CITY OF BENTONVILLE

STORMWATER MANAGEMENT
AND
DRAINAGE MANUAL

2008

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Ordinance Number #2008-87

CITY OF BENTONVILLE
117 W. CENTRAL
BENTONVILLE, ARKANSAS 72712

STORMWATER MANAGEMENT
AND
DRAINAGE MANUAL
FOR
BENTONVILLE, ARKANSAS

October 2008

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SECTION I - SUBMITTAL PROCEDURES

1.1 GENERAL

Submittal requirements, number of copies, and distribution are dependent on the type of development.

Large-Scale Development, Preliminary Plat, and or roadway drainage shall follow requirements associated with their submittal applications.

If drainage improvements are not associated with a Large-Scale Development, Preliminary Plat, and/or roadway drainage then follow In-House Large-Scale Development application requirements.

Applications are available at the Bentonville Community Development office.

In order to minimize review time by the City Engineer's staff, all submittals shall include: (1) Title Sheet, (2) Master Site Plan, (3) Drainage Plan(s), (4) Right-of-Way Sheet, (5) Plan and Profile Sheet(s), (6) Standard and Special Detail Sheets, (7) Drainage Area Map, and (8) Calculations. Combining of the above items is allowed when legibility and readability is maintained.

A written drainage report shall accompany the plan submittal. Examples of additional submittal requirements may include a grading permit, a stormwater pollution prevention plan, and erosion control plan

The word "improvement" means the construction of public or private infrastructures, roadways, drainage, utilities, and buildings.

On combination roadway-drainage projects, it is not the intent that completely separate storm drainage plans be prepared. Where the required details of the proposed storm drainage system can be adequately shown on the roadway plans without sacrificing clarity, the roadway plans will be sufficient. If a combined project submittal is made for review of only roadway or only storm drainage aspects of the project, this fact shall be clearly indicated in large, bold lettering on the Title Sheet.

Plans and Specifications for storm drainage plans are to be signed and stamped by a professional engineer registered in the State of Arkansas. Because all plans, specifications, and calculations are retained by the City for use as permanent records, neatness, clarity and completeness are very important and lack of these qualities will be considered sufficient basis for submittal rejection.

The suggested plan sheet size is 24" x 36" with all sheets in a given set of plans the same size. The Master Site Plan should include the overall scope of the project on one sheet regardless of the scale. Plan drawings will be prepared with a maximum horizontal scale of 1" = 50'. Profile drawings for storm sewers should be drawn to a suggested horizontal scale of 1" = 20' with a maximum scale of 1" = 50'; and a minimum vertical scale of 1" = 5'. Drainage ditch profiles should be drawn at the suggested horizontal scale of 1" = 20' with a maximum scale of 1" = 50'; and a minimum vertical scale of 1" = 5'. Special cases may warrant use of larger or smaller scale drawings for increased clarity or conciseness of the plans and may be used with prior permission of the City Engineer.

Each sheet in a set of Plans shall contain a sheet number, the total number of sheets in the Plans, proper project identification and the date. Revised sheets submitted must contain a revision block with identifying notations and dates for revisions, and the revised areas must be clouded.

1.2 TITLE SHEET

Title shall include:

1. The designation of the project, which includes the nature of the project, the name or title, city, and state.

2. Index of sheets.

3. Location maps showing project location in relation to streets, railroads, and physical features. The location map shall have a north arrow and appropriate scale.

4. A project control bench mark identified as to the location and elevation.

   Horizontal and Vertical Datum:
   All drainage improvements in the City of Bentonville shall be tied to the City of Bentonville Survey Monumentation System based upon the State Plane Coordinate System, Arkansas North Zone using the North American Datum of 1983 (NAD 83). All information for newly constructed streets and roads at the time of approval shall be delivered to the City of Bentonville Engineering Department, georeferenced, in an AutoCAD compatible digital format for review and acceptance.

   All drainage construction shall use the above mentioned coordinate system and shall identify with monuments that were used for horizontal and vertical control. Elevation of controlling points shall be based on USGS NAVD 88 datum.

5. The name and address of the owner of the project and the engineer preparing the plans.

6. Engineer's seal.

1.3 Drainage Plan

The Drainage Plan shall include:

1. North arrow and scale.

2. Legend of symbols, which will apply to all sheets.

3. Name of subdivision, if applicable, and all street names and an accurate tie to at least one quarter section corner. Unplatted tracts should have an accurate tie to at least one quarter section corner.

4. Boundary line or project area.

5. Location and description of existing major drainage and utility facilities within or adjacent to the project area.

6. Location and description (size, material, etc.) of major proposed drainage facilities, along with other proposed improvements.

7. Name and description (size, material, utility owner, etc.) of each utility within or adjacent to the project area.

8. If more than one sheet is required, a match line should be used to show continuation of coverage from one sheet to the next sheet. A key should be included to show the sheet's location in relation to the overall project.

9. The registration seal of the Engineer of Record shall be placed in a convenient place on each set of plans.

10. Elevations on profiles of sections or as indicated on plans shall have U.S.G.S. data. At least one permanent bench mark in the vicinity of each project shall be noted on the first drawing of each project, and their location and elevation shall be clearly defined.

    Horizontal and Vertical Datum:

    All drainage improvements in the City of Bentonville shall be tied to the City of Bentonville Survey Monumentation System based upon the State Plane Coordinate System, Arkansas North Zone using the North American Datum of 1983 (NAD 83). All information for newly constructed streets and roads at the time of approval shall be delivered to the City of Bentonville Engineering Department, georeferenced, in an AutoCAD compatible digital format for review and acceptance.

All drainage construction shall use the above mentioned coordinate system and shall identify with monuments that were used for horizontal and vertical control. Elevation of controlling points shall be based on USGS NAVD 88 datum.

11. The top of each page shall be either north or east. The stationing of street plans and profiles shall be from left to right and downstream to upstream in the case of channel improvement/construction projects unless approved by the City Engineer.

12. Each project shall show at least 100' of topography beyond the project limits. At least 100' of topography shall be shown in areas of channel flow at the property boundary. For sites one (1) acre or smaller, the project shall show at least 50' of topography beyond the project limits. City aerial topography can be used outside of the project limits. All existing topography and any proposed changes, including utilities, telephone installations, etc., shall be shown on the plans and profiles.

13. Revisions to drawings shall be indicated above the title block in a revision block and shall show the nature of the revision and the date made. The revised area shall be clouded, unless the entire sheet is affected.

14. Utilizing the standard symbols for engineering plans, all existing utilities, telephone installations, sanitary and storm sewers, pavements, curbs, inlets, and culverts, etc., shall be shown with a broken line; proposed facilities with a solid line; land, lot, and property lines to be shown with a slightly lighter solid line. Easements shall be shown.

15. Lot lines and dimensions shall be shown where applicable.

16. Minimum floor elevation shall be shown a minimum of 3 ft. above the 100 year flood elevation, on each lot when located in a designated floodplain and in areas where flooding is known to occur or 3 ft. above the highest adjacent ground if no flood elevation is specified. All occupied buildings, whether in or out of a designated floodplain shall have the finished floor elevation a minimum of 12" above the land immediately surrounding the building.

17. It shall be understood that the requirements outlined in these standards are only minimum requirements and shall only be applied when conditions, design criteria, and materials conform to the City Specifications and are normal and acceptable to the City Engineer. When unusual subsoil or drainage conditions are suspected, an investigation should be made and a special design prepared in line with good engineering practice.

DRAINAGE DESIGN CHECKLIST
CITY OF BENTONVILLE, ARKANSAS
REVISION NO. _______________
DATE: ____________________

______ 1. PROJECT TITLE AND DATE

______ 2. PROJECT LOCATION MAP

______ 3. PROJECT DESCRIPTION

______ 4. NAME OF OWNER AND ENGINEER – With addresses and telephone numbers.

______ 5. SITE AREA – With a 1 mile radius.

______ 6. UPSTREAM AND DOWNSTREAM DRAINAGE - Brief description of the drainage path from the proposed site shown on a 1" = 200' minimum scale, 2' contour topographic map. (Include an exhibit, if required.)

______ 7. AREA DRAINAGE PROBLEMS

______ 8. HYDROLOGIC COMPUTATIONS - Include complete runoff computations for the design frequency storm specified in the Manual for each specific type drainage system

______ 9. OPEN CHANNEL FLOW DESIGN - Include computations for normal depth and velocity (Use Figure 9.2 or equal)

______ 10. PAVEMENT DRAINAGE DESIGN - Include width of spread for design flow (Use Figure 7.12 or equal). Show flow in gutter for Q_{10} and Q_{100} in plan view.

______ 11. CULVERT DESIGN - Include all computations and check for inlet/outlet control (Use Table 4.3 or equal)

______ 12. STORM SEWER INLET DESIGN - Include all computations (Use Figure 7.12 or equal)

______ 13. STORM SEWER DESIGN - Include all computations (Use Figure 8.1 and/or 8.2 or equal)

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DRAINAGE DESIGN CHECKLIST

Figure 1.1

_____ 14. STORMWATER DETENTION DESIGN - Include the following computations and backup/support data:

SUMMARY OF RUNOFF - A Table with minimum 2, 10, 25, 50, and 100 year storm flow comparisons for existing and proposed conditions and detention volumes required if applicable - Also describe method used for determining stormwater runoff flows.

RECOMMENDATIONS/SUMMARY - Description of any drainage improvements to be made to the site - Also, the following backup/support data:

_____ a. Runoff coefficient/RCN computations (existing and proposed conditions)

_____ b. Complete runoff computations for the 2, 10, 25, 50, and 100-year storms (existing and proposed conditions)

_____ c. Detention basin size requirement computations (using an approved method)

_____ d. Release structure design computations (include release rate computations for the 2, 10, 25, 50, and 100 year storms)

_____ e. Stage-Storage and Stage-Discharge curves for the detention facility

_____ 15. DESIGN STORM DESIGNATED BY Q _____ = and design flow rate for each street crossing or drainage structure

_____ 16. Detail Plans and Specifications as required by the City of Bentonville Drainage Manual, including Project Location (Street Address and Vicinity Map).

_____ 17. AS-BUILT DRAWINGS AND CERTIFICATION that drainage facility is constructed to the City of Bentonville Standards and Ordinances and signed and sealed by an Arkansas registered engineer.

_____ 18. ADD THE FOLLOWING PARAGRAPH TO THE DRAINAGE LETTER:

Improvements as outlined in this report and depicted on the preliminary plat and design drawings shall not increase the risk of endangerment to life or have negative impacts on adjacent or downstream property or watersheds.

____

Signed and Sealed by Professional Engineer

_____ 19. OTHER

____

____

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SECTION II - DETERMINATION OF STORM RUNOFF

2.1 GENERAL

Continuous records over many years on the amounts and rates of runoff from the City's streams would provide the best source of data on which to base the design of storm drainage and flood protection systems. Unfortunately, stream flow records of adequate history are not available for the City's drainage ways. Experience based prediction of the probable frequencies and amounts of runoff are not available as a standard practice in determining stormwater runoff and flood flows.

The accepted practice, therefore, is to relate runoffs to rainfall events; events which enjoy a very lengthy period of record. The correlation of the rainfall events to runoff amounts is a widely accepted practice. Direct correlation provides a means for predicting the rates and amounts of runoff expected from the City's watersheds at various recurrence intervals since runoff events are directly based on known frequency of occurrence for various rainfall events.

2.2 CITY OF BENTONVILLE DRAINAGE METHODS

There are numerous methods of rainfall computations on which the design of storm drainage and flood control systems are based. Three widely used methods include: The Rational Method, the Soil Conservation Service Technical Release - 55 Synthetic Hydrograph Method, and the use of the Corps of Engineers HEC-I / HEC-II computer programs or a method authorized by the Little Rock Office of the Corps of Engineers. One of these three methods should be the basis of all drainage analysis in the City of Bentonville. The area limits and/or ranges for the analysis methods are:

Rational Method	Less than 10 Acres
Rational Method, SCS TR-55 / TR-20 or HEC-1	10 to 100 Acres
SCS TR-55 / TR-20 Hydrograph Method Or HEC I / HEC II	100 to 2,000 Acres
HEC-I Methods or other Corps of Engineers authorized methods	Greater than 2,000 Acres or within Designated FEMA Flood Plain Areas

Computer programs may be used in the satisfaction of the above minimum standards. The City Engineer may disallow any specific software about which there are concerns of the accuracy thereof, or which produce printed calculations that are inadequate to define the design process, or which are difficult to review.

Criteria for the above methods are specified in the following Sections:

2.3 RATIONAL METHOD

The Rational Method is probably the most frequently used rainfall-runoff method in urban hydrology in the United States. The Rational Method formula is expressed as:

$$Q = C (I) (A)$$

"Q" is defined as the peak rate of runoff in cubic feet per second. Actually, Q is in units of acre inches per hour, but calculator results differ from cubic feet by less than 1 percent. Since the difference is so small, the "Q" value calculated by the equation is universally taken as cubic feet per second or "CFS".

"C" is the dimensionless coefficient of runoff represented in the ratio of the amount of runoff to the amount of rainfall.

"I" is the average intensity of rainfall in inches per hour for a period of time equal to the critical time of full contribution of the drainage area under consideration. This critical time for full contribution is commonly referred to as "time of concentration".

"A" is the area in acres that contributes to runoff at the point of design or the point under consideration.

Basic assumptions associated with use of the Rational Method are as follows:

1. The computed peak rate of runoff to the design point is the function of the average rainfall rate during the time of concentration to that point.

2. The time of concentration is the critical value in determining the design rainfall intensity and is equal to the time required for water to flow from the hydraulically most distant point in the watershed to the point of design.

3. The ratio of runoff to rainfall, "C", is uniform during the entire duration of the storm event.

4. The rate of rainfall or rainfall intensity, "I", is uniform for the entire duration of the storm event and is uniformly distributed over the entire watershed area.

2.3.1 RUNOFF COEFFICIENT ("C")

The proportion of the total rainfall that runs off depends on the relative porosity or imperviousness of the ground surface, the surface slope, and the ponding character of the surface. Impervious surfaces, such as asphalt pavements and roofs of buildings, will be subject to nearly 100 percent runoff regardless of the slope, after the surfaces have become thoroughly wet. On-site inspections and aerial photographs are valuable in estimating the nature of the surfaces within the drainage area.

2.3.2 SOIL

The runoff coefficient "C" in the Rational formula is also dependent on the character of the soil. The type and condition of the soil determines its ability to absorb precipitation. The rate at which a soil absorbs precipitation generally decreases if the rainfall continues for an extended period of time. The soil absorption or infiltration rate is also influenced by the presence of soil moisture before a rain (antecedent condition), the rainfall intensity, the proximity of the ground water table, the degree of soil compaction, the porosity of the subsoil, vegetation, ground slopes, and surface depressions.

2.3.3 SELECTION OF RUNOFF COEFFICIENTS

It should be noted that the runoff coefficient "C" is the variable of the Rational Method, which is least susceptible to precise determination. Proper selection requires judgment and experience on the part of the Engineer, and its use in the formula implies a fixed ratio for any given drainage area, which in reality is not the case. A reasonable coefficient must be chosen to represent the integrated effects of infiltration, detention storage, evaporation, retention, flow routing, and interception, all of which affect the time distribution and peak rate of runoff. However, to standardize City Design Computations, Table 2.1 represents standard runoff coefficient values by land use and composite analysis. The values for respective land uses shall govern for all drainage analysis and design projects using the Rational Method.

2.3.4 RAINFALL INTENSITY ("I")

Rainfall intensity is the design rainfall rate in inches per hour for a particular drainage basin or subbasin. The rainfall intensity is selected on the basis of the design rainfall duration and frequency of occurrence. The design duration is equal to the time of concentration for a drainage area under consideration. Once the time of concentration is known, the design intensity of rainfall may be determined from the rainfall intensity curves (see Figure 2.5). The frequency of occurrence is a statistical variable, which may be established by the City standards or chosen by the Engineer as a design parameter.

2.3.5 TIME OF CONCENTRATION

The time of concentration used in the Rational Method is a measure of the time of travel required for runoff to reach the design point or the point under consideration. The critical time of concentration is the time to the peak of the runoff hydrograph at the design point. Runoff from a watershed usually reaches a peak at the time when the entire watershed area is contributing to flow. The critical time of concentration, therefore, is assumed to be the flow time measured from the most remote part of the watershed to the design point. A trial and error procedure is usually required to select a most remote point of a watershed since type of flow, ground slopes, soil types, surface treatments and improved conveyances all effect flow velocity and time of flow. The types of flow used in calculating the design time of concentration are overland flow, shallow concentrated flow, and channelized flow. Overland flow is defined as that portion of the flow pattern which results in thin sheet flow across a given area. Overland flow often becomes shallow concentrated flow when it enters a poorly defined channel. Channelized flow is that which allows significant depth accumulation in a defined ditch, natural channel, improved channel, or pipe system.

Figures 2.2, 2.3, or Figure 2.4, depending upon type of flow shall be used for all time of concentration flow computations. In Figure 2.3, the known ground slope plus the type of surface treatment is used to determine the average flow velocity in feet per second. Interpolation can be used for estimating velocities for surface treatments other than those shown. Overland flow distances will rarely exceed 300 feet in developed areas. After 300 feet, overland flow usually turns to shallow concentrated flow or channelized flow. If the overland flow time is calculated to be in excess of 20 minutes, the designer should check to be sure that the time is reasonable considering the projected ultimate development of the area.

2.3.6 CHANNELIZED FLOW

Channelized flow is that part of the flow pattern which is not shallow, sheet-type flow. Channelized flow paths may consist of pipe systems, defined natural channels, ditches, swales, and improved ditches in any combination. (See Figure 2.2)

2.3.7 DESIGN INTENSITY

The design rainfall intensity can be obtained from Figure 2.5. If a watershed involves a design time of concentration (storm duration) of over

30 minutes, applicability of the Rational Method should be checked according to the criteria of Section 2.2.

The calculated time of concentration for the watershed is taken as the duration of the rainfall event required to produce peak runoff at the design point. This relation and the Rational Formula state that the rate of runoff is equal to the rate of supply (rainfall excess) if the rainfall event lasts long enough to permit the entire watershed to contribute. These assumptions may not involve significant errors for watersheds several acres in size. However, errors may be involved with significant channel and overland flow storage effects.

2.3.8 DRAINAGE AREA ("A")

The drainage area or the area from which runoff is to be estimated is measured in acres when using the Rational Method. Drainage areas should be calculated using planimetric-topographic maps, supplemented by field surveys where topographic data has changed or where the contour interval is too great to distinguish the exact direction of overland flows.

2.4 SOIL CONSERVATION SERVICE METHOD, TABULAR TR-55

2.4.1 GENERAL

The Soil Conservation Service tabular method is a synthetic hydrograph method developed specifically for use in urbanized and urbanizing areas. This method is similar to the Rational Method in that runoff is directly related to rainfall amounts through use of runoff curve numbers (RCN's) (See Table 2.1). The basic equation used with the tabular method is also very similar to that used for the Rational Method.

q = (DRO) x (DA) x (HDO)

q = Hydrograph coordinate discharge in CFS

DRO = Direct runoff amount in inches

DA = Drainage area in square miles

HDO = Hydrograph distribution ordinate in CSM/inch

CSM/inch = Cubic feet per second per square mile per inch of runoff

Hydrograph coordinates are computed from the hydrograph distribution data in the TR-55 Manual. A coordinated value is computed for each time shown in the distribution data. The calculated "q" results, when plotted against the corresponding times, constitute the runoff hydrograph.

The tabular method is useful in analyzing watersheds involving several subareas with complex runoff patterns. The method is most useful in analyzing changes in runoff volume due to development and in evaluating runoff control measures. The SCS tabular method as described herein shall be used in all cases where watershed problems involve two or more interacting subareas. The SCS tabular method is the suggested method to be used in evaluating the runoff effects of urbanization and the evaluation/design of runoff control measures.

2.4.2 METHOD FUNDAMENTALS

The Soil Conservation Service has completed extensive research in the runoff potential from native soils under specific conditions of pre-wetting and rainfall events. This research has also extended to correlation of native soil types and land uses in assessing runoff potential. Runoff curve number or RCN values have been developed which approximate the runoff potential from various types of development with respect to native soils. These RCN values are similar to runoff coefficient values used in the Rational Method in that they can be used to estimate the amount of rainfall, which will actually result in runoff. The amount of runoff, which will occur for a given RCN value, is a function of the design rainfall, and is termed direct runoff amount (DRO). The RCN values differ from runoff coefficients in that:

1. Their development encompasses a wide range of land uses.

2. Runoff potentials from native soil types are taken into account.

3. The amount of runoff, which will occur, is the function of both the RCN value and the design rainfall.

Design rainfalls used with a tabular method are 24-hour rainfall amounts taken from the U.S. Weather Bureau data. The data includes reoccurrence intervals or frequencies of occurrence of 10, 25, 50, and 100 years.

Hydrograph distribution ordinates used in the tabular method were developed by computer analysis of many watersheds of various sizes and configurations. The distribution data published in Technical Release No. 55 was developed specifically by computing hydrographs for a one square mile drainage area for a range of times of concentration and routing of the hydrographs through stream reaches with a range of travel times.

One advantage of using the empirically-based hydrograph distributions over simpler methods is that the channel storage and overland flow storage effects are taken into account. This feature is particularly useful in the cases involving larger, more complex watersheds.
The biggest advantage of the tabular method over simpler methods is that the runoff effects of different development patterns (both in land use and in drainage facilities) can be easily measured. The effects of a wide variety of runoff control measures can also be measured since the method's work result is in hydrograph form. These features are extremely valuable in watershed management efforts since differences in flow magnitudes are often more important in design decisions than are determinations of precise peak flow values for given conditions. Also, volumetric effects of runoff can be considered with hydrograph methods.

2.4.3 LIMITATIONS ON TABULAR METHOD USE

The tabular method should not be used when large changes in RCN values occur among watershed subareas and when runoff volumes are less than about 1-1/2 inches for RCN values less than 60.

The tabular method should not be used for watersheds that have several subareas with times of concentration below six minutes. In these cases, subareas should be combined so as to produce a time of concentration of at least six minutes (0.10 hours) for the combined areas.

2.4.4 TABULAR METHOD USE

2.4.4.1 DETERMINATION OF RUNOFF CURVE NUMBER (RCN)

The runoff curve number determines the amount of runoff that will occur with the given rainfall. Soil types and land use are used to determine the runoff potential.

Calculation of the RCN values for a watershed or subarea proceeds in the same fashion as the calculation of weighted runoff coefficients used in the Rational Method. Area calculations are completed for each land use type within the study area. Table 2.1 lists runoff curve numbers for various land uses. A more complete table listing RCN values for specific soil types and land coverages can be found in the TR-55 Manual. These values are used along with the area calculations to arrive at a weighted runoff curve number for the watershed or subarea under consideration. Figure 2.6 is a worksheet, which is useful in tabulating weighted runoff curve numbers for watersheds and watershed subareas. Areas can be measured either in acres or square miles. Weighted RCN values should be rounded to the nearest whole number.

2.4.4.2 DESIGN STORM DATA

The tabular method is based on 24-hour rainfall amounts for various design recurrence intervals or frequency of occurrence. These rainfall amounts are taken from the U.S. Weather Bureau Technical Paper No. 40 for Bentonville and are as follows: 4.08 inches for the 2-year frequency rainfall; 6.00 inches for the 10-year frequency rainfall; 6.96 inches for the 25-year frequency; 7.92 inches for the 50-year frequency; and 8.64 inches for the 100-year frequency.

2.4.4.3 DIRECT RUNOFF AMOUNTS FROM DESIGN STORM (DRO VALUES)

Figure 2.7 is a generalized table of direct runoff amounts for given rainfalls and runoff curve numbers. This Table can be used to interpolate runoff amounts (DRO values) from any combination of RCN between 60 and 98 and rainfall amounts between 1 and 12 inches.

2.4.4.4 MODERN APPROVED COMPUTERIZATION

Modern approved computerization of this design method by experienced engineers is encouraged.

TABLE 2.1

RUNOFF COEFFICIENTS

City of Bentonville, Arkansas

BENTONVILLE ZONING		SCS CURVE NO.* (TR-55/HEC-1)	RUNOFF COEFFICIENT (RATIONAL)
A-1	Agricultural	74 – 84	0.30 - 0.60
R-E	Residential Estate	77	0.35
R-1	Low Density Residential		
	1 Acre Lots	79	0.40
	1/2 Acre Lots	80	0.50
	1/3 Acre Lots	81	0.55
	1/4 Acre Lots	83	0.65
	1/8 Acre Lots	90	0.80
R-2	Duplex and Patio Home Residential	86	0.75
R-3	Medium Density Residential	86	0.75
R-4	High Density Residential	90	0.80
R-MH	Manufactured Home Residential	86	0.75
R-ZL	Zero Lot Line	90	0.80
R-O	Residential Office	90	0.80
R-C2	Central Residential – Moderate Density	90	0.80
R-C3	Central Residential – High Density	95	0.90 - 0.95
C-1	Neighborhood Commercial	90	0.80
C-2	General Commercial	94	0.90
C-3	Central Commercial	95	0.90 - 0.95
C-4	Shopping Center Commercial	94	0.90
I-1	Light Industrial	90	0.70 - 0.90
I-2	Heavy Industrial	96	0.80 - 0.90
Church		84 – 92	0.70 - 0.90
School		82 – 92	0.50 - 0.90
Park		74 – 84	0.30 - 0.70
Cemetery		74 – 82	0.30 - 0.50

* SCS RCN values are based on Hydrologic Soil Group C, which has been selected as the average soil type in Bentonville, Arkansas. The user should refer to the TR-55 Manual for soil types not falling into this category.

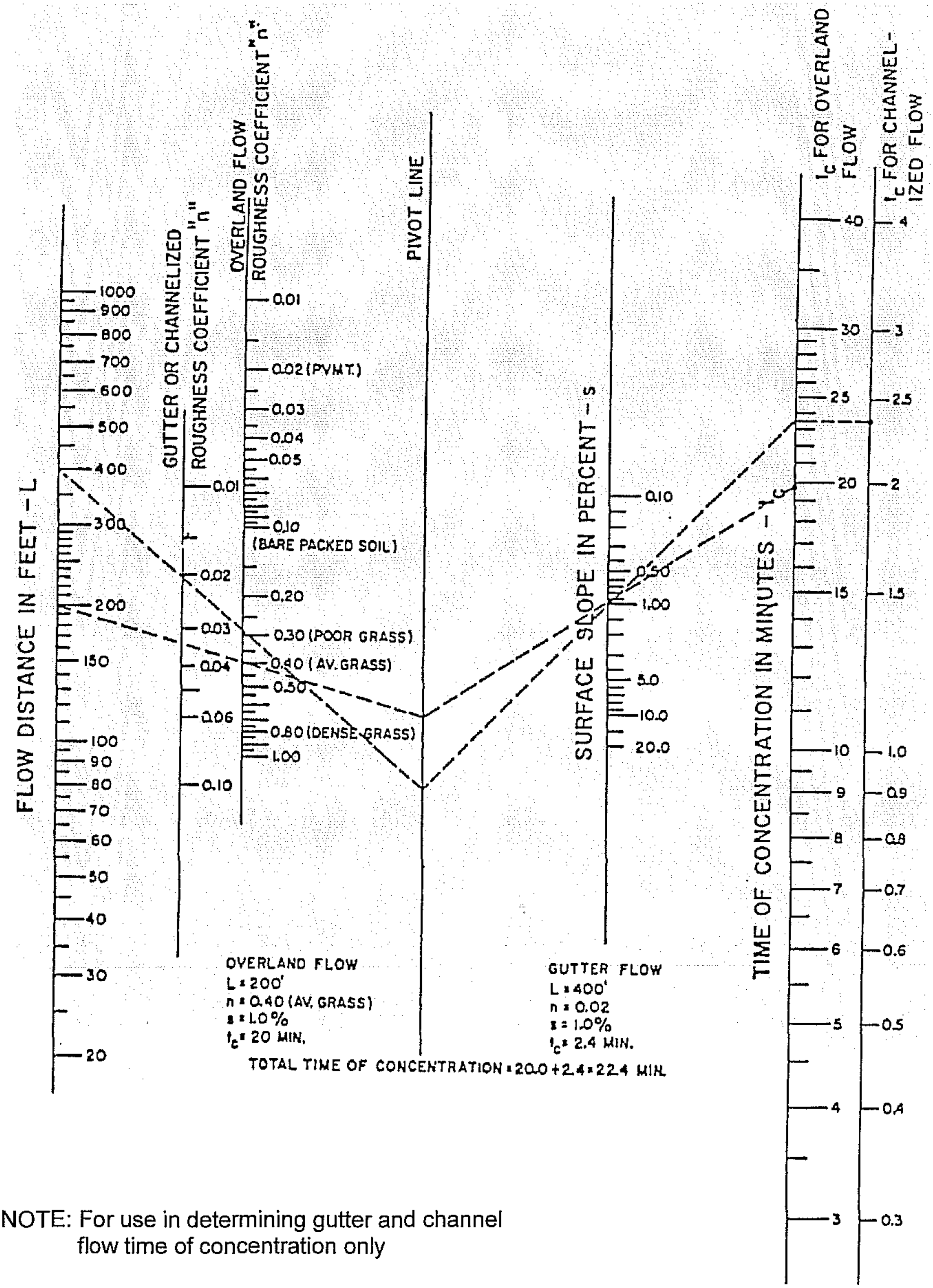
NOTE: Composite Curve Numbers and Runoff Coefficients can be calculated for a specific site.

TABLE 2.1 (Continued)

RUNOFF COEFFICIENTS FOR RATIONAL METHOD COMPOSITE ANALYSIS

City of Bentonville, Arkansas

CHARACTER OF SURFACE	RUNOFF COEFFICIENTS
Undeveloped Areas:	
Historic Flow Analysis, Greenbelts, Agricultural, Natural Vegetation	
Clay Soil	
Flat, 2%	.30
Average, 2-7%	.40
Steep, 7%	.50
Sandy Soil	
Flat, 2%	.12
Average, 2-7%	.20
Steep, 7%	.30
Streets, Parking Areas, Drives, and Walks:	
Paved	.90
Gravel	.60
Roofs:	.90
Lawns:	
Clay Soil	
Flat, 2%	.18
Average, 2-7%	.22
Steep, 7%	.35
Sandy Soil	
Flat, 2%	.10
Average, 2-7%	.15
Steep, 7%	.20

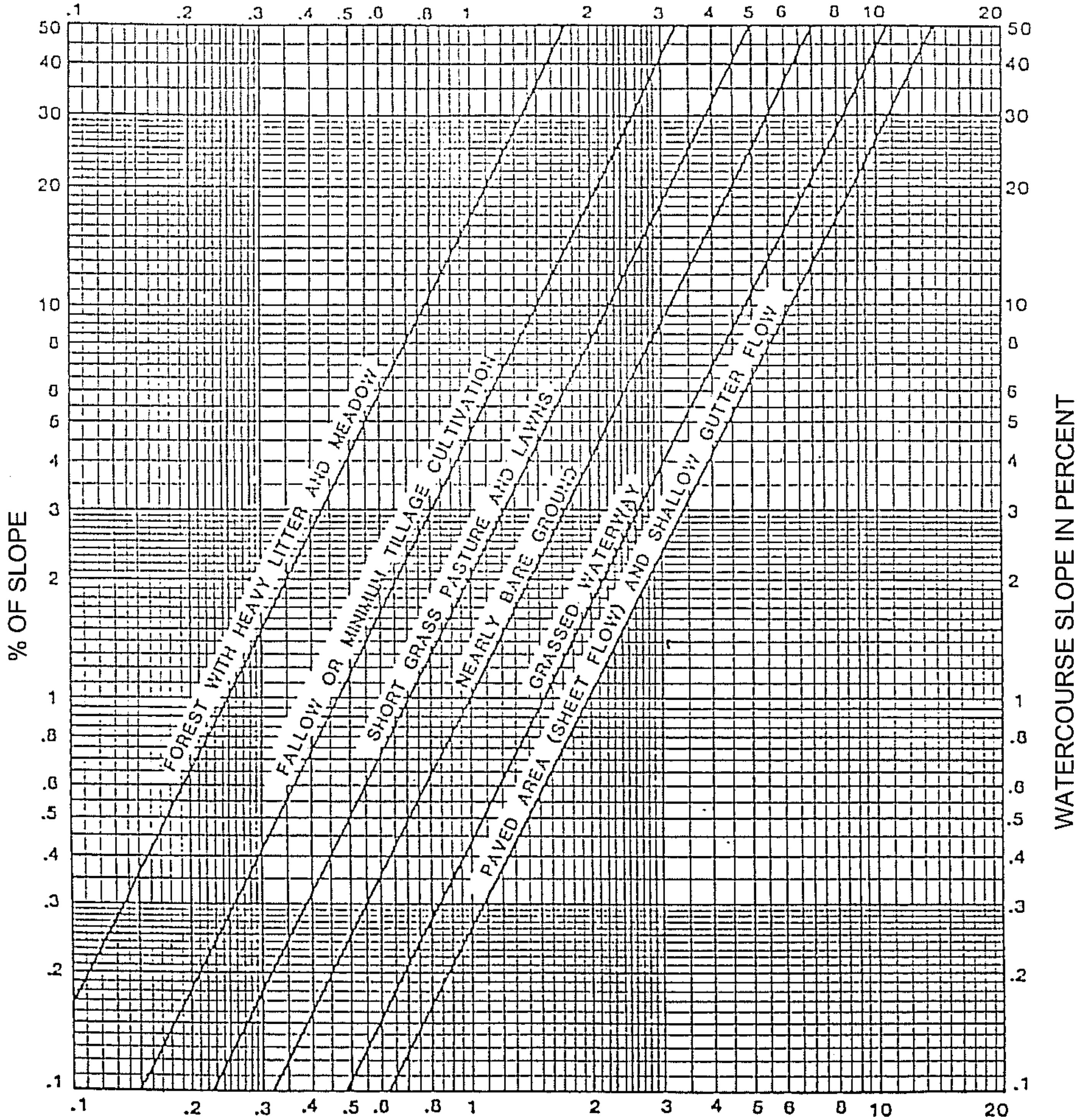


NOTE: For use in determining gutter and channel flow time of concentration only

THE CITY OF BENTONVILLE

NOMOGRAPH FOR TIME OF CONCENTRATION
SOURCE: City of Fort Worth, TX

Figure 2.2



VELOCITY IN FEET PER SECOND

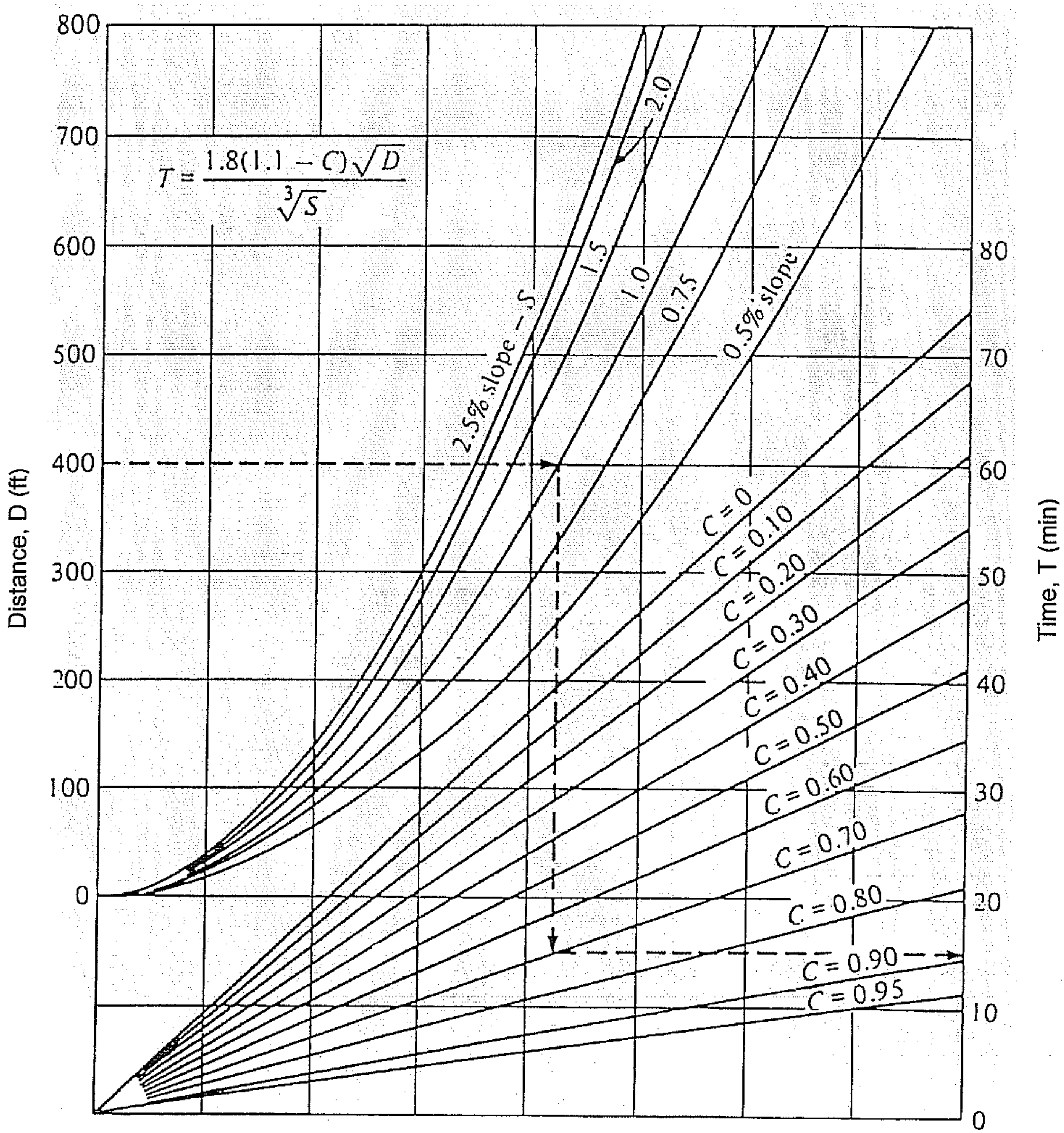
$T = \dfrac{L}{60V}$

T = time of concentration (min.)
L = length of flow (ft)
V = velocity of flow (ft/s)

SOURCE:
U.S. SOIL CONSERVATION SERVICE
TECHNICAL RELEASE #55

AVERAGE VELOCITIES FOR SHALLOW CONCENTRATED FLOW

Figure 2.3



T = Time of concentration in minutes
C = Average Runoff Coefficient
D = Length of overland flow in feet
S = Slope percentage

Note = For use in determining overland flow, time of concentration

THE CITY OF B BENTONVILLE	TIME OF CONCENTRATION NOMOGRAPH FAA METHOD Source: "Airport Drainage" Federal Aviation Agency Department of Transportation Figure 2.4

DURATION MINUTES	2 YEAR	5 YEAR	10 YEAR	25 YEAR	50 YEAR	100 YEAR
5	5.54	6.58	7.34	8.46	9.35	10.22
6	5.35	6.34	7.07	8.15	9.00	9.85
7	5.10	6.09	6.80	7.80	8.68	9.50
8	4.92	5.85	6.54	7.52	8.34	9.14
9	4.72	5.64	6.30	7.29	8.06	8.80
10	4.58	5.45	6.08	7.06	7.78	8.50
11	4.41	5.28	5.88	6.78	7.50	8.25
12	4.27	5.10	5.70	6.55	7.25	7.92
13	4.12	4.92	5.50	6.32	7.00	7.70
14	4.00	4.78	5.34	6.15	6.81	7.45
15	3.88	4.65	5.18	6.00	6.61	7.24
16	3.78	4.54	5.04	5.84	6.45	7.05
17	3.67	4.38	4.91	5.69	6.30	6.90
18	3.55	4.29	4.80	5.55	6.15	6.73
19	3.47	4.17	4.70	5.43	6.00	6.55
20	3.38	4.06	4.59	5.32	5.88	6.43
21	3.29	3.98	4.49	5.20	6.76	6.30
22	3.20	3.89	4.39	5.10	5.65	6.27
23	3.13	3.80	4.30	4.98	5.52	6.08
24	3.05	3.73	4.20	4.89	5.43	5.93
25	2.99	3.66	4.12	4.80	5.32	5.85
26	2.93	3.58	4.06	4.72	5.24	5.75
27	2.87	3.50	3.96	4.62	5.13	5.65
28	2.80	3.44	3.90	4.54	5.05	5.55
29	2.73	3.37	3.83	4.47	4.97	5.46
30	2.69	3.30	3.76	4.40	4.90	5.38
31	2.62	3.24	3.70	4.31	4.80	5.30
32	2.58	3.19	3.64	4.25	4.74	5.20
33	2.52	3.12	3.57	4.18	4.67	5.12
34	2.48	3.07	3.51	4.11	4.60	5.04
35	2.42	3.02	3.46	4.06	4.51	4.98
36	2.40	2.97	3.40	3.99	4.45	4.90
37	2.37	2.92	3.33	3.92	4.40	4.83
38	2.30	2.89	3.28	3.87	4.33	4.78
39	2.28	2.82	3.24	3.81	4.28	4.70
40	2.23	2.79	3.18	3.76	4.20	4.62
41	2.20	2.75	3.13	3.70	4.15	4.58
42	2.16	2.70	3.10	3.65	4.10	4.50
43	2.12	2.67	3.07	3.60	4.05	4.43
44	2.10	2.63	3.01	3.56	3.97	4.40
45	2.07	2.60	2.97	3.51	3.92	4.33
46	2.04	2.55	2.94	3.46	3.87	4.28
47	2.00	2.52	2.90	3.42	3.82	4.22
48	1.98	2.49	2.86	3.37	3.78	4.18
49	1.97	2.47	2.82	3.33	3.72	4.12
50	1.96	2.42	2.79	3.29	3.69	4.08
51	1.90	2.40	2.74	3.25	3.63	4.03
52	1.88	2.36	2.71	3.20	3.60	3.98
53	1.86	2.33	2.69	3.17	3.55	3.92
54	1.84	2.31	2.65	3.14	3.50	3.88
55	1.82	2.29	2.62	3.10	3.46	3.83
56	1.80	2.26	2.59	3.06	3.44	3.80
57	1.79	2.23	2.56	3.02	3.39	3.75
58	1.76	2.21	2.54	2.98	3.35	3.70
59	1.74	2.19	2.50	2.96	3.30	3.67
60	1.73	2.17	2.48	2.90	3.26	3.62
120	1.12	1.41	1.61	1.86	2.09	2.32
180	0.79	1.04	1.20	1.37	1.53	1.72
6 HR	0.48	0.62	0.73	0.84	0.93	1.03
12 HR	0.29	0.37	0.44	0.50	0.56	0.62
24 HR	0.17	0.22	0.25	0.29	0.33	0.36

Source: 5-60 min. NOAA HYDRO-35
60-120 min. interpolated
120 min. – 24 hr. Technical Paper No. 40

THE CITY OF BENTONVILLE

RAINFALL INTENSITY CHART FOR
THE CITY OF BENTONVILLE, ARKANSAS
(in inches per hour)

Figure 2.5

RUNOFF CURVE NUMBER WORKSHEET

Subbasin ______________________________

____

____

LAND USE	RCN	ACRES	RCN X ACRES
	TOTALS		

WEIGHTED RCN = $\frac{\text{Total (RCN x Acres)}}{\text{TOTAL ACRES}}$ =

DIRECT RUNOFF VALUES BY RCN'S AND RAINFALL AMOUNTS

Rainfall (inches)	Curve Number (CN)1/								
	60	65	70	75	80	85	90	95	98
1.0	0	0	0	0.03	0.08	0.17	0.32	.56	.79
1.2	0	0	0.03	0.07	0.15	0.28	0.46	.74	.99
1.4	0	0.02	0.06	0.13	0.24	0.39	0.61	.92	1.18
1.6	0.01	0.05	0.11	0.20	0.34	0.52	0.76	1.11	1.38
1.8	0.03	0.09	0.17	0.29	0.44	0.65	0.93	1.29	1.58
2.0	0.06	0.14	0.24	0.38	0.56	0.80	1.09	1.48	1.77
2.5	0.17	0.30	0.46	0.65	0.89	1.18	1.53	1.96	2.27
3.0	0.33	0.51	0.72	0.96	1.25	1.59	1.98	2.45	2.78
4.0	0.76	1.03	1.33	1.67	2.04	2.46	2.92	3.43	3.77
5.0	1.30	1.65	2.04	2.45	2.89	3.37	3.88	4.42	4.76
6.0	1.92	2.35	2.80	3.28	3.78	4.31	4.85	5.41	5.76
7.0	2.60	3.10	3.62	4.15	4.69	5.26	5.82	6.41	6.76
8.0	3.33	3.90	4.47	5.04	5.62	6.22	6.81	7.40	7.76
9.0	4.10	4.72	5.34	5.95	6.57	7.19	7.79	8.40	8.76
10.0	4.90	5.57	6.23	6.88	7.52	8.16	8.78	9.40	9.76
11.0	5.72	6.44	7.13	7.82	8.48	9.14	9.77	10.39	10.76
12.0	6.56	7.32	8.05	8.76	9.45	10.12	10.76	11.39	11.76

1/ To obtain runoff depths for CN's and other rainfall amounts not shown in this Table, use an arithmetic interpolation.

Source: U.S. Soil Conservation Service
Technical Release No. 55

Sub Area	T_c (Hr)	T_t (Hr)	Area (Sq. Mi.)	RCN	Run-Off (Inch)		DISCHARGE CSM/IN - CFS														
							Hr	Hr	Hr	Hr	Hr	Hr	Hr	Hr	Hr	Hr	Hr	Hr	Hr	Hr	Hr

THE CITY OF BENTONVILLE

TR-55 WORKSHEET
TR-55 TABULAR HYDROGRAPH METHOD

Figure 2.8

TABLE OF CONTENTS - SECTION III

SECTION III - FLOW IN STORM DRAINS AND DRAINAGE APPURTENANCES

SECTION III - FLOW IN STORM DRAINS AND DRAINAGE APPURTENANCES

3.1 GENERAL

A general description of storm drainage systems and quantities of storm runoff is contained in this Section and Section II of this manual. It is the purpose of this section to consider the significance of the hydraulic elements of storm drain system.

Hydraulically, storm drainage systems are conduits (open or closed) in which unsteady and non-uniform free flow exists. 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.

3.2 STORM SEWER DESIGN REQUIREMENTS

In preparation of storm sewer design, the following is a list of minimum requirements:

1. A plan of the drainage area that ties to the City of Bentonville monumentation network, which is available on the City of Bentonville's GIS map at www.bentonvillear.com. Contours shall be a minimum of 2 feet, regardless of the plan scale. This plan shall include all proposed street, drainage, and grading improvements with flow quantities and direction at all critical points. All areas and subareas for drainage calculations shall be clearly distinguished.

2. Complete hydraulic data showing all calculations, including a copy of all nomographs, graphs, charts and tables used for the calculations shall be submitted. Computer generated computations and output are accepted and subject to review by City Engineer.

3. A plan and profile of all proposed improvements at a scale of 1" = 20' to 1" = 50' horizontal and 1" = 5' vertical shall be submitted. This plan shall include the following: Location, sizes, flow line elevations and grades of pipes, channels, boxes, manholes and other structures drawn on standard plan-profiles; a list of the kind and quantities of materials; typical sections of all boxes and channels; location of property lines, street paving, sanitary sewers and other utilities; and standard installation details for all facilities.

4. A field study of the downstream capacity is highly suggested of all drainage facilities and the effect of additional flow from the area to be improved shall be submitted. If the effect is to endanger property or life, the problem must be resolved before the plan will be given approval.

5. Stormwater flow quantities in the street shall be shown at all street intersections and all inlet openings and locations where flow is removed from the streets. This shall include the hydraulic calculations for all inlet openings and street flow capacities. The street flow shall be limited according to Section VI, Flow in Streets.

6. Any additional information deemed necessary by the City Engineer for an adequate consideration of the storm drainage effect on the City of Bentonville and surrounding areas must be submitted.

3.3 REQUIREMENTS RELATIVE TO IMPROVEMENTS

3.3.1 DESIGN CRITERIA / STORM FREQUENCIES

Minimum design frequencies shall be as follows:

a.	Dams	100 Year
b.	BRIDGES for HIGHWAYS or ARTERIAL or COLLECTOR STREETS	100 Year
c.	Conduits for HIGHWAYS and major ARTERIAL STREETS	50 Year
d.	Conduits for minor ARTERIAL STREETS	25 Year
e.	Conduits for COLLECTOR STREETS	25 Year
f.	Conduits, other	10 Year
g.	Conduits for local or Subdivision streets	10 Year
h.	Open Channels	25 Year
i.	Sidewalk and Trail conduits	10 Year
j.	Other Land Use Areas	Refer to Street system, precipitation event table.

Note: For all scenarios, overflow limits for 100 year events shall be maintained.

Design frequencies above are minimum requirements. It is the Engineer's responsibility to comply with state and federal regulations and guidelines. It is also the Engineer's responsibility to ensure storm sewer design will not adversely impact adjacent properties.

3.3.2 BRIDGES AND CULVERTS

Bridges or culverts shall be provided where streets or alleys cross water courses and shall be designed to accommodate a <u>100</u>-year storm and meet Federal Emergency Management Agency (FEMA) regulations on FEMA regulated floodways or floodplains. Additionally, the following requirements shall be met: A 50-year frequency storm without overtopping on principal arterial roads and streets, 25-year frequency storm without overtopping for minor arterials and collectors, and a 10-year frequency storm without overtopping for all other streets. The structure shall be designed in accordance with current Arkansas Highway and Transportation Department specification materials and to carry a minimum H-20 loadings in any case.

Where same structure is to be constructed in a location other than existing or proposed street right-of-way, H-10 loadings may be used.

3.3.3 CLOSED STORM SEWER

Closed storm sewers for all conditions other than required in Section 3.3.1 above shall be designed to accommodate a 10-year frequency storm, based on the drainage area involved. Same shall either be R.C. box culverts for minimum H-20 loadings on street right-of-way or H-10 loadings elsewhere, or R.C. pipe ASTM Class III when sufficient cover is provided or ASTM Class IV when less than one-foot under paving or less than two feet of cover.

HDPE, corrugated metal, and other material pipe may be allowed at the discretion of the City Engineer.

3.3.4 MINIMUM GRADES

Storm drains should operate with velocities of flow sufficient to prevent excessive deposition of solid material; otherwise, objectionable clogging may result. The controlling velocity is near the bottom of conduits and considerably less than the mean velocity. Storm drains shall be designed to have a minimum velocity flowing full of 2.5 fps. Table 3.1 indicates the grades for both concrete pipe (n = 0.012) and for corrugated metal pipe (n = 0.024) to produce a velocity of 2.5 fps, which is considered to be the lower limit of scouring velocity. Grades for closed storm sewers and open paved channels shall be designed so that the velocity shall not be less than 2.5 fps nor exceed 12 fps. All other structures such as junction boxes or inlets shall be in accordance with City standard drawings. The minimum slope for standard construction procedures shall be 0.50 percent when possible. Any variance must be approved specifically in writing by the City Engineer.

Table 3.1
Minimum Slope Required
to Produce Scouring Velocity

Pipe Size (Inches)	Concrete Pipe Slope ft./ft.	Corrugated Metal Pipe 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

Closed storm sewers extending to farthest downstream point of development shall give consideration to velocities and discharge energy dissipaters to prevent erosion and scouring along downstream properties.

3.3.5 OPEN DITCHES (EARTH CHANNELS)

Open earth ditches shall be designed to carry the 25-year frequency storm and to accommodate the 100-year frequency storm without encroaching on existing buildings, infrastructures, or improvements. The 100-year water surface elevation must not be increased in conjunction with the ditch.

Ditches shall have a gradient to keep the velocity within 1.5 to 5.0 feet per second in unpaved channels unless approved by City Engineer. **Sod shall be required to the 25-year storm depth unless approved by the City Engineer. Side slopes shall have a minimum slope ratio of 3:1 unless approved specifically in writing by the City Engineer.** Designer's attention is directed to the fact that the Subdivision Ordinance prohibits encroachment of buildings and improvements on natural or designated drainage channels, or the channel's floodways. Floodplains are areas of land adjacent to an open channel (not in closed storm sewers) that may flood during a 100-year rain. Such floodways and floodplains shall be indicated on drainage improvement plans and individual plot plans.

3.3.6 OPEN PAVED CHANNELS

Open paved channels are to be used where flow velocity exceeds 5 fps or channel grade is less than 1.00%, unless approved by the City Engineer. Open paved storm drainage channels shall be designed to carry a 25-year frequency storm and to accommodate a 100-year frequency storm without encroaching on existing buildings, infrastructures, or improvements. Such channels may be of different shapes according to existing conditions. **The channel shall be of concrete with a minimum four-inch thickness paved to a point 1' above the 25-year storm depth. Six-inch minimum thickness is required where maintained by machinery.** Thickness of concrete and amount of reinforcing steel shall depend upon conditions at site and size of channel. Gabion or riprap lined channels may be used in place of paved channels where approved by the City Engineer.

3.4 FULL OR PART FULL FLOW IN STORM DRAINS

3.4.1 GENERAL

The size of closed storm sewers, open channels, culverts and bridges shall be designed so that their capacity will not be less than the volume computed by using the Manning Formula. All storm drains shall be designed by the application of the continuity equation and Manning Formula either through the appropriate charts and nomographs, or by direct solutions of the equations as follow:

$Q = AV$ and

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

Q = Capacity = discharge in cubic feet per second

A = Cross-sectional area in conduit or channel in square feet

R = Hydraulic radius = $A \div P$

P = Wetted perimeter (feet)

S_o = Slope of pipe (feet per feet)

S_f = Friction slope of energy grade line

n = Coefficient of roughness of pipe

V = Velocity in pipe (feet per second)

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 follow:

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. An 18" pipe diameter is the minimum acceptable pipe diameter for maintenance purposes. Where used, arch pipe sizes shall be hydraulically equivalent to the round pipe size.

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 sizes, match the soffits or crown 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 slopes should be kept below critical slope if at all possible. This also removes the possibility of a hydraulic jump within the line.

3.4.2 PIPE FLOW CHARTS

Pipe flow charts are nomographs for determining flow properties in circular pipe, elliptical pipe and pipe-arches. Figures 3.1 through 3.9 are nomographs based upon a value of "n" of 0.024 for corrugated metal and 0.012 for concrete. The charts are self-explanatory, and their use is demonstrated by the example in Figure 3.1.

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

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

Q_c = Flow based upon n_c

n_c = Value of "n" other than 0.012

Q_n = Flow from 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 the known properties, you must use the formula to convert Q_c (based on n_c) to Q_n (based on 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 3.1 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_n = 22.7 (0.018)/(0.012) = 34 cfs, enter Figure 3.1 to determine d/D = 0.6. Now enter Figure 3.3 to determine V = 7.5 fps (answer).

3.4.3 ROUGHNESS COEFFICIENTS

Roughness coefficients for storm drains are as follows on Table 3.2.

Table 3.2

Roughness Coefficients "n" for Storm Drains

Materials of Construction	Design Manning Coefficient	Range of Manning Coefficient
Concrete Pipe	0.013	0.011-0.015
Corrugated Metal Pipe		
o Plain or Coated	0.024	0.022-0.026
o Paved Invert	0.020	0.018-0.022

3.4.4 MANHOLE LOCATIONS

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 a curved section of storm sewer. The maximum spacing between manholes for various pipe sizes shall be in accordance with the Chart below. The required manhole size shall be as follows:

Table 3.3
Manhole Size

Sewer Diameter	Round Manhole Inside Diameter
18" – 24"	4'
27" – 36"	5'
Larger than 36"	Not Allowed

Sewer Diameter	Rectangular Manhole Inside Lengths
18" – 36"	4' x 4'
42" – 48"	5' x 5'
Larger than 48"	Approved by City

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.

Table 3.4
Storm Sewer Alignment
And Size Criteria

Vertical Dimension of Pipe (inches	Maximum Allowable Distance Between Manholes and/or Cleanouts
18 and larger	500 feet

3.4.5 PIPE CONNECTIONS

Connections will be made by inlet or junction boxes. Precast structures are not allowed in public drainage systems.

3.4.6 MINOR HEAD LOSSES AT STRUCTURES

The following total energy head losses at structures shall be determined for inlets, manholes, wye branches or bends and other junctions in the design of closed conduit. See Figures 3.10 and 3.11 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 are both upstream and downstream velocity, takes the form as set forth below with the various values of the coefficient of K_j shown in Tables 3.5, 3.6 and 3.7.

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

h_j = junction or structure head loss in feet.

v_1 = velocity in upstream pipe in feet per second.

v_2 = velocity in downstream pipe in feet per second.

K_j = junction or structure coefficient of loss.

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

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

Short radius bends may be used on 24 inch or larger pipes where 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 downstream end of such short radius bends.

The values of the coefficient "K_j" for determining the loss of head due to obstructions in pipe are shown in Table 3.6 and the coefficients are used in the following equation to calculate the head loss at the obstruction:

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

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 3.7 and the coefficients are used in the following equation to calculate the head loss at the change in Section:

$$h_j = \frac{K_j V^2}{2g}$$

3.5 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, sanitary sewer, electric, and communication lines.

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

Due to the difficulty and expense to the public with regard to hand cleaning, clearing, and other ditch maintenance, the following ditch requirements are specified to expedite small equipment cleaning and access to drainage easements and ditches:

- Manholes are not allowed in drainage ditches, unless approved by the City Engineer.
- Access easements shall be required every 600 feet. Access to be provided from public street to drainage facility.
- Utility crossings above the channel flowline shall not be allowed unless approved specifically in writing by the City Engineer.
- Utilities shall not be located beneath a concrete ditch bottom except at crossings.
- Minimum drainage easement width shall be 20'.

See Figure 3.12 for dimensions of utility easements required when drainage facilities are installed within the same easement.

Table 3.5

Junction or Structure
Coefficient of Loss

Case No.	Reference Figure	Description of Condition	Coefficient K_j
I		Inlet on Main Line **	0.50
II		Inlet on Main Line with Branch Lateral **	0.25
III		Manhole on Main Line with 45° Branch Lateral	0.50
IV		Manhole on Main Line with 90° Branch Lateral	0.75
V		45° Wye Connection or Cut-in	0.25
VI		Inlet on Manhole at Beginning of Line	1.25
VII		Conduit on Curves for 90° ***	
		Curve Radius = Diameter	0.50
		(2 to 8) Diameter	0.40
		Curve Radius = (8 to 20) Diameter	0.25
VIII		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

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

** Must be approved by City Engineer.
*** 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%

TABLE 3.6

Head Loss Coefficients Due To Obstructions

A* / A.	K_j	A.	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	10.0	121.0
2.5	9.70		

* $\frac{A}{A.}$ = Ratio of area of pipe to opening at obstruction.

