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# 5.0 STORM SEWERS AND OVERLAND FLOW

## 5.1 GENERAL

Due to the flat terrain in Fort Bend County, it is infeasible in certain areas to convey the runoff from extreme rainfall events entirely via an underground storm sewer system. Local flooding will occur in areas away from the primary drainage channels because it is simply uneconomical to provide a storm sewer pipe large enough to totally carry the infrequent, severe storm events. For this reason, a sheet flow analysis is required so that street design and alignment assure that excess runoff from extreme storm events will be conveyed to primary drainage channels safely. The development or project is not allowed to increase flows into the receiving channel, ditch or drainage system without sufficient mitigation and supporting calculations. Sheet flow corridors and extreme event swales shall be designated and all required right-of-way shall be established. Special consideration must also be given for off-site sheet flows and their impacts on the planned subdivision.

The discussion presented in this section will be directed primarily at curb-and-gutter streets with underground storm sewers. Roadside ditch systems are acceptable in certain instances, but are not preferred.

## 5.2 RUNOFF ANALYSES

### 5.2.1 Frequency Considerations

Flooding in Fort Bend County is generally associated with one of two types of severe rainfall events. The first type is a localized high intensity rainfall of short duration which floods a small localized area causing ponding of water and interruption of traffic flow. The second type is a more generalized rainfall of longer duration, which can cause more widespread flooding and can result in severe damage and loss of life. This second of storm event is generally used to design channels for drainage large areas.

In designing storm sewers for draining small developments, it is the localized high intensity, short duration rainfall event which is used. However, since these storm sewers usually drain into open channels, which are used to convey the runoff from larger areas, the design must take into consideration the interaction of these two systems.

Figure 5-1 illustrates the effect on the hydraulic grade line of a storm sewer for three outlet conditions. Assuming the outlet channel is at its 25-year water level, it can be seen from Part A of Figure 5-1 that the hydraulic grade line for the standard design condition remains at or below the gutter level at the furthest inlet. For this condition, there is no street ponding and the storm sewers are functioning at or below their design capacity.

Parts B and C of the Figure show the case where the tailwater condition is above the design level. Street ponding begins to occur throughout the storm sewer drainage system, as the storm sewers are unable to operate at their design capacity. This local flooding situation could also occur when the tailwater is below design conditions if local rainfall is in excess of that used in the design of the storm sewer system. As this widespread street ponding starts to occur, provisions must be made to limit the depth of ponding to a level below that which will cause significant property damage. In general, flood elevations shall be considered unacceptable when they exceed the lowest of the following: 1) one foot over natural ground; 2) one foot over top of curb; or 3) one foot below the lowest slab elevation.

### 5.2.2 General Design Guidelines

Storm sewers shall be designed to carry the design storm peak flow (See Section 5.2.3 for design frequency). To obtain the design storm peak flow, the Rational Method can be used for drainage areas less than 200 acres. For areas from 200 to 2000 acres, the discharge curves can be used to obtain the peak flows. To obtain peak flow for larger drainage areas, hydrologic modeling with HEC-HMS should be used. A detailed description of these techniques is contained in Section 2.0 of this manual.

The grading of the development or lots shall conform to the project plans. The grading of the site or lots shall be from the back of the lots or development to the front of the lot or to an applicable designated drainage system that has been designed to convey the project flows.

When filling lots adjacent to a channel, a transition of the back of the lot to natural ground at the channel right-of-way must occur so as to not hinder maintenance operations within the channel right-of-way. The design should also minimize the amount of the back of lot draining directly to the channel right-of-way. Review of the preliminary design by the Fort Bend County Drainage District Engineer should be obtained before any detailed engineering is performed.

For all storm sewer systems or for enclosing an existing open channel, the hydraulic calculations and hydraulic profiles along with the construction plans of the closed-conduit system must be submitted to the Fort Bend County Drainage District Engineer for review.

A preliminary design should first be performed utilizing the design storm and the Rational Method and partially summarized in Table 5-1. Then, if necessary, adjust the sizes of the pipes or boxes to meet the required hydraulic grade line criteria outlined in Section 5.2.3 which follows.

Generally, no more than one storm sewer outfall per 1000 feet of channel or one outfall per smaller tract will be allowed on each side of the receiving channel, detention basin or waterway.

### 5.2.3 Specific Design Flow Frequency Criteria

The recommended design flow frequency criteria to be used for continuous closed-conduit systems are given below:

1. For all drainage areas, the design flows shall be determined utilizing the Rational Method and storm sewer curves (See Figure 5-2) shown in Table 5-1 as a minimum. The conduit shall be designed in accordance with methodology as outlined in Section 5.3.2.

2. For portions of the system serving areas between 100 acres and 200 acres, it is additionally required that the 25-year hydraulic grade line be at or below the gutter line for the portion of the system which drains 100 or more acres. For this computation, the 25-year discharge for fully developed conditions based on the Rational Method (See Section 2.4) should be used. A 25-year design water surface should be assumed in the outfall channel.

3. For portions of the system serving an area larger than 200 acres, the 100-year flow for fully developed conditions should be used (based on the hydrological modeling using HEC-HMS) to insure that the 100-year hydraulic grade line will be below the natural ground elevation at all points along this portion of the closed system. A 25-year design water surface should be assumed in the outfall channel.

4. For systems designed in accordance with (2) or (3), sufficient additional inlet capacity shall be provided to allow for entry into the closed-conduit system of runoff in excess of the runoff conveyed through the storm sewer system up to the design capacity of the closed-conduit system.

5. For all areas, overland flow shall be considered as discussed in Section 5.4.

6. Closed systems adjoined to an upstream open channel shall be designed for the 100-year ultimate discharge.

## 5.3 STORM SEWERS

### 5.3.1 Design Criteria

The following specific criteria and requirements shall apply to the design and construction of storm sewers in Fort Bend County. The following criteria were taken primarily from General Design Requirements for Sanitary Sewers, Storm Sewers, Water Lines, and Paving, City of Houston (1983 or latest version).

1. Calculation of the hydraulic grade line for design conditions in a specific branch of storm sewer shall proceed upstream from the level of the 25-year water surface elevation in the outfall channel.

2. The minimum diameter of a pipe in a sewer line shall be 24”.

3. The Manning's "n" value to be used in a reinforced concrete pipe storm sewer shall be 0.013. For corrugated metal pipe, the "n" value shall be as shown in Table 5-2.

