SECTION 8
STORM SEWERS

CITY OF FORT LUPTON
STORM DRAINAGE DESIGN AND TECHNICAL CRITERIA

SECTION 8 STORM SEWERS

8.1 INTRODUCTION

Storm sewers are a part of the minor drainage system. They are required when the street or roadside ditch capacity is exceeded.

Except as modified herein, the design of storm sewers shall be in accordance with the MANUAL. The user is referred to the MANUAL and other references cited for additional discussion and basic design concepts.

A computer program for the design of a storm sewer system will be permitted.

8.2 CONSTRUCTION MATERIALS

Reinforced Concrete Pipe (RCP) shall have a minimum pipe class of Class-II and be in accordance with the CITY STANDARD.

Corrugated Metal Pipe (CMP) in accordance with ASSHTO M-190, M-196, M-197, or M-219; PVC pipe in accordance with ASTM F679 for solid wall pipe (SDR35) and ASTM F794, ASTM F1803, or AASHTO M304 for closed profile wall pipe; and HDPE pipe in accordance with AASHTO M294, AASHTO Section 18, or ASTM F667 are **only** permitted in privately owned and maintained installations. CMPs shall be invert paved, have a minimum thickness of 12-gauge, and meet H-20 loading criteria standards. The actual depth of cover, live load, and field conditions may require structurally stronger pipe than the minimums stated here. A typical design strength calculation shall be submitted to the CITY for approval. CMP or PVC is **not** permitted within public rights-of-way or public easements without the written consent of the Public Works Director/City Engineer.

8.3 HYDRAULIC DESIGN

Storm sewers shall be designed to convey the storm flows that exceed the allowable street capacity. To ensure that this objective is achieved, the hydraulic grade line and energy grade line of the storm sewer shall be calculated by accounting for the total hydraulic losses which include friction, expansion, contraction, bend, and junction losses. The methods for estimating these losses are presented in the following sections. Table 801 is provided to assist in calculating the losses, energy grade line, and hydraulic grade line. The final energy grade line shall be at or below the proposed ground surface. The hydraulic grade line and energy grade line for all storm sewers shall be shown on the profile sheets and included in the Phase III Drainage Report.

8.3.1 Pipe Friction Losses

The Manning's "n" values to be used in the calculation of storm sewer capacity and velocity are presented in Table 802.

8.3.2 Pipe Form Losses

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

$$H_L = K\frac{V^2}{2g} \qquad (8\text{-}1)$$

Where:
H_L = head loss (feet)
K = loss coefficient
V = average flow velocity (feet per second)
g = gravitational acceleration (32.2 ft/sec^2)

The following is a discussion of a few of the common types of form losses encountered in storm sewer system design. The reader is referred to References 1 and 6 for additional discussion.

1. 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:

$$H_L = K_e \frac{V_1^2}{2g} (1-\frac{A_1}{A_2})^2 \qquad (8\text{-}2)$$

Where:
A = the cross-section area (square feet)
V = the average flow velocity (ft/sec)
K_e = the loss coefficient.
The value of K_e is 1.0 for a sudden expansion and 0.2 for a well-designed expansion transition. Table 803 presents the expansion loss coefficients for various flow conditions.
g = gravitational acceleration (32.2 ft/sec^2)

Subscripts 1 and 2 denote the upstream and downstream sections, respectively.

2. Contraction Losses

The form loss due to contraction is:

$$H_L = K_c \frac{V_2^2}{2g} (1-\frac{A_2}{A_1})^2 \qquad (8\text{-}3)$$

Where:

A = cross-sectional area (square feet)

V = average flow velocity (ft/sec)

K_c = the contraction coefficient.

K_c is equal to 0.5 for a sudden contraction and 0.1 for a well-designed transition. Table 803 presents the contraction loss coefficients for various flow conditions.

g = gravitational acceleration (32.2 ft/sec^2)

Subscripts 1 and 2 denote the upstream and downstream sections, respectively.

3. Bend Losses

The head losses for bends, in excess of that caused by an equivalent length of straight pipe, may be expressed by the equation:

$$H_L = K_b \frac{V^2}{2g} \qquad (8\text{-}4)$$

Where:

V = average flow velocity (ft/sec)

g = gravitational acceleration (32.2 ft/sec^2)

K_b = the bend coefficient.

The bend coefficient has been found to be a function of (a) the ratio of the radius of curvature of the bend to the width of the conduit, (b) deflection angle of the conduit, (c) geometry of the cross-section of flow, and (d) the Reynolds number and relative roughness. The recommended bend loss coefficients are presented in Table 804 and Figure 804.

4. Junction and Manhole Losses

A junction occurs where one or more storm sewers enter a main storm sewer, usually at manholes. 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.

The head loss for a straight-through manhole or at an inlet entering the storm sewer is calculated from Equation 8-1.

The head loss at a junction can be calculated from:

$$H_L = \frac{V_2^2}{2g} - K_j \left(\frac{V_1^2}{2g}\right) \qquad (8\text{-}5)$$

Where: V = average flow velocity (ft/sec)
g = gravitational acceleration (32.2 ft/sec^2)
K_j = loss coefficient.
The coefficients for various junction configurations are presented in Figure 805.
Subscripts 1 and 2 denote the upstream and downstream sections, respectively.

8.3.3 Storm Sewer Outlets

When the storm sewer system discharges into the major drainage system (usually an open channel), additional losses occur at the outlet in the form of expansion losses as outlined in Section 8.3.2. For a headwall and no wingwalls, the loss coefficient (K_e) equals 1.0 while for a flared-end section, the loss coefficient is 0.5 or less. Table 803 presents the recommended loss coefficients.

8.3.4 Partially Full Pipe Flow

When a storm sewer is not flowing full, the storm sewer acts like an open channel and the hydraulic properties of the pipe can be calculated using open channel techniques. For convenience, charts for various pipe shapes have been developed for calculating the hydraulic properties (Figures 801, 802, 803). The data presented in these figures assumes that the friction coefficient, Manning's "n" value, does not vary by depth.