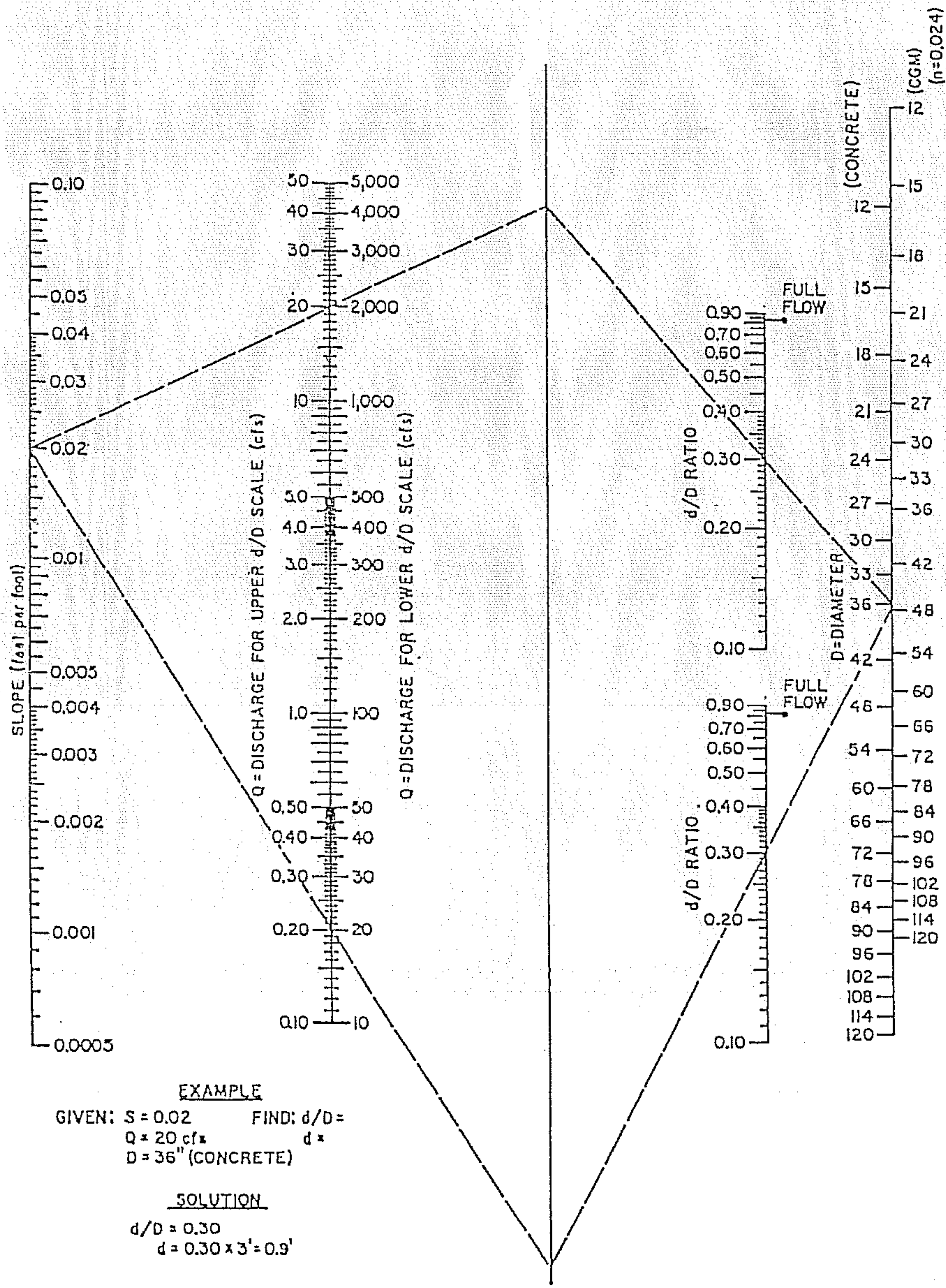
TABLE 3.7

Head Loss Coefficients Due To Sudden Enlargements And Contractions

D_2** / D_1	Sudden Enlargements K_j	Sudden Contractions K_j
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

**$\frac{D_2}{D_1}$ = Ratio of larger to smaller diameter.

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



EXAMPLE

GIVEN: S = 0.02 FIND: d/D =
Q = 20 cfs d =
D = 36" (CONCRETE)

SOLUTION

d/D = 0.30
d = 0.30 x 3' = 0.9'

SOURCE: AHTD

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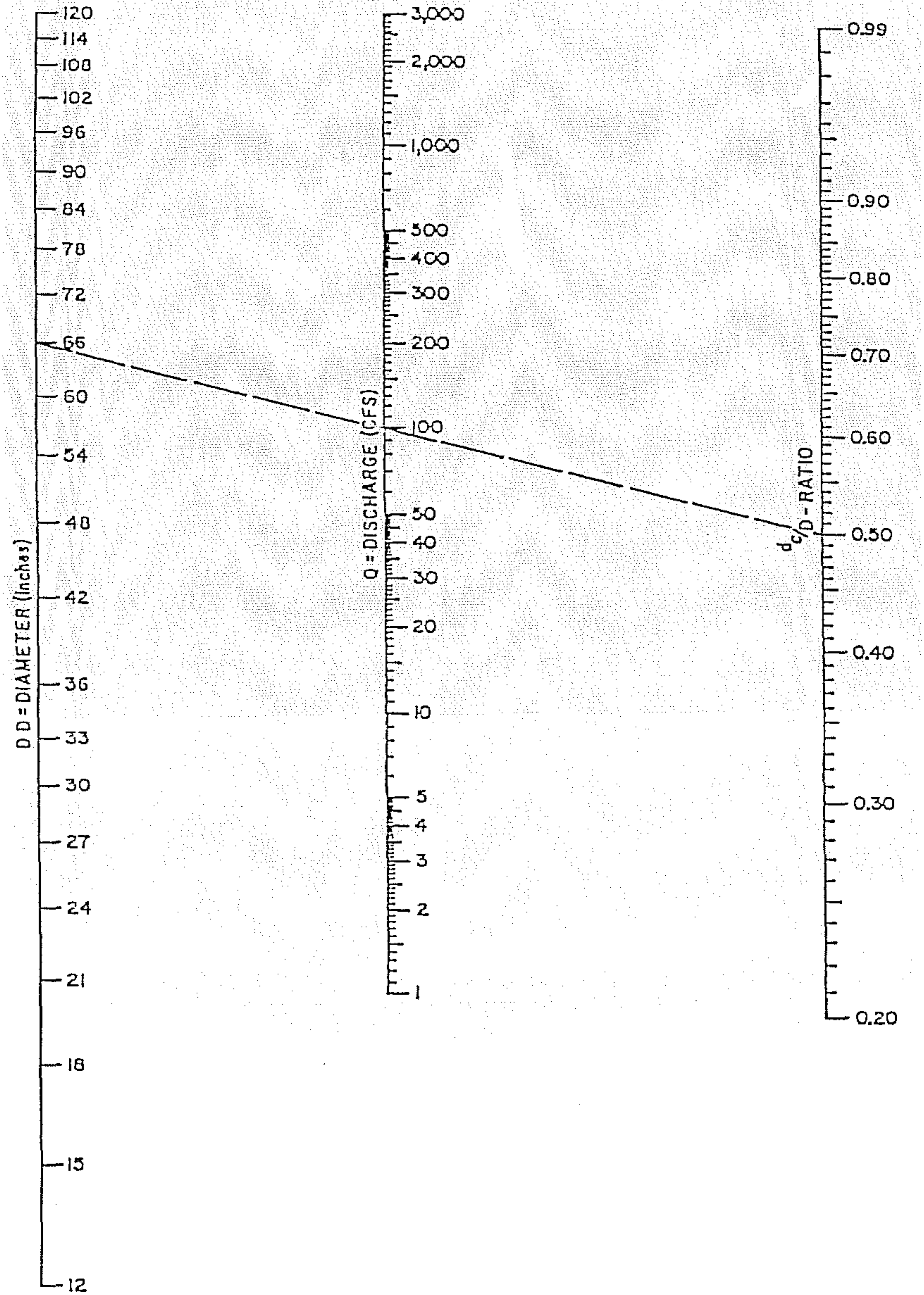
UNIFORM FLOW FOR PIPE CULVERTS

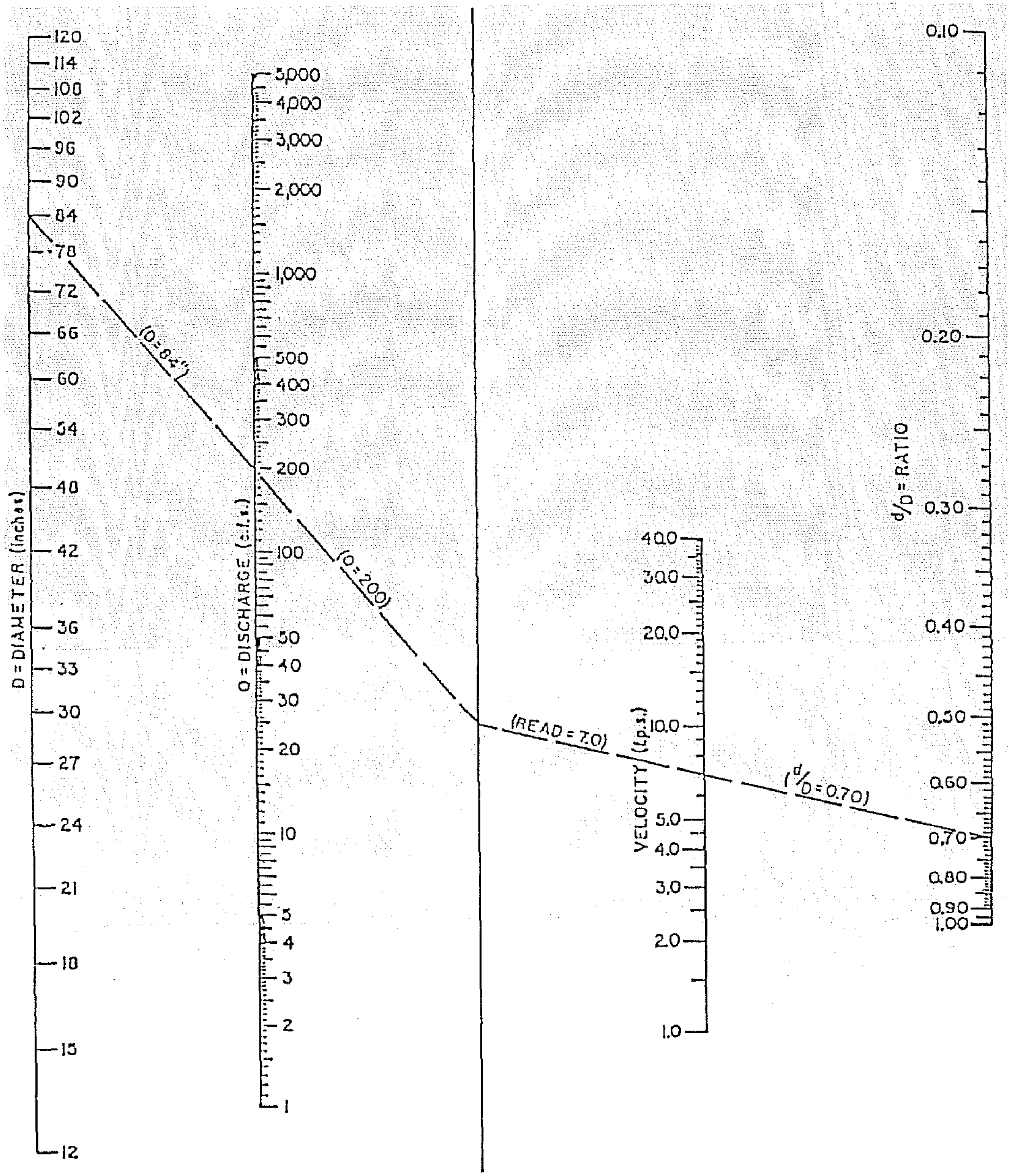
Figure 3.1



CRITICAL DEPTH OF FLOW FOR CIRCULAR CONDUITS

SOURCE: AHTD

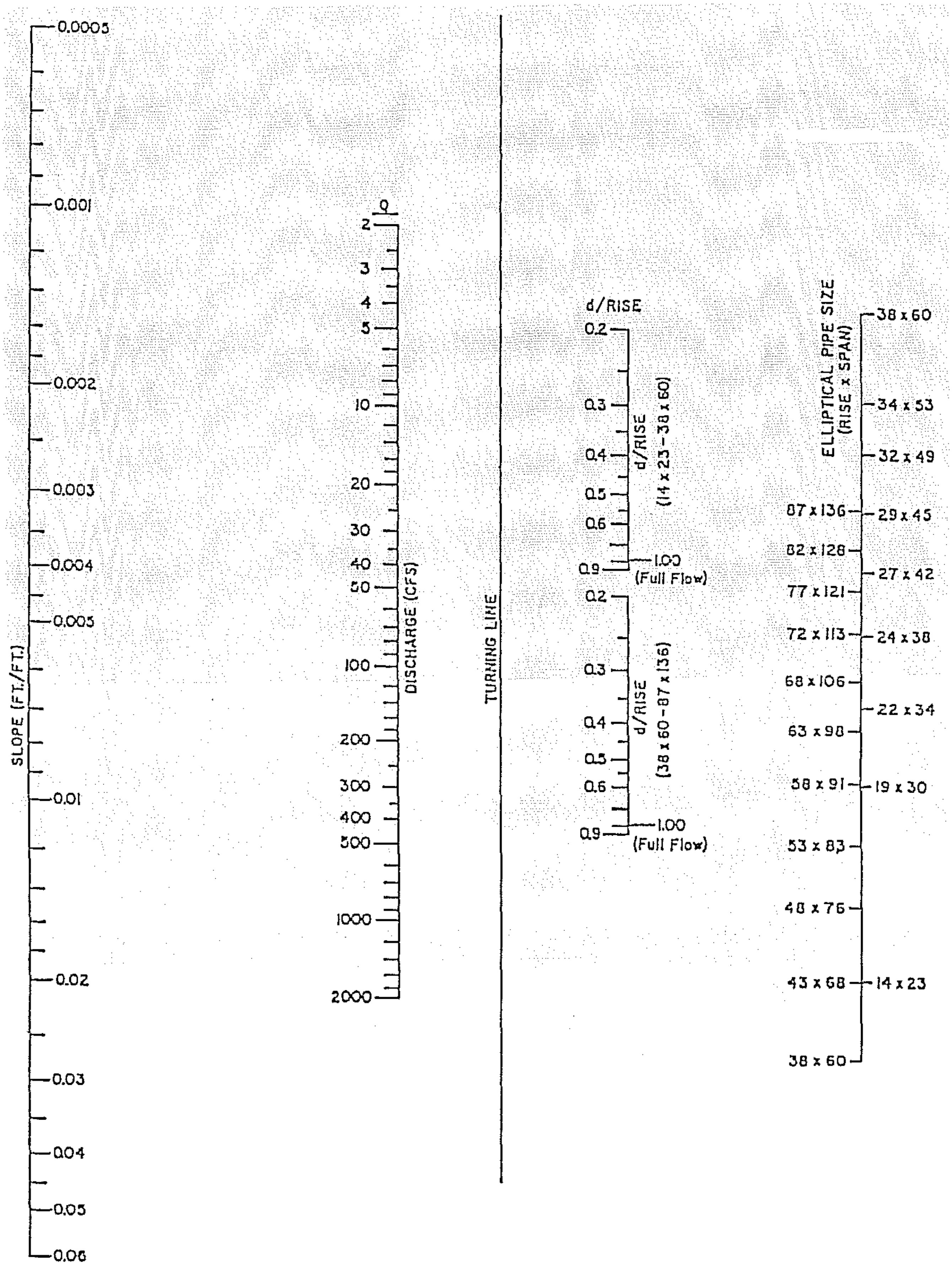
Figure 3.2



VELOCITY IN PIPE CONDUITS

Source: AHTD for Figures 3.3 – 3.9

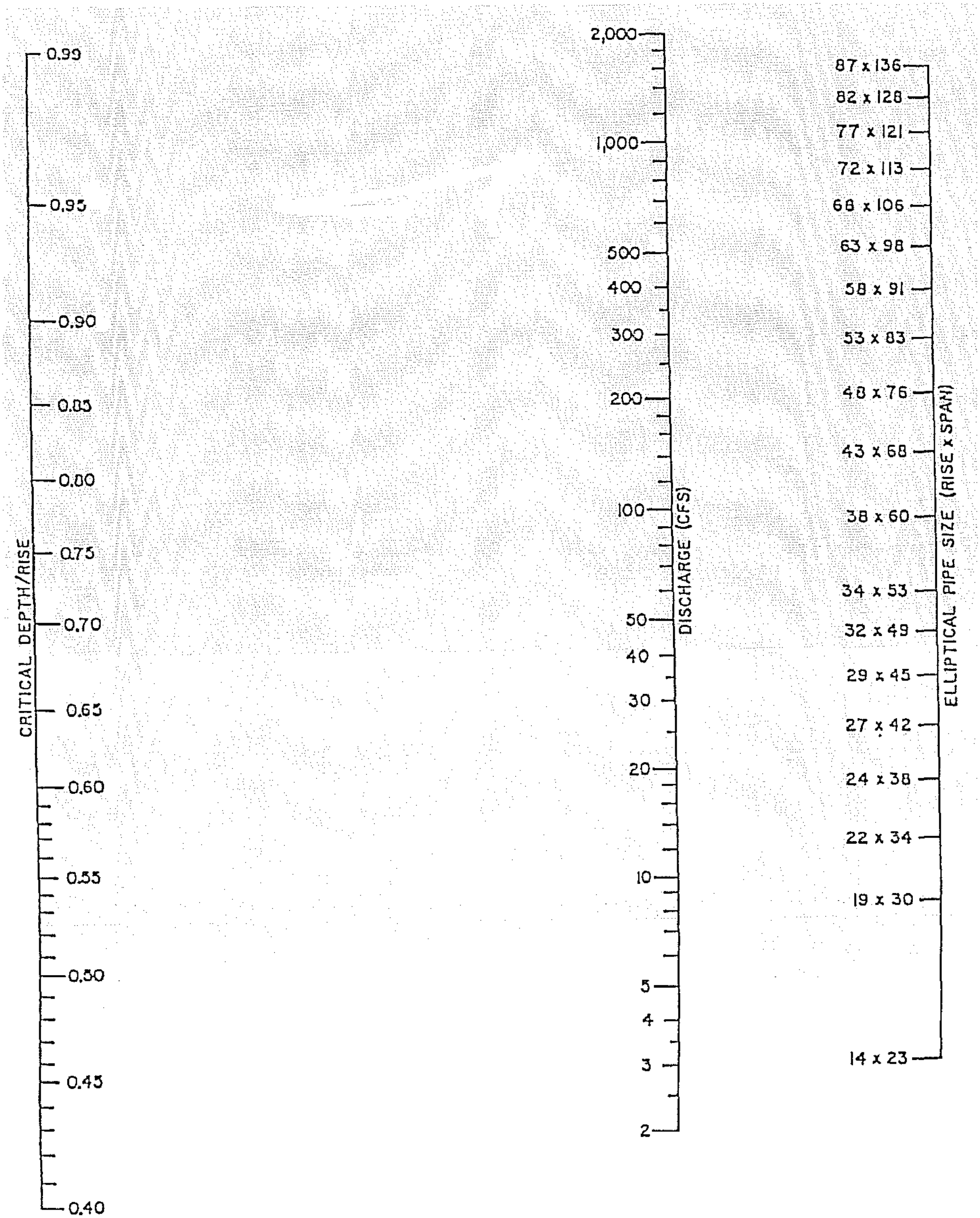
Figure 3.3



THE CITY OF BENTONVILLE

UNIFORM FLOW FOR CONCRETE ELLIPTICAL PIPE

Figure 3.4



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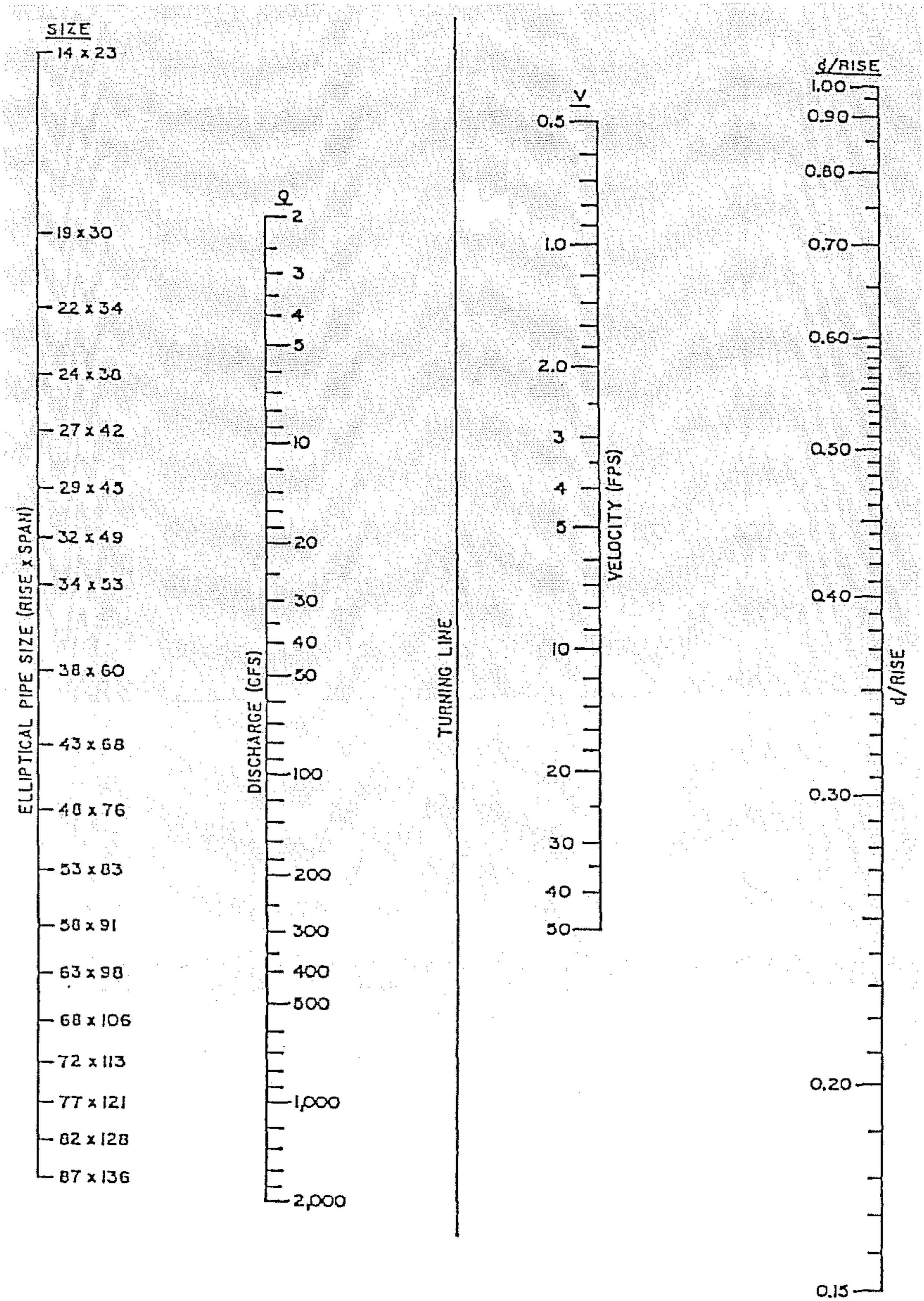
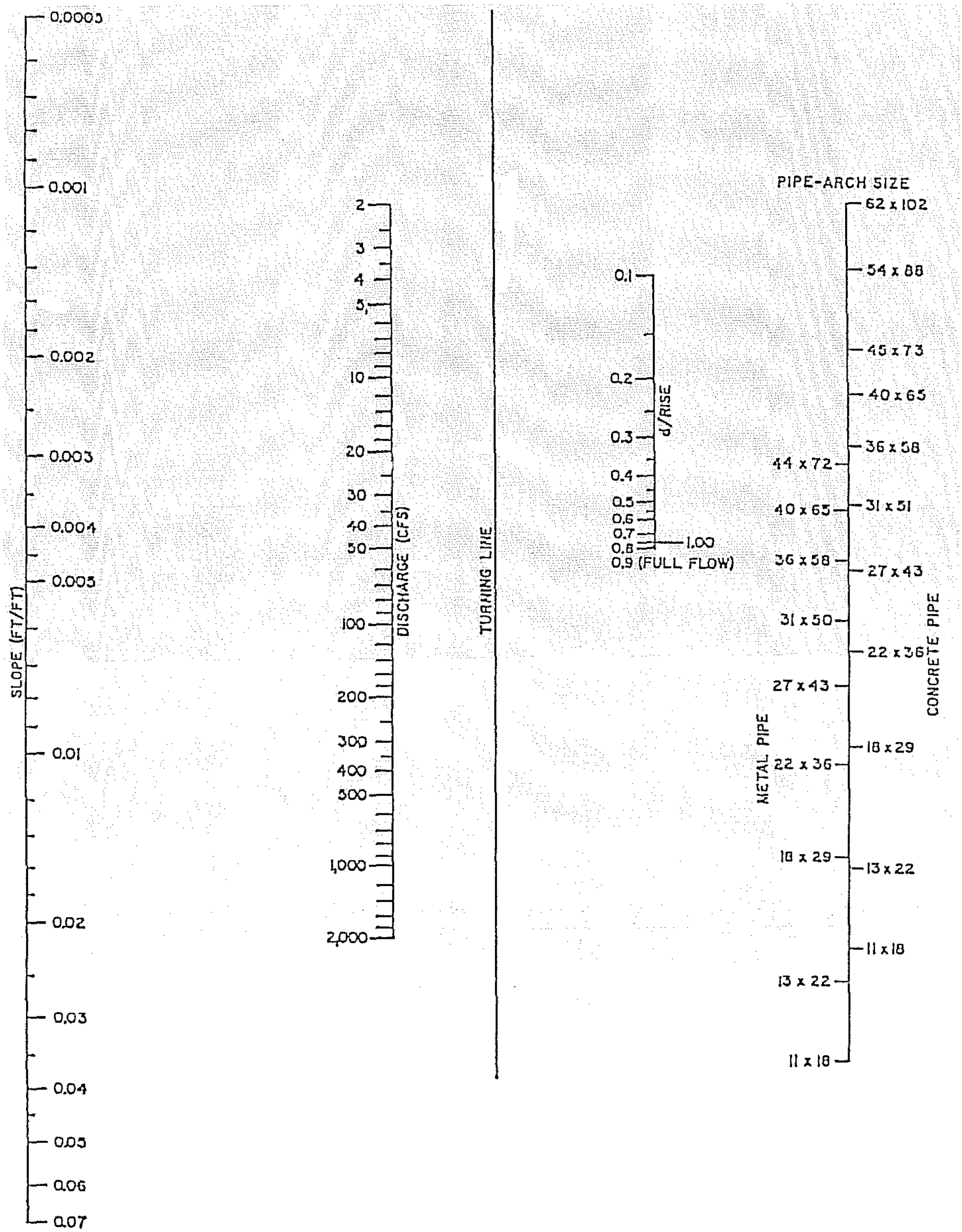
CRITICAL DEPTH FOR ELLIPTICAL PIPE

Figure 3.5



VELOCITY IN ELLIPTICAL PIPE

Figure 3.6



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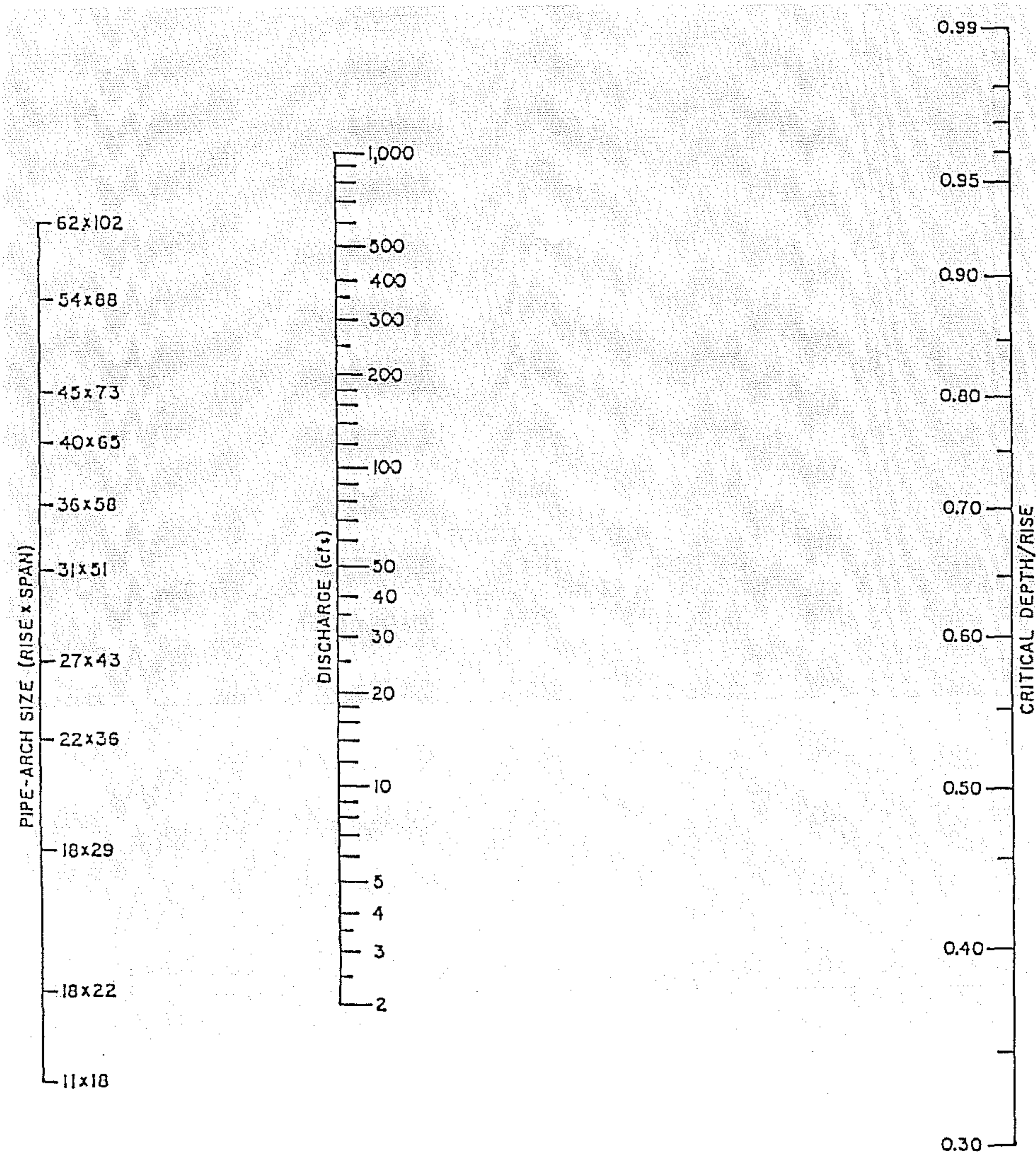
UNIFORM FLOW FOR ARCH PIPE

Figure 3.7



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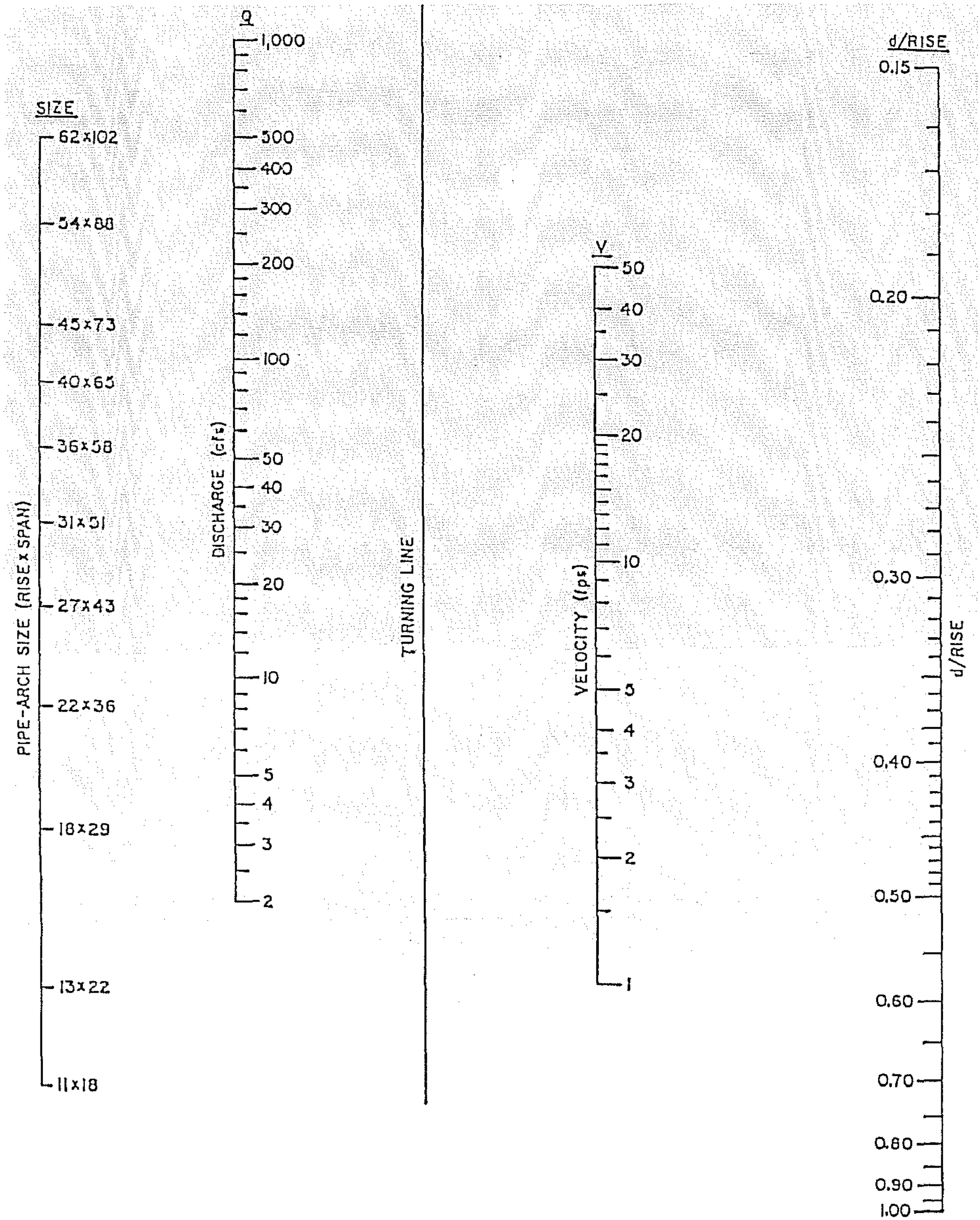
CRITICAL DEPTH FLOW FOR ARCH PIPE

Figure 3.8



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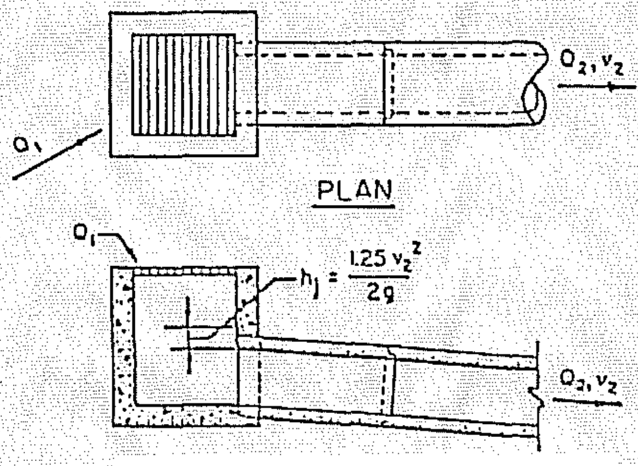
VELOCITY IN ARCH PIPE

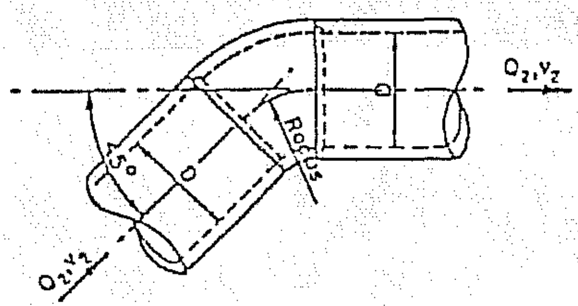
Figure 3.9



SECTION

CASE VI

INLET OR MANHOLE AT BEGINNING OF LINE



CASE VIII

BENDS WHERE RADIUS IS EQUAL TO DIAMETER OF PIPE

(AS APPROVED CASE BY CASE)

NOTE: Head loss applied at beginning of bend.

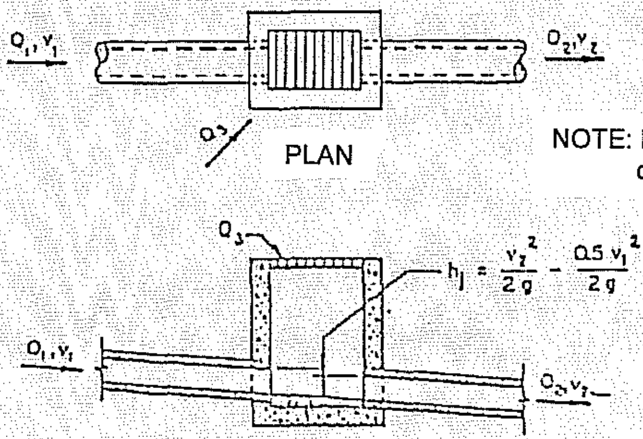
90° Bend $h_j = 0.50 \frac{v_2^2}{2g}$

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

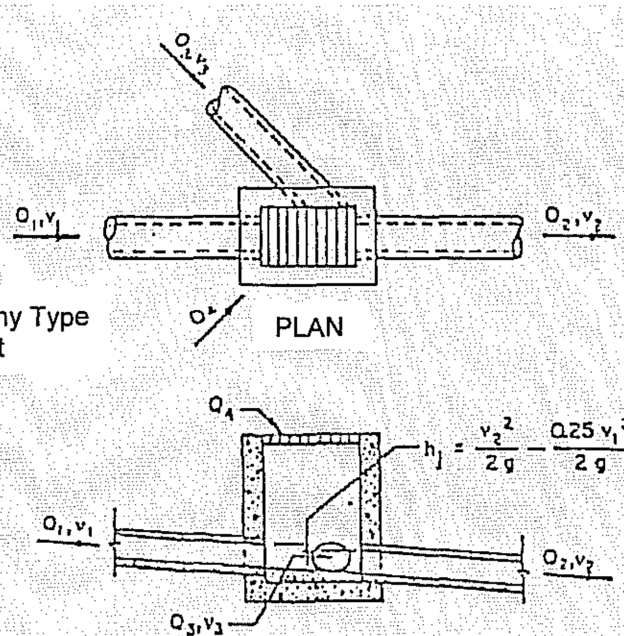
45° Bend $h_j = 0.35 \frac{v_2^2}{2g}$

22 1/2° Bend $h_j = 0.20 \frac{v_2^2}{2g}$

SOURCE: City of Austin, Tx.



NOTE: For Any Type of Inlet



SECTION

CASE I

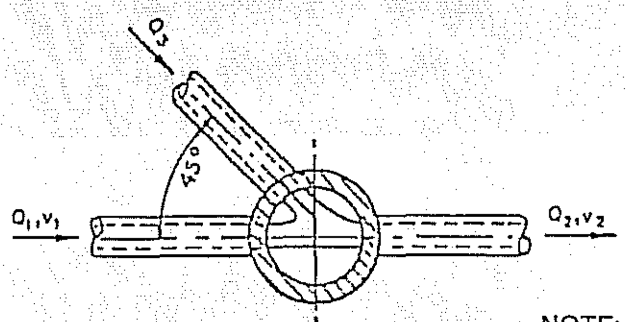
INLET ON MAIN LINE

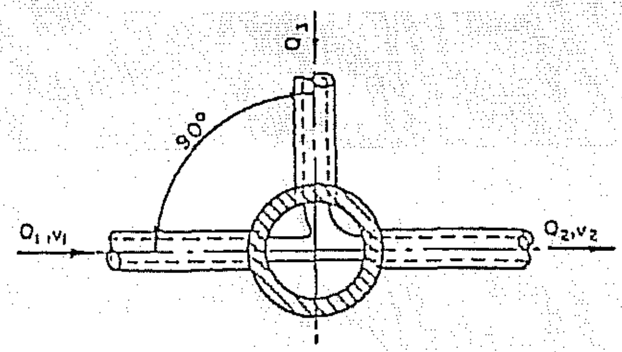
SECTION

CASE II

INLET ON MAIN LINE

WITH BRANCH LATERAL





NOTE:

60° Lateral $h_j = \frac{v_2^2}{2g} - \frac{0.35 v_1^2}{2g}$

22 1/2° Lateral $h_j = \frac{v_2^2}{2g} - \frac{0.75 v_1^2}{2g}$

SECTION

CASE III

MANHOLE ON MAIN LINE

WITH 45° BRANCH LATERAL

SECTION

CASE IV

MANHOLE ON MAIN LINE

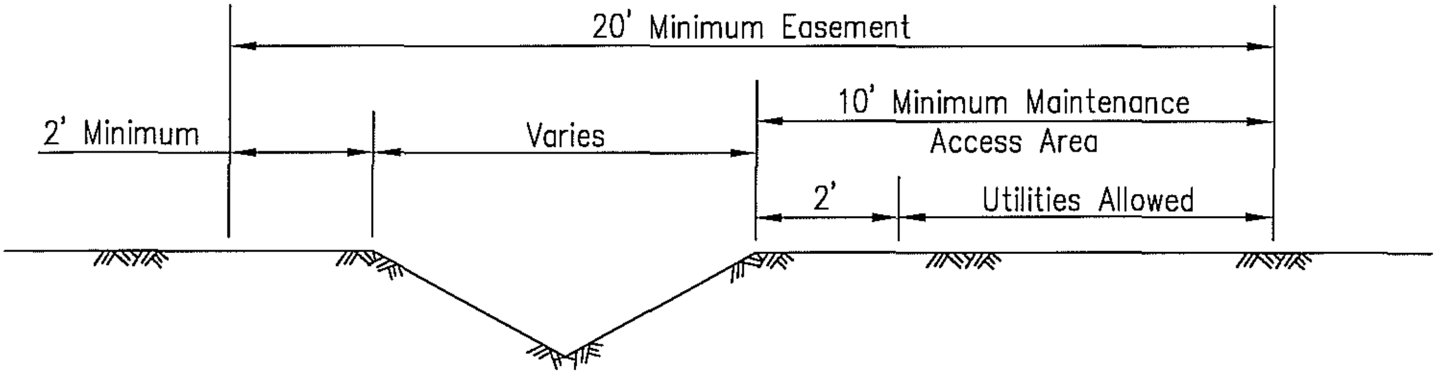
WITH 90° BRANCH LATERAL

SOURCE: City of Austin, Tx.

THE CITY OF BENTONVILLE

MINOR HEAD LOSSES DUE TO TURBULENCE AT STRUCTURES

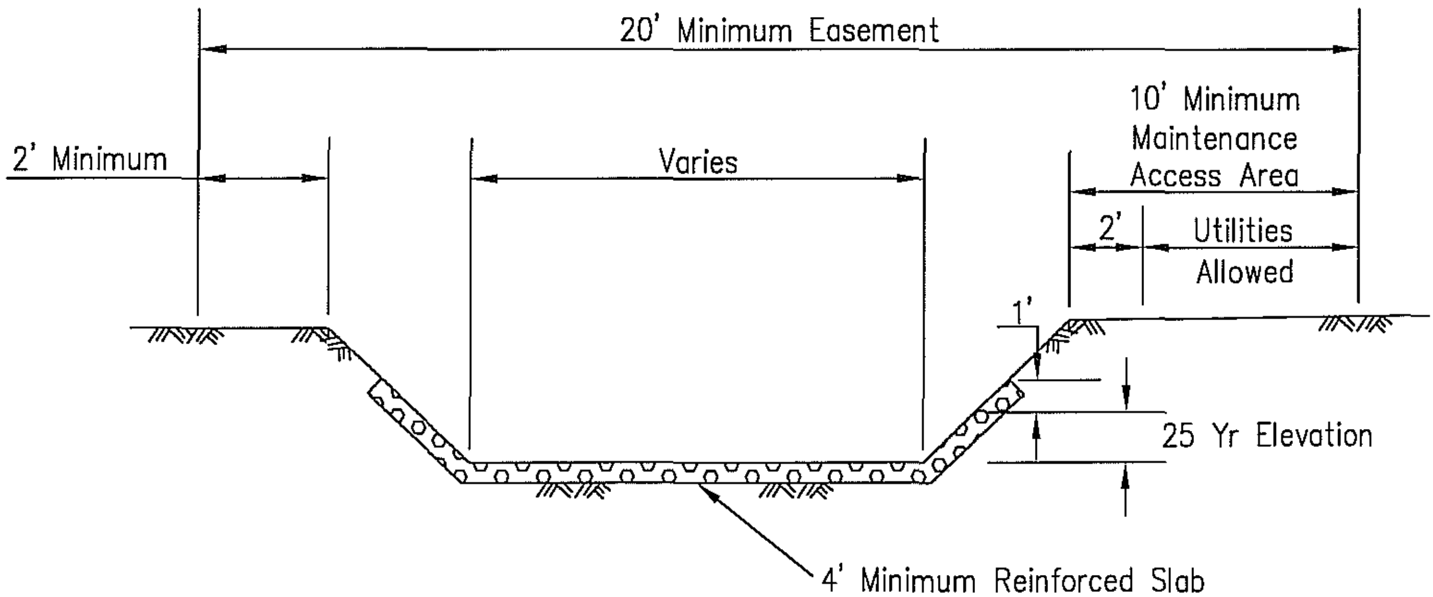
Figure 3.11



SMALL DITCH

GENERAL NOTES

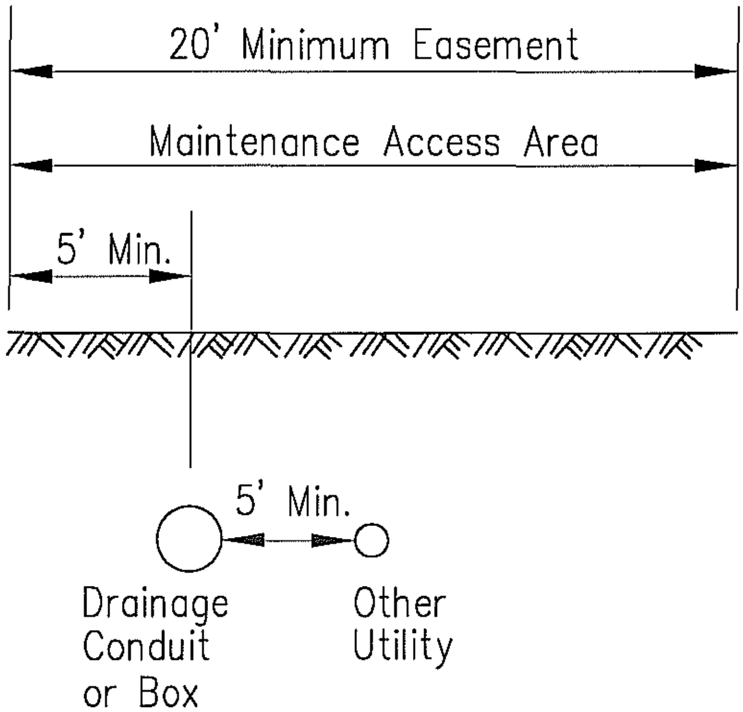
* Utility crossing limited to one per block
* Access easement required every 600' (from public street to facility)
* Utilities shall not be located beneath a a concrete bottom except at crossings
* Manholes not allowed in ditches



LARGE DITCH

MINIMUM EASEMENTS FOR DRAINAGE AND UTILITIES

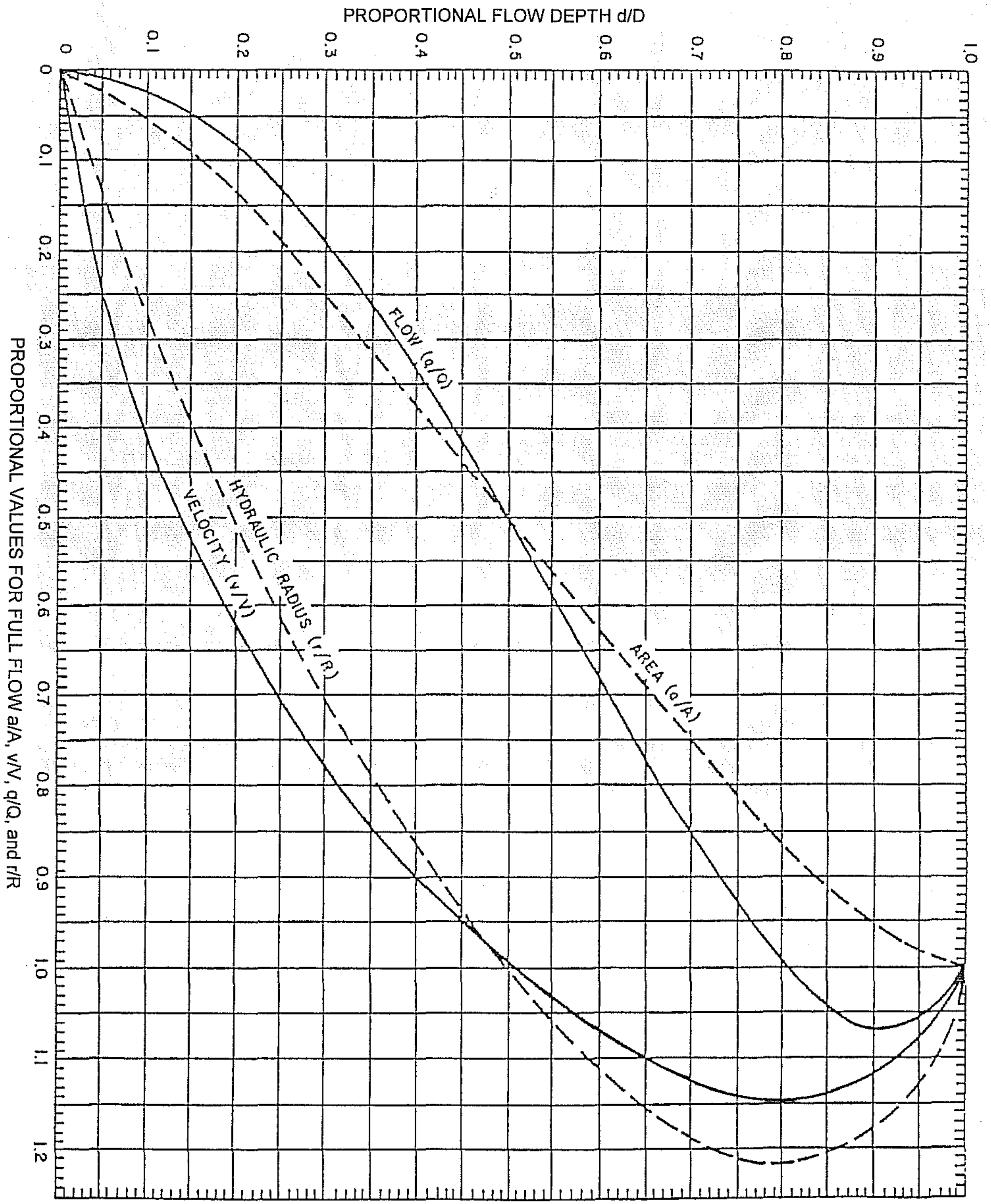
Figure 3.12



CONDUIT OR BOX

MINIMUM EASEMENTS FOR
DRAINAGE AND UTILITIES

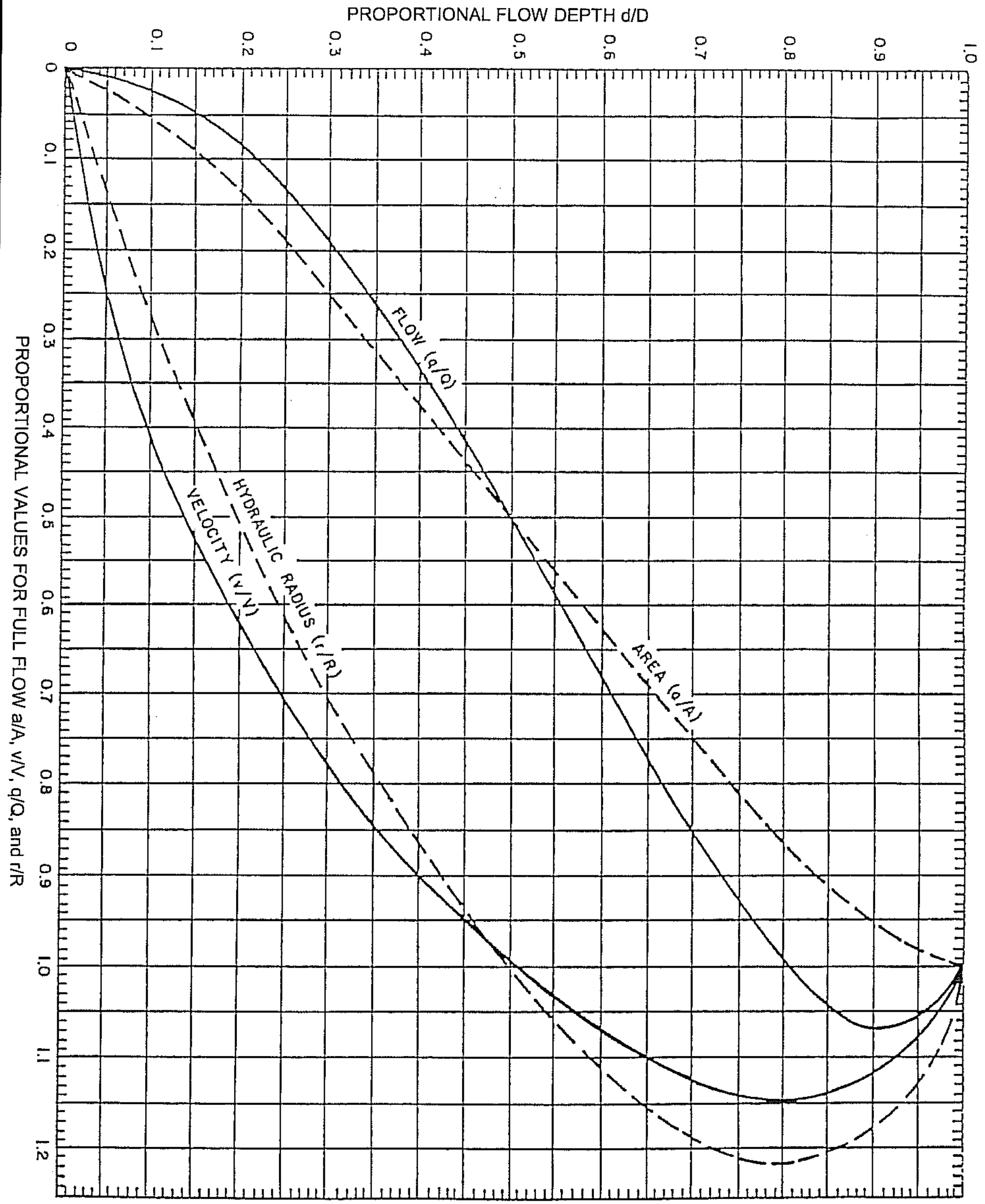
Figure 3.12 (cont)



THE CITY OF BENTONVILLE

RELATIVE VELOCITY, AREA, AND DISCHARGE
IN A CIRCULAR PIPE FOR ANY DEPTH OF FLOW

Figure 3.13



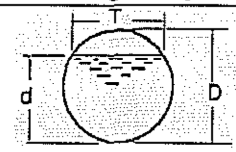
THE CITY OF BENTONVILLE

RELATIVE VELOCITY, AREA, AND DISCHARGE
IN A CIRCULAR PIPE FOR ANY DEPTH OF FLOW

Figure 3.13

Hydraulic Properties of Circular Conduits Flowing Partly Full

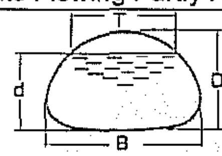
D = Depth of Flow, D = Diameter of pipe
d_c = Critical depth, A = Area of Flow
d_m = Mean depth, R = Hydraulic radius
T = Top width of flow



$\frac{d}{D}$ or $\frac{d_c}{D}$	$\frac{A}{D^2}$	$\frac{R}{D}$	$\frac{T}{D}$	$\frac{d_m}{D}$
1.00	0.7854	0.2500	----	----
0.95	0.7707	0.2865	0.4359	1.7681
0.90	0.7445	0.2980	0.6000	1.2408
0.85	0.7115	0.3033	0.7142	0.9962
0.80	0.6736	0.3042	0.8000	0.8420
0.75	0.6319	0.3017	0.8660	0.7297
0.70	0.5872	0.2962	0.9165	0.6407
0.65	0.5404	0.2882	0.9539	0.5665
0.60	0.4920	0.2776	0.9798	0.5021
0.55	0.4426	0.2649	0.9950	0.4448
0.50	0.3927	0.2500	1.0000	0.3927
0.45	0.3428	0.2331	0.9950	0.3445
0.40	0.2934	0.2142	0.9798	0.2994
0.35	0.2450	0.1935	0.9539	0.2568
0.30	0.1982	0.1709	0.9165	0.2163
0.25	0.1535	0.1466	0.8660	0.1773
0.20	0.1118	0.1206	0.8000	0.1397
0.15	0.0739	0.0929	0.7142	0.1035

Hydraulic Properties of Pipe Arch Conduits Flowing Partly Full

D = Depth of Flow, D = Diameter of pipe
d_c = Critical depth, A = Area of Flow
d_m = Mean depth, R = Hydraulic radius
T = Top width of flow



$\frac{d}{D}$ or $\frac{d_c}{D}$	$\frac{A}{BD}$	$\frac{R}{D}$	$\frac{T}{D}$	$\frac{d_m}{D}$
1.00	0.7879	0.2991	----	----
0.95	0.7762	0.3408	0.3489	2.225
0.90	0.7552	0.3549	0.4855	1.555
0.85	0.7283	0.3622	0.5848	1.245
0.80	0.6970	0.3649	0.6637	1.0503
0.75	0.6621	0.3639	0.7288	0.9085
0.70	06243	0.3595	0.7837	0.7966
0.65	0.5839	0.3520	0.8303	0.7033
0.60	0.5414	0.3415	0.8700	0.6223
0.55	0.4970	0.3282	0.9037	0.5500
0.50	0.4511	0.3120	0.9320	0.4840
0.45	0.4039	0.2928	0.9555	0.4227
0.40	0.3556	0.2705	0.9755	0.3616
0.35	0.3065	0.2451	0.9889	0.3100
0.30	0.2568	0.2162	0.9967	0.2577
0.25	0.2069	0.1839	0.9967	0.2076
0.20	0.1574	0.1484	0.9815	0.1603
0.15	0.10908	0.11022	0.9477	0.11505

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Hydraulic Properties of Circular Conduits Flowing Partly Full
Hydraulic Properties of Pipe Arch Conduits Flowing Partly Full

Figure 3.14

<u>TABLE OF CONTENTS - SECTION IV</u>

SECTION IV - CULVERT HYDRAULICS

4.0 GENERAL

The function of a drainage culvert is to pass the design storm flow under a roadway or railroad without causing excessive backwater and without creating excessive downstream velocities. The designer shall keep energy losses and discharge velocities within reasonable limits when selecting a structure.

Culvert flow may be separated into two major types of flow - inlet or outlet control. Under inlet control, the cross sectional area of the barrel, the shape of the inlet and the amount of ponding (headwater) at the inlet are primary design considerations. Outlet control is dependent upon the depth of water in the outlet channel (tailwater), the slope of the barrel, type of culvert material and length of the barrel.

4.1 INLET CONTROL

The size of a culvert operating with inlet control is determined by the size and shape of the inlet and the depth of ponding allowable (headwater) between the flowline elevation of a culvert and the elevation of a finished grade surface or surrounding buildings and facilities. See Figure 4.1. Factors not effecting inlet control design are the barrel roughness, slope and length and depth of the tailwater.

The headwater (HW) depth for a culvert of a given diameter or height (D) where a discharge is given can be determined by obtaining the HW/D value from current Hydraulic Design Series #5, FHWA. A desirable maximum headwater for a culvert should not be greater than the diameter or height plus 2'. The elevation of adjacent facilities (i.e., buildings, etc.) must be reviewed for flooding.

