Section 5

Flow In Storm Drains And Their Appurtenances

5.01 General

A general description of storm drainage systems and quantities of storm runoff is in Section 2 of this manual. It is the purpose of this section to consider the significance of the hydraulic elements of storm drains and their appurtenances to a storm drainage system. Corrugated metal pipe is not to be used in City of Lewisville maintained systems.

Hydraulically, storm drainage systems are conduits (open or enclosed) in which unsteady and non-uniform free flow exits. Storm drains accordingly are designed for open-channel flow to satisfy as well as possible the requirements for unsteady and non-uniform flow. Steady flow conditions may or may not be uniform.

5.02 Velocities And Grades

A. Minimum Grades

Storm drains should operate with velocities of flow sufficient to prevent excessive deposition of solid materials, otherwise objectionable clogging may result. The controlling velocity is near the bottom of the conduit and considerably less than the mean velocity. Storm drains shall be designed to have a minimum mean velocity flowing full of 3.0 fps. Table 5-1 indicates the grades for both concrete pipe ($n=0.013$) and for corrugated metal pipe ($n=0.024$) to produce a velocity of 3.0 fps, which is considered to be the lower limit of scouring velocity. The minimum slope for standard construction procedures shall be 0.45% when possible. Any variance must be approved by the Engineering Department.

B. Maximum Velocities

Maximum velocities in conduits are important mainly because of the possibilities of excessive erosion on the storm drain inverts. Table 5-2 shows the limits of maximum velocity.

C. Minimum Diameter

Pipes which are to become an integral part of the public storm sewer system shall have a minimum diameter of 18 inches.

*Table 5-1
Minimum Slope Required
To Produce Scouring Velocity

Pipe Size (Inches)	Concrete Pipe Slope ft./ft.	Corrugated Metal Pipe Slope ft./ft.
18	0.0018	0.0060
21	0.0015	0.0049
24	0.0013	0.0041
27	0.0011	0.0035
30	0.0009	0.0031
36	0.0007	0.0024
42	0.0006	0.0020
48	0.0005	0.0016
54	0.0004	0.0014
60	0.0004	0.0012
66	0.0004	0.0011
72	0.0003	0.0010
78	0.0003	0.0009
84	0.0003	0.0008
96	0.0002	0.0007

*From City of Waco, Texas Storm Drainage Design Manual

NOTE: Corrugated Metal Pipe is not to be used in systems maintained by the CIty of Lewisville.

*Table 5-2
Maximum Velocity In Storm Drains

Description	Maximum Permissible Velocity
Culverts (all Types)	15 fps
Storm Drains (inlet laterals)	No Limit
Storm Drains (collectors)	15 fps
Storm Drains (mains)	12 fps

*From City of Waco, Texas Storm Drainage Design Manual

5.03 Materials

In selecting a roughness coefficient for concrete pipe, between 0.011 and 0.015, consideration will be given to the average conditions during the useful life of the structure. An "n" value of 0.017 for concrete pipe shall be used primarily in analyzing old conduits where alignment is poor and joints have become rough. If, for example, concrete pipe is being designed at a location where it is considered suitable, and there is reason to believe that the roughness would increase through erosion or corrosion of the interior surface, slight displacement of joints, or entrance of foreign materials, a roughness coefficient will be selected which, in the judgement of the designer, will represent the average condition. Any selection of "n" values below the minimum or above the maximum, either for monolithic concrete structures, concrete pipe or corrugated metal pipe, must have the written approval of the Engineering Department.

The coefficient of roughness listed in Table 5-3 are for use in the nomographs contained herein, or for direct solution of Manning's Equation.

Table 5-3

Roughness Coefficients "n" For Storm Drains

Materials of Construction	Design Coefficient	Range of Manning Coefficient
Concrete Pipe	0.012	0.011-0.015
Corrugated-metal pipe		
Plain or Coated	0.024	0.022-0.026
Paved Invert	0.020	0.018-0.022
Plastic Pipe		
Smooth	0.010	0.009-0.012
Perforated	0.020	0.018-0.023

5.04 Full Or Part Full Flow In Storm Drains

A. General

All storm drains shall be designed by the application of the continuity equation and Manning's Equation either through the appropriate charts and nomographs or by direct solutions of the equations as follows:

$Q = AV$ and

$$Q = \frac{1.49}{n} A R^{2/3} S_f^{\ 1/2}$$

$Q =$ Pipe Flow (cfs)

$A =$ Cross-sectional area of pipe (ft.^{2})

$V =$ Velocity of flow (fps)

$N =$ Coefficient of roughness of pipe

$R =$ Hydraulic radius $= A/W_p$ (ft.)

$S_f =$ Friction slope in pipe (ft./ft.)

$W_p =$ Wetted perimeter

There are several general rules to be observed when designing storm sewer runs. When followed they will tend to alleviate or eliminate the common mistakes made in storm sewer design. These rules are as follows:

1. Select pipe size and slope so that the velocity of flow will increase progressively, or at least will not appreciably decrease, at inlets, bends, or other changes in geometry or configuration.
2. Do not discharge the contents of a larger pipe into a smaller one, even though the capacity of the smaller pipe may be greater due to steeper slope.
3. At changes in pipe size match the soffits of the two pipes at the same level rather than matching the flow lines.
4. Conduits are to be checked at the time of their design with reference to critical slope. If the slope of the line is greater than critical slope, the unit will likely be operating under entrance control instead of the originally assumed normal flow. Conduit slope should be kept below critical slope if at all possible. This also removes possibility of a hydraulic jump within the line.

5. Where laterals tie to trunk lines, the laterals will be made at a 60 degree angle with the trunk line and the connection will be where longitudinal centers intersect.

B. Pipe Flow Charts

Figures 5-1 through 5-9 are nomographs for determining flow properties in circular pipe, elliptical pipe, and pipe-arches. The nomographs are based upon a value of "n" of 0.012 for concrete and 0.024 for corrugated metal. The charts are self-explanatory, and their use is demonstrated by the example in Figure 5-1.

For values of "n" other than 0.012, the value of Q should be modified by using the formula below:

$$Q_c = \frac{Q_n(0.012)}{n_c} \text{ where}$$

Q_c = flow based upon n_c

n_c = Value of "n" other than 0.012

Q_n = Flow from the nomograph based on $n = 0.012$

This formula is used in two ways. If $n_c = 0.015$ and Q_c is unknown, use the known properties to find Q_n from the nomograph, and then use the formula to convert Q_n to the required Q_c. If Q_c is one of known properties, you must use the formula to convert Q_c (based on n_c) to Q_n (based on the $n = 0.012$) first, and then use Q_n and the other known properties to find the unknown value on the nomograph.

Example 1:

Given: Slope = 0.005, depth of flow $(d) = 1.8$, diameter $D = 36$, $n = 0.018$

Find: Discharge (Q)

First determine $d/D = 1.8'/3.0' = 0.6$. Then enter Figure 5-1 to read $Q_n = 34$ cfs. Using the formula $Q_c = 34(0.012/0.018) = 22.7$ cfs (Answer).

Example 2.

Given: Slope = 0.005; diameter $D = 36$, $Q = 22.7$ cfs, $n = 0.018$

Find: Velocity of flow (fps)

First convert Q_c to Q_n so that nomograph can be used. Using the formula $Q_R = 22.7(0.018)/(0.012) = 34cfs$, enter Figure 5-1 to determine d/D. Now enter Figure 5-3 to determined $V = 7.5fps$ (Answer).

5.05 Hydraulic Gradient And Profile Of Storm Drain

In storm drain systems flowing full, all losses of energy through resistance of flow in pipes, by changes of momentum or by interference with flow patterns at junction, must be accounted for by the accumulative head losses along the system from its initial upstream inlet to its outlet. The purpose of accurate determinations of head losses at junctions is to include these values in a progressive calculation of the hydraulic gradient along the storm drain system. In this way, it is possible to determine the water surface elevation which will exist at each structure.

After the computation of the quantity of storm runoff entering each inlet, the size and gradient of pipe required to carry off the design storm are to be determined. The City of Lewisville requires all hydraulic gradient calculations to begin at the outfall of the system. The following is the criteria for the starting elevation of the hydraulic gradient:

1. The 100-year water surface elevation in a creek, stream or other open channel is to be calculated for the time of peak pipe discharge in the same storm and that elevation used for beginning the hydraulic gradient.

