# CHAPTER 6

# STORM SEWERS

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## 6.1 INTRODUCTION

Storm sewers are a part of the Minor Drainage System, and are required when the other parts of the minor system, primarily curb, gutter, and roadside ditches no longer have capacity for additional runoff.

A computer program for the design of a storm sewer system will be permitted provided the model is calibrated to three or more design points using the procedures presented in these CRITERIA.

## 6.2 CONSTRUCTION MATERIALS

All storm sewers within the Town shall be constructed using one of the following materials and meet the applicable standard as presented below:

STORM SEWER STANDARDS

| Pipe Material | Standard |
|---|---|
| Reinforced Concrete | ASTM C-76/C-506/C-507/C-789/ C-850 OR AASHTO M-170/M-206/M-207 M-259/M-273 |
| Corrugated Steel | AASHTO M-190 |
| Corrugated Aluminum | AASHTO M-196/197/219 |
| Corrugated Plastic (HDPE) | ASTM D-1248 |
| Smooth Plastic (PVC) | AWWA C-900, ASTM D-1784/2412/2422/ 2444/2855/3213, AND ASTM F-402 |

The minimum class for RCP pipe shall be Class II. The minimum gauge for metal pipe shall be 12 gauge. All metal pipe shall be invert paved. For all pipe materials, the required pipe strength shall be determined from the actual depth of cover, true load, and proposed field conditions. A typical design strength calculation shall be submitted to the Town for approval.

## 6.3 HYDRAULIC DESIGN

Storm sewers shall be designed to convey the minor storm flood peaks without surcharging the sewer. To ensure that this objective is achieved, the hydraulic and energy grade lines shall 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 in the following sections. The final energy grade line shall be at or below the proposed ground surface.

### 6.3.1 Pipe Friction Losses

The Manning's "n" values to be used in the calculation of storm sewer capacity and velocity are presented in Section 3.5 of this Manual.

### 6.3.2 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:

$$H_L = K\frac{V^2}{2g} \qquad \text{Equation 601}$$

where

$H_L$ = head loss (feet)

$K$ = loss coefficient

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

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

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

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:

$$H_e = K_e \frac{V_1^2}{2g} \left[1 - \frac{A_1}{A_2}\right]^2 \qquad \text{Equation 602}$$

in which A is the cross-section area, V is the average flow velocity, and $K_e$ 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 602 presents the expansion loss coefficients for various flow conditions.

2. Contraction Losses

The form loss due to contraction is:

$$H_c = K_c \frac{V_2^2}{2g} \left[1 - \left(\frac{A_2}{A_1}\right)^2\right]^2 \qquad \text{Equation 603}$$

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 602 presents the contraction loss coefficient for various flow conditions.

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 relation

$$H_b = K_b \frac{V^2}{2g} \qquad \text{Equation 604}$$

in which $K_b$ is the bend coefficient. The bend coefficient has been found to be a function of, (a) the ratio of the 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. A table showing the recommended bend loss coefficients is presented in Table 602.

4. Junction and Manhole Losses

A junction occurs where one or more branch sewers enter a main 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 sewer is calculated from Equation 601. The head loss at a junction can be calculated from:

$$H_j = \frac{V_2^2}{2g} - K_j \left(\frac{V_1^2}{2g}\right) \qquad \text{Equation 605}$$

where $V_2$ is the outfall flow velocity and $V_1$ is the inlet velocity. The loss coefficient, $K_j$, for various junctions is presented in Table 603.

### 6.3.3 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 $K_e$ = 1.0 (refer to Table 602), and for a flared-end section the loss coefficient is approximately 0.5 or less.

### 6.3.4 Partially Full Pipe Flow

When a storm sewer is not flowing full, the sewer acts like an open channel, and the hydraulic properties can be calculated using open channel techniques. For convenience, charts for various pipe shapes have been developed for calculating the hydraulic properties (Figures 601, 602, 603). the data presented assumes that the friction coefficient, Manning's "n" value, does not vary throughout the depth.

## 6.4 VERTICAL ALIGNMENT

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

The minimum clearance between storm sewer and water main, either above or below, shall be 18 inches. Concrete encasement of the water line will be required for clearances of 18 inches or less.

The minimum clearance between storm sewer and sanitary sewer, either above or below, shall also be 18 inches. In addition, when a sanitary sewer main lies above a storm sewer, or within 18 inches below, the sanitary sewer shall have an impervious encasement or be constructed of structural sewer pipe for a minimum of 10 feet on each side of where the storm sewer crosses.

## 6.5 HORIZONTAL ALIGNMENT

Storm sewer alignment may be curvilinear for pipe with diameters of 48 inches or greater but only when approved in writing by the Town. The applicant must demonstrate the need for a curvilinear alignment. Generally, a curvilinear alignment will only be allowed where physical constraints dictate the use of a curvilinear alignment. The limitations on the radius for pulled-joint pipe are dependent on the pipe length and diameter, and amount of opening permitted in the joint. The maximum allowable joint pull shall by 3/4 inch. The minimum parameters for radius type pipe is shown in Table 601. The radius requirements for pipe bend sis dependent upon the manufacturer's specifications.

## 6.6 PIPE SIZE

The minimum allowable pipe size for storm sewers except for detention outlets is dependent upon a practical diameter from the maintenance standpoint. The length of the sewer also affects the maintenance and, therefore, the minimum diameter. Table 601 presents the minimum pipe size for storm sewers.