4.2 OUTLET CONTROL

A culvert will operate under outlet control when the depth of the tailwater, the length, the slope or roughness of the culvert barrel act as the control on the quantity of water able to pass through a given culvert. See Figure 4.2. Energy head required for a culvert to operate under outlet control is comprised of velocity head (H_v), entrance loss (H_e) and friction loss (H_f). This energy head (H) is obtained from current Hydraulic Design Series #5, FHWA, and entrance loss coefficients from Table 4.1.

The headwater depth (HW) at the culvert entrance is calculated by means of the following formula:

$HW = H + h_o - LS_o$

Where: H = energy head

L = length of culvert (ft.)

S_o = slope of barrel (feet per foot)

$h_o = \frac{d_c + D}{2}$ or TW, whichever is greater

d_c = critical depth of flow in the barrel.
Critical depth may be determined by using Hydraulic Design Series #5, FHWA.

D = height of pipe or box

TW = tailwater depth

The maximum desirable headwater depth for culverts operating under outlet control shall be the same as described in Section 4.1.

See Section 4.7 for detailed types of culvert flow. See Section 4.8 and Examples 1-6 for examples of culvert sizing computations. Computer generated computations and output are accepted and subject to review by City Engineer.

4.3 HEADWALLS AND ENDWALLS

4.3.1 GENERAL

The normal functions of properly designed headwalls and end walls are to anchor the culvert, to prevent movement due to the lateral pressures, to control erosion and scour resulting from excessive velocities and turbulence, and to prevent adjacent soil from sloughing into the waterway opening. Headwalls shall be constructed of reinforced concrete and may either be straight parallel headwalls, flared headwalls, or warped headwalls with or without aprons as may be required by site conditions. Multi-barrel culvert crossings of roadways at an angle of 15° or greater shall be accompanied by adequate inlet and outlet control sections.

4.3.2 CONDITIONS AT ENTRANCE

It is important to recognize that the operational characteristics of a culvert may be completely changed by the shape or condition at the inlet or entrance. Design of culverts involve consideration of energy losses that occur at the entrance. The entrance head losses may be determined by the following equation:

$$h_e = \frac{K_e \ (V_2^2 - V_1^2)}{2g}$$

h_e = entrance head loss in feet

V_2 = velocity of flow in culvert

V_1 = velocity of approach in feet per sec.

K_e = entrance loss coefficient as shown in Table 4.1.

TABLE 4.1

VALUES OF ENTRANCE LOSS COEFFICIENTS "K_e"

Type of Structure & Entrance Design	value of K_e
Box, Reinforced Concrete	
Submerged Entrance	
Parallel wing walls	0.5
Flared wing walls	0.4
Free Surface Flow	
Parallel wing walls	0.5
Flared wing walls	0.15
Pipe, Concrete	
Project from fill, socket end	0.2
Project from fill, square cut end	0.5
Headwall or headwall & wingwalls	
Socket end of pipe	0.2
Square – edge	0.5
End - Section conforming to fill slope	0.5
Pipe, or Pipe-Arch, Corrugated Metal	
Projecting from fill (No headwall)	0.9
Headwall or headwall and wingwalls	
Square – Edge	0.5
End - Section conforming to fill	0.5

4.3.3 SELECTION OF HEADWALL OR ENDWALL

In general, the following guidelines should be used in the selection of the type of headwalls or endwalls.

Straight/Parallel Headwalls and Endwalls

(1) Approach velocities are low (below 6 feet per sec.).

(2) Backwater pools may be permitted.

(3) Approach channel is undefined.

(4) Ample right-of-way or easement is available.

(5) Downstream channel protection is not required.

Flared Headwall and Endwall:

(1) Channel is well defined.

(2) Approach velocities are between 6 and 10 feet per second.

(3) Medium amounts of debris exists.

The wings of flared walls should be located with respect to the direction of the approaching flow instead of the culvert axis.

Warped Headwall and Endwall:

(1) Channel is well defined and concrete lined.

(2) Approach velocities are between 8 and 20 feet per second.

(3) Medium amounts of debris exists.

These headwalls are effective with drop down aprons to accelerate flow through the culvert, and are effective for transitioning flow from closed conduit flow to open channel flow. This type of headwall should be used only where the drainage structure is large and right-of-way or easement is limited.

4.4 CULVERT DISCHARGE VELOCITIES

The velocity of discharge from culverts should be limited as shown in Table 4.2. Consideration must be given to the effect of high velocities, eddies, or other turbulence on the natural channel, downstream property, and roadway embankment.

Table 4.2
Culvert Discharge - Velocity Limitations

Downstream Condition	Maximum Allowable Discharge Velocity (FPS)
Bare Earth (Only when adjacent to undeveloped areas)	2 FPS
Sodded Earth	5 FPS
Paved or Riprap Apron	15 FPS
Rock	15 FPS
Other	(as approved by City Engineer)

4.4.1 Energy Dissipators

Energy dissipators are used to dissipate excessive kinetic energy in flowing water that could promote erosion. An effective energy dissipator must be able to retard the flow of fast moving water without damage to the structure or to the channel below the structure.

Impact-type energy dissipators direct the water into an obstruction that diverts the flow in many directions and in this manner dissipates the energy in the flow. Baffled outlets and baffled aprons are two (2) impact-type energy dissipators.

Other energy dissipators use the hydraulic jump to dissipate the excess head. In this type of structure, water flowing at a higher than critical velocity is forced into a hydraulic jump, and energy is dissipated in the resulting turbulence. Stilling basins are this type of dissipator, where energy is diffused as flow plunges into a pool of water.

Generally, the impact-type of energy dissipator is considered to be more efficient than the hydraulic jump-type. Also the impact-type energy dissipator results in smaller and more economical structures.

The design of energy dissipators is based on the empirical data resulting from a comprehensive series of model structure studies by the U.S. Bureau of Reclamation, as detailed in its book Hydraulic Design of Stilling Basins and Energy Dissipators.

4.5 CULVERT TYPES AND SIZES

The permissible types of culverts under all roadways and embankments are reinforced concrete box, round, or elliptical concrete pipe or pipe arch.

The minimum size of pipe for all culverts shall be 18" or the equivalent sized elliptical pipe or arch pipe. Box culverts may be constructed in sizes equal to or larger than 3' x 2' (width versus height), except as approved by the City Engineer.

If material other than reinforced concrete pipe is to be used, it shall be approved by the City Engineer.

Flared, precast concrete and metal pipe aprons may be used in lieu of headwalls to improve the hydraulic capabilities of the culverts. Plastic and HDPE end sections are prohibited.

4.6 FILL HEIGHTS AND BEDDING

The minimum cover over any culvert or box culvert shall be 18", or a minimum of 6" from the bottom of the pavement subgrade, unless approved by City Engineer. Minimum cover less than these values shall be fully justified in writing and approved by the City Engineer prior to proceeding with final plans. Maximum fill heights shall be based on pipe manufacturer's recommendations. Bedding descriptions are shown on Figures 4.17 and 4.18. Box culverts shall be structurally designed to accommodate earth and live load to be imposed upon the culvert. Refer to the Arkansas Highway and Transportation Departments Standard Plans for Typical Box Culvert Designs. When installed within public right-of-ways, all culverts shall be capable of withstanding minimum H-20 loading.

Where culverts under railroad facilities are necessary, the designer shall obtain approval from the affected railroad.

4.7 TYPES OF CULVERT FLOW

Type I	Flow Part Full with Outlet Control and Tailwater Depth Below Critical Depth. (Figure 4.3)
Type II	Flowing Part Full with Outlet Control and Tailwater Depth Above Critical Depth. (Figure 4.4)
Type III	Flowing Part Full with Inlet Control. (Figure 4.5)
Type IVA	Flowing Full with Submerged Outlet. (Figure 4.6)
Type IVB	Flowing Full with Partially Submerged Outlet. (Figure 4.7)

4.8 CULVERT DESIGN PROCEDURE:

Computer generated computations and output are accepted and subject to review by City Engineer.

STEP 1 - SELECTING CULVERT SIZE:

The computations involved in selecting the smallest feasible barrel which can be used without exceeding the design headwater elevation is summarized in the tabulation sheet, titled "Culvert Computations", Table 4.3.

INITIAL DATA:

Enter initial data and complete required information for first approximation. The square feet of opening for the initial trial size may be estimated by the ratio of design discharge divided by 10.

TAILWATER:

The tailwater depth is influenced by conditions downstream of the culvert outlet. If the culvert outlet is located near the inlet of a downstream culvert, then the headwater elevation of the downstream culvert may define the tailwater depth for the upstream culvert. If the culvert outlet is operating in a free outfall condition then the tailwater is taken as 0.0.

If the culvert discharges into an open channel, then tailwater conditions should be determined by either backwater conditions, normal depth (subcritical flow) or critical depth (supercritical flow). Figure 9.1, provides a graphical solution for normal depth of flow which may be calculated by Mannings Formula:

$$Q = \frac{1.486}{n} AR^{2/3} S^{1/2}$$

In any case, the tailwater depth is defined as the depth of water measured from the flow line of the culvert (invert) at the outlet to the water surface elevation at the outlet.

Enter tailwater depth in Column 8 and applicable stream data in upper left hand portion of Culvert Computation Form.

STEP 2 - PERFORM OUTLET CONTROL CALCULATIONS (TABLE 4.3):

These calculations are performed before inlet control calculations in order to select the smallest feasible barrel, which can be used without the required headwater elevation in outlet control exceeding the allowable headwater elevation.

Column 1: Enter the span times height dimensions (or diameter of pipe) of culvert.

Column 2: Enter the type of structure and design of entrance.

Column 3: Enter the design discharge or quotient of design discharge divided by the applicable denominator.

Column 4: Enter the Entrance Loss Coefficient from Table 4.1.

Column 5: Enter the head from the applicable outlet control nomograph, in the example problem use Figure 4.11.

Column 6: Enter the critical depth from appropriate nomograph, in the example problem use Figure 4.8. Critical depth cannot exceed height of culvert opening.

Column 7: For tailwater elevations less than the top of the culvert at the outlet, hydraulic grade line is found by solving for h_o using the following equation:

$$h_o = \frac{d_c + D}{2}$$

where: h_o = vertical distance in feet from culvert invert at outlet to the hydraulic grade line in feet

d_c = critical depth in feet

D = height of culvert opening in feet

Column 8: Enter the tailwater elevation from initial data shown at top of form. Refer to tailwater comments under STEP 1 for additional guidelines.

Column 9: Enter the product of culvert length times the slope.

Column 10: Headwater elevation required for culvert to pass flow in outlet control (HW_o) is computed by the following equation:

$$HW_o = H + h_o - LS$$

Note: Use TW elevation in lieu of h_o where $TW > h_o$

Additional trials may be required. Space for additional trials is provided on Culvert Computations Form.

<u>STEP 3 - PERFORM INLET CONTROL CALCULATIONS FOR CONVENTIONAL AND BEVELED EDGE CULVERT</u>:

After minimum barrel size has been determined under STEP 2, the next procedure is similar to that used in FHWA's <u>Hydraulic Design Series #5</u>, "Hydraulic Design of Highway Culverts".

The computations involved in computing inlet headwater elevation is summarized in the tabulation sheet used in STEP 2, titled "Culvert Computations", Table 4.3.

Column 11: Enter ratio of headwater to height of structure from Figure 4.10.

Column 12: HW is derived by multiplying Column 11 by the height (or diameter) of culvert.

Column 13: Enter greater of two headwaters (Column 10 or 12).

Column 14: Inlet control governs, outlet velocity equals Q/A, where A is defined by the cross-sectional area of normal depth of flow in the culvert barrel. Figures 4.9 and 4.13 provide a graphical solution for estimating normal depth of flow and velocity. Manning's Formula may also be used:

$$V = \frac{1.486}{n} R^{2/3} S^{1/2}$$

If outlet control governs, outlet velocity equals Q/A, where A is the cross-sectional area of flow in the culvert barrel at the outlet.

Column 15: Figures shown in this column are believed to be self-explanatory.

IMPROVED INLETS:

A. See Arkansas Highway and Transportation Department's Manual for improved inlet or side tapered inlet design.

FORM HYD 4-1

PROJECT: ______

CULVERT COMPUTATIONS
(SQUARE AND BEVELED EDGES)

DESIGNER: ______

DATE: ______

HYDROLOGIC AND CHANNEL INFORMATION

HYDROLOGY

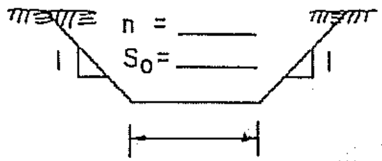
Q_1 ______ = ______ cfs

Q_2 ______ = ______ cfs

STREAM DATA

Tw_1 = ______

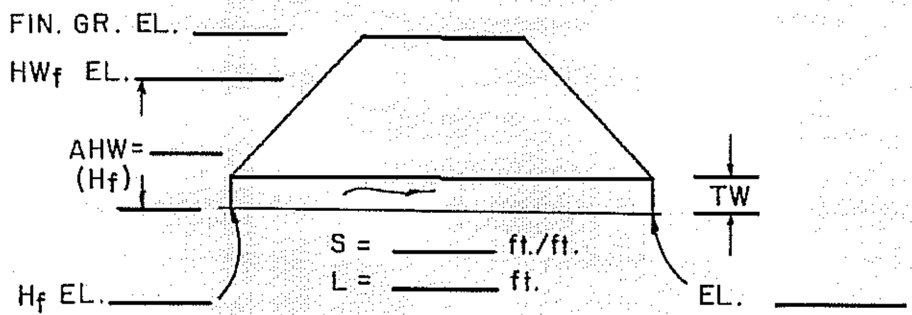
Tw_2 = ______



OUTLET CHANNEL
(APPROX. DIMENSIONS)

SKETCH

STATION: ______



TRIAL NO.	SIZE	STRUCTURE TYPE & ENTRANCE DESIGN	Q / Q/NB / $\frac{Q}{NBD^{3/2}}$	HEADWATER COMPUTATION: OUTLET CONTROL (a) K_e	H	(b) d_c	(c) h_o	(d) TW	LS	(e) HW_o	INLET CONT. (f) $\frac{HW}{D}$	HW	CONTROL-LING HW	OUTLET VELOCITY ft./sec.	COST	COMMENTS
0	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16

(a) Entrance loss coefficient, Refer to Table **4.1**, page **IV-4**

(b) "d_c" cannot exceed D.

(c) $h_o = \frac{d_c + D}{2}$ or TW, whichever is larger.

(d) TW = d_n in natural channel, or other downstream control.

(e) $HW_o = H + H_o - LS$

(f) **Use Inlet control nomographs.**

Table 4.3

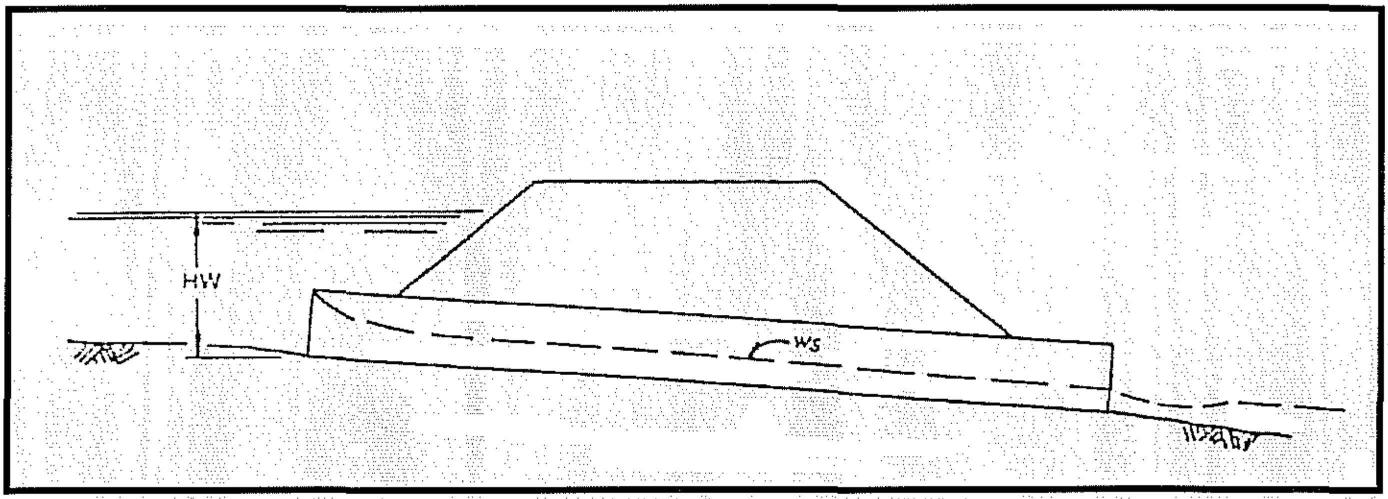
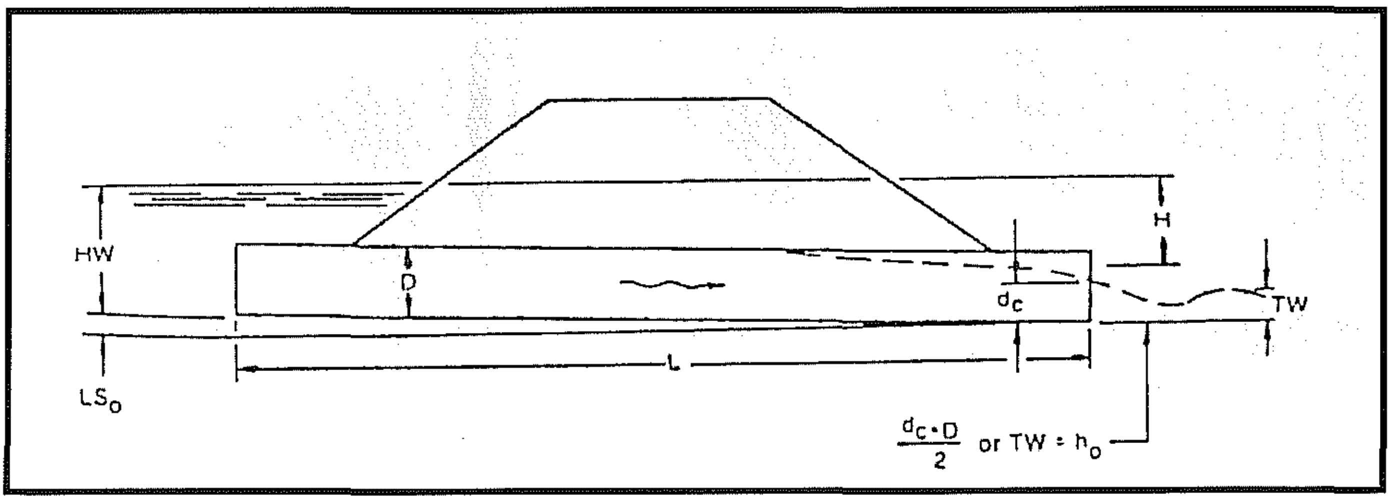


Figure 4.1

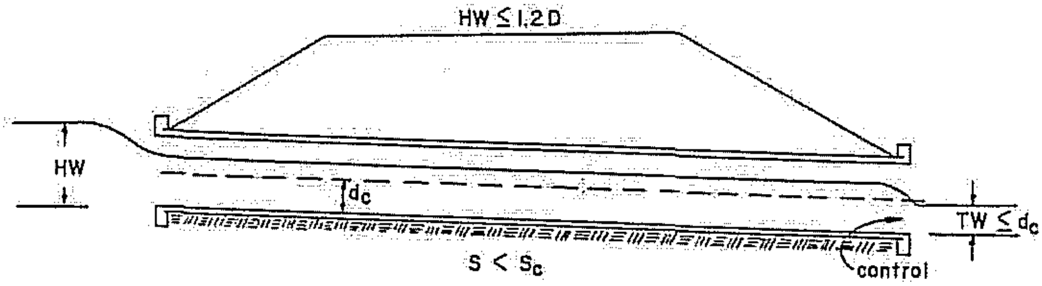
INLET CONTROL



OUTLET CONTROL

Figure 4.2

Type I
Culvert Flowing Part Full
With Outlet Control and Tailwater Depth
Below Critical Depth



Conditions

The entrance is unsubmerged (HW≤ 1.2D), the slope at design discharge is sub-critical (S_o<S_c), and the tailwater is below critical depth (TW≤ d_c).

The above condition is a common occurrence where the natural channels are on flat grades and have wide, flat flood plains. The control is critical depth at the outlet.

In culvert design, it is generally considered that the headwater pool maintains a constant level during the design storm. If this level does not submerge the culvert inlet, the culvert flows part full.

If critical flow occurs at the outlet the culvert is said to have "Outlet Control". A culvert flowing part full with outlet control will require a depth of flow in the barrel of the culvert greater than critical depth while passing through critical depth at the outlet.

The capacity of a culvert flowing part full with outlet control and tailwater depth below critical depth shall be governed by the following equation when the approach velocity is considered zero.

$$HW = d_c + \frac{V_c^2}{2g} + h_e + h_f - S_oL$$

HW = Headwater depth above the invert of the upstream end of the culvert in feet. Headwater must be equal to or less than 1.2D or entrance is submerged and Type IV operation will result.

d_c = Critical depth of flow in feet = $\sqrt[3]{\frac{q^2}{32.2}}$

D = Diameter of pipe or height of box.

q = Discharge in cfs per foot.

V_c = Critical Velocity in feet per second occurring at critical depth.

h_e = Entrance head loss in feet.

$$h_e = Ke\left(\frac{Vc^2}{2g}\right)$$

K_e = Entrance loss coefficient

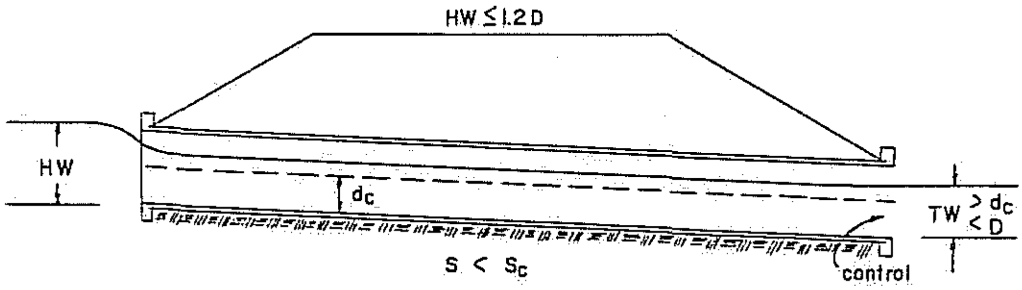
h_f = Friction head loss in feet = $S_f L$.

S_f = Friction slope or slope that will produce uniform flow. For Type I operation the friction slope is based upon $1.1d_c$

S_o = Slope of culvert in feet per foot.

L = Length of culvert in feet.

Type II
Culvert Flowing Part Full
With Outlet Control and Tailwater Depth
Above Critical Depth



Conditions

The entrance is unsubmerged (HW≤ 1.2D), the slope at design discharge is subcritical ($S_o<S_c$), and the tailwater is above critical depth (TW> d_c).

The above condition is a common occurrence where the channel is deep, narrow and well defined.

If the headwater pool elevation does not submerge the culvert inlet, the slope at the design discharge is subcritical, and the tailwater depth is above critical depth the control is said to occur at the outlet; and the capacity of the culvert shall be governed by the following equation when the approach velocity is considered zero.

$$HW = TW + \frac{V_{TW}^2}{2g} + h_e + h_f - S_oL$$

HW = Headwater depth above the invert of the upstream end of the culvert in feet. Headwater depth must be equal to or less than 1.2D or entrance is submerged and Type IV operation will result.

TW = Tailwater depth above the invert of the downstream end of the culvert.

V_{TW} = Culvert discharge velocity in feet per second at tailwater depth.

h_e = Entrance head loss in feet.

$$h_e = K_e \frac{V_{TW}^2}{2g}$$

K_e = Entrance loss coefficient.

h_f = Friction head loss in feet = S_fL.

S_f = Friction slope or slope that will produce uniform flow. For Type II operation the friction slope is based upon TW depth.

S_o = Slope of culvert in feet per foot.

L = Length of culvert in feet.

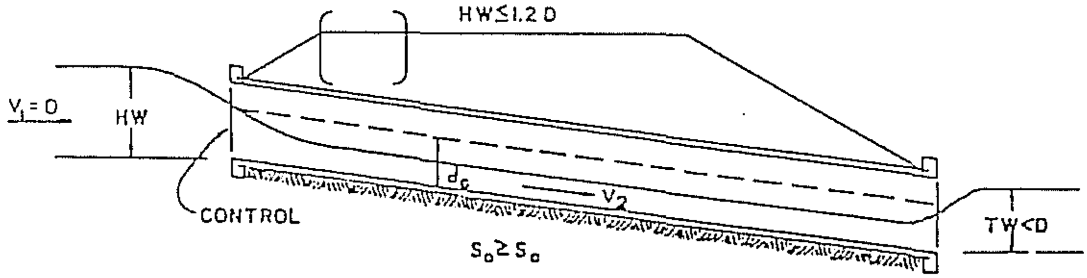
THE CITY OF BENTONVILLE

TYPES OF CULVERT FLOW-TYPE II

SOURCE: City of Austin, TX

Figure 4.4

Type III
Culvert Flowing Part Full With Inlet Control



Conditions

The entrance is unsubmerged (HW$\leq$ 1.2D) and the slope at design discharge is equal to or greater than critical (Supercritical) ($S_o \geq S_c$).

This condition is a common occurrence for culverts in rolling or mountainous country where the flow does not submerge the entrance. The control is critical depth at the entrance.

If critical flow occurs near the inlet, the culvert is said to have "Inlet Control". The maximum discharge through a culvert flowing part full occurs when flow is at critical depth for a given energy head. To assure that flow passes through critical depth near the inlet, the culvert must be laid on a slope equal to or greater than critical slope for the design discharge. Placing culverts which are to flow part full on slopes greater than critical slope will increase the outlet velocities but will not increase the discharge. The discharge is limited by the section near the inlet at which critical flow occurs.

The capacity of a culvert flowing part full with control at the inlet shall be governed by the following equation when the approach velocity is considered zero.

$$HW = d_c + \frac{V_2^2}{2g} + K_e \frac{V_2^2}{2g}$$

HW = Headwater depth above the invert of the upstream end of the culvert in feet. Headwater depth must be equal to or less than 1.2D or entrance is submerged and Type IV operation will result.

d_c = Critical depth of flow in feet = $\sqrt[3]{\frac{q^2}{32.2}}$

q = Discharge in cfs per foot.

V_2 = Velocity of flow in the culvert in feet per second.

The velocity of flow varies from critical velocity at the entrance to uniform velocity at the outlet provided the culvert is sufficiently long. Therefore, the outlet velocity is the discharge divided by the area of flow in the culvert.

K_e = Entrance loss coefficient.

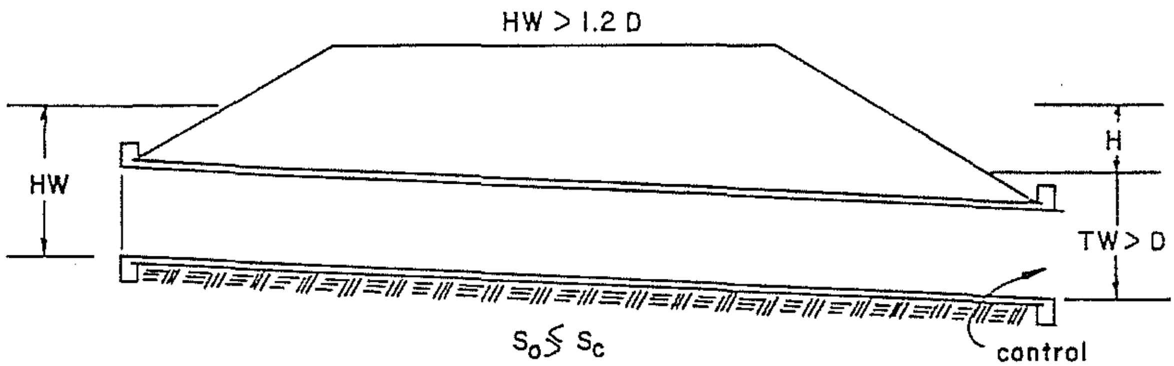
THE CITY OF BENTONVILLE

TYPES OF CULVERT FLOW – TYPE III

SOURCE: City of Austin, TX

Figure 4.5

Type IV-A
Culvert Flowing Full With Submerged Outlet



Conditions
(Submerged Outlet)

The entrance is submerged (HW>1.2D). The tailwater completely submerges the outlets.

Most culverts flow with free outlet, but depending on topography, a tailwater pool of a depth sufficient to submerge the outlet may form at some installation. Generally, these will be considered at the outlet. For an outlet to be submerged, the depth at the outlet must be equal to or greater than the diameter of pipe of height of box. The capacity of a culvert flowing full with a submerged outlet shall be governed by the following equation when the approach velocity is considered zero. Outlet Velocity is based on full flow at the outlet.

$$HW = H + TW - S_oL$$

HW = Headwater depth above the invert of the upstream end of the culvert. Headwater depth must be greater than 1.2D for entrance to be submerged.

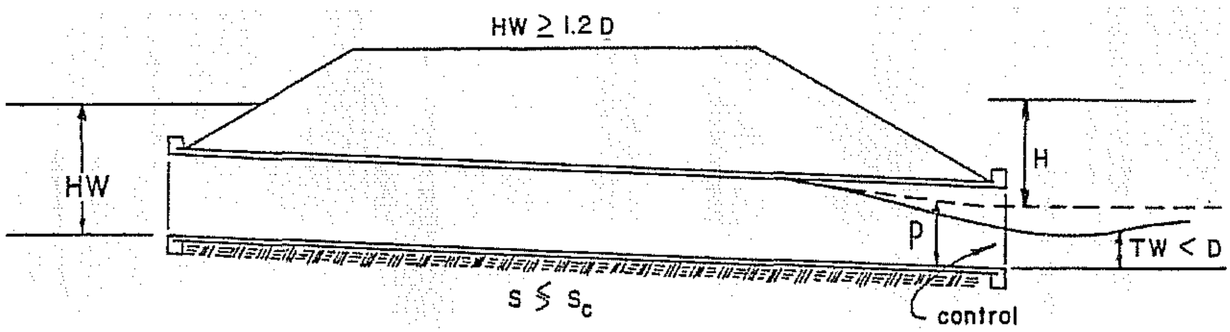
H = Head for culvert flowing full.

TW = Tailwater depth in feet.

S_o = Slope of culvert in feet per foot.

L = Length of culvert in feet.

Type IV-B
Culvert Flowing Full With Partially Submerged Outlet



Conditions
(Partially Submerged Outlet)

The entrance is submerged (HW>1.2D). The tailwater depth is less than D (TW<D).

The capacity of a culvert flowing full with a partially submerged outlet shall be governed by the following equation when the approach velocity is considered zero. Outlet velocity is based on critical depth if TW depth is less than critical depth. If TW depth is greater than critical depth, outlet velocity is based on TW depth.

$HW = H + P - S_oL$

HW = Headwater depth above the invert of the upstream end of the culvert. Headwater depth must be greater than 1.2D for entrance to be submerged.

H = Head for culvert flowing full.

P = Pressure line height = $\frac{d_c + D}{2}$

d_c = Critical depth in feet.

D = Diameter or height of structure in feet.

S_o = Slope of culvert in feet per foot.

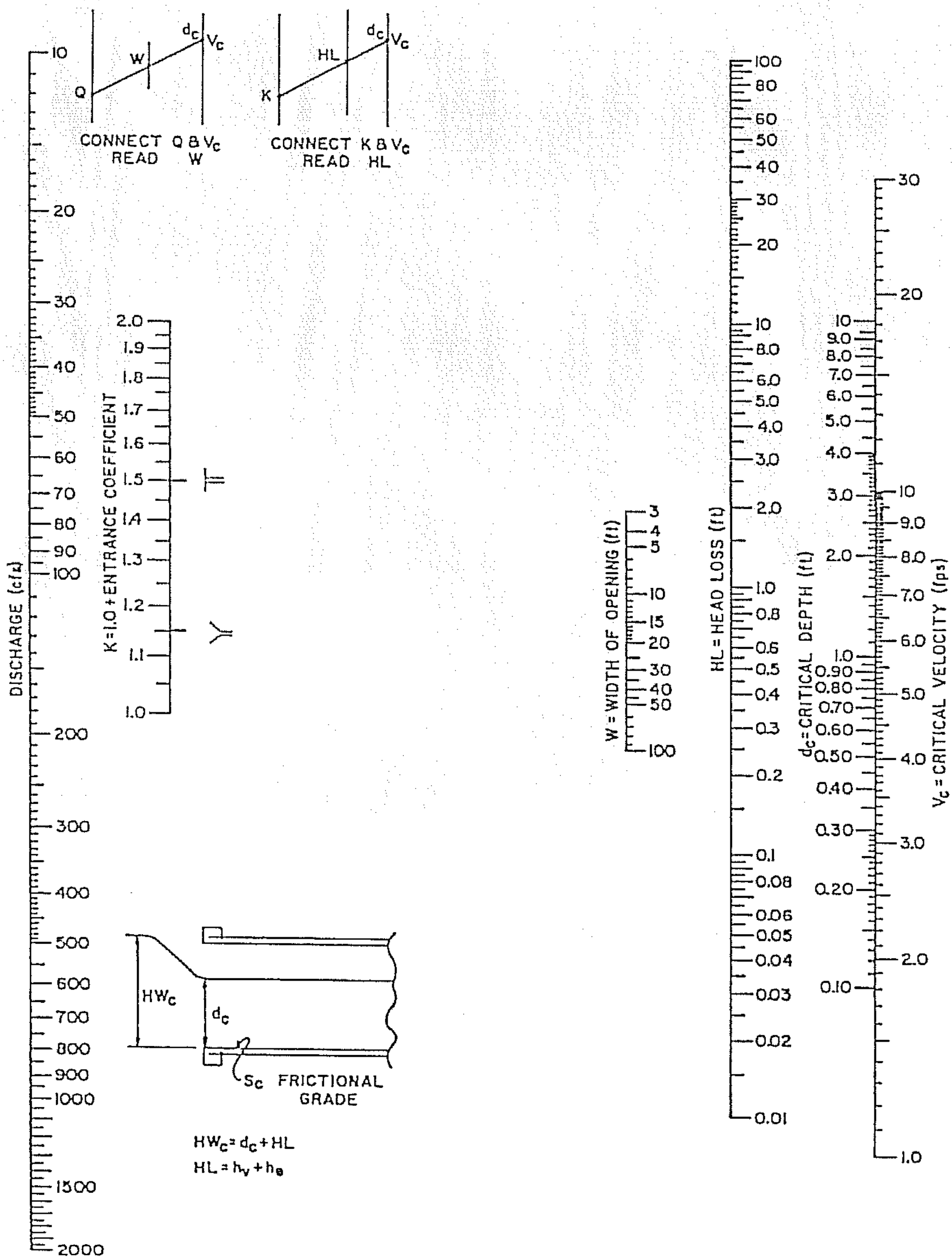
L = Length of culvert in feet.

THE CITY OF BENTONVILLE

TYPES OF CULVERT FLOW - TYPE IV-B

SOURCE: City of Austin, TX

Figure 4.7

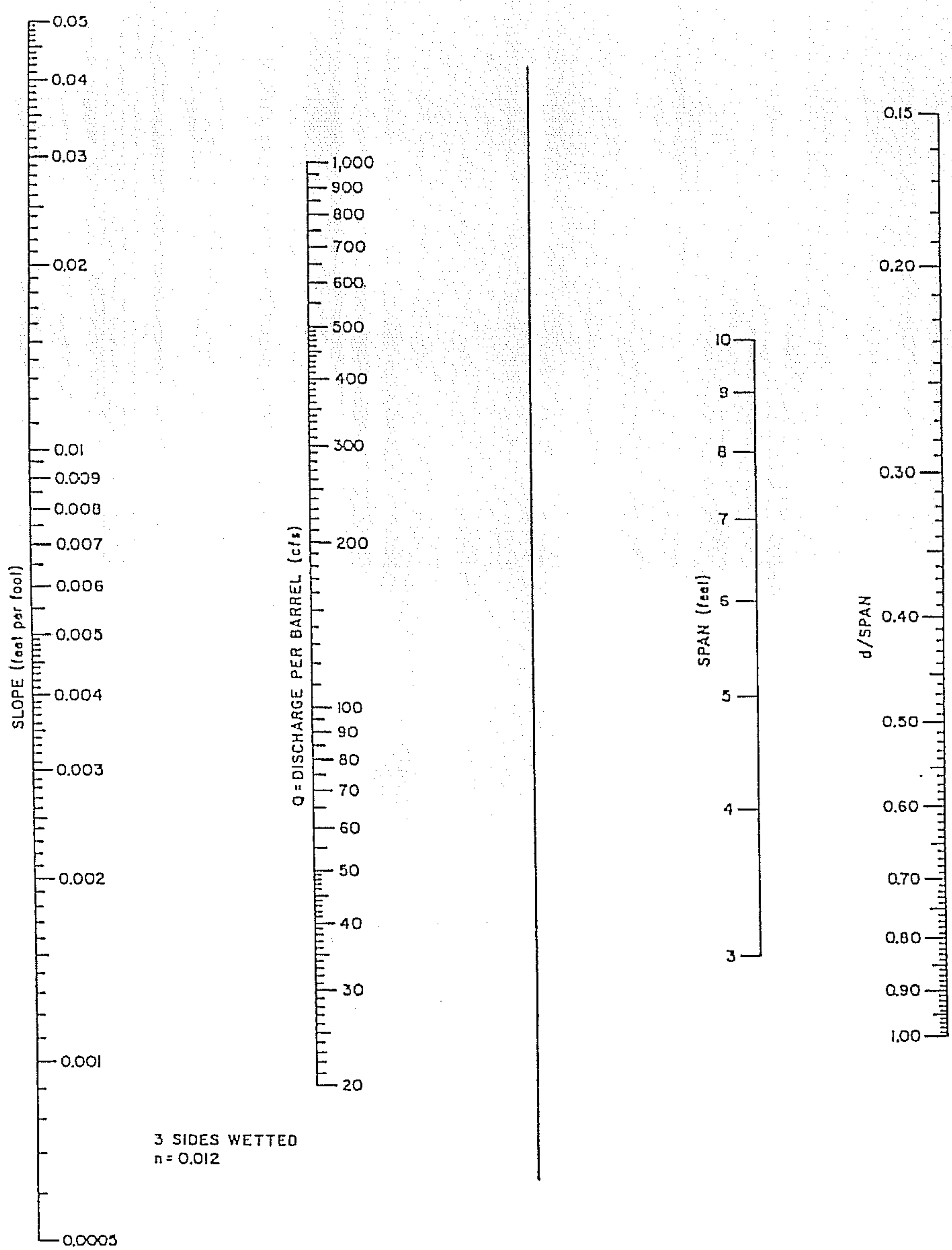


THE CITY OF BENTONVILLE

CRITICAL FLOW FOR BOX CULVERTS

SOURCE: Texas Highway Department

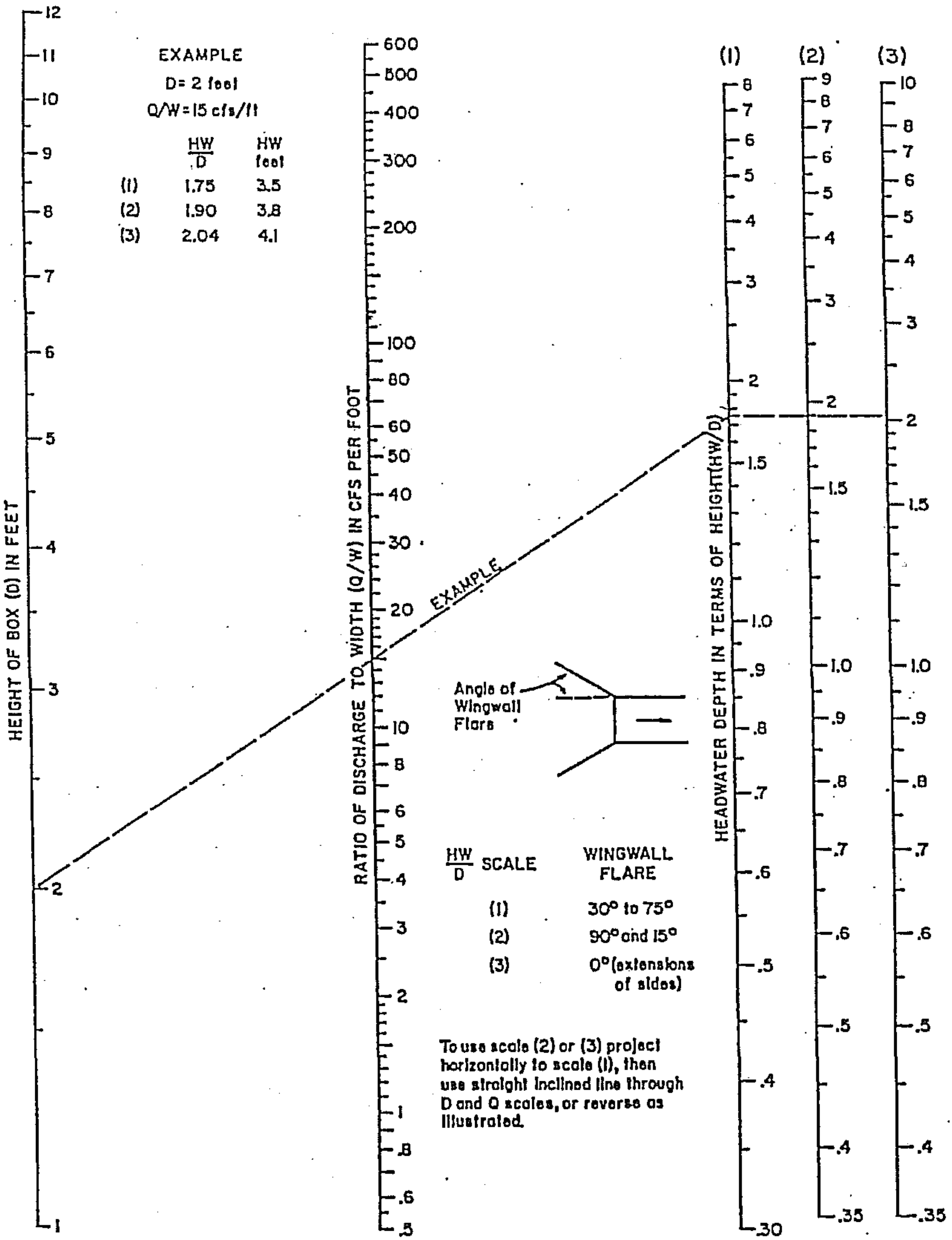
Figure 4.8



UNIFORM FLOW FOR BOX CULVERTS

SOURCE: Texas Highway Department

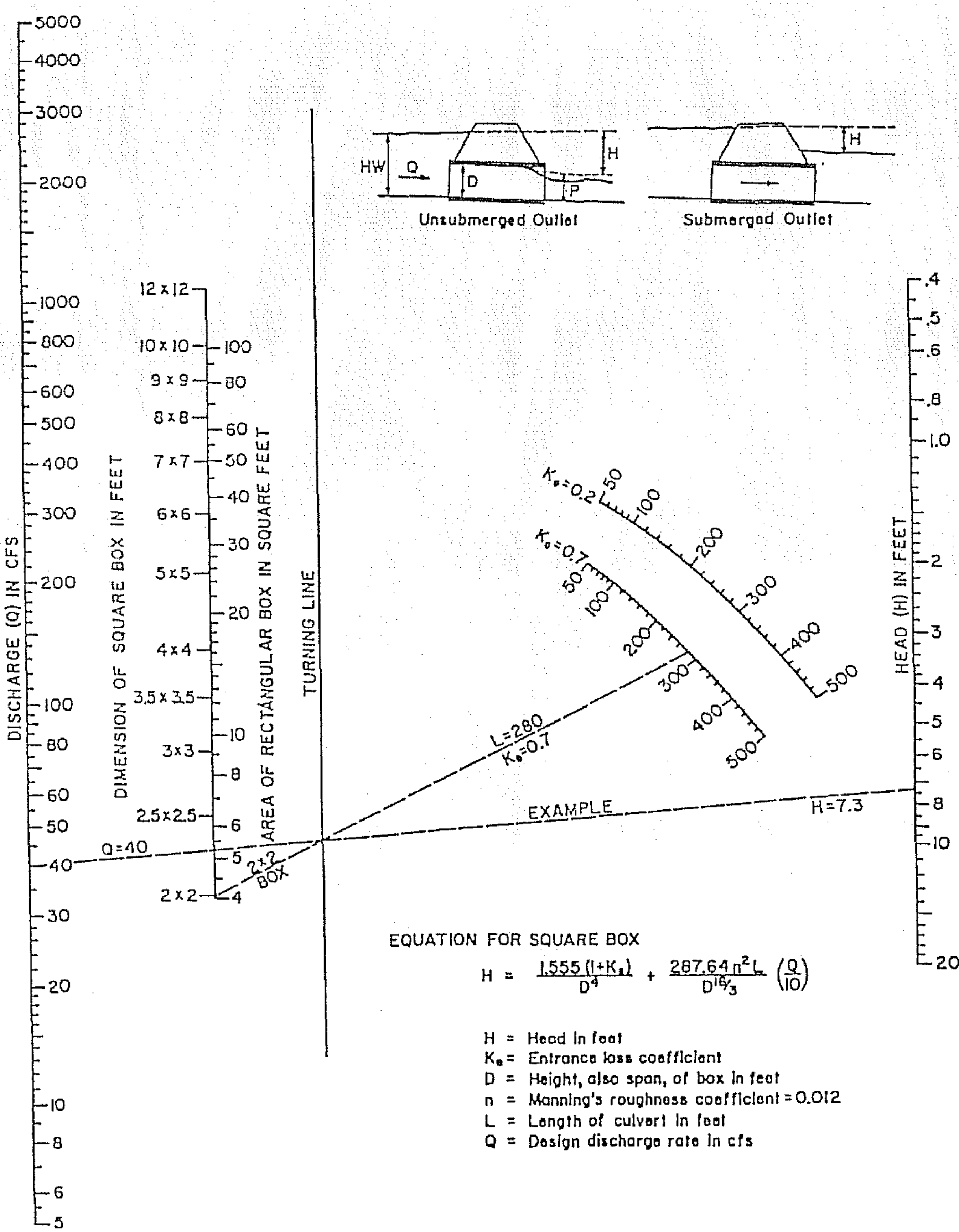
Figure 4.9



HEADWATER DEPTH FOR BOX
CULVERTS WITH INLET CONTROL

SOURCE: Texas Highway Department

Figure 4.10



EQUATION FOR SQUARE BOX

$$H = \frac{1.555(1+K_e)}{D^4} + \frac{287.64\, n^2 L}{D^{16/3}} \left(\frac{Q}{10}\right)$$

H = Head in feet
K_e = Entrance loss coefficient
D = Height, also span, of box in feet
n = Manning's roughness coefficient = 0.012
L = Length of culvert in feet
Q = Design discharge rate in cfs

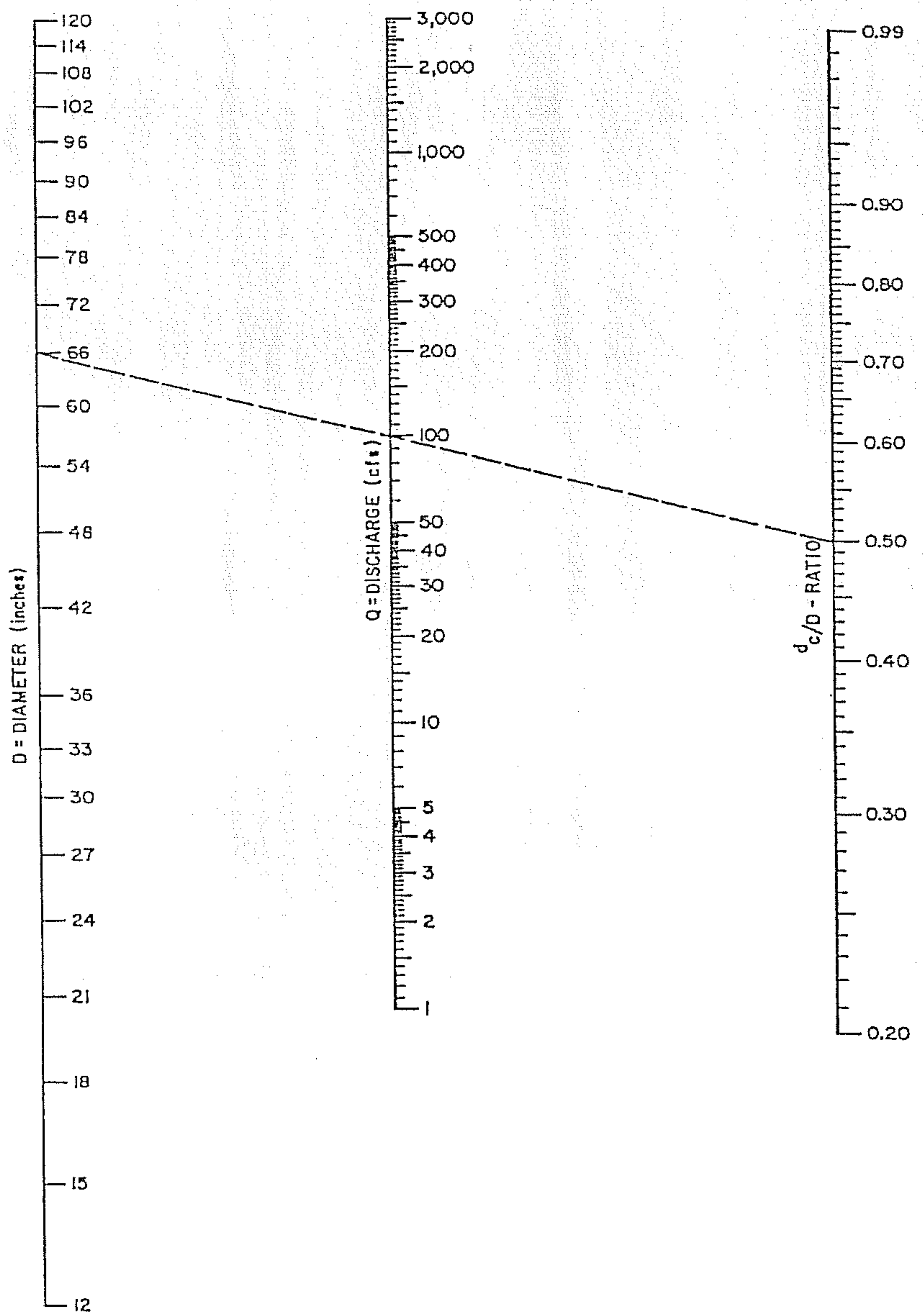
n = 0.012

THE CITY OF BENTONVILLE

HEAD FOR CONCRETE BOX
CULVERTS FLOWING FULL

SOURCE: Texas Highway Department

Figure 4.11

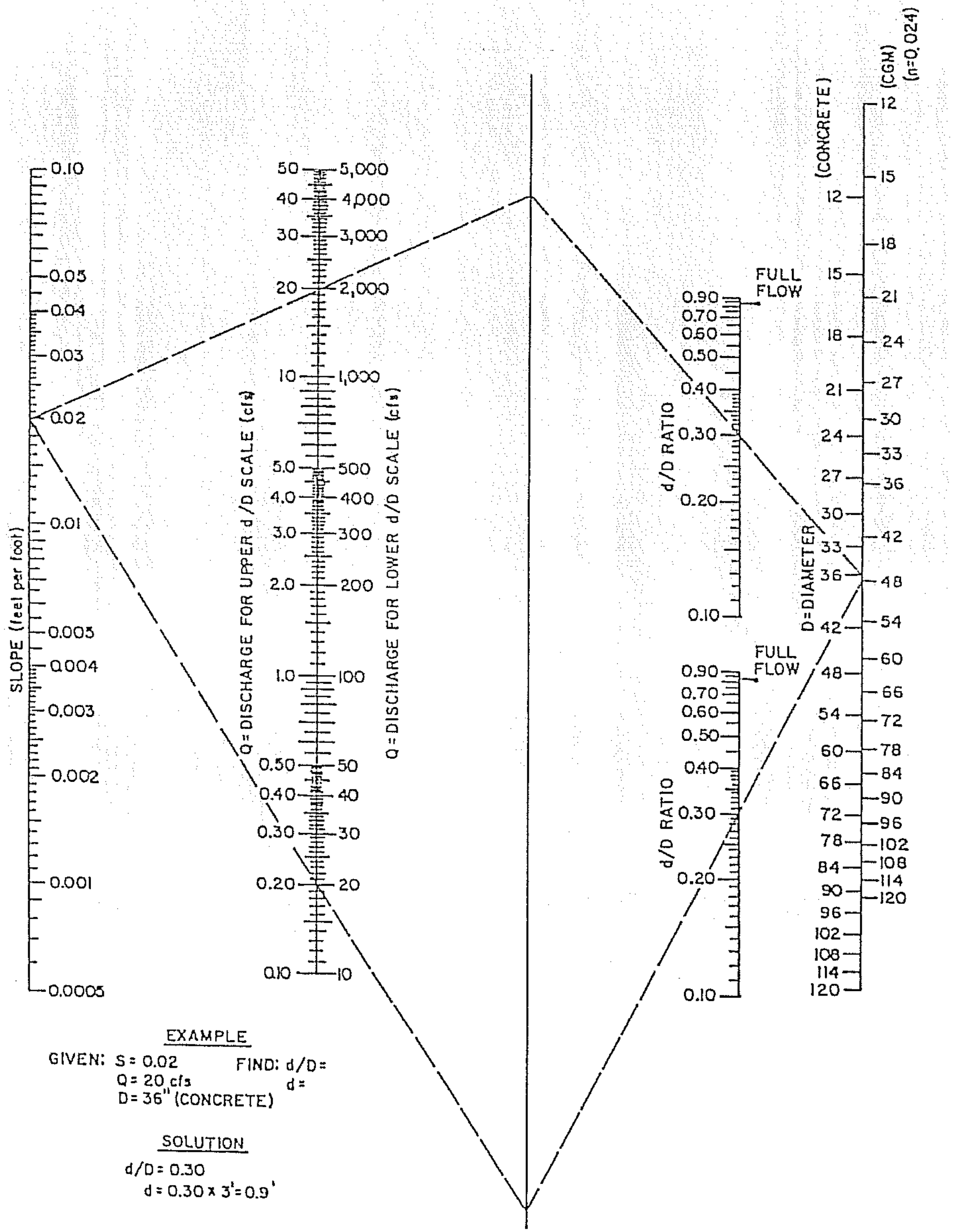


THE CITY OF BENTONVILLE

CRITICAL DEPTH OF FLOW
FOR CIRCULAR CONDUITS

SOURCE: Texas Highway Department

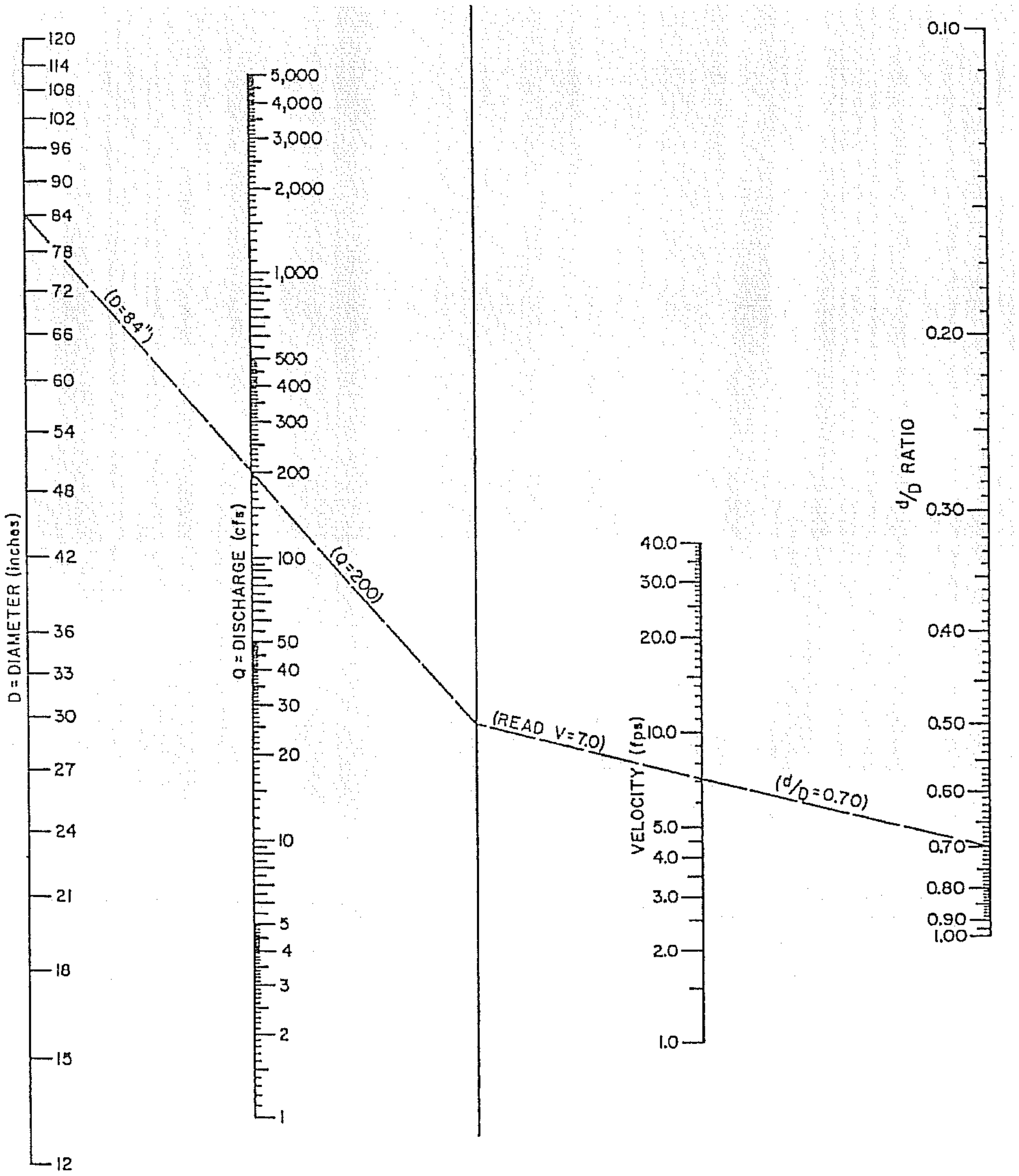
Figure 4.12



UNIFORM FLOW FOR PIPE CULVERTS

SOURCE: Texas Highway Department

Figure 4.13



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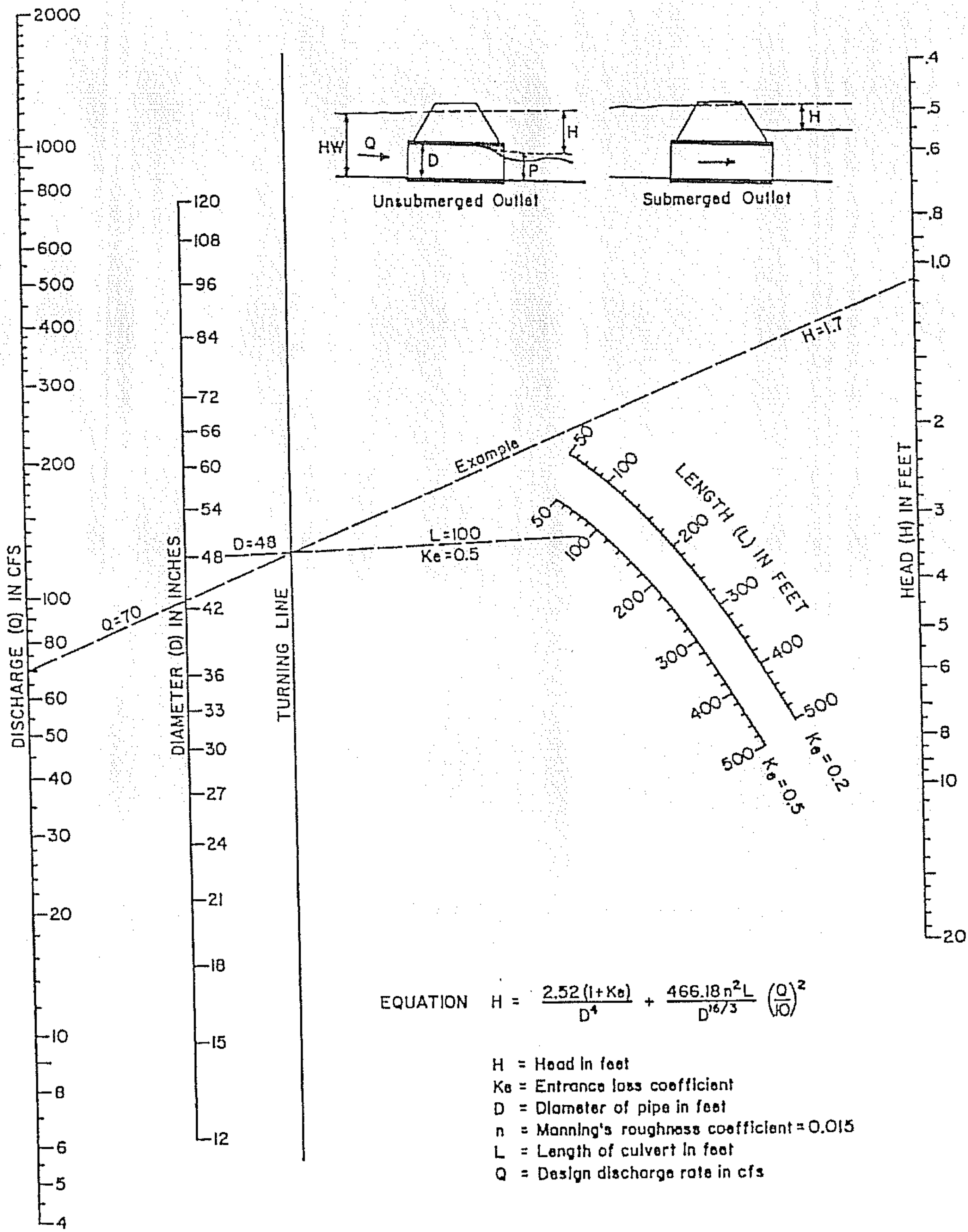
VELOCITY IN PIPE CONDUITS

