CHAPTER 6
STORM SEWER HYDRAULICS

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Synopsis

The general approach for storm sewer system design usually involves iterative sequences of system layout, hydrologic and hydraulic calculations, and outfall design. Basic criteria and procedures are presented for the design of storm sewer systems. Conditions requiring variance from these guidelines should be documented and approved by MWS.

6.1 Design Criteria

6.1.1 Return Periods

Closed conduits shall be designed for the total intercepted flow based on the design event (see Volume 1, Section 6.3.1). In general, design event return periods are as follows:

Minor Facilities	10-year
Major Facilities	100-year

Minor and major drainage facilities are defined in Volume 1.

6.1.2 Manning's n Values

Values for Manning's roughness coefficient for concrete pipe, concrete box culvert, and corrugated metal pipe (CMP) are given below:

Concrete pipes and box culverts (precast or cast-in-place)	n = 0.013
CMP (non-spiral flow, annular corrugations)	n = 0.024
CMP (full pipe spiral flow, helical corrugations)	
Sizes 15-24"	n = 0.017
Sizes 30-54"	n = 0.021
Sizes 60-96"+	n = 0.024

Additional details for selecting roughness coefficients for CMP can be obtained from FHWA-TS-80-216 (USDOT, FHWA, 1980).

Full pipe spiral flow occurs only for circular pipes longer than 20 diameters and free of sediment buildup when lining is not used. If the conditions for development of full pipe spiral flow are questionable, the conservative use of the n value for non-spiral flow is more desirable. Conditions where full spiral flow may be appropriate are down drains, detention outlet pipes, and free outlet or gravity storm sewer systems with a design velocity above 4 feet per second.

6.1.3 Slopes and Hydraulic Gradient

The standard recommended maximum and minimum slopes for storm sewers should conform to the following criteria:

1. The maximum hydraulic gradient should not produce a velocity that exceeds 20 feet per second.

2. The minimum desirable physical slope should be that which will produce a velocity of 2.5 feet per second when the storm sewer is flowing full.

Systems should generally be designed for non-pressure conditions. When hydraulic calculations do not consider minor energy losses such as expansion, contraction, bend, junction, and manhole losses (see Section 6.4.2), the elevation of the hydraulic gradient for design flood conditions should be at least 1.0 foot below ground elevation. As a general rule, minor losses should be considered when the velocity exceeds 6 feet per second (lower if flooding could cause critical problems). If all minor energy losses are accounted for, it is usually acceptable for the hydraulic gradient to reach the gutter elevation. The maximum hydraulic gradient allowed is 5 feet above the crown of the conduit (see Volume 1, Section 6.3.2).

6.1.4 Pipe Size and Length

A minimum pipe size of 15 inches is required when access spacing is 50 feet or less. When access spacing exceeds 50 feet, a minimum size of 18 inches is required. Designs should use standard pipe size increments of 6 inches for pipes larger than 18 inches.

A minimum box culvert size of 3 by 3 feet for precast units and 4 by 4 feet for cast-in-place units is recommended. Increments of 1 foot in the height or width should be used above this minimum. The span by height format is used for reporting box culvert dimensions, e.g., in the dimension 10 by 7, the span is 10 feet and the height is 7 feet.

Access spacing shall not exceed 400 feet for conduits less than 54 inches in diameter and shall not exceed 800 feet without approval from MWS. The two materials for pipes allowed within Right of Ways (or pipes that carry public water) are concrete and corrugated metal.

6.1.5 Minimum Clearances

Minimum clearances for storm sewer pipe shall comply with the following criteria:

1. A minimum of 1 foot is required between the bottom of the road base material and the outside crown of the storm sewer.

2. For utility conflicts that involve crossing a storm sewer alignment, the recommended minimum design clearance between the outside of the pipe and the outside of any conflicting utility should be 0.5 foot if the utility has been accurately located at the point of conflict. If the utility has been approximately located, the minimum design clearance should be 1 foot. Electrical transmission lines or gas mains should never come into direct contact with the storm sewer.

3. Storm sewer systems should not be placed parallel to or below existing utilities in a manner that could cause utility support problems. The recommended clearance is 2 feet extending from each side of the storm sewer and 1:1 side slopes from the trench bottom.

4. When a sanitary line or other utility must pass through a manhole, a minimum 1-foot clearance should be maintained between the bottom of the utility and the flow line of the storm main, and greater clearance is recommended. Flow will be less obstructed when the utility is placed above or as close as possible to the crown of the pipe. The head loss caused by an obstruction should be accounted for. (Note: Gas mains shall not pass through inlet and manhole structures.)

6.1.6 Inlet Location and Spacing

The location and spacing of inlets should be based on inlet capacity and width of spread calculations consistent with procedures and criteria presented in Chapter 4.

6.1.7 Easements

Easement requirements are given in Volume 1, Section 6.3.3.

6.2 General Approach

The design of storm sewer systems is usually an iterative process involving the following four steps:

1. System Layout: Selection of inlet locations and development of a preliminary plan and profile configurations consistent with design criteria in Section 6.1.

2. Hydrologic Calculations: Determination of design flow rates and volumes (see Section 6.3).

3. Hydraulic Calculations: Determination of pipe sizes required to carry design flow rates and volumes, as discussed in Section 6.4.

4. Outfall Design: Outlet protection or detention/retention may be required because of downstream constraints; see Chapter 8 for detention/retention , Chapter 10 and Volume 4 TCP-25 or PESC-07 for outlet protection.

6.3 Hydrologic Calculations

The two peak flow methods generally appropriate for hydrologic calculations for storm sewer systems are the Rational Method and the inlet hydrograph method. In general, as the time of concentration, drainage area, and variability in land use increase, more complex procedures are warranted. A rule-of -thumb is that flood hydrograph procedures should be considered when the time of concentration goes beyond the range of 30 to 45 minutes. In addition, the size and complexity of the storm sewer system should be considered. (See Chapter 2 for additional guidance on selecting hydrologic methods.)

To demonstrate the application of the peak flow methods identified above and to provide a point of comparison, the example storm sewer system layout shown in Figure 6-1 is evaluated below. Common data for calculating inlet flow rates are presented in Table 6-1.

6.3.1 Rational Method

The Rational Method, expressed in Chapter 2 as Equation 2-11, implicitly assumes that all runoff from the tributary area is intercepted by the storm sewer system. Bypass must be accounted for by adjusting the tributary drainage area. The method requires a determination of the tributary area, time of concentration, rainfall intensity, and runoff coefficient at each design point.

The time of concentration is the sum of the inlet travel time and the storm sewer travel time and must be calculated for each design point considered. Rainfall intensity is obtained from an IDF curve (see Figure 2-1), based on the time of concentration and design frequency. The runoff coefficient should be the composite factor based on tributary land use and soil conditions. Table 2-3 (see Section 2.3.1) can provide a good starting point for selecting the runoff coefficient for a 10-year return period, but other considerations should include examination of existing facilities and a comparison of historical performance with the results of design calculations, if possible.

Results of Rational Method calculations for the example storm sewer data presented in Figure 6-1 and Table 6-1 are shown in Table 6-2.

6.3.2 Inlet Hydrograph Method

The inlet hydrograph method is a simplified approach that accounts for channel storage and appears to provide better estimates of observed peak runoff rates than the Rational Method (Jens and McPherson, 1964). The following equation is used to route intercepted flow for each upstream inlet to the design point:

$$Q_o = Q_i \left(\frac{2T}{2T + 0.8\frac{L}{v}} \right) \qquad (6\text{-}1)$$

where:

Q_o = Outflow peak runoff rate at the design point, in cfs

Q_i = Intercepted flow peak runoff rate, in cfs

T = inlet travel time, in minutes

L = Length of storm sewer, in feet

v = Average velocity for storm sewer flow, in feet/minute

Having calculated the peak outflow, Q_o, for intercepted flow from each inlet, Q_i, the composite peak flow at the design point is obtained by summing the ordinates of triangular hydrographs for each inlet. This summation is accomplished graphically by drawing triangular hydrographs for the outflow from each inlet with a peak of Q_o, a rising limb time of T +0.8 (L/v), and a recession time of T. This procedure is illustrated in Figure 6-2, which also illustrates the inflow hydrograph with a peak flow rate equal to the inlet intercept and a time base of 2T. By plotting triangular outflow hydrographs for each inlet tributary to the design point on the same scale, the composite hydrograph can be developed by summing hydrograph ordinates. Dividers are helpful for accomplishing this summation.

Additional information on the use of the inlet hydrograph method can be found in publications by Jens and McPherson (1964) and Kaltenbach (1963).

Results of inlet hydrograph calculations for the example storm sewer data presented in Figure 6-1 and Table 6-1 are shown in Table 6-3. Graphical development of peak flows for each storm sewer segment is shown in Figures 6-3 through 6-6.

6.3.3 Example Comparison

Peak flow calculations from the two methods for the example storm sewer system in Figure 6-1 are compared in Table 6-4. The inlet hydrograph method consistently gave the lowest peak flow results, with the results obtained by the Rational Method corresponding closely. Pipe segments between inlets and manholes are not compared in Table 6-4 because each method produced the same results (because 100 percent intercept was assumed).

The design flow for the pipe between the first two manholes, M_1-M_2, did not vary a great deal between methods. Beginning with pipe segment M_2–M_3, the inlet hydrograph method has lower results. The final pipe segment, M_4-0, had a 13 percent reduction in peak flow rate for the inlet hydrograph method, as compared to the Rational Method.

6.4 Hydraulic Calculations

Hydraulic calculations are used to size conduits to handle the design flows determined from hydrologic calculations (see Section 6.3). The hydraulic capacity of a storm sewer conduit can be calculated for the two types of conditions typically referred to as gravity and pressure flow. Hydraulic procedures provided in this section represent a summary of information from publications by Brater and King (1976), Chow (1959), the American Society of Civil Engineers (1969), the University of Missouri (1958), and the American Iron and Steel Institute (1980). These publications should be consulted if additional details are required.