4. The minimum velocity of flow to be allowed in a section of storm sewer flowing full shall be 3 fps. The maximum velocity shall be 10 fps.

5. Provisions must be made for all adjacent undeveloped areas with natural drainage patters directing overland flow into and across planned development.

6. Before a particular storm sewer design will be reviewed, the following items must be presented:

   a. A contour and drainage area map showing all pertinent subareas, including contribution off-site areas.

   b. A listing of all relevant hydrologic design flow calculations, which shall include all contributing off-site flows.

   c. Calculations for determining the hydraulic gradient, along with a profile of its location.

   d. A plan showing the location of all manholes and inlets, and the alignment of all storm sewers in the right-of-way.

   e. A profile showing the placement of storm sewers and the location of all pipe size changes, grade changes, and pipe intersections.

7. All storm sewers and appurtenant construction shall conform to the City of Houston Department of Public Works publication Specifications for Sewer Construction, Form E-14-62, City of Houston Drawing Nos. 529-S-1, 530-S-1, 530-S-2, and all subsequent revisions, or approved equal. All outfalls into ditches, channels, streams or detention ponds shall include the use of erosion protection in accordance with Figures 3-4 and 3-5.

8. All storm sewers shall be constructed with reinforced concrete pipe, or approved equal. Corrugated galvanized metal pipe, or other approved equal, may be used only at the storm sewer outfall into unlined channels. The use of polymer or other approved coatings for outfall pipes is required. Submit applicable coating information to the FBCDD Engineer for consideration.

9. All cast-in-place concrete storm sewers shall follow the alignment of the right-of-way or easement.

10. All pre-cast concrete pipe storm sewers shall be typically designed in a straight line or shall conform to the Texas Department of Transportation Specifications and all subsequent revisions or approved equal.

11. All storm sewer inlet leads shall be designed in a straight line alignment.

12. Storm sewers shall be located in public street rights-of-way or in easements that will not prohibit future maintenance access.

13. In most cases where easements are restricted to storm sewers, the pipe should be centered within the limits of the easement.

14. For all storm sewers having a cross-sectional area equivalent to a forty-two inch (42”) inside diameter pipe or larger, soil borings with logs shall be made along the alignment of the storm sewer at intervals not to exceed five hundred feet (500’) and to a depth not less than three feet (3’) below the flowline of the sewer. The required bedding of the storm sewer as determined from these soil borings shall be shown in the profile of each respective storm sewer. The design engineer shall inspect the open trench and may authorize changes in the bedding indicated on the plans. Such changes shall be shown on the record drawings and, along with soil boring logs, submitted to the County Drainage District Office. All bedding shall be constructed as specified in the Texas Department of Transportation Specifications and all subsequent revisions, or approved equal.

15. All storm sewer outfalls shall conform to the requirements and specifications defined in Section 3.0, Open Channel Flow, and Figure 3-5.

5.3.2 General Design Methodology

It is recommended that design of a storm sewer system proceed as follows:

1. Determine the 25-year water surface elevation in the channel at the storm sewer outfall using appropriate backwater calculations.

2. Determine the design flow rates for all sections of storm sewer based on drainage area size.

3. Assuming storm sewer pipes are full at design flows, determine the appropriate sizes for all sections of storm sewer using Manning's equation and assuming uniform flow conditions.

4. Begin calculation at the 25-year water surface elevation in the outfall channel and plot the hydraulic gradient for the design storm. Include all relevant energy losses. The hydraulic gradient must not exceed the roadway gutter flowline elevation.

5.3.3 Head Losses

Head losses at structures shall be determined for inlets and manholes in the design of closed conduits. The design engineer should determine the relative significance of the minor losses and their applicability to the design. If they are insignificant, they may be omitted.

5.3.3.1 Head Losses at Structures

The equation for the head loss (feet) at an inlet or manhole is as follows:

$$\text{Head loss} = \frac{V_2^{\,2} - K V_1^{\,2}}{2g} \qquad (5\text{-}1)$$

where

$V_1$ = velocity in the upstream pipe (fps).

$V_2$ = velocity in the downstream pipe (fps).

K = junction or structure coefficient of loss. (See Table 5-3)

5.3.3.2 Entrance Losses

A special case of sudden contraction is the entrance loss for pipes. The equation for head loss at the entrance to a pipe is given as follows:

$$\text{head loss} = K\frac{V^2}{2g} \qquad (5\text{-}2)$$

where

K = entrance loss coefficient. (See Table 5-4.)

V = flow velocity in pipe (fps).

## 5.3.4 Manholes

Manholes shall be placed at the location of all pipe size or cross section changes, pipe sewer intersections or P.I.'s, pipe sewer grade changes, street intersections, at maximum intervals of 500 feet measured along the centerline of the pipe sewer; and at all inlet lead intersections with the pipe sewer where precise concrete pipe sewers are designed.

## 5.3.5 Inlets

Three types of inlets are recommended for use in Fort Bend County, the Type "BB" Inlet, Type "C-1" Inlet and the Type "H-2" Inlet (as identified by the City of Houston). All inlets shall be constructed as specified in the Fort Bend County Design Standards and Details, or approved equal.

5.3.5.1 Inlet Capacity

The capacity of inlets shall be determined as shown in Step 7 of Section 5.4.3 of this manual. All inlets shall be designed to carry at least the design storm frequency runoff.

5.3.5.2 Inlet Spacing

Curb inlets must be spaced to handle the design storm discharge so that the hydraulic gradient does not exceed the roadway gutter elevation. Inlets shall be spaced so that the maximum travel distance of water in the gutter will not exceed six hundred feet (600') one way for residential streets and three hundred feet (300') one way on major thoroughfares and streets within commercial developments. Curb inlets shall be located on intersection side streets to major thoroughfares for all original designs or developments. Special conditions warranting other locations of inlets shall be determined on a case-by-case basis.

## 5.4 STREET DRAINAGE OF STORM SEWER OVERFLOW

When the capacity of the underground system is exceeded and street ponding begins to occur, careful planning can reduce or eliminate the flood hazard for adjacent properties. Street layout and pavement grades along with extreme event swales are the key components in developing a successful system which can convey the storm sewer overflows to the outfall channel designed to carry the 100-year storm runoff. The following design methodology and example is derived from typical criteria in Harris County.