8.4 VERTICAL ALIGNMENT

The storm sewer longitudinal slope shall be such that a minimum cover is maintained. The minimum cover depends upon the pipe size, type, class, and soil bedding condition. The pipe shall be able to withstand AASHTO HS-20 (or as designated by the CITY) loading on the pipe but the cover shall not be less than 18-inches at any point along the pipe.

8.5 HORIZONTAL ALIGNMENT

A storm sewer alignment may be curvilinear for pipe with diameters of 48-inches or greater but only when approved by the Public Works Director/City Engineer. The applicant must demonstrate the need for a curvilinear alignment. Generally, a curvilinear alignment will only be allowed where physical constraints dictate its use.

The limitations on the radius for pulled-joint pipe is dependent on the pipe length and diameter, and the amount of opening permitted in the joint. The maximum allowable joint pull shall be 3/4-inches. The radius requirements for pipe bends are dependent upon the manufacturer's specifications with the minimum radius requirements given in Table 805. The pipe must be specifically manufactured for curvilinear alignment.

8.6 PIPE SIZE

The minimum allowable pipe size for storm sewers is dependent upon a practical diameter and length from a maintenance standpoint. Table 805 presents the minimum pipe size for storm sewers. At detention pond outlets, orifice plates shall be designed and installed to control the pond outflow.

8.7 MANHOLES

Manholes or maintenance access ports will be required whenever there is a change in size, direction, elevation, slope, or at a junction of two or more sewers. Also, the maximum spacing between manholes for various pipe sizes shall be in accordance with Table 805. Additional manholes may be required in a curvilinear reach.

The required manhole size shall be as follows:

MANHOLE SIZE

Storm Sewer Diameter	**Sum of Pipe Diameter (3 or More)**	**Manhole Diameter**
15" to 18"	less than 54"	4'
21" to 27"	less than 72"	5'
30" to 33"	less than 90"	6'
36" and larger	greater than 90"	Refer to the STANDARD

Larger manhole diameters or a junction structure may be required when storm sewer alignments are not straight through, more than one storm sewer line goes through the manhole, or pipe entry angles do not allow adequate space between pipes at the manhole. "Tee" manholes are acceptable for storm sewers equal to or larger than 60-inch in diameter.

8.8 DESIGN EXAMPLE

The following example, which is shown on Table 806, Figure 806, and Figure 807, was obtained from Modern Sewer Design, AISI, Washington, D.C., 1980 (Reference 26). The example was edited in the calculation of manhole and junction losses in accordance with this section. The following procedure is based on full-flow conditions. If the pipe is flowing substantially full (i.e., greater than 80 percent), 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.

EXAMPLE: HYDRAULIC DESIGN OF STORM SEWERS

Given:
(a) Plan and Profile of storm sewer (Figure 806 and Figure 807)
(b) Station 0+00 (outlet) data as follows:

Design discharge	Q	=	145 cfs	[9]
Invert of pipe	El	=	94.50'	[2]
Diameter	D	=	66" RCP	[3]
Starting water surface	W.S.	=	100 ft	[4]
Area of pipe	A	=	23.76 sq.ft.	[6]
Velocity	V	=	6.1 ft/s	[8]

Note:
(1) Number in brackets refers to the columns on Table 806.
(2) Sizes of the storm sewer were determined during the preliminary design phases

Find: Hydraulic Grade Line and Energy Grade Line for storm sewer.

Procedure:

Step 1:

The normal depth is greater than critical depth, $d_n > d_c$; therefore, the flow is subcritical and the calculations need to begin at outlet and proceed upstream.

Compute the following parameters:

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

$$C = \frac{2gn^2}{2.21} = \frac{(2)(32.2)(0.013)^2}{2.21} = 0.00492 \qquad [7]$$

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

Step 2:

Velocity head: $$H_V = \frac{V^2}{2g} = \frac{(6.1)^2}{(2)(32.2)} = 0.58 \qquad [10]$$

Step 3:

For the initial calculation, the Energy Grade Line is calculated as:

$$E.G. = W.S. + H_V = 100 \text{ ft} + 0.58 \text{ ft} \qquad [11]$$

$$E.G. = 100.58 \text{ ft}$$

For subsequent calculations, the water surface elevation is calculated as follows:

$$W.S. = E.G. - H_V \qquad [11]$$

This equation is used since the losses computed in Step 8 are energy losses, which are added to the downstream energy grade elevation. The velocity head is then subtracted to compute the water surface elevation (hydraulic grade line).

Step 4:

$$S_f = \mathbb{C}\frac{H_V}{R^{\frac{4}{3}}} = (0.00492)\frac{(0.58)}{(1.375)^{\frac{4}{3}}} = 0.0019 \qquad [12]$$

Where: R = the hydraulic radius of the pipe.

Step 5:

$$\text{Avg. } S_f \qquad [13]$$

This is the average value between S_f of the station being calculated and the previous station. For the first station, Avg. $S_f = S_f$. The entries are placed in the next row since they represent the calculated losses between two stations.

Step 6:

sewer length L, ft [14]

Step 7:

$$\text{Friction loss } H_f = (\text{Avg. } S_f)(L) \qquad [15]$$
$$H_f = (0.0019)(110) = 0.21$$

Step 8:

Calculate the form losses for bends, junctions, manholes, and transitions (expansion or contraction) using Equations 8-1, 8-2, 8-3, 8-4, and 8-5. 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)

$H_b =$ $K_b H_V$, where the degree of bend is 60^o
$K_b =$ 0.20 (Table 804, Case I)
$H_b =$ (0.20)(0.58) = 0.12 ft

[16]

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

$$H_L = K_e \frac{V_1^2}{2g} (1-\frac{A_1}{A_2})^2$$

$K_e =$ 1.06 (Table 803) for D_2/D_1 = 1.5, and $\theta = 45^o$
$H_L =$ (1.06)(1.29) $[1 - (15.9/23.76)]^2$ = 0.15 ft

(c) station 3 + 55.5 Column 19 (manhole, straight through)

$H_m =$ $K_m H_v$
$K_m =$ 0.05 (Figure 805, Case I)
$H_m =$ (0.05)(1.29) = 0.06 ft

[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 (Figure 805, Case III), $\theta = 30^o$
$H_j =$ 1.29 - (0.62)(0.99) = 0.68 ft [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 (Figure 805 Case III), $\theta = 70^o$
$H_j =$ 0.99 - (0.33)(0.64) = 0.78 ft for one lateral

Step 9:

Sum all the form losses in 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 ft.