2. For pipes that flow into sumps the design flood elevation for that sump should be obtained from the Drainage Design Engineer. The hydraulic gradient should start at the design flood sump elevation.

3. When a proposed storm sewer is to be connected to an existing storm sewer system that has a flow less than the proposed, the hydraulic gradient for the proposed storm sewer should start at the elevation of the existing storm sewers' hydraulic gradient.

Lacking these conditions or when it is desired to check the system against a larger flood than that used in sizing the pipes, and if the tailwater is known, then the hydraulic grade line and energy shall be computed and plotted. The friction head loss shall be determined by direct application of Manning's Equation or by appropriate nomographs in this section. Minor losses due to turbulence at structures shall be determined by the procedure of Paragraph 5.08. The hydraulic grade line shall in no case be closer to the surface of the ground or street than 1.5 feet unless otherwise approved by the Engineering Department. If the storm sewer system is to be extended at some future date, present and future operation of the system must be considered.

5.06 Manhole Location

Manholes shall be located at intervals not to exceed 600 feet for pipe 30 inches in diameter or smaller. Manholes shall preferably be located at street intersections, conduit junctions, changes of grade and changes of alignment.

Manholes for pipe greater than 30 inches in diameter shall be located at points where design indicates entrance into the conduit is desirable; however, in no case shall the distance between openings or entrances be greater than 1,200 feet.

5.07 Pipe Connections

Prefabricated wye and tee connections are available up to and including 24" x 24". Connections larger than 24 inches will be made by field connections. This recommendation is based primarily on the fact that field connections are more easily fitted to a given alignment than are precast connections. regardless of the amount of care exercised by the Contractor in laying the pipe, gain in footage invariably throws precast connections slightly out of alignment. This error increases in magnitude as the size of pipe increases.

5.08 Minor Head Losses At Structure

The following total energy head losses at structures shall be determined for inlets, manholes, wye branches or bends in the design of closed conduits. See Figure 5-10 for details of each case. Minimum head loss used at any structure shall be 0.10 foot, unless otherwise approved.

The basic equation for most cases, where there is both upstream and downstream velocity, takes the form as set forth below with the various conditions of the coefficient K_j

shown in Tables 5-4, 5-5 and 5-6.

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

h_j = Junction or structure head loss in feet.

v_1 = Velocity in upstream pipe in fps.

v_2 = Velocity in downstream pipe in fps.

K_j= Junction or structure coefficient of loss.

In the case where the initial velocity is negligible, the equation for head loss becomes:

$$h_g = K_j \frac{V_2{}^2}{2g}$$

Short radius bends may be used on 24" and larger pipes when flow must undergo a direction change at a junction or bend. Reductions in head loss at manholes may be realized in this way. A manhole shall always be located at the end of such short radius bends.

*Table 5-4
Junction Or Structure
Coefficient Of Loss

Case No.	Reference Figure	Description Of Condition	Coefficient K_j
I	5-10	Inlet on Main Line **	0.50
II	5-10	Inlet on Main Line with ** Branch Lateral	0.25
III	5-10	Manhole on Main Line with 45° Branch Lateral	0.50
IV	5-10	Manhole on Main Line with 90° Branch Lateral	0.25
V	5-10	45° Wye Connection or cut-in	0.75
		Inlet or Manhole at Beginning of Line	1.25
VI	5-10	Conduit on Curves for 90° ***	
VII	5-10	Curve radius = diameter	0.50
		Curve radius = (2 to 8) diameter	0.40
		Curve radius = (8 to 20) diameter	0.25
VIII	5-10	Bends where radius is equal to diameter	
		90° Bend	0.50
		60° Bend	0.48
		45° Bend	0.35
		22-1/2° Bend	0.20
		Manhole on line with 60° Lateral	0.35
		Manhole on line with 22-1/2° Lateral	0.75

* From City of Waco, Texas Storm Drainage Design Manual
** Must be approved by Director of Engineering
*** Where bends other than 90° are used, the 90° bend coefficient can be used with the following percentage factor applied:

60° Bend - 85%; 45° Bend - 70%; 22-1/2° Bend - 40%

The values of the coefficient "K_j"

for determining the loss of head due to obstructions in pipes are shown in Table 5-5 and the coefficients are used in the following equation to calculate the head loss at the obstruction: $h_j = K_j \frac{(V_2)^2}{2g}$

*Table 5-5
Head Loss Coefficients Due To Obstructions

$\frac{A^{**}}{A_{\cdot}}$	K_j	$\frac{A^{**}}{A_{\cdot}}$	K_j
1.05	0.10	3.0	15.0
1.1	0.21	4.0	27.3
1.2	0.50	5.0	42.0
1.4	1.15	6.0	57.0
1.6	2.40	7.0	72.5
1.8	4.00	8.0	88.0
2.0	5.55	9.0	104.0
2.2	7.05	10.00	121.0
2.5	9.70		

*From City of Waco, Texas Storm Drainage Design Manual
** $\frac{A}{A}$ = Ratio of area of pipe to area of opening at obstruction.

The values of the coefficient "K_j" for determining the loss of head due to sudden enlargements and sudden contractions in pipes are shown in table 5-6 and the coefficients are used in the following equation to calculate the head loss at the change in section:

$h_j = K_j \frac{V^2}{2g}$ where v = velocity in smaller pipe

*Table 5-6
Head Loss Coefficients Due To Sudden
Enlargements And Contractions

$\frac{D_2}{D_1}$ **	Sudden Enlargements K_l	Sudden Contractions K_l
1.2	0.10	0.08
1.4	0.23	0.18
1.6	0.35	0.25
1.8	0.44	0.33
2.0	0.52	0.36
2.5	0.65	0.40
3.0	0.72	0.42
4.0	0.80	0.44
5.0	0.84	0.45
10.0	0.89	0.46
	0.91	0.47

*From City of Waco, Texas Storm Drainage Design Manual
** $\frac{D_2}{D_1}$ = Ratio of larger to smaller diameter.

The values of the coefficient "K_l"

for determining the loss of head due to obstructions in pipes are shown in Table 5-5 and the coefficients are used in the following equation to calculate the head loss at the obstruction:

$$h_l = K_l \frac{(V_2)^2}{2g}$$

5.09 Utilities

In the design of a storm drainage system, the engineer is frequently confronted with the problem of grade conflict between the proposed storm drain and existing utilities such as water, gas and sanitary sewer lines.

When conflicts arise between a proposed drainage system and a utility system, the owner of the utility system shall be contacted and made aware of the conflict. Any adjustments necessary to either the drainage system or the utility can then be determined.