SOURCE: Texas Highway Department

Figure 4.14



EQUATION $H = \frac{2.52(1+Ke)}{D^4} + \frac{466.18 n^2 L}{D^{16/3}} \left(\frac{Q}{10}\right)^2$

H = Head in feet

Ke = Entrance loss coefficient

D = Diameter of pipe in feet

n = Manning's roughness coefficient = 0.015

L = Length of culvert in feet

Q = Design discharge rate in cfs

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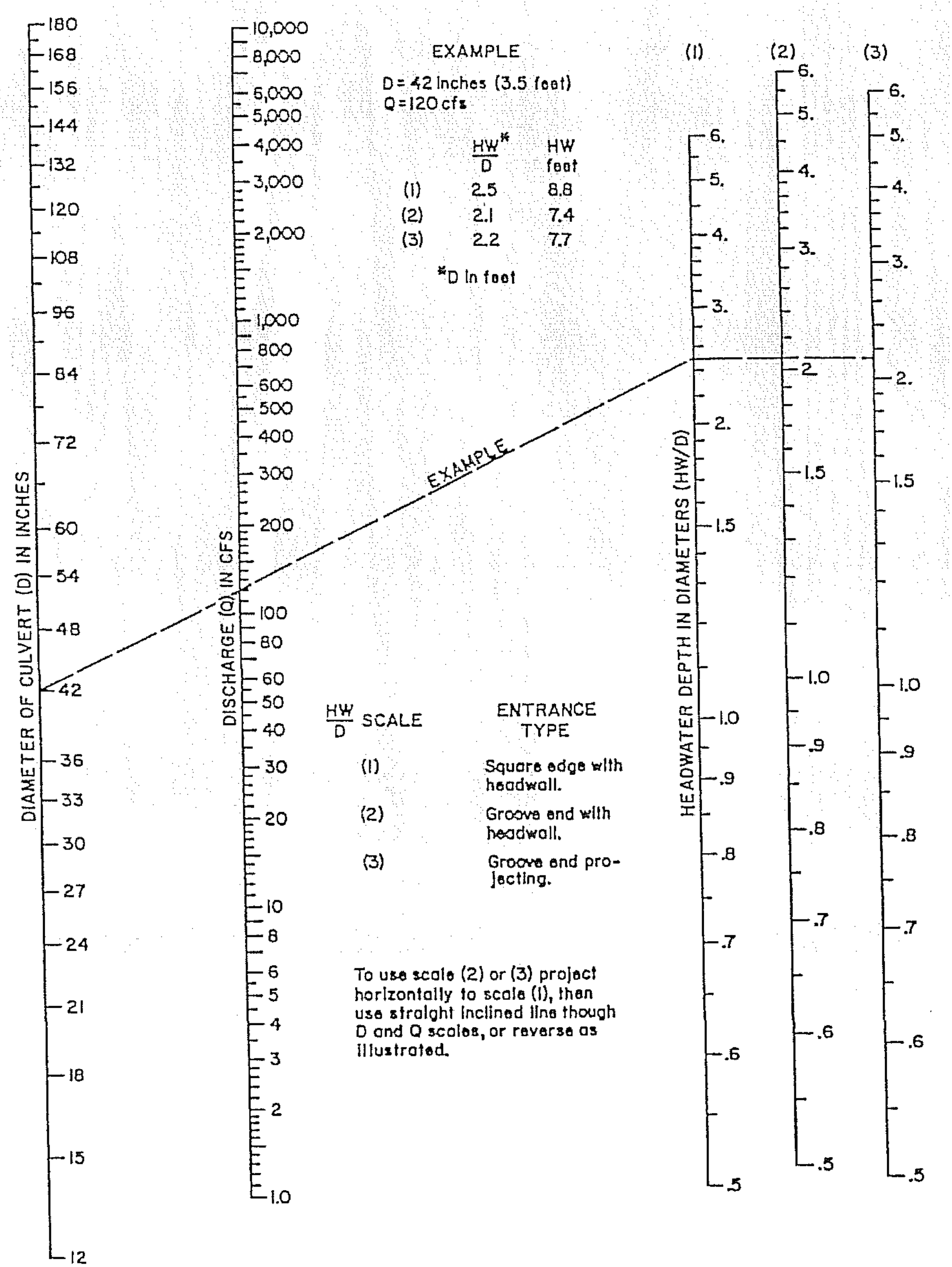
HEAD FOR CONCRETE PIPE CULVERTS FLOWING FULL

SOURCE: Texas Highway Department

Figure 4.15



HEADWATER DEPTH FOR CONCRETE PIPE CULVERTS WITH INLET CONTROL

Figure 4.16

CONCRETE PIPE

DIAM. OF PIPE D (INCHES)	CLASS "B" BEDDING (H) MAXIMUM ALLOWABLE COVER-FEET				CLASS "C" BEDDING (H) MAXIMUM ALLOWABLE COVER-FEET			
	II*	III	IV	V	II	III	IV	V
18	11	13	20	25	9	12	18	22
24	12	14	21	26	10	13	19	23
36	13	16	23	28	11	14	20	24
48	14	16	24	29	11	15	21	25
60	14	17	24	29	12	15	21	26
72	14	17	24	30	12	16	22	26
84	15	17	25	30	13	16	22	27
96	15	18	25	31	13	16	23	27
108	15	18	26	32	13	17	23	28

* ASTM C76-72 Table Designation

THE CITY OF BENTONVILLE

DEPTH OF COVER TABLES

Figure 4.17

CORRUGATED METAL PIPE

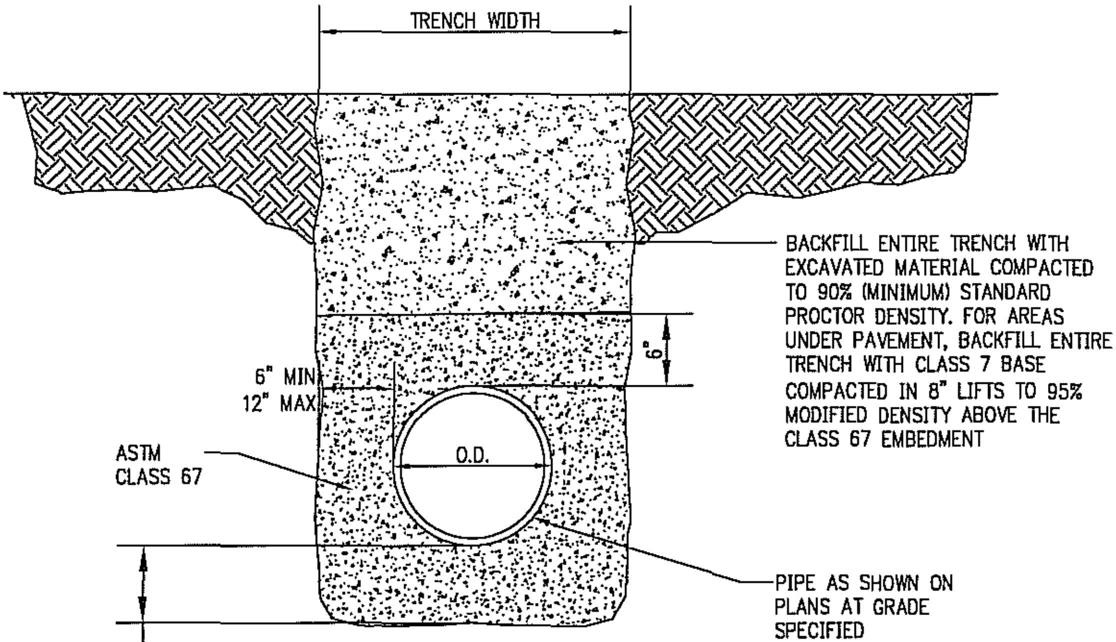
3" x 1" CORRUGATIONS											
DAIM. OF PIPE D (INCHES)	MIN. COVER ABOVE PIPE (INCHES)	(H) MAXIMUM ALLOWABLE COVER-FEET									
		16 GA. (0.064")		14 GA. (0.079")		12 GA. (0.109")		10 GA. (0.138")		8 GA. (0.168")	
		Round	Elong	Round	Elong	Round	Elong	Round	Elong	Round	Elong
36	18	27	40	31	50	40	74				
42	18	21	34	23	42	29	58				
48	18	17	30	19	37	23	46				
54	18	15	27	16	32	19	38				
60	18	13	24	15	29	16	33				
66	18	13	22	13	27	15	30				
72	18	12	20	12	25	14	27				
78	18	12	18	12	23	13	26				
84	18			12	21	12	24	13	26		
90	18					12	24	12	35	13	26
96	18					11	23	12	24	12	25
102	24							12	23	12	24
108	24									12	23
114	24									11	23
120	24									11	20

2 2/3" x 1/2" CORRUGATIONS											
DAIM. OF PIPE D (INCHES)	MIN. COVER ABOVE PIPE (INCHES)	(H) MAXIMUM ALLOWABLE COVER-FEET									
		16 GA. (0.064")		14 GA. (0.079")		12 GA. (0.109")		10 GA. (0.138")		8 GA. (0.168")	
		Round	Elong	Round	Elong	Round	Elong	Round	Elong	Round	Elong
12	18	70		76							
15	18	56		61							
18	18	40		48		64					
24	18	23		26		33					
30	18			18	30	22	43	25	51		
36	18			15	25	17	33	19	38		
42	18					14	28	16	31	17	34
48	18					13	25	14	27	15	29
54	18					12	24	13	25	13	26
60	18							12	23	12	25
66	18							11	22	12	23
72	18							11	17	11	21
78	18									11	17
84	18									11	13

CORRUGATED METAL PIPE ARCH

3" x 1" CORRUGATIONS								
SPAN (INCHES)	RISE (INCHES)	RC (INCHES)	MIN. COVER ABOVE PIPE (INCHES)	(H) MAXIMUM ALLOWABLE COVER-FEET				
				16 GA. (0.064")	14 GA. (0.079")	12 GA. (0.109")	10 GA. (0.138")	8 GA. (0.168")
43	27	7.75	18	6	6			
50	31	9	18	6	6			
58	36	10.5	18	6	6			
65	40	12	18	6	6			
72	44	13.25	18	6	6			
73	55	18	18	8	8			
81	59	18	18		7	7		
87	63	18	18		7	7		
95	67	18	18		6	6		
103	71	18	24			6		
112	75	18	24			5		
117	79	18	24			5		
128	83	18	24				5	

2 2/3" x 1/2" CORRUGATIONS								
SPAN (INCHES)	RISE (INCHES)	RC (INCHES)	MIN. COVER ABOVE PIPE (INCHES)	(H) MAXIMUM ALLOWABLE COVER-FEET				
				16 GA. (0.064")	14 GA. (0.079")	12 GA. (0.109")	10 GA. (0.138")	8 GA. (0.168")
18	11	3.5	18	6	6			
22	13	4	18	6	6			
25	16	4	18	5	5			
29	18	4.5	18	5	5			
36	22	5	18	5	5			
43	27	5.5	18	4	4			
50	31	6	18			4	4	4
58	36	7	18			4	4	4
65	40	8	18			4	4	4
72	44	9	18				4	4
79	49	10	18					4
85	54	11	18					4



STORM SEWER PIPE BEDDING DETAIL

NTS

FOR ALL STORM PIPE MATERIALS

NOTE: DIMENSIONS SHOWN ARE MINIMUM REQUIREMENTS.
MANUFACTURER'S SPECIFICATIONS MAY BE MORE STRINGENT.
MOST RESTRICTIVE SPECIFICATIONS SHALL GOVERN

Recommended Distance Between Pipes for Trenches with Multiple Pipes

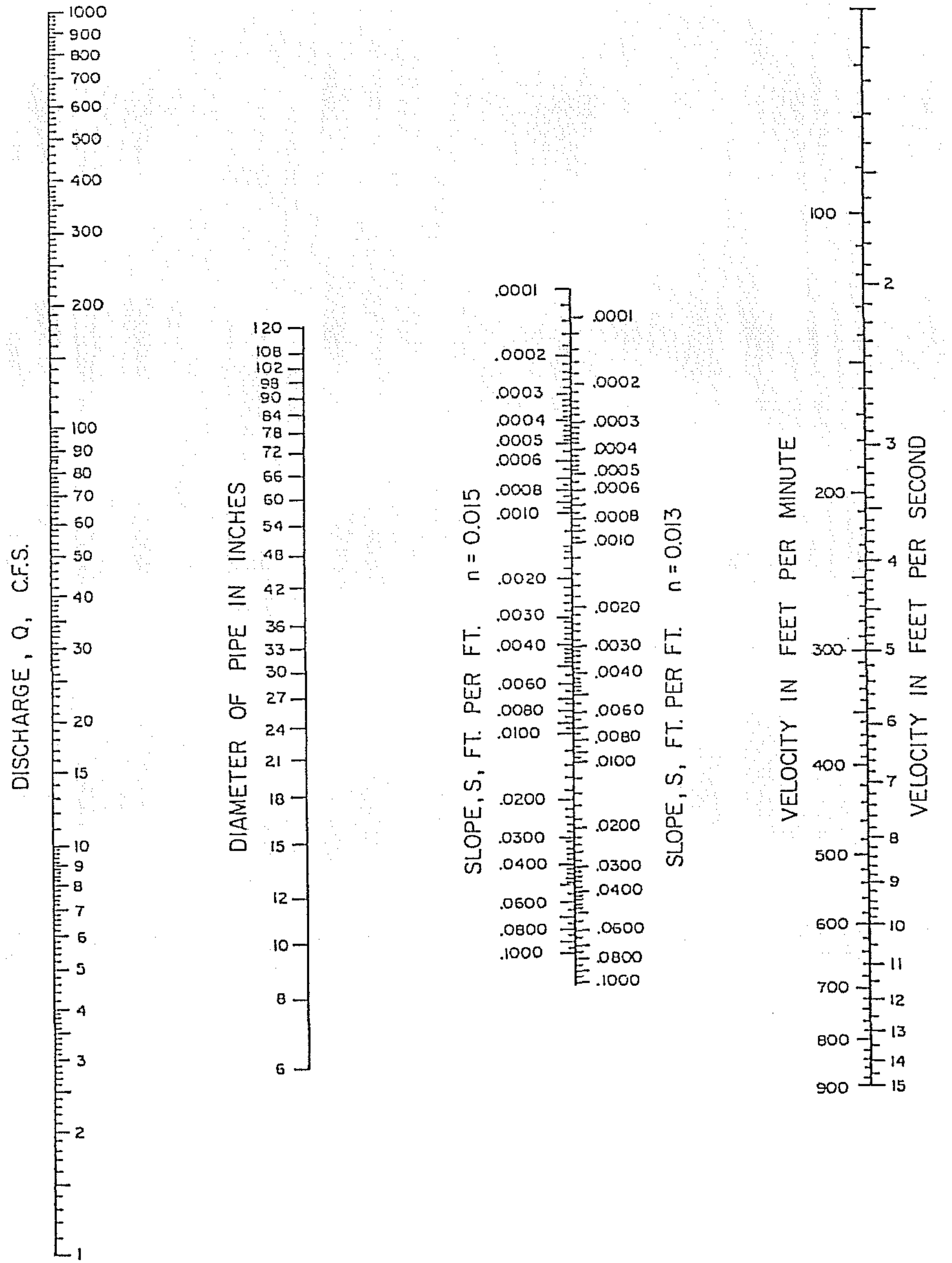
PIPE MATERIAL	MINIMUM DISTANCE BETWEEN MULTIPLE PIPES (FEET)
RCP	6"
CMP	1'
HDPE	1'
PVC	1'

NOTE: DIMENSIONS SHOWN ARE MINIMUM REQUIREMENTS.
MANUFACTURER'S SPECIFICATIONS MAY BE MORE STRINGENT.
MOST RESTRICTIVE SPECIFICATIONS SHALL GOVERN

THE CITY OF BENTONVILLE

PIPE BEDDING

Figure 4.18



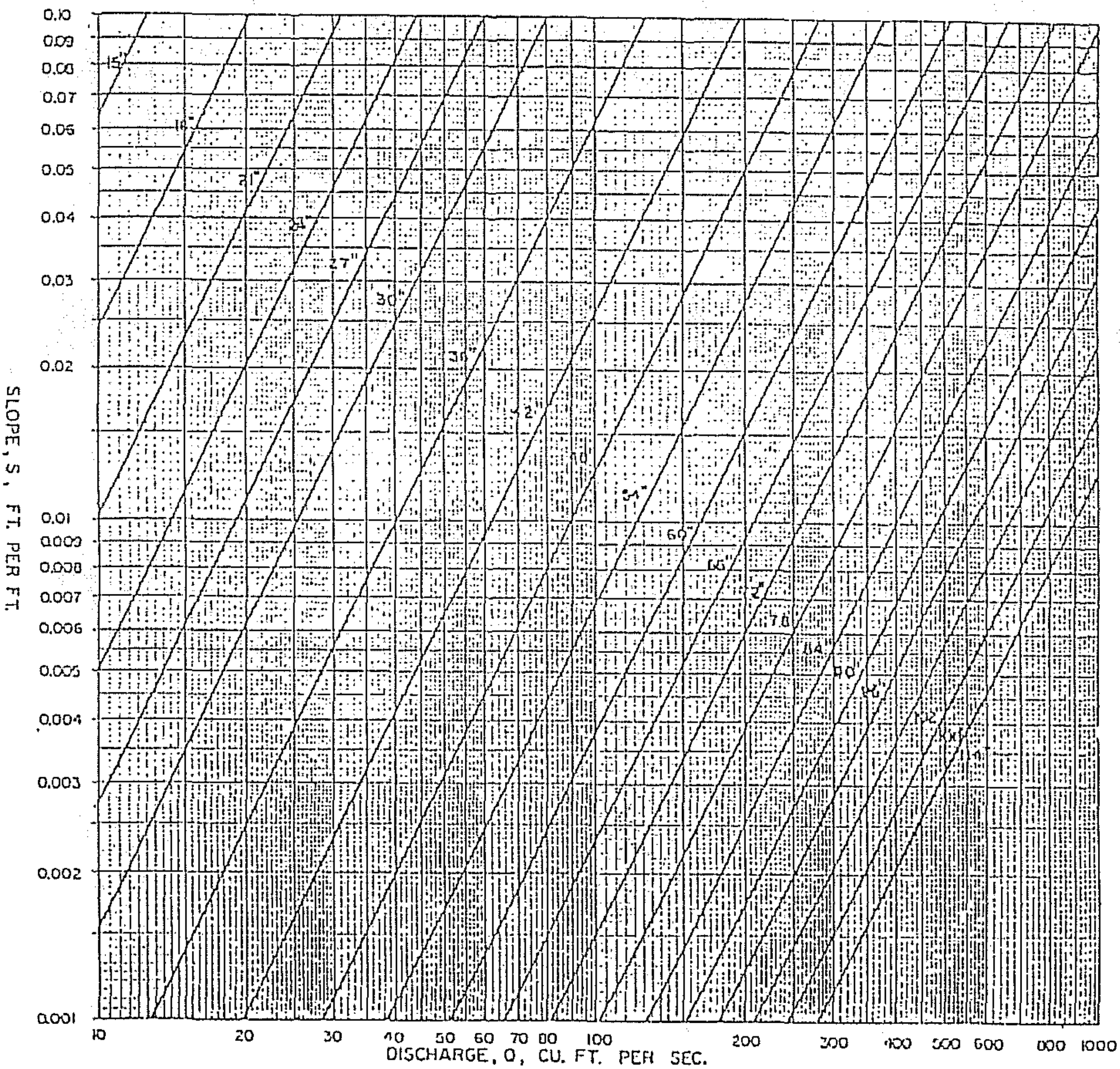
THE CITY OF BENTONVILLE

NOMOGRAPH FOR PIPES FLOWING FULL

n = 0.015 and 0.013

SOURCE: City of Shreveport, LA

Figure 4.19



DISCHARGE FOR CIRCULAR PIPE FLOWING FULL

n = 0.021

SOURCE: City of Shreveport, LA

Figure 4.20

4.9 Examples of Culvert Sizing Computations

Example 1:

Given:

$Q = 326$ cfs
$S_o = 0.002$ ft./ft.
Allowable headwater depth, HW=6.0 ft.
Allowable outlet velocity, V=8.0 fps
Length of Culvert, L=200 ft.±
Tailwater depth, TW=2.6 ft.
Flared Wingwalls

Required: The most economical concrete box culvert that will pass the design discharge.

Solution:

(1) Enter Figure 4.8 with Q=326 and V_c=8.0 and read approximate width of opening. W=20' and d_c=2.0', then connect K value for flared wings = 1.15 with Vc=8.0 and read HL=1.2'. Then

$HW_c = d_c + HL$ or $2.0 + 1.2 = 3.2'$

From the above calculations it appears that a culvert having a width of 20' and a height of 3.2' will adequately pass the design discharge. In order to fit a standard design it is decided to try a 4 - 5' x 4' multiple box culvert.

(2) The next step is to determine the type of culvert operation. This is accomplished by first determining the critical slope by entering Figure 4.9 with $\frac{d_c}{W=5} = \frac{2}{5} = 0.4$ and W=5 and establishing a point on the turning line. Connect the point on turning line with $Q = \frac{326}{4} = 81.5$ and read S_c=0.0037.

We have now assembled the following data:

Existing Channel	Culvert
$S_o = 0.002$ ft./ft.	$S_c = 0.0037$
TW = 2.6'	$d_c = 2.0'$
	D = 4.0'

Also we know the following:

$S_o < S_c$
$TW > d_c$
$TW < D$

This culvert will function as a Type II operation with the control at the outlet providing HW<1.2D.

(3) The next step is to determine the actual headwater depth and to confirm the Type II operation.

$$HW = TW + \left(\frac{V_{TW}}{2g}\right)^2 + h_e + h_f - S_oL$$

$TW = 2.6'$

$$\left(\frac{V_{TW}}{2g}\right)^2 = \frac{\left[\frac{Q}{A}\right]^2}{64.4} = \frac{\left(\frac{326}{20 \times 2.6}\right)^2}{64.4} = \frac{39.31}{64.4} = 0.61'$$

$$h_e = K_e \left(\frac{(V_{TW})^2}{2g}\right) = 0.15 \times 0.61 = 0.09$$

$h_f = S_f L$ Enter Figure 4.9 with

$\frac{d_{TW}}{W} = \frac{2.6}{5} = 0.52$, $W = 5$ and

$Q = \frac{326}{4} = 81.5$ and read $S_f = 0.0019$ ft./ft.

$h_f = 0.0019 \times 200 = 0.38'$

$S_o L = 0.002 \times 200 = 0.40'$

$HW = 2.60 + 0.61 + 0.09 + 0.38 - 0.40 = 3.28'$

The computation of the headwater depth confirms the Type II operation since HW≤1.2D.

(4) The outlet velocity $= \frac{Q}{A} = \frac{326}{20 \times 2.6} = 6.3$ fps

Since the calculated HW=3.27' which is substantially less than the allowable HW=6.0' and the calculated V=6.3 fps which is less than the allowable V = 8.0 fps, the above structure is considered uneconomical.

Example 2:
Given: Same data as in Example 1.
Try 2 – 6.5' x 4' multiple box culvert.
Solution:

(1) From Figure 4.8 d_c=2.65, V_c=9.30

(2) From Figure 4.9 S_c=0.0035 ft./ft.
since S_o<S_c
since TW<d_c
We have a Type I operation with control at the outlet providing HW≤1.2D.

(3) Check HW for Type I operations:

$$HW = d_c + \frac{V_c^2}{2g} + h_e + h_f - S_oL$$

d_c = 2.65'

$$\frac{V_c^2}{2g} = \frac{(9.30)^2}{64.4} = 1.34'$$

$$h_e = K_e\left(\frac{V^2}{2g}\right) = 0.15 \times 1.34' = 0.20'$$

$h_f = S_fL$ Enter Figure with

$$\frac{1.1\ d_c}{W} = \frac{1.1 \times 2.65}{6.5} = 0.45,\ W = 6.5'$$

$$Q = \frac{326}{2} = 163 \text{ and read } S_f = 0.00275 \text{ ft./ft.}$$

$h_f = S_fL$ = 0.00275 x 200 = 0.55'

S_oL = 0.002 x 200 = 0.40'

HW = 2.65 + 1.34 + 0.20 + 0.55 – 0.40 = 4.34'

Since HW<1.2D the installation will function as a Type I operation.

(4) Outlet Velocity = V_c = 9.30 fps.

HW is still lower than the allowable HW=6.0'; however, the outlet velocity is greater than the allowable which was assumed to be 8 fps. The designer has the choice to provide riprap in the downstream channel, select a multiple box culvert of greater width or consider Type IV operation.

Example 3:
Given: Same data as in Example 1.
Required: Multiple Box Culvert for Type IV operation.
Solution:

For the given data let us select a 2 – 5' x 4' multiple box culvert. HW must be equal to or greater than 1.2D, or HW=1.2 x 4.0 = 4.8' minimum. A partially submerged outlet (Type IV-B) will be considered. Under these conditions:

$$HW = H + P - S_oL$$

(1) Area of one barrel= 5x4=20 sq. ft. Length of Culvert= 200 ft. K_e (Flared Wingwalls)= 0.4

$$Q \text{ per barrel} = \frac{326}{2} = 163 \text{ cfs}$$

(2) Use Figure 4.11, Connect area of one barrel–20 sq. ft. with 200 ft. length on K_e=0.4 scale. The position of K_e=0.4 must be interpolated between the limits K_e=0.2 and K_e=0.7. Mark point on turning line. Connect this point with Q=163 and read H=2.3'

(3) According to the definition,

$$P = \frac{d_c + D}{2}$$

Enter Figure 4.8 with Q=326, W=10 and read d_c=3.1'

$$\text{Then } P = \frac{3.1 + 4.0}{2} = 3.55'$$

And HW = 2.3 + 3.55 – (0.002 x 200) HW=5.45'

(4) $V \text{ (outlet)} = \frac{Q}{A} = \frac{326}{10 \times 3.1} = 10.5 \text{ fps}$ (concrete apron req'd.)

Note: Had TW been higher than I) we would have had a submerged outlet and Type IV-A Flow would have controlled

$HW = H + TW - S_oL$ and $V \text{ (outlet)} \ \frac{Q}{A}$

Example 4:

Given: To illustrate Type III operation assume the same data as in Example 1 except that S_o = 0.005 and the allowable outlet velocity = 10.0 fps.

Required: To determine the size of concrete box culvert.

Solution:

(1) Enter Figure 4.8 with Q=326 cfs and V_c=10.0 fps and read W=10', d_c=3.1' and HL=1.3'. Then

$$HW_c = d_c + HL = 3.1 + 1.3 = 4.4'$$

(2) 10' x 5' single box culvert.

To determine the type of operation first find S_c by entering Figure 4.9 with $\frac{d_c}{W} = \frac{3.1}{10}$ = 0.31, W=10' and establish a point on the turning line. Connect this point with Q=326 cfs and read S_c=0.00295 ft./ft.

We now have assembled the following data:

Existing Channel	Culvert
S_o=0.005 ft./ft.	S_c=0.00295 ft./ft.
TW=2.6'	d_c=3.1'

Since $S_o > S_c$
And TW<D

indications are the structure will function as Type III operation providing the HW<1.2D.

(3) For Type III operation the control is critical depth at the entrance and

$$HW = \frac{HW}{D} \text{ (from Nomograph)} \times D$$

Check HW:

Enter Figure 4.10 with $\frac{Q}{W} = \frac{326}{10} = 32.6$ and D=5'

and determine $\frac{HW}{D} = 1.0$

Then HW = 1.0 x D = 1.0 x 5 = 5'

(4) The velocity for Type III culverts varies from critical velocity at the entrance to uniform velocity at the outlet provided the culvert is sufficiently long. We assume in this example that the outlet velocity is equal to the uniform velocity which is computed as follows:

Enter Figure 4.9 with S_o=0.005, Q=326 and W=10 and determine $\frac{d}{W} = 0.26$

d = 0.26W = 0.26 x 10 = 2.6

A = 10 x 2.6 = 26.0 sq. ft.

$V \text{ (uniform)} = \frac{Q}{A} = \frac{326}{26.0} = 12.5$ fps (Outlet requires riprap)

Example 5:
Given:

$Q = 326$ cfs

$S_o = 0.002$ ft./ft.

Allowable headwater depth, HW = 6.5 ft.

Allowable outlet velocity, V = 8.0 fps

Length of Culvert, L = 200 ft. ±

Tailwater depth, TW = 2.6 ft.

Square edge with headwall

Required: Determine size of concrete pipe culvert to pass the design discharge.

Solution:

(1) Use Figure 4.16, connect $\frac{HW}{D} = 1.2$ with Q = 326 and read approximate opening required = 80 inches. Since the allowable HW is restricted to 6.5' and HW for 80" pipe =1.2x 6.7=8.0' the designer tries 2 – 60" pipes, and HW =1.2 x 5.0 = 6.0'.

(2) Use Figure 4.12, connect $Q = \frac{326}{2} = 163$ with D=60" and read $\frac{d_c}{D} = 0.73$

$d_c = 0.73D = 0.73 \times 5.0 = 3.65'$

(3) Use Figure 4.13, connect 60" with $\frac{d_c}{D} = 0.73$ and intersect turning line. Connect turning line with Q=163 and determine $S_c = 0.0046$ for concrete pipe.

We now have assembled the following data:

Existing Channel	Culvert
S_o=0.002 ft./ft.	S_c=0.0046 ft./ft. (Conc.)
TW=2.6'	d_c=3.65'
	D = 5.0'

Since $S_o < S_c$ and TW< d_c, we have a Type I operation with control at the outlet, providing HW≤1.2D.

(4) The next step in this design is to determine the actual headwater depth and to confirm the Type I operation.

$$HW = d_c + \frac{V_c^2}{2g} + h_e + h_f - S_oL$$

$d_c = 3.65'$

For $\frac{d_c}{D} = 0.73'$ V_c (Figure 4.14) = 10.7 fps

$$\frac{V_c^2}{2g} = \frac{(10.7)^2}{64.4} = 1.77'$$

$h_e = 0.5 \times 1.77 = 0.89'$

h_f is calculated as follows:

$1.1\ d_c = 1.1 \times 3.65 = 4.01'$

$$\frac{1.1\ d_c}{D} = \frac{4.01}{5.0} = 0.8$$

To determine the friction slope, S_f,

enter Figure 4.13 with D = 60", $\frac{d_c}{D} = 0.8$

Q = 163 and determine $S_f = 0.0038$

$h_f = S_f L = 0.0038 \times 200 = 0.76'$

$S_o L = 0.002 \times 200 = 0.40'$

HW = 3.65 + 1.77 + 0.89 + 0.76 – 0.40 = 6.67'

(5) Since HW>1.2D for the concrete pipe, the concrete pipe will not function as Type I operation. Also the HW exceeds the allowable.

(6) The designer must now try another pipe size to carry the design flow. Try 2 – 66" pipes.

(7) Use Figure 4.12, connect Q = 163 cfs with D=66" and read $\frac{d_c}{D} = 0.65$

$\frac{d_c}{D} = 0.65D = 0.65 \times 5.5 = 3.58'$

(8) Use Figure 4.13, connect 66" with $\frac{d_c}{D} = 0.65$ and intersect turning line. Connect turning line with Q=163 and determine $S_c = 0.004$.

We have now assembled the following data:

Existing Channel	Culvert
S_o=0.002 ft./ft.	S_c=0.004 ft./ft.
TW=2.6'	d_c=3.58'
	D = 5.5'

Since $S_o < S_c$ and TW< d_c, we have a Type I operation, providing HW<1.2D.

(9) Check to determine the actual headwater depth and to confirm the Type I operation.

$$HW = d_c + \frac{V_c^2}{2g} + h_e + h_f - S_o L$$

$d_c = 3.58'$

For $\frac{d_c}{D} = 0.65$; from Figure 4.14, $V_c = 10.0$ fps

$$\frac{V_c^2}{2g} = \frac{(10)^2}{64.4} = 1.55'$$

$h_e = 0.5 \times 1.55 = 0.78'$

$$\frac{1.1\ d_c}{D} = \frac{1.1 \times (3.58)}{5.5} = 0.72$$

From Figure 4.13 with D = 66", $\frac{d}{c}$ = 0.72, and Q=163 determine S_f = 0.0032

$h_f = S_fL = 0.0032 \times 200 = 0.64'$

$S_oL = 0.002 \times 200 = 0.40'$

HW = 3.58+ 1.55 + 0.78 + 0.64 – 0.40 = HW = 6.1'

(10) Since HW<1.2D, the pipe will function as Type I operation. Also the headwater is calculated to be less than the allowable.

(11) Check outlet velocity to determine if within allowable.

Outlet velocity = V_c = 10 fps

This velocity is greater than allowable. The designer must consider providing riprap in the downstream channel or some type of energy dissipation method or try another size pipe culvert.

Example 6:

Given: To illustrate Type III operation assume the same data as in Example 5 except that $S_o = 0.02$ and the allowable outlet velocity = 15 fps due to a solid rock channel.
Solution:

Follow the same procedure as in Example 5 for determining the initial size, critical depth and critical slope which is summarized below:

Existing Channel	Culvert
S_o=0.02 ft./ft.	S_c=0.0046 ft./ft. (Conc.)
TW=2.6'	d_c=3.65'
	D = 5.0'

Since $S_o > S_c$ and TW< D, the installation will function as Type III operation, providing the entrance is unsubmerged, i.e. HW<1.2D.

(1) The next step in this design is to determine the actual headwater depth and to confirm the Type III operation.

$$HW = \frac{HW}{D} \times D$$

$\frac{HW}{D}$ (Figure 4.16 = 1.13 for concrete pipe.)

HW (Conc. – grooved end with headwall) = 1.13D = 1.13 x 5.0 = 5.65'

Since HW<1.2D the concrete pipe will function as Type III operation.

(2) The velocity for Type III operation varies from critical velocity at the entrance to uniform velocity at the outlet, providing the installation is sufficiently long and the TW depth = uniform depth.

Enter Figure 4.13 with S_o = 0.02, Q = 163

D = 60" and determine

$\frac{d}{D}$ = 0.45, d = 0.45D = 0.45 x 5.0 = 2.25

Since TW≥2.25 the outlet velocity is based on TW depth as follows:

$$\frac{d_{TW}}{D} = \frac{2.25}{5.0} = 0.45$$

Enter Figure 4.14 with D = 60", Q = 163 and the controlling

$\frac{d}{D}$ ratios and determine

V (outlet – Conc.) = 19.0 fps

Some provisions must be made to reduce the outlet velocity to the allowable velocity.

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SECTION V - STORMWATER DETENTION

5.1 GENERAL

Stormwater runoff and the velocity of discharge are considerably increased through development and growth of the City. Prior to the development of land, surface conditions provide a high percentage of permeability and a longer time of concentration. With the construction of buildings, parking lots, etc., permeability and the time of concentration are significantly decreased. These modifications may create harmful effects on properties downstream.

Criteria for differential runoff and detention guidelines are set out below to attempt to decrease the possible effects of development on downstream properties due to increased runoff.

Downstream Conditions: A field study of the downstream capacity of all drainage facilities and the effects from the area to be improved shall be submitted which includes an area equal to twenty (20) times the project area or one-half (1/2) mile minimum distance along the drainage path.

1. No increase in peak flow discharge from the one hundred (100) year precipitation event down to and including the two (2) year event shall be allowed into areas within the City of Bentonville, Arkansas, or the city's planning jurisdiction, except as provided in item 2 below.
2. A waiver of detention may be requested if circumstances exist.
3. An acceptable solution must be presented to the Planning Commission as a part of the PROJECT DRAINAGE PLAN, which may include detention design and/or other on-site and/or off-site improvements as required to meet the intent of these regulations and have no negative impact on adjacent property or watersheds.

5.2 VOLUME OF DETENTION

Volumes of detention shall be evaluated according to the following methods:

A. Volume of detention for basins with total drainage areas of less than 25 acres may be evaluated by the "Modified Rational Hydrograph Method".

B. For basins with total drainage areas larger than 25 acres, the City Engineer retains the right to require submittal of proposed method of evaluation for the sizing of the retention basin or detention basin. The method will be evaluated for a professional acceptance, applicability, and reliability by the City Engineer.

5.3 DESIGN CRITERIA

Stormwater detention ponds shall be designed to limit the peak stormwater discharge rate of the 2, 10, 25, 50, and 100 year storm frequencies after development to predevelopment rates.

5.4 METHOD OF DETENTION

The following conditions and limitations shall be observed in selection and use of the method of detention:

5.4.1 GENERAL LOCATION

Detention facilities shall be located within the parcel limits of the project under consideration. No detention or ponding will be permitted within public road right-of-ways. Location of detention facilities immediately upstream or downstream of the project will be considered by special request if proper documentation is submitted with reference to practicality, feasibility, and proof of ownership or right-of-use of the area proposed. Conditions for general location of detention facilities are identified in the following sections.

5.4.2 DRY RESERVOIRS

Wet weather ponds or dry reservoirs shall be designed with proper safety, stability, and ease of maintenance facilities, and shall not exceed eight (8) feet in depth. Maximum side slopes for grass reservoirs shall not exceed one (1) foot vertical for three (3) feet horizontal (3:1) unless adequate measures are included to provide for the above noted features. In no case shall the limits of maximum ponding elevation be closer than twenty (20) feet horizontally from any building and less than one and a half (1.5) feet vertically below the lowest sill or floor elevation. The entire reservoir area shall be sodded as required prior to final plat approval or issuance of certificate of occupancy. The reservoir area shall include bottom, all side slopes (interior and exterior), and top of berm/embankment. Any area susceptible to, or designed as, overflow by higher design intensity rainfall shall be sodded or paved depending upon the outflow velocity. Concrete trickle channel shall be constructed from all inlets into pond to discharge.

5.4.3 DETENTION IN OPEN CHANNELS

Open channels may be used as detention areas provided that the limits of the maximum ponding elevation are not closer than twenty (20) feet horizontally from any buildings, and not less than one and a half (1.5) feet below the lowest sill or floor elevation of any building. No ponding will be permitted within public road right-of-way unless approval is given in writing by the City Engineer. Maximum depth of detention in open channels shall be four (4) feet. Minimum flow line grade shall be 1.0 percent for grass or untreated bottoms or 0.5 percent for paved channels.

For trapezoidal sections, the maximum side slopes of the channel used for detention shall not exceed one (1) foot vertical for three (3) feet horizontal (3:1). For design of other typical channel sections, the features of safety, stability, and ease of maintenance shall be observed by the Design Engineer.

Unless concrete lined, the entire reservoir area of the open channel shall be sodded as required in the original design. The hydraulic or water surface elevations resulting from channel detention shall not adversely effect adjoining properties.

5.4.4 PERMANENT LAKES OR RETENTION PONDS/WET PONDS

Permanent lakes with fluctuating volume controls may be used as detention areas provided that the limits of maximum ponding elevations are no less than one and a half (1.5) feet below the lowest sill or floor elevation of any building.

Maximum side slopes for the fluctuating area of permanent lakes shall be one (1) foot vertical to three (3) feet horizontal (3:1) unless provisions are included for safety, stability, and ease of maintenance.

Special consideration such as fencing, shoreline slope, depth of water, and abrupt change of grade in inundated areas of two (2) feet or more depth, etc., shall be given to the safety of small children and the public in design of permanent lakes in residential areas.

The entire fluctuating area of the permanent reservoir shall be sodded. Also, calculations must be provided to ensure adequate "live storage" is provided for the 100 year storm. Any area susceptible to or designed as overflow by higher design intensity rainfall (100-year frequency) shall be sodded or paved, depending on the design velocities. An analysis shall be furnished of any proposed earthen dam construction soil. A boring of the foundation for the earthen dam may be requested by the City

Engineer. Earthen dam structures shall be designed by a Professional Engineer.

Aeration devices required for lakes/ponds less than 10-acre water surface area.

5.4.5 PARKING LOTS

Detention is permitted in parking lots to maximum depths of 12 inches. In no case should the maximum limits of ponding be designed closer than twenty (20) feet from a building.

The minimum freeboard and the maximum ponding elevation to the lowest sill or floor elevation shall be one and a half (1.5) feet for the 100 year precipitation event.

5.4.6 OTHER METHODS

Underground detention is acceptable if designed conditions warrant. Parking lots and associated curb and gutter is to serve as the overflow and/or freeboard area. Freeboard is used to account for issues such as sedimentation, clogged discharge, factor of safety, etc. If a parking lot or overflow area is available on site, there is no need for additional storage. If a parking lot or overflow area is not available on site, and/or any overflow would immediately discharge to adjacent property, extra storage/volume should be provided (designed for 105%).

Other methods of detention such as seepage pits, French drains, etc., are discouraged. If other methods are proposed, proper documentation of soil data, percolation, geological features, etc., will be needed for review and consideration.

5.4.7 VERIFICATION OF ADEQUACY

Projects shall provide documented verification of adequacy according to the scope and complexity of design signed originally and certified as-built by the same Arkansas registered professional engineer, if feasible.

5.4.8 CONTROL STRUCTURES

Detention facilities shall be provided with effective control structures. Plan view and sections of the structure with adequate details shall be included in Plans.

The structure selected shall have documented evidence that it will control the 2, 10, 25, 50, and 100 year storms.

The overflow opening or spillway shall be designed to accept the total peak runoff of the improved tributary area. Conveyance for any off-site drainage shall also be provided for.

5.4.9 DISCHARGE SYSTEMS

For site-specific runoff, the effectiveness of local detention structures can be acknowledged in the design of any on-site downstream drainage facilities assuming that the detention facilities comply with all criteria and that they are properly constructed and maintained.

In the case of regional detention basins, sizing of the system below the control structure shall be for the total improved peak runoff tributary to the structure with no allowance for detention unless approved in writing by the City Engineer.

In the event the Engineer desires to incorporate the flow reduction benefits of existing upstream detention ponds, the following field investigations and hydrologic analysis <u>will be required</u>: (Please note that under no circumstances will the previously approved construction plans of the upstream pond suffice as an adequate analysis. While the responsibility of the individual site or subdivison plans rests with the Engineer of Record, any subsequent engineering analysis must assure that all the incorporated ponds work collectively.)

1. A field survey of the existing physical characteristics of both the outlet structure and ponding volume. Any departure from the original engineer's design must be accounted for. If a dual use for the detention pond exists, (e.g., storage of equipment), then this too should be accounted for.

2. A comprehensive hydrologic analysis which simulates the attenuation of the contributing area ponds. This should not be limited to a linear additive analysis, but rather a network of hydrographies which considers incremental timing of discharge and potential coincidence of outlet peaks.

5.4.10 OWNERSHIP OF STORMWATER DETENTION PONDS

The ownership of stormwater detention ponds shall remain with the DEVELOPER, his/her successors or assigns, or the PROPERTY OWNER, or a Property Owners Association (POA). The City assumes no responsibility for ownership or maintenance of stormwater detention ponds unless currently owned by the City.

5.4.11 EASEMENTS

Easements shall be provided in Plans for detention facilities.

Easements shall be dedicated in conjunction with platting of subject property, or by separate document in the case of existing platted property. In either case, document to be approved by City Engineer prior to execution.

All detention and retention facilities within a subdivision shall be shown on the final plat as an individual lot and said lot shall be a drainage easement to allow for inspection and maintenance of outfall structure by the City.

Access to the detention facility shall be provided by a minimum 20' wide unobstructed drainage/access easement between public street and facility when the facility and associated easement is not located adjacent to a public right-of-way.

A detention facility located on an individual commercial development does not require a drainage easement unless the detention is shared or located off-site. In this case, the detention facility shall be enclosed within a drainage easement along with necessary access easement.

5.4.12 MAINTENANCE

a. Detention facilities, when mandatory, are to be built in conjunction with the storm sewer installation and grading. Since these facilities are intended to control increased runoff, they must be partially or fully operational soon after the initial clearing of vegetation.

b. Silt and debris connected with early construction shall be removed periodically from the detention area and control structure in order to maintain close to full storage capacity.

c. The responsibility of maintenance of detention facilities in residential subdivision projects shall remain with the DEVELOPER, PROPERTY OWNER, or POA.

d. The responsibility of maintenance of detention facilities in commercial developments shall be the responsibility of the PROPERTY OWNER.

e. Regarding the responsibility of maintenance for detention facilities, if the Developer, property owner, or POA fail to provide a reasonable degree of maintenance and the detention facilities become inoperative or ineffective, the City of Bentonville, Arkansas, may perform remedial work and assess the OWNER the cost of repair and maintenance. (Ord. No. 86-31, Sec. 7)

f. The responsibility of maintenance of the detention facilities and single lot development projects shall remain with the general contractor until final inspection of the development is performed and approved, and a legal occupancy permit is issued. After legal occupancy of the project, the maintenance of detention facilities shall be vested with the OWNER of the detention pond.

5.5 DETENTION BASIN DESIGN PROCEDURE
(Using the Modified Rational Method)

Computer generated computations and output are accepted and subject to review by City Engineer.

1. Compute existing (predevelopment) and proposed (developed) site characteristics:

   A. Drainage Area
   B. Composite Runoff Coefficient
   C. Time of Concentration (use Figures 2.2 and/or 2.4)

2. Determine rainfall intensity for existing conditions (2 through 100 year storm) from City of Bentonville Rainfall Intensity-Duration Curves (Figure 2.5).

3. Compute existing peak runoff rates using Rational Formula Q=CiA - These will also be the maximum allowable release rates from the detention basin.

4. Determine inflow hydrograph using Modified Rational Method (see Figure 5.2 and Example).

5. Find estimated detention volume using Modified Rational Method.

6. Size detention basin based on estimated required volume. Develop stage-storage curve for the detention basin.

7. Size release structure based on allowable release flow. Develop stage-discharge curve for the release structure.

8. Route the inflow hydrographs (developed using Modified Rational Method for the 2 through 100 year storms) through the detention basin using Modified Puls Method. (See Exhibit 5.1).

9. Check routed hydrographs to insure flows do not exceed predevelopment peaks. Adjust detention basin and release structure, if necessary.

5.5.1 MODIFIED RATIONAL METHOD DETENTION BASIN DESIGN PROCEDURE EXAMPLE

<u>Given</u>: A 10 acre site currently agricultural use is to be developed for townhouses. The entire area is the drainage area of the proposed detention basin.

<u>Determine</u>: Maximum Release rate and required detention storage.

<u>Solution</u>:

<u>Step 1</u>:

Determine 100-year peak runoff rate prior to site development. This is the maximum release rate from site after development.

NOTE: Where a basin is being designed to provide detention for both its drainage area and a bypass area; the maximum release rate is equal to the peak runoff rate prior to site development for the total of the areas minus the peak runoff rate after development for the bypass area. This rate for the bypass area will vary with the duration being considered.

Present Conditions $Q = CiA$

$C = .30$
$Tc = 20$ min.
$i100 = 7.0$ in./hr.
$Q100 = .30 (7.0) 10 = 21.0$ cfs (Maximum release rate)

Step 2:

Determine inflow hydrograph for storms of various durations in order to determine maximum volume required with release rate determined in Step 1.

NOTE: Incrementally increase durations by 10 minutes to determine maximum required volume. The duration with a peak inflow less than the maximum release rate or where required storage is less than storage for the prior duration is the last increment.

Future Conditions (Townhouses)

C = .80
Tc = 15 min.
i100 = 7.0 in./hr.
Q100 = .80 (7.7) 10 = 56.0 cfs

Check various duration storms

20 min.	i = 7.0	Q = .80 (7.0)	10 = 56.0 cfs
30 min.	i = 5.8	Q = .80 (5.8)	10 = 46.4 cfs
40 min.	i = 5.0	Q = .80 (5.0)	10 = 40.0 cfs
50 min.	i = 4.4	Q = .80 (4.4)	10 = 35.2 cfs
60 min.	i = 4.0	Q = .80 (4.0)	10 = 32.0 cfs
70 min.	i = 3.7	Q = .80 (3.7)	10 = 29.6 cfs
80 min.	i = 3.4	Q = .80 (3.4)	10 = 27.2 cfs
90 min.	i = 3.1	Q = .80 (3.1)	10 = 24.8 cfs

NOTE: Rainfall intensities are for illustrative purposes only and do not represent actual values for the City of Bentonville.

Maximum Storage Volume is determined by deducting the volume of runoff released during the time of inflow from the total inflow for each storm duration. See Figure 5.1.

V = time x Qin x 60 sec/min - 0.5 x Qout x (Time +Tc) x 60 sec/min.

15 min. Storm Inflow 15 (61.6) 60 sec/min =	55,440 cf
Outflow (0.5) 30 (21.0) 60 sec/min =	18,900 cf
Storage	36,540 cf

20 min. Storm Inflow 20 (56.0) 60 sec/min =	67,200 cf
Outflow (0.5) 35 (21.0) 60 sec/min =	22,050 cf
Storage	45,150 cf

30 min. Storm Inflow 30 (46.4) 60 sec/min =	83,520 cf
Outflow (0.5) 45 (21.0) 60 sec/min =	28,350 cf
Storage	55,170 cf
40 min. Storm Inflow 40 (40.0) 60 sec/min =	96,000 cf
Outflow (0.5) 55 (21.0) 60 sec/min =	34,650 cf
Storage	61,350 cf
50 min. Storm Inflow 50 (35.2) 60 sec/min =	105,600 cf
Outflow (0.5) 65 (21.0) 60 sec/min =	40,950 cf
Storage	64,650 cf
60 min. Storm Inflow 60 (32.0) 60 sec/min =	115,200 cf
Outflow (0.5) 75 (21.0) 60 sec/min =	47,250 cf
Storage	67,950 cf
70 min. Storm Inflow 70 (29.6) 60 sec/min =	124,320 cf
Outflow (0.5) 85 (21.0) 60 sec/min =	53,550 cf
Storage	70,770 cf
80 min. Storm Inflow 80 (27.2) 60 sec/min =	130,560 cf
Outflow (0.5) 95 (21.0) 60 sec/min =	59,850 cf
Storage	70,710 cf
90 min. Storm Inflow 90 (24.8) 60 sec/min =	133,920 cf
Outflow (0.5) 105 (21.0) 60 sec/min =	66,150 cf
Storage	67,770 cf

Step 3:

Route design storm hydrograph through the detention basin using the Modified Puls Routing Method or another approved method, based on final detention basin and release structure design. Computer programs to accomplish this task are readily available. See Exhibit 5.1 for typical required output format.