## 6.7 MANHOLES

Manholes or maintenance access ports will be required whenever there is a change in size, direction, elevation, grade, or where there is a junction of two or more sewers. a manhole may be required at the beginning and/or at the end of the curved section of storm sewer. The maximum spacing between manholes for various pipe sizes shall be in accordance with Table 601. The required manhole size shall be as follows:

MANHOLE SIZE

| SEWER DIAMETER | MANHOLE DIAMETER |
|---|---|
| 15" to 18" | 4' |
| 21" to 42" | 5' |
| 48" to 54" | 6' |
| 60" and larger | Std. Detail SD-6 |

Larger manhole diameters or a junction structure may be required when sewer alignments are not straight through or more than one sewer line goes through the manhole.

## 6.8 DESIGN EXAMPLE

The following calculation example, including the calculation Table 604 and Figure 604, were obtained from Modern Sewer Design, AISI, Washington, D.C., 1980 (Ref. 26) and edited for the calculation of manhole and junction losses in accordance with this Section.

### EXAMPLE 1: HYDRAULIC DESIGN OF STORM SEWERS

Given:
(a) Plan and Profile of storm sewer (Figure 604)
(b) Station 0+00 (outfall) data as follows:

| | | | | |
|---|---|---|---|---|
| Design discharge | Q | = | 145 C.F.S. | [9] |
| Invert of pipe | | = | 94.50' | [2] |
| Diameter | D | = | 66" FCP | [3] |
| Starting water surface | W.S. | = | 100' | [4] |
| Area of pipe | A | = | 23.76 sq. ft. | [6] |
| Velocity = $\frac{Q}{A}$ | V | = | 6.1 f/s | [8] |

Note:
(1) Number in brackets refers to the columns on Table 604.

(2) Sizes of the storm sewer were determined during the preliminary design phases

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

DISCUSSION: The following procedure is based on full-flow pipe 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.

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:

$$¢ \text{ value [7]:} \quad ¢ = \frac{2gn^2}{2.21} = \frac{2 \times 32.2 \times 0.013^2}{2.21}$$

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]:

$$H_V = \frac{V^2}{2g} = \frac{6.1^2}{2 \times 32.2}$$

$$H_V = 0.58$$

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: $S_f$ value [12]:

$$S_f = ¢ \frac{H_V}{R^{4/3}} = \frac{0.00492 \times 0.58}{1.375^{4/3}}$$

NOTE: R = the hydraulic radius of the pipe.

$$S_f = 0.0019$$

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]:

$$H_f = (\text{Avg. } S_f)(L)$$
$$H_f = (0.0019)(110)$$
$$H_f = 0.21$$

STEP 8: Calculate the form losses for bends, junctions, manholes, and transition losses (expansion or contraction) using equations 601, 602, 603, and 605. 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°

$K_b$ = 0.20 (Table 602, Case 1)

$H_b$ = (0.20) (0.58) = 0.12, enter in column 16

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

$$H_e = K_e\ H_{v-1}\ \frac{1-A_1}{A_2}\ 2$$

$K_e$ = 1.06 (Table 802) for $\frac{D_2}{D_1} = 1.5$, and $\theta = 45°$

$$H_L = 1.06 \times 1.29 \times 1 - \frac{15.9}{23.76}\quad 2$$

$$= 0.15,\ \text{enter in column 19}$$

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

$H_m = K_m H_v$

$K_m = 0.05$ (Table 603, Case I)

$H_m = 0.05 \times 1.29 = 0.06$, enter in column 18

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

$H_j = H_{v-2} - K_j \times H_{v-1}$

$K_j = 0.62$ (Table 603, Case III), $\theta = 30°$

$H_j = 1.29 - (.62)\ (0.99) = 0.68$, 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

$K_j = 0.33$ (Table 804, Case III) $= 70°$

$H_j = 0.99 - (0.33)\ (0.64) = 0.78$ for one lateral

STEP 9: Sum all the form losses from columns 15 through 19 and enter in column 20. For the reach between Station 00+0 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+0) to compute the energy grade at the upstream end (Sta. 1+10 for this example).

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

STEP 11: Enter the new invert [2], pipe diameter D[3], pipe shape [5], pipe area A, [6], the compute 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).

$$W.S. = E.G. - H_V$$
$$= 100.79 - 0.58$$
$$= 100.21 \text{ (column 4)}$$

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 804).

DISCUSSION OF RESULTS:

The 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 C.F.S.) at the slope than the design flow (145 C.F.S.). 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" 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 above the top of the pipe due to the losses in the junction. In this case, however, the full flow capacity (100 C.F.S.) 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 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.

## 6.9 CHECKLIST

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

(1) Calculate energy grade line (EGL) and hydraulic grade line (HGL) for all sewers and show on the construction drawings or on a separate copy of the plans submitted with the construction drawings.

(2) Account for all losses in the EGL calculation including outlet, form, bend, manhole, and junction losses.

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

(4) Check for minimum pipe cover; strength of pipe with overburden, surcharge, and dynamic loads; and clearance from utilities.

# DESIGN TABLES & FIGURES

# STORM SEWER ALIGNMENT AND SIZE CRITERIA

| Vertical Dimension of Pipe (Inches) | 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" | 28.50 ft. |
| 57" to 72" | 32.00 ft. |
| 78" to 108" | 38.00 ft. |

Short radius bends shall not be used on sewers 42 inches or less in diameter.

