Closed Profile PVC	ASTM F 1803	N/A	46 psi	21" -60"	Smooth	Bell & Spigot	24'
	Composite PVC	ASTM D 2680	N/A	46 psi	8" -15"	Smooth	Bell & Spigot	32'
	Corrugated HDPE	AASHTO M 294	N/A	Varies by diameter ³	12"-60"	Smooth or Corrugated	Bell & Spigot	8' to 9'
	Corrugated HDPE Tubing	AASHTO M 252	N/A	35 psi	6"-10"	Smooth or Corrugated	Soil Tight	8'
	CMP	AASHTO M 36	Type I	N/A	6"-15"	Corrugated	Coupling Bands	N/A
	RCP	ASTM 36	Class III	N/A	12"-120"	Smooth	Tongue & Groove	7' to 40'
Footing drain sewer collectors	PVC	ASTM D 3034	SDR 35	46 psi	8"-15"	Smooth	Bell & Spigot	24'
	Corrugated PVC	ASTM F 949	N/A	46 psi	8"-36"	Smooth	Bell & Spigot	24'
	Corrugated HDPE ³	AASHTO M 252	N/A	35 psi	8"-10"	Smooth or Corrugated	Soil Tight	9'
	Corrugated HDPE ³	AASHTO M 294	N/A	Varies by diameter	12"-24"	Smooth or Corrugated	Bell & Spigot	8' to 9'

¹ Site conditions may dictate something other than the typical piping.

² See Part 9B - Trench Design. Installations may be designed to exceed indicated maximum depth by modification of assumed conditions.

³ If less than 46 psi, must pass 5% deflection test after installation.

E. Physical requirements

1. **Minimum cover over storm sewer pipes.** The recommended minimum cover over storm sewer pipes should be 1 foot below the pavement slab, except for HDPE pipe, which should have a minimum cover of 2 feet; or as specified by the type of pipe as described in Chapter 9, whichever is greater. Where the clearance is less than 1 foot below the pavement, the Project Engineer will provide a design method to maintain the integrity of the pipe and pavement. For storm sewer pipe outside of the pavement, the minimum cover should be 1 foot 6 inches or as specified by the type of pipe (described in Chapter 9, Utilities), whichever is greater.
2. **Minimum flow line depth for footing drain sewers:** 3 feet 6 inches.
3. **Minimum pipe size**
 - a. **Storm sewers:** 15 inches in diameter.
 - b. **Subdrains:** 6 inches in diameter.
 - c. **Footing drain collector sewers in public right-of-way:** 8 inches in diameter.
 - d. **Building storm sewer stubs:** 4 inches in diameter
4. **Velocity within storm sewer pipe**
 - a. Minimum flow (1/2 full pipe) = 3 fps cleaning velocity
 - b. Maximum flow (1/2 full pipe) = 15 fps
5. **Velocity at outlet of pipe.** Energy dissipation is required when discharge velocities exceed those allowed for downstream channel. (See Section 2O-2, Tables 3 and 4).
 - a. With flared end section, maximum of 5 fps, and according to Section 2O-2, Tables 3 and 4, without any energy dissipation.
 - b. Maximum with flared end section, footing, and rip-rap = 10 fps
 - c. Maximum with energy dissipation device = 15 fps
6. **Partially full pipe flow.** For convenience, charts for various pipe shapes have been developed for calculating the hydraulic properties (Section 2M-4, Figures 1, 2, and 3). The data presented assumes that the friction coefficient, Manning's "n" value, does not vary throughout the depth.
7. **Minimum storm sewer and footing drain grades**
 - a. Velocity sets minimum grade for storm sewers and footing drain sewers, 2 fps for low-flow and 3 fps for design storm.
 - b. Cross runs - grades 1% minimum. Desired minimum velocity of 3 fps for design storm.
 - c. All other sewers - see Section 2M-4, Figure 3, Discharge of Circular Pipe-CFS
 - d. Building storm sewer stubs - 1% minimum

- e. Subdrains – 0.5% minimum
- 8. **Intakes:** See Section 2M-3.
- 9. **Manholes:** See Section 2M-3.

F. Horizontal alignment

Sewer will be laid with a straight alignment between structures with the following one exception: in subdivisions where street layouts are such that a straight alignment is difficult and the storm sewers are 54 inches in diameter or greater the sewers may be curved. The curvature will be factory fabricated pipe bends and should be concentric with the curvature of the street. The radius of curvature must not be less than 200 feet. The pipe manufacturer's recommended maximum deflection angle may not be exceeded.

2M-2 Groundwater Barriers and Outlets

A. Introduction

When there exists a possibility that groundwater may be diverted and follow the path of the new sewer, groundwater barriers should be constructed in adequate numbers to prevent groundwater migration down sewer trenches.

Subsurface barriers are designed to prevent or control groundwater flow into, through, or from a certain location. Barriers keep fresh groundwater from coming into contact with a contaminated aquifer zone or ground water from existing areas of contamination from moving into areas of clean groundwater. Usually it is necessary to incorporate other technologies, such as pump-and-treat systems, with groundwater barriers.

B. Groundwater barriers

The types of barriers commonly used include:

- Slurry trench walls
- Grout curtains
- Vibrating beam walls
- Bottom sealing
- Block displacement
- Sheet piles
- Sheet curtains

1. **Slurry trench walls.** Slurry trench walls are placed either upgradient from a waste site to prevent flow of groundwater into the site, downgradient to prevent off-site flow of contaminated water, or around a source to contain the contaminated groundwater. A slurry wall may extend through the water-bearing zone of concern, or it may extend only several feet below the water table to act as a barrier to floating contaminants. In the former case, the foundation should lie on, or preferably in, an underlying unit of low permeability so that contaminants do not flow under the wall. A slurry wall is constructed by excavating a trench at the proper location and to the desired depth, while keeping the trench filled with a clay slurry composed of a 5% to 7% by weight suspension of bentonite in water. The slurry maintains the vertical stability of the trench walls and forms a low permeability filter cake on the walls of the trench. As the slurry trench is excavated, it is simultaneously backfilled with a material that forms the final wall. The three major types of slurry backfill mixtures are soil bentonite, cement bentonite, and concrete. Slurry walls, under proper conditions, can be constructed to depths of about 100 feet.

Slurry trench walls are reported to have a long service life and short construction time, cause minimal environmental impact during construction, and be a cost-effective method for enclosing large areas under certain conditions. A concern regarding the use of a slurry wall where contaminated materials are in direct contact with the wall is the long-term integrity of the wall. In such cases, the condition of the wall needs to be verified over time by groundwater monitoring.

2. **Grouting curtains.** Grouting is the process of pressure-injecting stabilizing materials into the subsurface to fill, and thereby seal, voids, cracks, fissures, or other openings. Grout curtains are underground physical barriers formed by injecting grout through tubes. The amount of grout needed is a function of the available void space, the density of the grout, and the pressures used in setting the grout. Two or more rows of grout are normally required to provide a good seal. The grout used may be either particulate (i.e., portland cement) or chemical (i.e., sodium silicate) depending on the soil type and the contaminant present. Grouting creates a fairly effective barrier to groundwater movement, although the degree of completeness of the grout curtain is difficult to ascertain. Incomplete penetration of the grout into the voids of the earth material permits leakage through the curtain.
3. **Vibrating beam walls.** A variation of the grout curtain is the vibrating beam technique for placing thin (approximately 4 inches) curtains or walls. Although this type of barrier is sometimes called a slurry wall, it is more closely related to a grout curtain since the slurry is injected through a pipe in a manner similar to grouting. A suspended I-beam connected to a vibrating driver-extractor is vibrated through the ground to the desired depth. As the beam is raised at a controlled rate, slurry is injected through a set of nozzles at the base of the beam, filling the void left by the beam's withdrawal. The vibrating beam technique is most efficient in loose, unconsolidated deposits, such as sand and gravel.
4. **Bottom sealing.** Another method that uses grouting is bottom sealing, where grout is injected through drill holes to form a horizontal or curved barrier below the site to prevent downward migration of contaminants.
5. **Block displacement.** Block displacement is a relatively new plume management method, in which a slurry is injected so that it forms a subsurface barrier around and below a specific mass or "block" of material. Continued pressure injection of the slurry produces an uplift force on the bottom of the block, resulting in a vertical displacement proportional to the slurry volume pumped.
6. **Sheet piles.** Sheet pile cutoff walls have been used for many years for excavation bracing and dewatering. Where conditions are favorable, depths of 100 feet or more can be achieved. Sheet piling cutoff walls can be made of wood, reinforced concrete, or steel, with steel being the most effective material for constructing a ground-water barrier. The construction of a sheet pile cutoff wall involves driving interlocking sheet piles down through unconsolidated materials to a unit of low permeability. Individual sheet piles are connected along the edges with various types of interlocking joints. Unfortunately, sheet piling is seldom water-tight and individual plates can move laterally several to several tens of feet while being driven. Acidic or alkaline solutions, as well as some organic compounds, can reduce the expected life of the system.
7. **Sheet curtains.** Membrane and synthetic sheet curtains can be used in applications similar to grout curtains and sheet piling. With this method, the membrane is placed in a trench surrounding or upgradient of the plume, thereby enclosing the contaminated source or diverting groundwater flow around it. Placing a membrane liner in a slurry trench application also has been tried on a limited basis. Attaching the membrane to an underlying confining layer and forming perfect seals between the sheets is difficult but necessary in order for membranes and other synthetic sheet curtains to be effective.

Source: The Pan American Center for Sanitary Engineering and Environmental Sciences, CEPIS.

C. Outlets

1. Where a storm sewer discharges into a natural channel or irrigation ditch, an outlet structure should be provided that will blend the storm sewer discharge into the natural channel flow in such a way as to prevent erosion of the bed or banks of the channel. As a minimum, all storm sewer pipes that outlet to drainageways will require flared end sections with apron guard for pipe diameters 18 inches or larger. Storm sewers 30 inches in diameter or greater require a footing at the outlet. Footings may be required for pipe diameters less than 30 inches.
2. In an instance where the discharge velocity is high (higher than those outlined in Section 2O-2, Tables 3 and 4) or supercritical, prevention of erosion of the natural channel bed or banks in the vicinity of the outlet requires an energy dissipating structure, such as:
 - Riprap
 - Concrete slab
 - Gabions
 - Headwalls and wing wall with stilling basins
3. Outlets should drain at a receiving drainageway or connect to an existing storm sewer. Outlets will not drain across sidewalks or directly to streets. Outlets should not be located on slopes without adequate erosion protection and means of conveyance between the outlet and receiving drainageway or storm sewer. Erosion protection on a slope that does not extend beyond the outlet is often inadequate, as runoff velocity will increase down grade of the outlet.

2M-3 Street Flow and Intake/Manhole Capacity

A. Introduction

Storm sewer intakes are the main access points by which urban runoff enters the storm sewer system. In fact, the storm sewer intake is an important element of the design in its own right. The hydraulics of flow into an inlet are based on principles of weir and orifice flow, modified by laboratory and field observation of entrance losses under controlled conditions.

Curb and gutter intakes are installed along street sections having curbs and gutters to intercept stormwater runoff and to allow its passage into a storm sewer. Inlets can be located at low points (sumps), directly upstream from street intersections and at intermediate locations as well. The spacing of these intermediate curb inlets depends on several criteria but is usually controlled by rate of flow and the permissible water spread toward the street crown. The type of road is also important since the greater the speed and volume of traffic, the greater the potential for hydroplaning. On the other hand, it is also considered acceptable practice to allow some periodic and temporary flooding of low volume streets (see Section 2A-4 for criteria).

B. Definitions

Frontal flow: The portion of the flow that passes over the upstream side of a grate.

Side-flow interception: Flow that is intercepted along the side of a grate inlet, as opposed to frontal interception.

Slotted drain inlet: A drainage inlet composed of a continuous slot built into the top of a pipe that serves to intercept, collect, and transport the flow.

Splash-over: Portion of the frontal flow at a grate that skips or splashes over the grate and is not intercepted.

Storm sewer intake: A storm sewer intake is an opening into a storm sewer system for the entrance of surface storm runoff. There are four basic types of intakes:

- Curb-grate opening
- Curb opening (or open throat)
- Combination intakes (street intake with manhole)
- Grate intake only (parking lot, driveway, or ditch intake)

In addition, intakes may be further classified as being on a continuous grade or in a low point. The term "continuous grade" refers to an intake so located that the grade of the street has a continuous slope past the intake and therefore ponding does not occur at the intake. The sump or low point condition exists whenever water is restricted to the inlet area because the intake is located at a low point. A low point condition can occur at a change in grade of the street from positive to negative or due to the crown slope of a cross street when the intake is located at an intersection.

Storm sewer manhole: A storm sewer manhole is an access structure for storm sewers.