6.4.1 Pressure Versus Gravity Flow

Guidance is presented in Figure 6-7 for determining whether pressure or gravity flow conditions occur in a storm sewer system. In general, if the hydraulic grade line is above the crown of the pipe, pressure flow hydraulic calculations are appropriate. Conversely, if the hydraulic grade line is below the crown of the pipe, gravity flow calculations are appropriate. Storm sewer systems should generally be designed as gravity systems (see Volume 1, Section 6.3.2).

For storm sewers designed to operate under pressure flow conditions, inlet surcharging and possible manhole lid displacement can occur if the hydraulic grade line rises above the ground surface. A design based on gravity conditions must be carefully planned as well, including evaluation of the potential for excessive and inadvertent flooding created when a storm event larger than the design storm pressurizes the system.

Existence of the desired flow condition should be verified for design conditions. Storm sewer systems can alternate between pressure and gravity flow conditions from one section to another.

The discharge point of the storm sewer system usually establishes a starting point for evaluating the condition of flow. If the discharge is submerged, as when the water level of the receiving waters are above the crown of the storm sewer, the exit loss should be added to the water level and calculations for head loss in the storm sewer system started from this point, as illustrated in Figure 6-7. If the hydraulic grade line is above the pipe crown at the next upstream manhole, pressure flow calculations are indicated; if it is below the pipe crown, then gravity flow calculations should be used at the upstream manhole.

When the discharge point is not submerged, a flow depth should be determined at a known control section to establish a starting elevation. As illustrated in Figure 6-7, the hydraulic grade line is then projected from the starting elevation to the upstream manhole. Pressure flow calculations may be used at the manhole if the hydraulic grade is above the pipe crown.

The assumption of straight hydraulic grade lines, as shown in Figure 6-7, is not entirely correct, since backwater and drawdown conditions can exist, but is generally reasonable. It is also usually appropriate to assume the hydraulic grade calculations begin at the crown of the outlet pipe for simple non-submerged systems. If additional accuracy is needed, as with very large conduits or where the result can have a significant effect on design, backwater and drawdown curves should be developed.

6.4.2 Energy Losses

The following energy losses should be considered for storm sewer systems:

1. Friction
2. Entrance
3. Exit

Additional energy loss parameters should be evaluated for complex or critical systems. The following losses are especially important when failure to handle the design flood has the potential to flood offsite areas:

1. Expansion
2. Contraction
3. Bend
4. Junction and manhole

<u>Friction Loss</u>

The energy loss required to overcome friction caused by conduit roughness is generally calculated as:

$$H_f = \left(\frac{29n^2 L}{R^{1.33}}\right)\frac{v^2}{2g} \qquad (6\text{-}2)$$

where:

H_f = Energy loss due to friction, in feet

n = Manning's roughness coefficient

L = Conduit length, in feet

R = Hydraulic radius of conduit, in feet

v = Average velocity, in feet/second

g = Acceleration due to gravity, 32.2 feet/second2

Entrance, Exit, Expansion, Contraction, and Bend Losses

These head losses due to pipe form conditions are generally calculated as:

$$H_L = K \frac{v^2}{2g} \qquad (6\text{-}3)$$

where:

H_L = Head loss due to pipe form conditions, in feet

K = Loss coefficient for pipe form conditions

v = Average velocity, in feet/second

g = Acceleration due to gravity, 32.2 feet/second2

The loss coefficient, K, is different for each category of pipe form loss and should be based on operating characteristics of the specific system. Values for the entrance loss coefficient are the same as those developed for culverts (see Chapter 5). Expansion and contraction loss coefficients for circular pipes can be selected based on data from Brater and King (1976) presented in Tables 6-5 and 6-6.

The bend loss coefficient for storm sewer systems can be evaluated using Figure 6-8, which provides various relationships between the angle of a bend and the loss coefficient. Relationships are presented for bends at manholes with and without deflectors, and for curved drain alignments with r/D values equal to 2 and greater than or equal to 6.

Junction and Manhole Losses

Losses associated with junctions and manholes should be evaluated with the procedures reported by the University of Missouri (1958). Although details of the procedures are not given in this manual, the application of important results is discussed below and head loss coefficients for typical manholes and junctions are presented in Table 6-7.

For straight flow-through conditions, the University of Missouri (1958) indicates that pipes should be positioned vertically between the limits of inverts aligned or crowns aligned. An offset in the plan is allowable, provided that the projected area of the smaller pipe falls within that of the larger. It is probably most effective to align the pipe inverts, as the manhole bottom will then support the bottom of the jet issuing from the upstream pipe.

When two laterals intersect at a manhole, pipes should not be oppositely aligned, since the jets could impinge upon each other. If directly opposing laterals are necessary, the installation of a deflector (as shown in Figure 6-9) will significantly reduce losses. The research conducted on this type of deflector is limited to the ratios of outlet pipe to lateral pipe diameters equal to 1.25. In addition, lateral pipes should be located such that their centerlines are separated laterally by at least the sum of the two lateral pipe diameters.

Jets from upstream and lateral pipes must be considered when attempting to shape the inside of manholes. Results reported by the University of Missouri (1958) for pressurized pipe flow conditions indicate that very little, if anything, is gained by shaping the bottom of a manhole to conform to the pipe invert. Shaping the manhole bottom to match the pipe invert may even be detrimental when pressurized laterals flowing full are involved, as the shaping tends to deflect the jet upwards, causing unnecessary head loss. Limited shaping of the manhole bottom for open channel flow conditions is required.

Figure 6-9 depicts several types of deflectors that can be efficient in reducing losses at junctions and bends for full flow conditions. In all cases, the bottoms are flat or only slightly rounded (to handle low flows). As a contrast, several inefficient manhole shapes are shown in Figure 6-10. Several of these inefficient devices would appear to be improvements, indicating that special shapings deviating from those in Figure 6-9 should be used with caution.

6.4.3 Gravity Flow

The capacity of storm sewers designed to operate under gravity flow conditions should be sized using the following form of Manning's Equation:

$$v = \frac{0.592}{n} D^{2/3} S^{1/2} \qquad (6\text{-}4)$$

$$Q = vA \qquad (6\text{-}5)$$

$$Q = \frac{0.465}{n} D^{8/3} S^{1/2} \qquad (6\text{-}6)$$

where:

Q = Design flow rate, in cfs

v = Average velocity of flow, in feet/second

n = Manning's roughness coefficient

D = Pipe diameter, in feet

A = Cross-sectional area, in square feet

S= Slope of the energy gradient, in feet/foot

Storm sewer capacity calculations based on Manning's Equation can be accomplished using Figures 6-11, 6-12, and 6-13 as discussed below or procedures published by Brater and King (1976), the American Concrete Pipe Association (1978 and 1980), Chow (1959), and the American Iron and Steel Institute (1980).

Nomograph

The following steps are used for solving Manning's Equation using the circular pipe nomograph in Figure 6-11:

1. Determine input data, including slope in feet/foot, Manning's n value, and pipe diameter in inches or feet.

2. Connect a line from the slope scale, Point 1, to the Manning's n scale, Point 2, and note the point of intersection on the turning line, Point 3.

3. Connect a line from the pipe diameter, Point 5, to the point of intersection obtained in Step 2, Point 3.

4. Extend the line from Step 3 to the discharge and velocity scales to read the discharge at Point 4 and the velocity at Point 6.

Partial Flow Charts

For partial flow in a circular pipe. Figures 6-12 and 6-13 can be used for capacity and velocity calculations as follows:

1. Determine input data including design discharge, Q, Manning's n value, pipe diameter, D, and channel slope, S.

2. Calculate the circular pipe conveyance factor using the equation:

$$K_p = \frac{Qn}{D^{8/3} S^{1/2}} \qquad (6\text{-}7)$$

where:

K_p = Circular pipe open channel conveyance factor

Q = Discharge rate for design conditions, in cfs

n = Manning's roughness coefficient (see Section 6.1.2)

D = Pipe diameter, in ft

S = Slope of the energy grade line, in feet/foot

3. Enter the x-axis of Figure 6-12 with the value of K_p calculated in Step 2 and run a line vertically to the curve.

4. From the point of intersection obtained in Step 3, run a horizontal line to the y-axis and read a value of the normal depth of flow over the pipe diameter, d/D.

5. Multiply the d/D value from Step 4 by the pipe diameter, D, to obtain the normal depth of flow.

6. Enter the y-axis of Figure 6-13 with the d/D value from Step 4 and run a line horizontally to the curve.

7. From the point of intersection obtained in Step 6, run a line vertically downward and read a value of k_v, which equals $vn/D^{2/3}\ S^{1/2}$, from the x-axis.

8. Calculate the average velocity by the equation:

$$v = \frac{K_v D^{2/3} S^{1/2}}{n} \qquad (6\text{-}8)$$

where:

v = Average velocity, in feet/second

k_v = Pipe velocity factor from Figure 6-13 (Step 7)

D = Pipe diameter, in feet

S = Slope of the energy grade line, in feet/foot

n = Manning's roughness coefficient (see Section 6.1.2)

6.4.4 Pressure Flow

The capacity of storm sewers designed to operate under pressure flow conditions can be sized using inlet and outlet control nomographs developed for the evaluation of culverts (see Chapter 5). A more general procedure involves the application of the Energy Equation, which can be developed to consider unsteady flow conditions.

The capacity of storm sewers flowing full can be evaluated by considering velocity head, pipe form, and friction losses, expressed as:

$$H = H_v + H_L + H_f \qquad (6\text{-}9)$$

or

$$H = \left[1 + K_L + \frac{29n^2 L}{R^{1.33}}\right]\frac{v^2}{2g} \qquad (6\text{-}10)$$

where:

H = Head, determined as the difference between the hydraulic grade line at the downstream pipe and the energy grade line at the upstream pipe, in feet

H_v = Velocity head, in feet

H_L = Head loss due to pipe form conditions, in feet

H_f = Head loss due to friction, in feet

K_L = Loss coefficient for pipe form losses

n = Manning’s roughness coefficient

L = Length of storm sewer segment, in feet

R = Hydraulic radius, in feet

v = Average velocity of flow, in feet/second

g = Acceleration due to gravity, 32.2 feet/second^2

If H can be determined, the storm sewer capacity is calculated by rearranging Equation 6-8 as follows:

$$v = \left[2gH \div \left(1 + K_L + \frac{29n^2L}{R^{1.33}} \right) \right]^{1/2} \qquad (6\text{-}11)$$

or

$$Q = A\left[2gH \div \left(1 + K_L + \frac{29n^2L}{R^{1.33}} \right) \right]^{1/2} \qquad (6\text{-}12)$$

where:

v = Average velocity of flow, in feet/second

Q = Storm sewer capacity, in cfs

g = Acceleration due to gravity, 32.2 feet/second^2

H = Head, determined as the difference between the hydraulic grade line at the downstream pipe and the energy grade line at the upstream pipe, in feet

K_L = Loss coefficient for pipe form losses

n = Manning's roughness coefficient

L = Length of storm sewer segment, in feet

R = Hydraulic radius, in feet

The determination of H will generally involve an evaluation of energy losses to establish the hydraulic and energy gradients. Since the velocity is a required input to energy loss calculations, an iterative trial and error procedure is generally required.