## Minimum Pipe Diameter

| Type | Minimum Equivalent Pipe Diameter | Minimum Cross-sectional Area |
|---|---|---|
| Main Trunk | 18 in. | 1.77 sq. ft. |
| *Lateral from inlet | 15 in. | 1.23 sq. ft. |

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

## Manning's N-Value

| Sewer Type | Capacity Calculation | Velocity Calculation |
|---|---|---|
| Concrete (newer pipe) | .013 | .011 |
| Concrete (older pipe) | .015 | .012 |
| Concrete (preliminary sizing) | .015 | .012 |
| Plastic | .011 | .009 |
| Corrugated Metal | Table 1101 | Table 1101 |

Date:
Rev:

REFERENCE:
"URBAN STORM DRAINAGE CRITERIA MANUAL", DRCOG, 1969

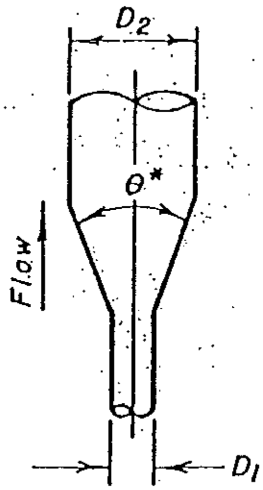
# STORM SEWER ENERGY LOSS COEFFICIENT

(EXPANSION, CONTRACTION)

(a) 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 θ is the angle in degrees between the sides of the tapering section.




(b) Pipe Entrance from Reservoir

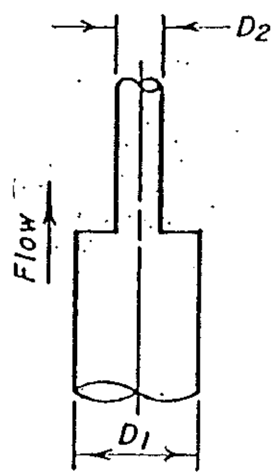
Bell-mouth $H_L = 0.04 \frac{V2}{2g}$

Square-edge $H_L = 0.5 \frac{V2}{2g}$

Groove end U/S For Concrete Pipe $H_L = 0.2 \frac{V^2}{2g}$

(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 |






REFERENCE: Linsley and Franzini "Water Resources Engineering" McGraw-Hill, 1964

# STORM SEWER ENERGY LOSS COEFFICIENT

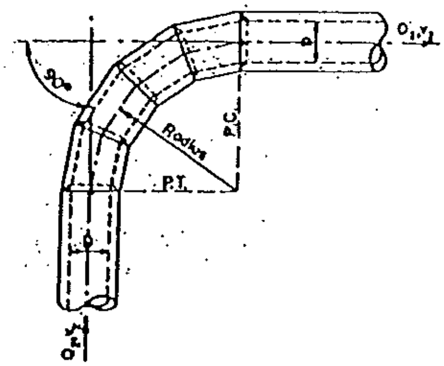
(BENDS)

## CASE I
## CONDUIT ON 90° CURVES

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

$$K_b = 0.25\sqrt{\frac{\theta}{90}}$$

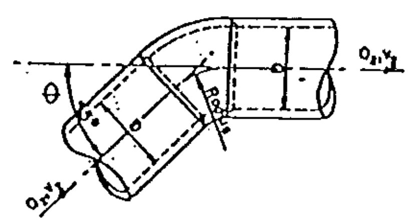
| $\theta$ | $K_b$ |
|---|---|
| 90 | 0.25 |
| 60 | 0.20 |
| 45 | 0.18 |
| 30 | 0.14 |




## CASE II
## BENDS WHERE RADIUS IS
## EQUAL TO DIAMETER OF PIPE

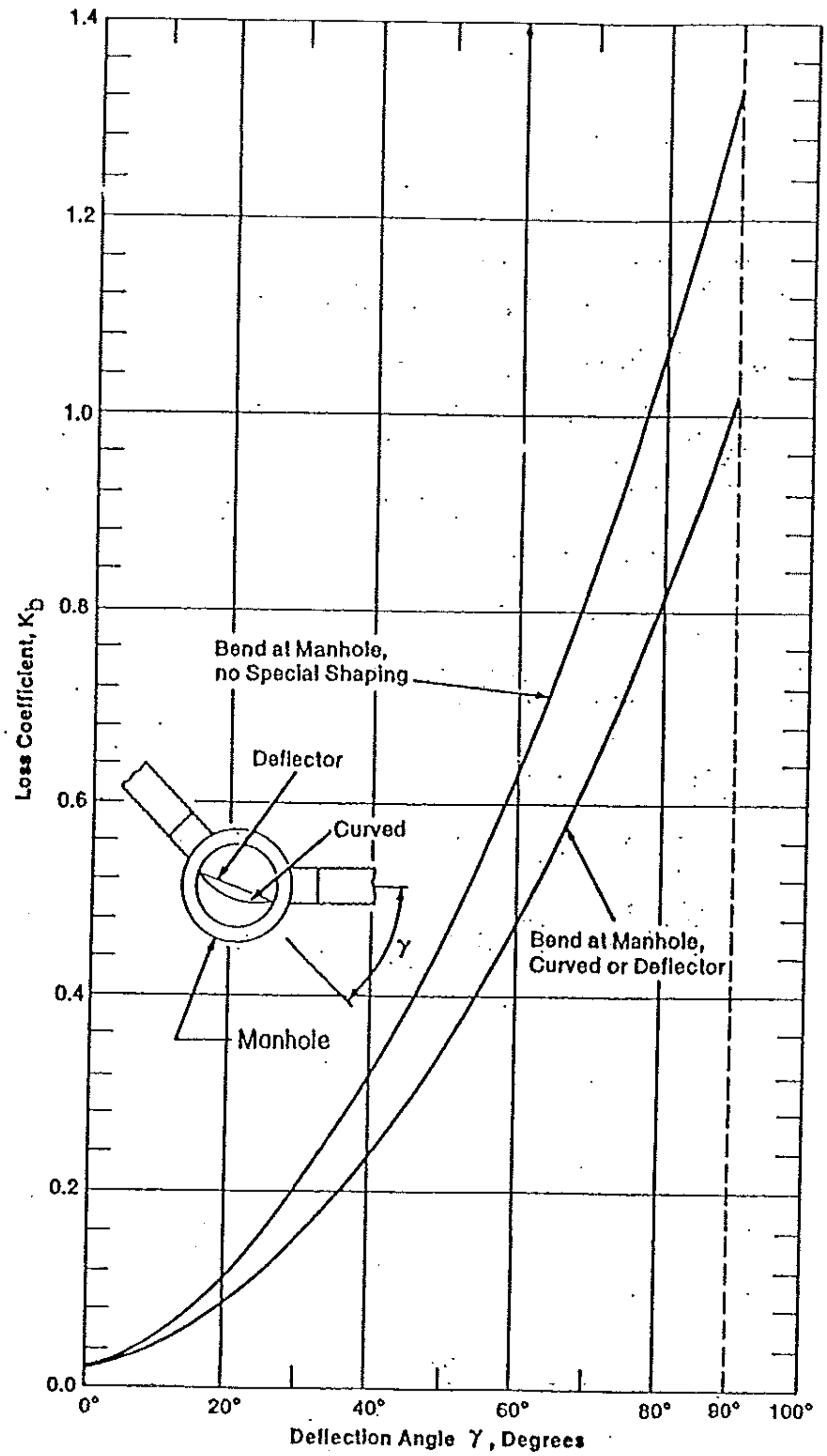
NOTE: Head loss applied at begining of bend