1. **Design flow.** Design flow is defined as that quantity of water at a given point calculated from the design storm runoff. For gutter applications, design flow should include bypass flow from upstream intakes.
2. **Bypass flow.** Bypass flow is defined as the flow in the gutter that is not intercepted by a given intake. Bypass flow is calculated by subtracting the allowable capacity of the given intake from the design flow assigned to that intake. Bypass flow is added to the design storm runoff for the next downstream intake. As a minimum, intakes at a low point will have design capacity equal to the assigned storm discharge plus upstream bypass flows.

C. Intercepting flows

Storm sewer intakes should be designed to intercept design flow with the following allowable bypass from the system:

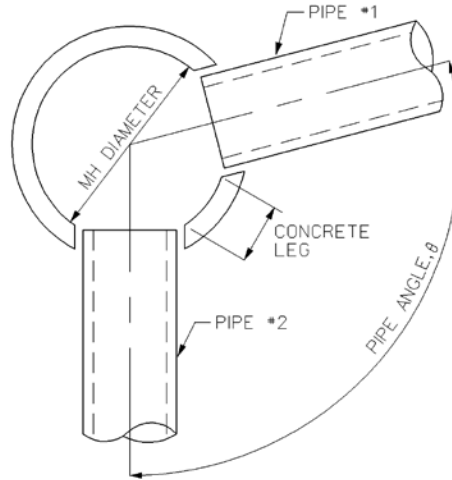
1. **Streets on continuous grade.** Downstream intakes should be designed to intercept no less than 50% of the design flow.
2. **Low points and dead-end streets on down grade.** Unless otherwise approved by the Jurisdictional Engineer, intakes should be designed to intercept 100% of the design Flow, or according to Figure 1.

D. Storm sewer structure locations

1. **Access ports (manholes/intakes).** Manholes or other access ports (intakes) are required under the following conditions:
 - a. At the end of each sewer line
 - b. At all changes in pipe size, elevation and grade, or alignment, and at all bends
 - c. At all sewer pipe intersections, except where the size of the storm sewer conduit (54 inches diameter or greater pipe) eliminates the need for a maintenance access. Manholes are required for 54 inches or greater pipes when direct access is desired every 400 feet.
 - d. At all sewer pipe intersections and at intervals not exceeding 400 feet, unless cleaning equipment allows a greater spacing interval. Maximum interval is 500 feet.
2. **Openings**
 - a. **Standard.** The minimum size for an access port (manhole) is 48 inches in diameter. Jurisdictions require concentric manholes, without built-in steps, with the manhole opening over the centerline of the pipe or on an offset not to exceed 12 inches. Some Jurisdictions may allow for eccentric manholes.
 - b. **Special.** For square or rectangular manholes, the manhole openings should be over the centerline of the pipes or on an offset not to exceed 12 inches. The distance from the centerline of the manhole opening to the face of the inside manhole wall should not exceed 30 inches to better facilitate video inspection and maintenance equipment. This may require more than one manhole opening.

c. **Determining diameters.** When utilizing circular precast manholes, it is necessary to determine the diameter required to maintain the structural integrity of the manhole. As a general rule, a minimum concrete leg of 6 inches should remain between the manhole blockouts for adjacent pipes. Determining the required manhole diameter to provide this minimum distance may be done as follows:

- 1) Determine the diameters of, and the angle between, the two pipes in question. If more than two pipes connect at the manhole, the adjacent pipes with the critical configuration (i.e. smallest angle and largest pipes) should be selected. If the critical configuration is not apparent, calculations may be required for all adjacent pipes.



- 2) Determine the blockout diameter. The blockout is the opening provided in the manhole for the pipe. Blockout dimensions are based on the outside diameter of the pipe. For storm sewer, a circular or doghouse type opening is provided with additional clearance to allow for the insertion of the pipe and sufficient space to accommodate placement of concrete grout in the opening. Typical blockout dimensions for various pipe sizes and materials are given in Table 1 below.

Table 1: Manhole Blockout Sizes

Pipe Dia.	Manhole Blockout, in		
	RCP	PVC	DIP
12"	21"	16"	16"
14"	N/A	16"	18"
15"	24"	19"	N/A
16"	N/A	N/A	20"
18"	28"	22"	23"
20"	N/A	N/A	24"
21"	31"	25"	N/A
24"	35"	28"	29"
27"	38"	31"	N/A
30"	42"	35"	36"
33"	47"	N/A	N/A
36"	48"	42"	41"
42"	57"	N/A	N/A
48"	64"	N/A	N/A
54"	71"	N/A	N/A
60"	78"	N/A	N/A

- 3) Determine the diameter of the manhole required to provide the minimum concrete leg dimension. This diameter may be calculated with the following equation:

$$MH_d = \frac{BO_1 + BO_2 + 2CL}{\theta \times \left(\frac{\pi}{180}\right)} \quad \text{Equation 1}$$

where:

MH_d = Manhole Diameter, inches

BO = Blockout Diameter, inches

CL = Minimum Concrete Leg Length, inches (6 inches)

θ = Angle between pipe centerlines, degrees

- 4) Round the minimum manhole diameter calculated, up to the next standard manhole size (48 inches, 60 inches, 72 inches, 84 inches, 96 inches, 108 inches, or 120 inches).
- 5) Verify that the manhole diameter calculated is sufficient for the largest pipe diameter (See Table 2).

Table 2: Minimum Manhole Diameter Required for Pipe Size

<i>Pipe Dia.</i>	Minimum Manhole Diameter Required for Pipe		
	<i>RCP</i>	<i>PVC</i>	<i>DIP</i>
8"	N/A	48"	48"
10"	N/A	48"	48"
12"	48"	48"	48"
14"	N/A	N/A	48"
15"	48"	48"	N/A
16"	N/A	N/A	48"
18"	48"	48"	48"
20"	N/A	N/A	48"
21"	48"	48"	N/A
24"	48"	48"	48"
27"	*60"	48"	N/A
30"	*60"	*60"	*60"
33"	*60"	N/A	N/A
36"	*60"	*60"	*60"
42"	*72"		
48"	*84"		
54"	*96"		
60"	*96"		

*48 inch diameter Tee-section manhole may be used for pipes 27 inches and greater.

2. **Combination intakes.** Intakes with combined manholes will be used when the size of the connecting pipes so indicate or when horizontal clearance is necessary behind the back of curb. The Project Engineer is encouraged to place intakes combined with manholes for storm sewers that are parallel to the street. This will prevent storm sewers from being installed under pavement and thus improves future maintenance access without removing pavement. Approval will be required by the Jurisdictional Engineer when storm sewers or footing drains parallel to the street are placed under the pavement.

3. **Cleanouts.** Lamp holes or clean-out structures are required at the beginning of footing drains and subdrains in street right-of-way. Cleanouts may be permitted in place of a manhole at the end of lines which are less than 150 feet in length.
4. **Access spacing.** Storm sewer structures (manholes, intakes, combination intakes, or cleanouts) in street right-of-way must be located in areas which allow direct access by maintenance vehicles. Areas outside the street right-of-way will be subject to the approval of the Jurisdictional Engineer.
 - a. **Manhole spacing**
 - 1) Manholes are to be spaced at intervals not exceeding 400 feet for sewers 24 inches or less at intervals not exceeding 500 feet when adequate cleaning equipment is available.
 - 2) Spacing of manholes over 500 feet may be permitted in sewers larger than 24 inches if the owner has adequate cleaning equipment.
 - b. **Intake spacing.** Locate street intakes upgrade from intersections, sidewalk ramps, and outside of intersection radius. At least one intake is to be installed at the low point of the street grade.
 - 1) **First intake.** An intake should be located no further than 500 feet from the street high point.
 - 2) **Remaining intakes.** To be spaced at a distance no greater than 400 feet regardless of gutter flow capacity.
1. **Invert drop.** When there is a change in pipe size at 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 sewer of 45° or greater, the suggested minimum manhole drop is 0.10 foot.

E. Intake capacity

The capacity of an intake is decreased by such factors as debris plugging, pavement overlaying, etc. Therefore, the allowable capacity of an intake is determined by applying the applicable reduction factor from the following table to the theoretical capacity calculated from the design procedures outlined in this section. These reduction factors are based on vane grates, which are required on all curb grate intakes within the street. Other intake grates may be approved by the Jurisdictional Engineer outside of the street right-of-way. The Iowa DOT normally requires curb only intakes on primary roads.

Table 3: Reduction Factors to Apply to Intakes

Intake Type	Location	Reduction Factor ¹	Intake Description
SW-501, SW-502, SW-503, and SW-504	Continuous Grade	90% Vane Grates with Curb	Single Grate with Curb Opening
	Low Point	80% Vane Grates with Curb	
SW-505 and SW-506	Continuous Grade	90% Vane Grates with Curb	Double Grate with Curb Opening
	Low Point	80% Vane Grates with Curb	
SW-507 and SW-508	Continuous Grade	80% Curb Only (No Grate)	Single Open Throat
	Low Point	70% Curb Only (No Grate)	
SW-509 and SW-510	Continuous Grade	80% Curb Only (No Grate)	Double Open Throat
	Low Point ²	70% Curb Only (No Grate)	
SW-501 and SW-502 (Driveway Grate)	Continuous Grade	75% Grate Only (No Curb)	Single Grate Only
	Low Point	65% Grate Only (No Curb)	

¹ Minimum reduction factor is to be used to reduce intake capacity.

² Use of driveway grates at low points is discouraged due to their tendency to become plugged with debris and flood the surrounding area. Obtain permission of the Jurisdictional Engineer prior to placing a driveway grate in a low point. If allowed, the Jurisdictional Engineer may also require installation of standard curb intake(s) immediately upstream of the driveway.

1. Abbreviations

- a = Intake Depression in Inches
- d = Depth of Flow in Gutter at Face of Curb
- h = Height of the Curb Opening
- H = Total Head in Feet = d + a in Feet
- K = Value Used in Equation $Q = Kd$ (See Figures 3 and 5)
- L = Length of Curb Opening in Feet
- n = Manning's Roughness Coefficient = 0.016
- Q = Discharge in cfs
- Q_{CO} = Flow Intercepted by the Curb Opening
- Q_G = Flow Intercepted by the Grate
- Q_I = Allowable Flow Intercepted by the Intake
- R_F = Reduction Factor
- S_L = Longitudinal Street Slope
- S_T = Transverse Street Slope or Crown
- T = Spread of Water in Gutter From Face of Curb
- Z = 1/S_T = Reciprocal of Transverse Slope

2. Equations

- a. **Capacity of gutter for straight crown.** Figure 1 is the nomograph used to determine the gutter capacity for a straight crown or segmented straight crown. Figure 1 can also be used to approximate the capacity of curved crowns.
- b. **Capacity of grate type intakes on a continuous grade.** The allowable capacity of SW-501, 502, 503, 504, 505, and 506 grate-type intakes on a continuous grade is determined by the following equation:

$$Q_I = K \left(d^{\frac{5}{3}} \right) (R_F)$$

Figure 3 is used to determine "K" for a vane grate and includes the curb hood. Figure 4 gives "K" for a driveway condition where no curb hood can be used. The appropriate reduction factor from Table 1 must then be applied to obtain the actual flow intercepted by the intake.

- c. **Capacity of curb opening (open-throat) intakes on a continuous grade.** Figure 2 is used to determine the interception ratio of the SW-507, 508, 509, and 510 intakes. This theoretical interception ratio (Q_i/Q) multiplied by the design flow in the gutter and the reduction factor equals the flow intercepted by the intake.
- d. **Capacity of grate-type and curb opening-type intakes at a low point.** Figure 5 is used to determine the capacity Q of a grate with curb opening, grate only, and curb-opening type intakes at a low point. The appropriate reduction factor must be applied to the results.