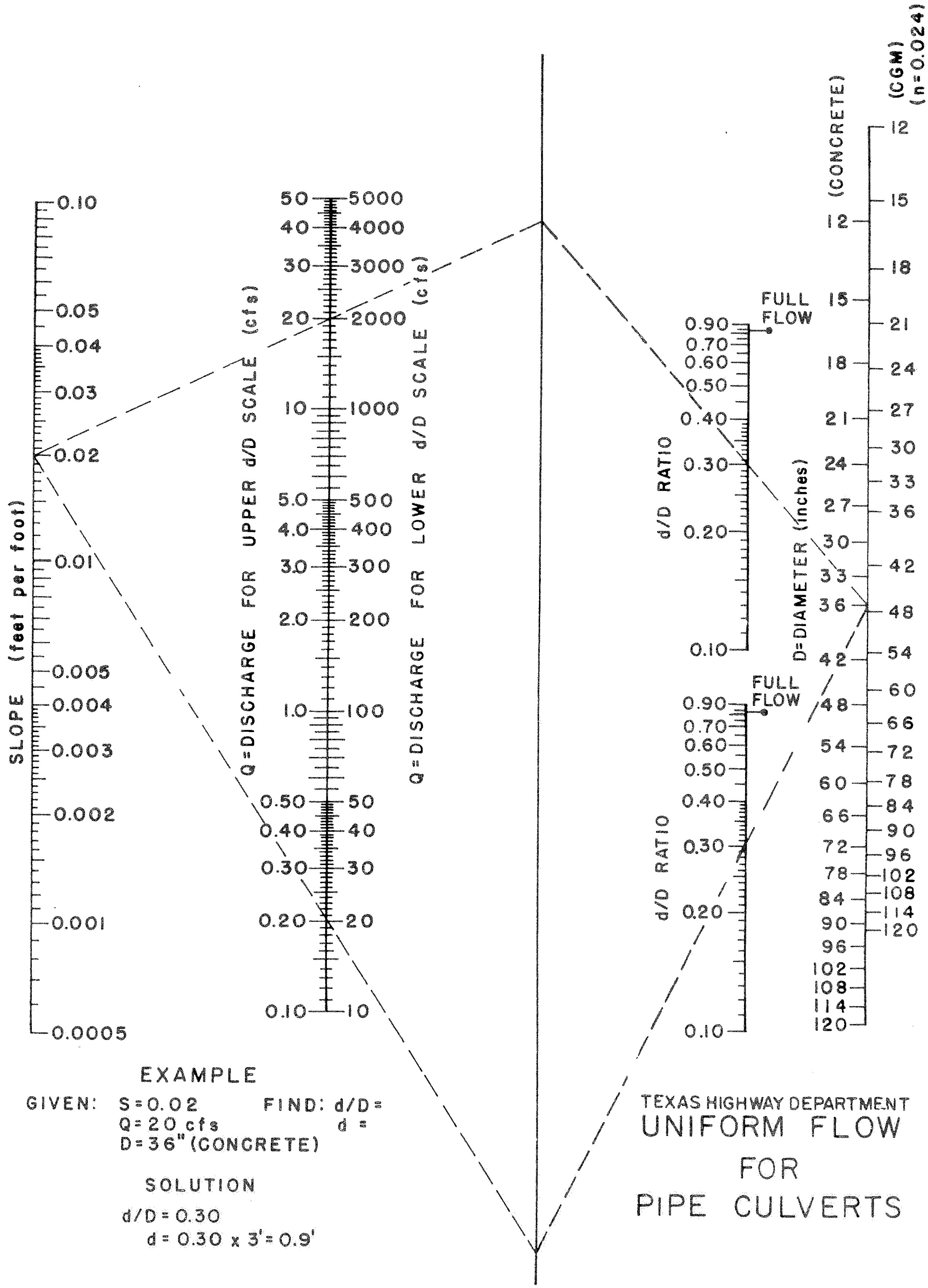
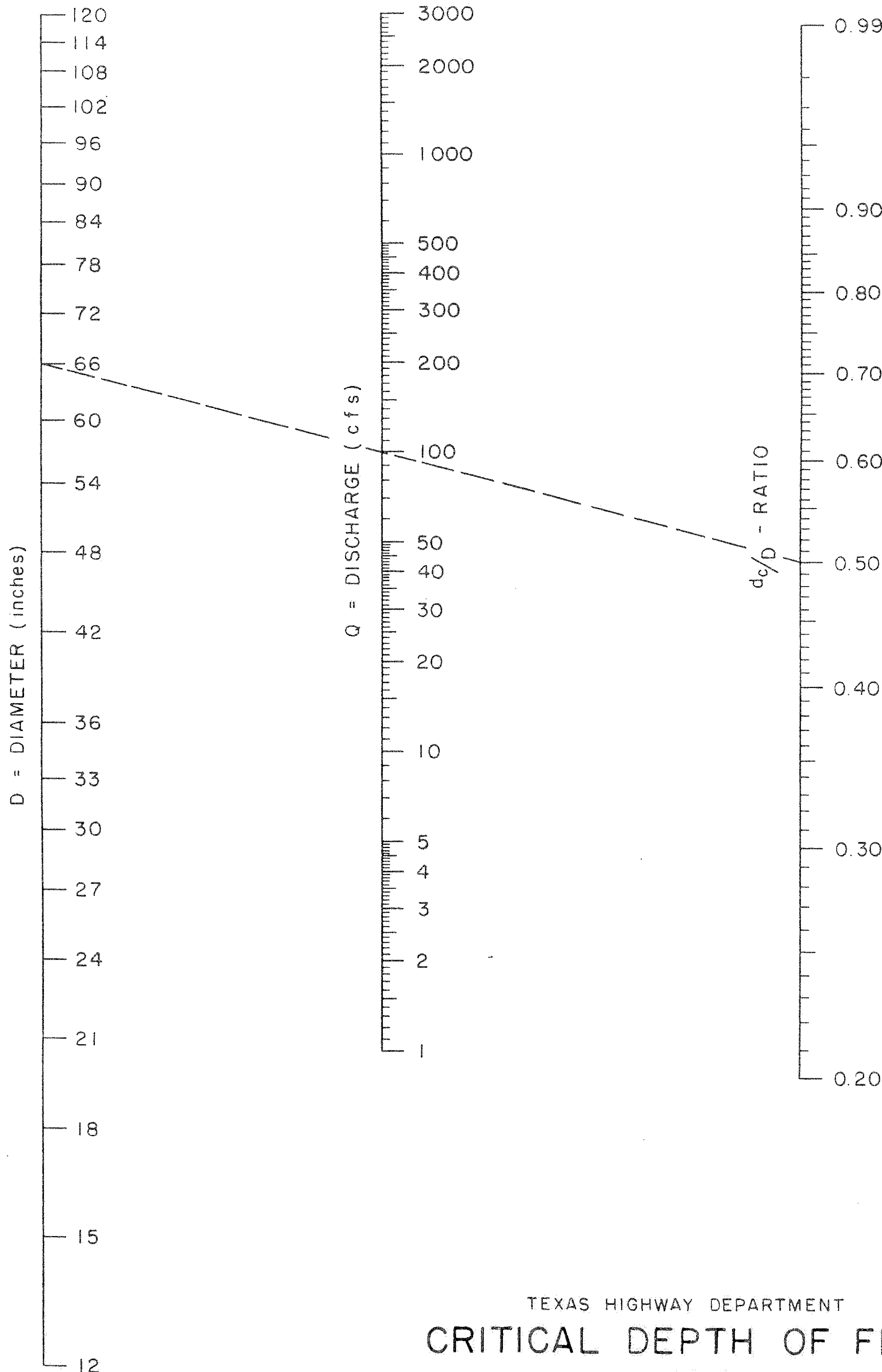

SLOPE (feet per foot)
0.10
0.05
0.04
0.03
0.02
0.01
0.005
0.004
0.003
0.002
0.001
0.0005
Q=DISCHARGE FOR UPPER d/D SCALE (cfs)
Q=DISCHARGE FOR LOWER d/D SCALE (cfs)
50 — 5000
40 — 4000
30 — 3000
20 — 2000
10 — 1000
5.0 — 500
4.0 — 400
3.0 — 300
2.0 — 200
1.0 — 100
0.50 — 50
0.40 — 40
0.30 — 30
0.20 — 20
0.10 — 10
d/D RATIO
FULL FLOW
0.90
0.70
0.60
0.50
0.40
0.30
0.20
0.10
D=DIAMETER (inches)
(CONCRETE)
(CGM) (n=0.024)
12 15 18 21 24 27 30 33 36 42 48 54 60 66 72 78 84 90 96 102 108 114 120
12 15 18 21 24 27 30 33 36 42 48 54 60 66 72 78 84 90 96 102 108 114 120
EXAMPLE
GIVEN: S=0.02
Q=20 cfs
D=36" (CONCRETE)
FIND: d/D=
d =
SOLUTION
d/D=0.30
d=0.30 x 3'=0.9'
TEXAS HIGHWAY DEPARTMENT
UNIFORM FLOW FOR PIPE CULVERTS