### 5.4.1 Land Plan and Street Layout

Designing an effective internal system must begin with the land plan and street layout. Awareness of overland flow problems in this early phase of the development process can reduce costly revisions and delays later on in the project. When designing drainage systems, attention needs to be given to special problems created by the topography. Excessive street cuts which can create ponding levels that hamper vehicle access and/or present a flood hazard must be avoided.

Some examples of undesirable sheet flow patterns are depicted in Figure 5-3 and include:

a) Cul-de-sac streets sloping downhill designed so that sheet flow can only escape through building lots.

b) The placing of a curve or turn in a roadway in a low area so that sheet flow into that curve or turn can escape only through existing building lots.

c) Many streets “T”ing into one street which is lower than the intercepting streets so that sheet flow down the streets can escape only through existing building lots.

Proper engineering foresight in the design of items such as emergency relief swales or underground systems can solve these potential problems. Some examples of acceptable sheet flow patterns are shown in Figure 5-4.

The maximum allowable ponding level for a new street is the lowest of the following: (1) one foot above natural ground; (2) one foot above top of curb; or (3) one foot below the lowest slab elevation. The design engineer must check to see if the storm drainage system can convey flows from a 100-year storm event without ponding water in the street at levels that exceed the maximum allowable level. The 100-year discharge can be obtained by following the procedures outlined in Section 2.4. A 25-year tailwater condition should be assumed in the outlet channel. If the maximum level would be exceeded, the engineer must analyze the route along the street system that will convey the overflows to the major drainage channel and verify that the overflows will not exceed the maximum allowable ponding level. In making this analysis, the engineer can account for the portion of flows that would be carried by the sewer system in addition to the street system, assuming a 25-year tail-water condition. The engineer must also verify that there are no increases in flows into the receiving channel. These increases are not allowed without appropriate mitigation and approval by the Fort Bend County Drainage District.

### 5.4.2 Conveyance of Surface flow to Primary Channels

Once it has been determined that ponding levels are excessive and where the collective sheet flow is going to go, provisions must be made to get the overflows into the appropriate drainage channel. This may be done through the use of additional pipe capacity and inlets or by using a surface swale.

The surface flow conveyance system shall be contained within an easement dedicated to the Drainage District. The easement shall be of sufficient width to operate and maintain the system.

Since a surface swale system would act only under emergency conditions and would not function under normal circumstances, all precautions must be taken to insure that the relief system will function when needed. The recommended design procedure for sizing storm sewers for sheet flow conveyance is presented in Section 5.4.3. The design procedure recommended for sizing of the surface swale is similar to the procedure for the pipe outfall as described in Section 5.4.3. First, the appropriate values from steps one and two are computed, then the required extreme event swale cross-section is determined by normal depth calculations, sizing the swale such that an acceptable water surface is achieved.

### 5.4.3 Design Procedure for Pipe Outlet

This section outlines the procedure recommended for designing an underground pipe system to convey overflows to a primary drainage channel. Because the majority of subdivisions in Fort Bend County are designed with curb-and-gutter streets, modification of the last storm sewer reach is generally all that is necessary to handle the overflow.

The recommended procedure is given below along with an example based on the drainage system presented in Figure 5-3 (c).

1. Determine the 100-year peak flow at the point of concentration from all existing and future contributing drainage areas for 100% development conditions. In the example, the contributing drainage area is 40 acres and the 100-year discharge is 147 cfs.

2. Determine the 25-year frequency water-surface elevation in the drainage channel at the pipe outfall point. Based on a 25-year backwater profile, the water surface elevation in the channel for the example is 97.0 feet.

3. Determine the maximum energy head, H, available between the outfall point and ponding area by subtracting the maximum allowable ponding elevation in the ponding area from the channel's 25-year water surface elevation.

   With a slab elevation of 101.5 feet and a top of curb and natural ground elevation of 100.0 feet in this example, the maximum allowable ponding elevation is the lowest of the follows: 1) one foot over natural ground; 2) one foot over the top of curb; or 3) one foot below the lowest floor elevation. In this case, the maximum elevation is controlled by the lowest floor elevation and is 100.5 feet. There is 3.5 feet of head available (H).

4. Establish a size of the storm sewer pipe and compute the head loss using the following equation:

$$h_p = \frac{4.66 Q^2 n^2 L}{D^{16/3}} \quad (5\text{-}3)$$

where

$h_p$ = Head loss in feet

Q = 100-year discharge in cubic feet per second

n = Manning's "n" value

D = Diameter of pipe in feet

L = Length of pipe in feet

For this example, 65 linear feet of 60-inch corrugated metal pipe (cmp) with a Manning's "N" value of 0.024 and 120 linear feet of 60-inch reinforced concrete pipe (rcp) with a Manning's "n" value of 0.013 was selected. The head loss is as follows:

$$h_p = \frac{4.66 (Q^2)}{D^{16/3}} \left(n^2_{cmp} L_{cmp} + n^2_{rcp} L_{rcp}\right)$$

$$h_p = \frac{4.66 (147)^2}{5^{16/3}} \left( (0.024)^2 (65) + (0.013)^2 (120) \right)$$

$$= 1.09 \text{ feet}$$

5. Compute the head loss through the leads, $h_L$, using Equation 5-3. Experience has shown that the 24-inch diameter leads generally cause excessive head loss. The 30-inch diameter leads are satisfactory in most cases, while the 36-inch leads are too large for the most common street inlets type "B-B" and "C-1". Therefore, the 30-inch diameter was selected.

Estimate the percentage of 100-year runoff flowing through each lead.

Assume the 147 cfs to be divided between three leads as follows:

Lead 1 20-foot lead with a flow of 56 cfs.

Lead 2 20-foot lead with a flow of 56 cfs.

Lead 3 45-foot lead with a flow of 37 cfs.

$$h_{L_1} = h_{L_2} = \frac{4.66\, n^2 Q^2 L}{D^{16/3}} = \frac{4.66\,(0.013)^2\,(56)^2\,(20)}{2.5^{16/3}}$$

$$= 0.37 \text{ foot}$$

$$h_{L_3} = \frac{4.66\,(0.013)^2\,(37)^2\,(45)}{2.5^{16/3}}$$

$$= 0.37 \text{ foot}$$

6. Determine the energy head available at each inlet using the equation:

$$h_i = H = h_p - h_L \qquad (5\text{-}4)$$

If $h_i$ is negative, the hydraulic grade line is above the maximum ponding elevation. Increase the capacity of the system and repeat steps 4, 5, and 6.