Step 10:

Add the total form loss in Column 20 to the energy grade at the downstream end (Sta. 00+0) to compute the energy grade at the upstream end (Sta. 1+10)

$$\begin{aligned} \text{E.G. (U/S)} &= \text{E.G. (D/S)} + \text{TOTAL LOSS} \\ &= 100.58 \text{ ft} + 0.21 \text{ ft} \\ &= 100.79 \text{ ft} \qquad [11] \end{aligned}$$

Step 11:

Enter the new pipe invert elevation [2], pipe diameter [3], pipe shape [5], pipe area [6], the computed constant ℂ from Step 1 [7], the computed velocity [8], the new flow rate [9], and the computed velocity head [10].

Step 12:

Compute the new water surface elevation for the upstream station (1+10 for this example).

$$\begin{aligned} &\text{W.S.} = \text{E.G.} - H_V \\ &= 100.79 \text{ ft} - 0.58 \text{ ft} = 100.21 \text{ ft} \qquad [4] \end{aligned}$$

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 drawing (Figure 806).

<u>Discussion of Results</u>:

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" RCP has a greater capacity (approximately 175 cfs) at the given slope than the design flow (145 cfs); thus, the pipe is not flowing full but is substantially full (i.e., 145 cfs /175 cfs = 0.84 which is greater than 0.80). At the outlet, the computed HGL is below the crown of the pipe. However, the actual HGL is higher since the outlet of the 54" RCP is submerged by the headwater for the 66" 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 below the top of the pipe. In this case, the full flow capacity (100 cfs) is the same as the design capacity and the HGL remains parallel to the top of the pipe. A similar situation occurs at Station 5+65.5 except that the HGL remains above and parallel to the top of the pipe.

If the pipe invert entering a manhole or junction is at an elevation significantly above the manhole invert, a discontinuity in the 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.

8.9 CHECKLIST

To aid the designer and reviewer, the following checklist has been prepared:

1. Calculate the energy grade line (EGL) and hydraulic grade line (HGL) for all storm sewers. Provide the calculations and resulting profiles in the Phase III Drainage Report.

2. Account for all losses in the EGL calculations including friction, outlet, form, bend, manhole, and junction losses. Provide all calculations in the Phase III Drainage Report.

3. Provide adequate protection at the outlet of all storm sewers into open channels.

4. Check for minimum pipe cover.

5. Check for adequate clearance with other utilities.

Table 801
Water Surface and Energy Grade Line
Calculations for a Storm Sewer

Pipe Material ______________
Manning's n ______________

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)
station	Invert	Pipe Dia.	W.S. Elev	Pipe Shape	Area	¢	Velocity	Flow Rate	H_v	Energy Grade Line	S_f	Avg S_f	Length	H_f	H_b	H_j	H_m	H_t	Total Loss
	(ft)	(in)	(ft)		(ft^2)		(fps)	(cfs)	(ft)	(ft)			(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)

Total Friction Loss =

¢ $= (2g(n^2))/2.21$

Total Form Loss =

Total Loss=

$S_f = ($¢$H_v)/R^{1.33}$

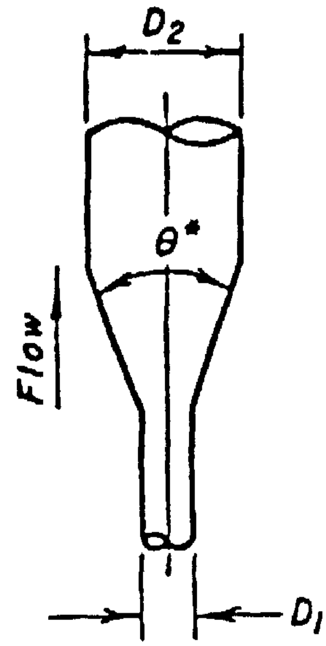
Table 802
Pipe Roughness Coefficients

Manning's n-value		
Sewer Type	**Capacity Calculation**	**Velocity Calculation**
RCP	0.015	0.011
PVC	0.015	0.011
CMP –25% paved	0.021	0.017
CMP – fully paved	0.017	0.013

Table 803
Energy Loss Coefficient
Expansion and Contraction

Expansion K_e		
θ	$\frac{D_2}{D_1} = 3$	$\frac{D_2}{D_1} = 1.5$
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 is the angle in degrees between the sides
- of the tapering section.



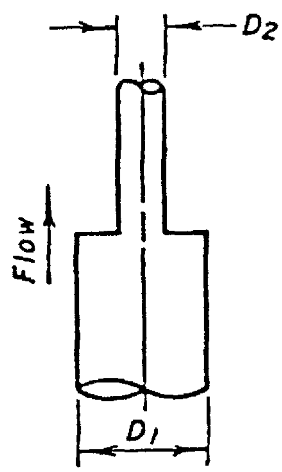
Pipe Entrance from Reservoir

Bell-mouth $H_L = 0.04 \frac{V^2}{2g}$

Square-edge $H_L = 0.5 \frac{V^2}{2g}$

Groove end upstream for concrete pipe $H_L = 0.2 \frac{V^2}{2g}$

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



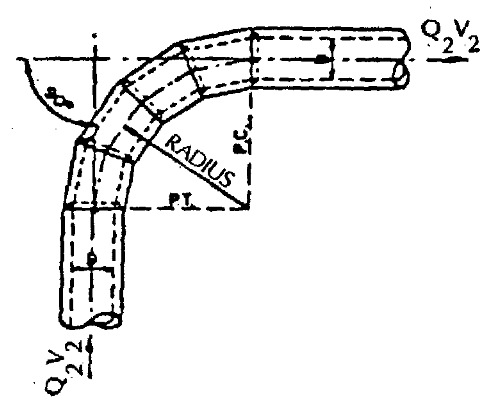
Reference: Linsley and Franzini "Water Resource Engineering," McGraw-Hill, 1964.