EXHIBIT 5.1

D E T A I N v1.0 -- Copyright (c) 1992 by -Mate Software

JOB DESCRIPTION: Sample Project for DETAIN v1.0
DATE: 05-18-1992
TIME: 01:00:00

D R A I N A G E R E P O R T

INPUT/OUTPUT FILENAMES:

Report output filename:	REPORT.OUT
Intensity/duration curve filename:	INT_100.DAT

DRAINAGE BASIN CHARACTERISTICS:

Drainage area = 1.50 acres
Length of overland flow = 500.0 feet
Average overland slope = 3.50%
Existing runoff coefficient = 0.40
Developed runoff coefficient = 0.80

CHANNEL CHARACTERISTICS:

Length of channel flow = 300.0 feet
Average channel slope = 0.0050 ft/ft
Average channel width = 4.0 feet
Average channel depth = 1.00 feet
Manning's "n" value = 0.040
Average channel velocity = 2.01 ft/sec

FLOW COMPUTATIONS:

Q = C * i * A (Runoff in cfs)
C = Average runoff coffecient
i = Average rainfall intensity (in/hr)
A = Drainage area (ac)

DESCRIPTION	EXISTING INTENSITY (in/hr)	DEVELOPED INTENSITY (in/hr)	EXISTING RUNOFF (cfs)	DEVELOPED RUNOFF (cfs)	RUNOFF INCREASE (cfs)
100 Year Storm	6.30	8.39	3.78	10.07	6.29
50 Year Storm	5.76	7.66	3.45	9.19	5.73
25 Year Storm	5.20	6.94	3.12	8.32	5.21
10 Year Storm	4.49	5.99	2.69	7.19	4.50
5 Year Storm	3.98	5.38	2.39	6.45	4.06
2 Year Storm	3.29	4.51	1.97	5.41	3.43

EXHIBIT 5.1 (Continued)

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JOB DESCRIPTION: Sample Project for DETAIN v1.0
DATE: 05-18-1992
TIME: 01:00:00

T I M E   O F   C O N C E N T R A T I O N

TIME OF CONCENTRATION (OVERLAND FLOW):

$$To = \frac{1.8 * (1.1 - C) * L^{0.5}}{S^{0.33}}$$

C = Average runoff cofficient
L = Length of basin (ft)
S = Average basin slope (%)

Existing time of concentration = 18.6 minutes
Developed time of concentration = 8.0 minutes

TIME OF TRAVEL (CHANNEL FLOW):

$$Tt = \frac{L}{V}$$

L = Length of channel
V = Average channel velocity

Time of travel = 2.5 minutes

TOTAL TIME OF CONCENTRATION:

Tc = To + Tt

Existing time of concentration = 21.0 minutes
Developed time of concentration = 10.4 minutes

EXHIBIT 5.1 (Continued)

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JOB DESCRIPTION: Sample Project for DETAIN v1.0
DATE: 05-18-1992
TIME: 01:00:00

C O M P O S I T E   R U N O F F   C O E F I C I E N T S

PREDEVELOPMENT AREAS:

DESCRIPTION	AREA (ac)	C
Predevelopment Area No. 1	0.5	.6
Predevelopment Area No. 2	1.0	.3

TOTAL AREA = 1.50 ACRES; AVERAGE RUNOFF COEFFICIENT = 0.40

POSTDEVELOPMENT AREAS:

DESCRIPTION	AREA (ac)	C
Postdevelopment Area No. 1	1.0	.9
Postdevelopment Area No. 2	0.5	.6

TOTAL AREA = 1.50 ACRES; AVERAGE RUNOFF COEFFICIENT = 0.80

D E T E N T I O N   C O M P U T A T I O N S

MODIFIED RATIONAL METHOD:

Volume = Time * Qin * 60 sec/min - 0.5 * Qout * (Time + Tc) * 60 sec/min
Time = Storm duration (min)
Qin = Peak runoff from storm (cfs)
Qout = Maximum discharge (cfs)
Tc = Time of concentration (min)

STORM DURATION (min)	AVERAGE INFLOW (cfs)	MAXIMUM RELEASE (cfs)	REQUIRED DETENTION (cu ft)
10	10.20	3.78	3802.56
11	9.90	3.78	4103.18
12	9.50	3.78	4298.69
13	9.24	3.78	4549.63
14	8.94	3.78	4738.65
15	8.69	3.78	4934.88
16	8.46	3.78	5123.90
17	8.28	3.78	5334.52
18	8.08	3.78	5497.63
19	7.86	3.78	5622.57
20	7.72	3.78	5808.00
21	7.56	3.78	5961.02
22	7.52	3.78	6253.72
23	7.30	3.78	6277.15
24	7.12	3.78	6342.33
25	7.02	3.78	6511.91
26	6.90	3.78	6632.54
27	6.78	3.78	6738.76
28	6.66	3.78	6830.59
29	6.55	3.78	6928.89
30	6.46	3.78	7035.83
31	6.36	3.78	7131.26
32	6.24	3.78	7169.08
33	6.14	3.78	7240.02
34	6.05	3.78	7299.45
35	5.98	3.78	7397.75
36	5.88	3.78	7435.58
37	5.80	3.78	7488.52
38	5.74	3.78	7586.10
39	5.64	3.78	7592.25
40	5.54	3.78	7586.87
41	5.50	3.78	7688.05
42	5.40	3.78	7662.52
43	5.32	3.78	7656.42
44	5.28	3.78	7766.97
45	5.20	3.78	7743.59
46	5.14	3.78	7776.37
47	5.06	3.78	7768.12
48	5.02	3.78	7820.34
49	4.94	3.78	7796.24
50	4.90	3.78	7835.51
51	4.84	3.78	7832.29

STORM DURATION (min)	AVERAGE INFLOW (cfs)	MAXIMUM RELEASE (cfs)	REQUIRED DETENTION (cu ft)
52	4.78	3.78	7821.88
53	4.70	3.78	7766.10
54	4.66	3.78	7779.44
55	4.60	3.78	7747.43
56	4.56	3.78	7788.85
57	4.50	3.78	7743.88
58	4.44	3.78	7691.70
59	4.40	3.78	7717.28
60	4.34	3.78	7652.15
61	4.31	3.78	7663.26
62	4.27	3.78	7669.91
63	4.23	3.78	7672.09
64	4.20	3.78	7669.81
65	4.16	3.78	7663.07
66	4.12	3.78	7651.86
67	4.08	3.78	7636.19
68	4.05	3.78	7616.05
69	4.01	3.78	7591.45
70	3.97	3.78	7562.38
71	3.94	3.78	7569.75
72	3.92	3.78	7573.81
73	3.89	3.78	7574.55
74	3.86	3.78	7571.98
75	3.83	3.78	7566.10
76	3.81	3.78	7556.91
77	3.78	3.78	7544.41 X
78	3.75	3.78	7528.59 X
79	3.72	3.78	7509.46 X

END OF COMPUTATIONS -- MAXIMUM VOLUME = 7835.51; AT TIME = 50

EXHIBIT 5.1 (Continued)

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JOB DESCRIPTION: Sample Project for DETAIN v1.0
DATE: 05-18-1992
TIME: 01:00:00

D E T E N T I O N H Y D R O G R A P H

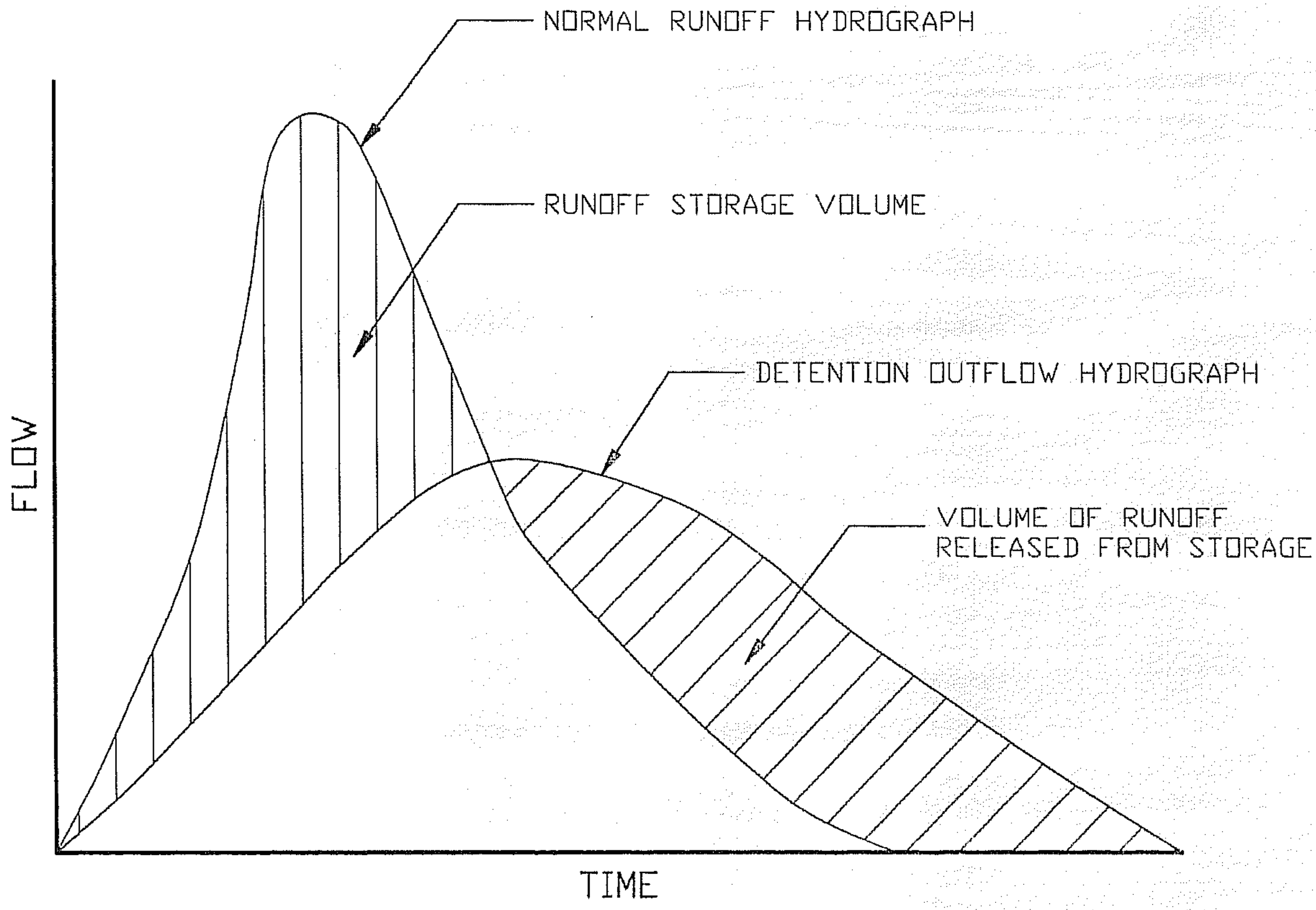
INPUT/OUTPUT FILENAMES:

Inflow hydrograph filename: INFLOW.DAT
Stage/storage curve filename: STORAGE.DAT
Stage/discharge curve filename: OUTFLOW.DAT

TIME (min)	INFLOW (cfs)	STAGE (ft)	OUTFLOW (cfs)	VOLUME (cu ft)
0	0.00	0.00	0.00	0.00
1	0.47	0.02	0.05	13.28
2	0.94	0.08	0.19	48.92
3	1.41	0.12	0.30	104.16
4	1.88	0.16	0.40	182.01
5	2.34	0.21	0.51	280.15
6	2.81	0.24	0.60	401.46
7	3.28	0.29	0.71	545.20
8	3.75	0.33	0.80	710.98
9	4.22	0.37	0.89	899.39
10	4.69	0.41	0.99	1109.87
11	4.90	0.45	1.09	1338.47
12	4.90	0.49	1.18	1564.33
13	4.90	0.52	1.29	1784.28
14	4.90	0.55	1.41	1997.04
15	4.90	0.58	1.52	2202.77
16	4.90	0.61	1.64	2401.70
17	4.90	0.64	1.75	2593.95
18	4.90	0.66	1.85	2779.72
19	4.90	0.71	2.02	2957.67
20	4.90	0.75	2.14	3126.53
21	4.90	0.80	2.27	3287.93
22	4.90	0.84	2.38	3442.21
23	4.90	0.88	2.49	3589.68
24	4.90	0.92	2.60	3730.64

TIME (min)	INFLOW (cfs)	STAGE (ft)	OUTFLOW (cfs)	VOLUME (cu ft)
25	4.90	0.96	2.70	3865.38
26	4.90	1.00	2.80	3994.17
27	4.90	1.03	2.84	4118.69
28	4.90	1.06	2.88	4240.61
29	4.90	1.09	2.93	4360.00
30	4.90	1.12	2.97	4476.91
31	4.90	1.15	3.01	4591.39
32	4.90	1.18	3.05	4703.49
33	4.90	1.20	3.09	4813.26
34	4.90	1.23	3.12	4920.75
35	4.90	1.26	3.16	5026.00
36	4.90	1.28	3.20	5130.09
37	4.90	1.31	3.23	5230.99
38	4.90	1.33	3.27	5329.80
39	4.90	1.36	3.30	5426.55
40	4.90	1.38	3.33	5521.30
41	4.90	1.40	3.37	5614.07
42	4.90	1.43	3.40	5704.92
43	4.90	1.45	3.43	5793.87
44	4.90	1.47	3.46	5880.98
45	4.90	1.49	3.49	5966.28
46	4.90	1.51	3.53	6049.54
47	4.90	1.52	3.58	6129.82
48	4.90	1.53	3.63	6207.14
49	4.90	1.54	3.68	6281.63
50	4.89	1.54	3.72	6353.36
51	4.42	1.55	3.76	6408.37
52	3.95	1.55	3.77	6433.75
53	3.49	1.55	3.77	6430.59
54	3.02	1.55	3.75	6399.93
55	2.55	1.54	3.72	6342.80
56	2.08	1.53	3.66	6260.16
57	1.61	1.52	3.60	6152.97
58	1.14	1.50	3.52	6022.11
59	0.67	1.47	3.45	5867.58
60	0.20	1.42	3.39	5688.46

END OF SIMULATION -- MAXIMUM VOLUME = 6433.75; AT TIME = 52

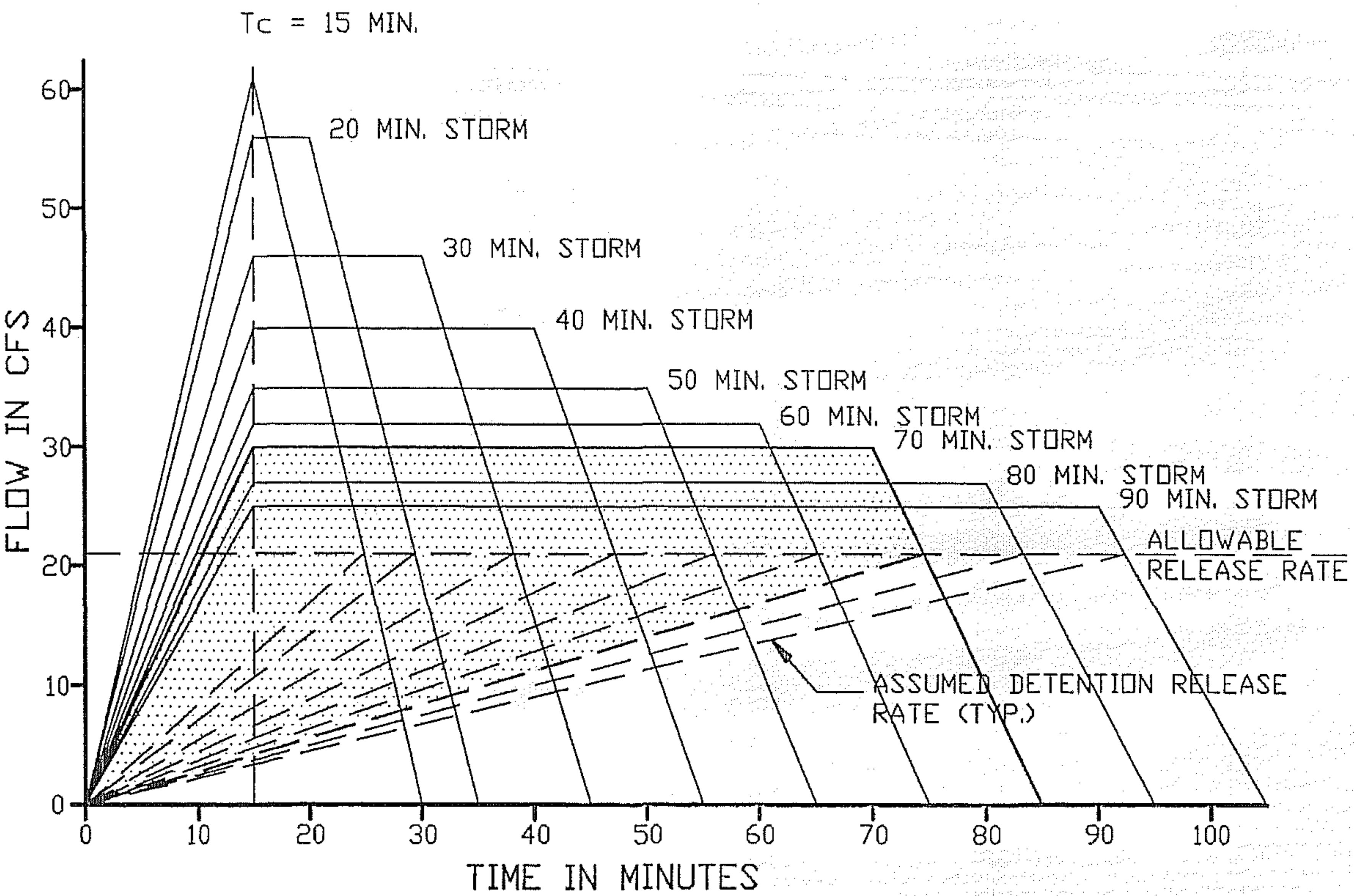


CONCEPT OF DETENTION

CONCEPT OF DETENTION POND

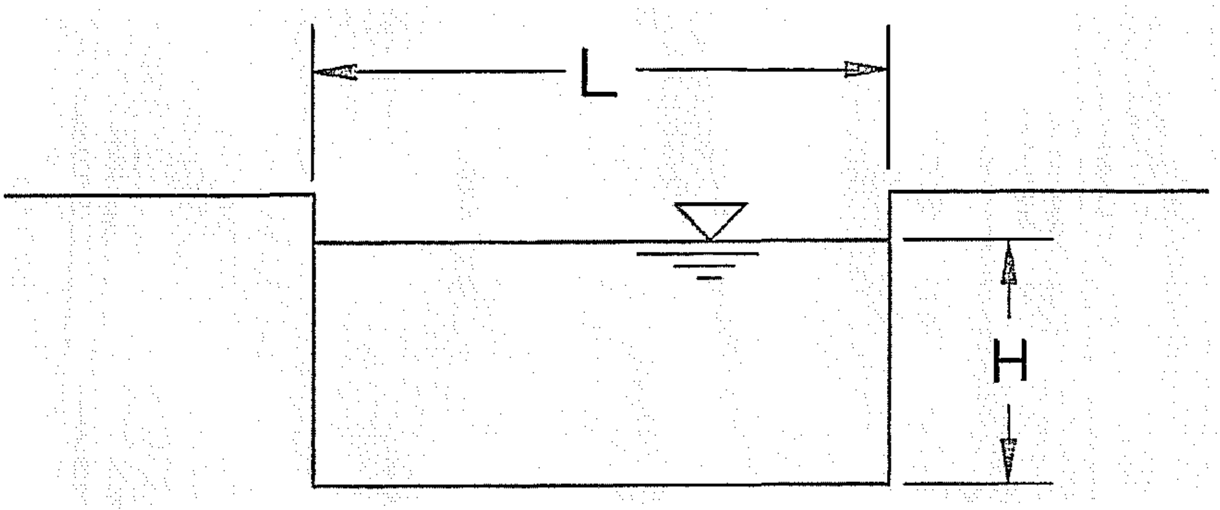
Figure 5.1

EXAMPLE OF MODIFIED RATIONAL METHOD



MODIFIED RATIONAL METHOD EXAMPLE

Figure 5.2



RECTANGULAR WEIR FLOW EQUATION

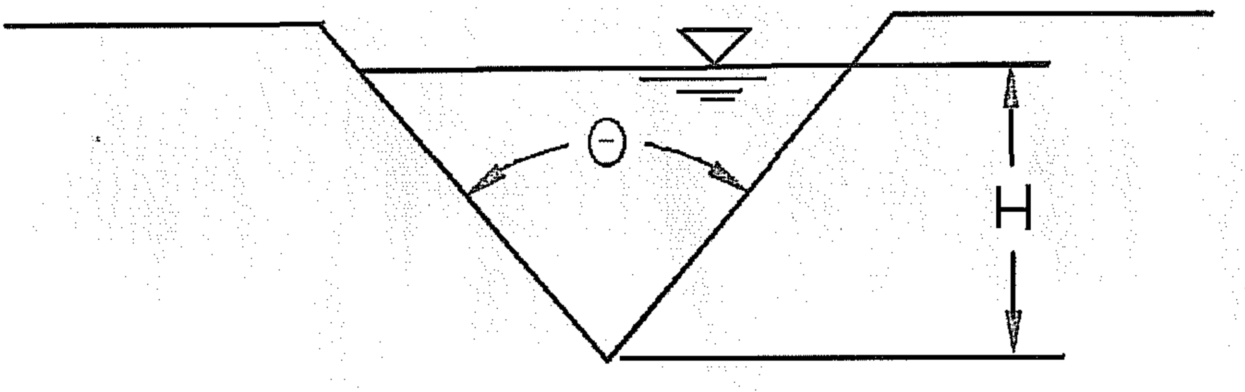
$$Q = CLH^{3/2}$$

where

- Q = Weir discharge in CFS
- C = Weir coefficient
- L = Horizontal length of the weir in feet
- H = Head on the weir in feet

RECTANGULAR WEIR

Figure 5.3



V-NOTCH WEIR FLOW EQUATION

$Q = C \tan(\theta/2)H^{5/2}$

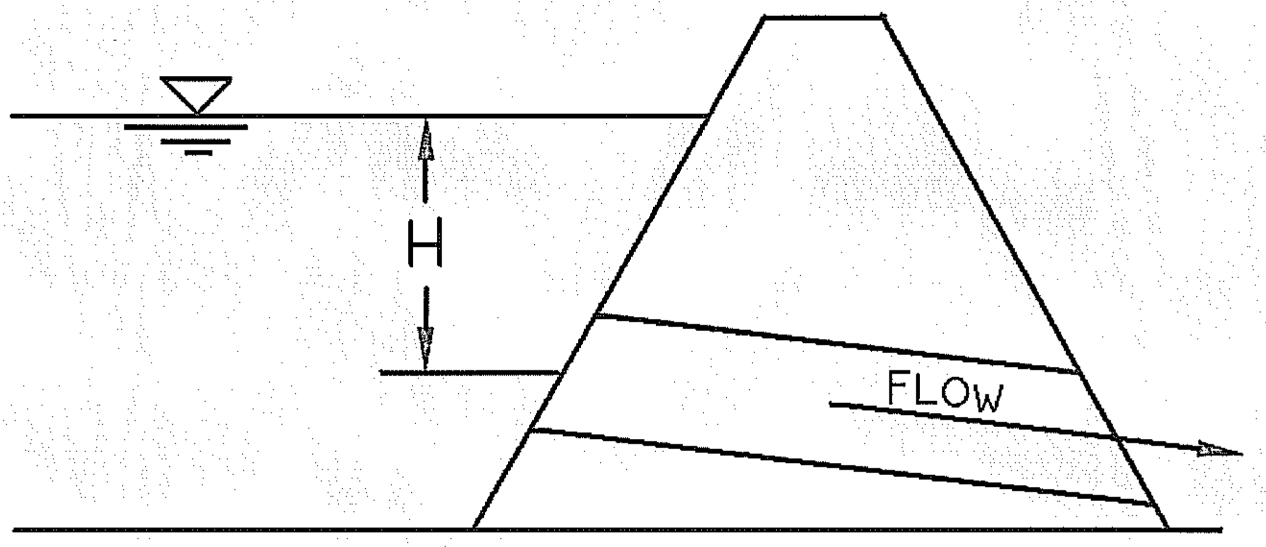
where

Q = Weir discharge in CFS

C = Weir coefficient

θ = Angle of the weir notch in degrees

H = Head on the weir in feet



ORIFICE FLOW EQUATION

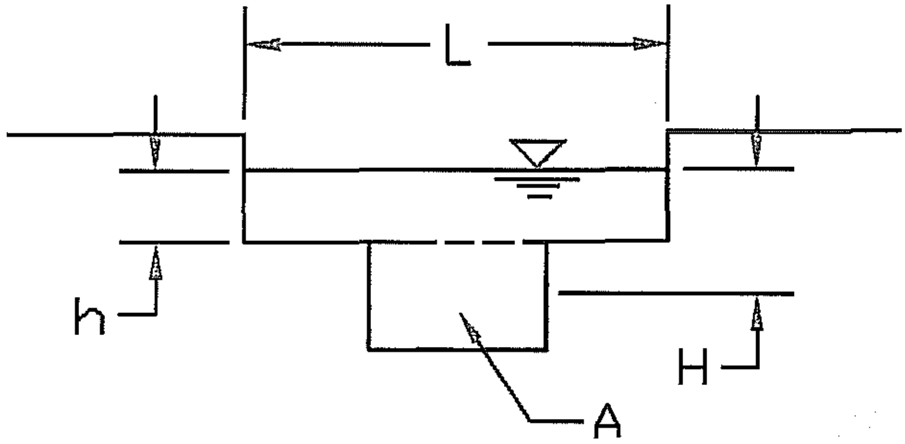
$$Q = CA\,(2gH)^{1/2}$$

where

- Q = Orifice discharge in CFS
- C = Orifice coefficient
- A = Area of orifice in square feet
- g = Gravitational constant (32.2 FT/S^2)
- H = Head on the orifice measured from the centerline in feet

ORIFICE DESIGN

Figure 5.5

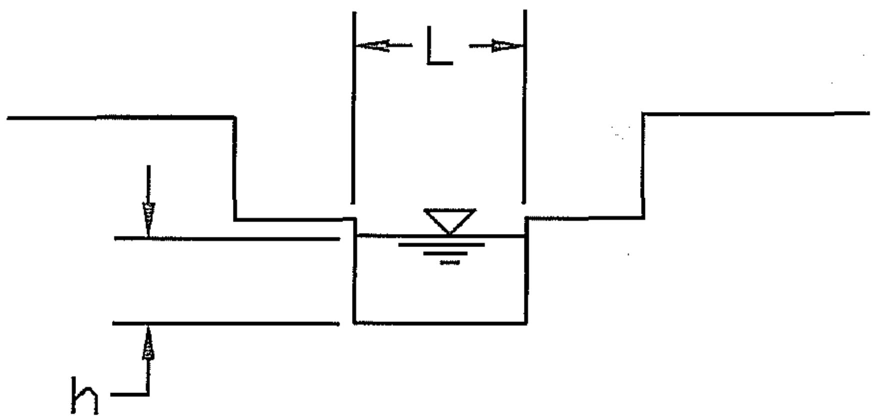


RECTANGULAR WEIR AND ORIFICE FLOW

$$Q_W = C_W L h^{3/2}$$
$$Q_O = C_O A(2gH)^{1/2}$$

where

Q_W = Weir discharge in CFS
Q_O = Orifice discharge in CFS
C_W = Weir coefficient
C_O = Orifice Coefficient
A = Area of orifice in square feet
L = Horizontal length of the weir in feet
h = Head on the weir in feet
H = Head on the orifice in feet



RECTANGULAR WEIR FLOW ONLY

$$Q_W = C_W L h^{3/2}$$

where

Q_W = Weir discharge in CFS
C_W = Weir coefficient
L = Horizontal length of the weir in feet
h = Head on the weir in feet

MULTISTAGE RELEASE STRUCTURES

Figure 5.6

TABLE OF CONTENTS - SECTION VI

SECTION VI - FLOW IN STREETS

6.1 GENERAL

The location of inlets and permissible flow of water in the streets should be related to the extent and frequency of interference to traffic and the likelihood of flood damage to surrounding property. Interference to traffic is regulated by design limits on the spread of water into traffic lanes, especially in regard to arterials. Flooding of surrounding property from streets is controlled by limiting runoff building up to the top of the curb for a 10-year storm.

6.1.1 INTERFERENCE DUE TO FLOW IN STREETS

Water which flows in a street, whether from rainfall directly on to the pavement surface or overland flow entering from adjacent land areas, will flow in the gutters of the street until it reaches an overflow point or some other outlet, such as a storm sewer inlet. As the flow progresses downhill and additional areas contribute to the runoff, the width of flow will increase and progressively encroach into a traffic lane. On streets where parking is not permitted, as with many arterial streets, flow widths exceeding a few feet become a traffic hazard. Field observations show that vehicles will crowd adjacent lanes to avoid curb flow.

As the width of flow increases further, it becomes impossible for vehicles to operate without moving through water and they must use the now inundated lane. Splash from vehicles traveling in the inundated lane obscures the vision of drivers of vehicles moving at a high rate of speed in the open lane. Eventually, if width and depth of flow becomes great enough, the street loses its effectiveness as a traffic-carrier. During these periods, it is imperative that emergency vehicles such as fire trucks, ambulances, and police cars be able to traverse the streets by moving along the crown of the roadway.

6.1.2 INTERFERENCE DUE TO PONDING

Storm runoff ponded on the street surface because of grade changes or the crown slope of intersecting streets has a substantial effect on the street's traffic carrying capacity. Because of the localized nature of ponding, vehicles moving at a relatively high speed may enter a pond. The manner in which ponded water affects traffic is essentially the same as for curb flow, that is, the width of spread into the traffic lane is critical. Ponded water will often completely halt all traffic. Ponding in streets has the added hazard of surprise to drivers of moving vehicles, often producing erratic and dangerous responses.

6.1.3 INTERFERENCE DUE TO WATER FLOWING ACROSS TRAFFIC LANE

Whenever stormwater runoff, other than limited sheet flow, moves across the traffic lane, a serious and dangerous impediment to traffic flow occurs. The cross-flow may be caused by super elevation of the curb, a street intersection, overflow from the higher gutter on a street with cross fall, or simply poor street design. The problem associated with this type of flow is the same as for ponding in that it is localized in nature. Vehicles may be traveling at high speed when they reach the location. If vehicular movement is slow and the street is lightly traveled, as on residential streets, limited cross flows do not cause sufficient interference to be unacceptable.

The depth and velocity of cross flows shall be maintained within such limits that do not have sufficient force to threaten moving traffic.

6.1.4 EFFECT ON PEDESTRIANS

In areas with heavily used sidewalks, splash due to vehicles moving through water adjacent to the curb is a serious problem.

Streets should be classified with respect to pedestrian traffic as well as vehicular traffic. As an example, streets which are classified as residential and located adjacent to a school are arterials for pedestrian traffic. The allowable width of gutter flow and extent of ponding should reflect this fact.

6.1.5 REDUCTION OF ALLOWABLE CARRYING CAPACITY

As the stormwater flow approaches an arterial street, tee intersection, or cul-de-sac, the allowable carrying capacity shall be calculated by multiplying the reduction factor from Figure 6.1 times the theoretical gutter capacity. The grade used to determine the reduction factors shall be the same effective grade used to calculate the theoretical capacity.

6.1.6 STREET CROSS FLOW

Whenever storm runoff, other than limited sheet flow, moves across a traffic lane, a serious and dangerous impediment to traffic flow occurs, therefore, cross flow is not allowed. In case of superelevation of a curve or overflow from the higher gutter on a street with cross fall, potential cross flow is to be collected by inlets prior to the superelevation transition.

6.1.7 ALLOWABLE FLOW OF WATER THROUGH INTERSECTIONS

As the storm water flow approaches an arterial street or tee intersection, an inlet is required. Concrete swales may be used to convey water across residential streets at the intersection of a residential street and a larger capacity street. Swales are not allowed across streets if the design cross flow exceeds 2 cfs..

6.2. PERMISSIBLE SPREAD OF WATER

The depth of flow in the street shall be limited to the top of curb except in FEMA controlled floodplains, where FEMA guidelines shall govern.

6.2.1 PRINCIPAL ARTERIAL STREETS

Inlets shall be spaced at such an interval as to provide one clear traffic lane in each direction during the peak flows of the design storm.

Gutter depressions may not exceed 3 inches unless specifically approved by the City Engineer. The design storm will have a 10 year return frequency. A design storm of 100-year frequency must be accommodated within the limits of the street right-of-way unless approved in writing by City Engineer.

Example:

Street width 60 feet; two 12-foot lanes to remain clear.

Therefore: Street flow in each gutter shall not exceed (60 - 24)/2 = 18 feet.

6.2.2 MINOR ARTERIAL AND COLLECTOR STREETS

The flow of water in gutters of the minor arterial streets shall be limited so that one standard lane will remain clear during the peak runoff from the design storm. Inlets shall be located at low points or wherever the flow exceeds the one standard lane requirement. Gutter depression at the inlets shall not exceed 3 inches. The design storm will have a 10 year return frequency. A design storm of 100-year frequency must be accommodated within the limits of the street right-of-way unless approved in writing by City Engineer.

Example: Street width 49 ft.; one 12-foot traffic lane to remain clear.

Therefore; Street flow in each gutter shall not exceed (49 - 12)/2 = 18.5 ft.

6.2.3 RESIDENTIAL COLLECTOR STREETS (LOCAL)

The flow of water in gutters of a residential collector street shall be limited so that one standard lane will remain clear during the peak runoff from the design storm. Inlets shall be located at low points or wherever the flow exceeds the one standard lane requirement. Gutter depression at the inlets shall not exceed 3 inches. The design storm will have a 10 year return frequency. A design storm of 100-year frequency must be accommodated within the limits of the street right-of-way unless approved in writing by City Engineer.

Example: Street width - 36 ft.; one 12-foot traffic lane to remain clear.

Therefore: Street flow in each gutter shall not exceed (36 - 12)/2 = 12 ft.

6.2.4 RESIDENTIAL STREETS

The flow of water in gutters of a residential street shall be limited to a depth of flow at the curb of 6 inches or wherever the street is just covered, whichever is the least depth. Inlets shall be located at low points, or wherever the gutter flow exceeds the permissible spread of water.

Gutter depression at the inlets shall not exceed 3 inches. The design storm will have a 10-year return frequency. A design storm of 100-year frequency must be accommodated within the limits of the street right-of-way unless approved in writing by City Engineer.

6.3 BYPASS FLOW

Flow bypassing each inlet must be included in the total gutter flow to the next inlet downstream. A bypass of 10 to 20 percent per inlet will result in a more economical drainage system. Refer to Section VII for inlet design.

6.4 MINIMUM AND MAXIMUM VELOCITIES

To ensure cleaning velocities at very low flows, the gutter shall have a minimum slope of 0.01 feet per foot (1.00%, unless otherwise approved by City Engineer). The maximum velocity of curb flow shall be 10 feet per second. Along sharp horizontal curves, peak flows tend to jump behind the curb line at driveways and other curb breaks. Water running behind the curb line can result in considerable damage due to erosion and flooding. In a gutter where the slope is greater than 0.10 feet per foot (10%) and the radius is 400 feet or less, design depth of flow shall not exceed 4 inches at the curb.

6.5 DESIGN METHOD

6.5.1 STRAIGHT CROWNS

Flow in gutters which are straight crown pavements is normally calculated by using Manning's equation for various hydraulic properties for uniform flow in pavement gutters and triangular channels. The equation is:

$$Q_o = 0.56 \frac{z}{n} S_o^{1/2} Y_o^{8/3}$$

Q_o = gutter discharge (CFS)

z = reciprocal of the crown slope (Ft./Ft.)
(foot per foot)

S_o = street or gutter slope (Ft./Ft.)
(foot per foot)

n = roughness coefficient

Y_o = depth of flow in gutter (Ft.)

The nomograph in Figure 6.2 provides for direct solution of flood conditions for triangular channels most frequently encountered in urban street drainage design. For a standard concrete gutter, a value of 0.016 for "n" is recommended.

6.5.2 PARABOLIC CROWNS

Flow in gutters, which are on parabolic crown pavements is calculated from a variation of Manning's equation for steady flow in a prismatic open channel.

$$\log Q = K_o + K_1 \log S_o + K_2 \log Y_o$$

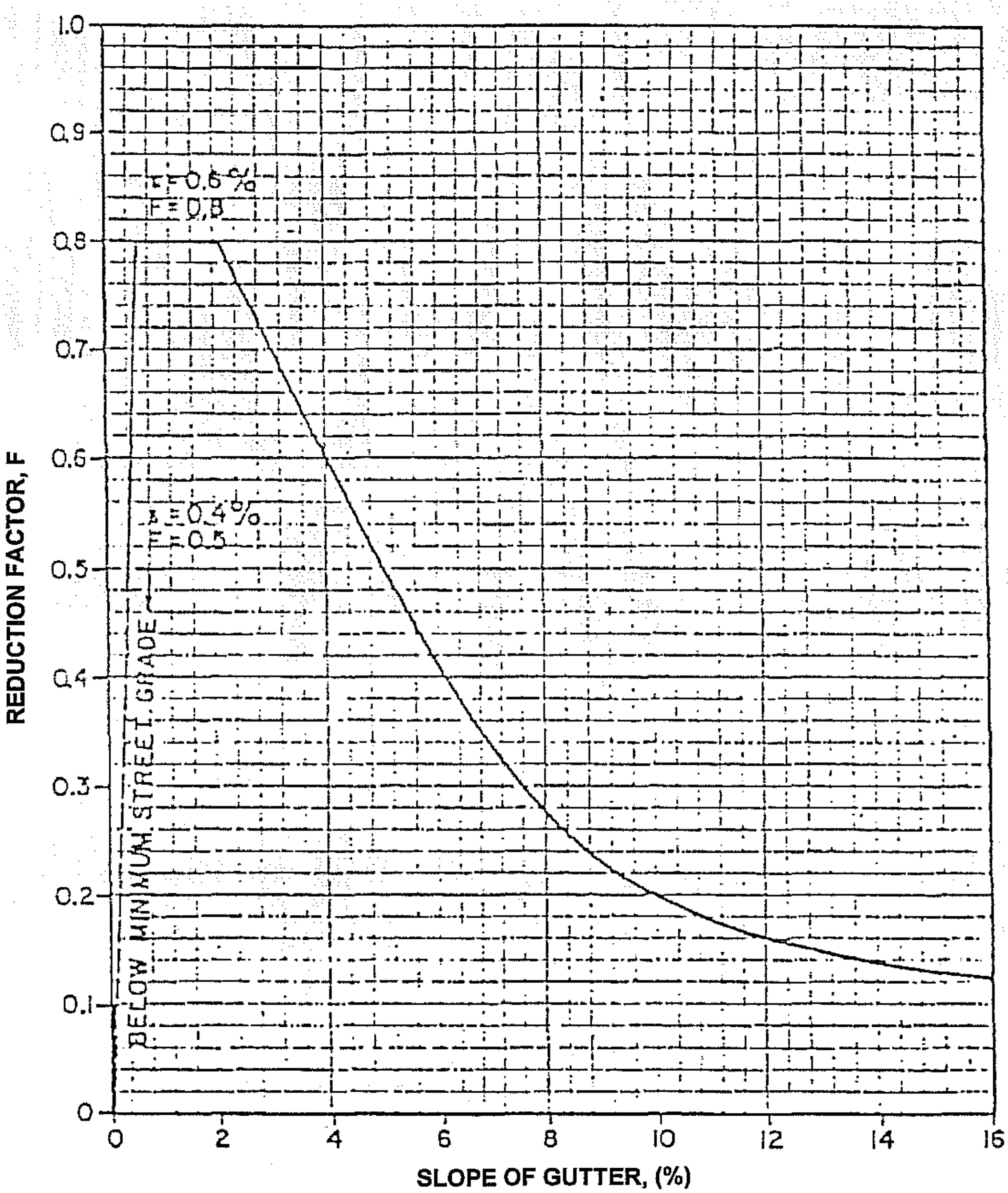
Where,
Q = Gutter flow, cfs
S_o = Street grade, ft/ft
Y_o = Water depth in the gutter, feet
K_o, K_1, K_2 = Constant coefficients for different street widths

Coefficients for Parabolic Streets

Street Width* (ft)	Coefficients K_o	K_1	K_2
30	2.85	0.50	3.03
36	2.89	0.50	2.99
40	2.85	0.50	2.89
44	2.84	0.50	2.83
48	2.83	0.50	2.78
60	2.85	0.50	2.74

* Note: Based on the Transportation Criteria Manual, the street width is measured from face of curb to face of curb (FOC-FOC).

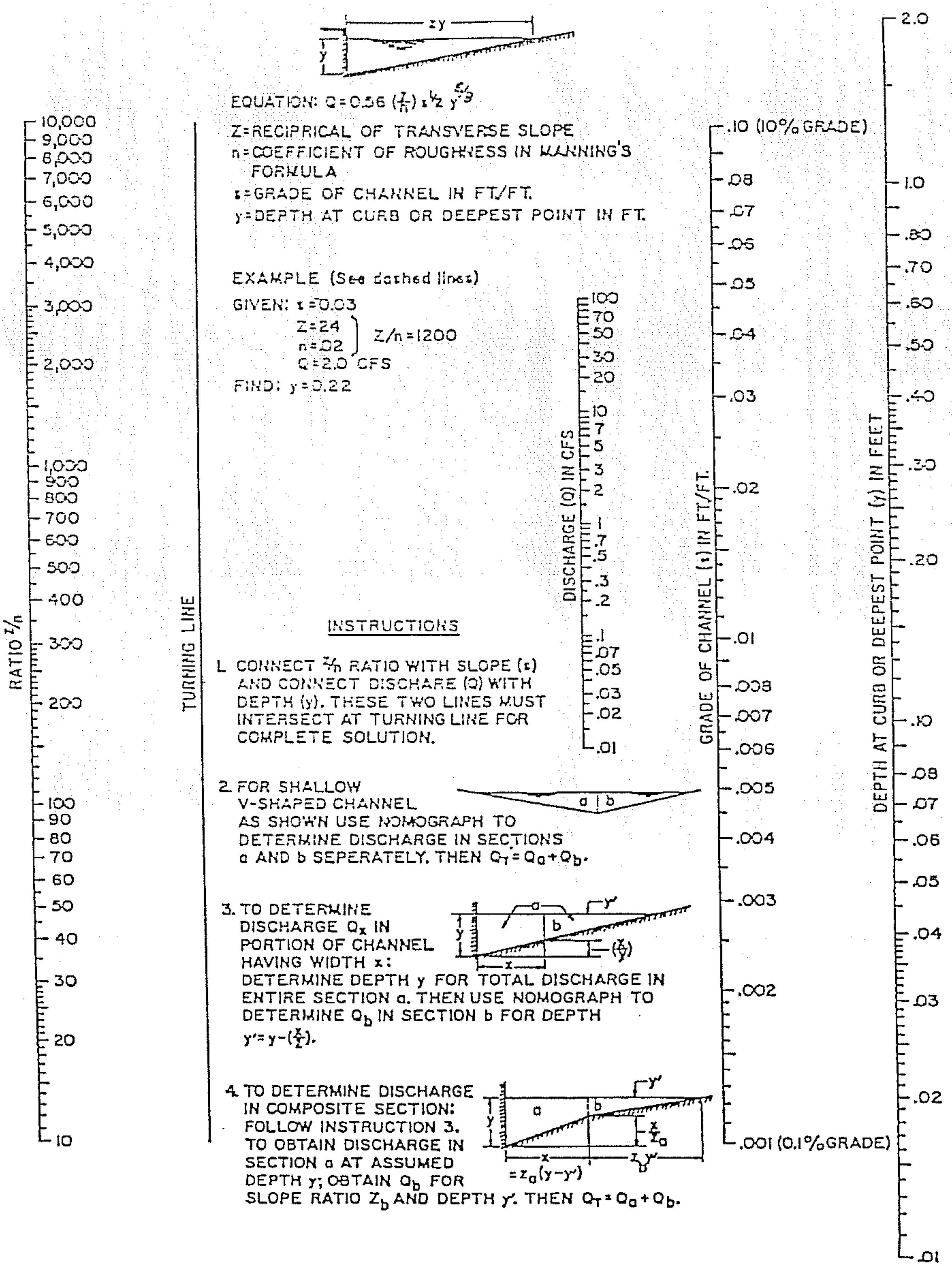
Source: City of Austin, Watershed Management Division



REDUCTION FACTOR FOR ALLOWABLE GUTTER CAPACITY

SOURCE: City of Austin, TX

Figure 6.1



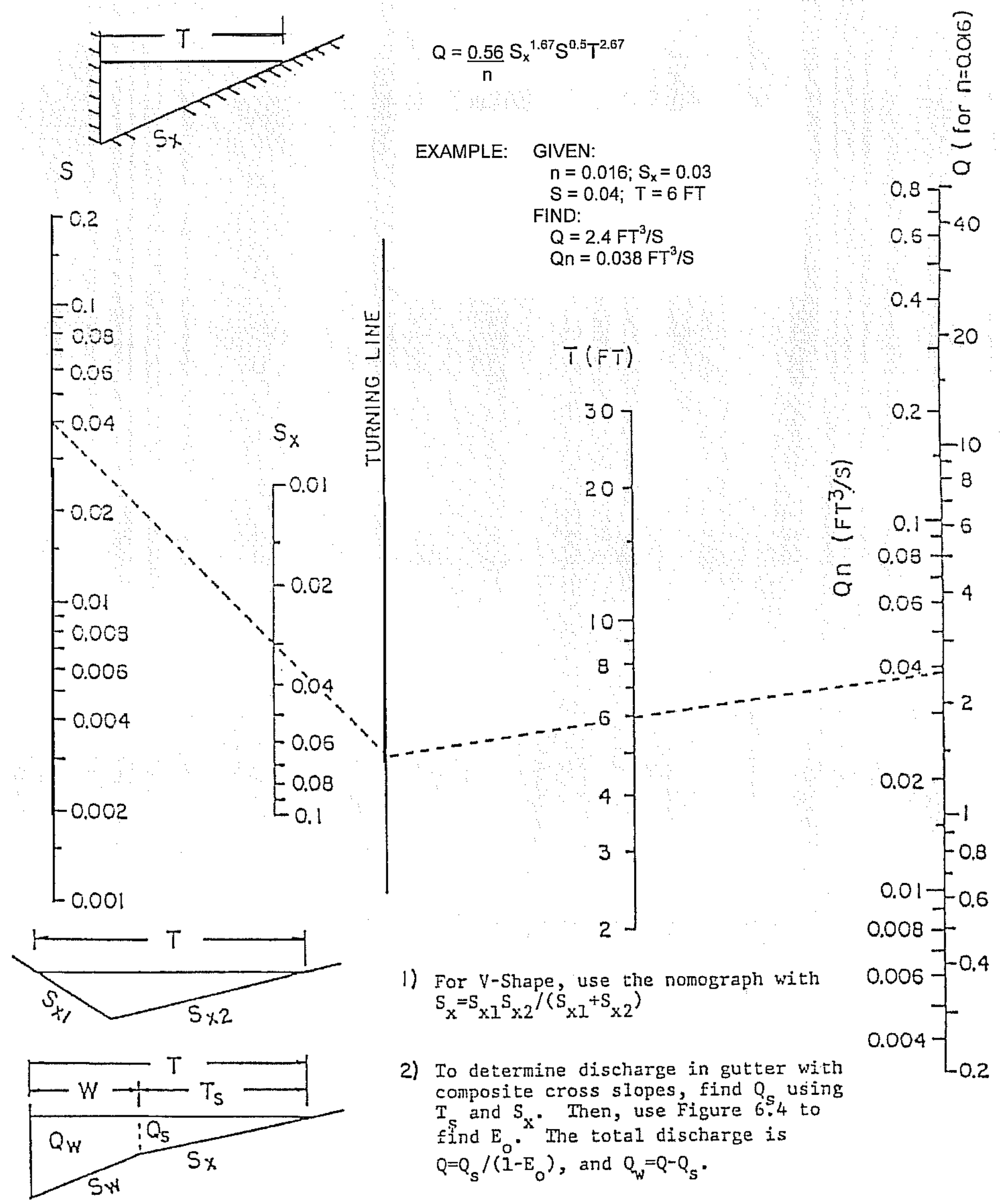
NOMOGRAPH FOR FLOW IN TRIANGULAR CHANNELS

SOURCE: AHTD

Figure 6.2

$$Q = \frac{0.56}{n} S_x^{1.67} S^{0.5} T^{2.67}$$

EXAMPLE: GIVEN:
$n = 0.016$; $S_x = 0.03$
$S = 0.04$; $T = 6$ FT
FIND:
$Q = 2.4$ FT^3/S
$Qn = 0.038$ FT^3/S



1) For V-Shape, use the nomograph with $S_x = S_{x1}S_{x2}/(S_{x1}+S_{x2})$

2) To determine discharge in gutter with composite cross slopes, find Q_s using T_s and S_x. Then, use Figure 6.4 to find E_o. The total discharge is $Q = Q_s/(1-E_o)$, and $Q_w = Q - Q_s$.

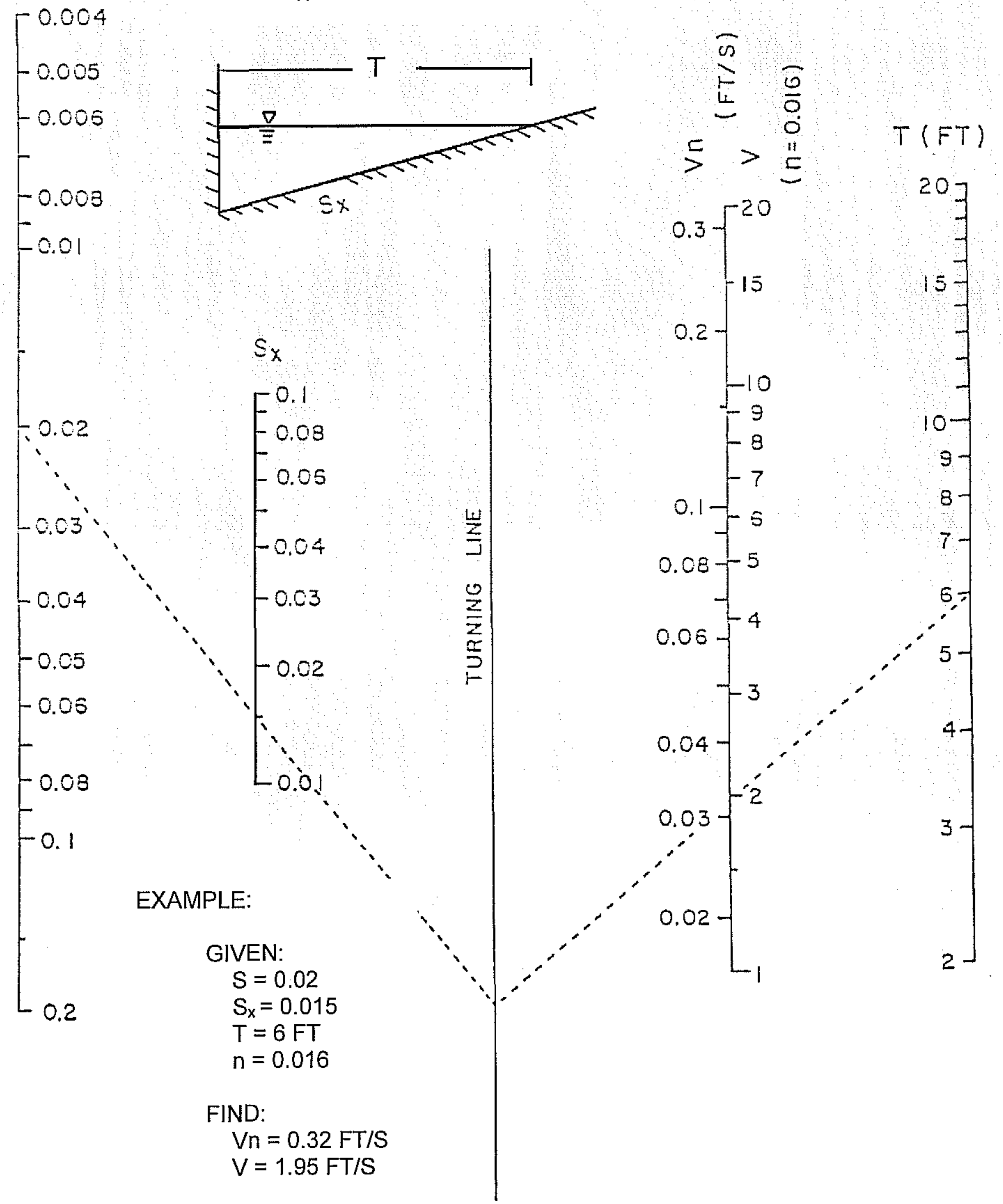
THE CITY OF BENTONVILLE

FLOW IN TRIANGULAR GUTTER SECTIONS

SOURCE: Federal Highway Administration

Figure 6.3

$$V = \frac{1.12}{n} S^{0.5} S_x^{0.67} T^{0.67}$$

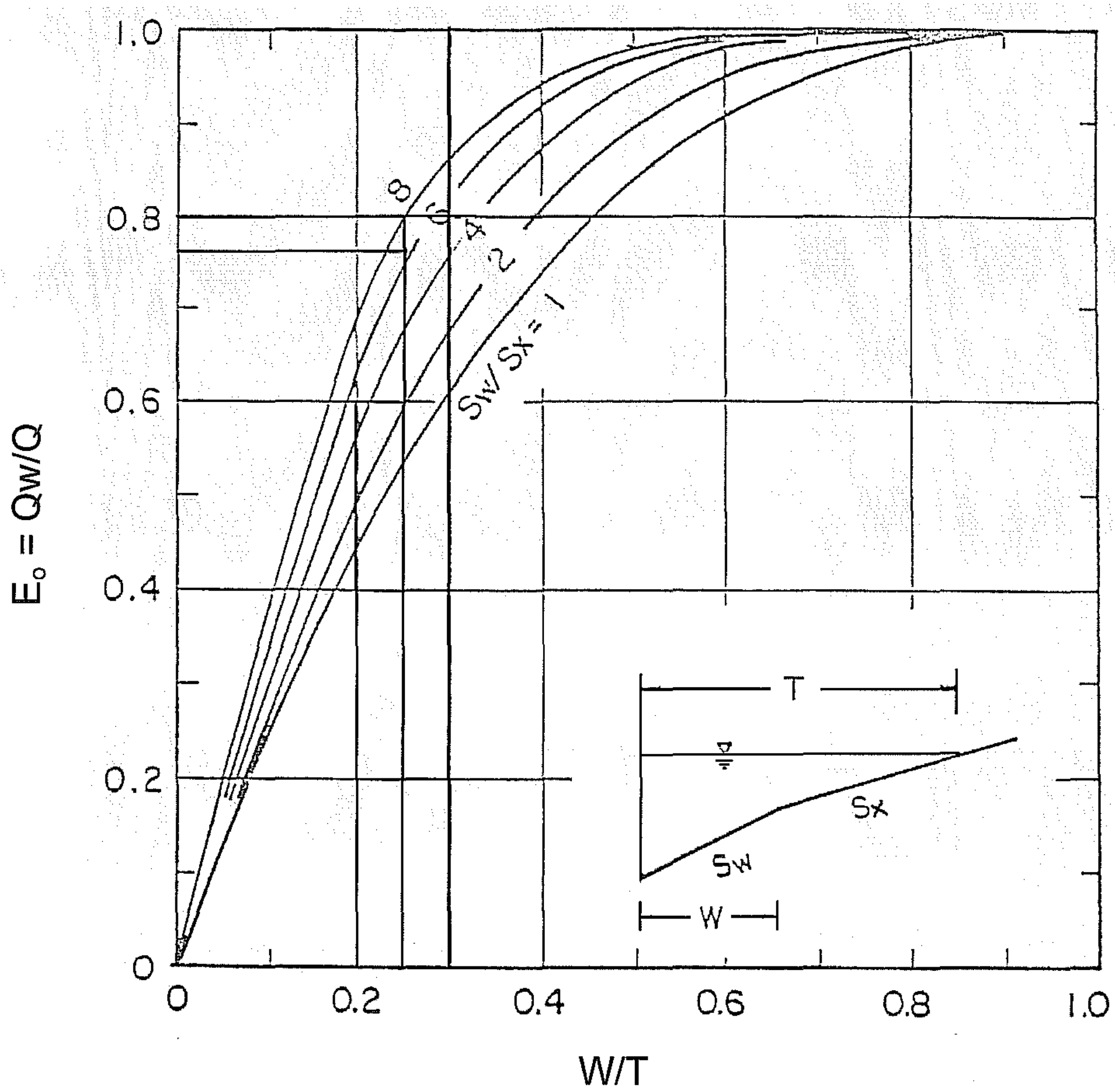


THE CITY OF BENTONVILLE

VELOCITY IN TRIANGULAR GUTTER SECTIONS

SOURCE: Federal Highway Administration

Figure 6.4



RATIO OF FRONTAL FLOW TO TOTAL GUTTER FLOW

SOURCE: Federal Highway Administration

Figure 6.5

FLOW IN TRIANGLE CURB SECTIONS AT DIFFERENT STREET SLOPES

TOTAL FLOW Qt (CFS)	n	RDWY SLOPE S (%)	CROSS SLOPE Sx (%)	PONDED WIDTH T (FT)	DEPTH AT CURB d (FT)	VEL V (FPS)
< 0.50% Street Slope>						
1.0	0.015	0.50%	3.33%	5.8	0.19	1.78
2.0	0.015	0.50%	3.33%	7.5	0.25	2.12
3.0	0.015	0.50%	3.33%	8.8	0.29	2.34
4.0	0.015	0.50%	3.33%	9.8	0.33	2.52
5.0	0.015	0.50%	3.33%	10.6	0.35	2.66
6.0	0.015	0.50%	3.33%	11.4	0.38	2.79
7.0	0.015	0.50%	3.33%	12.0	0.40	2.90
8.0	0.015	0.50%	3.33%	12.7	0.42	3.00
9.0	0.015	0.50%	3.33%	13.2	0.44	3.09
10.0	0.015	0.50%	3.33%	13.8	0.46	3.17
12.5	0.015	0.50%	3.33%	15.0	0.50	3.35
< 1.00% Street Slope>						
1.0	0.015	1.00%	3.33%	5.1	0.17	2.30
2.0	0.015	1.00%	3.33%	6.6	0.22	2.74
3.0	0.015	1.00%	3.33%	7.7	0.26	3.04
4.0	0.015	1.00%	3.33%	8.6	0.29	3.26
5.0	0.015	1.00%	3.33%	9.3	0.31	3.45
6.0	0.015	1.00%	3.33%	10.0	0.33	3.61
7.0	0.015	1.00%	3.33%	10.6	0.35	3.76
8.0	0.015	1.00%	3.33%	11.1	0.37	3.88
9.0	0.015	1.00%	3.33%	11.6	0.39	4.00
10.0	0.015	1.00%	3.33%	12.1	0.40	4.11
17.9	0.015	1.00%	3.33%	15.0	0.50	4.75
< 2.00% Street Slope>						
1.0	0.015	2.00%	3.33%	4.5	0.15	2.99
2.0	0.015	2.00%	3.33%	5.8	0.19	3.56
3.0	0.015	2.00%	3.33%	6.8	0.23	3.94
4.0	0.015	2.00%	3.33%	7.5	0.25	4.23
5.0	0.015	2.00%	3.33%	8.2	0.27	4.47
10.0	0.015	2.00%	3.33%	10.6	0.35	5.32
15.0	0.015	2.00%	3.33%	12.4	0.41	5.90
20.0	0.015	2.00%	3.33%	13.8	0.46	6.34
25.0	0.015	2.00%	3.33%	15.0	0.50	6.70

TOTAL FLOW Qt (CFS)	n	RDWY SLOPE S (%)	CROSS SLOPE Sx (%)	PONDED WIDTH T (FT)	DEPTH AT CURB d (FT)	VEL V (FPS)
< 3.00% Street Slope>						
1.0	0.015	3.00%	3.33%	4.2	0.14	3.48
2.0	0.015	3.00%	3.33%	5.4	0.18	4.14
3.0	0.015	3.00%	3.33%	6.3	0.21	4.58
4.0	0.015	3.00%	3.33%	7.0	0.23	4.92
5.0	0.015	3.00%	3.33%	7.6	0.25	5.21
10.0	0.015	3.00%	3.33%	9.8	0.33	6.20
15.0	0.015	3.00%	3.33%	11.5	0.38	6.86
20.0	0.015	3.00%	3.33%	12.8	0.42	7.38
25.0	0.015	3.00%	3.33%	13.9	0.46	7.80
30.0	0.015	3.00%	3.33%	14.9	0.49	8.17
31.0	0.015	3.00%	3.33%	15.0	0.50	8.23
< 4.00% Street Slope>						
1.0	0.015	4.00%	3.33%	3.9	0.13	3.87
2.0	0.015	4.00%	3.33%	5.1	0.17	4.61
3.0	0.015	4.00%	3.33%	5.9	0.20	5.10
4.0	0.015	4.00%	3.33%	6.6	0.22	5.48
5.0	0.015	4.00%	3.33%	7.2	0.24	5.80
10.0	0.015	4.00%	3.33%	9.3	0.31	6.90
15.0	0.015	4.00%	3.33%	10.9	0.36	7.64
20.0	0.015	4.00%	3.33%	12.1	0.40	8.21
25.0	0.015	4.00%	3.33%	13.1	0.44	8.69
30.0	0.015	4.00%	3.33%	14.1	0.47	9.09
35.5	0.015	4.00%	3.33%	15.0	0.50	9.49
< 5.00% Street Slope>						
1.0	0.015	5.00%	3.33%	3.8	0.13	4.21
2.0	0.015	5.00%	3.33%	4.9	0.16	5.01
3.0	0.015	5.00%	3.33%	5.7	0.19	5.55
4.0	0.015	5.00%	3.33%	6.3	0.21	5.96
5.0	0.015	5.00%	3.33%	6.9	0.23	6.31
10.0	0.015	5.00%	3.33%	8.9	0.30	7.50
15.0	0.015	5.00%	3.33%	10.4	0.35	8.31
20.0	0.015	5.00%	3.33%	11.6	0.39	8.93
25.0	0.015	5.00%	3.33%	12.6	0.42	9.45
30.0	0.015	5.00%	3.33%	13.5	0.45	9.89
40.0	0.015	5.00%	3.33%	15.0	0.50	10.63

TOTAL FLOW Qt (CFS)	n	RDWY SLOPE S (%)	CROSS SLOPE Sx (%)	PONDED WIDTH T (FT)	DEPTH AT CURB d (FT)	VEL V (FPS)
< 6.00% Street Slope>						
1.0	0.015	6.00%	3.33%	3.7	0.12	4.51
2.0	0.015	6.00%	3.33%	4.7	0.16	5.36
3.0	0.015	6.00%	3.33%	5.5	0.18	5.94
4.0	0.015	6.00%	3.33%	6.1	0.20	6.38
5.0	0.015	6.00%	3.33%	6.7	0.22	6.75
10.0	0.015	6.00%	3.33%	8.6	0.29	8.03
15.0	0.015	6.00%	3.33%	10.1	0.34	8.90
20.0	0.015	6.00%	3.33%	11.2	0.37	9.56
30.0	0.015	6.00%	3.33%	13.0	0.43	10.59
40.0	0.015	6.00%	3.33%	14.5	0.48	11.38
43.5	0.015	6.00%	3.33%	15.0	0.50	11.62
< 8.00% Street Slope>						
1.0	0.015	8.00%	3.33%	3.5	0.12	5.02
2.0	0.015	8.00%	3.33%	4.5	0.15	5.97
3.0	0.015	8.00%	3.33%	5.2	0.17	6.62
4.0	0.015	8.00%	3.33%	5.8	0.19	7.11
5.0	0.015	8.00%	3.33%	6.3	0.21	7.52
10.0	0.015	8.00%	3.33%	8.2	0.27	8.95
15.0	0.015	8.00%	3.33%	9.5	0.32	9.91
20.0	0.015	8.00%	3.33%	10.6	0.35	10.65
30.0	0.015	8.00%	3.33%	12.4	0.41	11.79
40.0	0.015	8.00%	3.33%	13.8	0.46	12.67
50.0	0.015	8.00%	3.33%	15.0	0.50	13.40
< 10.00% Street Slope>						
1.0	0.015	10.00%	3.33%	3.3	0.11	5.46
2.0	0.015	10.00%	3.33%	4.3	0.14	6.50
3.0	0.015	10.00%	3.33%	5.0	0.17	7.19
4.0	0.015	10.00%	3.33%	5.6	0.19	7.73
5.0	0.015	10.00%	3.33%	6.1	0.20	8.18
10.0	0.015	10.00%	3.33%	7.9	0.26	9.73
20.0	0.015	10.00%	3.33%	10.2	0.34	11.58
30.0	0.015	10.00%	3.33%	11.9	0.39	12.82
40.0	0.015	10.00%	3.33%	13.2	0.44	13.78
50.0	0.015	10.00%	3.33%	14.4	0.48	14.57
56.0	0.015	10.00%	3.33%	15.0	0.50	14.99

Figure 6.6

TABLE OF CONTENTS - SECTION VII

SECTION VII - STORM DRAIN INLETS

7.1 GENERAL

The primary purpose of storm drain inlets is to intercept excess surface runoff and deposit it in a drainage system, thereby reducing the possibility of surface flooding.

The most common location for inlets is in streets which collect and channelize surface flow making it convenient to intercept. Because the primary purpose of streets is to carry vehicular traffic, inlets must be designed so as not to conflict with that purpose.

The following guidelines shall be used in the design of inlets to be located in streets:

1. Minimum transition for depressed inlets shall be 10 feet.
2. When recessed inlets are used, they shall not interfere with the intended use of the sidewalk.
3. The capacity of a recessed inlet on grade shall be calculated the same as the capacity of a similar unrecessed inlet.
4. Design and location of inlets shall take into consideration pedestrian and bicycle traffic.
5. Inlet design and location must be compatible with the criteria established in Section III of this manual.
6. Inlet lengths shall be in 4' increments.
7. Curb inlets with vertical openings larger than 8" will require a grate to block the openings.
8. Grate inlets adjacent to a curb must be a combination inlet.