If $h_i$ is positive, check the elevation of the hydraulic grade line relative to the maximum ponding elevation. For grade lines above the gutter line, use $h_i$ as the energy head on the inlet; otherwise, make the value of $h_i$ equal to the maximum ponding elevation minus the gutter elevation.

For this example, assume the hydraulic grade line is above the gutter elevation. Since the head loss through the three leads in the example are similar, the available head at each inlet is:

$$h_i = 3.5\text{-}1.1\text{-}0.37 = 2.03$$

7. Determine the type of inlets required to handle the portion of the 100-year flow reaching the ponding area. The flow through the inlet(s) must be equal to or

greater than the flows estimated in Step 5 for each lead. Use the following orifice equation to compute the flow into each inlet.

$$Q = CA \quad (2gh_i)^{1/2} \tag{5-5}$$

Where

Q = discharge in cubic feet per second

C = orifice coefficient (0.8 for inlets)

A = area of inlet opening. (Type “B-B” = 2.14 square feet and Type “C-1” = 6.50 square feet.)

G = acceleration of gravity (32.2 ft/sec$^3$)

$h_i$ = as defined in Step 6

Type “C-1” inlets are selected for Inlet 1 and Inlet 2 and
Type “B-B” inlets are selected for Inlet 3 across the street.

$$Q_{C\text{-}1} = 0.8(6.50) \quad (64.4\ (2.0)\ )^{1/2} = 59 \text{ cfs}$$

$$2Q_{B\text{-}B} = 2\ (0.8(2.14) \quad (64.4\ (2.0)^{1/2}) = 38 \text{ cfs}$$

Thus, it is shown that a Type “C-1” inlet at Inlet 1, a Type “C-1” inlet at Inlet 2, and two Type “B-B” inlets at Inlet 3 will convey the 100-year sheet flow to the channel with the energy head available. If this inlet choice is adequate, the design is complete.

8. Repeat Steps 4 through 7 until the combination of storm sewer pipe, leads, and inlets adequately conveys the 100-year sheet flow to the channel with the energy head available, and is the most economical.

### 5.4.4 Design Procedure for Extreme Event Swale

Design calculations shall be provided that substantiate the design elevation of the extreme event swale and the needed design requirements of the extreme event swale. The design and construction of the extreme event swale shall be consistent with the needed capacity at each location.

The extreme event swale within the right-of-way of detention ponds and outfall channels shall have a minimum 6-foot bottom width and 6:1 side slopes. The swale will be designed to use interlocking concrete blocks or concrete slope paving to protect from erosion.

5.4.5 Roadside Ditch Drainage

Under certain conditions, roadside ditch drainage is acceptable as an alternative to curb-and-gutter systems. However, a similar potential for flooding exists when flow in roadside ditches exceeds capacity. Provisions must be made to assure that the amount of water ponded behind an elevated roadway does not reach damaging levels. Projects or developments that drain to roadside ditches are only allocated a pro rata share of the existing ditch capacity. (See Figure 5-5 for typical roadside ditch drain detail.)

5.4.5.1 Preliminary Approval

Preliminary approval for the use of roadside ditch systems must be obtained from the Fort Bend County Drainage District Engineer prior to the submittal of contour and drainage area maps, and hydrologic and hydraulic calculations.

5.4.5.2 Design Criteria

The following requirements taken from the General Design Requirements for Sanitary Sewers, Storm Sewers, Water Lines and Paving, City of Houston (1983 or latest version) must also be met in the design of roadside ditch systems in Fort Bend County:

1. The design flow shall be determined based on the projected land use and the rainfall runoff curves from Figure 5-2.

2. Minimum acceptable ditch section shall have a side slope no steeper than 4 horizontal to 1 vertical.

3. The minimum bottom width for roadside ditches shall be two feet.

4. The “n” coefficient for the ditch calculations shall be a minimum of 0.04. All values must be justified.

5. The minimum grade or slope of the ditches shall be 0.10%.

6. Hydraulic design computations must be submitted for each drainage ditch system. Computations shall include the effect of future driveway culverts, which shall be sized taking into account design flow and ditch depth.

7. The computed water surface of the ditches shall be a minimum of 0.5 feet below finished grade elevations along the street edge of pavement.

8. The entire ditch must be revegetated immediately after construction to minimize erosion.

9. Erosion control methods shall be utilized in the ditch designs where velocities of flow are calculated to be greater than five feet per second or where soil conditions dictate their need.