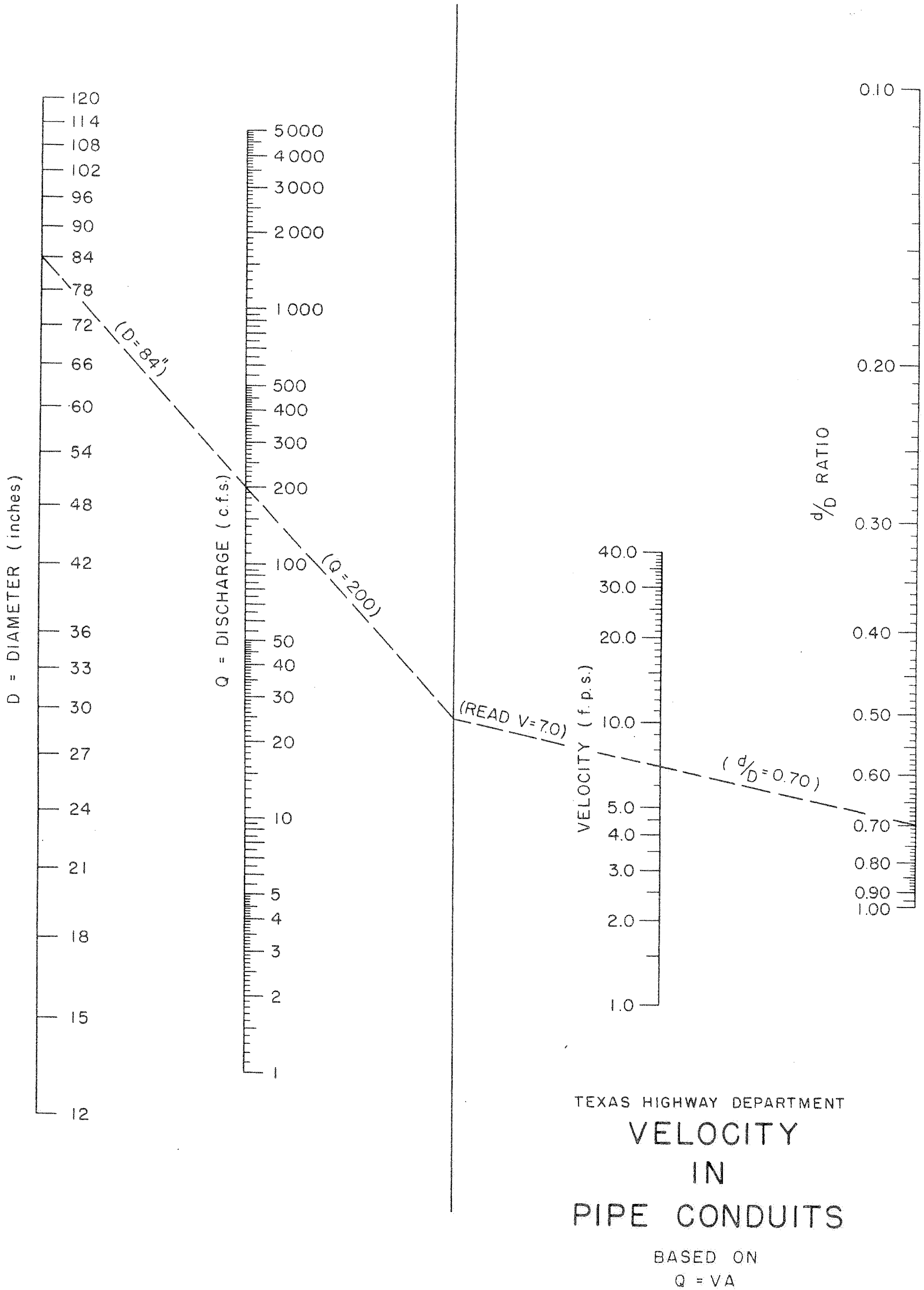
Figure 5-1



TEXAS HIGHWAY DEPARTMENT

CRITICAL DEPTH OF FLOW FOR CIRCULAR CONDUITS

Figure 5-2



D = DIAMETER (inches)
120
114
108
102
96
90
84
78
72
66
60
54
48
42
36
33
30
27
24
21
18
15
12
Q = DISCHARGE (c.f.s.)
5000
4000
3000
2000
1000
500
400
300
200
100
50
40
30
20
10
5
4
3
2
1
(D=84")
(Q=200)
(READ V=7.0)
VELOCITY (f.p.s.)
40.0
30.0
20.0
10.0
5.0
4.0
3.0
2.0
1.0
(d/D=0.70)
d/D RATIO
0.10
0.20
0.30
0.40
0.50
0.60
0.70
0.80
0.90
1.00
TEXAS HIGHWAY DEPARTMENT
VELOCITY
IN
PIPE CONDUITS
BASED ON
Q = VA

Figure 5-3

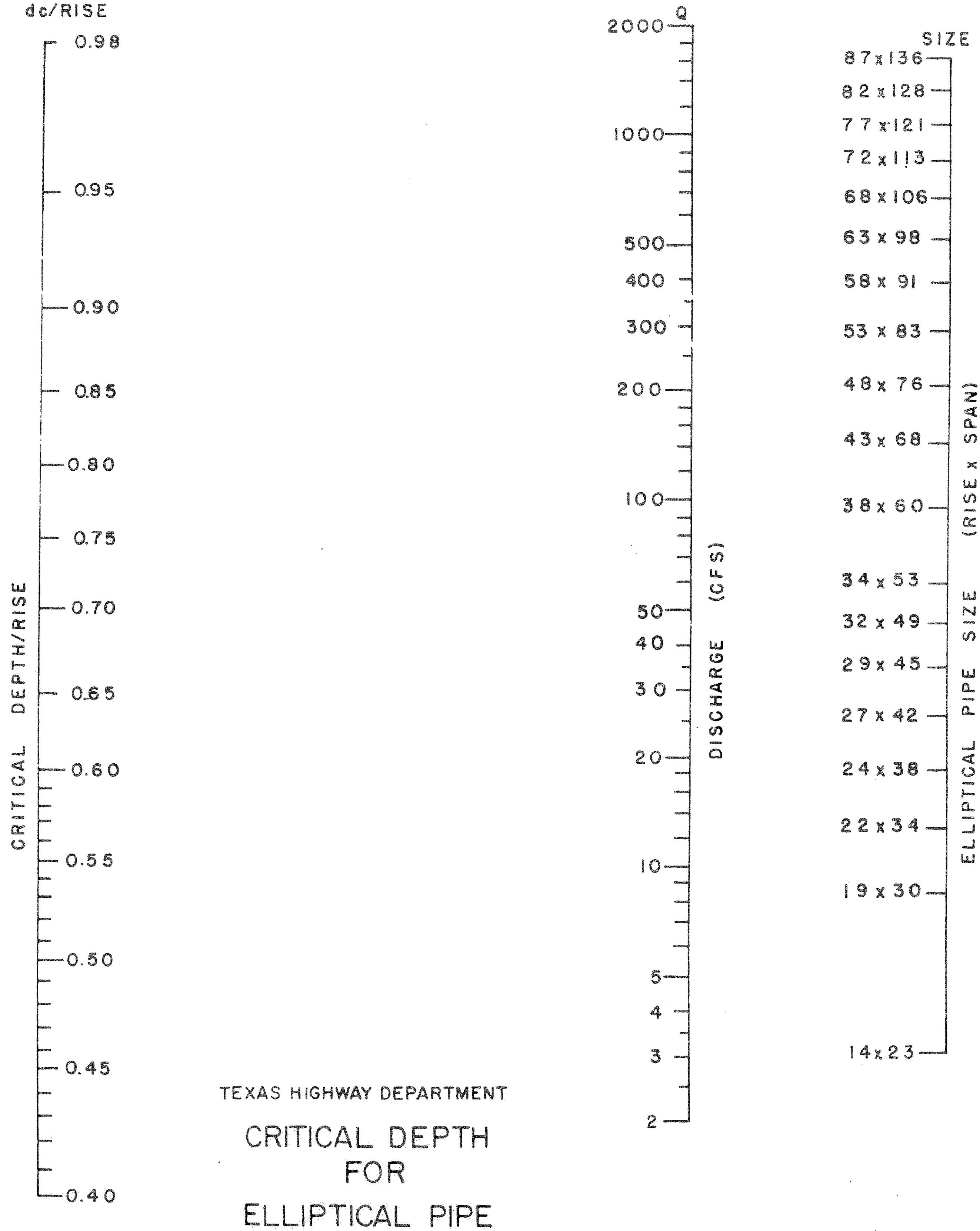

dc/RISE
0.98
0.95
0.90
0.85
0.80
0.75
0.70
0.65
0.60
0.55
0.50
0.45
0.40
CRITICAL DEPTH/RISE
Q
2000
1000
500
400
300
200
100
50
40
30
20
10
5
4
3
2
DISCHARGE (CFS)
SIZE
87x136
82x128
77x121
72x113
68x106
63x98
58x91
53x83
48x76
43x68
38x60
34x53
32x49
29x45
27x42
24x38
22x34
19x30
14x23
ELLIPTICAL PIPE SIZE (RISE x SPAN)
TEXAS HIGHWAY DEPARTMENT
CRITICAL DEPTH FOR ELLIPTICAL PIPE