6.5 Construction and Maintenance Considerations

An important step in the design process involves identifying whether special provisions are warranted to properly construct or maintain proposed facilities. Maintenance concerns of storm sewer system design focus on adequate physical access for cleaning and repair. Volume 4 CP-18 and 20 should be considered as a part of the design process.

Table 6-1
DATA FOR DEMONSTRATING THE APPLICATION OF STORM SEWER HYDROLOGIC METHODS

Inlet [a]	Drainage Area (acres)	Time of Concentration (minutes)	Rainfall [b] Intensity (inches/hr)	Runoff Coefficient	Inlet Flow Rate [c] (cfs)
1	2.0	8.0	6.4	.9	11.5
2	3.0	10.0	6.1	.9	16.5
3	2.5	9.0	6.2	.9	14.0
4	2.5	9.0	6.2	.9	14.0
5	2.0	8.0	6.4	.9	11.5
6	2.5	9.0	6.2	.9	14.0
7	2.0	8.0	6.4	.9	11.5

[a] Inlet and storm sewer system configuration are shown in Figure 6-1.

[b] Data for example calculations only. See Chapter 2 for Nashville IDF data.

[c] Calculated using the Rational Equation (see Chapter 2).

Table 6-2
RESULTS OF RATIONAL METHOD CALCULATIONS FOR THE
HYPOTHETICAL STORM SEWER SYSTEM IN FIGURE 6-1

Storm Sewer Segment	Tributary Area [a] (acres)	Time of Concentration [b] (minutes)	Rainfall Intensity [c] (inches/hr)	Runoff Coefficient	Design Flow Rate (cfs)
$I_1 - M_1$	2.0	8.0	6.4	.9	11.5
$I_2 - M_1$	3.0	10.0	6.1	.9	16.5
$M_1 - M_2$	5.0	10.5	6.0	.9	27.0
$I_3 - M_2$	2.5	9.0	6.2	.9	14.0
$I_4 - M_2$	2.5	9.0	6.1	.9	13.7
$M_2 - M_3$	10.0	11.5	5.7	.9	51.3
$I_5 - M_3$	2.0	8.0	6.4	.9	11.5
$I_6 - M_3$	2.5	9.0	6.2	.9	14.0
$M_3 - M_4$	14.5	13.5	5.4	.9	70.5
$I_7 - M_4$	2.0	8.0	6.4	.9	11.5
$M_4 - O$	16.5	14.7	5.2	.9	77.2

[a] Tributary area data are presented in Table 6-1.

[b] See Figure 6-1 for details.

[c] Data for example calculations only. See Chapter 2 for Nashville IDF data.

Table 6-3
RESULTS OF INLET HYDROGRAPH CALCULATIONS FOR THE HYPOTHETICAL STORM SEWER SYSTEM IN FIGURE 6-1

A. Segment $M_1 - M_2$

Inlet	2T (minutes)	L/v (minutes)	$\frac{2T}{2T+0.8\frac{L}{v}}$	Q_i (cfs)	Q_o (cfs)
1	16	0.2	0.99	11.5	11.4
2	20	0.5	0.98	16.5	16.2

Inflow to Segment $M_1 - M_2$ = 24.5 cfs (from Figure 6-3)

B. Segment $M_2 - M_3$

Inlet	2T (minutes)	L/v (minutes)	$\frac{2T}{2T+0.8\frac{L}{v}}$	Q_i (cfs)	Q_o (cfs)
1	16	1.2	0.94	11.5	10.8
2	20	1.5	0.94	16.5	15.5
3	18	0.2	0.99	14.0	13.9
4	18	0.5	0.98	14.0	13.7

Inflow to $M_2 - M_3$ = 50.0 cfs (from Figure 6-4)

C. Segment $M_3 - M_4$

Inlet	2T (minutes)	L/v (minutes)	$\frac{2T}{2T+0.8\frac{L}{v}}$	Q_i (cfs)	Q_o (cfs)
1	16	3.2	0.86	11.5	9.9
2	20	3.5	0.88	16.5	14.5
3	18	2.2	0.91	14.0	12.7
4	18	2.5	0.90	14.0	12.6
5	16	0.2	0.99	11.5	11.4
6	18	0.5	0.98	14.0	13.7

Inflow to $M_3 - M_4$ = 65.5 cfs (from Figure 6-5)

Table 6-3
(Continued)

D. Segment $M_4 - O$

Inlet	2T (minutes)	L/v (minutes)	$\frac{2T}{2T+0.8\frac{L}{v}}$	Q_i (cfs)	Q_o (cfs)
1	16	4.4	0.82	11.5	9.4
2	20	4.7	0.84	16.5	13.9
3	18	3.2	0.88	14.0	12.3
4	18	3.7	0.86	14.0	12.0
5	16	1.4	0.93	11.5	10.7
6	18	1.7	0.93	14.0	13.0
7	16	0.5	0.98	11.5	11.3

Inflow to $M_4 - 0$ = 67.0 cfs (from Figure 6-6)

Note: Q_i values are calculated in Table 6-1.

$$Q_o = Q_i\left(\frac{2T}{2T+0.8\frac{L}{v}}\right) \quad Equation\,6-1$$

Table 6-4
COMPARISON OF HYDROLOGIC METHODS FOR THE HYPOTHETICAL STORM SEWER SYSTEM IN FIGURE 6-1

Storm Sewer [a] Segment	Rational Method [b] (cfs)	Inlet Hydrograph [c] (cfs)
$M_1 - M_2$	27.0	24.5
$M_2 - M_3$	51.3	50.0
$M_3 - M_4$	70.5	65.5
$M_4 - O$	77.2	67.0

[a] Storm sewer configuration is shown in Figure 6-1.

[b] Results obtained from Table 6-2.

[c] Results obtained from Table 6-3.

Table 6-5
VALUES OF K_2 FOR DETERMINING LOSS OF HEAD DUE TO SUDDEN EXPANSION IN PIPES, FROM THE FORMULA
$H_2 = K_2 (V_1{}^2/2g)$

d_2/d_1 = Ratio of larger pipe to smaller pipe diameter

v_1 = Velocity in smaller pipe

$\frac{d_2}{d_1}$	Velocity, v_1 (feet/second)												
	2	3	4	5	6	7	8	10	12	15	20	30	40
1.2	.11	.10	.10	.10	.10	.10	.10	.09	.09	.09	.09	.09	.08
1.4	.26	.26	.25	.24	.24	.24	.24	.23	.23	.22	.22	.21	.20
1.6	.40	.39	.38	.37	.37	.36	.36	.35	.35	.34	.33	.32	.32
1.8	.51	.49	.48	.47	.47	.46	.46	.45	.44	.43	.42	.41	.40
2.0	.60	.58	.56	.55	.55	.54	.53	.52	.52	.51	.50	.48	.47
2.5	.74	.72	.70	.69	.68	.67	.66	.65	.64	.63	.62	.60	.58
3.0	.83	.80	.78	.77	.76	.75	.74	.73	.72	.70	.69	.67	.65
4.0	.92	.89	.87	.85	.84	.83	.82	.80	.79	.78	.76	.74	.72
5.0	.96	.93	.91	.89	.88	.87	.86	.84	.83	.82	.80	.77	.75
10.0	1.00	.99	.96	.95	.93	.92	.91	.89	.88	.86	.84	.82	.80
∞	1.00	1.00	.98	.96	.95	.94	.93	.91	.90	.88	.86	.83	.81

Reference: Brater and King (1976).

Table 6-6
VALUES OF K_3 FOR DETERMINING LOSS OF HEAD DUE TO SUDDEN CONTRACTION IN PIPES, FROM THE FORMULA
$H_3 = K_3 (V_2{}^2/2g)$

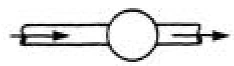
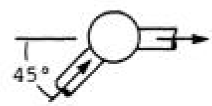
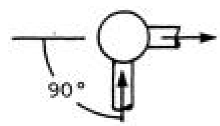
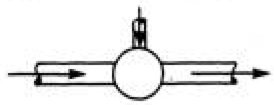
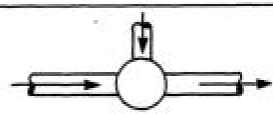
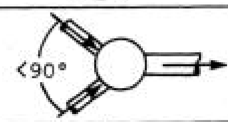
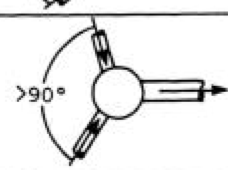
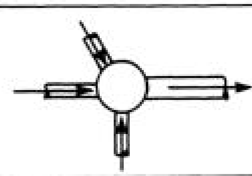
d_2/d_1 = Ratio of larger pipe to smaller pipe diameter

v_2 = Velocity in smaller pipe

$\frac{d_2}{d_1}$	Velocity, v_2 (feet/second)												
	2	3	4	5	6	7	8	10	12	15	20	30	40
1.1	.03	.04	.04	.04	.04	.04	.04	.04	.04	.04	.05	.05	.06
1.2	.07	.07	.07	.07	.07	.07	.07	.08	.08	.08	.09	.10	.11
1.4	.17	.17	.17	.17	.17	.17	.17	.18	.18	.18	.18	.19	.20
1.6	.26	.26	.26	.26	.26	.26	.26	.26	.26	.25	.25	.25	.24
1.8	.34	.34	.34	.34	.34	.34	.33	.33	.32	.32	.31	.29	.27
2.0	.38	.38	.37	.37	.37	.37	.36	.36	.35	.34	.33	.31	.29
2.2	.40	.40	.40	.39	.39	.39	.39	.38	.37	.37	.35	.33	.30
2.5	.42	.42	.42	.41	.41	.41	.40	.40	.39	.38	.37	.34	.31
3.0	.44	.44	.44	.43	.43	.43	.42	.42	.41	.40	.39	.36	.33
4.0	.47	.46	.46	.46	.45	.45	.45	.44	.43	.42	.41	.37	.34
5.0	.48	.48	.47	.47	.47	.46	.46	.45	.45	.44	.42	.38	.35
10.0	.49	.48	.48	.48	.48	.47	.47	.46	.46	.45	.43	.40	.36
∞	.49	.49	.48	.48	.48	.47	.47	.47	.46	.45	.44	.41	.38

Reference: Brater and King (1976).