Table 804
Energy Loss Coefficients
Bends

Case I **Conduit on 90 degree curves**	
θ	**K_b**
90	0.25
60	0.20
45	0.18
30	0.14

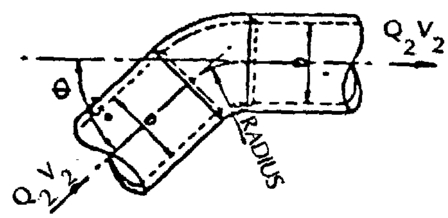
Note: Head loss applied at P.C. for length

$K_b = 0.25 \left(\frac{\theta}{90}\right)^{0.5}$



Case II **Bends where radius of curve is equal to diameter of pipe**	
θ	**K_b**
90	0.50
60	0.43
45	0.35
22 1/2	0.20

Note: Head loss is applied at beginning of bend



Reference: APWA Special Report No. 49, 1981

Table 805
Alignment and Size

Minimum Pipe Diameter		
Type	**Minimum Pipe Diameter**	**Minimum Cross-sectional area**
Main trunk	18 inch	1.77 sq. feet
Lateral from the inlet	15 inch	1.23 sq. feet

Note: Minimum size of the lateral shall also be based upon a water surface inside the inlet at a minimum distance of 1-foot below the grate or throat.

Diameter of Pipe	Maximum Allowable Distance between Manholes and/or Cleanouts
15” to 36”	400 feet
42” and larger	500 feet

Minimum Radius for Radius Pipe	
Diameter of Pipe	**Radius of Curvature**
48” to 54”	28.5 feet
57” to 72”	32.0 feet
78” to 108”	38.0 feet

Reference: “Urban Storm Drainage Criteria Manual”, DRCOG, 1969

Table 806
Design Example for Storm Sewer

Pipe Material _RCP_

Manning's *n* _0.013_

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)
station	Invert	Pipe Dia.	W.S. Elev	Pipe Shape	Area	ϕ	Velocity	Flow Rate	H_v	Energy Grade Line	S_f	Avg S_f	Length	H_f	H_b	H_j	H_m	H_t	Total Loss
	(ft)	(in)	(ft)		(ft^2)		(fps)	(cfs)	(ft)	(ft)			(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)
0+00	94.50	66	100	RND	23.75	0.00492	6.1	145	0.58	100.58	0.0019	0.0019	110	0.21					0.21
1+10	94.71	66	100.21	RND	23.75	0.00492	6.1	145	0.58	100.79	0.0019	0.0019	42.4	0.08	0.12				0.20
1+52.4	94.91	66	100.41	RND	23.75	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	23.75	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	12.56	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	12.56	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								