7.2 CLASSIFICATION

Inlets are classified into three major groups, mainly: Inlets in sumps (Type A), inlets on grade without gutter depression (Type B), and inlets on grade with gutter depression (Type C). Each of the three major classes include several varieties. The following are presented herein and are likely to find reasonable wide use. (See Figures 7.1 - 7.7)

Inlets in Sumps

1.	Curb opening	Type A-1
2.	Grate	Type A-2
3.	Combination (Grate & Curb Opening)	Type A-3
4.	Drop	Type A-4
5.	Drop (Grate Covering)	Type A-5

Inlets on Grade Without Gutter Depression

1.	Curb Opening	Type B-1
2.	Grate	Type B-2
3.	Combination (Grate & Curb Opening)	Type B-3

Inlets on Grade With Gutter Depression

1.	Curb Opening	Type C-1
2.	Grate	Type C-2
3.	Combination (Grate & Curb Opening)	Type C-3

Recessed inlets are identified by the suffix (R), (i.e.: A-1 (R)).

Engineering Department review of the proposed Drainage Plan shall include examination of the supporting calculations. **Computations must be submitted either as a part of the Plans or on separate tabulations sheets convenient for review and use as a permanent record in order to speed review.**

7.3 INLETS IN SUMPS

Inlets in sumps are inlets placed in low points of surface drainage areas to relieve ponding. The capacity of inlets in sumps must be known in order to determine the depth and width of ponding for a given discharge. The charts in this section may be used in the design of any inlet in a sump, regardless of its depth of depression.

7.3.1 CURB OPENING INLETS AND DROP INLETS (TYPE A-1 and A-4)

Unsubmerged curb opening inlets (Type A-1) and drop inlets (Type A-4) in a sump at low points are considered to function as rectangular weirs with

a coefficient of discharge of 3.0. Their capacity shall be based on the following equation:

$$Q = 3.0\ Y^{3/2}\ L$$

Q = capacity in CFS of curb opening inlet or capacity in CFS of drop inlet

Y = head at the inlet in feet when Y is less than the height of the opening

L = length of opening through which water enters the inlet in feet

Figure 7.8 provides for direct solution of the above equation.

7.3.2 GRATE INLETS (TYPE A-2 and A-5)

A grate inlet, Type A-2 or A-5 in a sump can be considered an orifice with the coefficient of discharge of 0.67. The capacity shall be based on the following:

$$Q = 5.37\ A_g\ Y^{1/2}$$

Q = Capacity in CFS

A_g = area of clear opening in square feet

Y = depth at inlet or head at sump in feet when Y is less than height of opening

The curve shown in Figure 7.9 provides for direct solution of the above equation.

Grate inlets in sumps have a tendency to clog when flows carry debris such as leaves and papers. Clogging shall be taken into consideration when calculating grate inlet capacity.

7.3.3 COMBINATION INLETS (TYPE A-3)

The capacity of a combined inlet type A-3 consisting of a grate and curb opening inlet in a sump shall be considered to be the sum of the capacities obtained from Figures 7.8 and 7.9. When the capacity of the gutter is not exceeded, the grate inlet accepts the major portion of the flow. Under severe flooding conditions, however, the curb inlet will accept most of the flow since its capacity varies with $y^{1.5}$ whereas the capacity of the grate varies as $y^{0.5}$.

7.4 INLETS ON GRADE WITHOUT GUTTER DEPRESSION

7.4.1 CURB OPENING INLETS (UNDEPRESSED: TYPE B-1)

The capacity of the curb inlet, like any weir depends upon the head and length of the overfall. In the case of an undepressed curb opening inlet, the head at the upstream end of the opening is the depth of flow in the gutter. In streets where grades are greater than one percent (1%), the velocities are high and the depths of flow are usually small, as there is little time to develop cross flow into the curb openings. Therefore, undepressed inlets are inefficient when used in streets of appreciable slope, but may be used satisfactorily where the grade is low and the crown slope high or the gutter channelized. Undepressed inlets do not interfere with traffic and usually are not susceptible to clogging. Inlets on grade should be designed and spaced so that 20 to 40 percent of gutter flow reaching each inlet will carry over to the next inlet downstream, provided the water carry-over does not inconvenience pedestrian or vehicular traffic.

The capacity of an undepressed inlet shall be determined by use of Figures 7.10 and 7.11. An example of the use of Figures 7.10 and 7.11 is included at the end of this section.

7.4.2 GRATE INLETS ON GRADE (UNDEPRESSED: TYPE B-2)

Undepressed grate inlets on grade have a greater hydraulic capacity than curb inlets of the same length so long as they remain unclogged. Undepressed inlets on grade are inefficient in comparison to grate inlets in sumps. For flow capacity through grade inlets, the Engineer should refer to Federal Highway publication H.E.C. 12 or refer to appropriate vendor catalog. Grate inlets should be designed and spaced so that 20 to 40 percent of the gutter flow reaching each inlet will carry over to the next downstream inlet, provided the carry-over does not inconvenience pedestrian or vehicular traffic.

Grates shall be certified by the manufacturer as bicycle-safe. For flows on streets with grades less than one percent (1%), little or no splashing occurs regardless of the direction of the bars.

Vane grate inlets are the recommended grates for best hydraulic capacity and should be the first grate type considered on any project. Clogging shall be taken into consideration when calculating grate inlet capacity.

7.4.3 COMBINATION INLETS ON GRADE (UNDEPRESSED: TYPE B-3)

The interception capacity of a combination inlet consisting of a curb opening and grate placed side-by-side, is not appreciably greater than that of the grate alone. Capacity is computed by neglecting the curb opening. A combination inlet is sometimes used with the curb opening or a part of the curb opening placed upstream of the grate. The curb opening in such an installation intercepts debris which might otherwise clog the grate and has been termed a "sweeper" by some. A combination inlet with a curb opening upstream of the grate has an interception capacity equal to the sum of the two inlets, except that the frontal flow and thus the interception capacity of the grate is reduced by interception by the curb opening.

The capacity of a Type B-3 inlet without extensions shall be considered the same as the capacity of a Type B-2 inlet. (considering reduction due to clogging).

7.5 INLETS ON GRADE WITH GUTTER DEPRESSION

7.5.1 CURB OPENING INLETS ON GRADE (DEPRESSED: TYPE C-1)

The depression of the gutter at a curb opening inlet below the normal level of the gutter increases the cross-flow toward the opening, thereby increasing the inlet capacity. Also, the downstream transition out of the depression causes backwater which further increases the amount of water captured. Depressed inlets should be used on continuous grades that exceed one percent (1%) except that their use in traffic lanes shall conform with the requirements of Section VI of this manual.

The depression depth, width, length, and shape all have significant effects on the capacity of an inlet. Reference to Section VI of this manual must be made for permissible gutter depressions.

The capacity of a depressed curb inlet will be determined by the use of Figures 7.10 and 7.11.

7.5.2 GRATE INLETS ON GRADE (DEPRESSED: TYPE C-2)

The depression of the gutter at a grate inlet decreases the flow past the outside of a grate. The effect is the same as that when a curb inlet is depressed, mainly the cross slope of the street directs the outer portion of flow towards the grate.

The bar arrangements for depressed grate inlets on streets with grades greater than one percent (1%) greatly affect the efficiency of the inlet. Grates with longitudinal bars eliminate splash which causes the water to jump and ride over the cross bar grates, and it is recommended that grates have a minimum of transverse cross bars for strength and spacing only.

For low flows or for streets with grades less than one percent (1%), little or no splashing occurs regardless of the direction of the bars. However, as the flow or street grade increases, the grate with longitudinal bars becomes progressively superior to the cross bar grate. A few small rounded cross bars, installed at the bottom of the longitudinal bars as stiffeners or a safety stop for bicycle wheels do not materially affect the hydraulic capacity of the longitudinal bar grates. A bicycle safe grate must be used.

The capacity of a Type C-2 inlet on grades less than one percent (1%) shall be the capacity determined from Figure 7.9. The capacity of C-2 inlets on grades greater than one percent (1%) shall be **90 percent** of the capacity as determined from Figure 7.9.

Grate inlets and depressions have a tendency to clog when gutter flow carries debris such as leaves and papers. Clogging shall be taken into consideration when calculating grate inlet capacity.

7.5.3 COMBINATION INLETS ON GRADE (DEPRESSED: TYPE C-3)

The interception capacity of a combination inlet consisting of a curb opening and grate placed side-by-side, is not appreciably greater than that of the grate alone. Capacity is computed by neglecting the curb opening. A combination inlet is sometimes used with the curb opening or a part of the curb opening placed upstream of the grate. The curb opening in such an installation intercepts debris which might otherwise clog the grate and has been termed a "sweeper" by some. A combination inlet with a curb opening upstream of the grate has an interception capacity equal to the sum of the two inlets, except that the frontal flow and thus the interception capacity of the grate is reduced by interception by the curb opening.

The capacity of a Type C-3 inlet without extensions shall be considered the same as the capacity of a Type C-2 inlet. (considering reduction due to clogging).

7.6 USE OF FIGURES 7.10 AND 7.11

Example 1

Given: $S_x = 0.03$
$S = 0.035$

$Q = 5$ ft.^{3}/S
$n = 0.016$

Find: (1) Q_i for a 10-ft. curb-opening inlet
(2) Q_i for a depressed 10-ft. curb opening inlet
a = 2 in.
W = 2 ft.

Solution:

(1) T = 8 ft. (Figure 6.3)
L_T = 41 ft. (Figure 7.10
$L/{}^{L}T = 10/41 = 0.24$
E = 0.39 (Figure 7.11)
$Q_i = EQ = 0.39 \times 5 = 2.0$ ft^3/S

(2) T = 7.0 ft. (Figure 6.3)
$W/T = 2/7 = 0.29$
E_o = 0.72 (Figure 6.5)
$S_e = S_x + S_wE_o = 0.03 + 0.083(0.72)$
$= 0.09$

L_T = 23 ft. (Figure 7.10)
$L/LT = 10/23 = 0.43$
E = 0.64 (Figure 7.11)
$Q_i = 0.64 \times 5 = 3.2$ ft.^{3}/S

7.7 INLET FLOW CALCULATION TABLE

Runoff and inlet computations for Inlet Flow Calculation Table (Figure 7.12) Computer generated computations and output are accepted and subject to review by City Engineer.

Column 1:	Inlet number. All inlets are classified with a designated number.
Column 2:	Inlet location. Location or station of inlet.
Column 3:	A - Drainage area in acres contributing runoff to the inlet.
Column 4:	C - Average or composite runoff coefficient of the area, A, contributing runoff to the inlet.
Column 5:	Tc - Time of concentration for the drainage area in minutes. See section II.
Column 6:	i - Rainfall intensity in inches per hour for the design storm. Based on the time of concentration. See Figure 2.5.
Column 7:	CA for the drainage area. Equal to Column 3 multiplied by Column 4.
Column 8:	Carry over, CA, from preceding inlet (Column 27).
Column 9:	Q_t - Total flow at the inlet. Equal to the sum of the values in Column 7 and Column 8 multiplied by the value in Column 6 or $Q_t = i * SCA$
Column 10:	n - Manning's roughness coefficient for the gutter section.
Column 11:	S - The slope of the gutter profile in feet per foot.
Column 12:	S_x - Cross slope of the roadway section at the inlet in feet per foot. Not applicable for parabolic street sections.
Column 13:	T - Ponded width of flow in the street/gutter in feet. Obtained from Figure 6.3.
Column 14:	d - Depth of flow in the gutter section of the inlet in feet. Obtained from Figure 6.2 or $d = T * S_x$

Column 15: V - Velocity of flow in gutter in feet per second. Equal to Column 8 divided by one half of Column 12 multiplied by Column 13 or $V = Q/A$

Column 16: L - Length of the inlet in feet.

Column 17: a - Depth of the gutter depression at the inlet in inches.

Column 18: W - Width of the gutter depression at inlet in feet.

Column 19: E_o - Ratio of frontal flow to total flow. Obtained from Figure 6.5 or $E_o = Q_w/Q - 1 - (1 - W/T)^{2.67}$

Column 20: S_e - Equivalent cross slope of the pavement at the inlet in feet per foot:

$$S_e = S_x + (a/12w) * \frac{E_o}{E}$$

Column 21: Lt - Required length of inlet in feet for total flow interception. Obtained from Figure 7.10.

Column 22: E - Efficiency of the inlet of length L. Obtained from Figure 7.11.

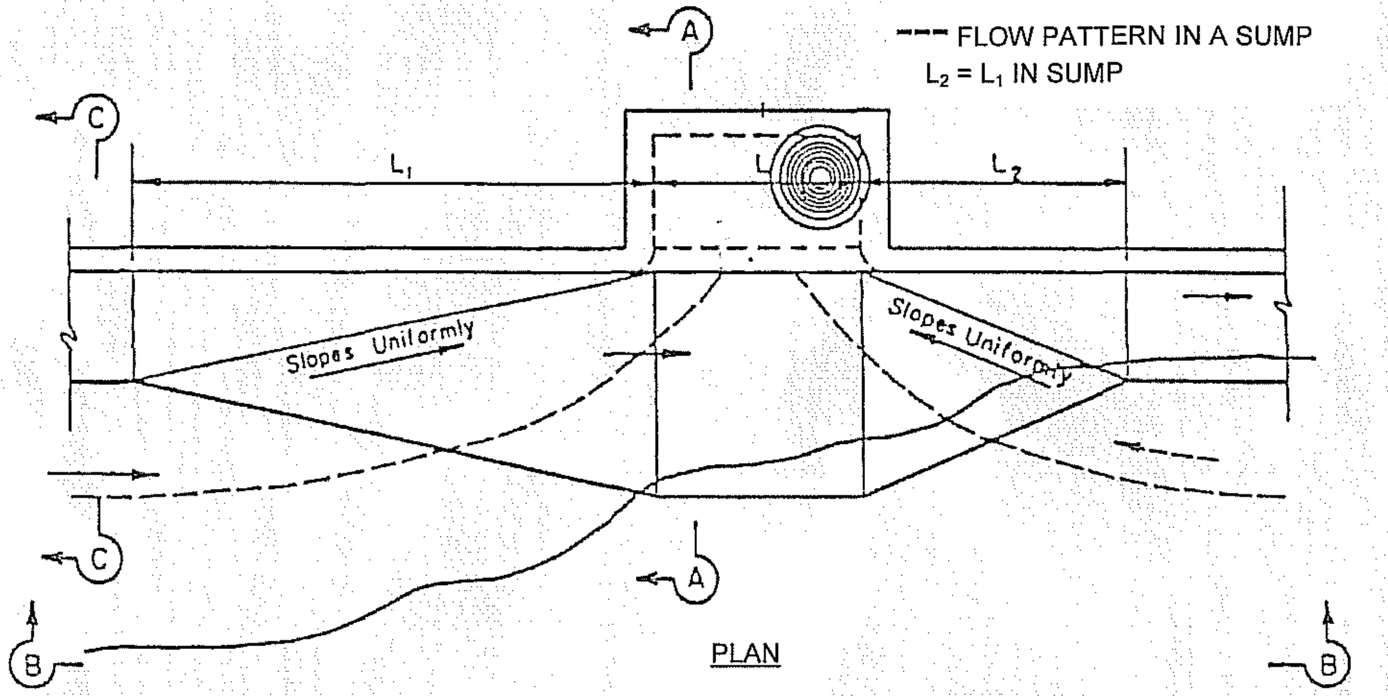
Column 23: Q_i - Flow intercepted by the inlet of length L in CFS. Equal to Column 22 multiplied by Column 9 or $Q_i = Q_t * E$

Column 24: RF - Clogging reduction factor for the inlet.

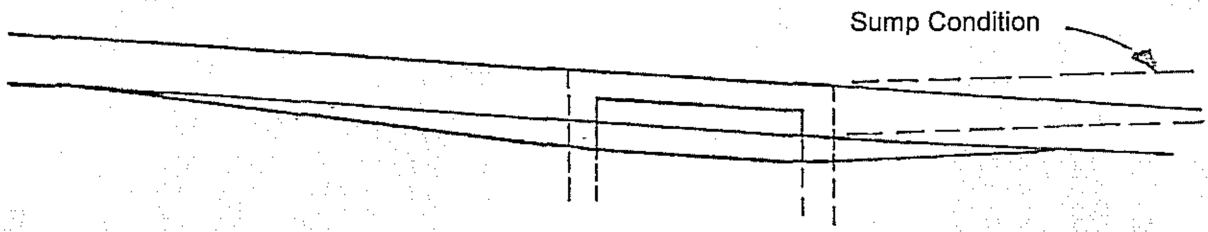
Column 25: Q_a - Actual flow intercepted by the inlet in CFS. Equal to Column 23 multiplied by Column 24 or $Q_a = Q_i * RF$

Column 26: Q_p - Bypass flow in CFS. Equal to Column 25 subtracted from Column 9 or $Q_p = Q_t - Q_a$

Column 27: Carry over, CA, for the next downstream inlet. Equal to Column 26 divided by Column 6 or Carry over = Q_p/i

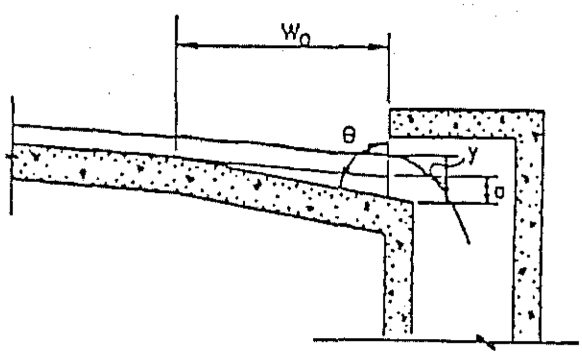


PLAN

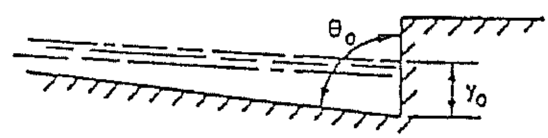


SECTION B - B

$$E = y_o + \frac{V_o^2}{2g} + a$$



SECTION A - A



SECTION C - C

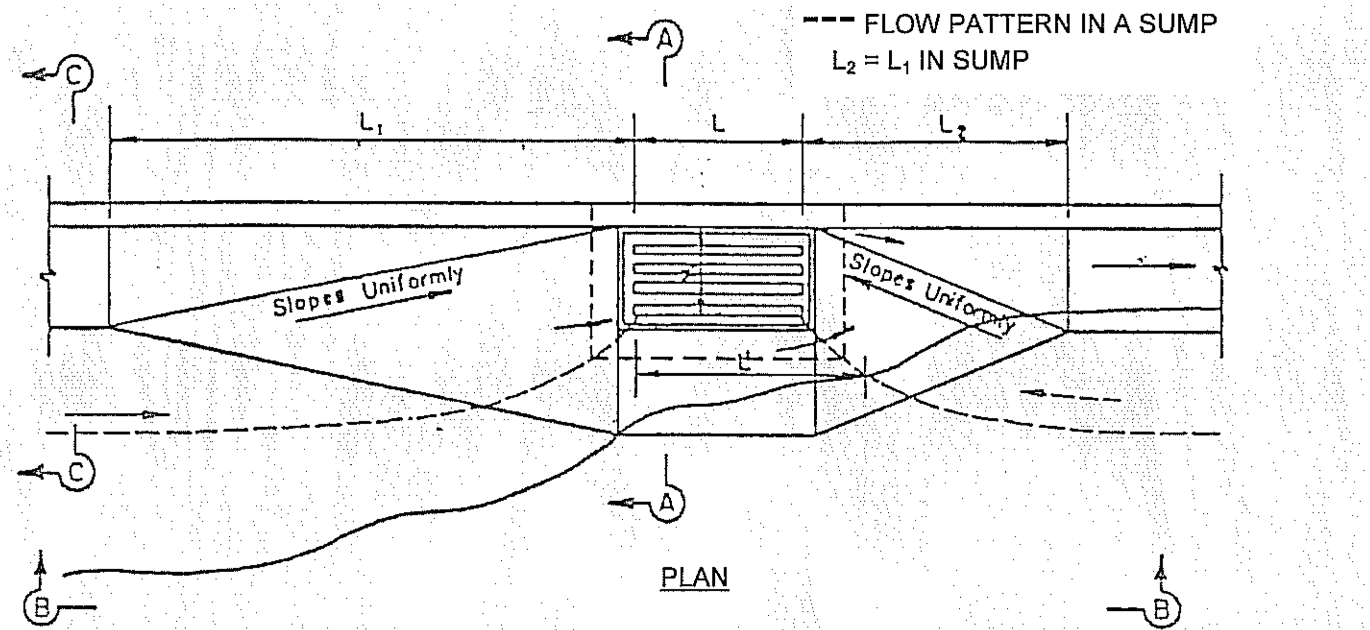
* Detail for Information and Calculation Purposes. Not for Construction

THE CITY OF BENTONVILLE

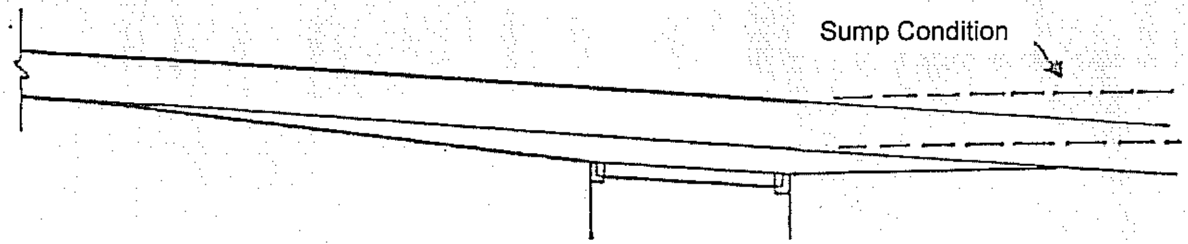
DEPRESSED CURB-OPENING INLET
TYPE A-1 & C-1

SOURCE: City of Austin, TX

Figure 7.1

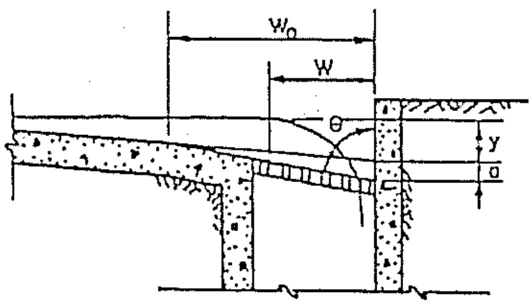


PLAN

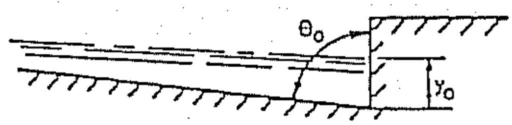


SECTION B - B

$$E = y_o + \frac{V_o^2}{2g} + a$$



SECTION A - A



SECTION C - C

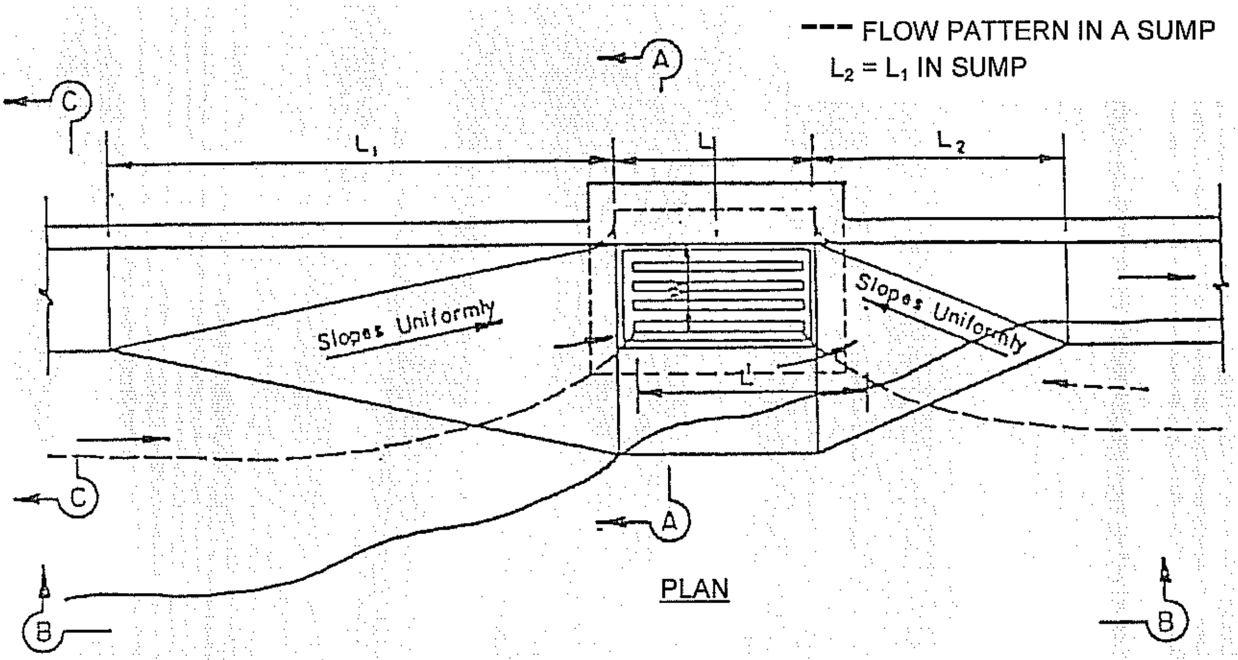
* Detail for Information And Calculation Purposes. Not For Construction

THE CITY OF B BENTONVILLE

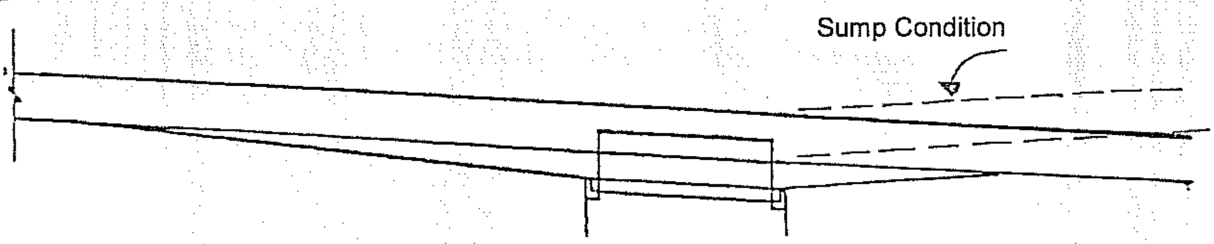
DEPRESSED GRATED INLET
TYPE A-2 & C-2

SOURCE: City of Austin, TX

Figure 7.2

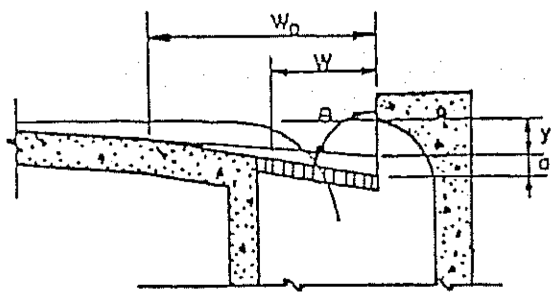


PLAN

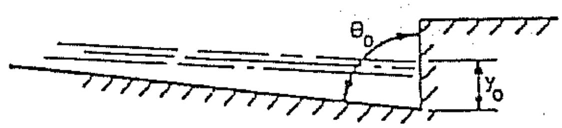


SECTION B - B

$$E = y_o + \frac{V_o^2}{2g} + a$$



SECTION A - A



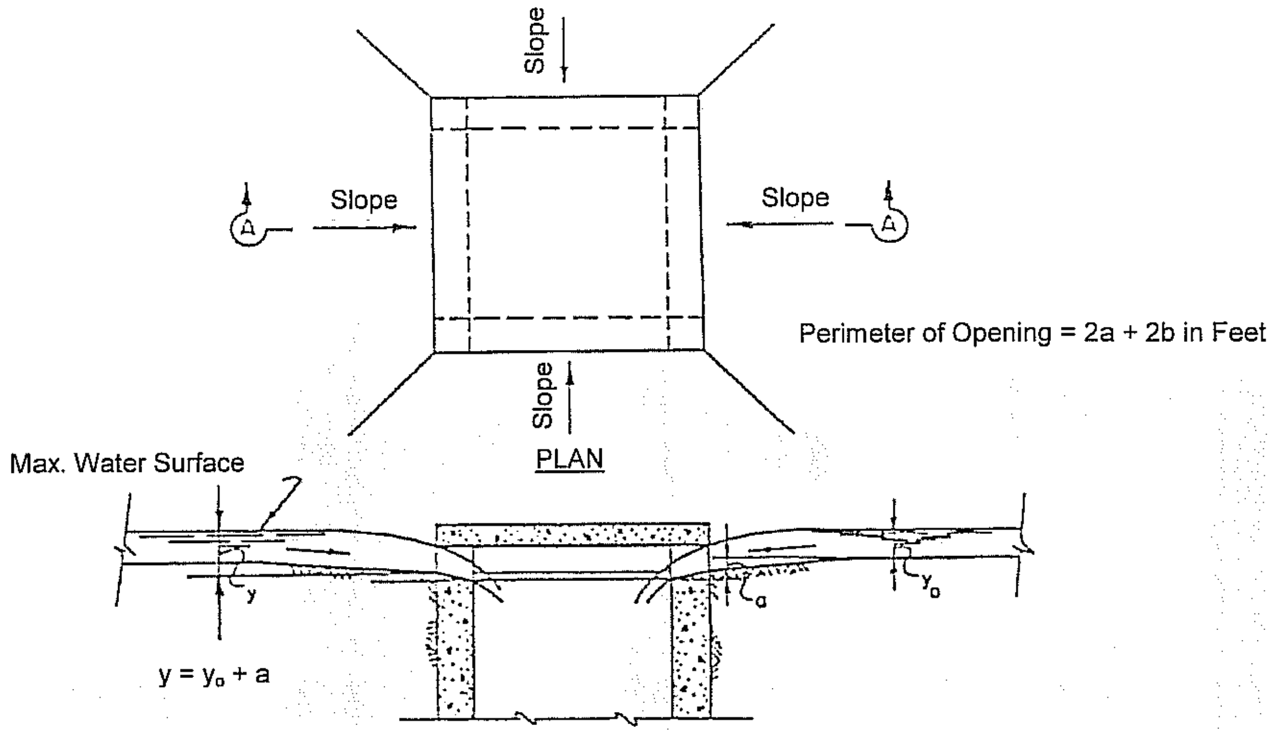
SECTION C - C

* Detail for Information and Calculation Purposes. Not for Construction

DEPRESSED COMBINATION INLET
TYPE A-3 & C-3

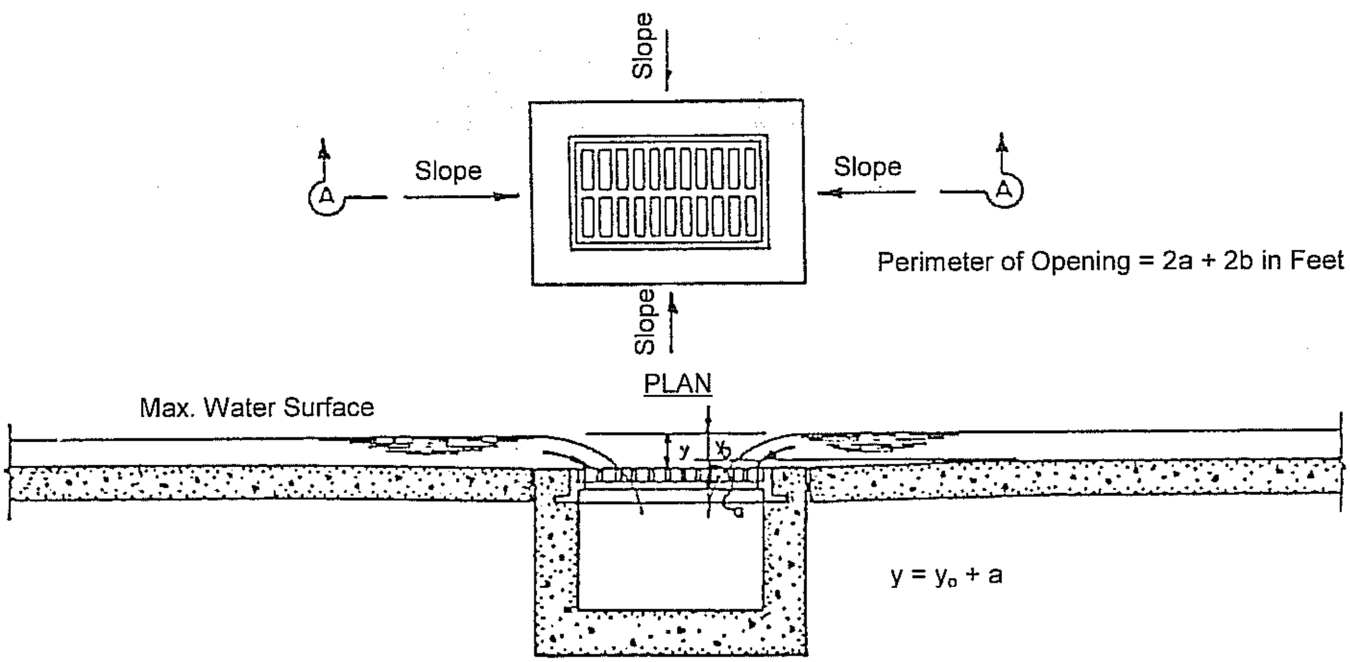
SOURCE: City of Austin, TX

Figure 7.3



SECTION A – A

DROP INLET
TYPE A-4



SECTION A – A
DROP INLET
(Grate Covering)

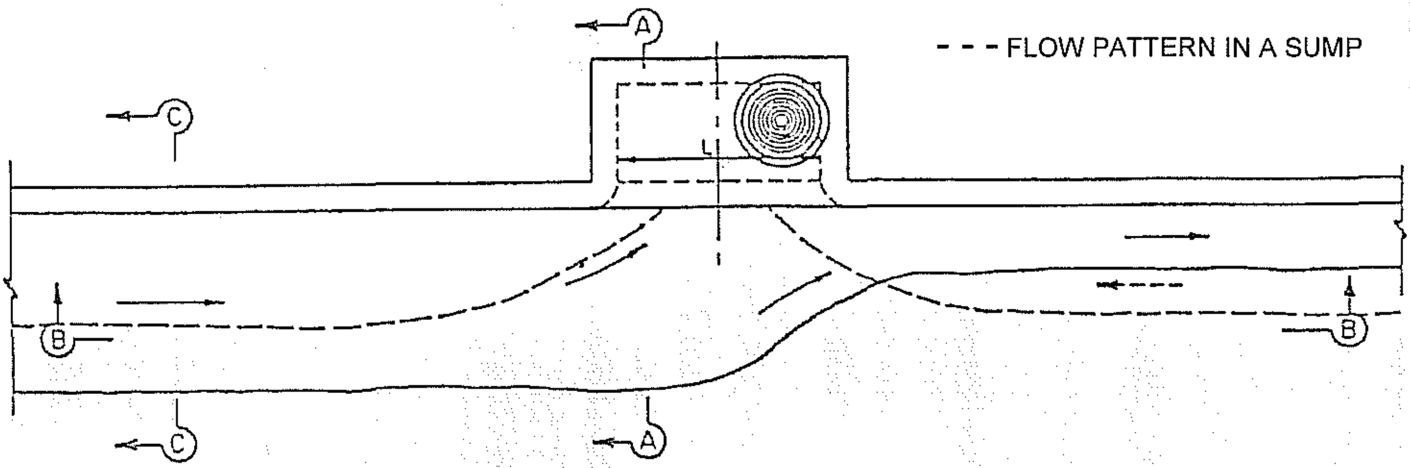
* Detail for Information and Calculation Purposes. Not for Construction

THE CITY OF BENTONVILLE

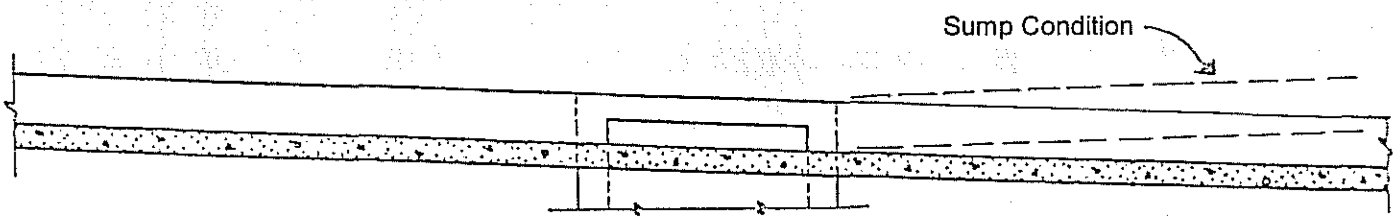
DROP INLET (Grate Covering)
TYPE A-4 & TYPE A-5

SOURCE: City of Austin, TX

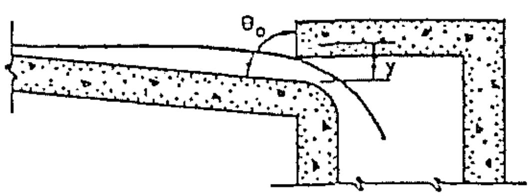
Figure 7.4



PLAN

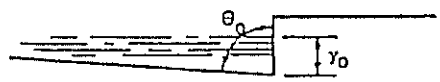


SECTION B - B



SECTION A – A

$y = y_o$



SECTION C – C

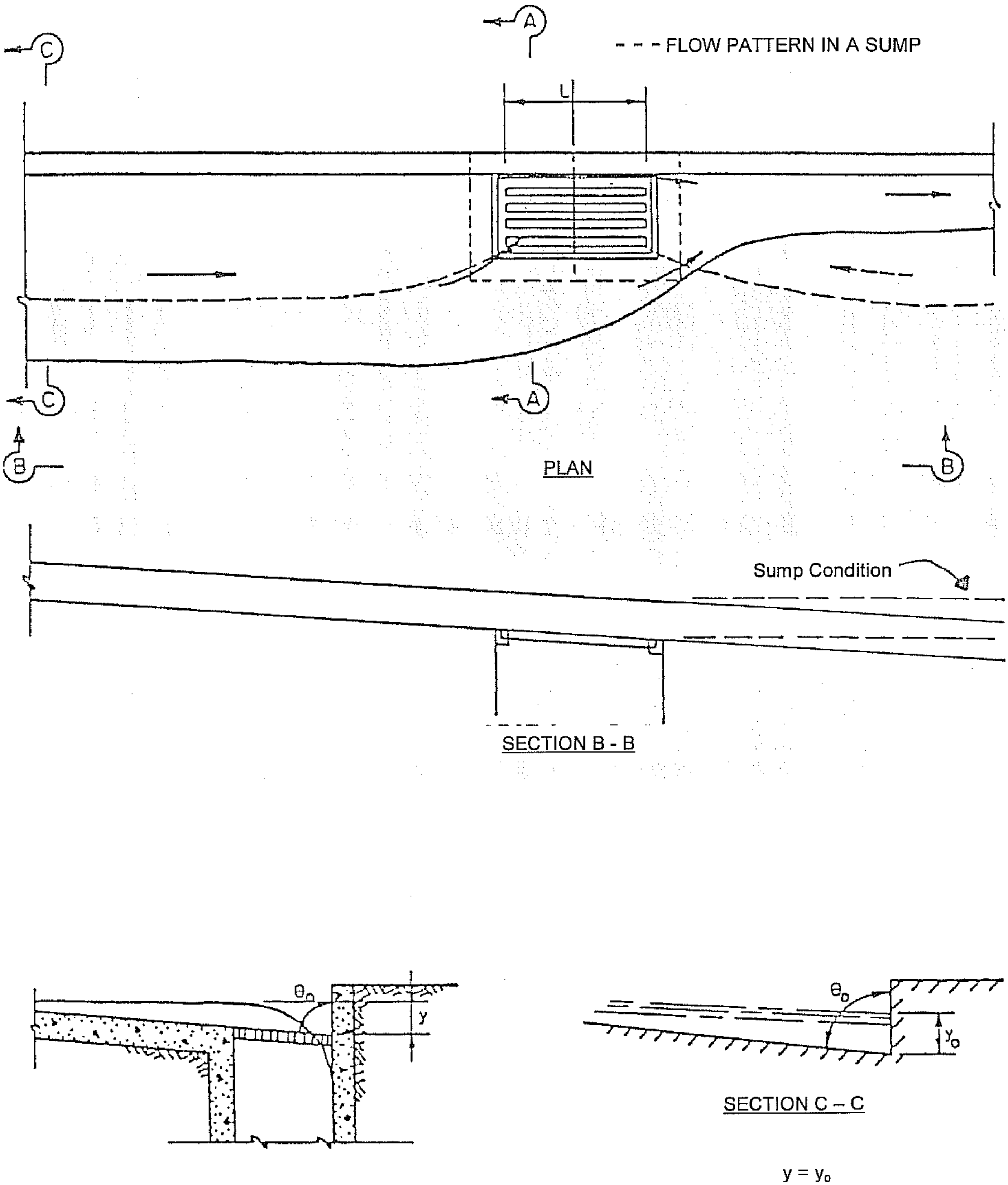
* Detail for Information and Calculation Purposes. Not for Construction

THE CITY OF BENTONVILLE

UNDEPRESSED CURB-OPENING
INLET TYPE A-1 & B-1 (RECESSED)

SOURCE: City of Austin, TX

Figure 7.5



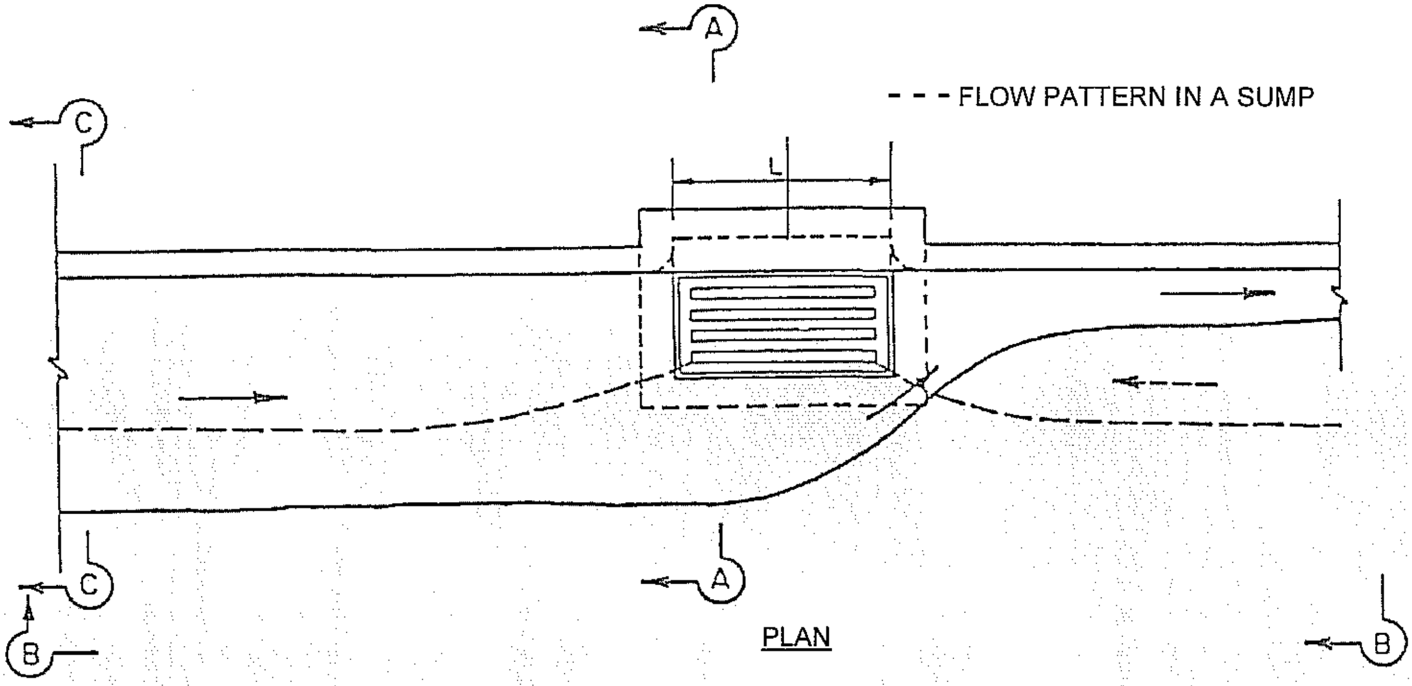
SECTION A – A

* Detail for Information and Calculation Purposes. Not for Construction

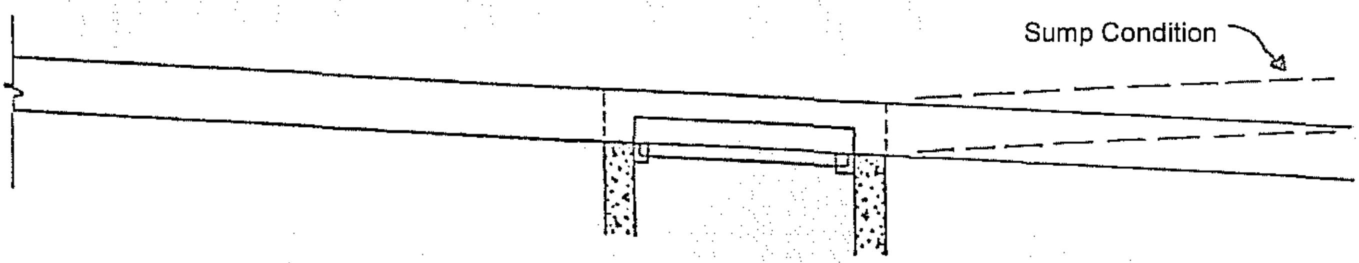
UNDEPRESSED GRATE INLET
TYPE A-2 & B-2

SOURCE: City of Austin, TX

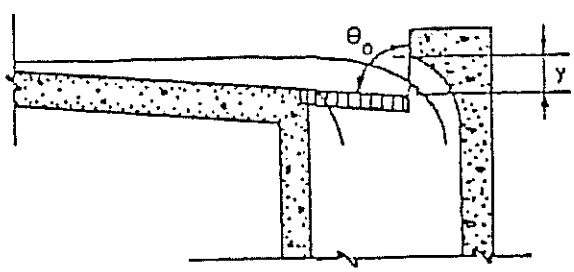
Figure 7.6



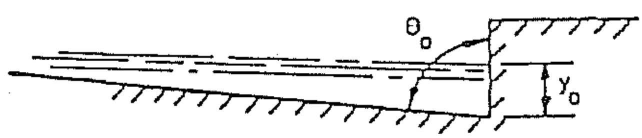
PLAN



SECTION B - B



SECTION A – A



SECTION C – C

$y = y_o$

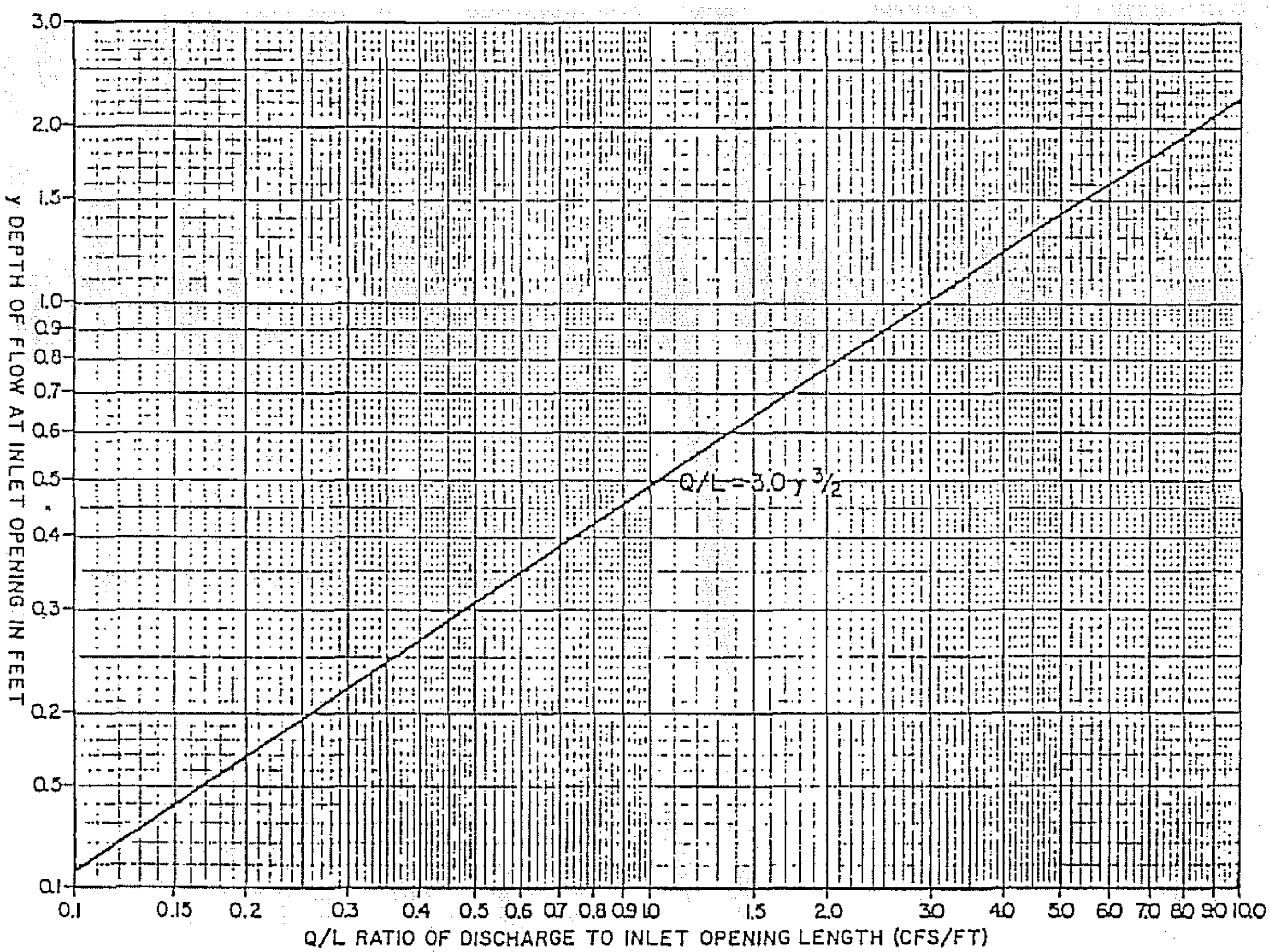
* Detail for Information and Calculation Purposes. Not for Construction

THE CITY OF BENTONVILLE

UNDEPRESSED COMBINATION INLET
TYPE A-3 & B-3 (RECESSED)

SOURCE: City of Austin, TX

Figure 7.7

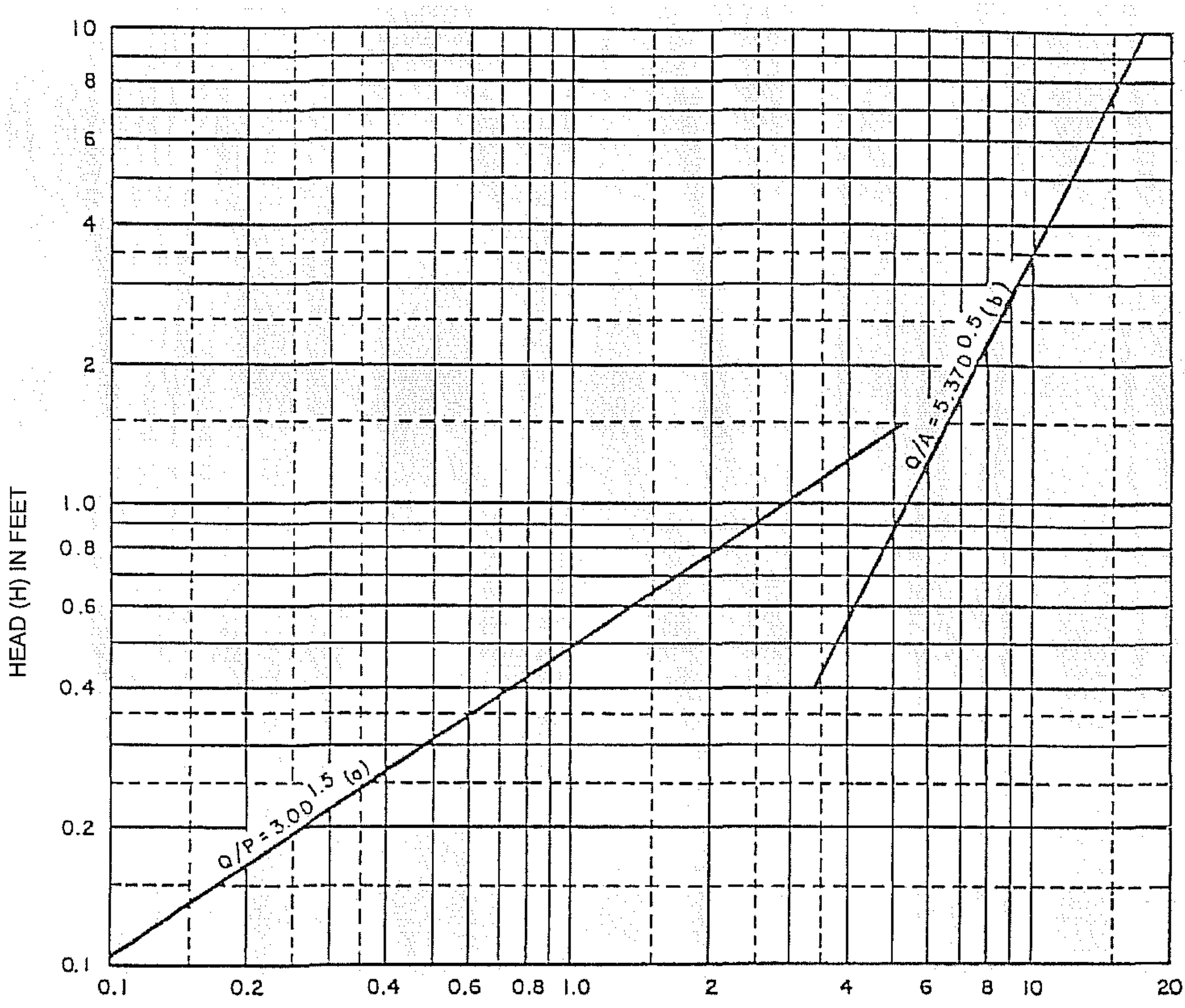


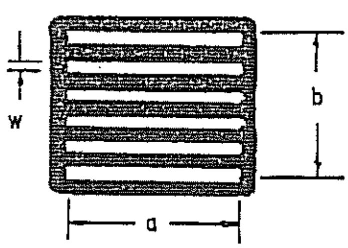
THE CITY OF BENTONVILLE

INLET CAPACITY
TYPE A-1 & A-4

SOURCE: City of Little Rock, AR

Figure 7.8





$P = 2\ (a + b)$
$A = 6\ aw$

HEADS UP TO 0.4, CURVE (a) APPLIES
HEADS ABOVE 1.4, CURVE (b) APPLIES
HEADS BETWEEN 0.4 & 1.4, TRANSITION
SECTOR, USE LESSOR VALUE OF DISCHARGE

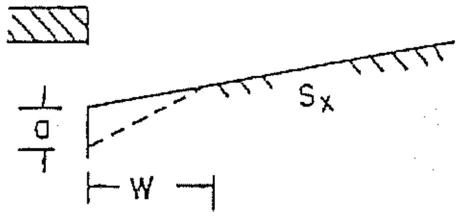
CAPACITY OF GRATE INLET IN SAG

THE CITY OF BENTONVILLE

INLET CAPACITY
TYPE A-2 & A-5

SOURCE: AHTD

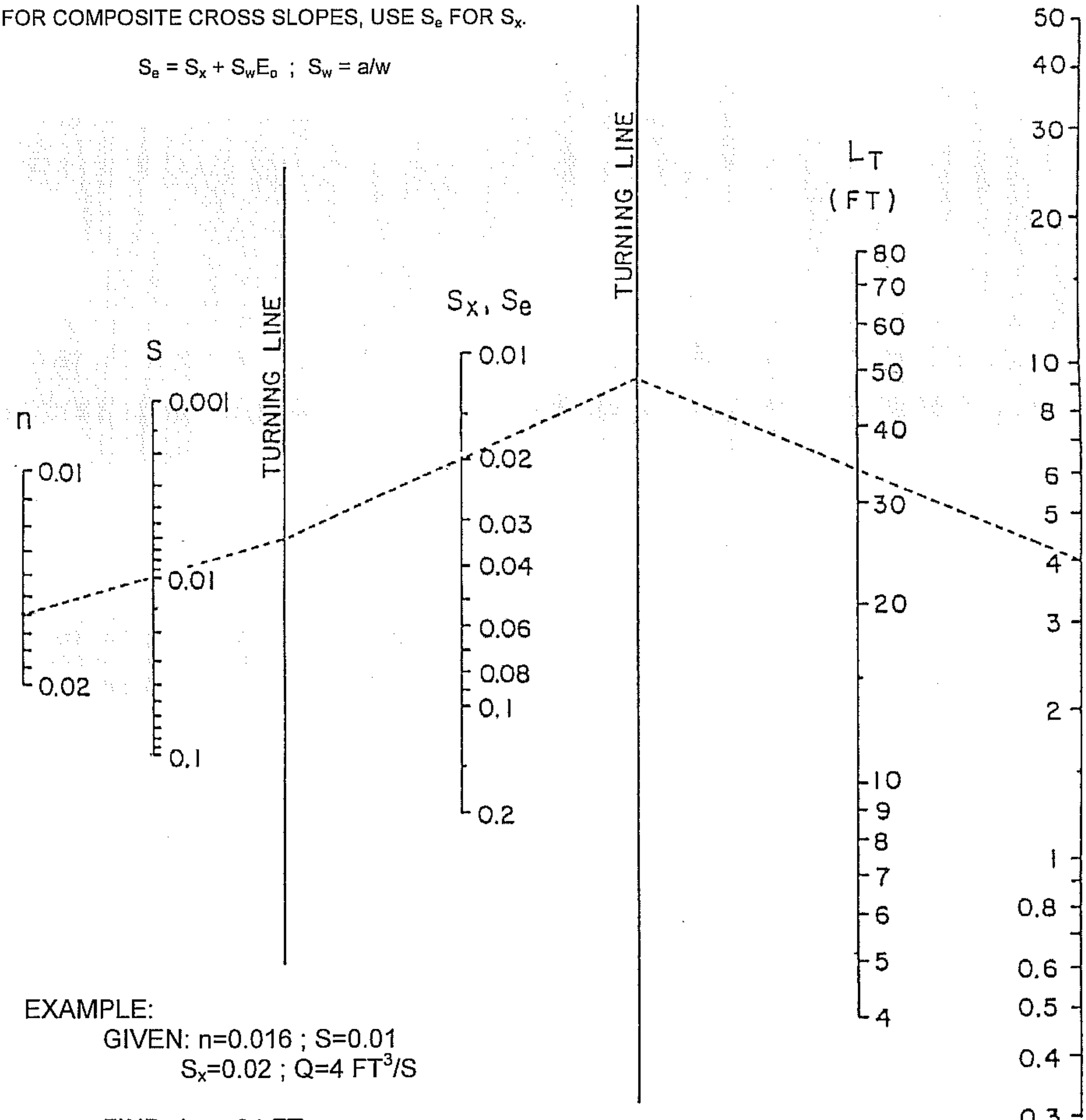
Figure 7.9



$$L_T = 0.6Q^{0.42}S^{0.3}(1/nS_x)^{0.6}$$

FOR COMPOSITE CROSS SLOPES, USE S_e FOR S_x.