Table 6-7
HEAD LOSS COEFFICIENTS FOR MANHOLES/JUNCTIONS

Single Pipe Junctions	
Type of Manhole/Junction	**Head Loss Coefficient (K)**
Trunkline only with no bend at junction 	0.5
Trunkline only with 45° bend at junction 	0.6
Trunkline only with 90° bend at junction 	0.8
Multiple Pipe Junctions	
Type of Manhole/Junction	**Head Loss Coefficient (K)**
Trunkline with one small lateral 	0.6
Trunkline with one large lateral 	0.7
Two roughly equivalent entrance lines with angle of <90° between lines 	0.8
Two roughly equivalent entrance lines with angle of >90° between lines 	0.9
Three or more entrance lines 	1.0

Reference: Golding (1987).

Note: Above values of K are to be used to estimate energy or head losses through surcharged junctions/manholes in pressure flow portions of a storm sewer system. The energy loss equation is

$$h_j(ft) = K \frac{[v(ft/sec)]^2}{64.4}$$

with v = larger velocity in main entrance or exit line of junction/manhole.

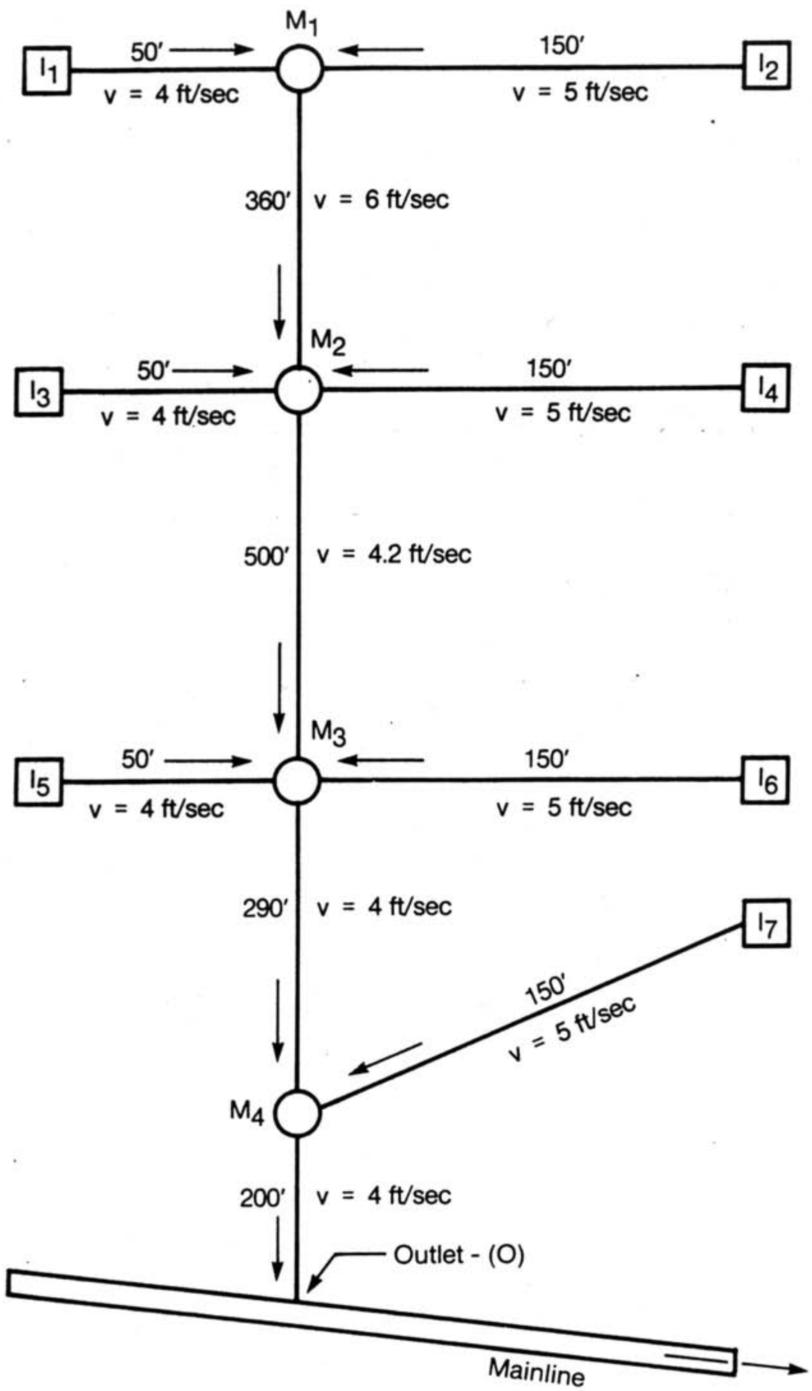


Figure 6-1
Hypothetical Storm Sewer System Layout
For Demonstrating Hydrologic Calculations