| $\theta$°BEND | $K_b$ |
|---|---|
| 90 | 0.50 |
| 60 | 0.43 |
| 45 | 0.35 |
| 22-1/2 | 0.20 |



# STORM SEWER ENERGY LOSS COEFFICIENT

## (BENDS AT MANHOLES)

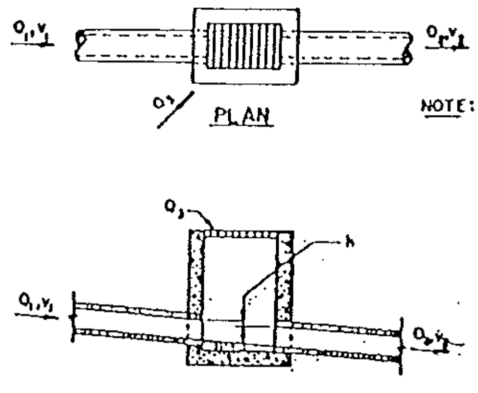



NOTE: Head loss applied at outlet of manhole.

DATE:
REV:

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

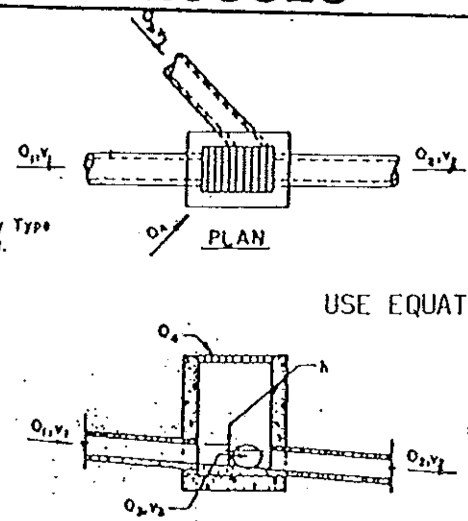
# MANHOLE AND JUNCTION LOSSES




NOTE: For Any Type of Inlet.

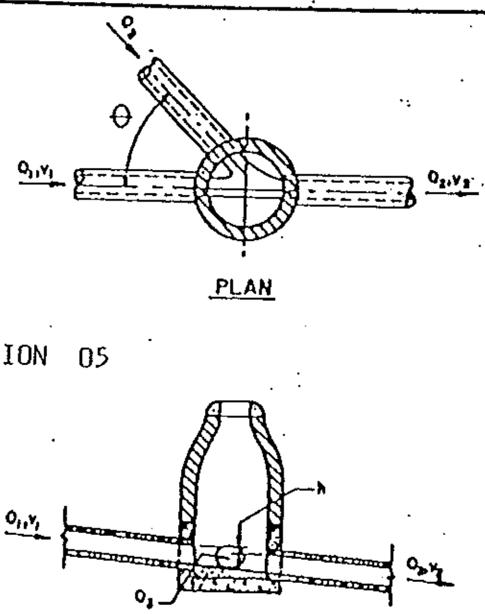
USE EQUATION 01

CASE I
INLET OR STRAIGHT THROUGH
MANHOLE ON MAIN LINE



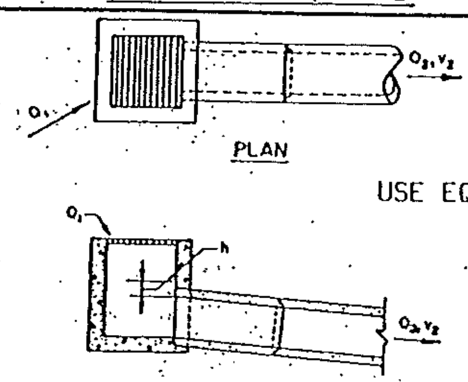

USE EQUATION 05

CASE II
INLET ON MAIN LINE
WITH BRANCH LATERAL




USE EQUATION 05

CASE III
MANHOLE ON MAIN LINE
WITH θ° BRANCH LATERAL




USE EQUATION .01

CASE IV
INLET OR MANHOLE AT
BEGINNING OF LINE

| CASE NO. | $K_j$ |
|---|---|
| I | 0.05 |
| II | 0.25 |
| IV | 1.25 |

CASE III

| θ° | $K_j$ |
|---|---|
| 22-1/2 | 0.75 |
| 45 | 0.50 |
| 60 | 0.35 |
| 90 | 0.25 |
| No Lateral | See Case I |