Figure 5-4

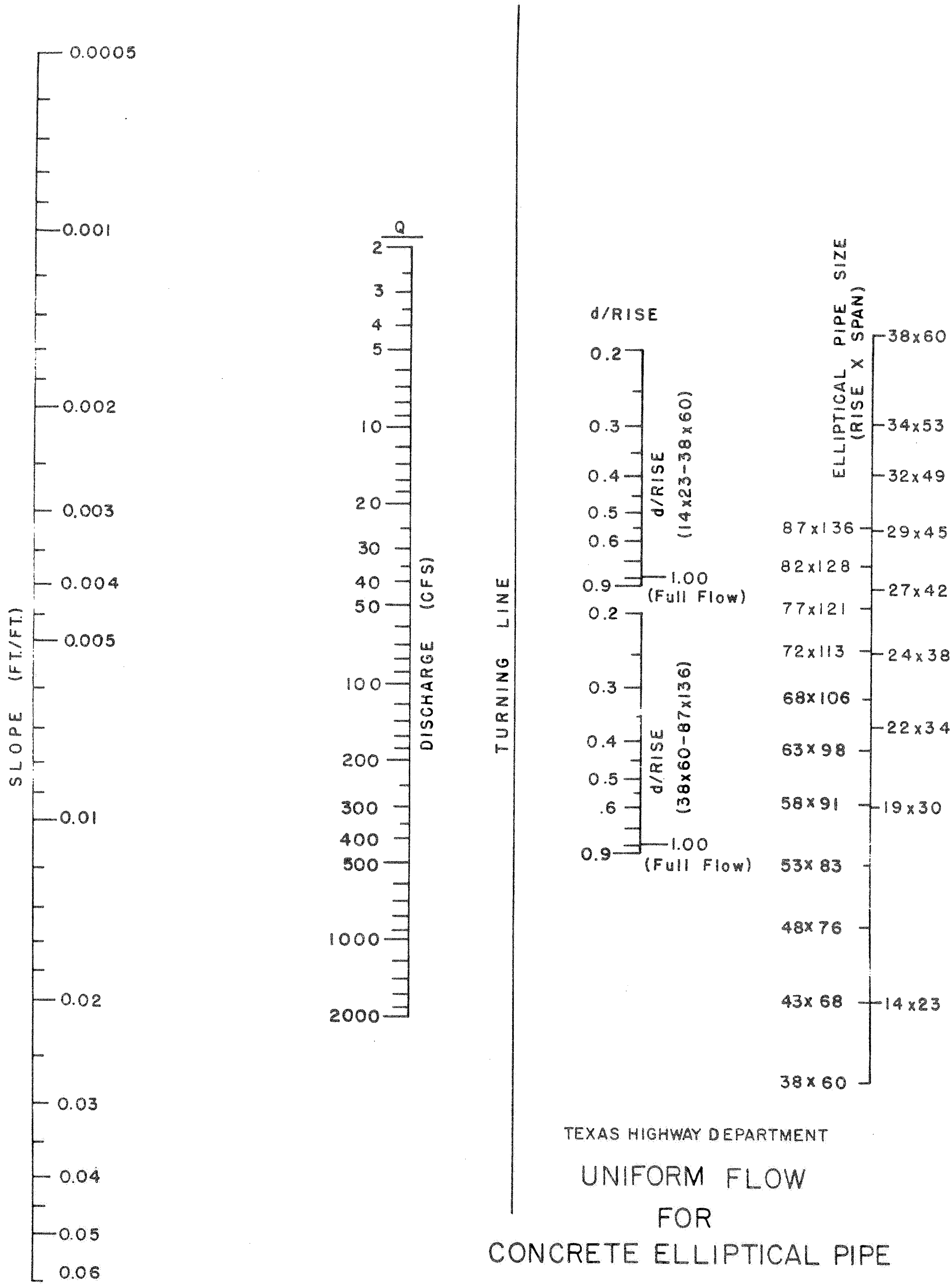

SLOPE (FT./FT.)
0.0005
0.001
0.002
0.003
0.004
0.005
0.01
0.02
0.03
0.04
0.05
0.06
Q
DISCHARGE (CFS)
2
3
4
5
10
20
30
40
50
100
200
300
400
500
1000
2000
TURNING LINE
d/RISE
0.2
0.3
0.4
0.5
0.6
0.9
1.00 (Full Flow)
d/RISE (14x23-38x60)
0.2
0.3
0.4
0.5
.6
0.9
1.00 (Full Flow)
d/RISE (38x60-87x136)
ELLIPTICAL PIPE SIZE (RISE X SPAN)
38x60
34x53
32x49
29x45
27x42
24x38
22x34
19x30
14x23
87x136
82x128
77x121
72x113
68x106
63x98
58x91
53x83
48x76
43x68
38x60
TEXAS HIGHWAY DEPARTMENT
UNIFORM FLOW FOR CONCRETE ELLIPTICAL PIPE