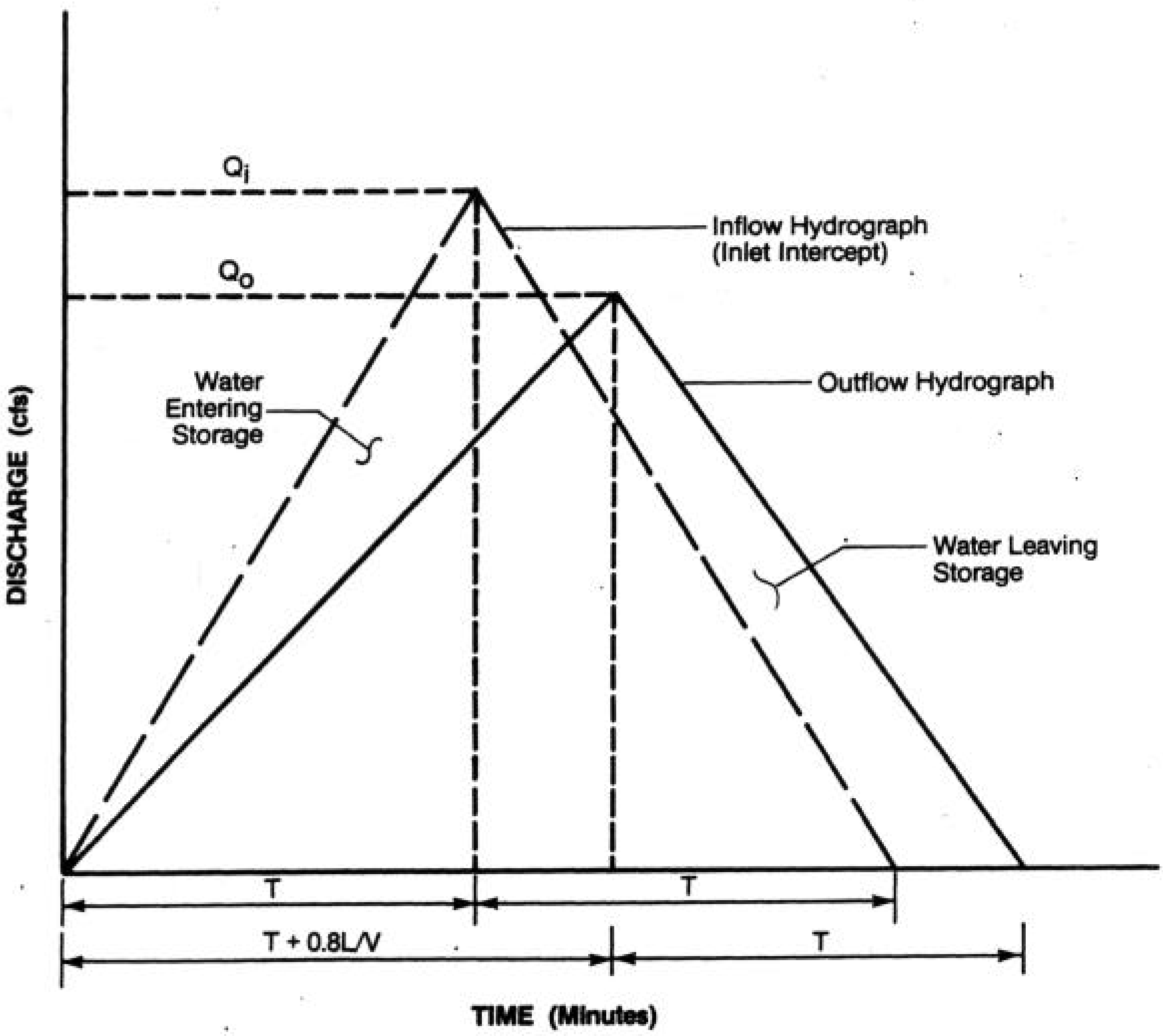


Figure 6-2
Triangular Approximation of Inlet Hydrographs

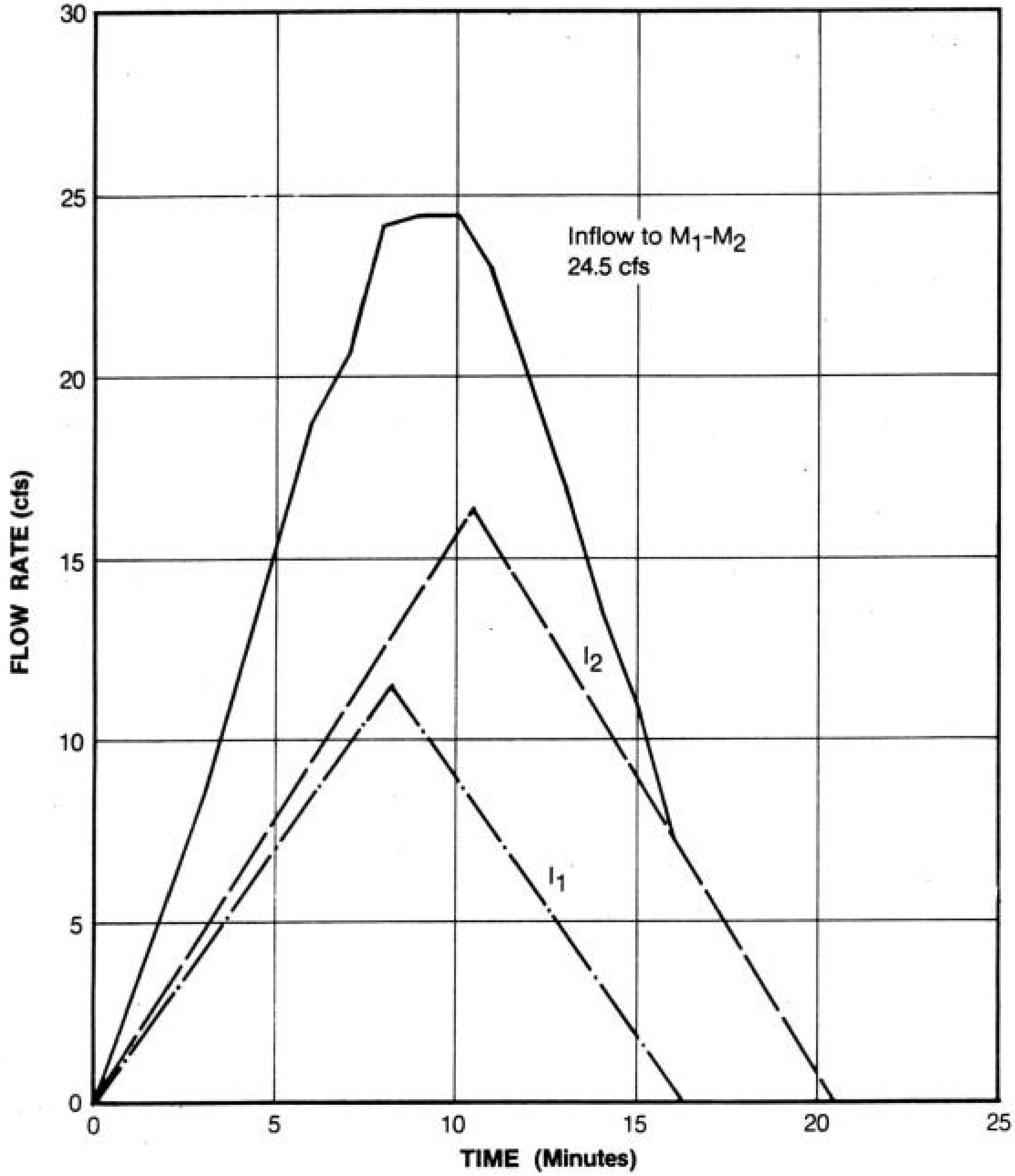


Figure 6-3
Inlet Hydrograph Results for Segment M_1-M_2
of the Hypothetical System in Figure 6-1

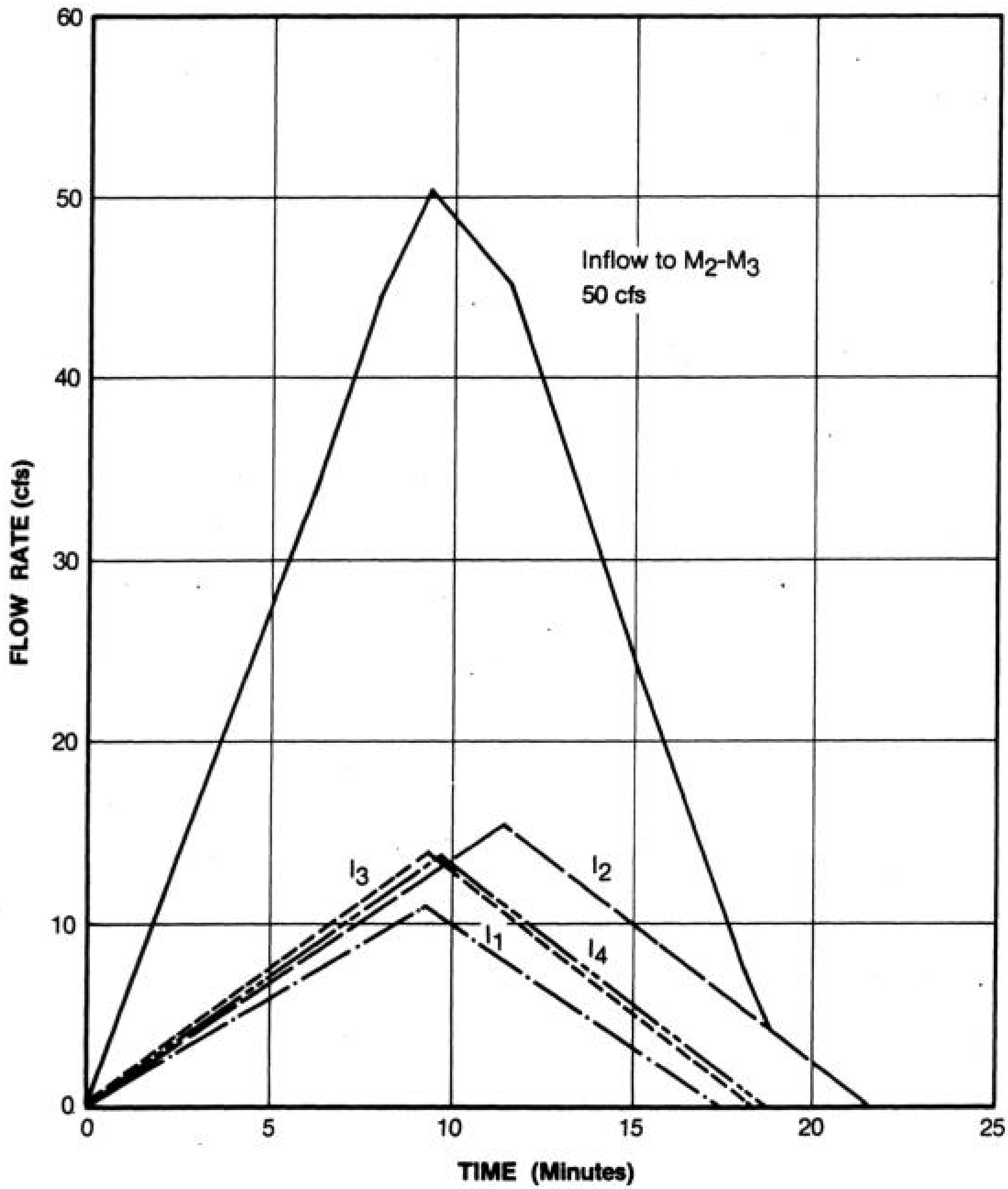


Figure 6-4
Inlet Hydrograph Results for Segment M_2-M_3
of the Hypothetical System in Figure 6-1

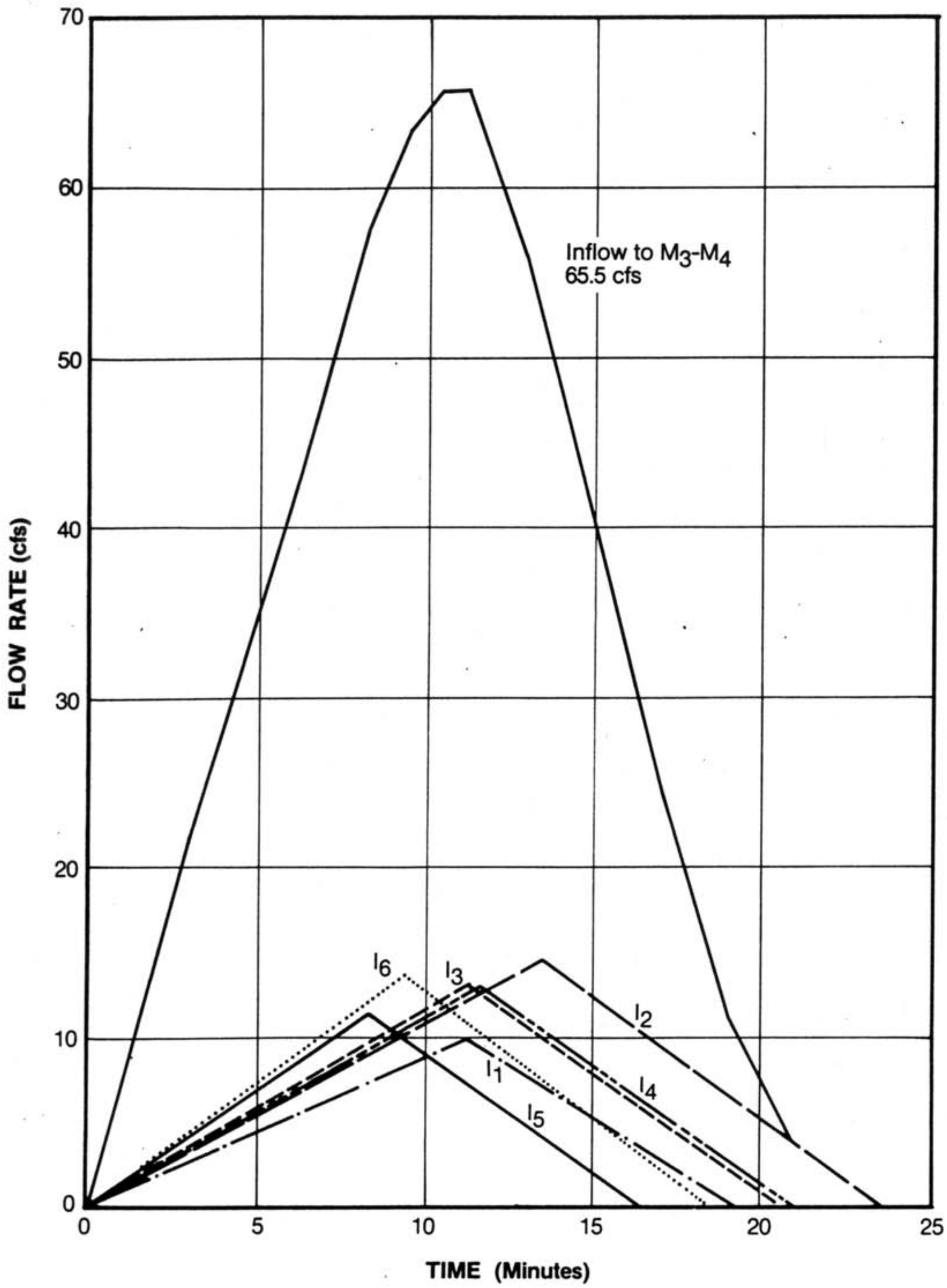


Figure 6-5
Inlet Hydrograph Results for Segment M_3-M_4
of the Hypothetical System in Figure 6-1