Date:
Rev:

REFERENCE: APWA Special Report No. 49, 1981

| 1 STA | 2 INVERT | 3 D | 4 W.S. | 5 PIPE SHAPE | 6 A | 7 $\phi$ | 8 V | 9 Q | 10 Hv | 11 E.G. | 12 Sf | 13 AVG. Sf | 14 L | 15 Hf | 16 Hb | 17 HJ | 18 Hm | 19 Ht | 20 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+10 | 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 | - | - | - | 0.20 |
| 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 | | | | | | | | |

TOTAL FRICTION LOSS = 3.43

TOTAL FORM LOSS = 2.75

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

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

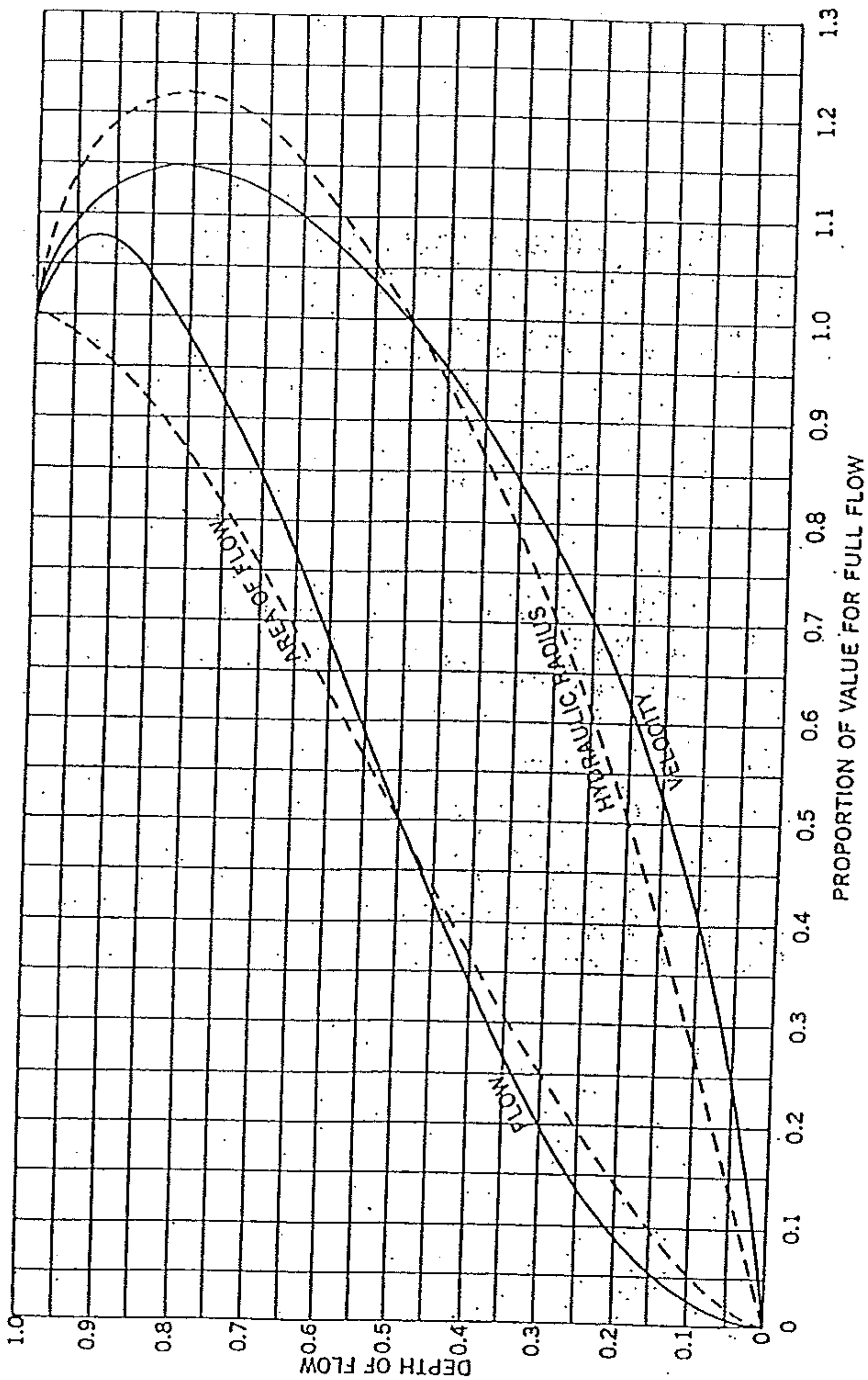
(NOTE: see figure 04A and 04B)