Figure 1: Nomograph for Capacity of the Gutter for Straight Crown

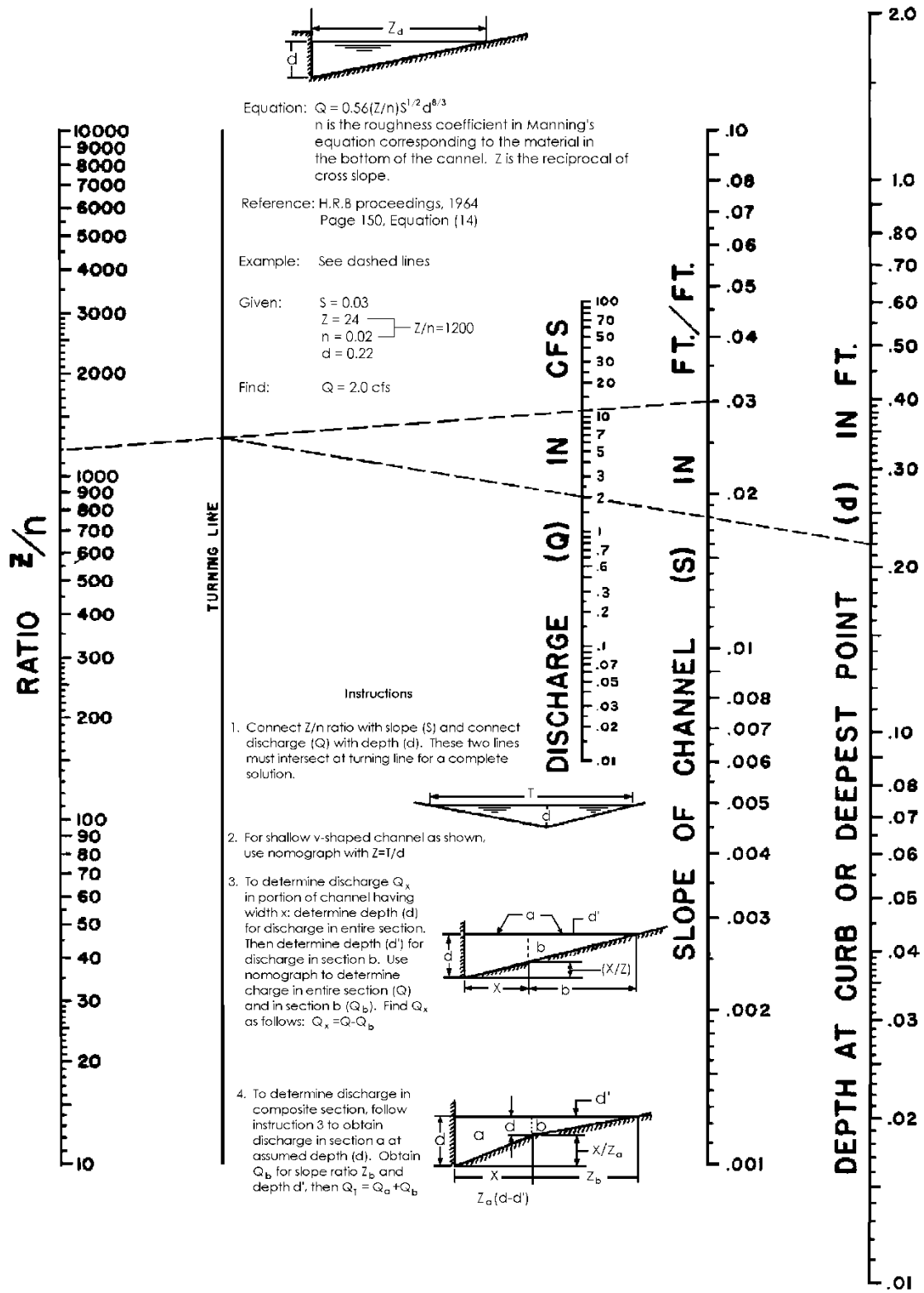


Figure 2: Interception Ratio for Standard Type SW-507, 508, 509, and 510 Intakes on Continuous Grade

STANDARD CURB - OPENING INLET CHART

EXAMPLE

W = 2 ft
 a = 2 in
 h = 6 in

Given: $S_T = 0.02$ ft/ft
 T = 10 ft
 $S_L = 0.03$ ft/ft
 Find: $L_i = 4$ ft $L_i = 8$ ft

$Q_i/Q = .22$ $Q_i/Q = 0.48$
 $d_w = S_T(T-2)$

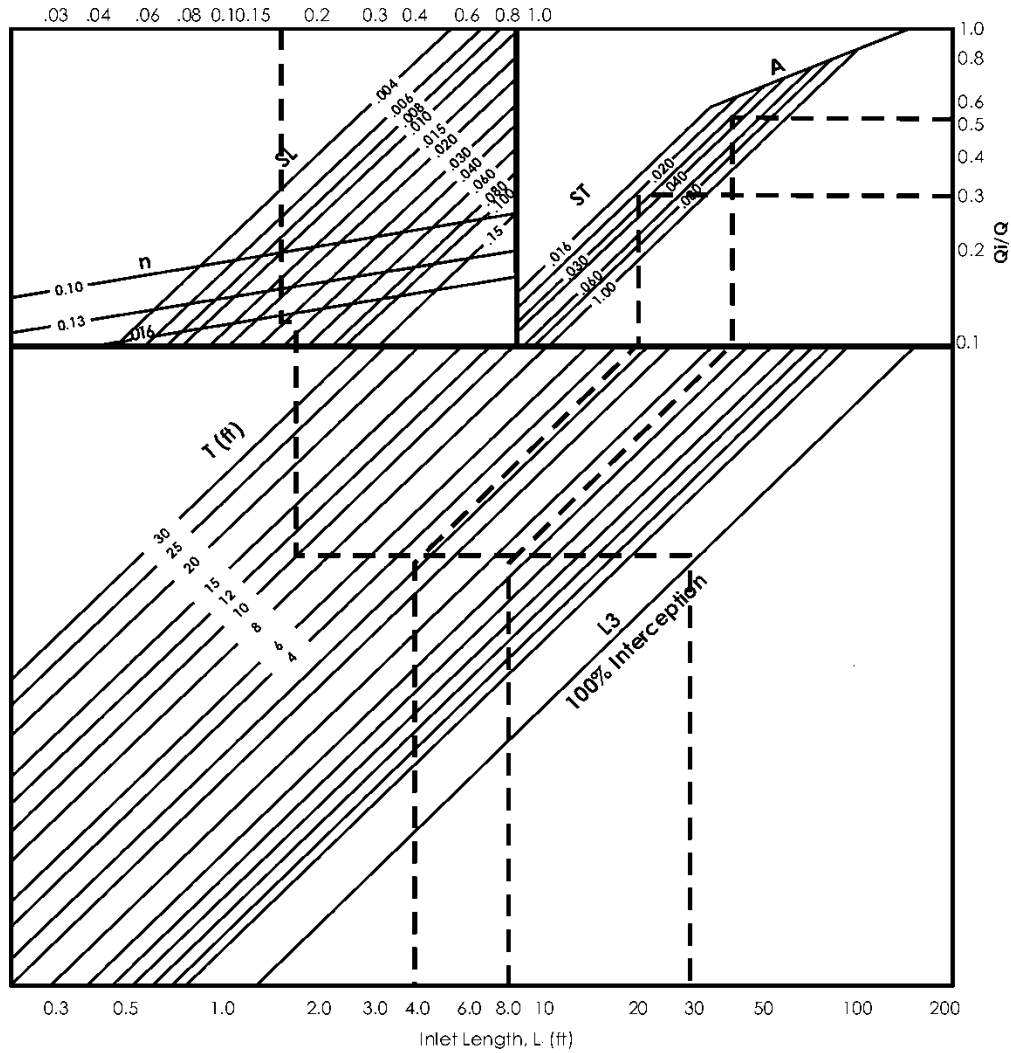


Figure 3: "K" Values for Standard Type SW-501, 502, 503, 504, 505, and 506 Intakes - Vane Grate and Curb Hood

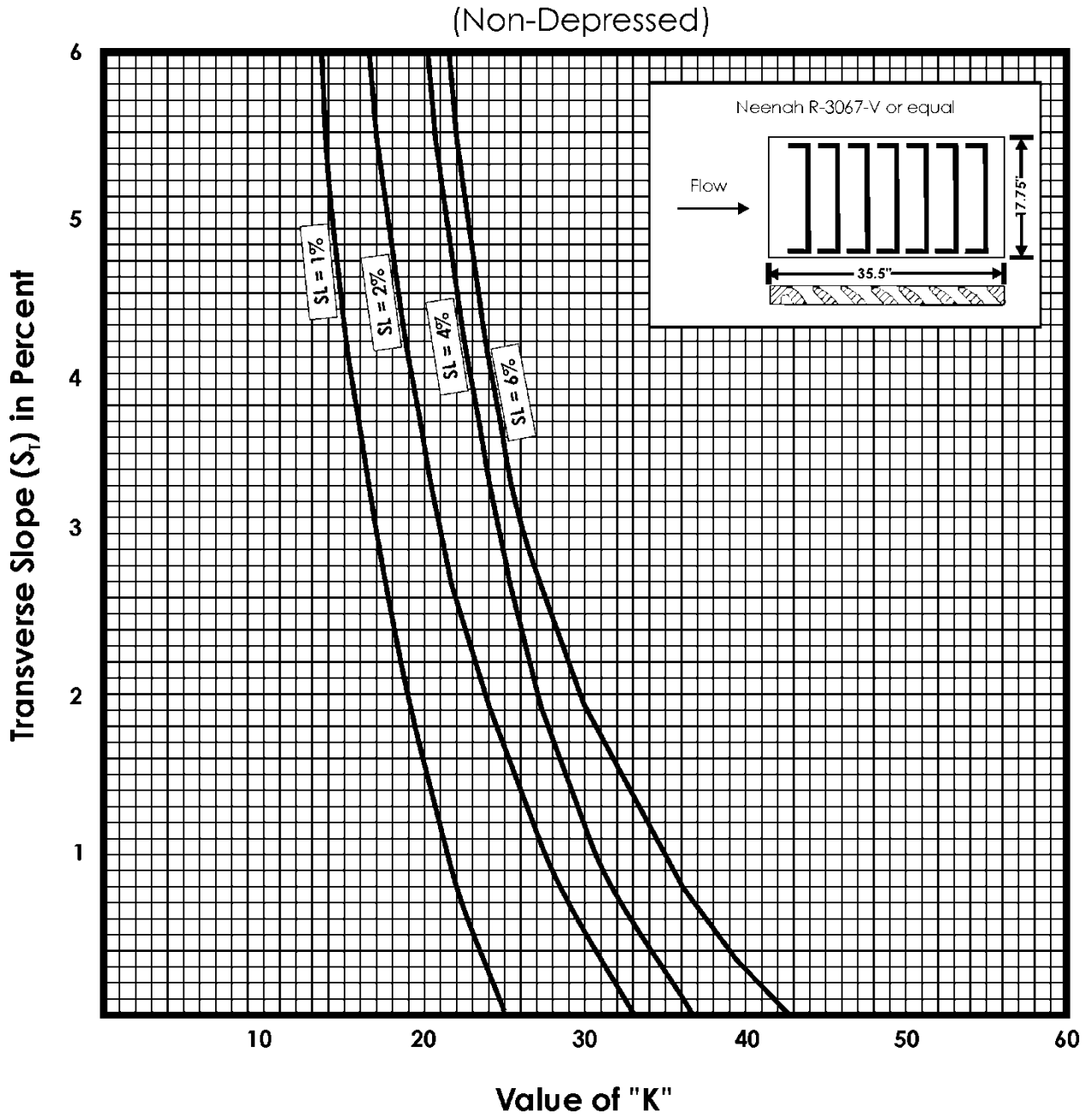


Figure 4: "K" Values for Driveway Grate Intake
(Non-Depressed)