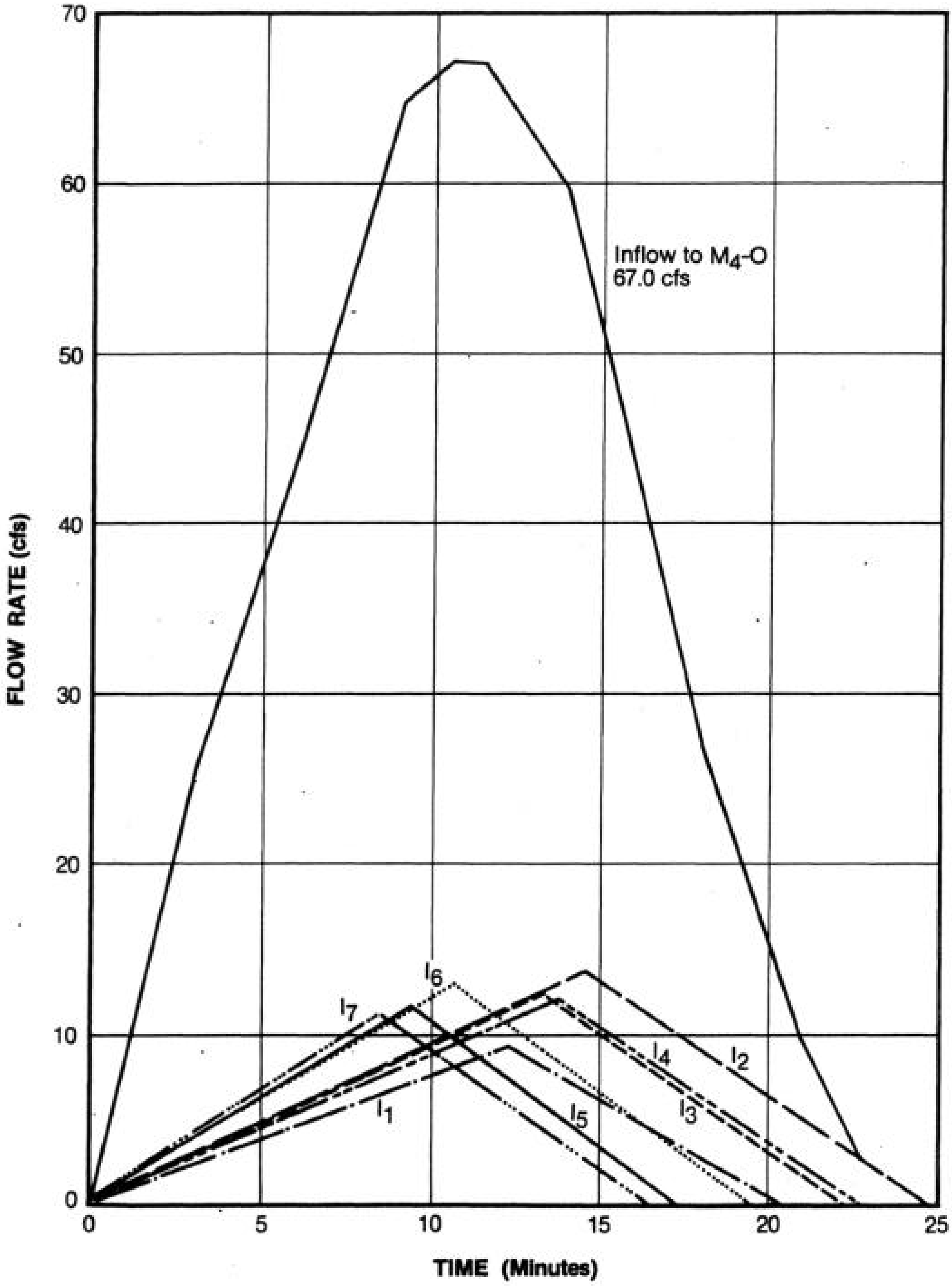
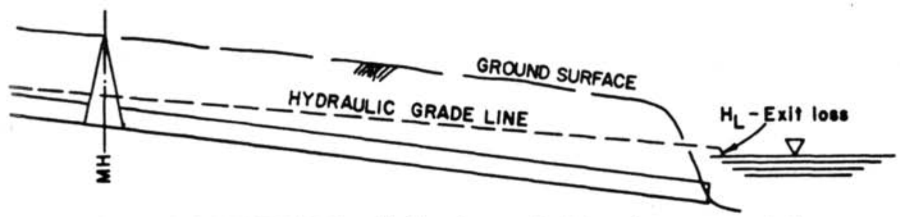
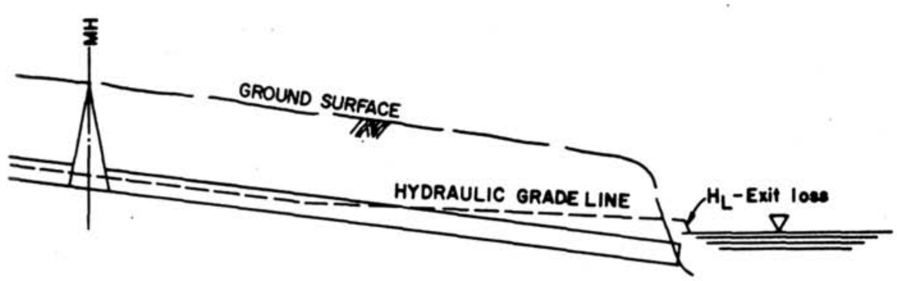


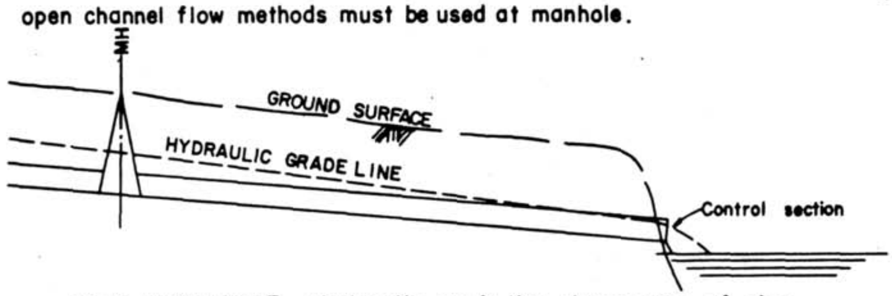
Figure 6-6
Inlet Hydrograph Results for Segment M_4-O
of the Hypothetical System in Figure 6-1



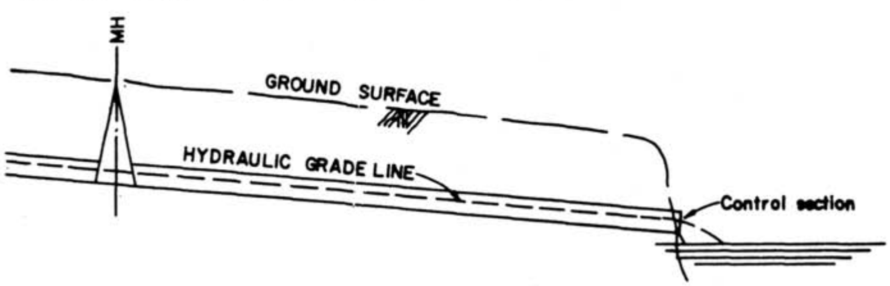
SUBMERGED DISCHARGE – Hydraulic grade line above crown of pipe, full flow design methods may be used at manhole.



SUBMERGED DISCHARGED – Hydraulic grade line below crown of pipe, open channel flow methods must be used at manhole.