# DESIGN EXAMPLE FOR STORM SEWERS



REFERENCE: MODERN SEWER DESIGN, AISI, WASH DC 1980

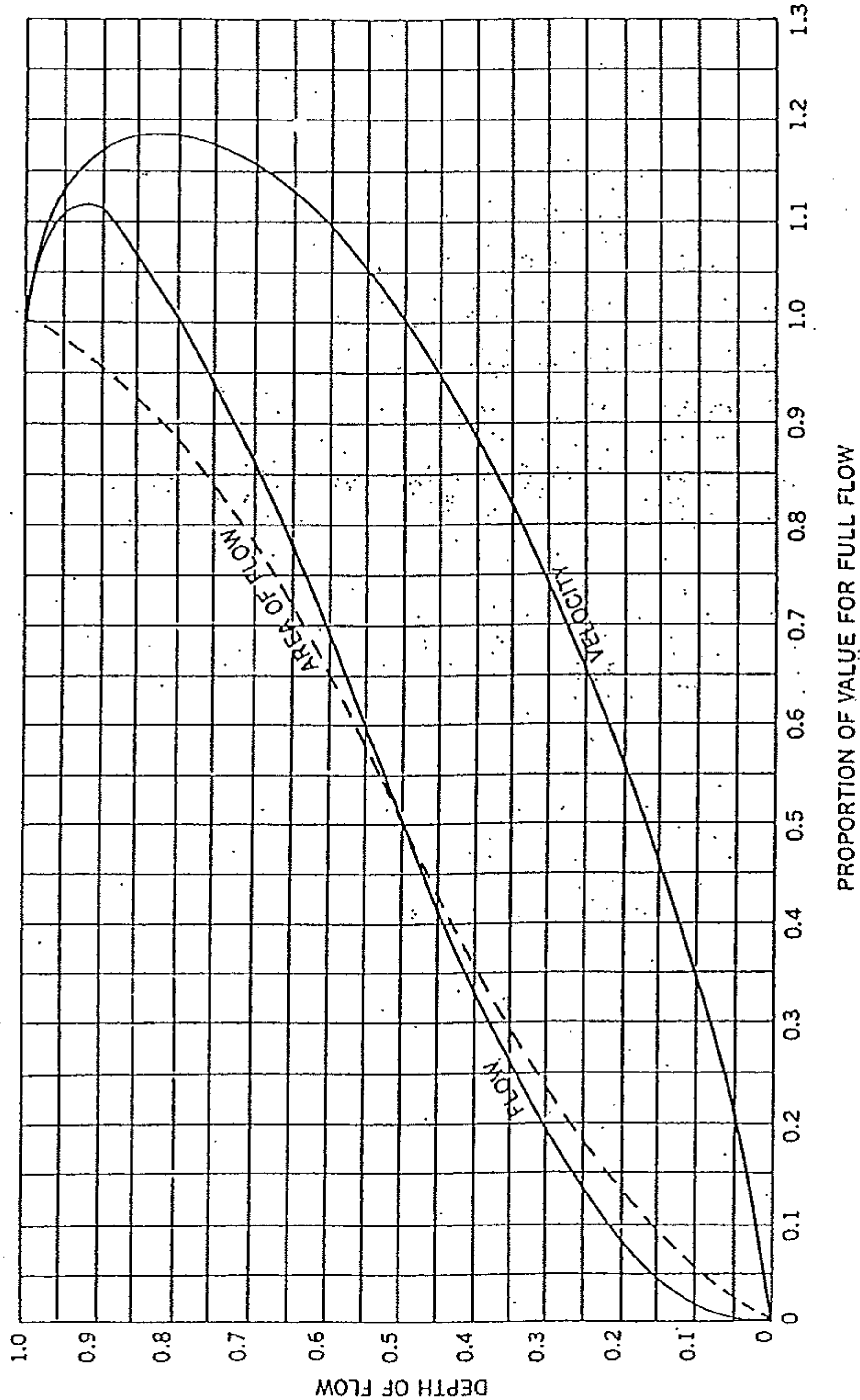
# HYDRAULIC PROPERTIES CIRCULAR PIPE




Date:
Rev:

REFERENCE: "CONCRETE PIPE DESIGN MANUAL" ACPA, 1970

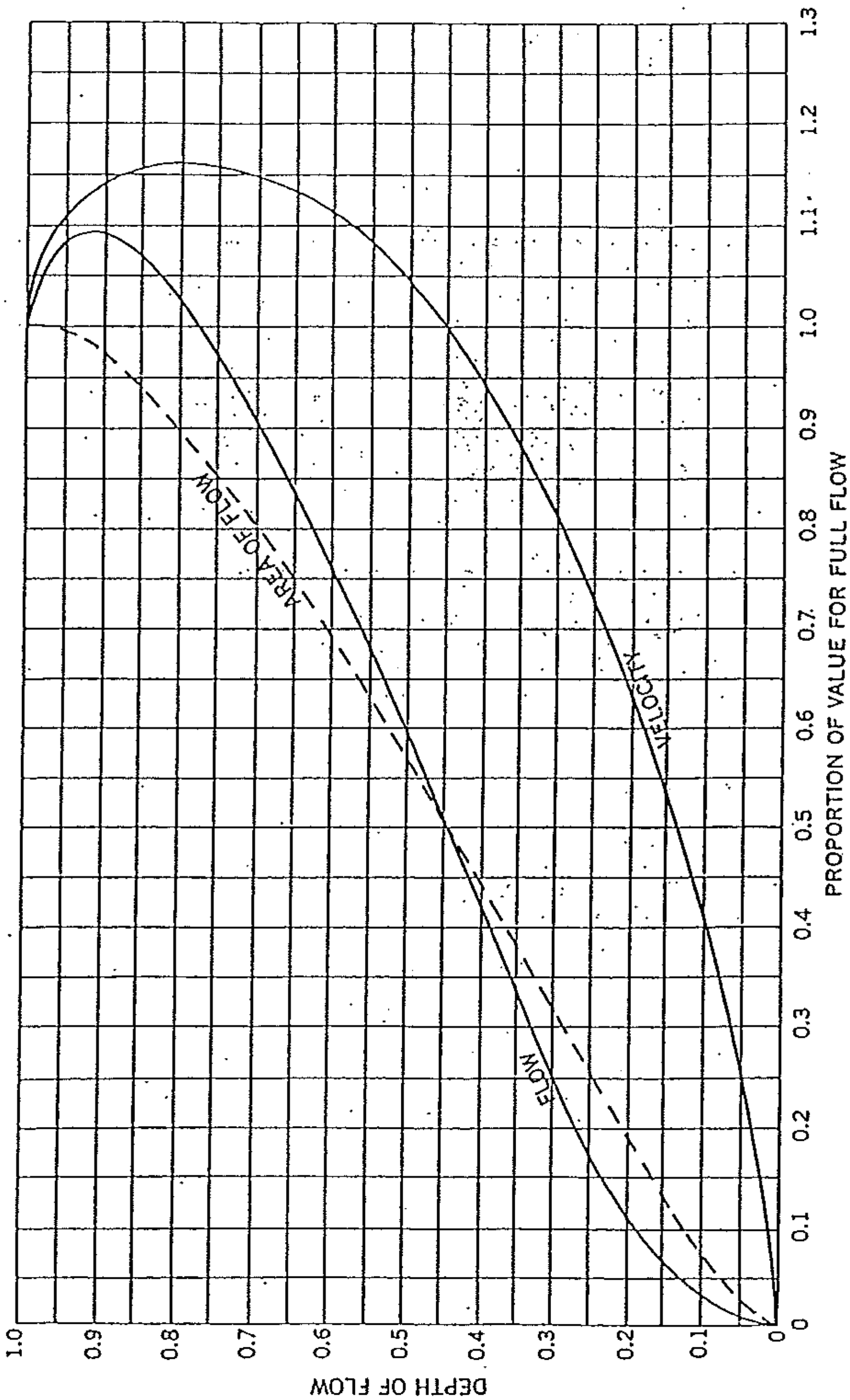
# HYDRAULIC PROPERTIES
# HORIZONTAL ELLIPTICAL PIPE






REFERENCE: "CONCRETE PIPE DESIGN MANUAL" ACPA, 1970

# HYDRAULIC PROPERTIES ARCH PIPE




Date:
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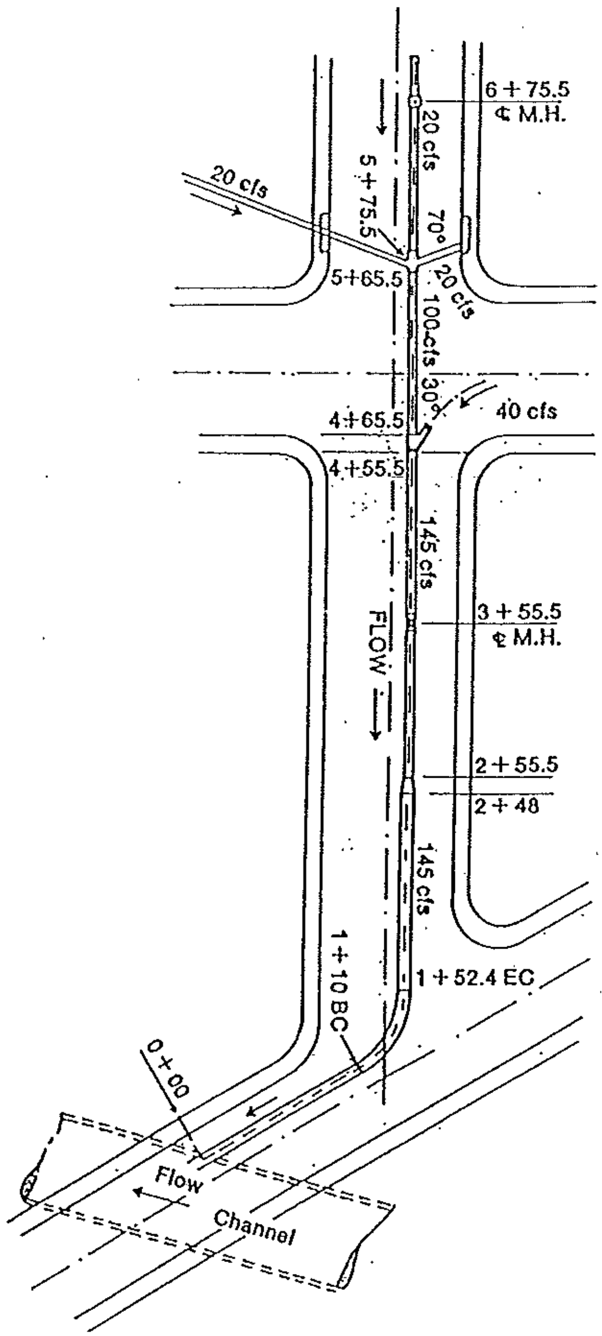
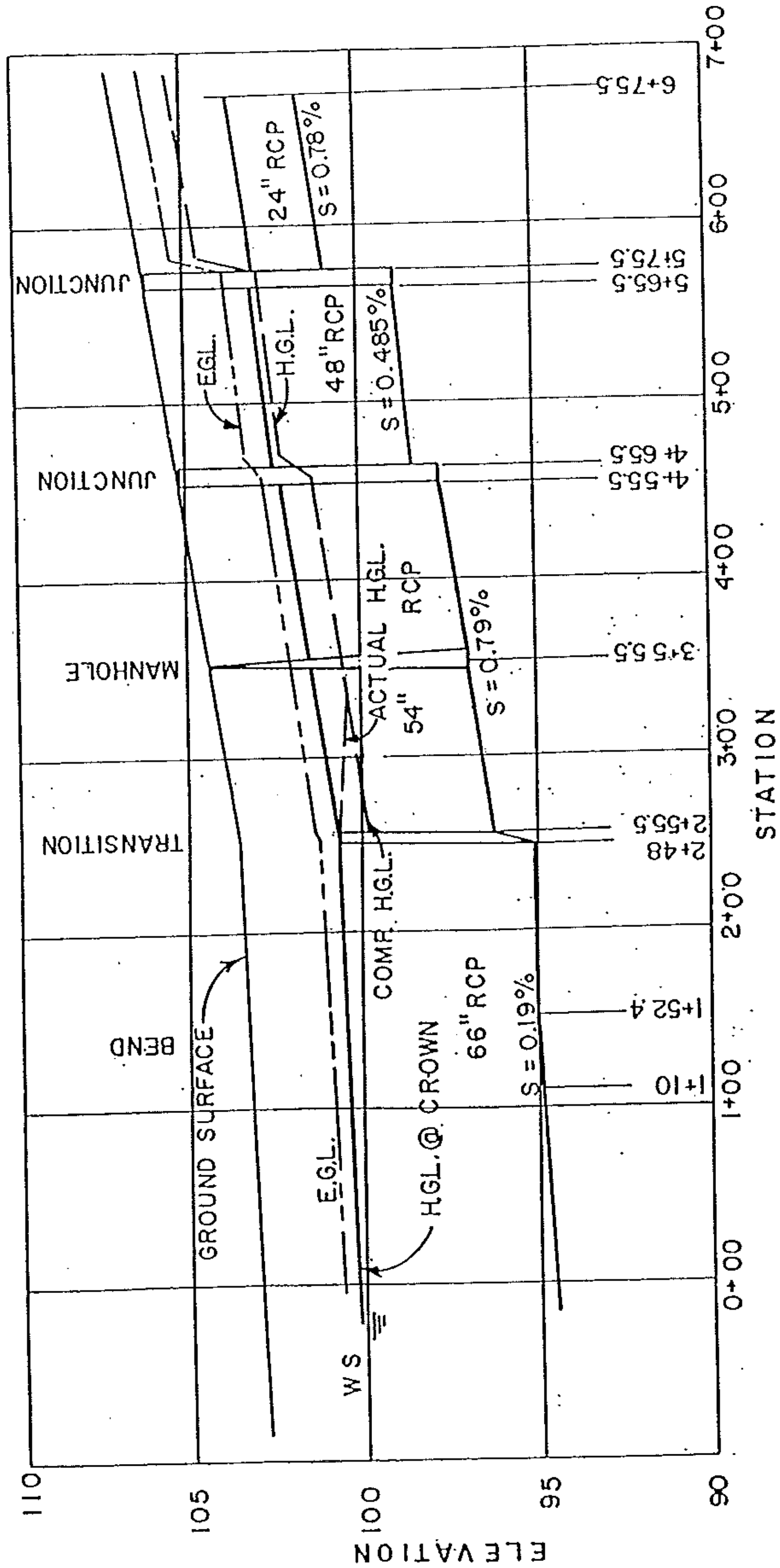
REFERENCE:
CONCRETE PIPE DESIGN MANUAL ACPA, 1970