$S_e = S_x + S_wE_o$; $S_w = a/w$



EXAMPLE:

GIVEN: n=0.016 ; S=0.01

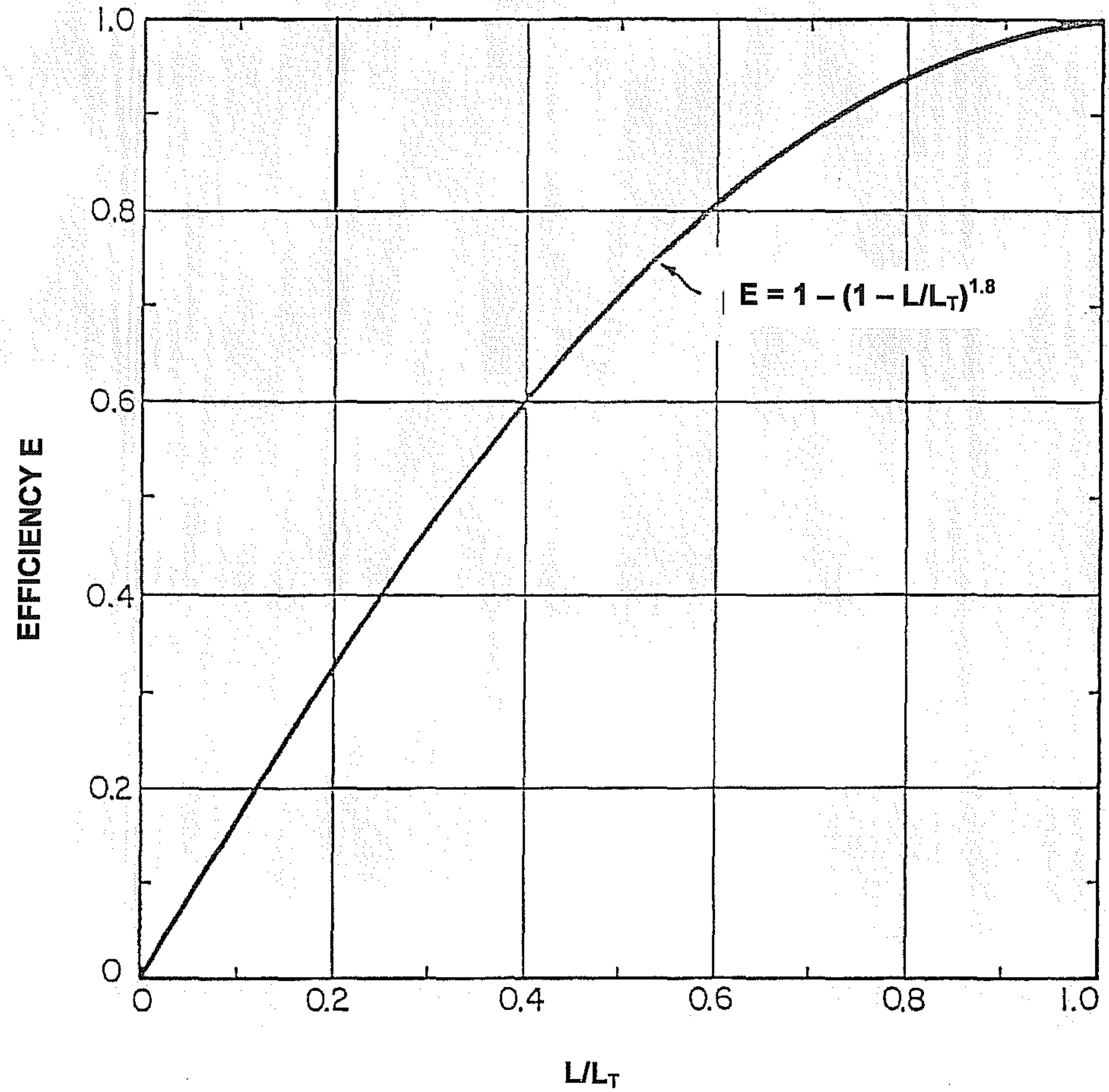
S_x=0.02 ; Q=4 FT^3/S

FIND: L_T = 34 FT

CURB-OPENING AND SLOTTED DRAIN INLET LENGTH FOR TOTAL INTERCEPTION

SOURCE: Federal Highway Administration – Circular 12

Figure 7.10



CURB-OPENING AND SLOTTED DRAIN INLET INTERCEPTION EFFICIENCY

SOURCE: Federal Highway Administration – Circular 12

Figure 7.11

INLET FLOW CALCULATION TABLE

Figure 7.12

CITY OF BENTONVILLE, ARKANSAS
INLET FLOW CALCULATION TABLE

PROJECT ______________________________

PLAN SHT. NO. ____________ DATE ______________

COMPUTED BY ____________ CKD. ______________

1 No	2 Location	3 Area (AC)	4 C	5 Tc (Min)	6 I (IN/HR)	7 Runoff At Inlet (CA)	8 Carry Over (CA)	9 Total Flow Qt (CFS)	10 n	11 RDWY Slope S	12 Cross Slope Sx	13 Ponded Width T (FT)	14 Depth At Curb d (FT)	15 VEL V (FPS)	16 L (FT)	17 a (IN)	18 W (FT)	19 Eo	20 Se	21 Lt (FT)	22 E	23 Q1 (CFS)	24 RF	25 Qa (CFS)	26 Bypass Qp (CFS)	27 Carry Over (CA)

TABLE OF CONTENTS - SECTION VIII

SECTION VIII - STORM SEWER DESIGN

SECTION VIII - STORM SEWER DESIGN

8.1 GENERAL

All storm drains shall be designed by the application of the Manning equation either directly or through appropriate charts or nomographs. In the preparation of hydraulic designs, a thorough investigation shall be made of all existing waterways and drainage structures along with their performance.

The design of the storm drainage systems should be governed by the following six conditions:

1. The system must accommodate all surface runoff resulting from selected design storm without serious damage to physical facilities or substantial interruptions of normal traffic.

2. Runoff resulting from storms exceeding the design storm must be anticipated and disposed of with minimum damage to physical facilities and minimum interruption of normal traffic.

3. The storm drainage system must have a maximum reliability of operation.

4. The construction cost of the system must be reasonable with relationship to the importance of the facilities it protects.

5. The storm drainage system must require minimum maintenance and must be accessible for maintenance operations.

6. The storm drainage system must be adaptable to future expansion with minimal additional costs.

An example of the design of the storm drainage system is outlined in Paragraphs 8.3 and 8.4. The design theory has been presented in the preceding sections with corresponding tables and graphs of information.

8.2 PRELIMINARY DESIGN CONSIDERATIONS

A. Prepare a Drainage Map of the entire area to be drained by proposed improvements. Contour maps serve as excellent area drainage maps when supplemented by field reconnaissance.

B. Make a tentative layout of the proposed storm system, locating all inlets, manholes, mains, laterals, ditches, culverts, etc.

C. Outline the drainage area for each inlet in accordance with present and future street development.

D. Indicate on each drainage area the size of the area, the direction of surface runoff by small arrows and coefficient of runoff for the area.

E. Show all existing underground utilities.

F. Establish design rainfall frequency.

G. Establish minimum inlet time of concentration.

H. Establish the typical cross section on each street.

I. Establish permissible spread of water on all streets within the drainage area.

J. All drainage improvements in the City of Bentonville shall be tied to the City of Bentonville Survey Monumentation System based upon the State Plane Coordinate System, Arkansas North Zone using the North American Datum of 1983 (NAD 83). All information for newly constructed streets and roads at the time of approval shall be delivered to the City of Bentonville Engineering Department, georeferenced, in an AutoCAD compatible digital format for review and acceptance.

All drainage construction shall use the above mentioned coordinate system and shall identify with monuments that were used for horizontal and vertical control. Elevation of controlling points shall be based on USGS NAVD 88 datum.

K. Include A. through I. with Plans submitted to the Engineering Department for review. The Drainage Map submitted shall be suitable for permanent filing in the Engineering Department and shall be a good quality reproducible.

8.3 INLET SYSTEM

Determining the size and location of inlets is largely a trial and error procedure. Using criteria outlined in earlier sections of this manual, the following steps will serve as a guide to the procedure to be used.

A. Beginning at the upstream end of the project drainage basin, outline a trial subarea and calculate the runoff from it.

B. Compare the calculated runoff to allowable street capacity. If the calculated runoff is greater than the allowable street capacity, reduce the size of the trial subarea. If the calculated runoff is less than street capacity, increase the size of the trial subarea. Repeat this procedure until the calculated runoff equals the allowable street capacity. This is the first point at which a portion of the flow must be removed from the street. The percentage of flow to be removed will depend on street capacities versus runoff entering the street downstream.

C. Record the drainage area, time of concentration, runoff coefficient, and calculated runoff for the subarea. This information shall be recorded on the Plans or in tabular form convenient for review. (Figure 7.12)

D. If an inlet is to be used to remove water from the street, size the inlet(s) and record the inlet size and amount of intercepted flow, and amount of flow carried over (bypassing the inlet).

E. Continue the above procedure for other subareas until a complete system of inlets has been established. Remember to account for carry-over from one inlet to the next.

F. After a complete system of inlets has been established, modification should be made to accommodate special situations such as point sources of large quantities of runoff, and variation of street alignments and grades.

G. Record information as in C. and D. for all inlets.

H. After the inlets have been located and sized, the inlet pipes can be designed.

I. Inlet pipes are sized to carry the volume of water intersected by the inlet and direct sources. Inlet pipe capacities may be controlled by the gradient available, or by entry condition into the pipe at the inlet. Inlet pipe sizes should be determined for both conditions and the larger size thus determined should be used.

8.4 STORM SEWER SYSTEM

After the computation of the quantity of runoff entering each inlet, the storm sewer system required to carry the runoff is designed. <u>It should be borne in mind that the quantity of flow to be carried by any particular section of the storm sewer system is not the sum of the inlet design quantities of all inlets above that section of the system, but is less than the straight total</u>. This situation is due to the fact that as the time of concentration increases the rainfall intensity decreases.

8.4.1 STORM SEWER PIPE

The ground line profile is now used in conjunction with the previous runoff calculations. **The maximum elevation of the hydraulic gradient is two feet (2') below the ground surface.** When this tentative gradient is set and the design discharge is determined, a Manning flow chart may be used to determine the pipe and velocity.

It is probable that the tentative gradient will have to be adjusted at this point since the intersection of the discharge in the slope on the chart will likely occur between standard pipe sizes. The smaller pipe should be used if the design discharge and corresponding slope does not result in an encroachment on the two-foot (2') criteria below the ground surface. If there is an encroachment, use the larger pipe which will establish a capacity somewhat in excess of the design discharge. Velocities can be read directly from a Manning flow chart based on a given discharge, pipe size, and gradient slope.

8.4.2 JUNCTIONS, INLETS, AND MANHOLES

A. Determine the hydraulic gradient elevation at the upstream end and downstream end of the pipe section in question. The elevation of the hydraulic gradient of the upstream end of the pipe is equal to the elevation of the downstream (hydraulic gradient) plus the product of the length of the pipe and the friction slope.

B. Determine the velocity of flow for incoming pipe (main line) at junction, inlet or manhole at design point.

C. Determine the velocity of flow for outgoing pipe (main line) at junction, inlet or manhole at design point.

D. Compute velocity head for outgoing velocity (main line) at junction, inlet, or manhole at design point.

E. Compute velocity head for incoming velocity (main line) at junction, inlet or manhole at design point.

F. Determine head loss coefficient "K" at junction, inlet, or manhole at design point from Tables 3.5, 3.6, 3.7, or Figures 3.10 or 3.11.

G. Compute head loss at junction, inlet, or manhole.

$$h_j = K_j\,(v_2^2 - v_1^2)/2_g$$

H. Compute hydraulic gradient at upstream end of junction as if junction were not there.

I. Add head loss to hydraulic gradient elevation determined to obtain hydraulic gradient elevation at upstream end of junction.

All information shall be recorded on the Plans or in tabular form convenient for review.

8.4.3 PROPORTIONING STORM SEWER PIPES.

The computations involved in proportioning various runs of sewer pipe are summarized in the tabulation sheet titled "Storm Sewer Computations", Figure 8.1.

Computer generated computations and output are accepted and subject to review by City Engineer.

Column 1: Inlet Number - Enter the inlet number.

Column 2: Inlet Location - Enter the station and location of the inlet.

Column 3: Inlet CA from the Inlet Flow Calculation Table, Figure 7.12, the quotient of Column 25 ÷ Column 6 or Column 27 is used to obtain the CA product to be entered in Column 3. Only structures contributing flow to the system should have values in Column 3.

Column 4: Other CA - Enter the CA product of flow from any contributing upstream structure.

Column 5: Structure No. - Number the inflowing structure.

Column 6: Total CA - Enter the sum of Columns 3 and 4.

Column 7, 8, & 9: The time of concentration is the time required for water to flow from the most remote part of the drainage area or areas involved to the upper end of the pipe run under consideration. The first run time of concentration is the inlet time for the first inlet. For all succeeding runs, time of concentration may be either the time as computed along the sewer line or the inlet time of the inlet at the beginning of the run under consideration, depending upon which of these two periods is longer. Accordingly, the larger of the two is used in determining "I" and "Q", unless this larger value is less than 10 minutes, in which case the established minimum time of 10 minutes is used.

The time of concentration shown in Column 7 is computed by taking the time of concentration for the preceding run and adding it to the time required for water to flow through the preceding run to the beginning of the run under consideration.

At junctions of lines, the larger value of the time of concentration is used.

Column 10: i - Rainfall intensity in inches per hour for the design storm. Base on T_c. See Figure 2.5.

Column 11: Q_t - Total flow in pipe in CFS. Equal to the product of Column 6 times Column 10.

Columns 12: 13, 14, & 15: Pipe Characteristics - The size and gradient of pipe as shown in Columns 12 and 14 must be chosen in such manner that the pipe when flowing full, but not under head, will carry an amount of water approximately equal or greater than the computed discharge, "Q". In other words,the "Capacity" shown in Column 15 must be approximately equal to or greater than the value "Q" shown in Column 11.

The capacity may be calculated by Manning's formula:

$$Q = \frac{1.486}{n} AR^{2/3}S^{1/2}$$

or capacity can be taken directly from the appropriate nomographs in Sections III and IV.

Whenever a pipe run is designed in such a manner that the capacity of a pipe as shown in Column 15 is less than the computed discharge shown in Column 11, a check of the hydraulic gradient above this run should be made to make such that the backwater head created by such a design is not large enough to cause blowouts at inlets or junctions above the run.

Column 16: The velocities shown in this column can be calculated by Manning's formula:

$$V = \frac{1.486}{n} R^{2/3}S^{1/2}$$

or the velocities can be taken directly from the appropriate graphs or figures in Sections III and IV.

Column 17: L - The length of each run as shown in this column is the length center to center of inlets or junctions in feet. This length is used in determining the time of flow from one inlet or junction to another.

Column 18: Pipe T_c - The time of concentration in the pipe under consideration is actual flow time, in minutes from the present inlet to the next junction point. Run time is calculated by dividing the length of run (Column 17) by velocity of flow (Column 16) and converting the answer to minutes by dividing by 60.

Columns 19 to 24: These columns are believed to be self-explanatory.

8.4.4 HYDRAULIC GRADE LINE

The final step in designing a storm sewer is to check the Hydraulic Grade Line (HGL). Computing the HGL will determine the elevation under design conditions to which water will rise in various inlets, manholes, junctions, and etc.

The HGL should be computed for all storm sewer systems. Computations are summarized in tabulation sheet entitled "Hydraulic Grade Line", Figure 8.2.

Computer generated computations and output are accepted and subject to review by City Engineer.

Column 1: Inlet Station - Enter the station for the junction immediately upstream of the outflow pipe. HGL computations begin at the outfall and are worked upstream taking each junction into consideration.

Column 2: Outlet Water Surface Elevation - Enter tailwater elevation in feet if the outlet will be submerged during the design storm or 0.8 diameter of pipe plus invert out elevation of the outflow pipe, whichever is greater.

Column 3: D_o - Enter diameter of outflow pipe in inches.

Column 4: Q_o - Enter design discharge for outflow pipe in CFS.

Column 5: L_o - Enter length of outflow pipe in feet.

Column 6: S_{fo} - Enter friction slope in feet/foot of the outflow pipe using the Manning's formula:

$$S_f = \left(\frac{Qn}{1.486\ AR^{2/3}} \right)^2$$

Column 7: H_f - Enter friction loss by multiplying Column 5 by Column 6.

Column 8: V_o - Enter velocity of the outflow pipe in feet per second.

Column 9: Q_i - Enter design discharge (Q_1, Q_2, Q_3...) in CFS for each pipe flowing into the junction.

Column 10: V_i - Enter velocity (V_1, V_2, V_3...) in feet per second for each pipe flowing into the junction.

Column 11: H_{tm} - Enter terminal junction losses in feet for the upper reach of each storm sewer run using the formula:

$$H_{tm} = \frac{V^2}{2g}$$

Column 12: H_e - Enter pipe entrance losses in feet for the upper reach of each storm sewer run using the formula:

$$H_e = \frac{K\,V^2}{2g}$$

where: K = 0.5 for square-edge

Column 13: Enter junction losses H_{jl} or H_{j2} in feet for each junction using the formula:

$$H_{jl} = \frac{V^2\ \text{outflow}}{2g}$$

or:

$$H_{j2} = \frac{Q_4V_4^2 - Q_1V_1^2 - Q_2V_2^2 + KQ_1V_1^2}{2gQ_4}$$

Column 14: H_b - Enter bend losses (changes in direction of flow) in feet for each inflowing pipe to the outflow pipe using the formula:

$$H_b = \frac{K V^2}{2g}$$

Refer to Section III for "K" values.

Column 15: H_t - Enter total head losses in feet using the formula:

$$H_t = H_f + H_{tm} + H_e + H_{jl} \text{ or } H_{j2} + H_b$$

Column 16: HGL - Enter the new Hydraulic Grade in feet by summing elevations in column 2 and column 15. This elevation is the potential water surface elevation for the junction under design conditions.

Column 17: Enter the top of junction cover or the gutter flow line, whichever is lowest and compare it with the HG in Column 16.

CITY OF BENTONVILLE, ARKANSAS
STORM SEWER COMPUTATIONS

PROJECT ____________________

PLAN SHT. NO. __________ DATE __________

COMPUTED BY __________ CKD. __________

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)		PIPE DESIGN							PIPE INVERTS			DROP INLET		
Inlet No.	Location	Inlet CA	Other CA	From Struct. No.	Total CA	Inlet Tc (Min)	Pipe Tc (Min)	Design Tc (Min)	I (IN/HR)	(11) Pipe Qt (CFS)	(12) Pipe Size (IN)	(13) Pipe n	(14) Pipe Slope (FT/FT)	(15) Pipe Cap. (CFS)	(16) Pipe Vp (FPS)	(17) Pipe L (FT)	(18) Pipe Tc (Min)	(19) Inlet Elev. (FT)	(20) Total Fall (FT)	(21) Outlet Elev. (FT)	(22) Flow Line (FT)	(23) Top Elev. (FT)	(24) HT (FT)

STORM SEWER COMPUTATIONS

Figure 8.1

CITY OF BENTONVILLE, ARKANSAS
HYDRAULIC GRADE LINE

PROJECT ____________________

PLAN SHT. NO. ____________ DATE ____________

COMPUTED BY ____________ CKD. ____________

(1) Inlet Station	(2) Outlet WSE	(3) Do	(4) Qo	(5) Lo	(6) Sfo	(7) Hf	VELOCITY HEAD LOSSES								(16) Total HGL	(17) Top Ring or Gutter
							(8) Vo	(9) Qi	(10) Vi	(11) Htm	(12) He	(13) Hj	(14) Hb	(15) HL		

HYDRAULIC GRADE LINE

Figure 8.2

TABLE OF CONTENTS - SECTION IX

SECTION IX - OPEN CHANNEL FLOW

9.1 GENERAL

Open channels for use in the major drainage system have significant advantage in regard to cost, capacity, multiple use for recreational and aesthetic purposes, and potential for detention storage. Disadvantages include right-of-way needs and maintenance costs. Careful planning and design are needed to minimize the disadvantages, and to increase the benefits.

Open channels may be used in lieu of storm sewers to convey storm runoff where:

(1) Sufficient right-of-way is available;

(2) Sufficient cover for storm sewers is not available;

(3) To maintain compatibility with existing or proposed developments; and

(4) Where economy of construction can be shown without long-term public maintenance expenditures.

Intermittent alternating reaches of opened and closed systems should be avoided. Closed systems are preferred due to the inherent hazard of open channels in urban areas and the tendency for trash to collect in open channels.

The ideal channel is a natural one carved by nature over a long period of time. The benefits of such a channel are:

(1) Velocities are usually low, resulting in longer concentration times and lower downstream peak flows.

(2) Channel storage tends to decrease peak flows.

(3) Maintenance needs are usually low because the channel is somewhat stabilized.

(4) The channel provides a desirable green belt and recreational area adding significant social benefits.

Generally speaking, the natural channel or the man-made channel which most nearly conforms to the character of the natural channel is the most efficient and the most desirable.

The City has adopted an ongoing ditch maintenance program that is based upon comprehensive field inventories and analysis, and a system of establishing priorities based upon flooding potentials.

In many areas facing urbanization, the runoff has been so minimal that natural channels do not exist. However, a small trickle path nearly always exists which provides an excellent basis for location and construction of channels. Good land planning should reflect even these minimal trickle channels to reduce development cost and minimize drainage problems. In most cases, the prudent utilization of natural water routes in the development of major drainage system will reduce the requirements for an underground storm sewer system.

Channel stability is a well recognized problem in urban hydrology because of the significant increases in low flows and peak storm runoff flows. A natural channel must be studied to determine the measures needed to avoid future bottom scour and bank cutting. Erosion control measures can be used. This also helps reduce public cost and maintaining the channel in the future.

Sufficient right-of-way or permanent easement should be provided adjacent to open channels to allow entry of City maintenance vehicles.

9.2 DESIGN CRITERIA

Open channels shall be designed to the following criteria:

(1) Channel shall carry the 25 year storm minimum with free board specified herein.

(2) Channel or adjacent <u>public</u> drainage easement, floodway, etc., shall be capable of carrying the 100 year storm.

(3) When open channel flow velocity exceeds 5 fps, the channel shall be paved to a point 1 foot above the design water surface or other forms of stabilization shall be used.

9.2.1 CHANNEL DISCHARGE - MANNING'S EQUATION

Careful attention must be given to the design of drainage channels to assure adequate capacity and minimum maintenance to overcome the results of vegetative growth, erosion, and silting. The hydraulic characteristics of channels shall be determined by Manning's equation.

$$Q = \frac{1.49}{n} AR^{2/3}S^{1/2}$$

Q = Total discharge in CFS

n = Coefficient of roughness

A = Cross-sectional area of channel (square feet)

R = Hydrologic radius of channel (feet)

S = Slope of channel (feet per foot)

For a given channel condition of roughness, discharge and slope, there is only one possible depth for maintaining a uniform flow. This depth is the normal depth. When roughness, depth, and slope are known at a channel section, there can only be one discharge for maintaining a uniform flow through the section. This discharge is the normal discharge.

If the channel is uniform in resistance and gravity forces are in exact balance, the water surface will be parallel to the bottom of the channel. This is the condition of uniform flow.

Uniform flow is more often a theoretical abstraction than an actuality. True uniform flow is difficult to find in the field or to obtain in the laboratory. Channels are sometimes designed on this assumption that they will carry uniform flow at the normal depth, but because of conditions difficult, if not impossible, to evaluate and hence not taken into account, the flow will actually have depths considerably different from uniform depth. The Engineer must be aware of the fact that uniform flow computation provides only an approximation of what will occur; however, such computations are useful and necessary for planning.

The normal depth is computed so frequently in trapezoidal channels that it is convenient to use nomographs for such types of cross sections to eliminate the need for trial and error solutions, which are time-consuming. A nomograph for uniform flow is given in Figure 9.1.

Open channel flow in urban drainage systems is usually nonuniform because of bridge openings, curbs, and structures. This necessitates the use of backwater computations for all final channel design work.

A water surface profile must be computed for all channels and shown on all final drawings. Computation of the water surface profile should utilize standard backwater methods or acceptable computer routines, taking into consideration all losses due to the changes in velocity, drops, bridge openings, and other obstructions.

Channels should have trapezoidal sections of adequate cross-sectional areas to take care of uncertainties in runoff estimates, changes in channel coefficients, channel obstructions, and silt accumulations.

Accurate determinations of the "n" value is critical in the analysis of the hydraulic characteristics of a channel. The "n" value of each channel reach should be based on experience and judgment with regard to the individual channel characteristics. Table 9.1 gives a method of determining the composite roughness coefficient based on actual channel conditions.

Where practical, unlined channels should have sufficient gradient, depending upon the type of soil, to provide velocities that will be self-cleaning but will not be so great as to create erosion. Lined channels, drop structures, check dams, or concrete spillways may be required to control erosion that results from the high velocities of large volumes of water. Unless approved otherwise by the City Engineer, channel velocities in paved man-made channels shall not exceed ten (10) feet per second.

Where velocities in excess of five (5) feet per second are encountered, riprap, pavement, or other approved protective erosion shall be required. As minimum protection to reduce erosion, all open channels slopes shall be seeded or sodded as soon after grading as possible.

9.2.2 CHANNEL CROSS SECTIONS

The channel shape may be almost any type suitable to the location and to the environmental conditions. Often the shape can be chosen to suit open space and recreational needs to create additional benefits.

(1) Side Slope

Except in horizontal curves, the flatter the side slope, the better. Normally, slopes shall be no steeper than 3 horizontal:1 vertical (3:1), which is also the practical limit for mowing equipment, unless approved in writing by City Engineer.

Rock or concrete lined channels or those that for other reasons do not require slope maintenance may have slopes as steep as 1-1/2 horizontal:1 vertical (1.5:1), or rectangular vertical if walls are structurally designed, unless approved in writing by City Engineer.

(2) Depth

Deep channels are difficult to maintain and can be hazardous. Constructed channels should, therefore, be as shallow as practical, and they shall not exceed 4 feet unless approved in writing by City Engineer.

(3) Bottom Width

Channels with narrow bottoms are difficult to maintain and are conducive to high velocities during high flows. It is desirable to design open channels such that the bottom width is at least twice the depth unless approved in writing by City Engineer.

(4) Bend Radius

Twenty-five (25) feet or ten (10) times the bottom width, whichever is larger, is the minimum bend radius required for open channels.

(5) Trickle Channels

The low flows, and sometimes base flows, from urban areas must be given specific attention. If erosion of the bottom of the channel appears to be a problem, low flows shall be carried in a paved trickle channel which has a capacity of 5.0 percent of the design peak flow. Care must be taken to ensure that low flows enter the trickle channel without the attendant problem of the flow paralleling the trickle channel. Concrete trickle channels are required when channel slope is less than 1%.

(6) Freeboard

For channels with flow at high velocities, surface roughness, wave action, air bulking, and splash and spray are quite erosive along the top of the flow. Freeboard height should be chosen to provide a suitable safety margin. The height of freeboard should be a minimum of 1-foot for velocities up to 8 FPS and 2' for velocities over 8 FPS or provide an additional capacity of approximately one-third of the design flow. For deep flows with high velocities, one may use the formula:

Freeboard (in feet) = $1.0 + 0.025\ VD^{1/3}$, where

V = Velocity of flow

D = Depth of flow

For the freeboard of a channel on a sharp curve, extra height must be added to the outside bank or wall in the amount:

$$H = V^2 \frac{(T + B)}{2gR}$$

H = Additional height on outside edge of channel (feet)

V = Velocity of flow in channel (feet per sec.)

T = Width of flow at water surface (feet)

B = Bottom width of channel (feet)

R = Centerline radius of turn (feet)

g = Acceleration of gravity (32.2 feet per $sec.^2$)

If R is equal or greater than 3 x B, additional freeboard is not required.

(7) Connections

Connections at the junction of two or more open channels shall be smooth. Pipe and box culvert or sewers entering an open channel will not be permitted to project into the normal channel section, nor will they be permitted to enter an open channel at an angle which would direct flow from the culvert or sewer upstream in the channel.

9.3 CHANNEL DROP

The use of channel drops permits adjustment of channel gradients which are too steep for the design conditions. In urban drainage work, it is often desirable to use several low head drops in lieu of a few higher drops.

The use of sloped drops will generally result in lower installation cost. Sloped drops can easily be designed to fit the channel topography.

Sloped drops shall have roughened faces and shall be no steeper than 2:1. They shall be adequately protected from scour, and shall not cause an upstream water surface drop which will result in high velocities upstream. Side cutting just downstream from the drop is a common problem which must be protected against.

The length of the drop (L) will depend upon the hydraulic characteristics of the channel and drop. For a Q of 30 cubic feet per second/feet, L would be about 15 feet, that is, about 1/2 of the Q value. The L should not be less than 10 feet, even for low Q values. In addition, follow-up riprapping will often be necessary at most drops to more fully protect the banks and channel bottom. The criteria given is minimal, based on the philosophy that it's less costly to initially under protect with riprap, and to place additional protection after erosional tendencies are determined in the field. Project approvals are to be based on provisions for such follow-up construction.

9.4 BAFFLE CHUTES

Baffle chutes are used to dissipate the energy in the flow at a larger drop. They require no tailwater to be effective. They are useful where the water surface upstream is held at a higher elevation to provide head for filling a side storage pond during peak flows.

Baffle chutes are used in channels where water is to be lowered from one level to another. The baffle piers prevent undue acceleration of the flow as it passes down the chute. Since the flow velocities entering the downstream channel are low, no stilling basin is needed. The chute, on a 2:1 slope or flatter, may be designed to discharge up to 60 CFS per foot of width, and the drop may be as high as structurally feasible. The lower end of the chute is constructed to below stream bed level and backfilled as necessary. Degradation of the streambed does not, therefore, adversely affect the performance of the structure. In urban drainage design, the lower end should be protected from the scouring action.

The baffled apron shall be designed for the full discharge design flow. Baffle chutes shall be designed using acceptable methods such as those presented by A.J. Perterka, Engineering Monograph No. 25, "Hydraulic Design of Stilling Basins and Energy Dissipators", U.S. Department of the Interior, Bureau of Reclamation.

9.5 STRUCTURAL AESTHETICS

The use of hydrologic structures in the urban environment requires an approach not encountered elsewhere in the design of such structures. The appearance of these structures is very important. The treatment of the exterior should not be considered of minor importance. Appearance must be an integral part of the design.

Parks. It must be remembered that structures are often the only above-ground indication of the underground works involved in an expensive project. Furthermore, parks and green belts may later be developed in the area in which the structure will play a dominant environmental role.
Play Areas: An additional consideration is that the drainage structures offer excellent opportunities for neighborhood children to play. It is almost impossible to make drainage works inaccessible to children, and therefore, what is constructed should be made as safe as is reasonably possible. Safety hazards should be minimized and vertical drops protected with decorative fencing or rails.

Concrete Surface Treatment: The use of textured concrete presents a pleasing appearance and removes form marks. Exposed aggregate concrete is also attractive but may require special control of aggregate used in the concrete.

Rails and Fences: The use of rails and fences along concrete walls provides a pleasing topping to an otherwise stark wall, and also gives a degree of protection against someone inadvertently falling over the wall.

9.6 COMPUTATION FORMAT

Figure 9.2 is to be used for open channel design. Computer generated computations and output are accepted and subject to review by City Engineer.

The steps to follow in an open channel design are:

1. List all the design data (i.e., location, area, runoff coefficients, typical section, slope, etc.).

2. Determine the initial time of concentration (T_o).

3. Estimate travel time (T_d) through study reach and add to initial time of concentration to obtain time of concentration (T_c) at lower end of study reach.

4. Determine the discharge for the design storm using T_c.

5. Enter the discharge and slope in the appropriate channel design chart with the discharge in the slope to find the velocity and depth of flow.

6. Check the estimated travel time against the calculated velocity using Manning's equation.

   A. If the estimated travel time is comparable to the calculated travel time ($\pm$1.0 min.) proceed to Step 7.

   B. If the estimated travel time does not check with the calculated travel time, repeat Steps 3-6 until an agreement is reached.

7. If excessive velocities or water depths are determined, select another typical section, revise channel grade, or revise lining and repeat Steps 3-7.

8. Similar calculations are to be made to determine operational characteristics - freeboard, velocity, etc.

9.7 CHANNEL LINING DESIGN

9.7.1 UNLINED CHANNELS

The design charts for unlined channels (bare soils) are based on tests on 10 different classes of soils, ranging from cohesive clays to noncohesive sands and gravels. These are Figures 9.3 and 9.4. Generally, sandy, noncohesive soils tend to be very erodible, the large grained gravel clay-silt mixtures are erosion resistant, and the mixtures of sand, clay, and colloids are moderately erodible.

9.7.2 TEMPORARY LININGS

Temporary linings are flexible coverings used to protect a channel until permanent vegetation can be established using seeding. For the most part, the materials used are biodegradable. Listed below are some of the temporary linings that can be used, which are established in the charts for this section. Among the factors which should be known in order to use these are hydraulic radius, soil condition, and channel slope. When one or all of these factors are known, then a flow velocity or maximum flow depth can be determined from these charts.

1. *Fiber Glass Roving
2. *Jute Matting
3. *Wood Fiber

* Refer to the Arkansas Highway and Transportation Department's Standard Specifications for material descriptions and construction methods.

9.7.3 GRASS LININGS

Several different types of vegetative covers are listed and grouped according to degree of retardance in Table 9.2. This Table can be used in conjunction with seeding specification in the Arkansas Highway and Transportation Department's Standard Specifications. Figures 9.14 through 9.21 determine flow velocities or maximum flow depths given such factors as channel slope, hydraulic radius, and/or soil types. Table 9.3 is relatively good source to check permissible velocities for different types of grass linings in channels.

9.7.4 ROCK RIPRAP

The resistance of random riprap to displacement by moving water depends upon:

1. Weight, size, shape, and composition of the individual stones.
2. The gradation of the stone.
3. The depth of water over the stone blanket.
4. The steepness and stability of the protected slope and angle of repose of riprap.
5. The stability and effectiveness of the filter blanket on which the stone is placed.
6. The protection of toe and terminals of the stone blanket.

The size of stone needed to protect a streambank or highway embankment from erosion by a current moving parallel to the embankment is determined by the use of Figures 9.22, 9.23 and 9.24.

When rock riprap is used, the need for an underlying filter material must be evaluated. The filter material may be either a granular blanket or plastic filter cloth. All rip-rap shall include concrete slurry to increase stability and minimize vegetative growth.

9.8 DESIGN OF GRANULAR FILTER BLANKET

For a granular filter blanket, the following criteria should be met:

$$\frac{D_{15}\text{ filter}}{D_{85}\text{ base}} < 5 < \frac{D_{15}\text{ filter}}{D_{15}\text{ base}} < 40$$

and

$$\frac{D_{50}\text{ filter}}{D_{50}\text{ base}} < 40$$

In the above relationships, filter refers to the overlying material. The relationships must hold between the filter blanket and base material and the riprap and filter blanket.

9.9 CONCRETE

Concrete lined channels provide high capacities, but also have high outlet velocities so erosion problems become evident and must be dealt with. Capacity Figures 9.25 through 9.36 related velocity and discharge to the channel geometry, slope and resistance. The Manning equation may be solved through trial and error by using the trapezoidal channel charts. Maximum velocity of concrete lined channel shall be 10 fps unless otherwise approved in writing by City Engineer.

9.10 OTHER LINING OPTIONS

Other lining options shall be reviewed on a case by case basis and approved by the City Engineer.

TABLE 9.1

COMPUTATION OF COMPOSITE ROUGHNESS COEFFICIENT
FOR EXCAVATED AND NATURAL CHANNELS

$$n = (n_o + n_1 + n_2 + n_3 + n_4)\, m$$

	CHANNEL CONDITIONS	VALUE
Material Involved n_0	Earth	0.020
	Rockcut	0.025
	Final Gravel	0.024
	Coarse Gravel	0.028
Degree of Irregularity n_1	Smooth	0.000
	Minor	0.005
	Moderate	0.010
	Severe	0.020
Variation of Channel Cross Section n_2	Gradual	0.000
	Alternating Occasionally	0.005
	Alternating Frequently	0.010-0.015
Relative Effect Of Obstructions n_3	Negliible	0.000
	Minor	0.010-0.015
	Appreciable	0.020-0.030
	Severe	0.040-0.060
Vegetation n_4	Low	0.005-0.010
	Medium	0.010-0.025
	High	0.025-0.050
	Very High	0.050-0.100
Degree of Meandering m	Minor	1.000-1.200
	Appreciable	1.200-1.500
	Severe	1.500

Roughness Coefficient For Lined Channels

Concrete Lined - n = 0.017
Rubble RipRap - n = 0.022

<u>Open Channel Hydraulics</u>
Ven Te Chow, Ph.D.

TABLE 9.2

CLASSIFICATION OF VEGETEL COVERS AS TO DEGREE OF RETARDANCE

Note: Covers classified have been tested in experimental channels.
Covers were green and generally uniform.

Retardance	Cover	Condition
A	Weeping lovegrass.....................	Excellent stand, tall, (average 30")
	Yellow bluestem, Ischaemum........	Excellent stand, tall, (average 36")
B	Kudzu......................................	Very dense growth, uncut
	Bermudagrass..........................	Good Stand, tall (average 12")
	Native grass mixture, (little bluestem, blue grama, and other long and short mid-west grasses)...	Good Stand, unmowed
	Weeping Lovegrass....................	Good stand, tall, (average 24")
	Lespedeza sericea.....................	Good stand, not woody, tall average 19")
	Alfalfa......................................	Good stand, uncut (average 12")
	Weeping lovegrass.....................	Good stand, mowed, (average 18")
	Kudzu......................................	Dense growth, uncut
	Blue Grama..............................	Good stand, uncut, (average 18")
C	Crabgrass.................................	Fair Stand, uncut (10" to 48")
	Bermudagrass..........................	Good Stand, mowed (average 6")
	Common lespedeza....................	Good Stand, uncut (average 6")
	Grass legume mixture—summer (orchard grass, redtop, Italian ryegrass, and common lespedeza)..	Good Stand, uncut (average 6" to 2')
	Centipedegrass.........................	Very dense cover (average 6")
	Kentucky Bluegrass....................	Good Stand, headed (6" to 12")
D	Bermudagrass..........................	Good stand, cut to 2.5" height
	Common lespedeza....................	Excellent Stand, uncut (average 4"-6")
	Buffalograss............................	Good Stand, cut to 3"-6"
	Grass legume mixture—fall, spring (orchard grass, redtop, Italian ryegrass, and common lespedeza)..	Good Stand, uncut (4" to 5")
	Lespedeza Sericea.....................	After cutting to 2" height Very good stand before cutting
E	Bermudagrass..........................	Good Stand, cut to 1.5" height
	Bermudagrass..........................	Burned stubble

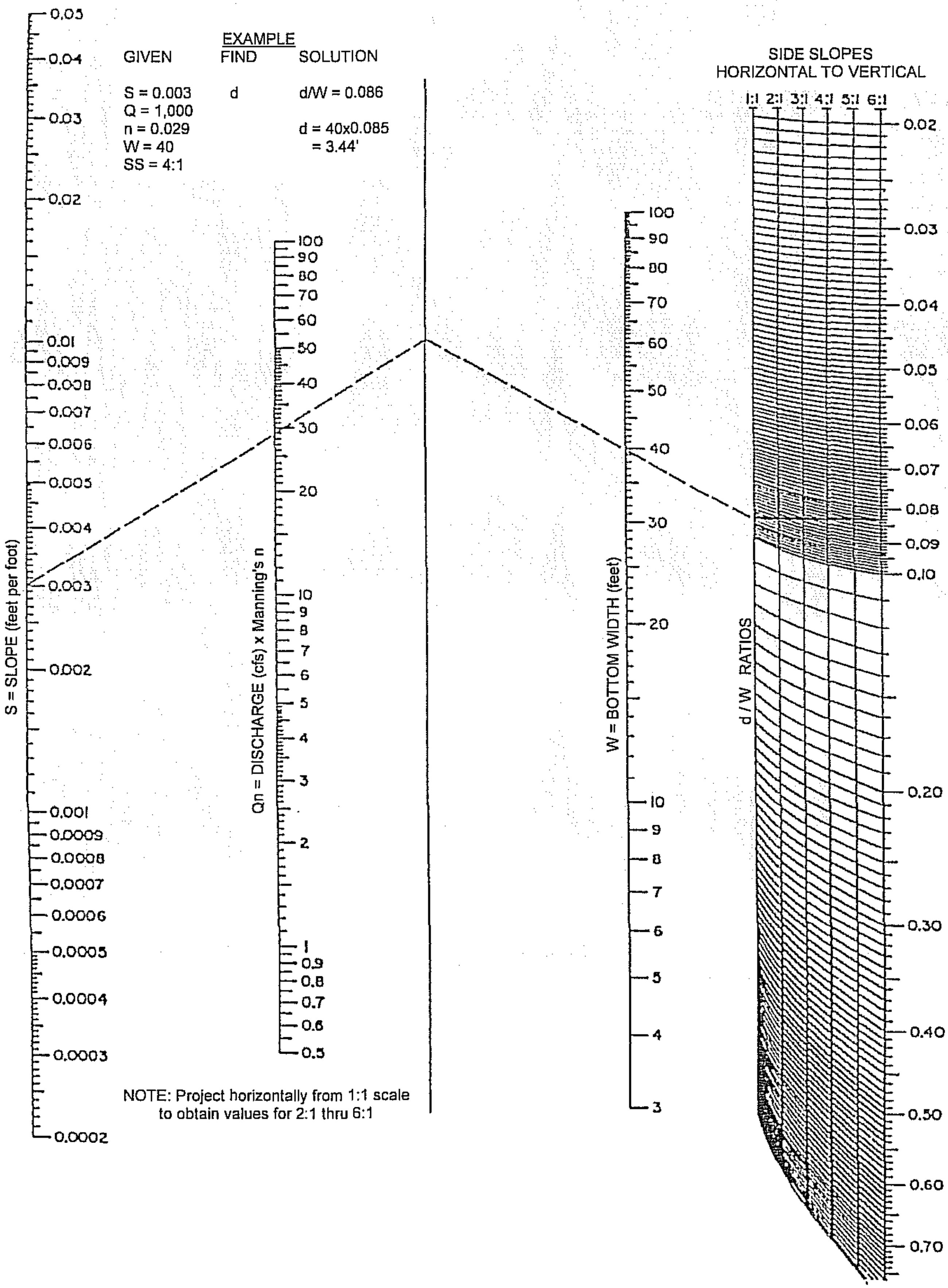
From: SCS "Handbook of Channel Design for Soil and Water Conservation".

TABLE 9.3

PERMISSIBLE VELOCITIES FOR CHANNELS LINED WITH GRASS

Cover	Slope Range %	Permissible Velocity, fps: Erosion-Resistant Soils	Permissible Velocity, fps: Easily Eroded Soils
Bermudagrass	0-5	5	5
	5-10	5	5
	>10	5	4
Buffalo Grass, Kentucky Bluegrass, smooth brome, blue grama	0-5	5	5
	5-10	5	4
	>10	5	3
Grass Mixture	0-5	5	4
	5-10	4	3
	Do not use on slopes steeper than 10%		
Lespedeza Sericea, weeping love grass, ischaemum, yellow bluestem, kudzu, alfalfa, crabgrass	0-5	3.5	2.5
	Do not use on slopes steeper than 5% except for side slopes in combination channel		
Annuals -- used on mild slopes or as temporary protection until permanent covers are established, common lespedeza, Sudan Grass	0-5	3.5	2.5
	Use on Slopes steeper than 5% not recommended		

Remarks: The values apply to average, uniform stands of each type of cover. Use velocities exceeding 5 fps only for paved channels.



UNIFORM FLOW FOR TRAPEZOIDAL CHANNELS

SOURCE: Texas Highway Department

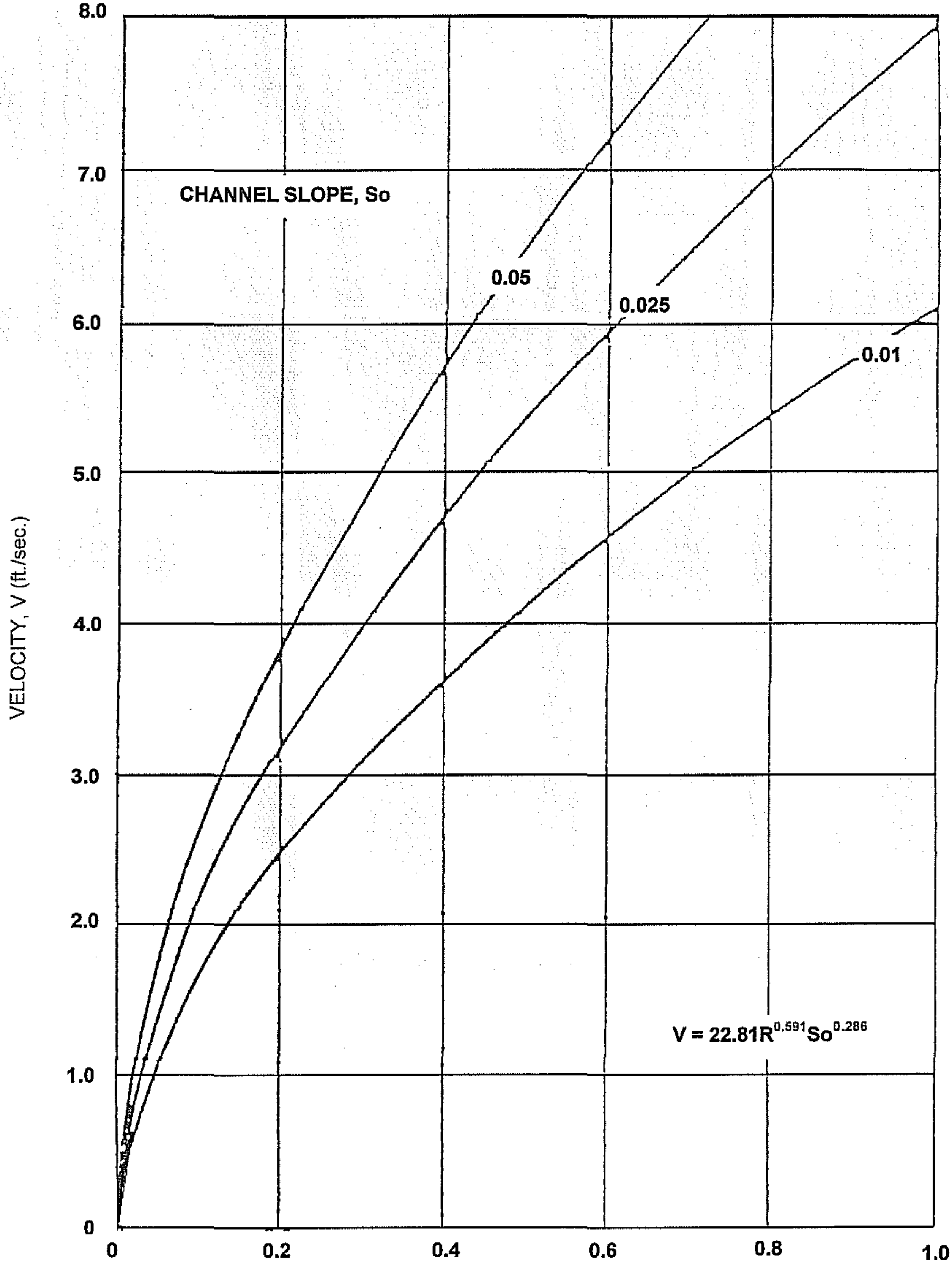
Figure 9.1

DITCH DESIGN FROM

Location	Area Acres	C	EA x C	ΣA x C	T_o Min	T_d Min	I IN/HR	Q CFS	Typical Section	So f/f	n	V. Calc Ft/Sec	Water Depth	Lining Depth	Type Lining	Remarks

DITCH DESIGN FORM

Figure 9.2

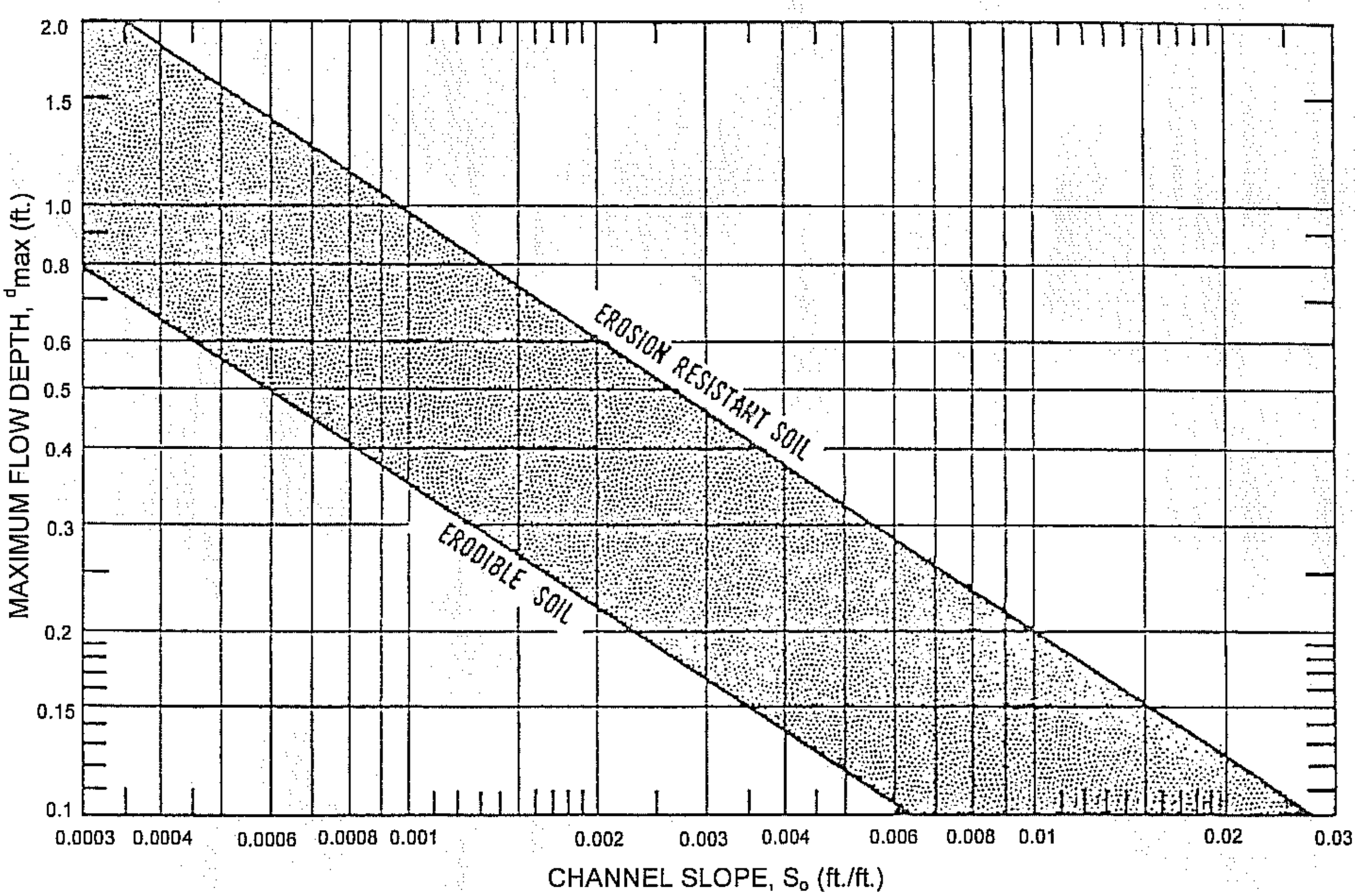


$$V = 22.81R^{0.591}So^{0.286}$$

FLOW VELOCITY FOR UNLINED CHANNELS
(BARE SOIL)

SOURCE: AHTD

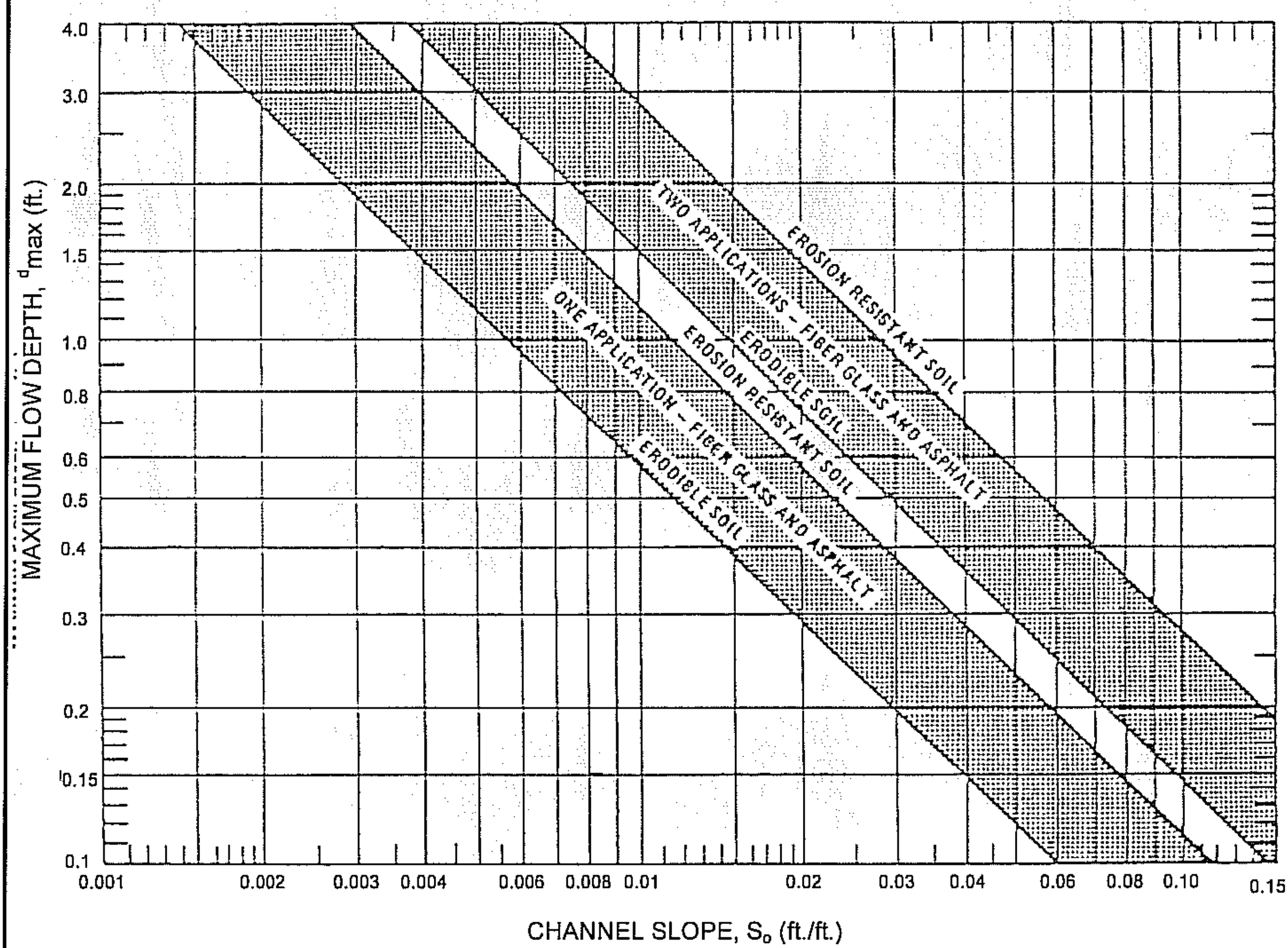
Figure 9.3



MAXIMUM PERMISSIBLE DEPTH OF FLOW (^{d}max.) FOR UNLINED CHANNELS (BARE SOIL)

SOURCE: AHTD

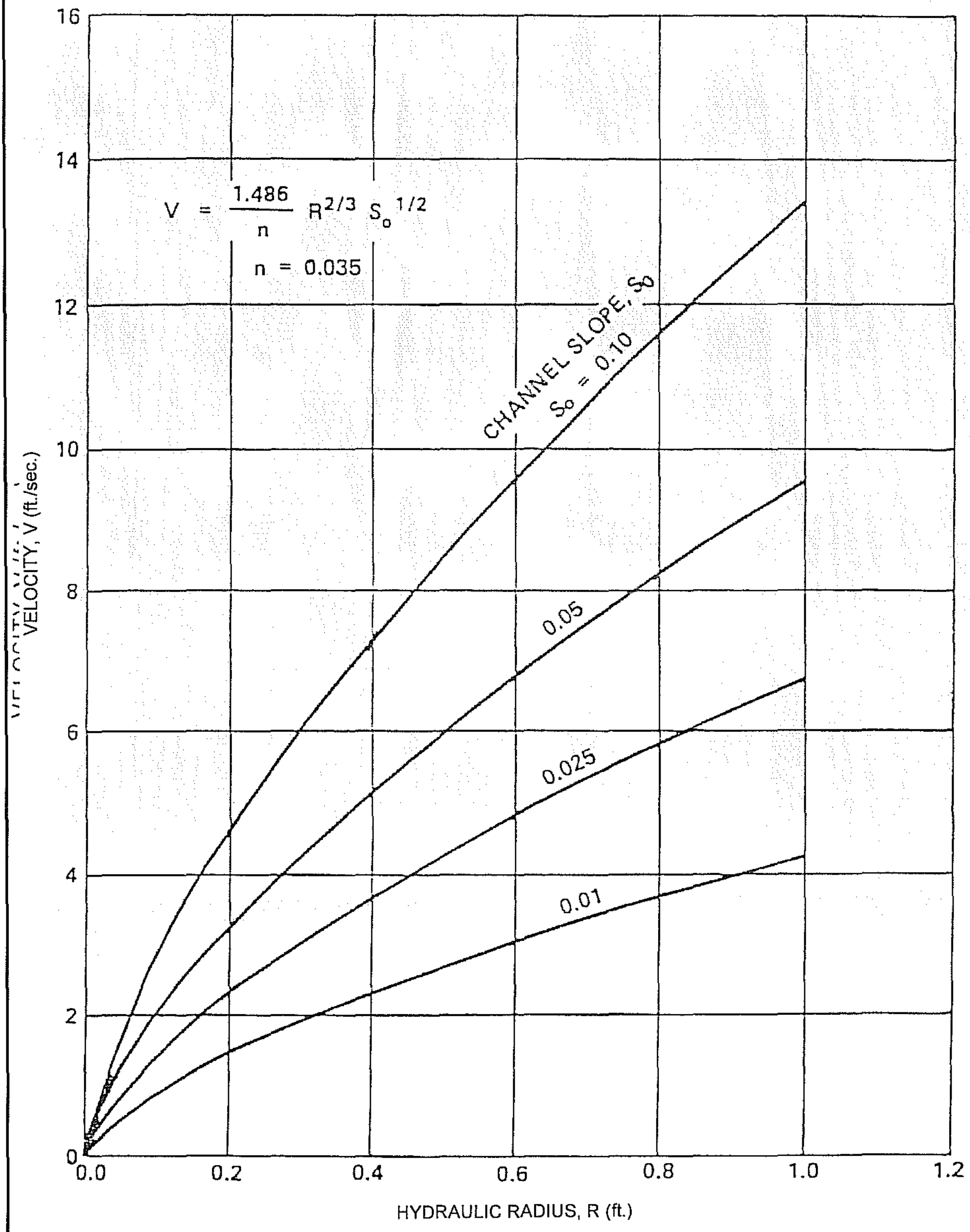
Figure 9.4



MAXIMUM PERMISSIBLE DEPTH OF FLOW (^{d}max.) FOR CHANNELS LINED WITH FIBER GLASS ROVING (SINGLE AND DOUBLE LAYER)