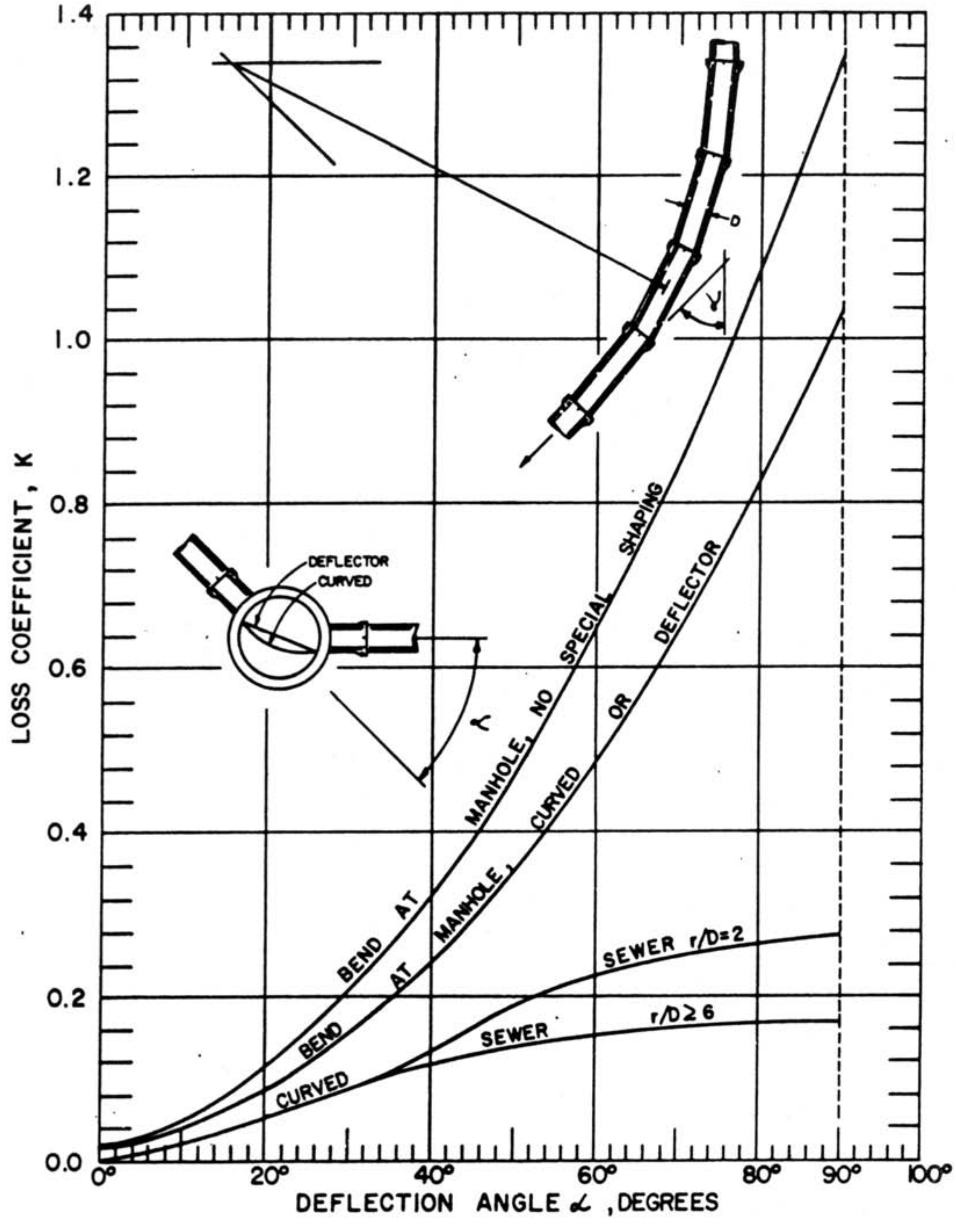
FREE DISCHARGE – Hydraulic grade line above crown of pipe, full flow design methods may be used at manhole.



FREE DISCHARGE – Hydraulic grade line below crown of pipe, open channel flow methods must be used at manhole.

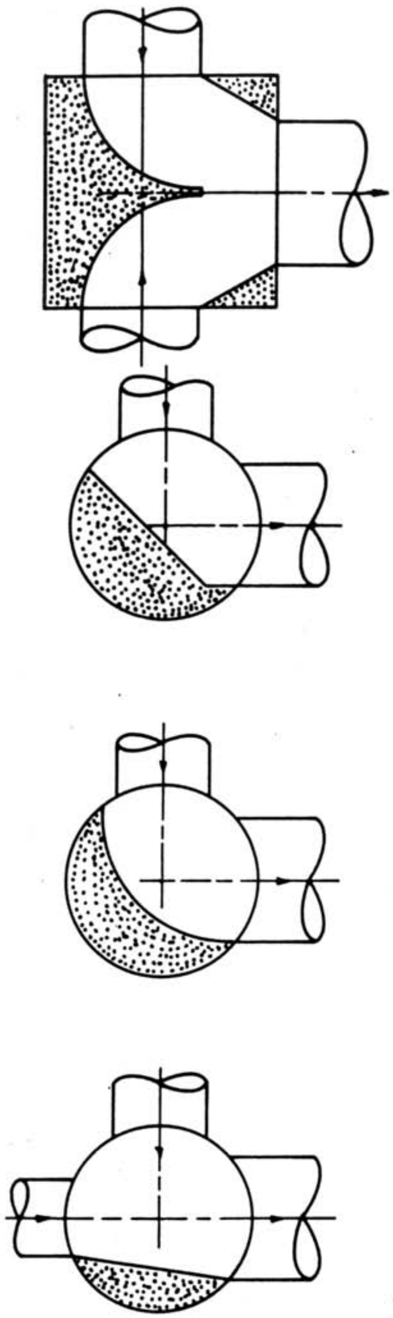
Reference: Wright-McLaughlin Engineers (1969).

Figure 6-7
Determination of Pressure vs. Open Channel Flow Conditions in Storm Sewer Systems



Reference: Wright-McLaughlin Engineers (1969).

Figure 6-8
Storm Sewer Bend Loss Coefficient



Directly opposed lateral with deflector
(head losses are still excessive with this method, but are significantly less than when no deflector exists.)

Bend with straight deflector

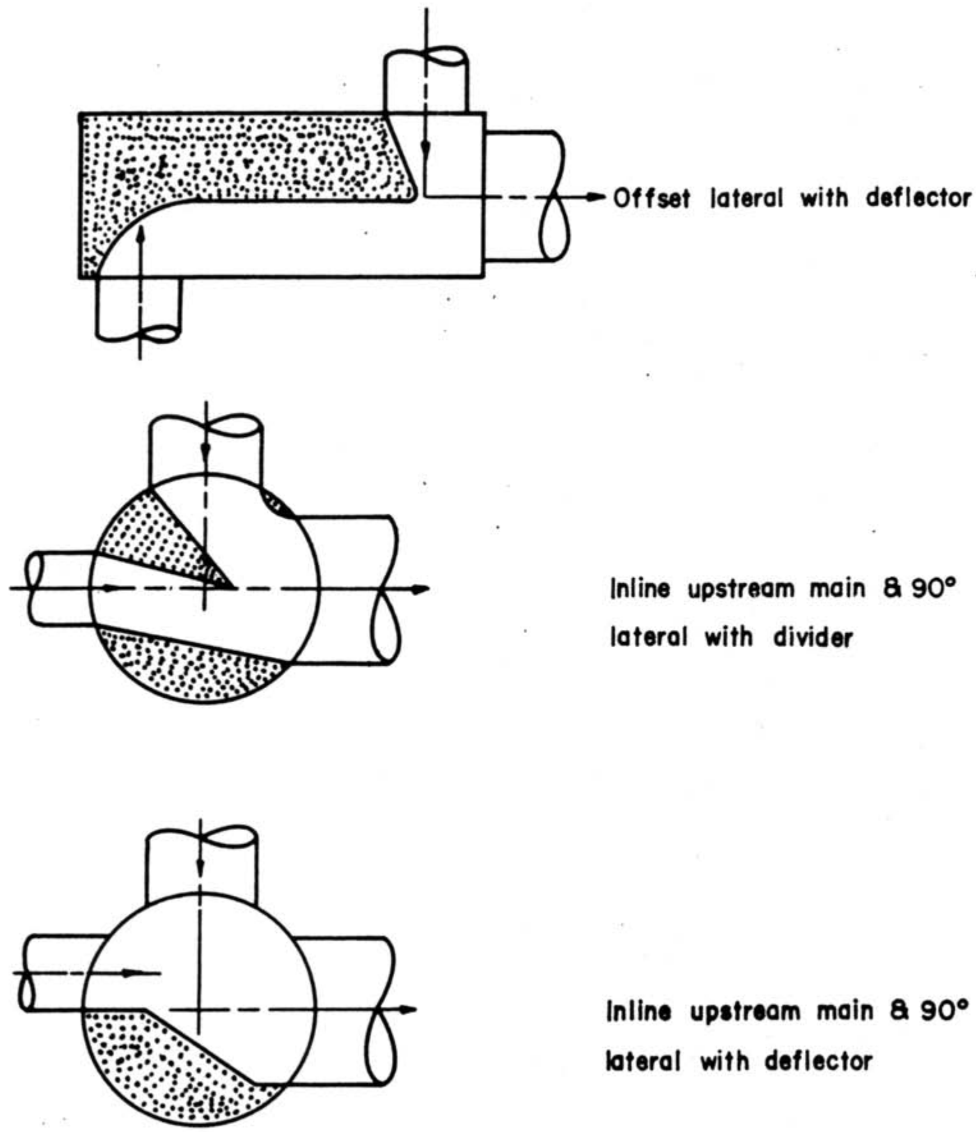
Bend with curved deflector

Inline upstream main & 90° lateral with deflector

These methods of shaping the interior of a manhole were found efficient in University of Missouri (1958) tests.

Reference: Wright-McLaughlin Engineers (1969).

Figure 6-9
Efficient Manhole Shaping



These methods of shaping the interior of a manhole were found inefficient in University of Missouri (1958) tests, either due to increased head loss or tendency to plug with trash.

Reference: Wright-McLaughlin Engineers (1969).