# DESIGN EXAMPLE FOR STORM SEWERS - PLAN

Date:
Rev:

REFERENCE:
MODERN SEWER DESIGN, AISI, WASH, DC 1980



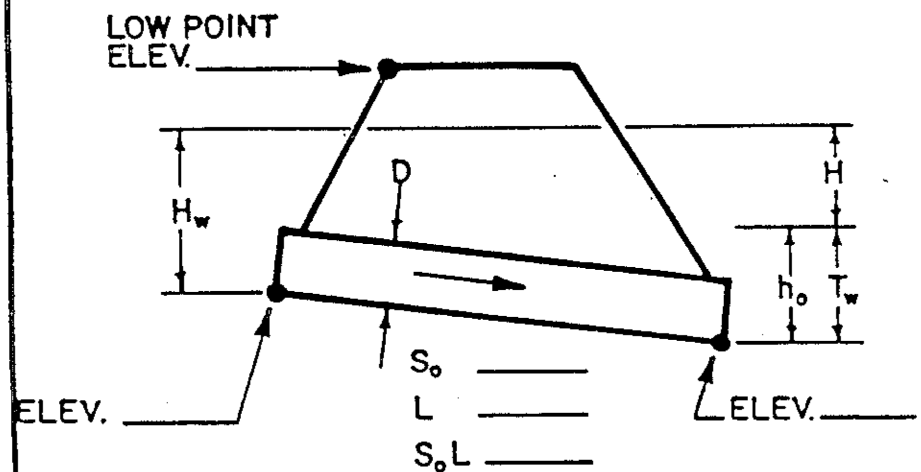

DESIGN EXAMPLE FOR STORM SEWERS – PROFILE



REFERENCE: MODERN SEWER DESIGN, AISI, WASH DC 1980

## CULVERT RATING

PROJECT: ______________________ LOCATION: ____________________ STATION: ________




### CULVERT DATA

TYPE: ____________________ n: ________

INLET: ____________________ $Q_{FULL}$: ________

$K_e$: ____________________ $V_{FULL}$: ________

### OUTLET CONTROL EQUATIONS

(1) $H_w = H + h_o - LS_o$

(2) For $T_w < D$; $h_o = \frac{d_c + D}{2}$ or $T_w$ (whichever is greater)

$T_w > D$; $h_o = T_w$

(3) For Box Culvert: $d_c = 0.315(Q/B)^{2/3} \le D$

| Q | INLET CONTROL | | OUTLET CONTROL | | | | | | CONT. | CONTROL | ELEV. |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | $T_w < D$ | | $T_w > D$ | | | | |
| | $\frac{H_w}{D}$ | $H_w$ | H | $T_w$ | $d_c$ | $\frac{d_c+D}{2}=h_o$ | $h_o$ | $H_w$ | $H_w$ | | |
| 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 |
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