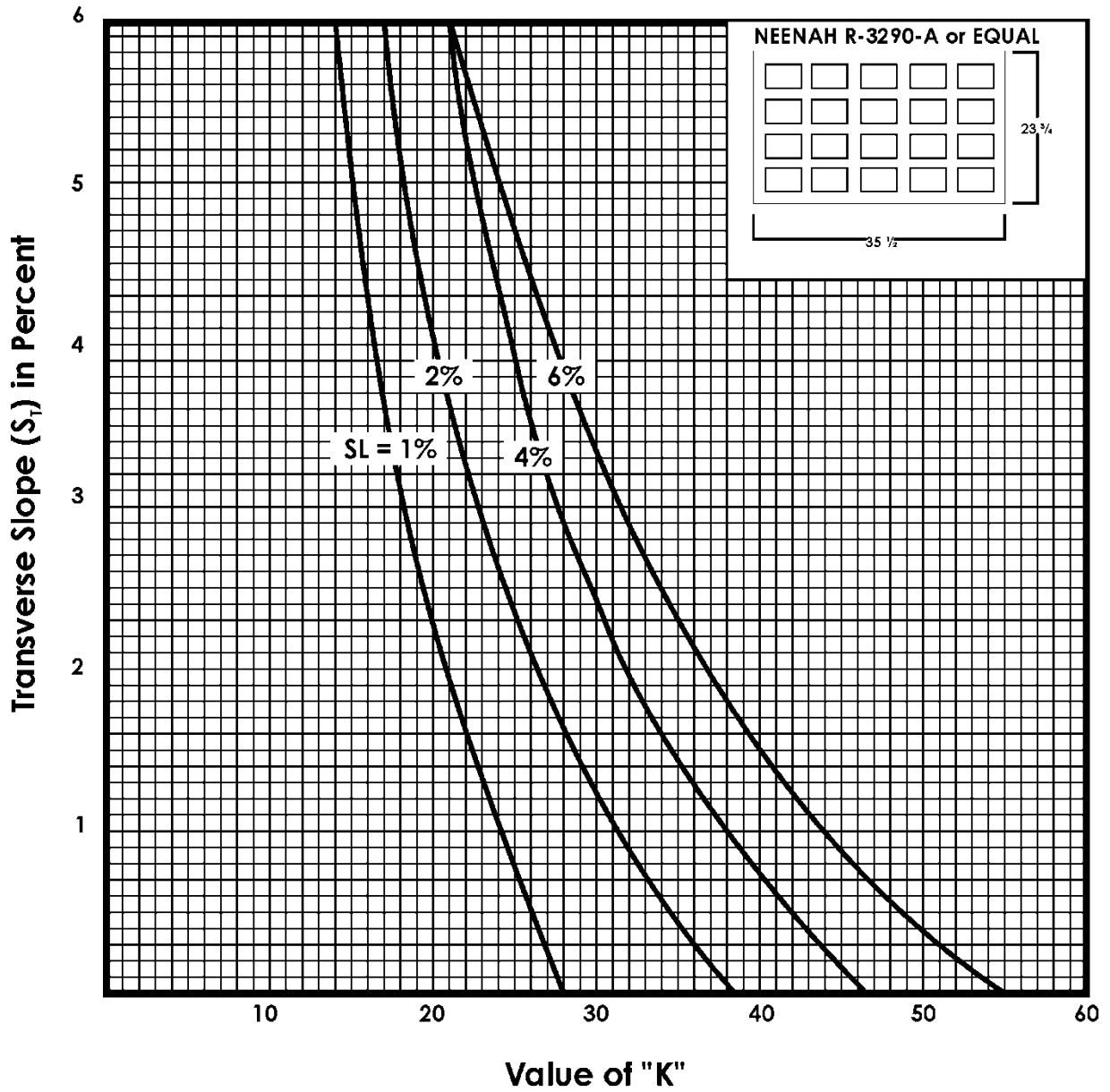
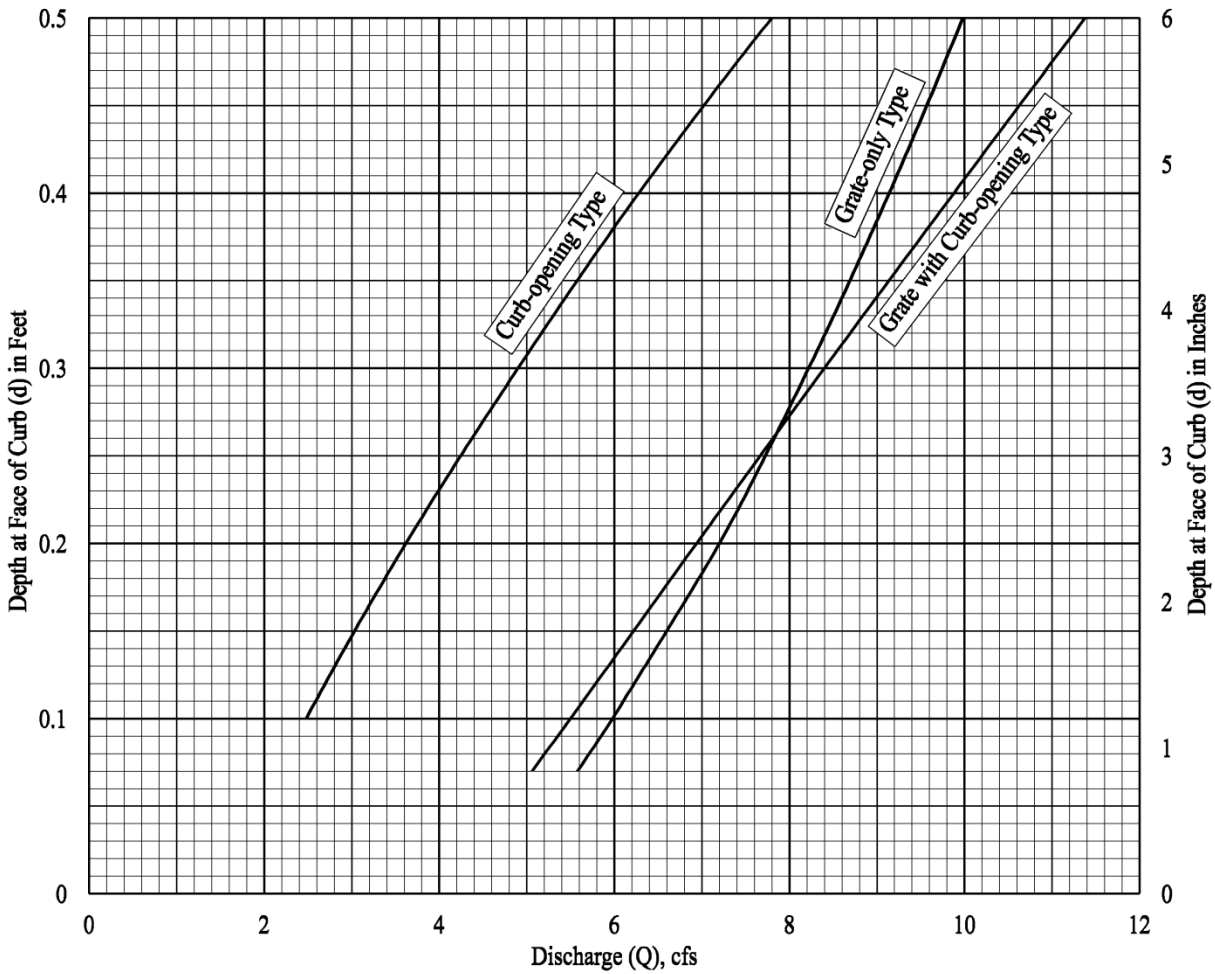


Figure 5: Capacity of Standard Single Type Intakes at a Low Point



Equations for above Curves

Curb-opening, Types SW-507, 508, 509, and 510

Grate with Curb-opening, Types SW-501, 502, 503, 504, 505, and 506

Grate only, Types SW-501 and 502 without curb hood

$$Q = 12h^{3/2}$$

$$Q = 8.44h^{1/2} + 8.25h^{3/2}$$

$$Q = 12.62h^{1/2}$$

where $h = d(\text{ft}) + A(\text{ft})$

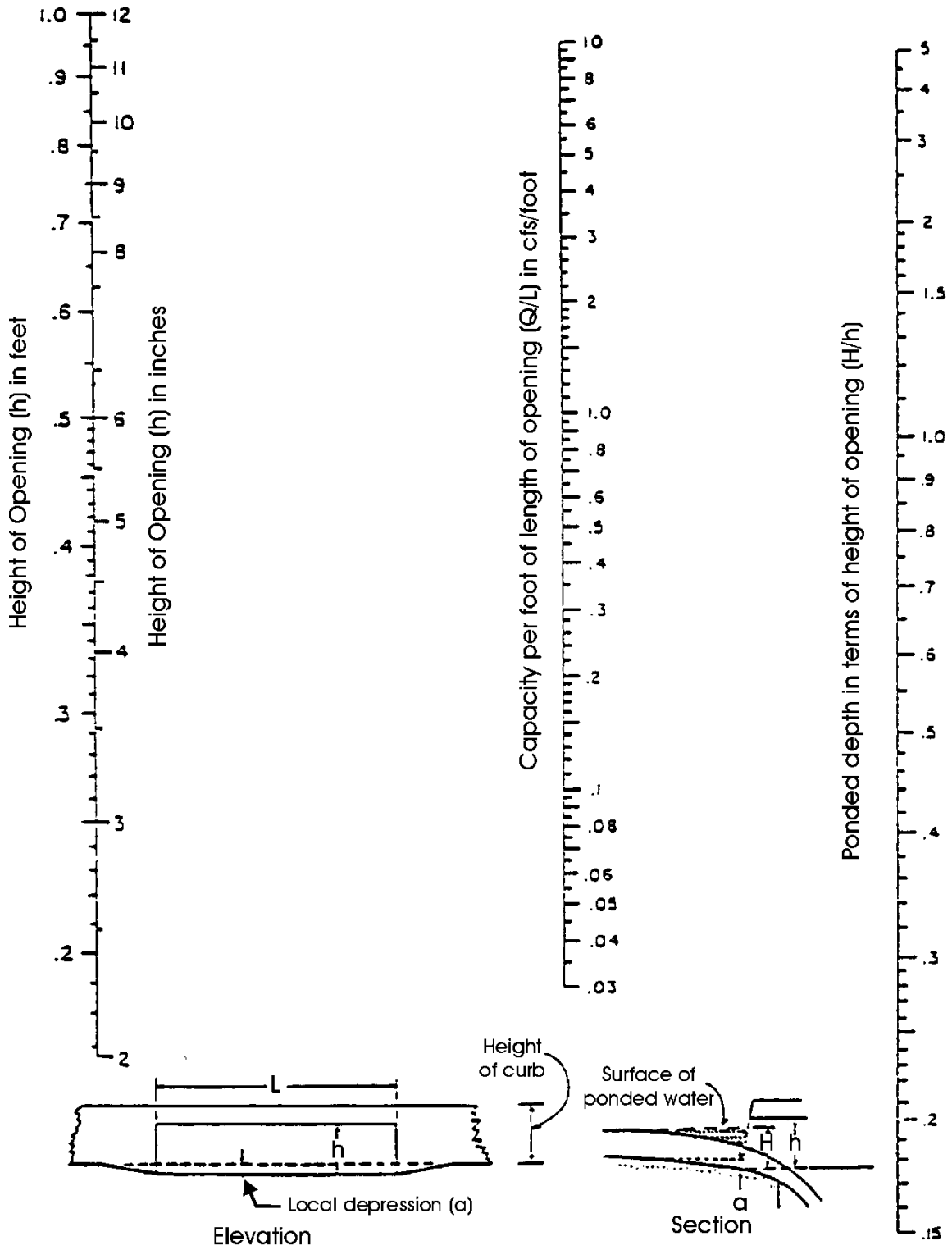
A = 3" (0.25') for Curb-opening

A = 2" (0.167') for Grate with Curb-opening

A = 1.5" (0.125') for Grate only

Note: For double intakes, take the values calculated for single intakes times two.

Figure 6: Capacity of Curb Opening Intake at Low Point



F. Manhole and intake standards

1. Manhole standards to be utilized:

Type	Description	Use	
		Main Pipe Size	Depth Restrictions
SW-401 Fig. 6010.401	Circular Storm Sewer Manhole	12" min. See table on Fig. 6010.401 for max. pipe size	N/A
SW-402 Fig. 6010.402	Rectangular Storm Sewer Manhole	12" to 54"	8' max.
SW-503 Fig. 6010.403	Deep Well Rectangular Storm Sewer Manhole	12" to 72"	12' max.
SW-404 Fig. 6010.404	Rectangular Base/Circular Top Storm Sewer Manhole	12" to 96"	12' min. - 22' max.
SW-405 Fig. 6010.405	Tee-section Storm Sewer Manhole	12" or greater	N/A

2. Manhole castings to be utilized:

Figure No. ¹	Casting Type	Number of Pieces	Ring/Cover	Bolted Frame	Bolted Cover (Floodable)	Gasket
6010.602	E	2	Fixed ²	Yes	No	No
6010.602	F	3	Adjustable ³	No	No	No
¹ The figure numbers listed in this table refer to figures from the SUDAS Standard Specifications.						
² Typically used with non-paved or flexible surfaces, including HMA, seal coat, gravel, and brick.						
³ Typically used with PCC surfaces, including castings in concrete boxouts.						