Total Friction Loss = 3.43

$\phi = (2g(n^2))/2.21$

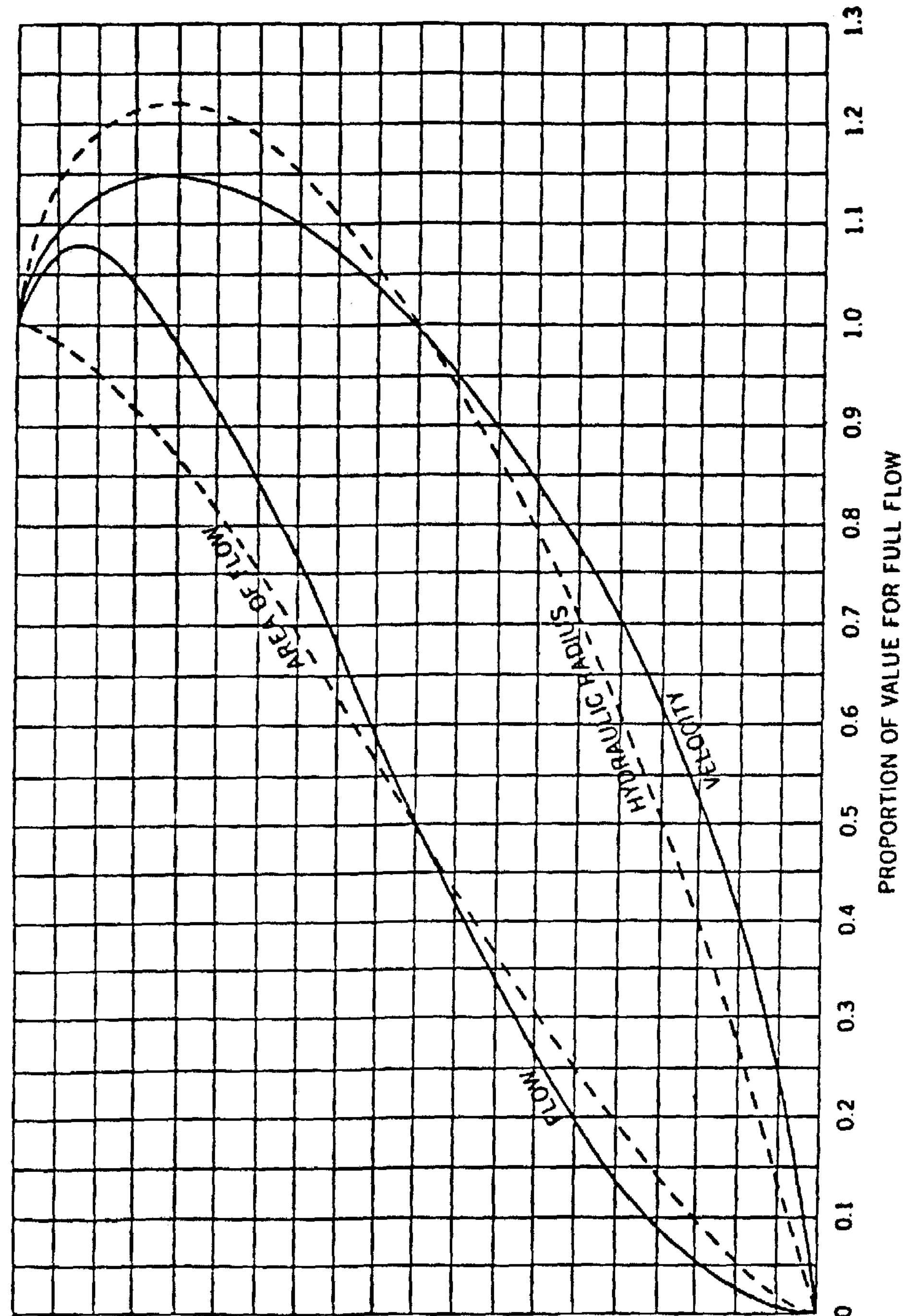
Total Form Loss = 2.60

Total Loss= 6.03

$S_f = (\phi H_v)/R^{1.33}$

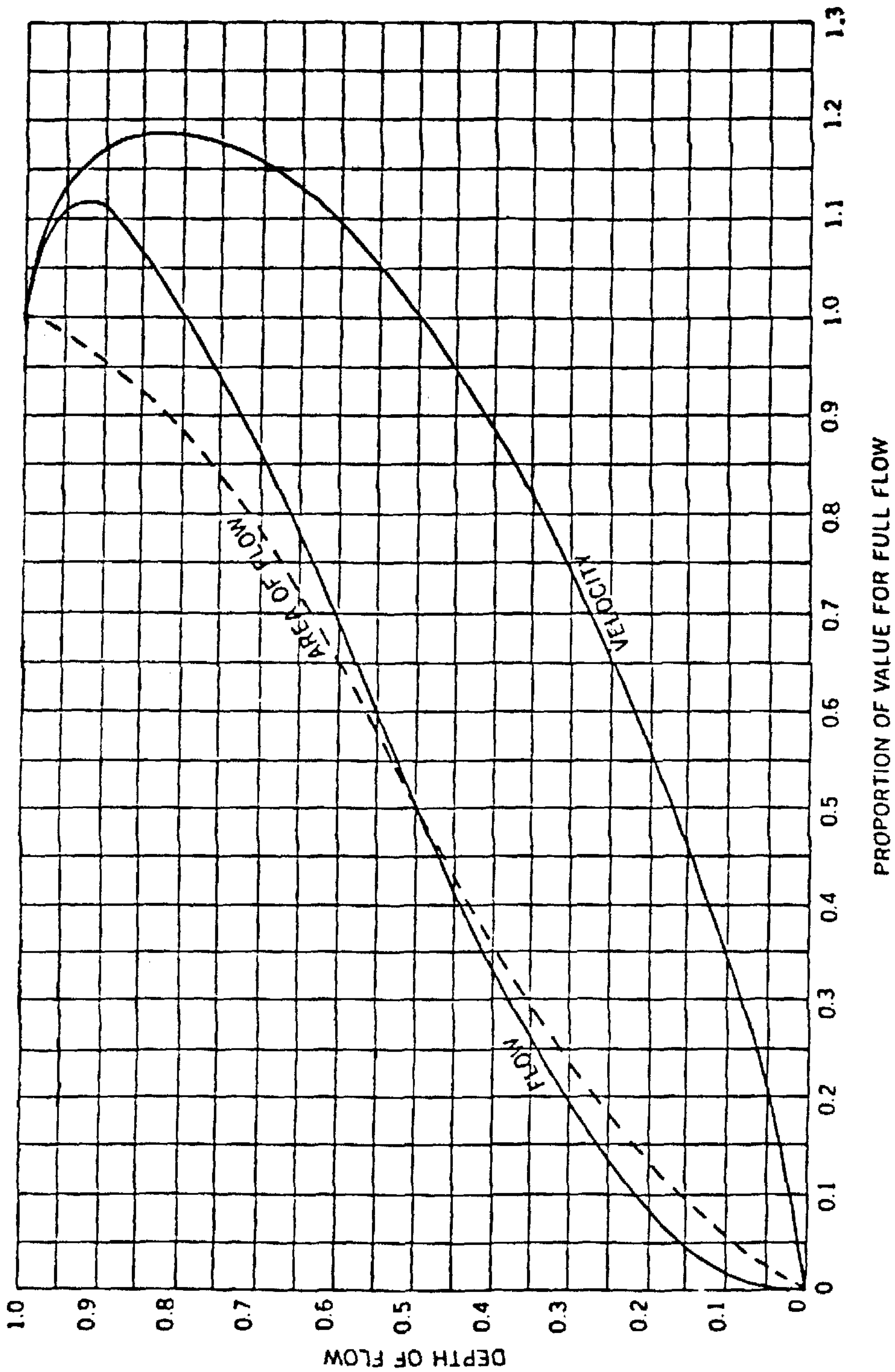
Reference: Modern Sewer Design, AISI, Washington DC 1980