10. The minimum depth of the ditches shall be 18 inches and the maximum depth shall be 4 feet.

TABLE 5-1

RAINFALL RUNOFF CURVES FOR
FORT BEND COUNTY

| Time of Concentration (minutes) | Ci Values for Curve No. | | | | | |
|---|---|---|---|---|---|---|
| | 1 | 2 | 3 | 4 | 5 | 6 |
| 10 | 2.06 | 1.72 | 1.38 | 1.15 | .92 | .58 |
| 11 | 2.03 | 1.70 | 1.36 | 1.13 | .91 | .57 |
| 12 | 2.00 | 1.67 | 1.34 | 1.11 | .90 | .56 |
| 13 | 1.97 | 1.65 | 1.32 | 1.10 | .88 | .55 |
| 14 | 1.95 | 1.63 | 1.31 | 1.09 | .87 | .55 |
| 15 | 1.92 | 1.61 | 1.29 | 1.07 | .86 | .54 |
| 16 | 1.91 | 1.59 | 1.28 | 1.06 | .85 | .54 |
| 17 | 1.89 | 1.57 | 1.27 | 1.05 | .84 | .53 |
| 18 | 1.87 | 1.56 | 1.25 | 1.04 | .83 | .52 |
| 19 | 1.85 | 1.55 | 1.24 | 1.03 | .83 | .52 |
| 20 | 1.84 | 1.54 | 1.23 | 1.02 | .82 | .51 |
| 21 | 1.82 | 1.53 | 1.22 | 1.02 | .82 | .51 |
| 22 | 1.80 | 1.51 | 1.21 | 1.01 | .81 | .51 |
| 23 | 1.79 | 1.50 | 1.20 | 1.00 | .80 | .50 |
| 24 | 1.78 | 1.49 | 1.19 | 1.00 | .80 | .50 |
| 25 | 1.77 | 1.48 | 1.18 | 1.00 | .80 | .50 |
| 26 | 1.75 | 1.47 | 1.17 | 1.00 | .80 | .50 |
| 27 | 1.75 | 1.46 | 1.17 | 1.00 | .80 | .50 |
| 28 | 1.74 | 1.45 | 1.16 | 1.00 | .80 | .50 |
| 29 | 1.73 | 1.44 | 1.16 | 1.00 | .80 | .50 |
| 30 | 1.72 | 1.43 | 1.15 | 1.00 | .80 | .50 |
| 31 | 1.71 | 1.43 | 1.15 | 1.00 | .80 | .50 |
| 32 | 1.70 | 1.42 | 1.14 | 1.00 | .80 | .50 |
| 33 | 1.69 | 1.41 | 1.13 | 1.00 | .80 | .50 |
| 34 | 1.68 | 1.40 | 1.13 | 1.00 | .80 | .50 |
| 35 | 1.67 | 1.40 | 1.12 | 1.00 | .80 | .50 |
| 36 | 1.66 | 1.39 | 1.12 | 1.00 | .80 | .50 |
| 37 | 1.65 | 1.38 | 1.11 | 1.00 | .80 | .50 |
| 38 | 1.65 | 1.38 | 1.11 | 1.00 | .80 | .50 |
| 39 | 1.64 | 1.37 | 1.10 | 1.00 | .80 | .50 |
| 40 | 1.64 | 1.37 | 1.10 | 1.00 | .80 | .50 |

TABLE 5-2
VALUES OF MANNING'S ROUGHNESS COEFFICIENT (n)
FOR CORRUGATED METAL PIPE

| Corrugation (Span x Depth) | "n" |
|---|---|
| 2-2/3" x 1/2" | 0.024 |
| 3" x 1" | 0.027 |
| 5" x 1" | 0.027 |
| 6" x 2" | 0.030 |

Source: Criteria Manual for the Design of Flood Control and Drainage Facilities in Harris County, Texas, February, 1984.

TABLE 5-3

COEFFICIENTS AT STRUCTURES

| Type of Structure | Coefficient (K) |
|---|---|
| Inlet on main line | 0.50 |
| Inlet on main line with branch lateral | 0.25 |
| Manhole on main line with 22-1/2º lateral | 0.75 |
| Manhole on main line with 45º lateral | 0.50 |
| Manhole on main line with 60º lateral | 0.35 |
| Manhole on main line with 90º lateral | 0.25 |

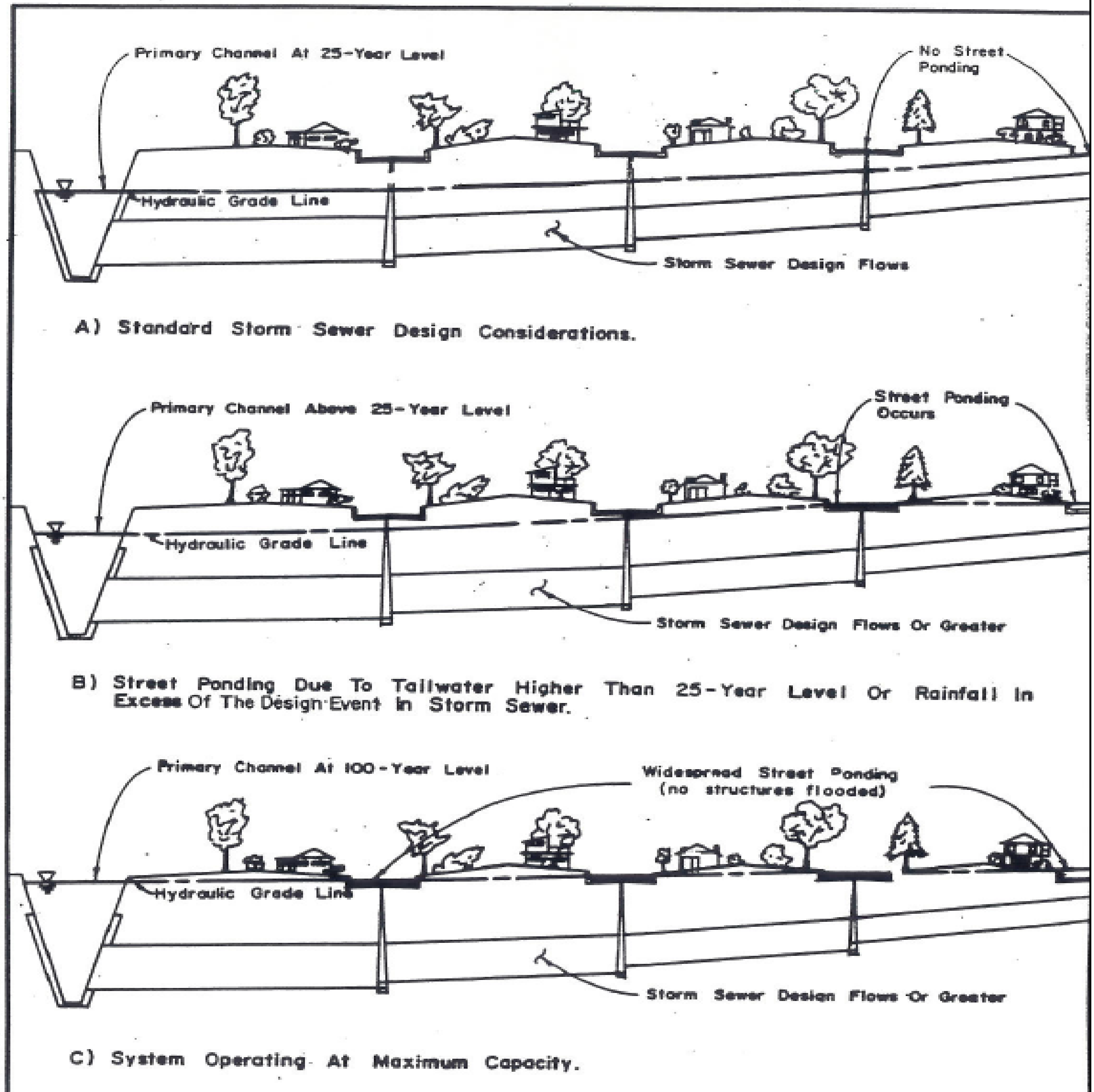
Source: City of Waco, Texas, Storm Drainage Design Manual