3. Intake standards to be utilized:

Intake Type	Intake Casting	Standard	Conditions
Curb-Grate SW-501	SW-603 Type Q	Single, poured 6" walls	Intake depth $\leq 7'$ Pipe size: 18" max. on 2' side, 30" max. on 3' side
Curb-Grate SW-502	SW-603 Type Q	Single, precast walls	Intake depth $> 7'$ Pipe size: 24" max. for 48" diameter
Curb-Grate (Combination) SW-503/SW-504	SW-603 Type Q	Single, poured 6" walls	Intake depth $\leq 6' 6''$ Pipe size: 30" max. on 3' side, 36" max. on 6' side
Curb-Grate SW-505	SW-603 Type Q	Double, poured 6" walls	Intake depth $\leq 7'$ Pipe size: 18" max. on 2' side, 66" max. on 6' 8" side
Curb-Grate (Combination) SW-506	SW-603 Type Q	Double, poured 6" walls	Intake depth $\leq 6' 6''$ Pipe size: 30" max. on 3' side, 36" max. on 6' side, 48" max. on 6' 8" side
Curb Only SW-507	N/A	Single open throat, poured 6" walls	Intake depth $\leq 10'$ Pipe size: 30" max. on 3' side, 36" max. on 4' side
Curb Only SW-508	N/A	Single open throat, poured 6" walls	Intake depth $\leq 16'$ Pipe size: 36" max.
Curb Only SW-509	N/A	Double open throat, poured 6" reinforced walls	Intake depth $\leq 10'$ Pipe size: 30" max. on 3' side, 66" max. on 8' side
Curb Only SW-510	N/A	Double open throat, poured 6" reinforced walls	Intake depth $< 10'$ Pipe size: 36" max. on 4' side, 66" max. on 8' side
Driveway or Alley Grate Intake SW-511	SW-604 Type 6	Single (Surface Intake) Poured 6" walls	Intake depth $\leq 7'$ Pipe size: 18" max. on 2' side, 30" max. on 3' side
Area Intake SW-512	SW-604 Type 3, 4, or 5	Precast, Area Intake	Intake depth $> 7'$ Pipe size varies on structure size
Ditch Intake SW-513	SW-602 Type E	Area Intake (side open intake) Poured 6" walls	Intake depth $\leq 7'$ Pipe size varies on structure size

4. Concrete poured walls are required for all other drainage structures. Cure time is required for poured wall intakes unless high early strength concrete is used, or concrete beams are taken. Upon approval of the Jurisdictional Engineer solid concrete block walls may be used on shallow structures when the depth from the gutter flowline to the pipe invert does not exceed 40 inches. Restrictions on the number of pipe connections and angle of entry may be imposed on solid concrete block intakes without combination manholes.
5. Combination Intakes may be required if utility locations and/or pipe size show a need for the manhole. Intakes with combined manholes will be used when the size of the connecting pipes so indicate or when horizontal clearance is necessary behind the back of curb. The Design Engineer is encouraged to place intakes combined with manholes for storm sewers that are parallel to the street. This will prevent storm sewers from being installed under pavement and thus improves future maintenance access without removing pavement. Approval will be required by the Jurisdictional Engineer when storm sewers or footing drains parallel to the street are placed under the pavement.

G. Intake capacity design example

1. **Capacity of gutter for straight crown.** Figure 1 is the nomograph used to determine the gutter capacity for a straight crown or segmented straight crown. Figure 1 can also be used to approximate the capacity of curved crowns.

- a. **Given:**

- 1) 26' B/B Street
- 2) $S_L = 4.0\%$
- 3) $S_T = 2.0\%$
- 4) $n = 0.016$
- 5) $Q = 2.5$ cfs

- b. **Find:**

- 1) d = depth of flow
- 2) T = spread of water from face of curb

- c. **Steps:** (Use Figure 1)

- 1) Calculate the value of "Z" which is the reciprocal of the transverse slope (S_T). $Z = 1/S_T = 1/0.020 = 50$.
- 2) Calculate the ratio $Z/n = 50/0.016 = 3125$.
- 3) Connect the Z/n ratio (3125) and the channel slope ($S_L = 4.0\%$) with a straight line. This will give a point of intersection on the turning line.
- 4) Connect this point and the discharge (2.5 cfs) with a straight line and read the depth at the face of curb ($d = 0.158$) in feet.
- 5) Calculate the value of the spread using the equation:
 $T = Zd$
 $T = 50 (0.158) = 7.9$ feet

2. **Capacity of grate with curb-opening type intake on a continuous grade.** The allowable capacity of an intake on a continuous grade will be determined by the following equation:

$$Q_I = K \left(d^{5/3} \right) (R_F)$$

Figure 3 is used to determine "K" for a vane grate and includes the curb hood. Figure 4 gives "K" for a driveway condition where no curb hood can be used. The appropriate reduction factor from Table 3 must then be applied to obtain the actual flow intercepted by the intake.

- a. **Given:** Street conditions as per “capacity of gutter for straight crown” example.

Design discharge	Q	= 145 cfs	[9]
Invert of pipe		= 94.50'	[2]
Starting water surface	W.S.	= 100'	[4]

Note: Number in brackets refers to the columns of Worksheet 2.

The pipe diameter needs to meet: 1) Low-flow cleaning velocity, 2) Slope for full flow, and 3) Surcharges in manhole or intake structures.

- b. **Find:** Flow intercepted by SW-501 Intake (Q_i)

- c. **Steps:**

- 1) Enter Figure 3 with the transverse slope ($S_T = 2.0\%$)
- 2) Extend horizontally to the appropriate curve for the longitudinal slope ($S_L = 4.0\%$); extend vertically downward and read the value of "K" equal to 23.9. The reduction factor for standard intake on continuous grade is 90% (from Table 3).

$$Q_i = K \left(d^{5/3} \right) (R_F)$$

$$Q_i = 23.9 \left(0.158^{5/3} \right) (0.90) = 0.99 \text{ cfs}$$

3. **Capacity of curb opening intakes on a continuous grade.** Figure 2 is used to determine the interception ratio of the intake. This theoretical interception ratio (Q_i/Q) multiplied by the design flow in the gutter and the reduction factor equals the flow intercepted by the intake.

- a. **Given:** Street conditions as per “capacity of gutter for straight crown” example

- b. **Find:** Flow intercepted by SW-507 or SW-509 intake (Q_i)

- c. **Steps:**

- 1) Enter Figure 2 at the top left hand edge with the depth of flow d_w ($d_w = S_T(T-2)$) (0.118 feet).
- 2) Follow vertically down to the line representing Manning’s n (0.016). Move horizontally across until intersecting the line representing the longitudinal slope, S_L (0.04). Follow vertically down to the flow spread, T (7.9 feet). Construct a horizontal line from this point.
- 3) Next, enter the chart from the bottom with the length of the inlet opening (4 feet for a standard SW-507 and 509 intake). Extend a line vertically from this point to the horizontal line constructed at the intersection with the flow spread (T). Move diagonally to the line formed at $Q_i/Q=0.1$. Extend a line vertically to S_T (0.02), or line A, and then horizontally to Q_i/Q (0.29).

$$Q_i = \left(\frac{Q_i}{Q} \right) (Q) (R_F)$$

$$Q_i = (0.29)(2.5)(0.9) = 0.653 \text{ cfs}$$

4. **Capacity of all intakes at a low point.** Figure 5 is used to determine the capacity Q of all intakes at a low point. The appropriate reduction factor from Table 3 must be applied to the results.

a. **Given:**

- 1) 26 feet B/B Street – Residential
- 2) $S = 2\%$ from East
- 3) $S = 1\%$ from West
- 4) $Q = 4.5$ cfs from East in each gutter
- 5) $Q = 6$ cfs from West in each gutter

b. **Find:** Q_I for Type SW-501 intake with vane grate

c. **Steps:**

- 1) Enter Figure 5 with d -max for 26 feet B/B street (6 inches). Extend vertically downward from the grate with curb opening curve and read the value of 11.4 cfs (or use equations provided).
 $Q_I = 11.4 (0.80) = 9.12$ cfs = Q_I maximum allowable.
- 2) Since Q_I is less than $Q_{total} = 4.5+6= 10.5$ cfs, additional intakes must be constructed to intercept flow so that flooding beyond the allowable limit does not occur.

5. **Capacity of SW-501 intakes at a low point**

a. **Given:** $Q_{total} = 9$ cfs

b. **Find:** d , if one SW-501 intake is built.

c. **Steps:**

- 1) Calculate the actual flow in the gutter (if the intake is partially clogged) so that the intake will intercept 10 cfs.

$$Q = \frac{Q_I}{R_F} = \frac{9.0}{0.80} = 11.25 \text{ cfs}$$

- 2) Use equations shown in Figure 5 (or use curves)

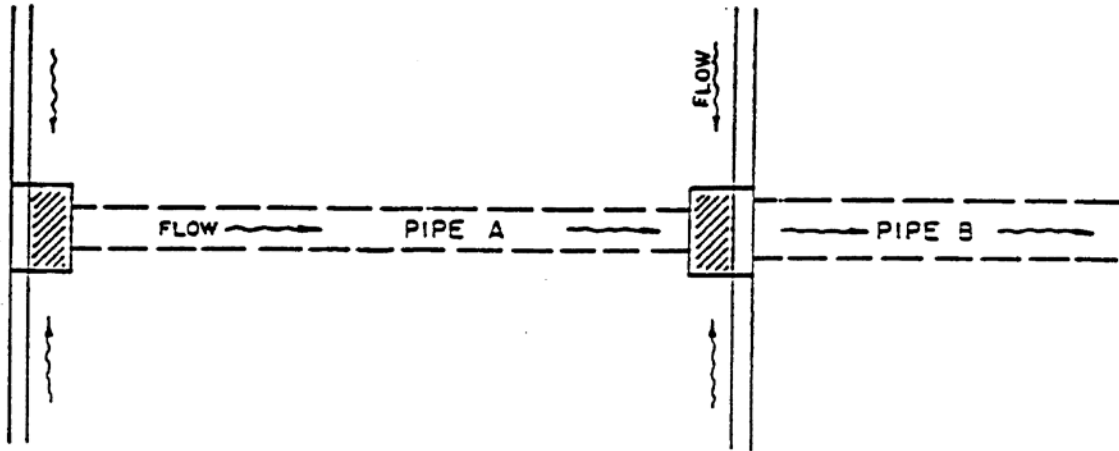
$$11.25 = 8.44 \left(H^{1/2} \right) + 8.25 \left(H^{3/2} \right) \text{ where } H = d + a$$

$$H = .66 \text{ ft.}$$

$$d = H - a = .66 - .17 = 0.49 \text{ ft.}$$

Figure 7: Pipe Standards at a Low Point for SW-501 Intake

(Unless otherwise approved by the Jurisdictional Engineer, the following pipe sizes and grades will be the minimum allowable at low points. (Based on $n = 0.013$)



Minimum for Single Intakes

Pipe A	Pipe B
Q = 7.6 CFS	Q = 15.2 CFS
18" Pipe @ 1.0% (Minimum)	24" Pipe @ 0.46%
15" Pipe @ 1.5%	18" Pipe @ 2.23%
	15" Pipe @ 6.0%

Minimum Desirable for Single Intakes

Pipe A	Pipe B
Q = 11.8 CFS	Q = 23.6 CFS
18" Pipe @ 1.4%	24" @ 1.1%
15" Pipe @ 3.6%	18" @ 5.4%

Minimum for Double Intakes

Pipe A	Pipe B
Q = 15.2 CFS	Q = 30.4 CFS
24" Pipe @ 0.46%	30" Pipe @ 0.55%
18" Pipe @ 2.3%	24" Pipe @ 1.8%
15" Pipe @ 6.0%	18" Pipe @ 9.0%

2M-4 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.

B. Flow formulas

1. Continuity Equation

$$Q = VA$$

Equation 1

where

Q = pipe discharge in cubic feet per second

V = pipe velocity in feet per second

A = pipe cross sectional area in square feet

2. Bernoulli Equation (conservation of energy)

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.

In open conduits, the flow is primarily controlled by the gravitational action on the moving fluid, which overcomes the hydraulic energy losses. The hydraulic grade line (HGL) in open conduit flow is equal to the water surface. For the case of pressure flow, the HGL is the piezometric surface, i.e. the height to which water will rise in a piezometer. It is often referred to as the piezometric head line (PHL). The energy grade line (EGL) is the line showing the total energy of the flow above some arbitrary horizontal datum. The slope of the EGL is called the energy slope or the friction slope and is designated S_f . The vertical difference between the HGL and the EGL is the velocity head.

For open (non-pressure) conduit flow:

$$\frac{V_1^2}{2g} + Y_1 + Z_1 = \frac{V_2^2}{2g} + Y_2 + Z_2 + h_f$$

Equation 2

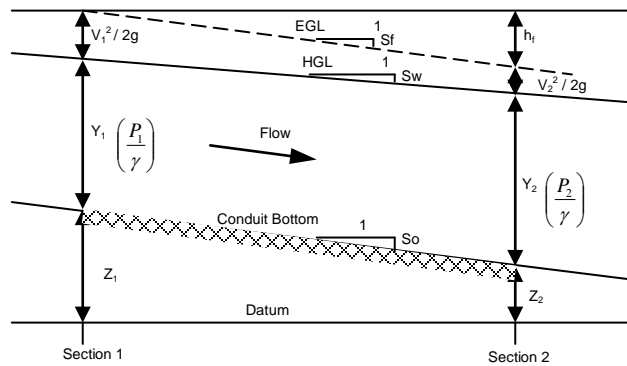
For pressure conduit flow, the Bernoulli Equation is:

$$\frac{V_1^2}{2g} + \frac{P_1}{\gamma} + Z_1 = \frac{V_2^2}{2g} + \frac{P_2}{\gamma} + Z_2 + h_f$$

Equation 3

where the total energy at Section 1 is equal to the energy at Section 2 plus the intervening head loss.