Figure 801
Hydraulic Properties of Circular Pipe



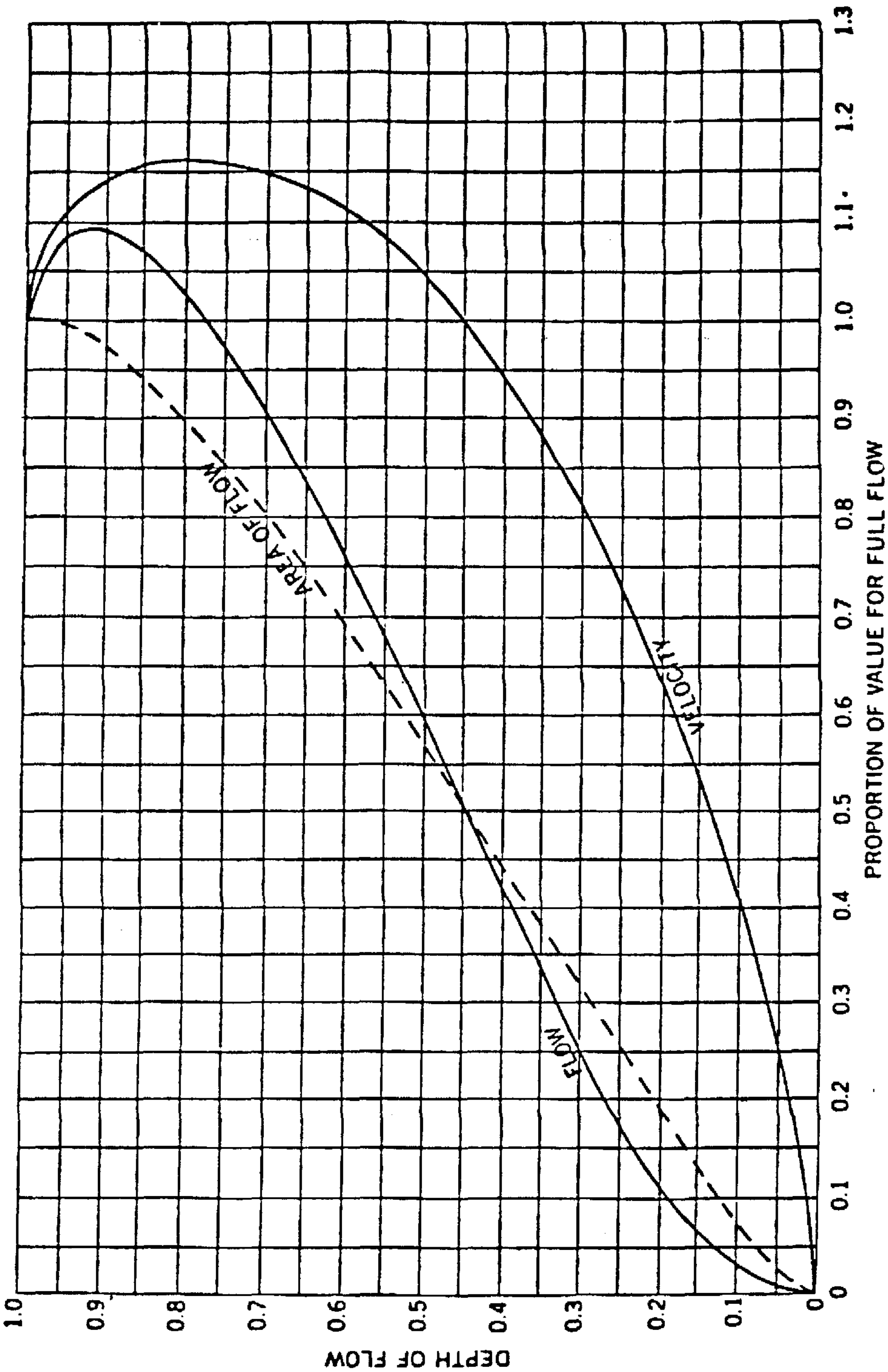
Reference: "Concrete Pipe Design Manual" ACPA, 1970

Figure 802
Hydraulic Properties of Horizontal Elliptical Pipe



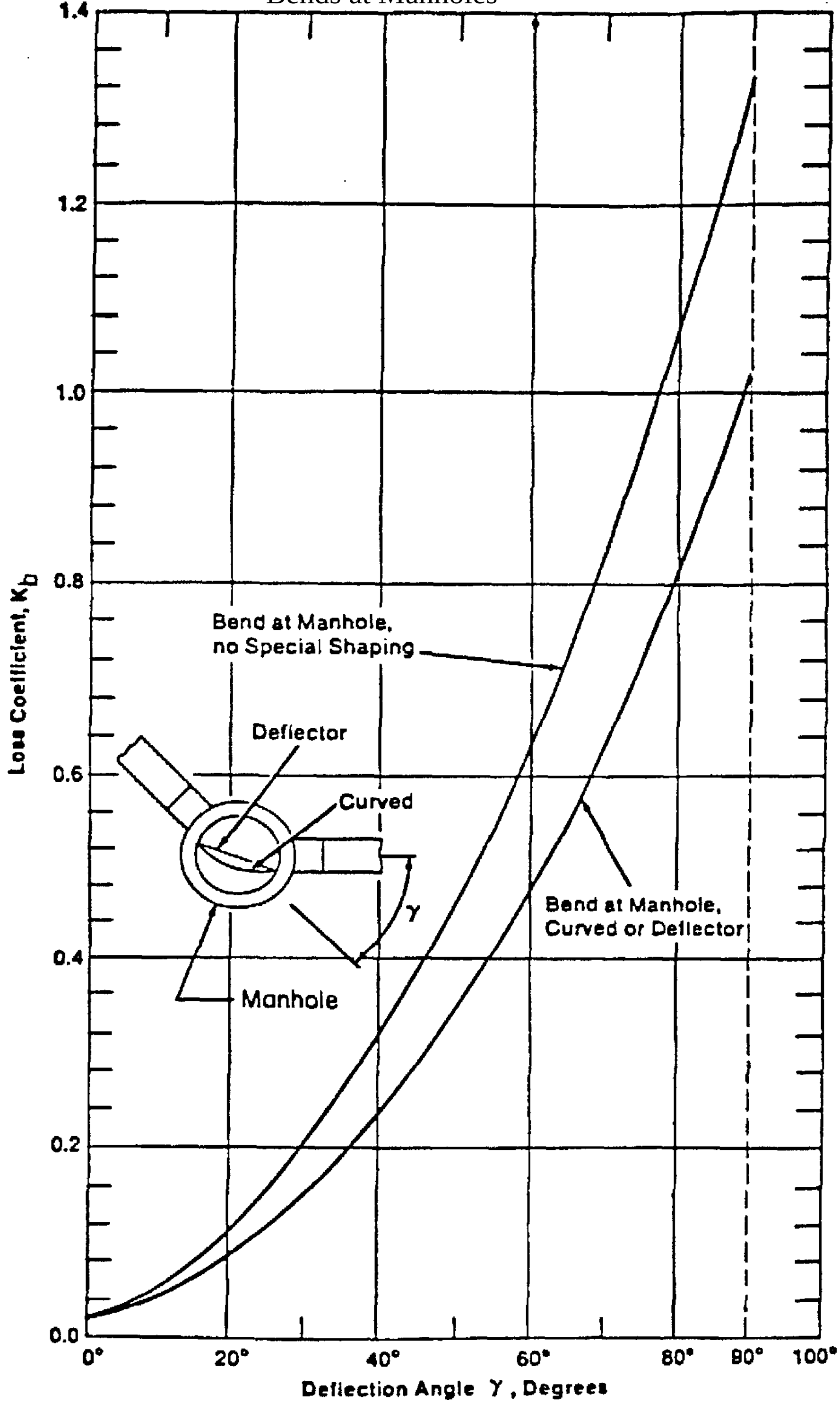
Reference: "Concrete Pipe Design Manual" ACPA, 1970