TABLE 5-4

COEFFICIENTS FOR ENTRANCE LOSSES

| Type of Entrance | Coefficient (K) |
|---|---|
| Pipe, Concrete[1] | |
| Projecting from fill, socket end (groove-end) | 0.2 |
| Projecting from fill, sq. cut end | 0.5 |
| Headwall or headwall and wingwalls | |
| Socket end of pipe (groove-end) | 0.2 |
| Square-edge | 0.5 |
| Rounded (radius = 1/12D) | 0.2 |
| Mitered to conform to fill slope | 0.7 |
| Inlet or Manhole at beginning of line[2] | 1.25 |

Source: (1) Hydraulic Charts for the Selection of Highway Culverts, U.S. Department of Commerce, December, 1965.
(2) City of Waco, Texas, Storm Drainage Design Manual.

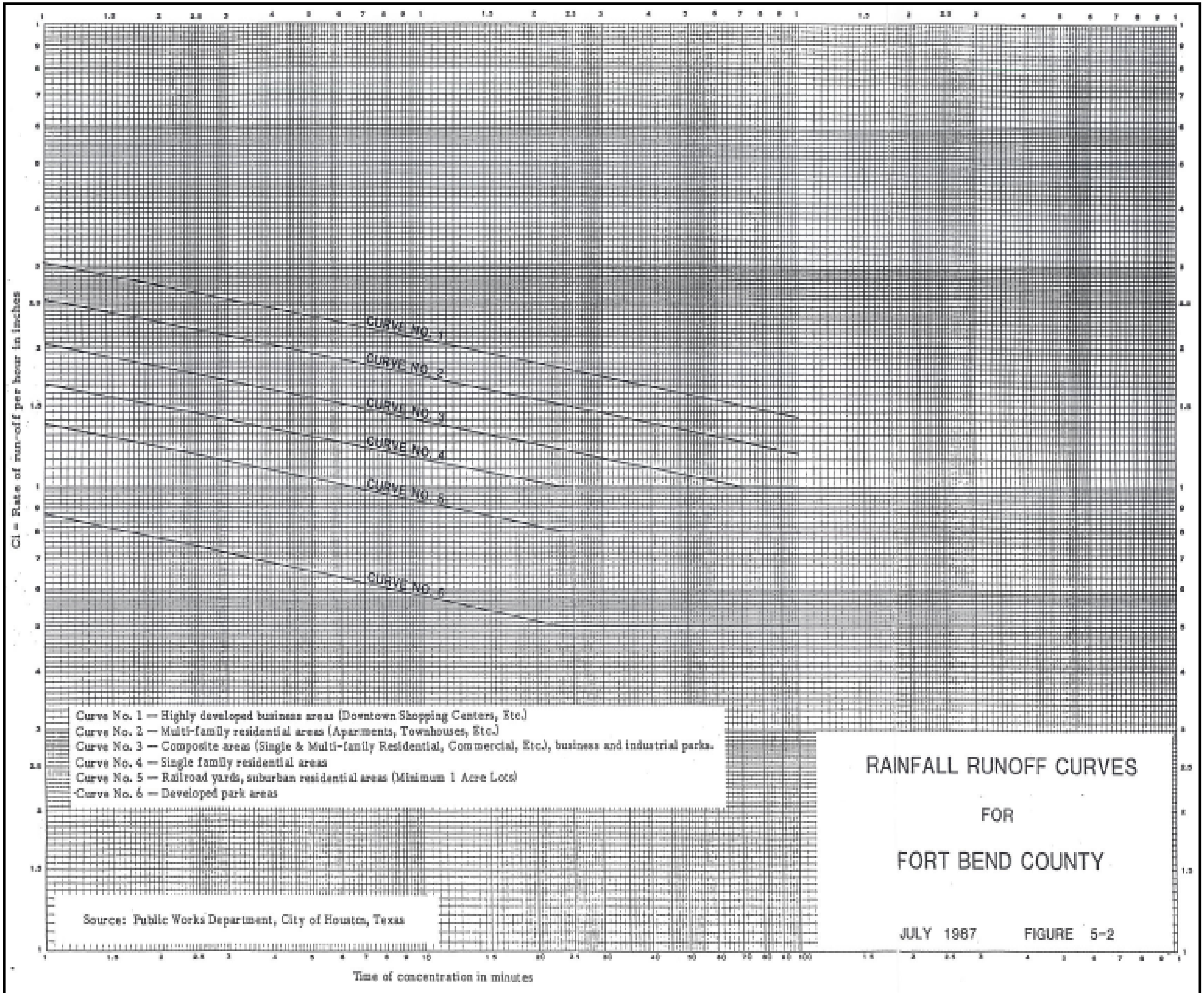


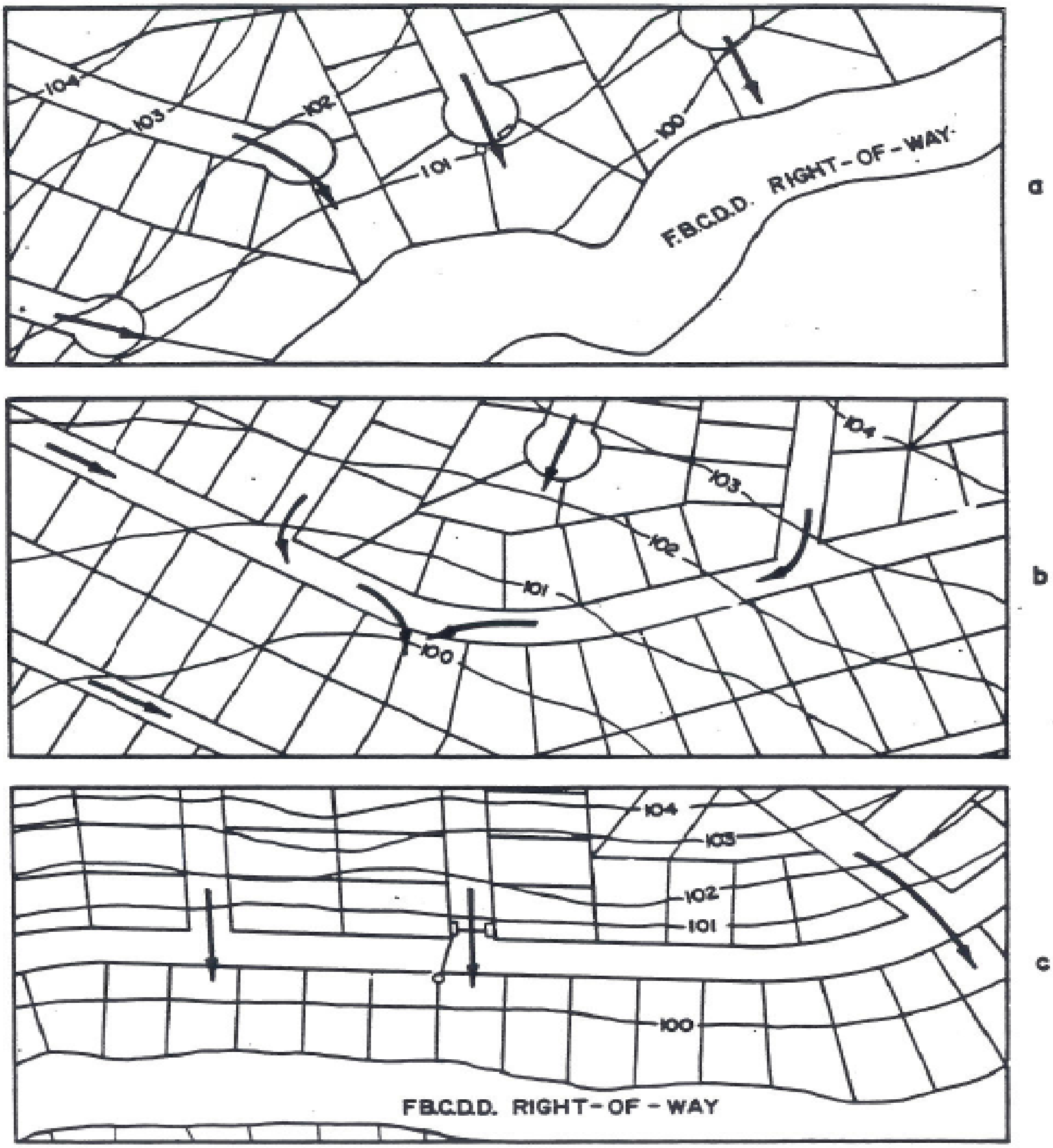

Source:

**Criteria Manual for Design of Flood Control and Drainage Facilities in Harris County, Texas, February, 1984.**

STORM SEWER-CHANNEL INTERACTION FOR FORT BEND COUNTY, TEXAS

August 1986 | FIGURE 5-1


CI = Rate of run-off per hour in inches
CURVE NO. 1
CURVE NO. 2
CURVE NO. 3
CURVE NO. 4
CURVE NO. 5
CURVE NO. 6
Curve No. 1 — Highly developed business areas (Downtown Shopping Centers, Etc.)
Curve No. 2 — Multi-family residential areas (Apartments, Townhouses, Etc.)
Curve No. 3 — Composite areas (Single & Multi-family Residential, Commercial, Etc.), business and industrial parks.
Curve No. 4 — Single family residential areas
Curve No. 5 — Railroad yards, suburban residential areas (Minimum 1 Acre Lots)
Curve No. 6 — Developed park areas
Source: Public Works Department, City of Houston, Texas
RAINFALL RUNOFF CURVES
FOR
FORT BEND COUNTY
JULY 1987
FIGURE 5-2
Time of concentration in minutes