Figure 1: Terms Used in the Energy Equation



where

EGL = Energy Grade Line

HGL = Hydraulic Grade Line

Y = Water Depth

$\frac{V^2}{2g}$ = Energy Head

V = Mean Velocity

Sf = Slope of EGL

Sw = Slope of HGL

g = acceleration of gravity (32.2 feet per second)

$\frac{P}{\gamma}$ = Pressure Head

P = pressure at given location (lb/ft²)

γ = specific weight of water (62.4 lb/ft³)

Z = elevation relative to some datum

So = Slope of Bottom

hf = Head Loss

3. Manning Equation

The Manning Equation is widely used in open channel flow, but may also be applied to closed conduit and pressure flows.

For a given channel geometry, slope and roughness, and a specified value of discharge Q, a unique value of depth (normal depth) occurs in a steady uniform flow.

The Manning Equation as written in USCS units is

$$V = \frac{Q}{A} = \frac{1.486}{n} r^{2/3} s^{1/2} \tag{Equation 4}$$

where

V = Average of Mean Velocity (feet per second)

Q = Discharge, cubic feet per second

A = Cross-sectional flow area (ft²)

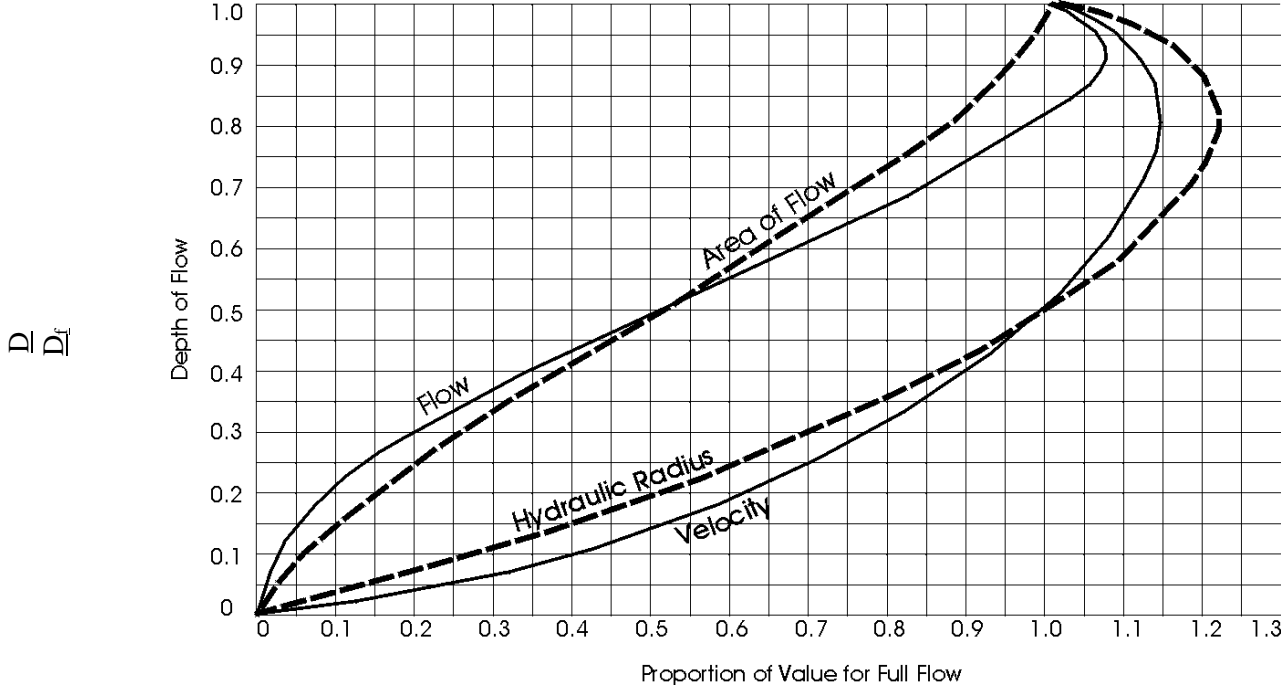
n = Roughness coefficient

$r = A/P = \text{Hydraulic radius (feet)}$ – Note: for pipes full or near full, $r = \frac{\text{Depth}}{4}$

p = Wetted perimeter (feet)

s = Slope of the hydraulic grade line (ft/ft)

Figure 2: Hydraulic Properties Circular Pipe for Partial Flow Depth

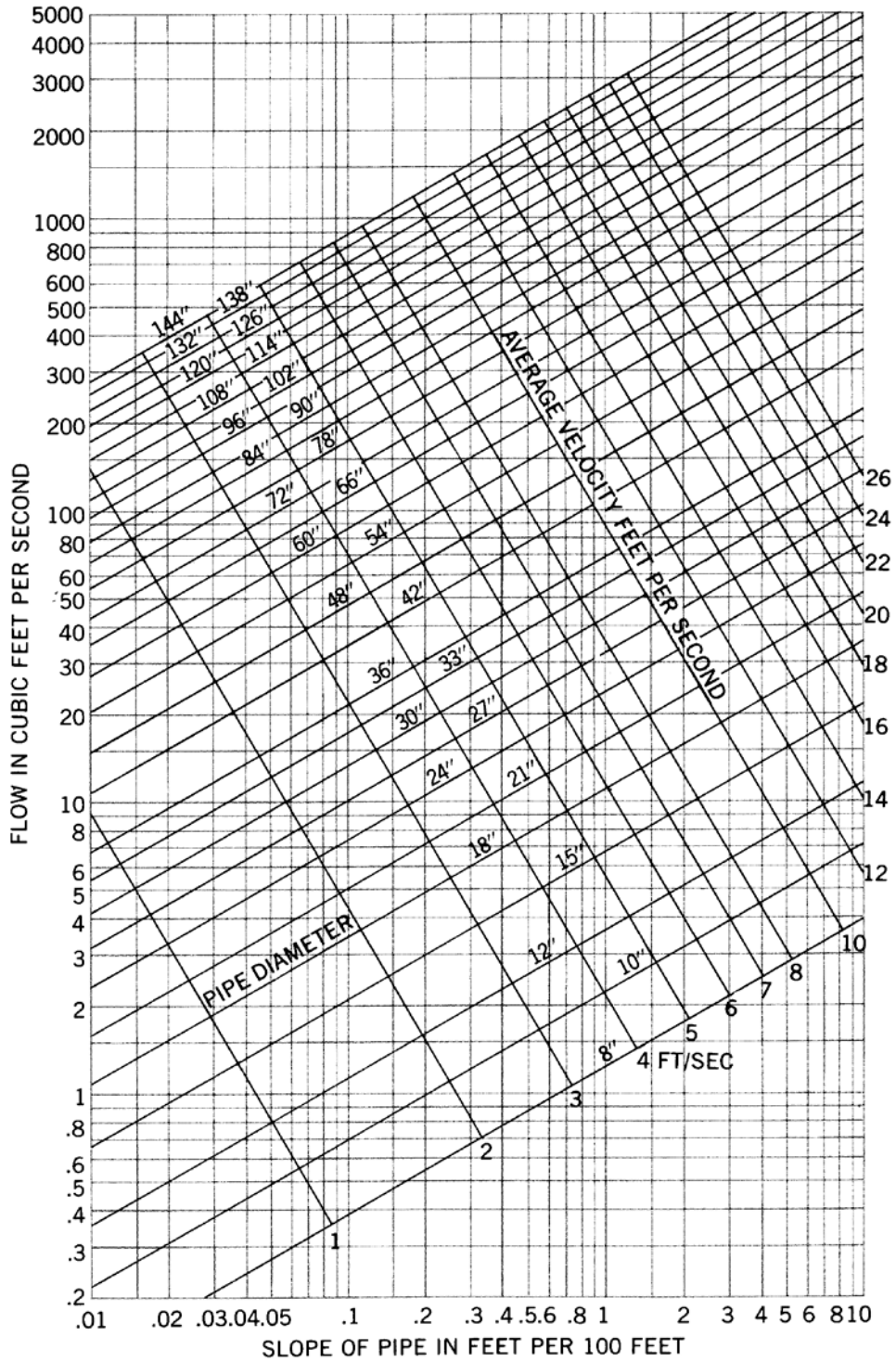


$$\frac{Q}{Q_f}, \frac{A}{A_f}, \frac{R}{R_f}, \frac{V}{V_m}$$

where

D = Diameter of Pipe
 D_f = Depth of Flow

Figure 3: Discharge of Circular Pipe Flowing Full (Based on Manning's Equation $n = 0.013$)



C. Hydraulic losses

Storm sewers should be designed to convey the minor storm flood 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 Manning's "n" values to be used in the calculation of storm sewer capacity and velocity are shown as follows:

Type of Pipe	Manning's "n"
Vitrified clay pipe	0.013
Plastic pipe (smooth wall)	0.010
Concrete pipe	0.013
Corrugated plastic pipe	0.020
CMP (2-2/3" x 1/2" corrugations)	0.024
(spun asphalt lined)	0.015
CMP (3" x 1" corrugations)	0.027
Structural Plate	0.032

2. **Pipe form losses.** Generally, between the inlet and outlet the flow encounters a variety of configurations in the flow passageway such as changes in pipe size, branches, bends, junctions, expansions, and contractions. These shape variations impose losses in addition to those resulting from pipe friction. Form losses are the result of fully developed turbulence and can be expressed as follows:

$$H_L = K \frac{V^2}{2g} \quad \text{Equation 5}$$

where

H_L = head loss (feet)

K = loss coefficient

$K \frac{V^2}{2g}$ = velocity head (feet)

g = gravitational acceleration (32.2 ft/sec)

The following is a discussion of a few of the common types of form losses encountered in sewer system design.

- a. **Pipe friction losses.** The friction slope is the energy slope in feet per foot for that run. The friction loss is simply the energy gradient multiplied by the length of the run. Energy losses from pipe friction may be determined by rewriting the Manning's equation with terms as previously defined:

$$S_{fo} = 0.453 \frac{Q^2 n^2}{A^2 R^{4/3}} \quad \text{Equation 6}$$

Then the head losses due to friction may be determined by the formula:

$$H_L = S_{fo} L \quad \text{Equation 7}$$

where

H_f = Fiction head loss (feet)

S_{fo} = Friction slope (feet/feet)

L = Length of outflow pipe (feet)

- b. **Expansion losses.** Expansion in a storm sewer conduit will result in a shearing action between the incoming high velocity jet and the surrounding sewer boundary. As a result, much of the kinetic energy is dissipated by eddy currents and turbulence. The loss of head can be expressed as:

$$H_L = K_e \frac{V_1^2}{2g} \left[1 - \left(\frac{A_1}{A_2} \right) \right]^2 \quad \text{Equation 8}$$

in which A is the cross section area, V is the average flow velocity, and K is the loss coefficient. Subscripts 1 and 2 denote the upstream and downstream sections, respectively. The value of K_e is about 1.0 for a sudden expansion, and about 0.2 for a well-designed expansion transition. Table 1 presents the expansion loss coefficients for various flow conditions.

- c. **Contraction losses.** The form loss due to contraction is:

$$H_L = K_c \frac{V_1^2}{2g} \left[1 - \left(\frac{A_2}{A_1} \right) \right]^2 \quad \text{Equation 9}$$

where K_c is the contraction coefficient. K_c is equal to 0.5 for a sudden contraction and about 0.1 for a well designed transition. Subscripts 1 and 2 denote the upstream and downstream sections, respectively. Table 1 presents the contraction loss coefficient for various flow conditions.

- d. **Bend losses.** The head losses for bends, in excess of that caused by an equivalent length of straight pipe, may be expressed by the relation:

$$H_L = K_b \frac{V^2}{2g} \quad \text{Equation 10}$$

in which K_b is the bend coefficient. The bend coefficient has been found to be a function of:

- 1) the ratio of the radius of curvature of the bend to the width of the conduit
- 2) deflection angle of the conduit
- 3) geometry of the cross section of flow
- 4) the Reynolds number and relative roughness.

Table 2 shows the recommended bend loss coefficients.

- e. **Junction and manhole losses.** A junction occurs where one or more branch sewers enter a main sewer, usually at manholes or intakes. The hydraulic design of a junction is in effect the design of two or more transitions, one for each flow path. Allowances should be made for head loss due to the impact at junctions. See Table 3 for K_j values. The head loss for a straight through manhole or at an inlet entering the sewer is calculated from Equation 1. The head loss at a junction can be calculated from:

$$H_L = \frac{V_2^2}{2g} - K_j \frac{V_1^2}{2g} \quad \text{Equation 11}$$

where V_2 is the outfall flow velocity and V_1 is the inlet velocity.