Figure 803
Hydraulic Properties of Arch Pipe



Reference: "Concrete Pipe Design Manual" ACPA, 1970

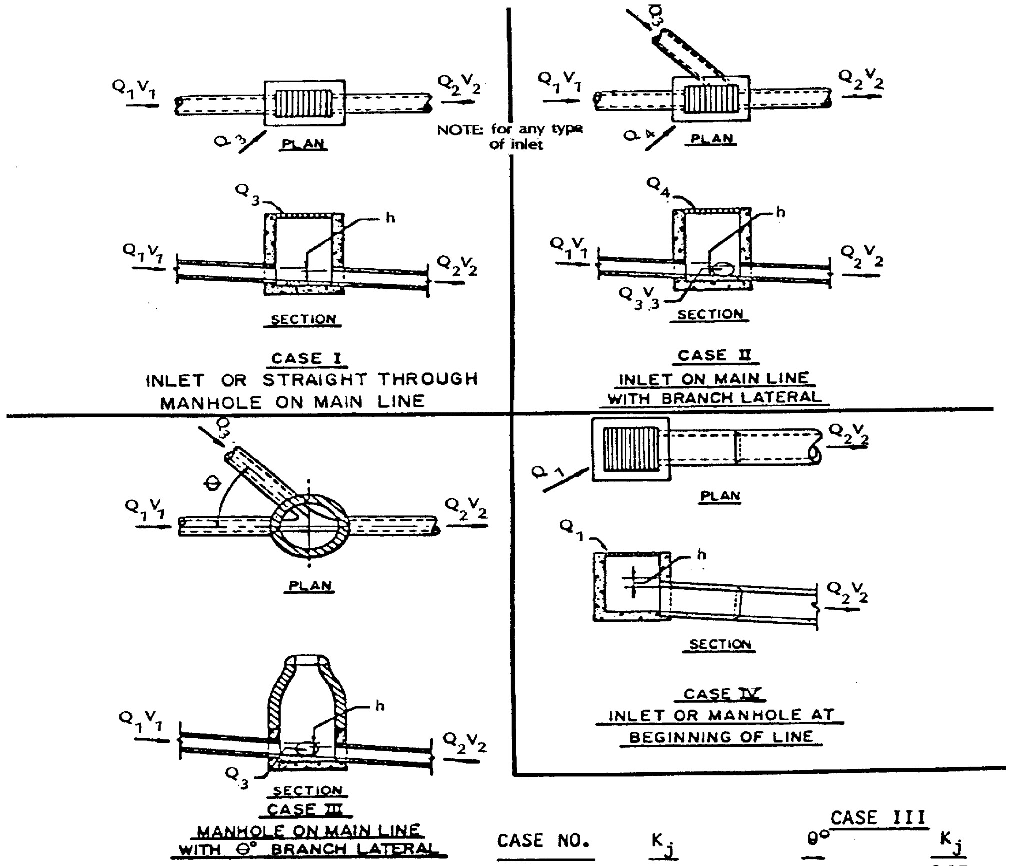
Figure 804
Energy Loss Coefficients
Bends at Manholes



Reference: Modern Sewer Design, AISI, Washington D.C., 1980

NOTE: Head loss applied at outlet of manhole.