Figure 5-5

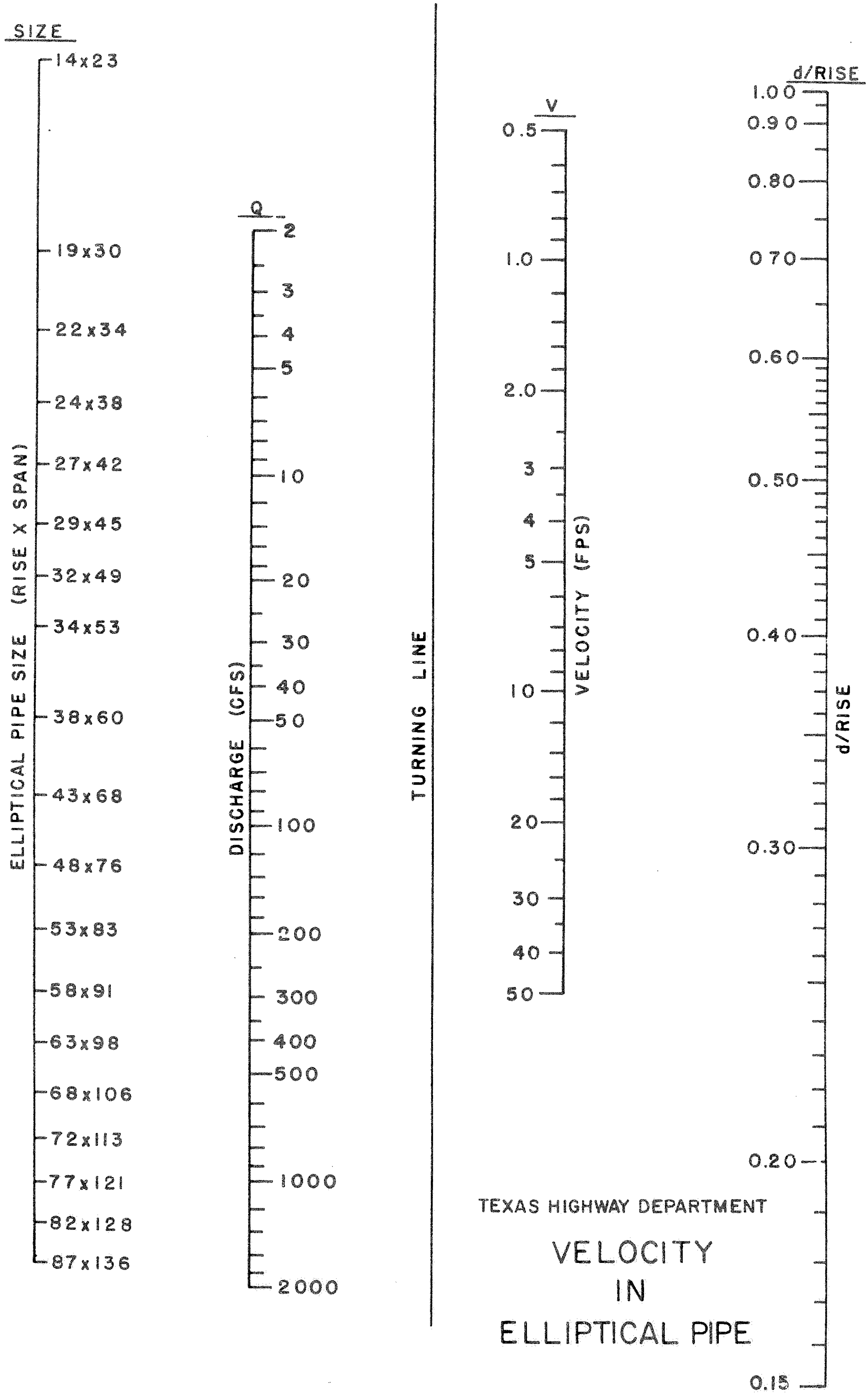

SIZE
14x23
19x30
22x34
24x38
27x42
29x45
32x49
34x53
38x60
43x68
48x76
53x83
58x91
63x98
68x106
72x113
77x121
82x128
87x136
ELLIPTICAL PIPE SIZE (RISE X SPAN)
Q
2
3
4
5
10
20
30
40
50
100
200
300
400
500
1000
2000
DISCHARGE (CFS)
TURNING LINE
V
0.5
1.0
2.0
3
4
5
10
20
30
40
50
VELOCITY (FPS)
d/RISE
1.00
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.15
d/RISE
TEXAS HIGHWAY DEPARTMENT
VELOCITY
IN
ELLIPTICAL PIPE

Figure 5-6

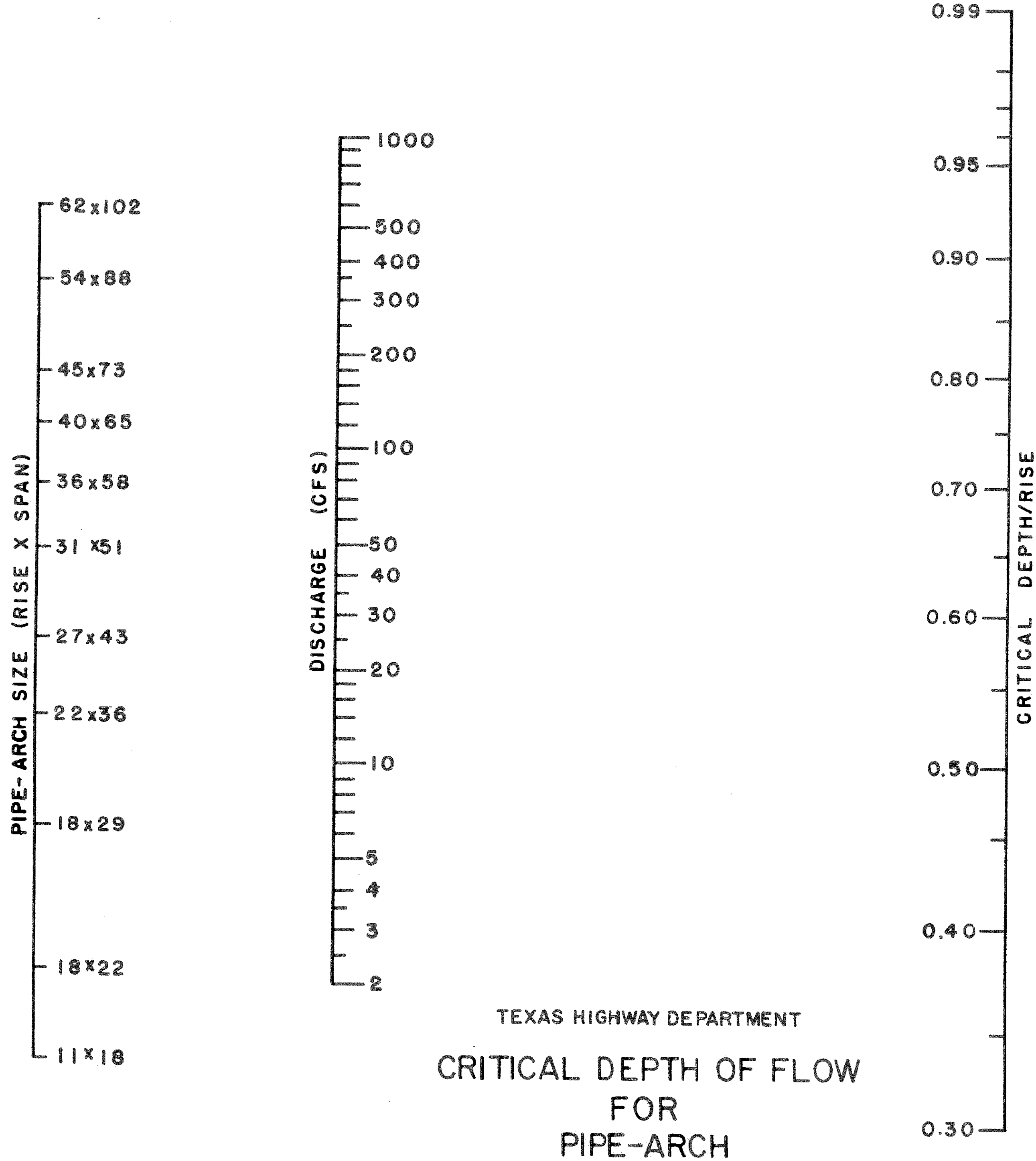

PIPE-ARCH SIZE (RISE X SPAN)
62x102
54x88
45x73
40x65
36x58
31x51
27x43
22x36
18x29
18x22
11x18
DISCHARGE (CFS)
1000
500
400
300
200
100
50
40
30
20
10
5
4
3
2
CRITICAL DEPTH/RISE
0.99
0.95
0.90
0.80
0.70
0.60
0.50
0.40
0.30
TEXAS HIGHWAY DEPARTMENT
CRITICAL DEPTH OF FLOW
FOR
PIPE-ARCH