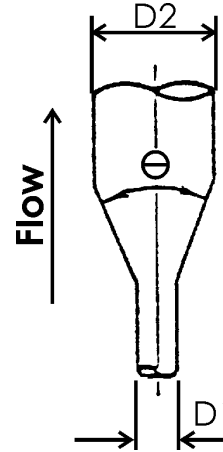
3. **Junction drop.** When there is an increase in sewer size of a smaller sewer connected with a larger one, the invert of the smaller sewer must be raised to maintain the same energy gradient. An approximate method of doing this is to place the 0.8 depth point of both sewers at the same elevation. When there is a change in alignment between storm sewer of 45° or greater the suggested minimum manhole drop is 0.10 foot.
4. **Storm sewer outlets.** When the storm sewer system discharges into the Major Drainageway System (usually an open channel), additional losses occur at the outlet in the form of expansion losses. For a headwall and no wingwalls, the loss coefficient $K_c = 1.0$ (refer to Table 6), and for a flared-end section the loss coefficient is approximately 0.5 or less.

Table 1: Storm Sewer Energy Loss Coefficient (Expansion, Contraction)

(a) Expansion (K_e)

θ^*	K_e	
	$\frac{D_2}{D_1} = 3$	$\frac{D_2}{D_1} = 1.5$
10	0.17	0.17
20	0.40	0.40
45	0.86	1.06
60	1.02	1.21
90	1.06	1.14
120	1.04	1.07
180	1.00	1.00

* The angle θ is the angle in degrees between the sides of the tapering section.



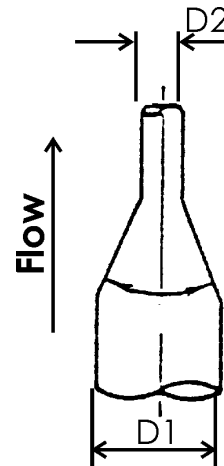
Expansion

(b) K_e for Pipe Entrance from Reservoir

Pipe Entrance Shape	K_e
Bell-mouth	0.1
Square-edge	0.5
Concrete Pipe Groove End	0.2

(c) Contractions (K_c)

$\frac{D_2}{D_1}$	K_c
0	0.5
0.4	0.4
0.6	0.3
0.8	0.1
1.0	0



Contraction

Table 2: Storm Sewer Energy Loss Coefficient (Bends)

$$K_6 = \left(0.13 + 1.85 \left(\frac{r}{R} \right)^{3.5} \right) \sqrt{\frac{\theta^\circ}{180^\circ}}$$

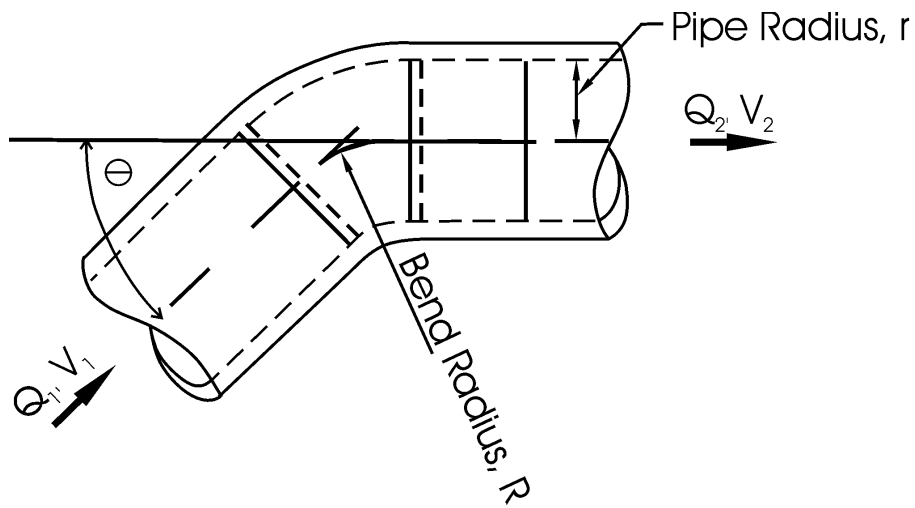
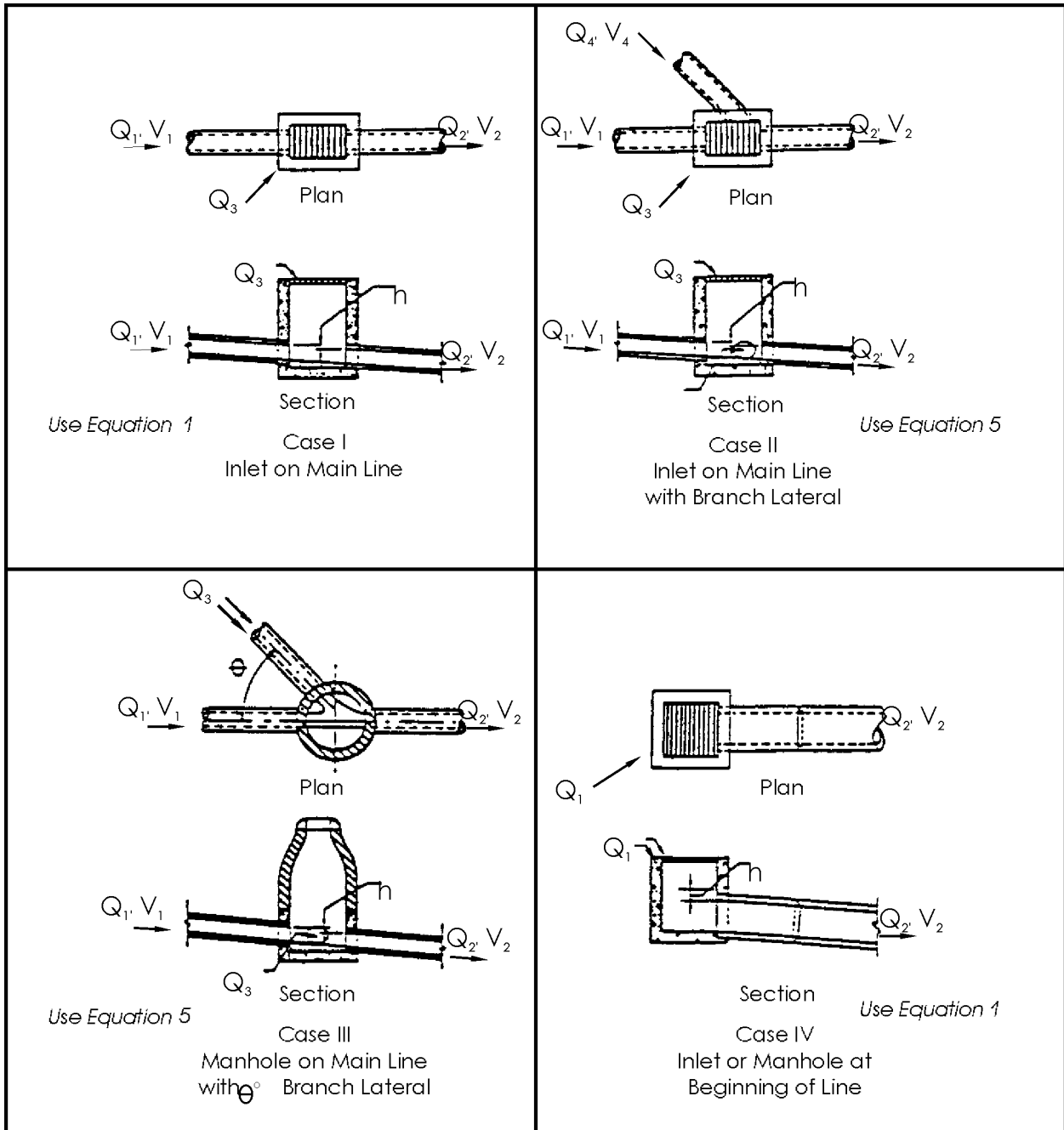


Table 3: Manhole and Junction Losses



Case	K_i	
I	0.05	
II	0.25	
III	$\theta = 22.5^\circ$	0.75
	$\theta = 45^\circ$	0.50
	$\theta = 60^\circ$	0.35
	$\theta = 90^\circ$	0.25
IV	1.25	

D. Hydraulic design of storm sewers

The following calculation example was obtained from Modern Sewer Design, AISI Wash., D.C., 1980 and edited for the calculation of manhole and junction losses according to this section.

1. Given:

- a. Plan and Profile of storm sewer (Figures 4 and 5).
- b. Station 0+00 (outfall) data as follows:

			<u>COL. #</u>
Design discharge	Q	= 145 cfs	[9]
Invert of pipe		= 94.50'	[2]
Starting water surface	W.S.	= 100'	[4]

Note: Number in brackets refers to the columns of Table 1.

The pipe diameter needs to meet: 1) Low-flow cleaning velocity, 2) Slope for full flow, and 3) Surcharges in manhole or intake structures.

2. Find: Pipe size to meet low-flow: Use 3 fps for cleaning velocity

STEP 1: Q mean annual storm = 10 cfs
 $Q_{MA}/Q_{10} = 0.07$ ratio of cleaning discharge to design discharge

STEP 2: Go to Figure 5 and using a 0.07 ratio on the horizontal axis and flow curve, the depth of flow in the pipe will equal 0.16 for cleaning velocity.

STEP 3: Using 0.16 for depth of flow on the vertical axis of Figure 5, and the ratio of
 $\frac{V_{CleaningVelocity}}{V_{FullFlow}}$ is 0.53.

STEP 4: $V_{ff} = \frac{3 \text{ fps}}{0.53} = 5.66$, or 6 fps

STEP 5: Area of Pipe = $\frac{145 \text{ cfs}}{6 \text{ fps}} = 24.2$ sq. ft.

$$R = \left(\frac{24.2}{3.14} \right)^{1/2} = 2.78 \text{ ft} = 5.55 \text{ ft}$$

Dia. = 5.5 ft or 66 in

3. Find: Slope of Pipe

STEP 1: $Q_{10} = 145$ cfs, Velocity ≈ 6 fps, and Pipe = 66 in

STEP 2: Using Manning's Equation (Figure 4) for $n = 0.13$, the slope is 0.175 ft/ft

4. **Find:** Hydraulic Grade Line and Energy Grade Line for storm sewer.
 The following procedure is based on full-flow pipe conditions. If the pipe is flowing substantially full (i.e., greater than 80%), the following procedures can be used with minimal loss of accuracy. However, the designer is responsible for checking the assumptions (i.e., check for full flow) to assure that the calculations are correct.

STEP 1: The normal depth is greater than critical depth, $d_n > d_c$; therefore, calculations to begin at outfall, working upstream. Compute the following parameters:

$$\text{Constant value [7]:} = \frac{2gn^2}{2.21} = \frac{(2)(32.2)(0.013)^2}{2.21}$$

This equation is derived from the Manning's equation by solving for velocity and converting to velocity head.

$$= 0.00492$$

This value remains constant for this design since the n-value does not change.

STEP 2: Velocity head [10]:
$$H_v = \frac{V^2}{2g} = \frac{(6.1)^2}{(2)(32.2)}$$

$$H_v = 0.58$$

STEP 3: Energy Grade Point, E.G. [11]: $E.G. = W.S. + H_v = 100 + 0.58$

$$E.G. = 100.58$$

For the initial calculation, the Energy Grade Line is computed as described above. For subsequent calculations, the equation is reversed, and the water surface is calculated as follows (see Step 12):

$$W.S. = E.G. - H_v$$

This equation is used since the losses computed in Step 8 are energy losses which are added to the downstream energy grade elevation as the new starting point from which the velocity head is subtracted as shown above.

STEP 4: Skin Friction:

$$S_f \text{ value [12]: } S_f = \phi \frac{H_v}{R^{4/3}} = \frac{(0.00492)(0.58)}{(1.375)^{4/3}}$$

NOTE: R = the hydraulic radius of the pipe.

$$\phi = \frac{2gn^2}{2.21}$$

$$S_f = 0.0019$$

STEP 5: Avg. S_f [13]: Average skin friction: This is the average value between S_f of the station being calculated and the previous station. For the first Station, Avg. $S_f = S_f$.

Beginning with Column [13], the entries are placed in the next row since they represent the calculated losses between two stations.

STEP 6: Enter sewer length, L , in column [14].

STEP 7: Friction loss H_f [15]: $H_f = (\text{Avg. } S_f)(L)$

$$H_f = \frac{0.0019}{110}$$

$$H_f = 0.21$$

STEP 8: Calculate the form losses for bends, junctions, manholes, and transition losses (expansion or contraction) using equations 1, 2, 3, 4, and 5. The calculation of these losses is presented below for the various sewer segments, since all the losses do not occur for one sewer segment.