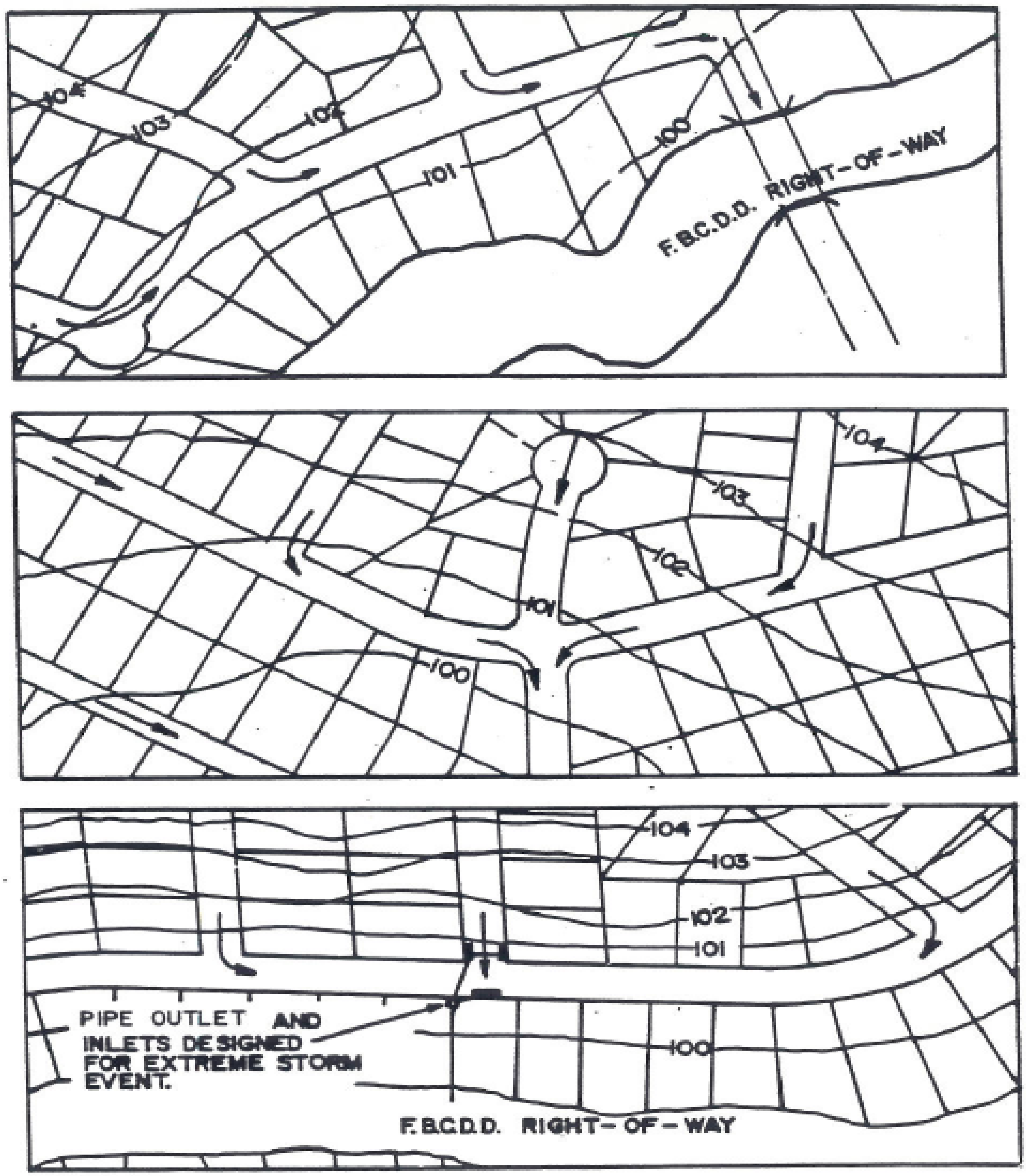



Source: Criteria Manual for Design of Flood Control and Drainage Facilities in Harris County, Texas, Feb. 1984.

UNDESIRABLE SHEET FLOW PATTERNS FOR FORT BEND COUNTY, TEXAS

August 1986

FIGURE 5-3

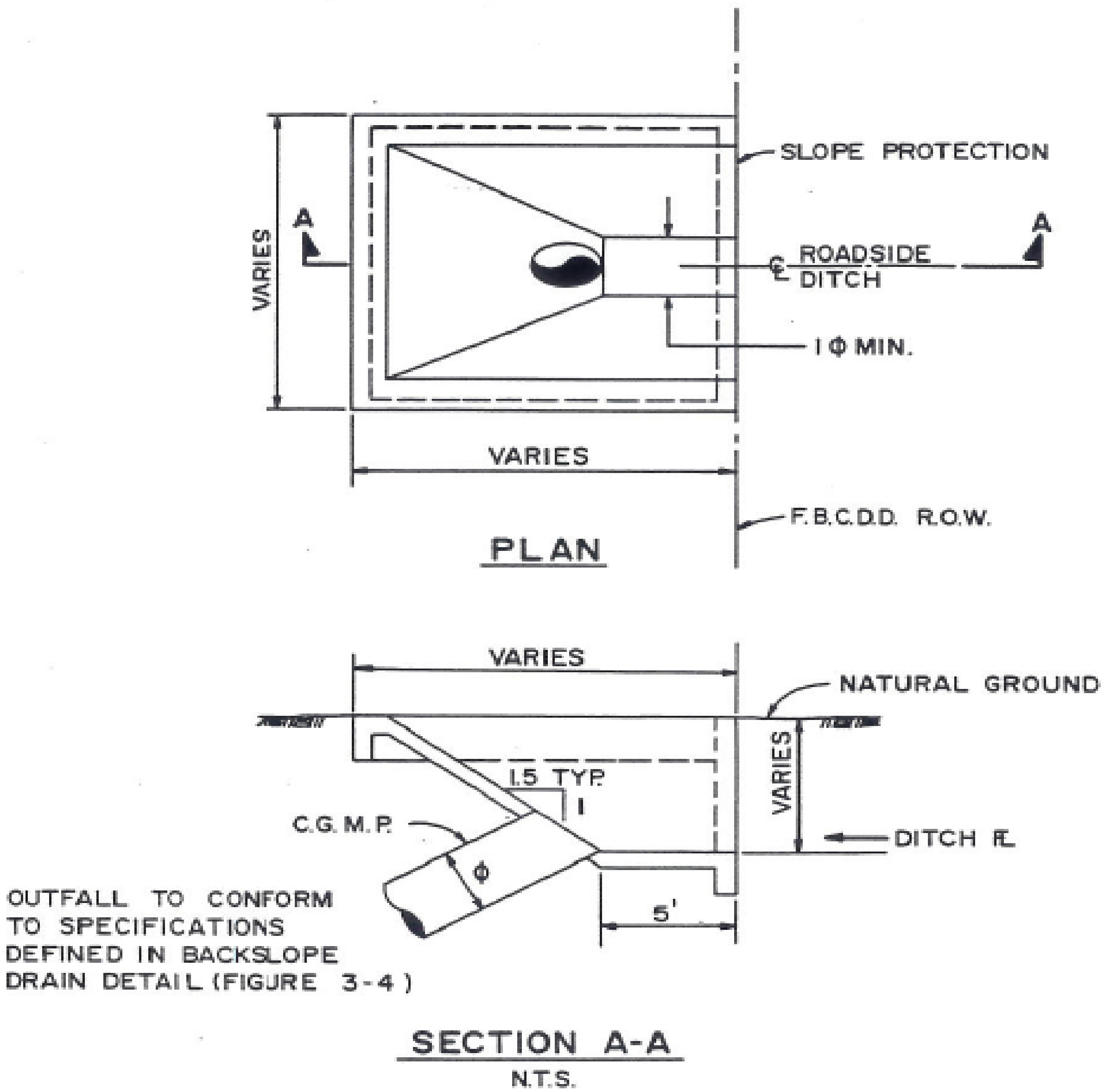



Source: Criteria Manual for Design of Flood Control and Drainage Facilities in Harris County, Texas, Feb. 1984.

ACCEPTABLE SHEET FLOW PATTERNS FOR FORT BEND COUNTY, TEXAS

August, 1986 | FIGURE 5-4

NOTE:
WHERE BACKSLOPE INTERCEPTOR SWALES FLOW TOWARDS ROADSIDE DITCH, MODIFY TO ACCOMODATE FLOW.




TYPICAL ROADSIDE DITCH DRAIN DETAIL FOR FORT BEND COUNTY, TEXAS

| | |
|---|---|
| August 1986 | FIGURE 5-5 |