Figure 5-7

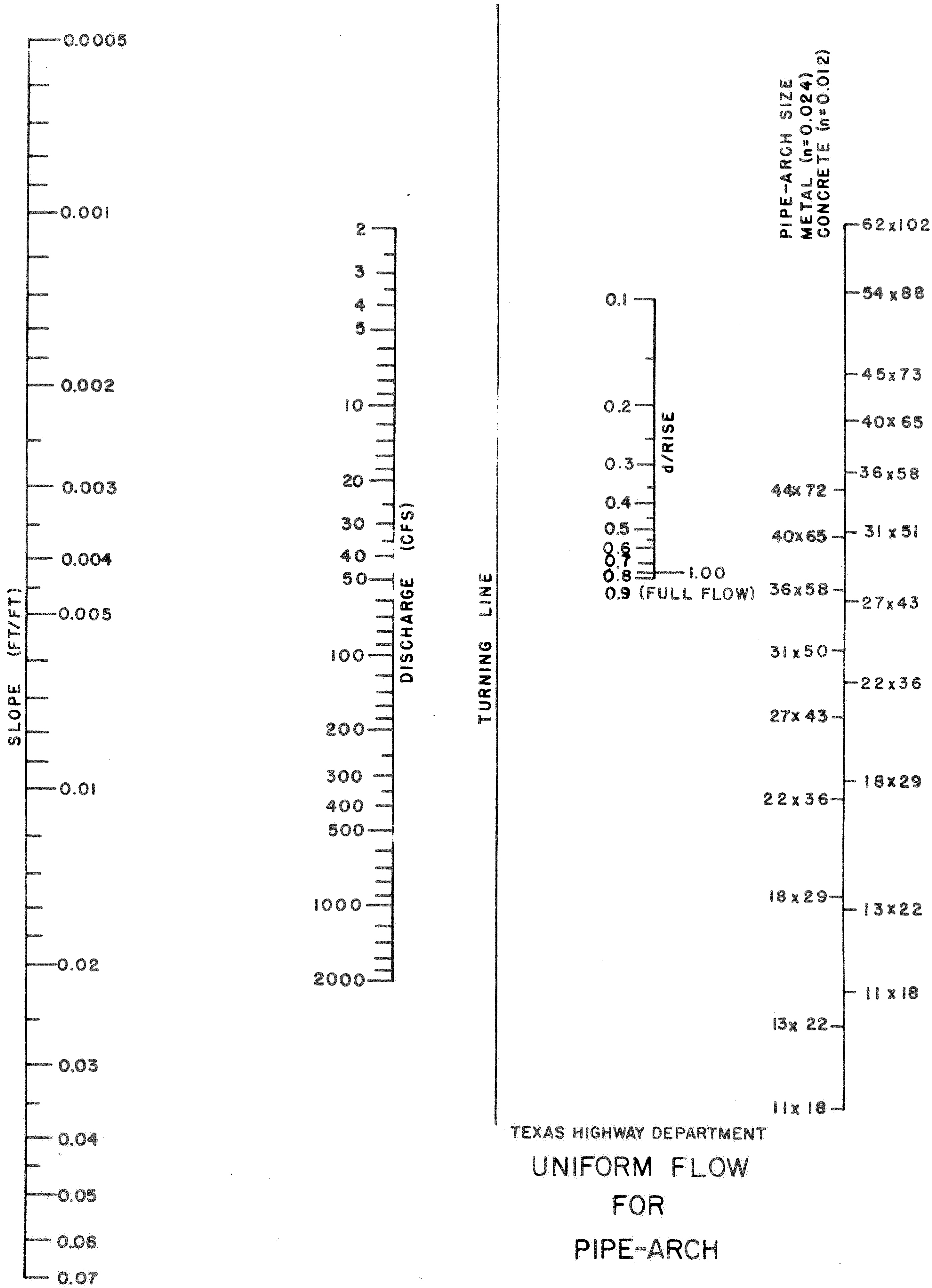

SLOPE (FT/FT)
0.0005
0.001
0.002
0.003
0.004
0.005
0.01
0.02
0.03
0.04
0.05
0.06
0.07
DISCHARGE (CFS)
2
3
4
5
10
20
30
40
50
100
200
300
400
500
1000
2000
TURNING LINE
d/RISE
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9 (FULL FLOW)
1.00
PIPE-ARCH SIZE
METAL (n=0.024)
CONCRETE (n=0.012)
62x102
54x88
45x73
40x65
36x58
31x51
27x43
22x36
18x29
13x22
11x18
44x72
40x65
36x58
31x50
27x43
22x36
18x29
13x22
11x18
TEXAS HIGHWAY DEPARTMENT
UNIFORM FLOW
FOR
PIPE-ARCH

Figure 5-8

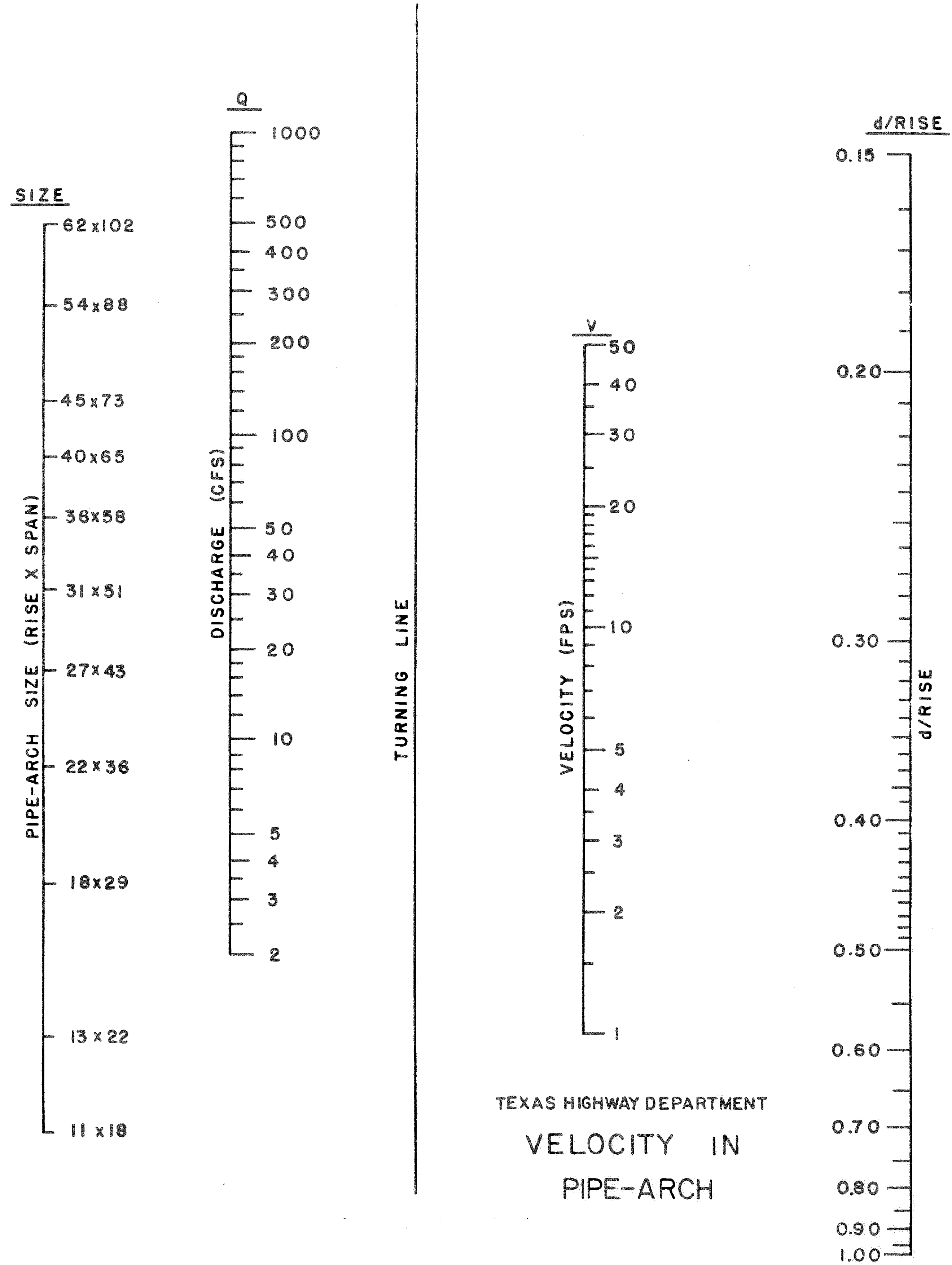
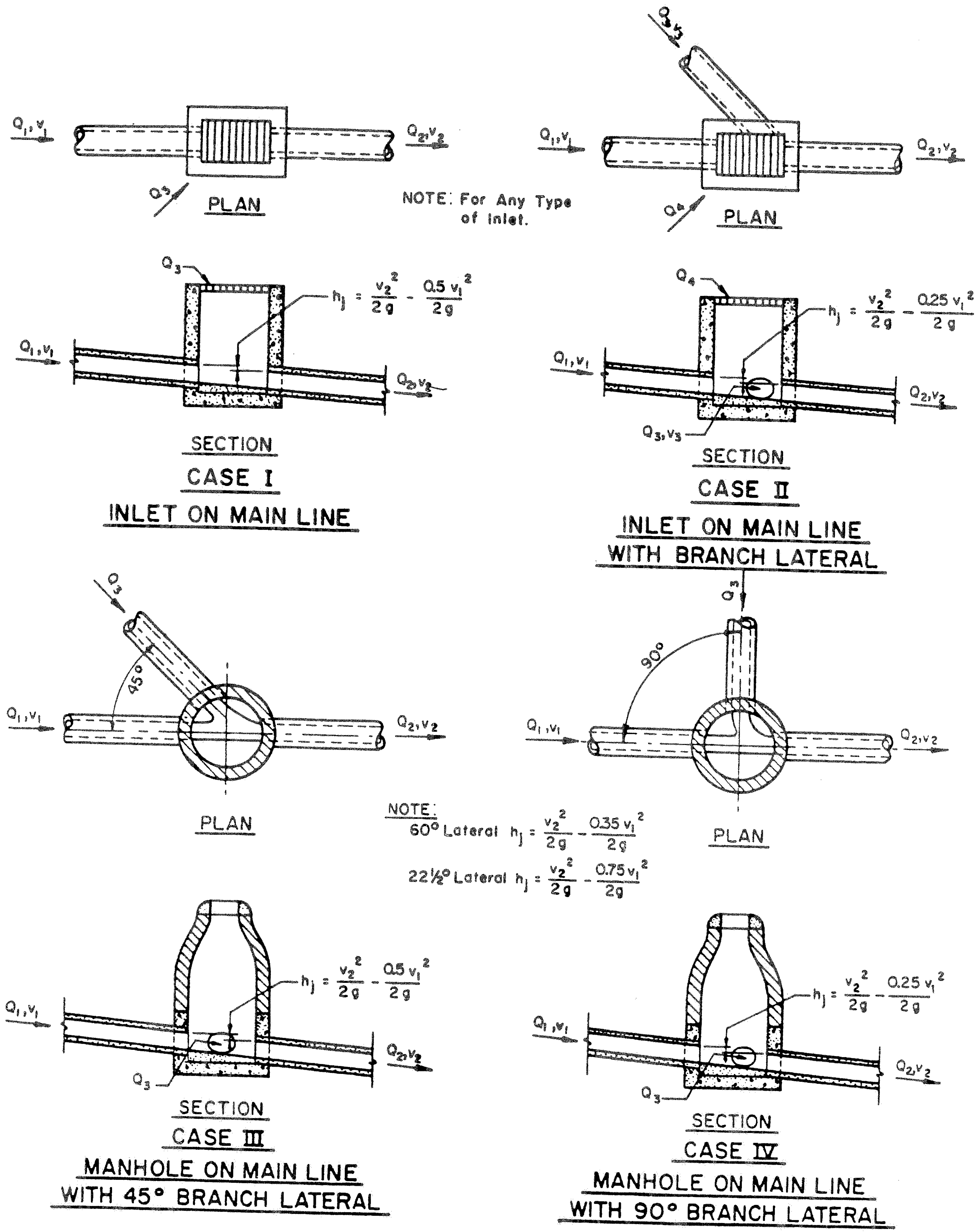
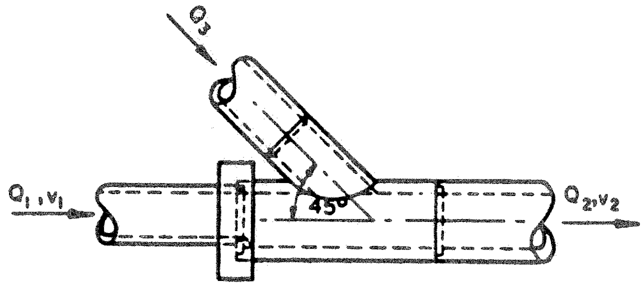