(a) Station 1 + 10 to 1 + 52.4 (bend)

$H_L = K_b H_v$, where the degree of bend is 60° .

$K_b = 0.20$ (Table 3, Case I)

$$H_L = \frac{0.20}{0.58} = 0.12, \text{ enter in column 16.}$$

(b) Station 2 + 48 to 2 + 55.5 (transition: expansion)

$$H_L = K_e H_v \left[1 - \left(\frac{A_1}{A_2} \right) \right]^2$$

$K_e = 1.06$ (Table 1) for $D_2/D_1 = 1.5$, and $= 45^\circ$.

$$H_L = (1.06)(1.29) \left[1 - \left(\frac{15.9}{23.76} \right) \right]^2 = 0.15, \text{ enter in column 19.}$$

(c) Station 3 + 55.5 (manholes, straight through)

$$H_L = K_m H_v$$

$K_m = 0.05$ (Table 3, Case I)

$$H_L = \frac{0.05}{1.29} = 0.06, \text{ enter in column 18.}$$

(d) Station 4 + 55.5 to 4 + 65.5 (junction)

$$H_L = \frac{V_2^2}{2g} - K_j \frac{V_1^2}{2g}$$

$$K_j = 0.62 \text{ (Table 3, Case III), } \theta = 30^\circ, V_1=0.99, V_2=1.29$$

$$H_L = 1.29 - \frac{0.62}{0.99} = 0.68, \text{ enter in column 17}$$

- (e) Station 5 + 65.5 to 5 + 75.5 (junction) - since there are two laterals, the loss is estimated as twice the loss for one lateral

$$K_j = 0.33 \text{ (Table 3, Case III), } \theta = 70^\circ$$

$$H_L = 0.99 - \frac{0.33}{0.64} = 0.78 \text{ for one lateral, 1.56 for 2 laterals}$$

STEP 9: Sum all the form losses from columns [15] through [19] and enter in column [20]. For the reach between Station 0+00 to 1+10, the total loss is 0.21.

STEP 10: Add the total loss in column [20] to the energy grade at the downstream end (Sta. 0+00) to compute the energy grade at the upstream end (Sta. 1+10) for this example).

$$\text{E.G. (Upstream)} = \text{E.G. (Downstream)} + \text{TOTAL LOSS}$$

$$= 100.58 + 0.21$$

$$= 100.79 \text{ (Column 11)}$$

STEP 11: Enter the new invert [2], pipe diameter D[3], pipe shape [5], pipe area A, [6], the computed constant from Step 1 in column [7], the computed velocity V in column [8], the new Q [9], and the computed velocity head H_v [10].

STEP 12: Compute the new water surface, W.S., for the upstream Station (1+10 for this example).

$$\text{W.S.} = \text{E.G.} - H_v$$

$$= 100.79 - 0.58$$

$$= 100.21 \text{ (column 4)}$$

STEP 13: Repeat Steps 1 through 12 until the design is complete. The hydraulic grade line and the energy grade line are plotted on the profile (Figure 2).

5. **Discussion of results:** The Hydraulic Grade Line (HGL) is at the crown of the pipe from Station 0+00 to 2+48. Upstream of the transition (Station 2+55.5) the 54-inch RCP has a greater capacity (approximately 175 cfs) at the slope than the design flow (145 cfs). The pipe is therefore not flowing full but is substantially full (i.e., $145/175 = 0.84$ greater than 0.80). The computed HGL is below the crown of the pipe. However, at the outlet, the actual HGL is higher, since the outlet of the 54-inch RCP is submerged by the headwater for the 66-inch RCP. To compute the actual profile, a backwater calculation would be required; however, this accuracy is not necessary for storm sewer design in most cases.

At the junction (Station 4+55.5), the HGL is above the top of the pipe due to the losses in the junction. In this case, however, the full flow capacity (100 cfs) is the same as the design capacity, and the HGL remains above and parallel to the top of the pipe. A similar situation occurs at the junction at Station 5+65.5.

If the pipe entering a manhole or junction is at an elevation significantly above the manhole invert, a discontinuity in the Energy Grade Line (EGL) may occur. If the EGL of the incoming pipe for the design flow condition is higher than the EGL in the manhole, then a discontinuity exists, and the higher EGL is used for the incoming pipe.

Figure 4: Design Example for Storm Sewers - Plan

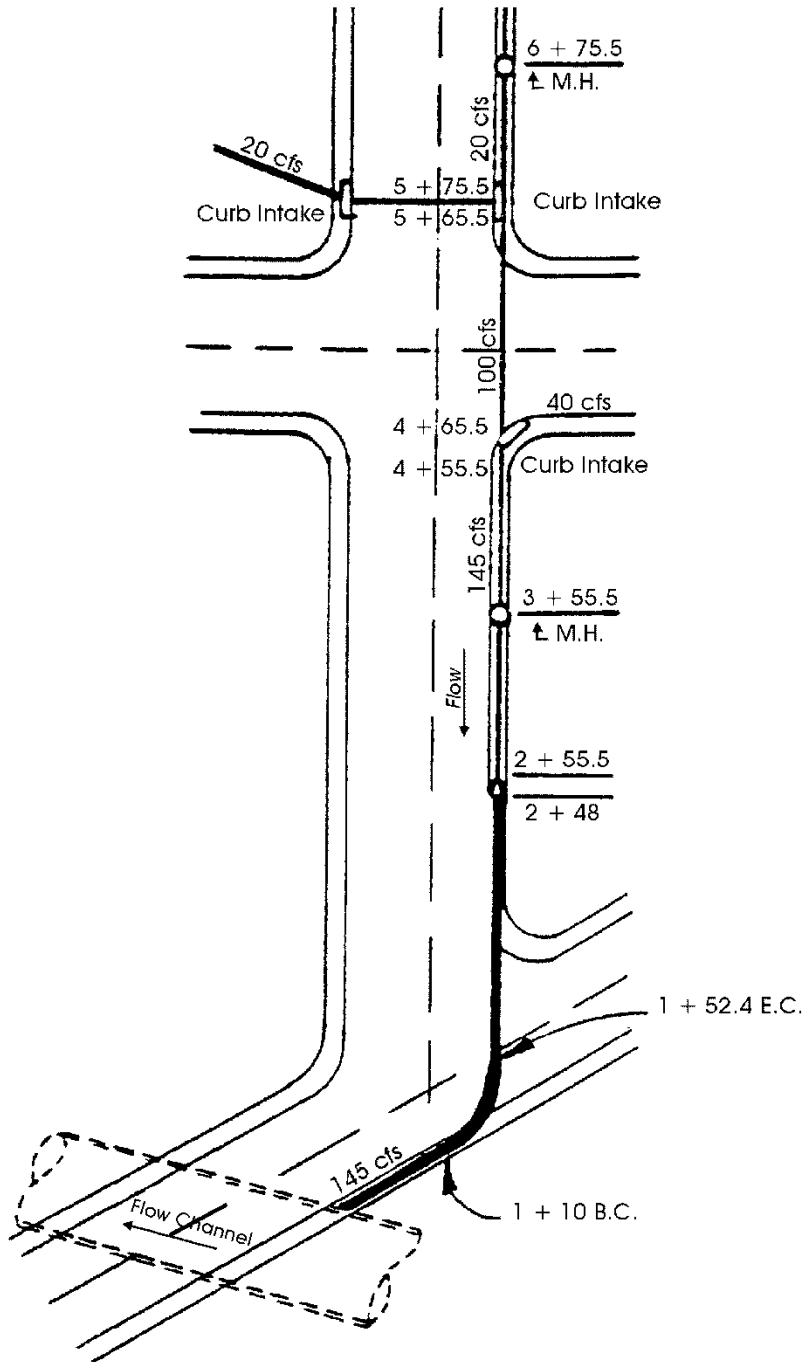


Figure 5: Design Example for Storm Sewers - Profile

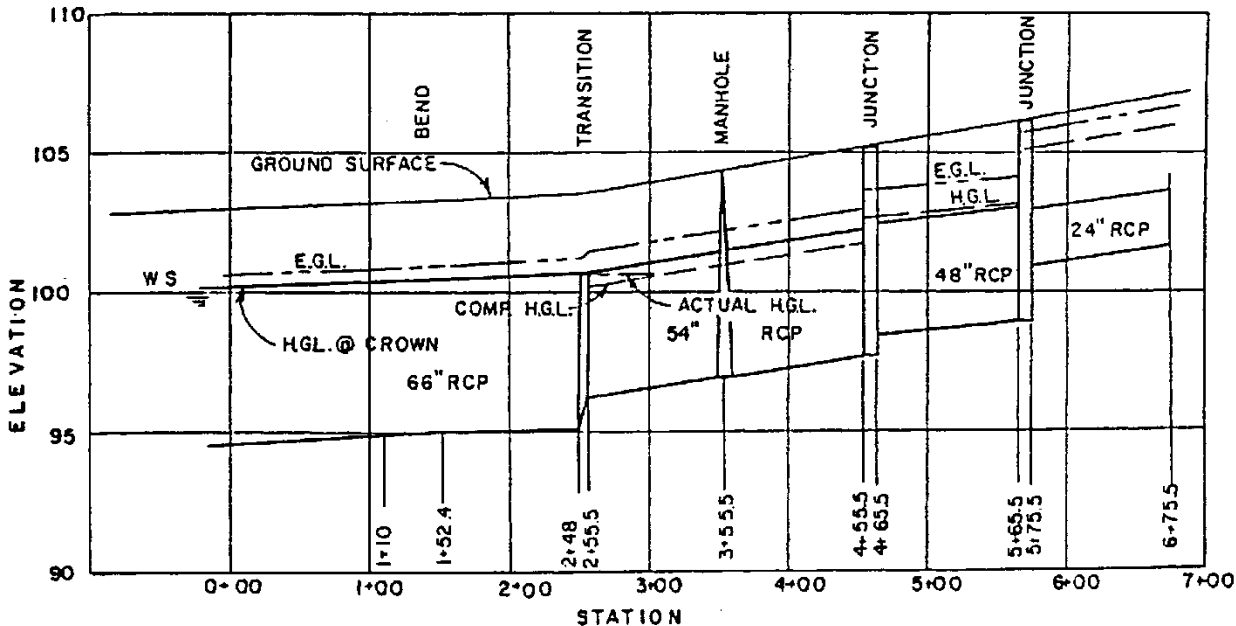


Table 4: Design Example for Storm Sewers

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
STA	INVERT	D	W.S.	PIPE SHAPE	A	φ	V	Q	Hv	E.G.	Sf	AVG. Sf	L	Hf	Hb	Hj	Hm	Ht	TOTAL LOSS
0+00	94.50	66	100.00	RND	23.76	0.00492	6.1	145	0.58	100.58	0.0019	0.0019	110	0.21	--	--	--	--	0.21
1+00	94.71	66	100.21	RND	23.76	0.00492	6.1	145	0.58	100.79	0.0019	0.0019	42.4	0.08	0.12	--	--	--	0.2
1+52.4	94.91	66	100.41	RND	23.76	0.00492	6.1	145	0.58	100.99	0.0019	0.0019	95.6	0.18	--	--	--	--	0.18
2+48	95.08	66	100.59	RND	23.76	0.00492	6.1	145	0.58	101.17	0.0019	0.0048	7.5	0.04	--	--	--	0.15	0.19
2+55.5	96.08	54	100.07	RND	15.90	0.00492	9.1	145	1.29	101.36	0.0076	0.0076	100	0.76	--	--	0.06	--	0.82
3+55.5	96.90	54	100.89	RND	15.90	0.00492	9.1	145	1.29	102.18	0.0076	0.0076	100	0.76	--	--	--	--	0.76
4+55.5	97.66	54	101.65	RND	15.90	0.00492	9.1	145	1.29	102.94	0.0076	0.0063	10	0.06	--	0.68	--	--	0.74
4+65.5	98.40	48	102.69	RND	12.57	0.00492	8.0	100	0.99	103.68	0.0049	0.0049	100	0.49	--	--	--	--	0.49
5+65.5	98.89	48	103.18	RND	12.57	0.00492	8.0	100	0.99	104.17	0.0049	0.0064	10	0.06	--	1.56	--	--	1.62
5+75.5	100.89	24	105.15	RND	3.14	0.00492	6.4	20	0.64	105.79	0.0079	0.0079	100	0.79	--	--	0.03	--	0.82
6+75.5	101.61	24	105.79	RND	3.14	0.00492	6.4	20	0.64	106.61	0.0079								

$$Sf = \frac{\phi Hv}{R^{1.33}}$$

TOTAL FRICTION LOSS = 3.43

$$\phi = \frac{2g(n^2)}{2.21}$$

TOTAL FORM LOSS = 2.75