Figure 6-10
Inefficient Manhole Shaping

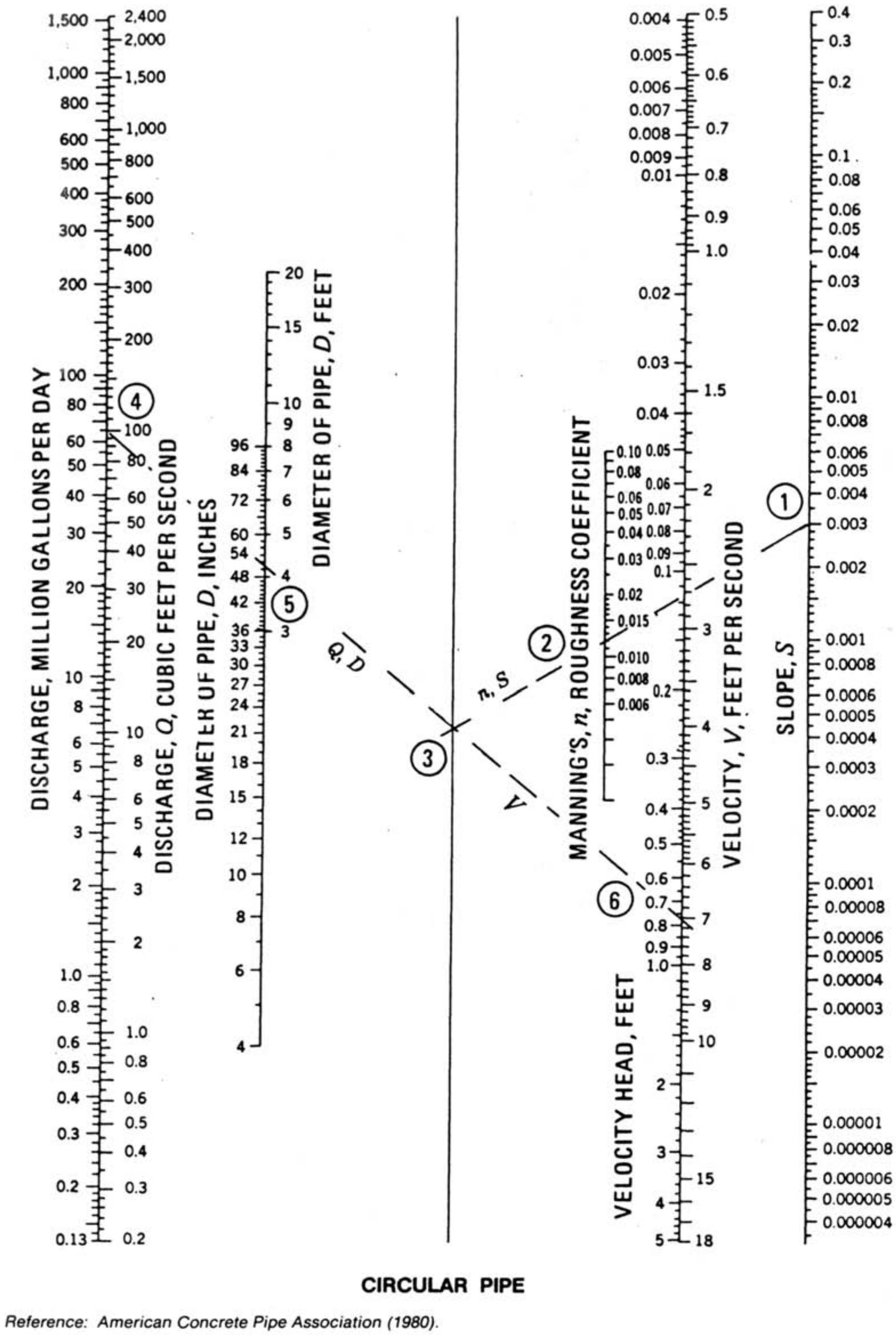


Figure 6-11
Circular Pipe Nomograph for Solving Manning's Equation

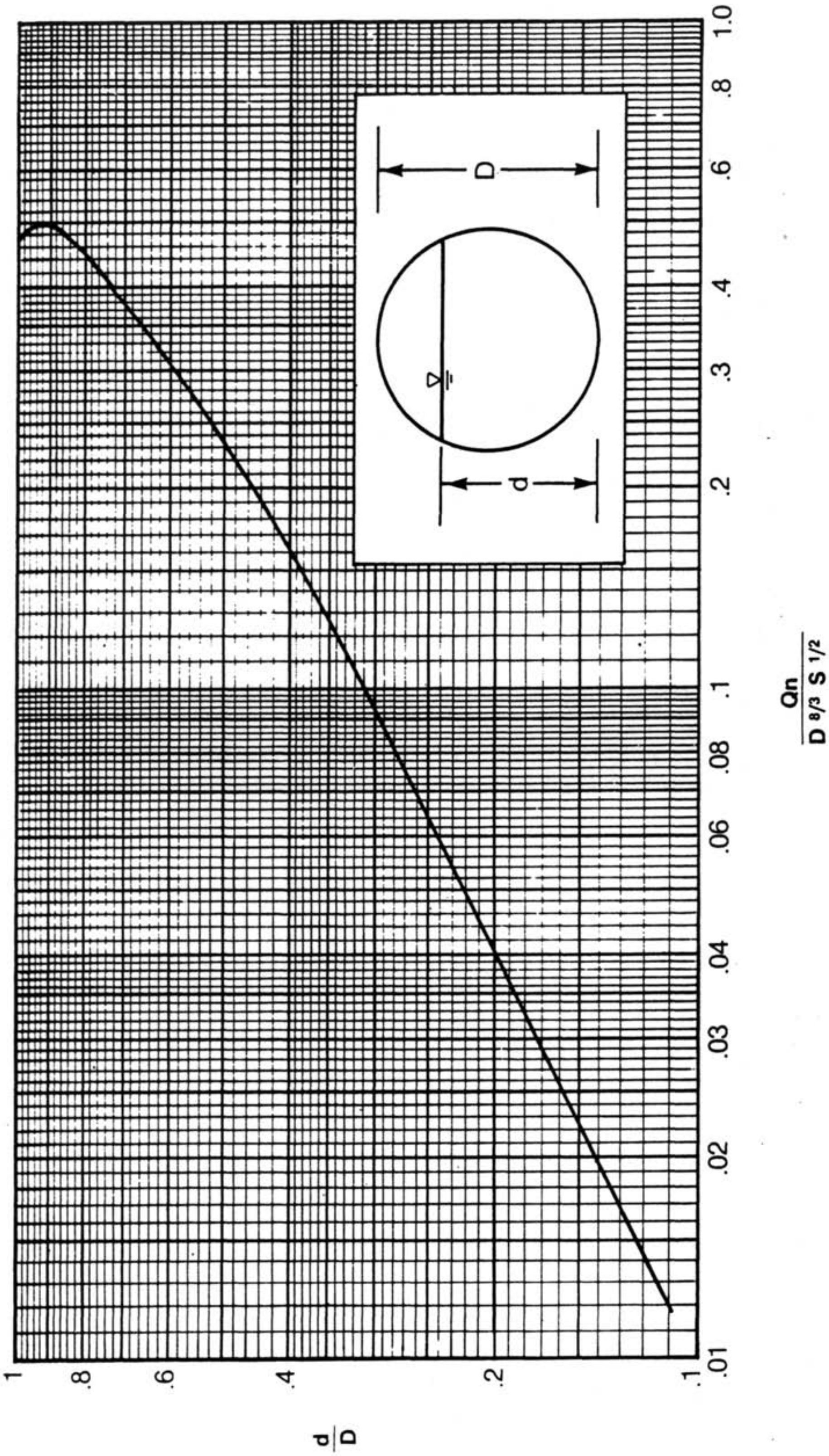


Figure 6-12
Circular Pipe Partial Flow Capacity Chart

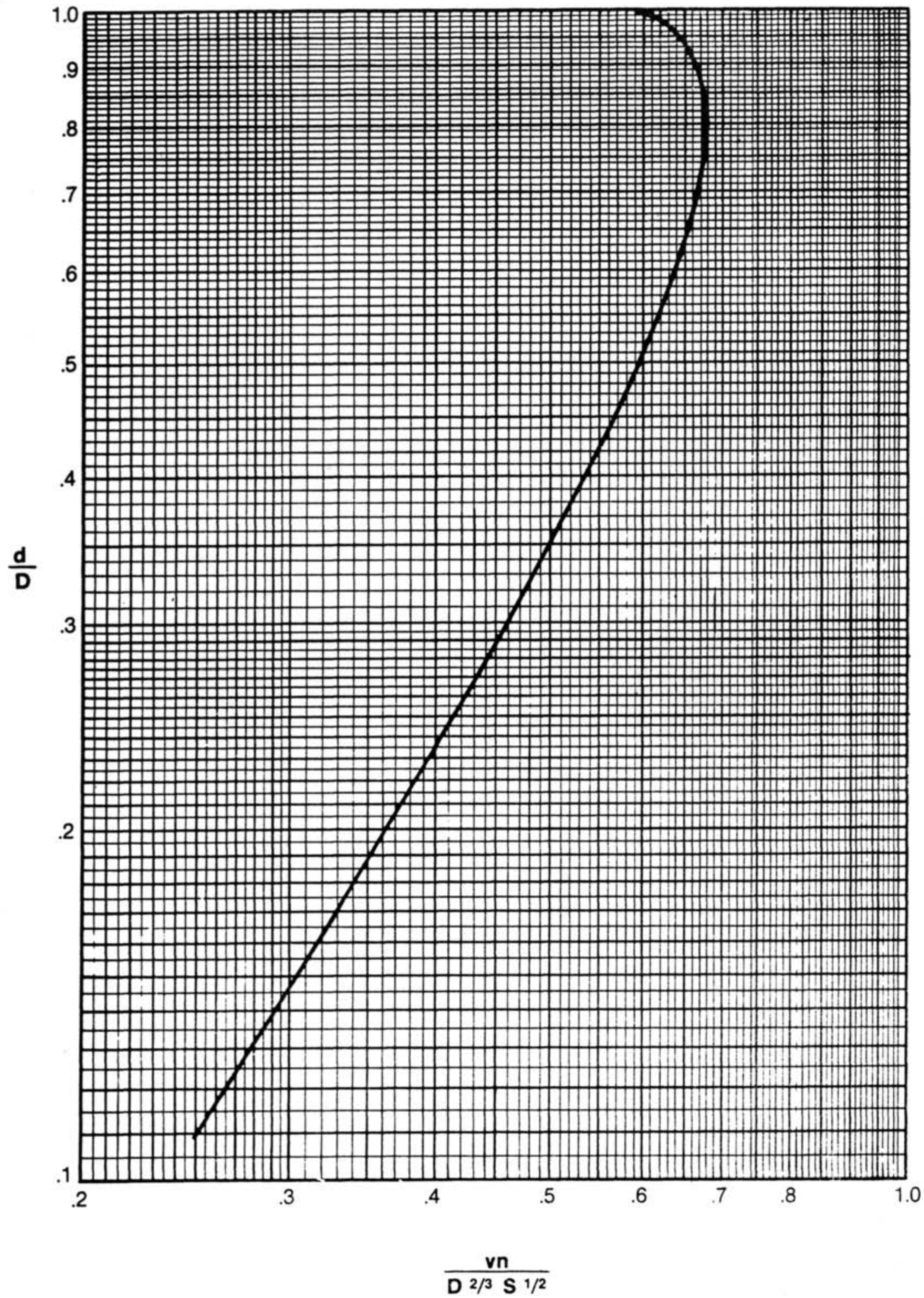


Figure 6-13
Circular Pipe Partial Flow Velocity Chart