SIZE
62x102
54x88
45x73
40x65
36x58
31x51
27x43
22x36
18x29
13x22
11x18
PIPE-ARCH SIZE (RISE X SPAN)
Q
1000
500
400
300
200
100
50
40
30
20
10
5
4
3
2
DISCHARGE (CFS)
TURNING LINE
V
50
40
30
20
10
5
4
3
2
1
VELOCITY (FPS)
d/RISE
0.15
0.20
0.30
0.40
0.50
0.60
0.70
0.80
0.90
1.00
d/RISE
TEXAS HIGHWAY DEPARTMENT
VELOCITY IN PIPE-ARCH

Figure 5-9

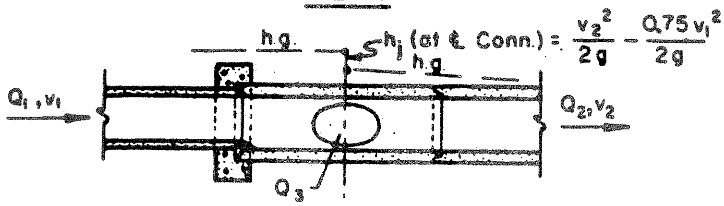


MINOR HEAD LOSSES DUE TO TURBULENCE AT STRUCTURES

Figure 5-10



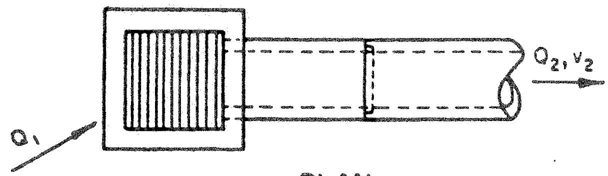
PLAN



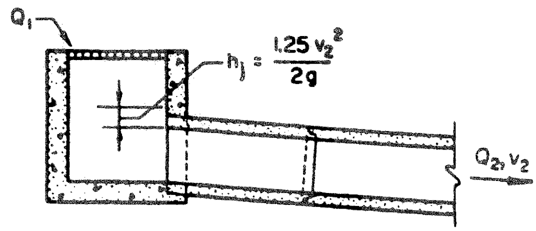
SECTION

CASE V

45° WYE CONNECTION OR CUT IN



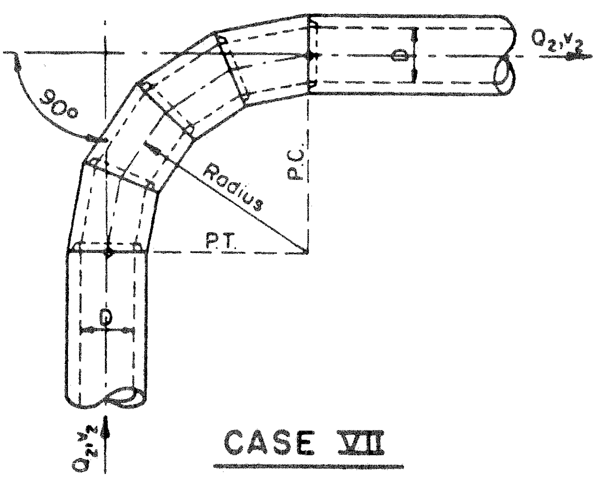
PLAN



SECTION

CASE VI

INLET OR MANHOLE AT BEGINNING OF LINE



CASE VII

CONDUIT ON 90° CURVES*

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

Radius = Dia. of Pipe $h_j = 0.50 \frac{v_2^2}{2g}$

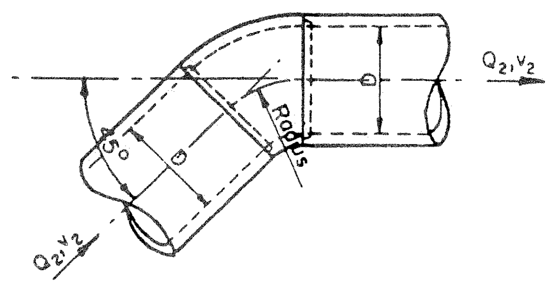
Radius = (2-8) Dia. of Pipe $h_j = 0.25 \frac{v_2^2}{2g}$

Radius = (8-20) Dia. of Pipe $h_j = 0.40 \frac{v_2^2}{2g}$

Radius = Greater than 20 Dia. of Pipe $h_j = 0$

*When curves other than 90° are used, apply the following factors to 90° curves.

60° curve 85%
45° curve 70%
22 ½° curve 40%



CASE VIII

BENDS WHERE RADIUS IS EQUAL TO DIAMETER OF PIPE

NOTE: Head loss applied at begining of bend

90° Bend $h_j = 0.50 \frac{v_2^2}{2g}$

60° Bend $h_j = 0.43 \frac{v_2^2}{2g}$

45° Bend $h_j = 0.35 \frac{v_2^2}{2g}$

22 ½° Bend $h_j = 0.20 \frac{v_2^2}{2g}$

MINOR HEAD LOSSES DUE TO TURBULENCE AT STRUCTURES