Figure 805
Manhole and Junction Losses



CASE NO.	K_j
I	0.05
II	0.25
IV	1.25

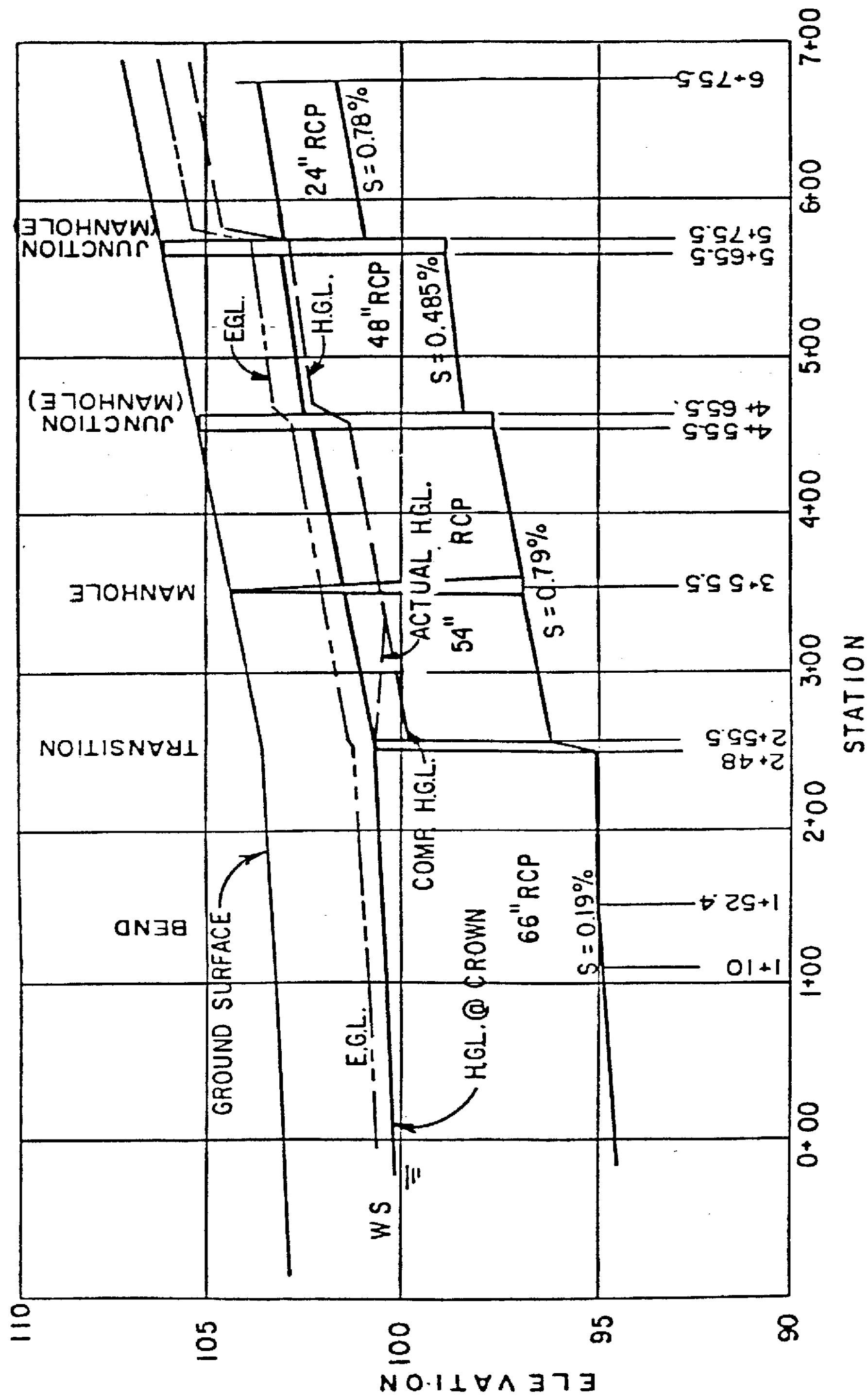
CASE III

θ°	K_j
0	0.95
22 1/2	0.75
45	0.50
60	0.35
90	0.25

NO LATERAL - SEE CASE I

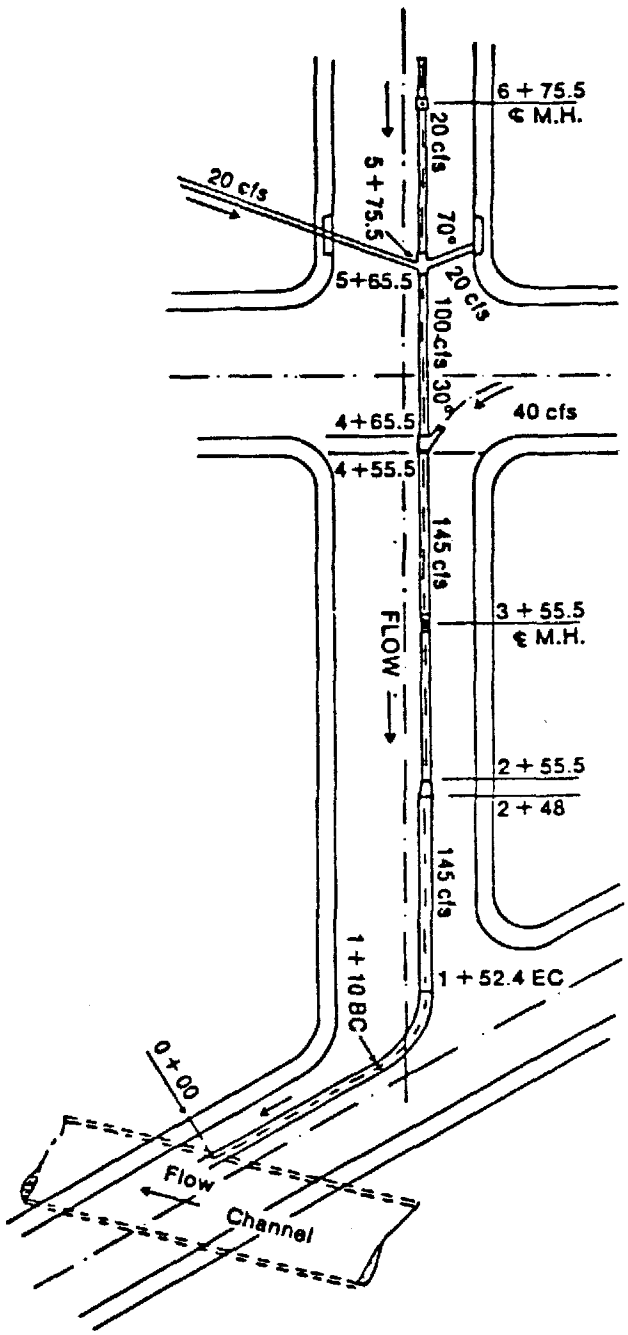
Reference: APWA Special Report No. 49, 1981

Figure 806
Design Example of Storm Sewer
Profile View



Reference: Modern Sewer Design, AISI, Washington DC 1980

Figure 807
Design Example of Storm Sewer
Plan View



Reference: Modern Sewer Design, AISI, Washington DC 1980