SOURCE: AHTD

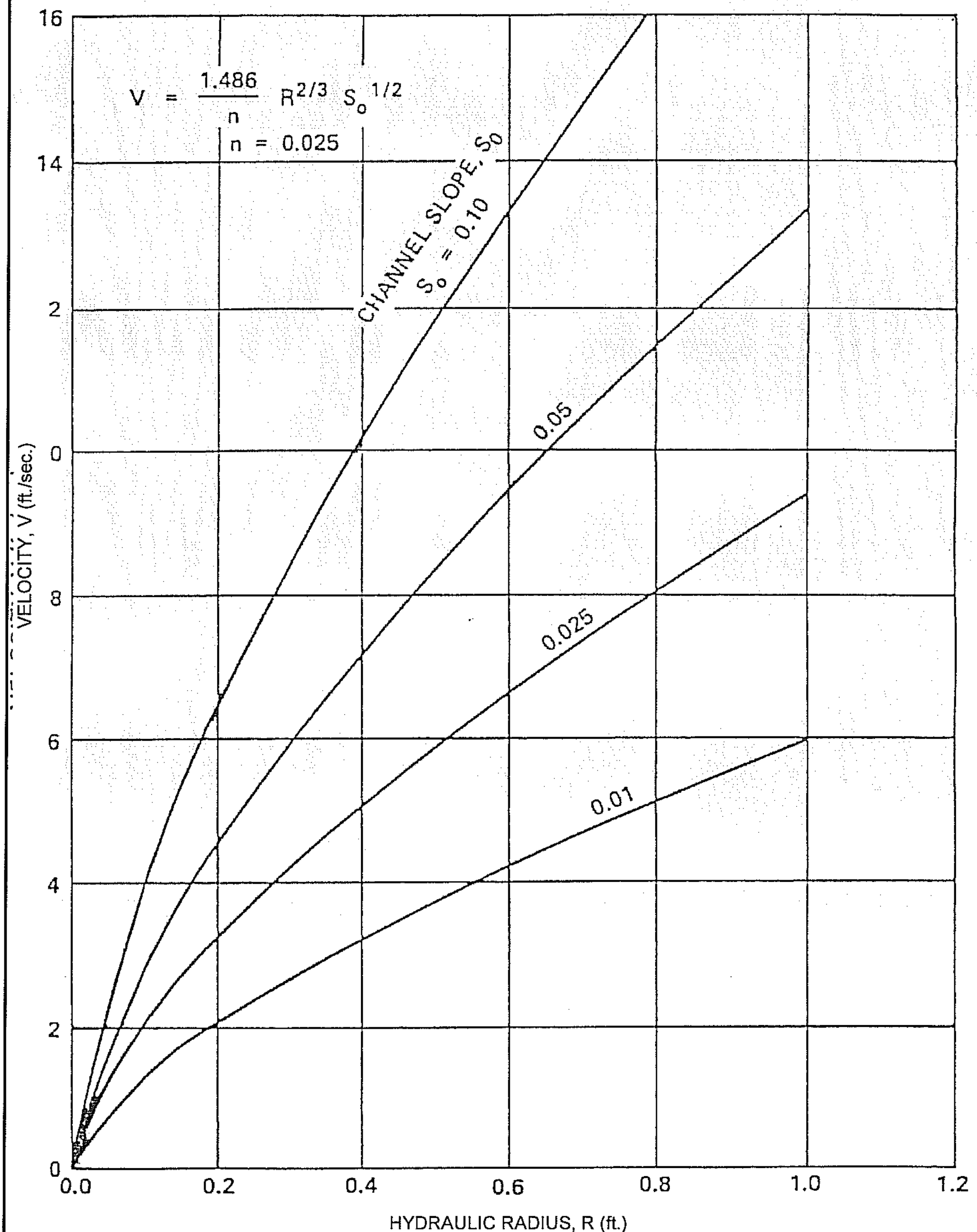
Figure 9.5



FLOW VELOCITY FOR CHANNELS LINED WITH
FIBER GLASS ROVING TACKED WITH ASPHALT
SINGLE LAYER

SOURCE: AHTD

Figure 9.6



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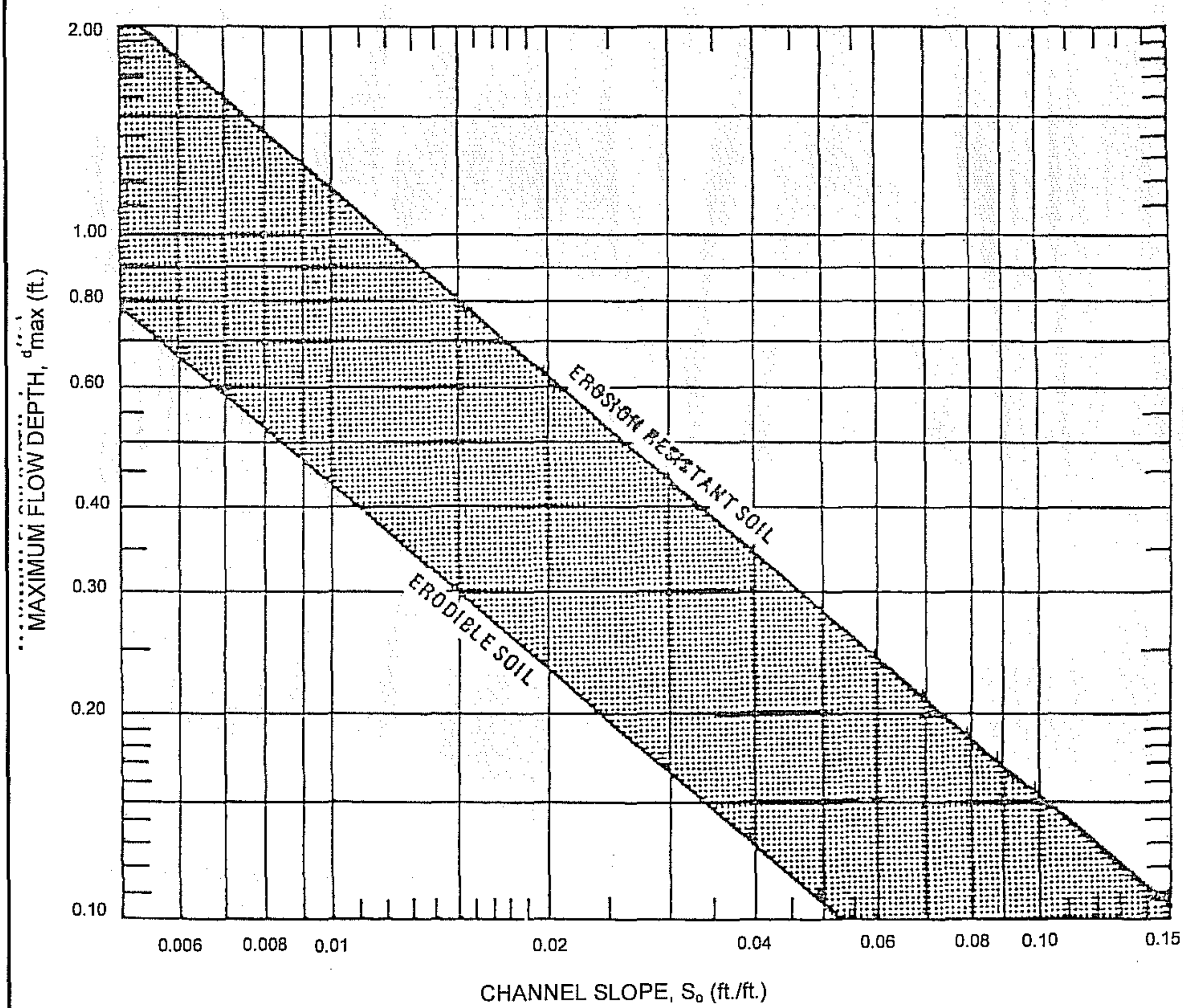
FLOW VELOCITY FOR CHANNELS LINED WITH FIBER GLASS ROVING TACKED WITH ASPHALT DOUBLE LAYER

SOURCE: AHTD

Figure 9.7



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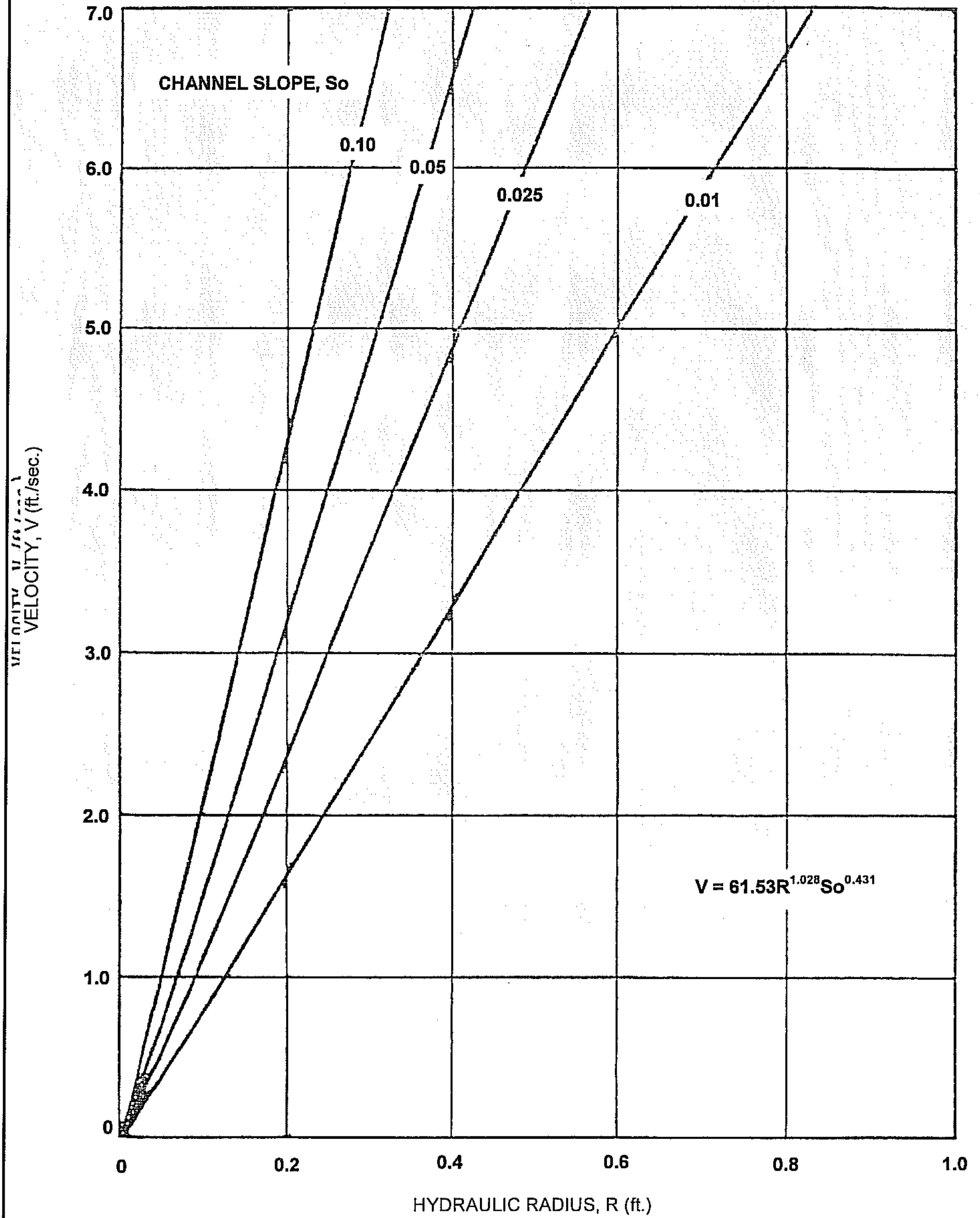
MAXIMUM PERMISSIBLE DEPTH OF FLOW (^{d}max.) FOR CHANNELS LINED WITH JUTE MESH

SOURCE: AHTD

Figure 9.8



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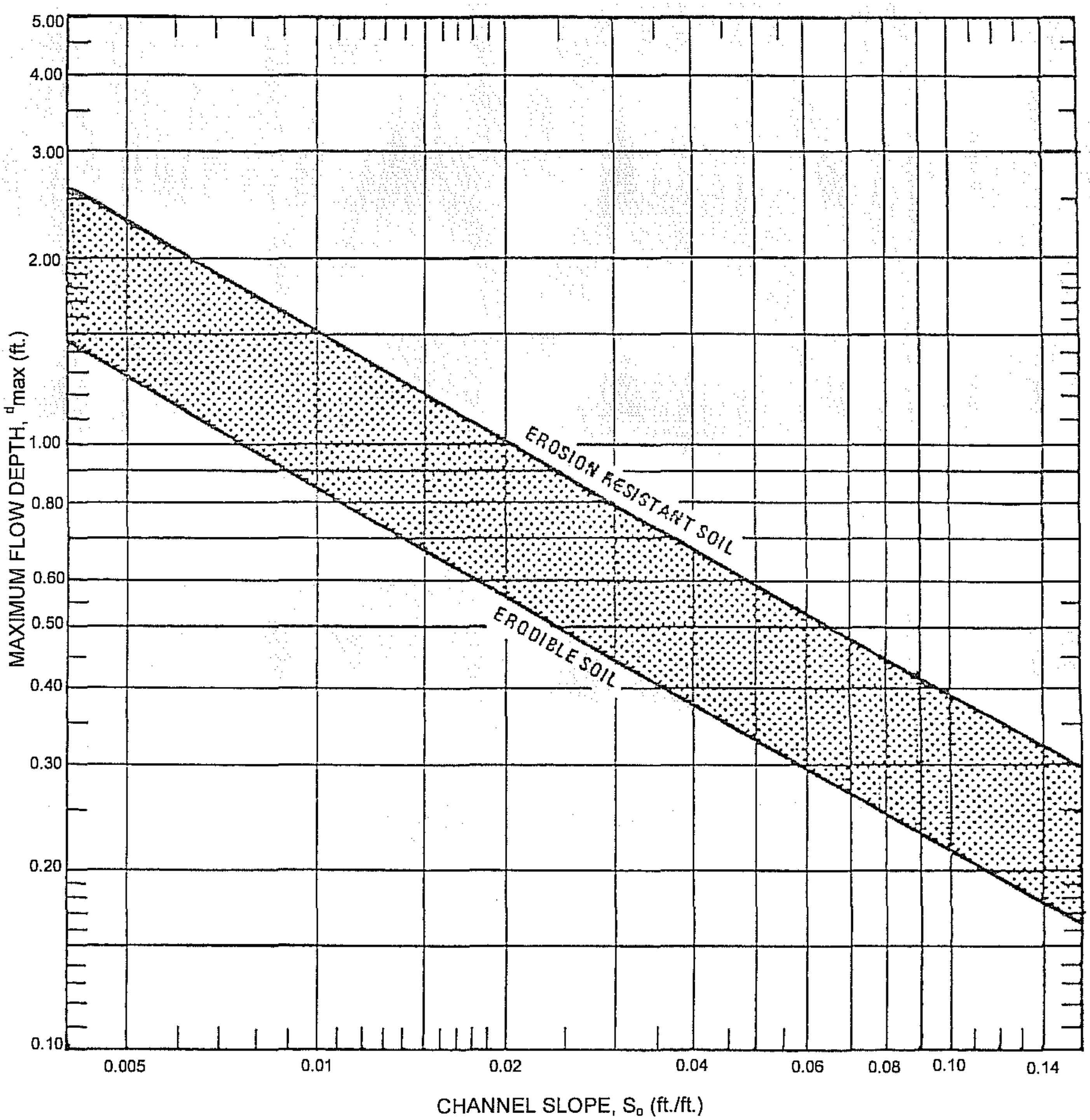
FLOW VELOCITY FOR CHANNELS LINED WITH JUTE MESH

SOURCE: AHTD

Figure 9.9



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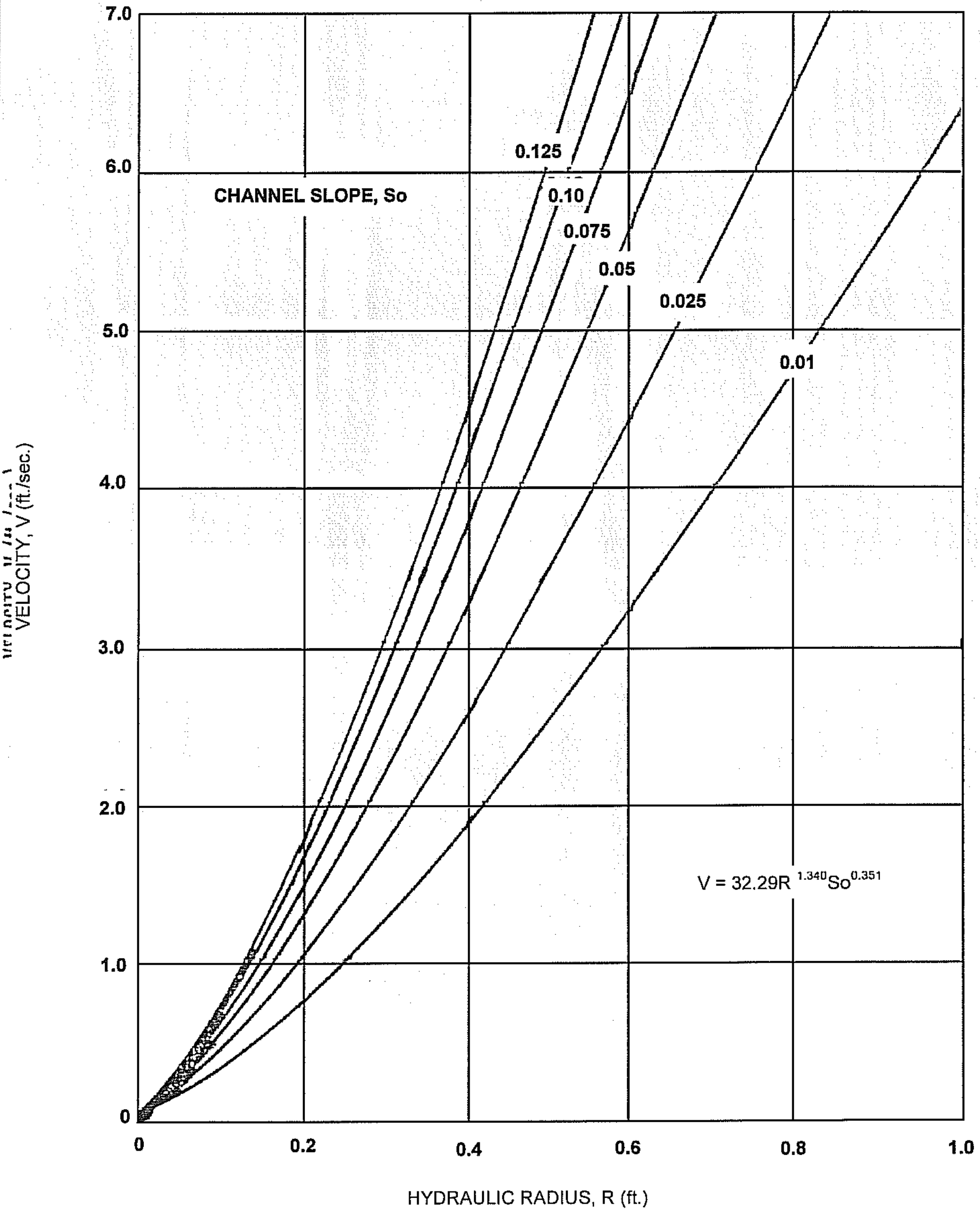
MAXIMUM PERMISSIBLE DEPTH OF FLOW (^{d}max.) FOR CHANNELS LINED WITH EXCELSIOR MAT

SOURCE: AHTD

Figure 9.10



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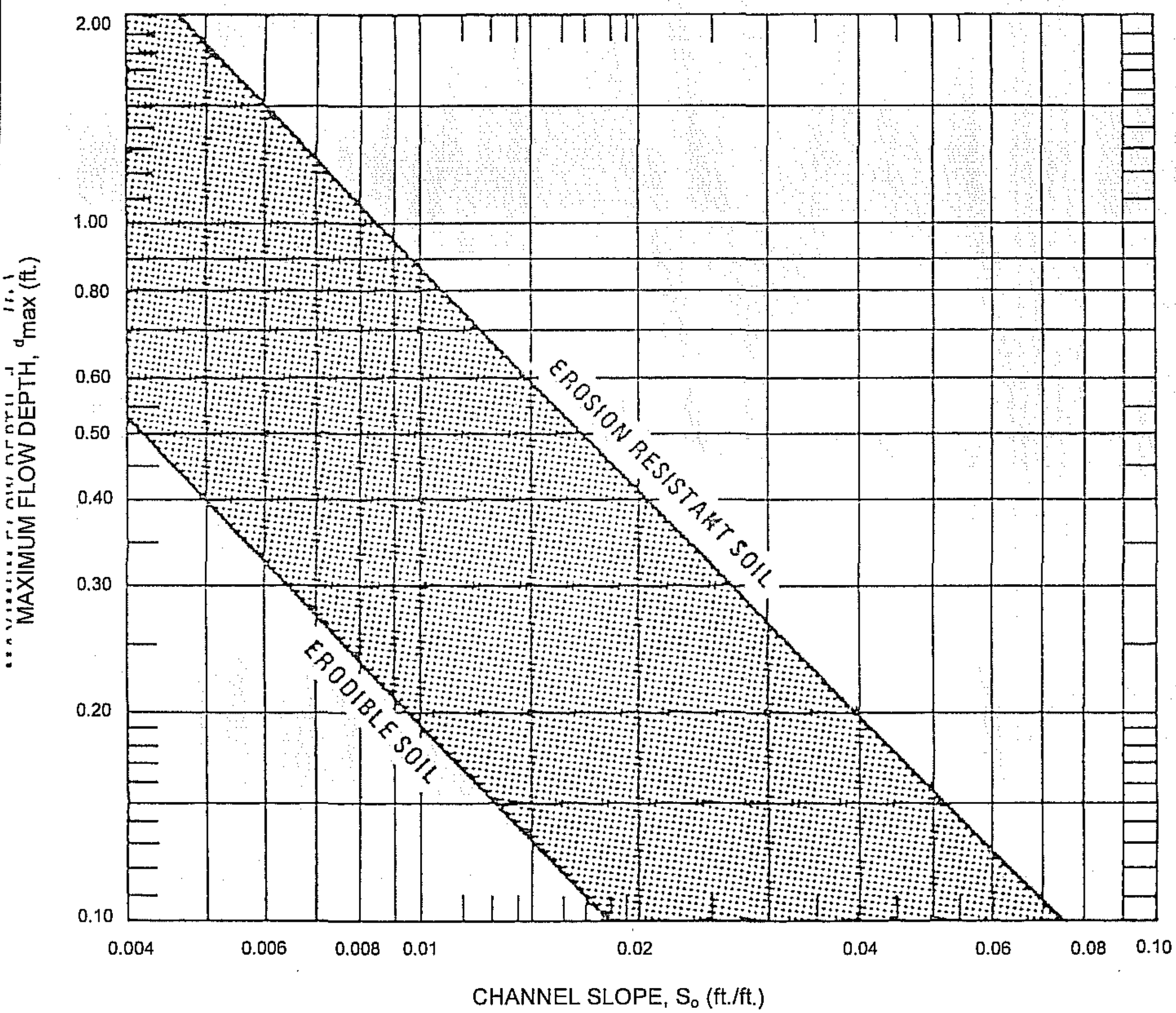
FLOW VELOCITY FOR CHANNELS LINED WITH EXCELSIOR MAT

SOURCE: AHTD

Figure 9.11

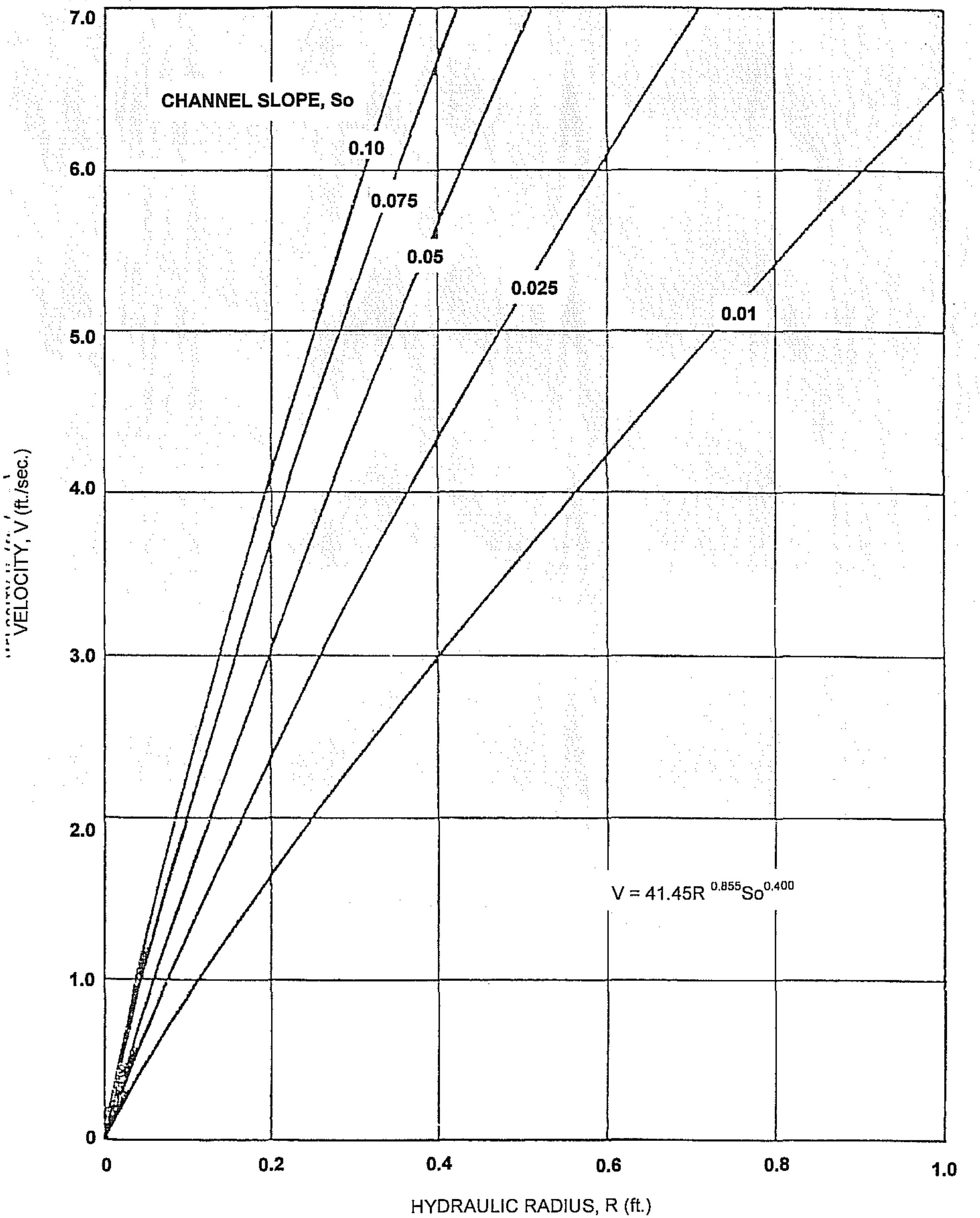


THE CITY OF B BENTONVILLE

MAXIMUM PERMISSIBLE DEPTH OF FLOW (^{d}max.) FOR CHANNELS LINED WITH EROSIONET

SOURCE: AHTD

Figure 9.12

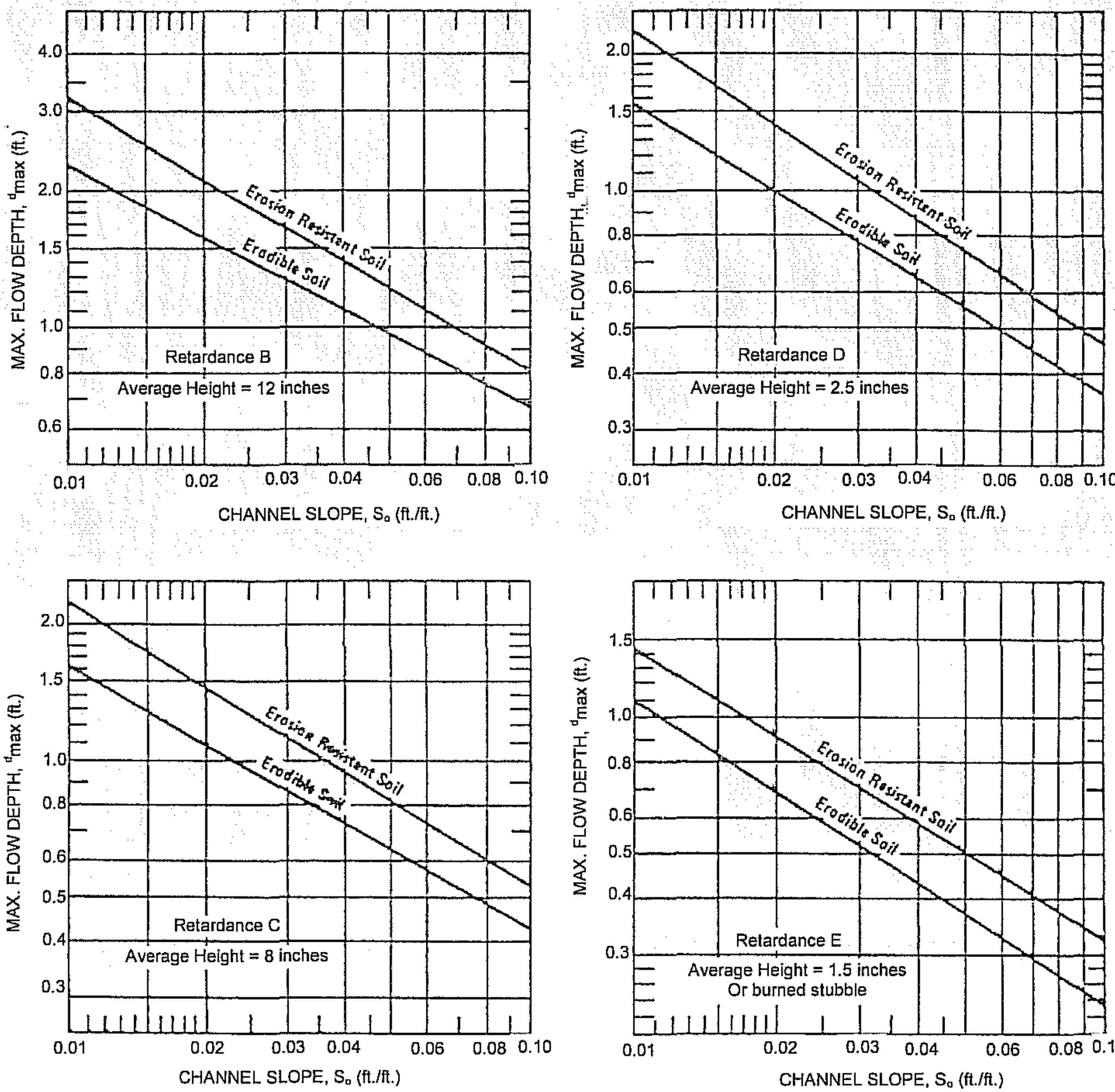


THE CITY OF BENTONVILLE

FLOW VELOCITY FOR CHANNELS LINED WITH EROSIONET

SOURCE: AHTD

Figure 9.13



NOTE: Use on slopes greater than 10% is not recommended.

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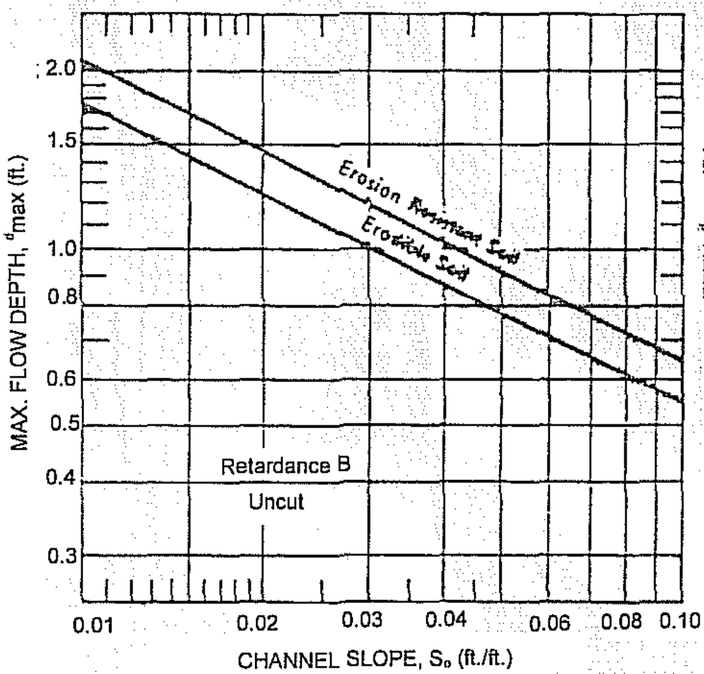
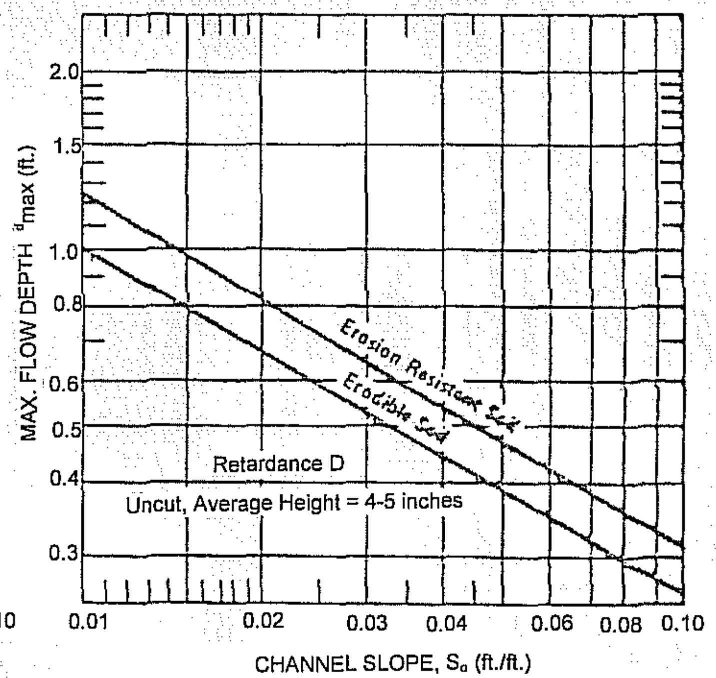
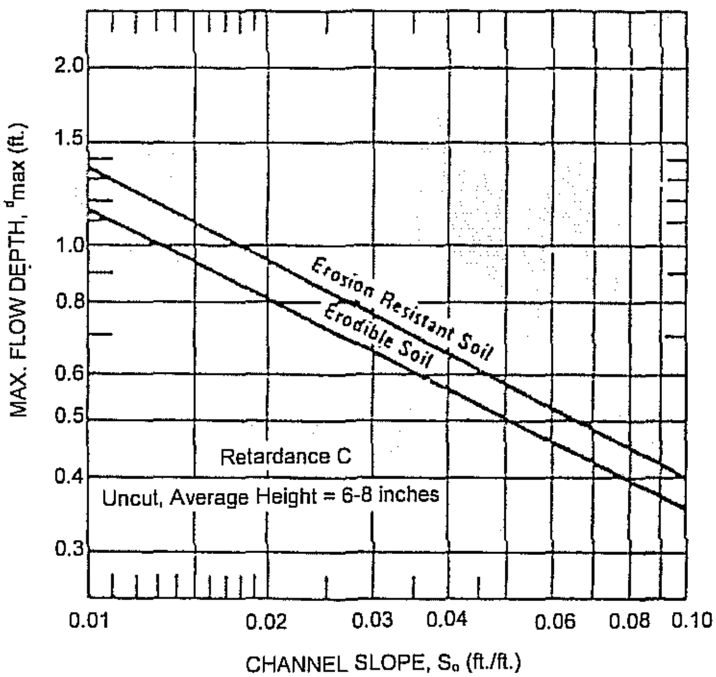
MAXIMUM PERMISSIBLE DEPTH OF FLOW (^{d}max.) FOR CHANNELS LINED WITH BERMUDA GRASS. GOOD STAND, CUT TO VARIOUS LENGTHS

SOURCE: AHTD

Figure 9.14







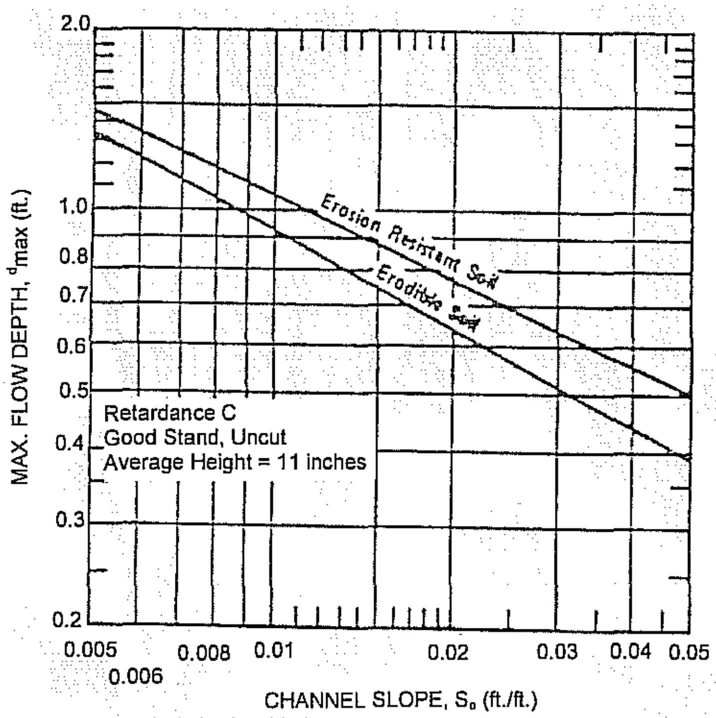
<u>Retardance B:</u> Native-Grass Mixture
Little Bluestem, Blue-Grama, Other
Long and Short Midwest Grasses.
<u>Retardance C:</u> Grass-Legume Mixture
Summer-Orchard Grass, Redtop,
Italian Ryegrass, Common Lespedeza
<u>Retardance D:</u> Grass-Legume Mixture
Fall, Spring – Orchard Grass, Redtop,
Italian Ryegrass, Common Lespedeza

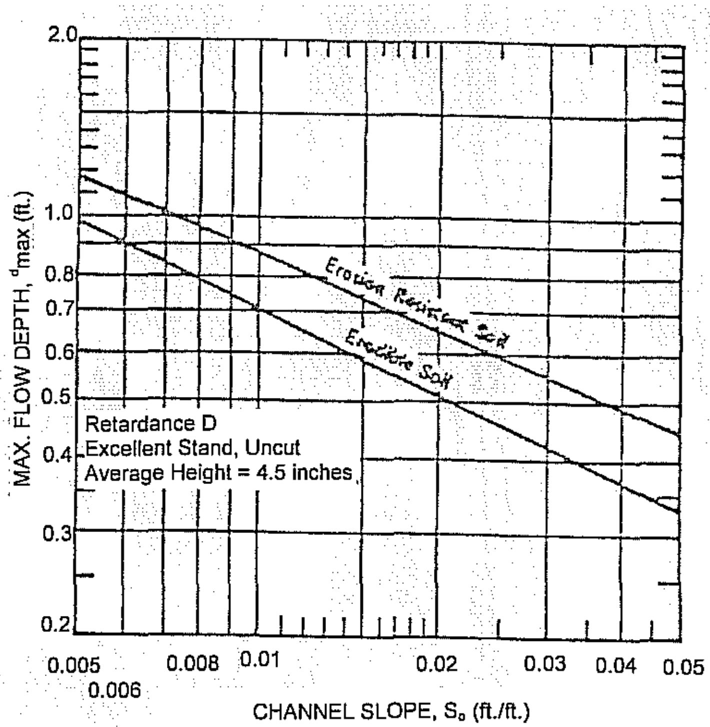
THE CITY OF BENTONVILLE

MAXIMUM PERMISSIBLE DEPTH OF FLOW (^{d}max.) FOR CHANNELS LINED WITH GRASS MIXTURES GOOD STAND, UNCUT

SOURCE: AHTD

Figure 9.15



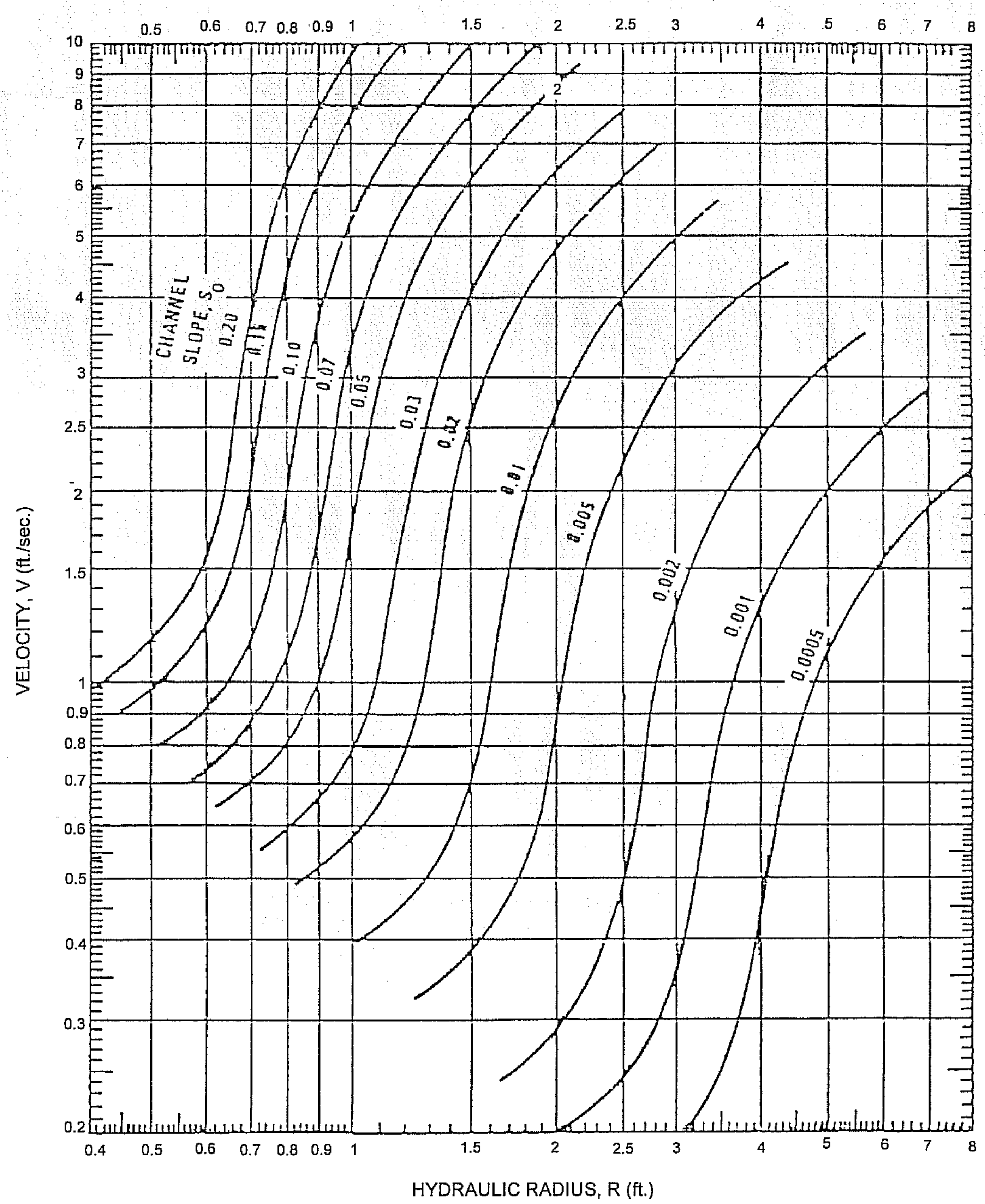


THE CITY OF BENTONVILLE

MAXIMUM PERMISSIBLE DEPTH OF FLOW (^{d}max.) FOR CHANNELS LINED WITH COMMON LESPEDEZA OF VARIOUS LENGTHS

SOURCE: AHTD

Figure 9.16

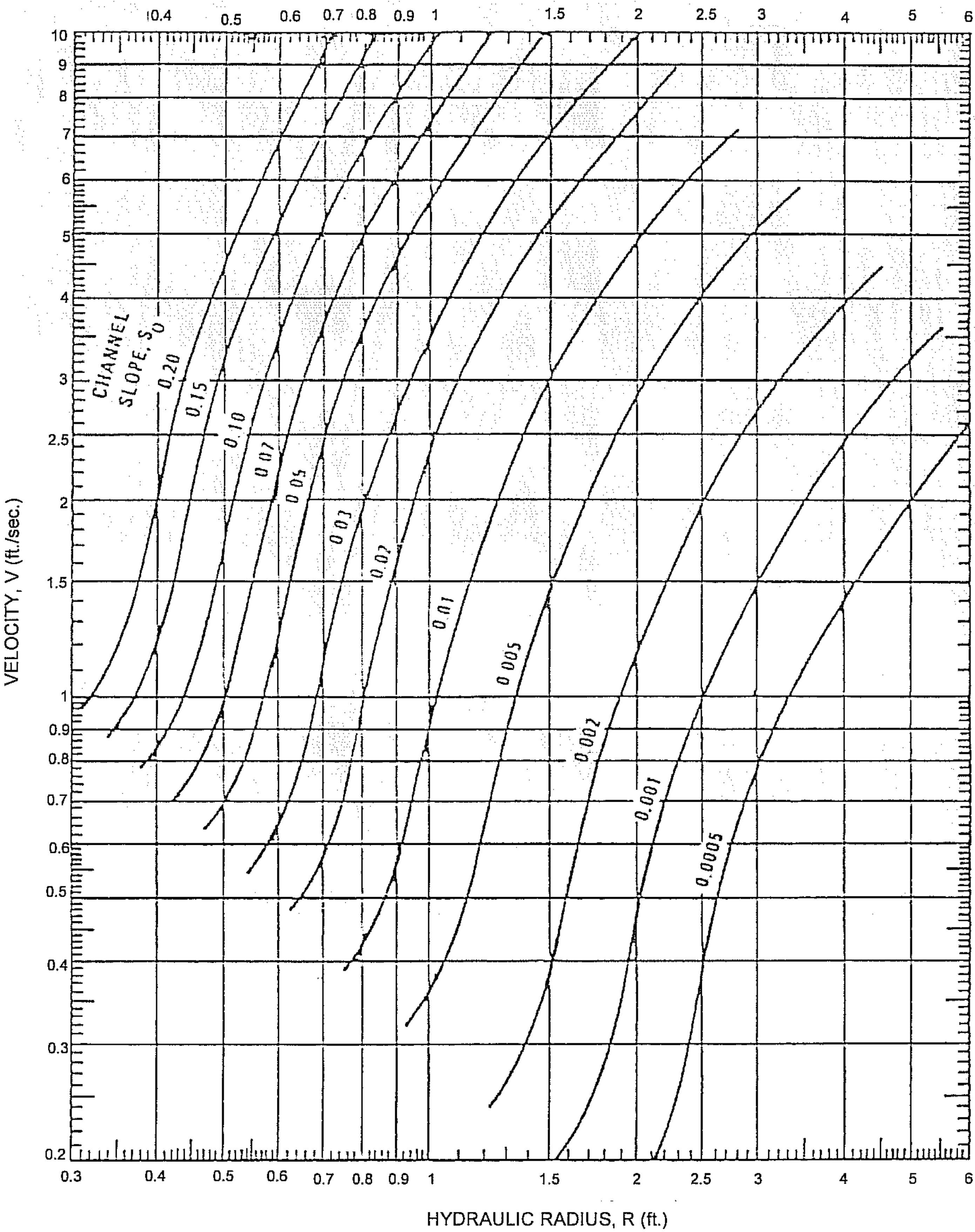


THE CITY OF BENTONVILLE

FLOW VELOCITY FOR CHANNELS LINED WITH VEGETATION OF RETARDANCE A

SOURCE: AHTD

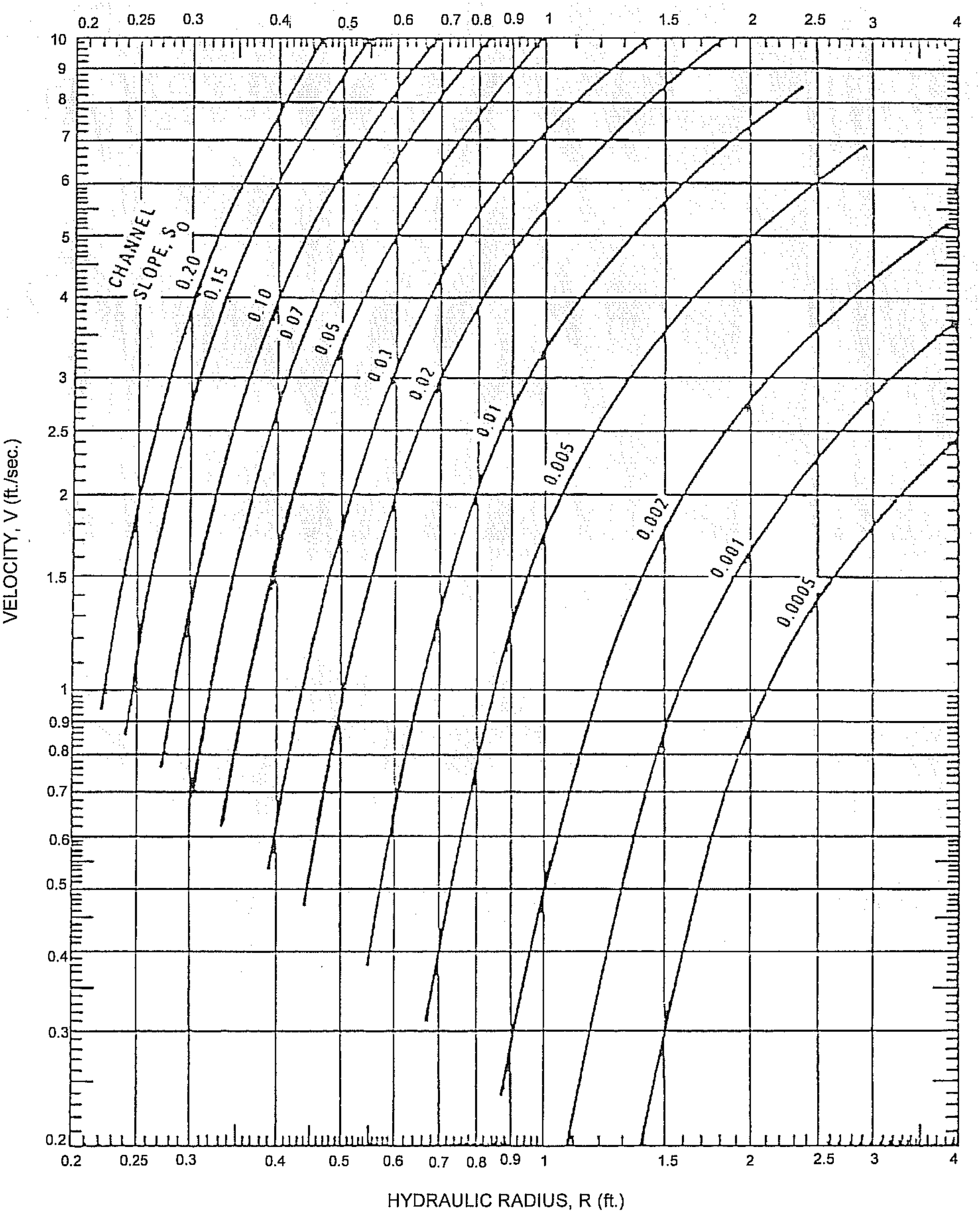
Figure 9.17



FLOW VELOCITY FOR CHANNELS LINED WITH VEGETATION OF RETARDANCE B

SOURCE: AHTD

Figure 9.18

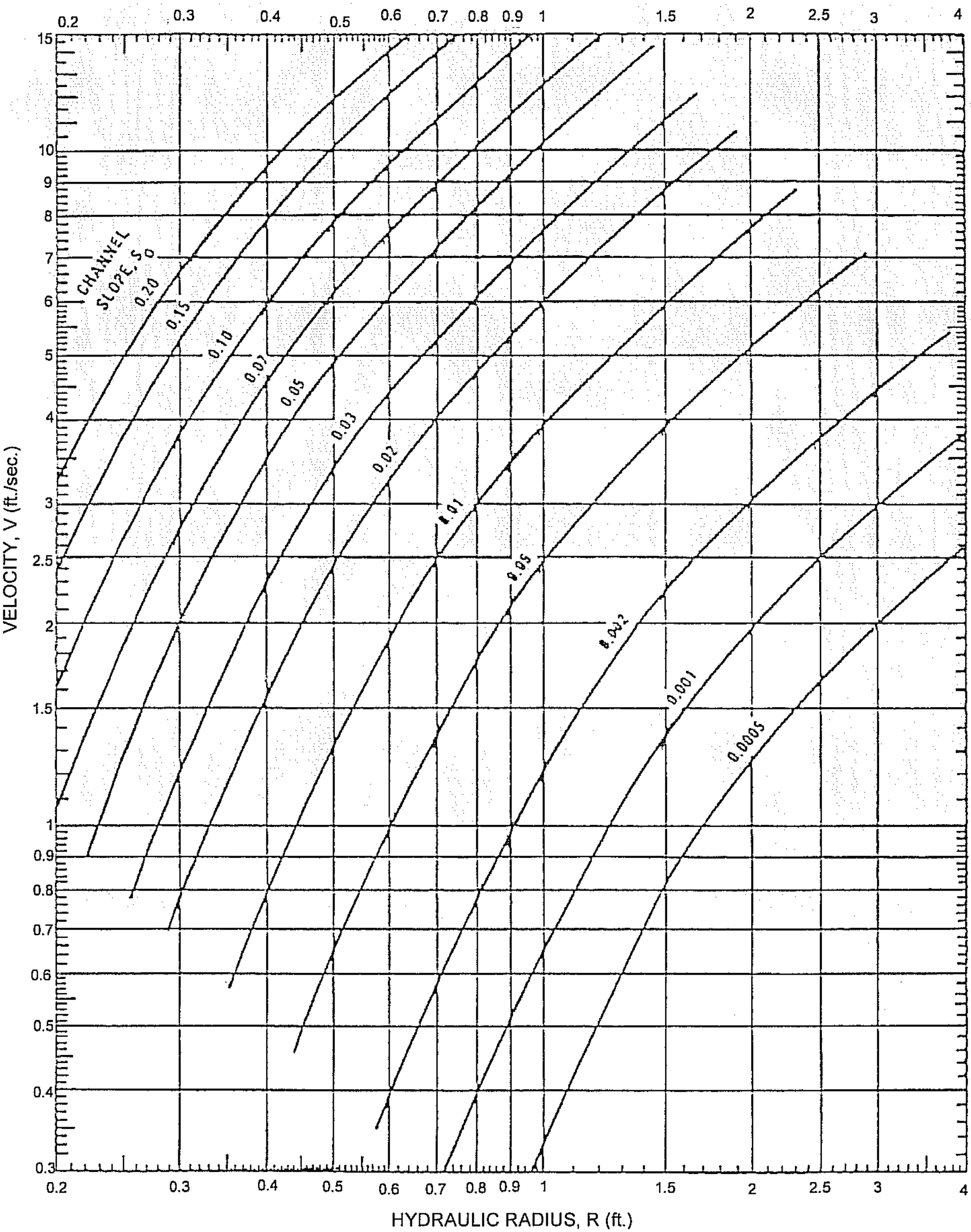


THE CITY OF BENTONVILLE

FLOW VELOCITY FOR CHANNELS LINED WITH VEGETATION OF RETARDANCE C

SOURCE: AHTD

Figure 9.19

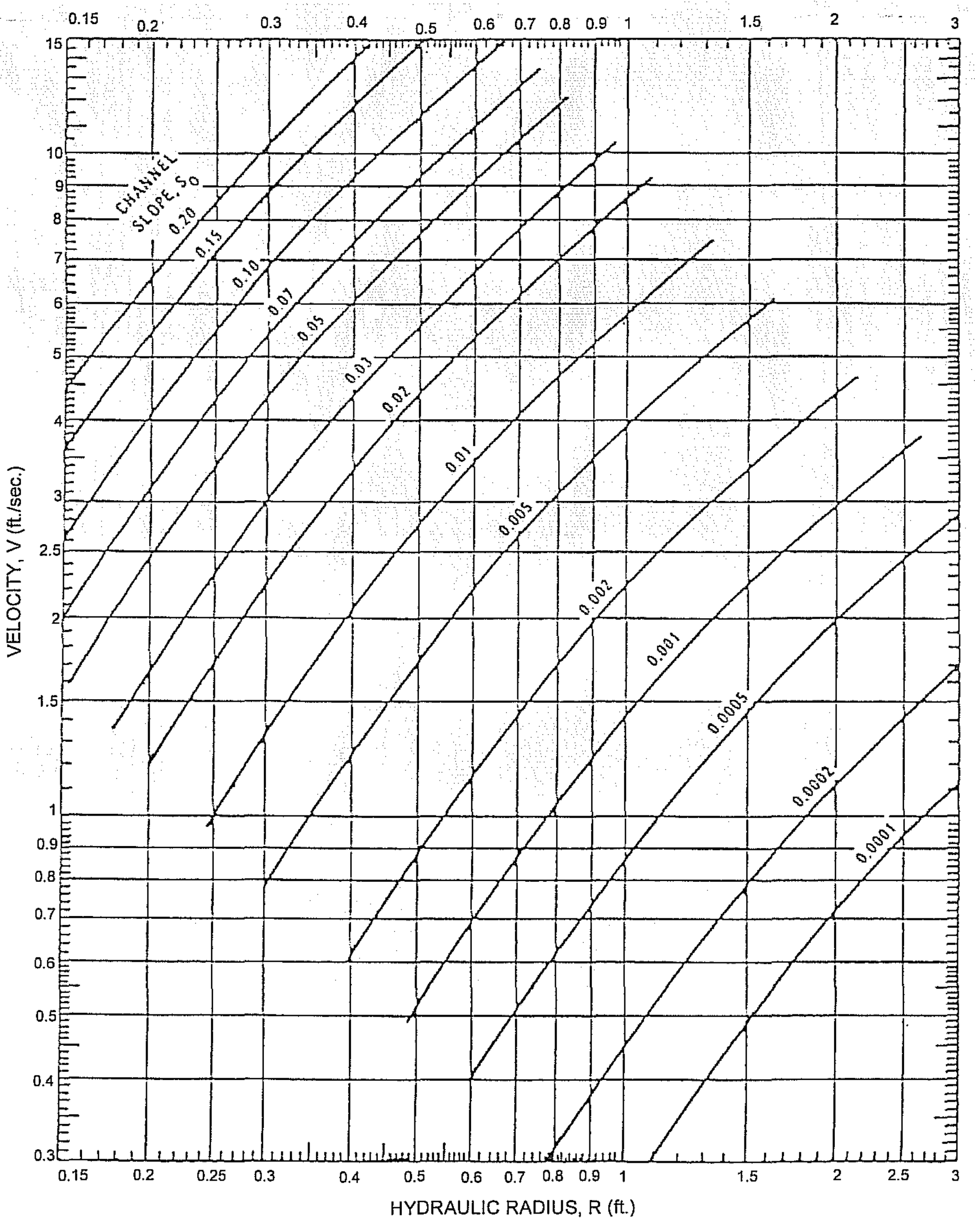


THE CITY OF BENTONVILLE

FLOW VELOCITY FOR CHANNELS LINED WITH VEGETATION OF RETARDANCE D

SOURCE: AHTD

Figure 9.20

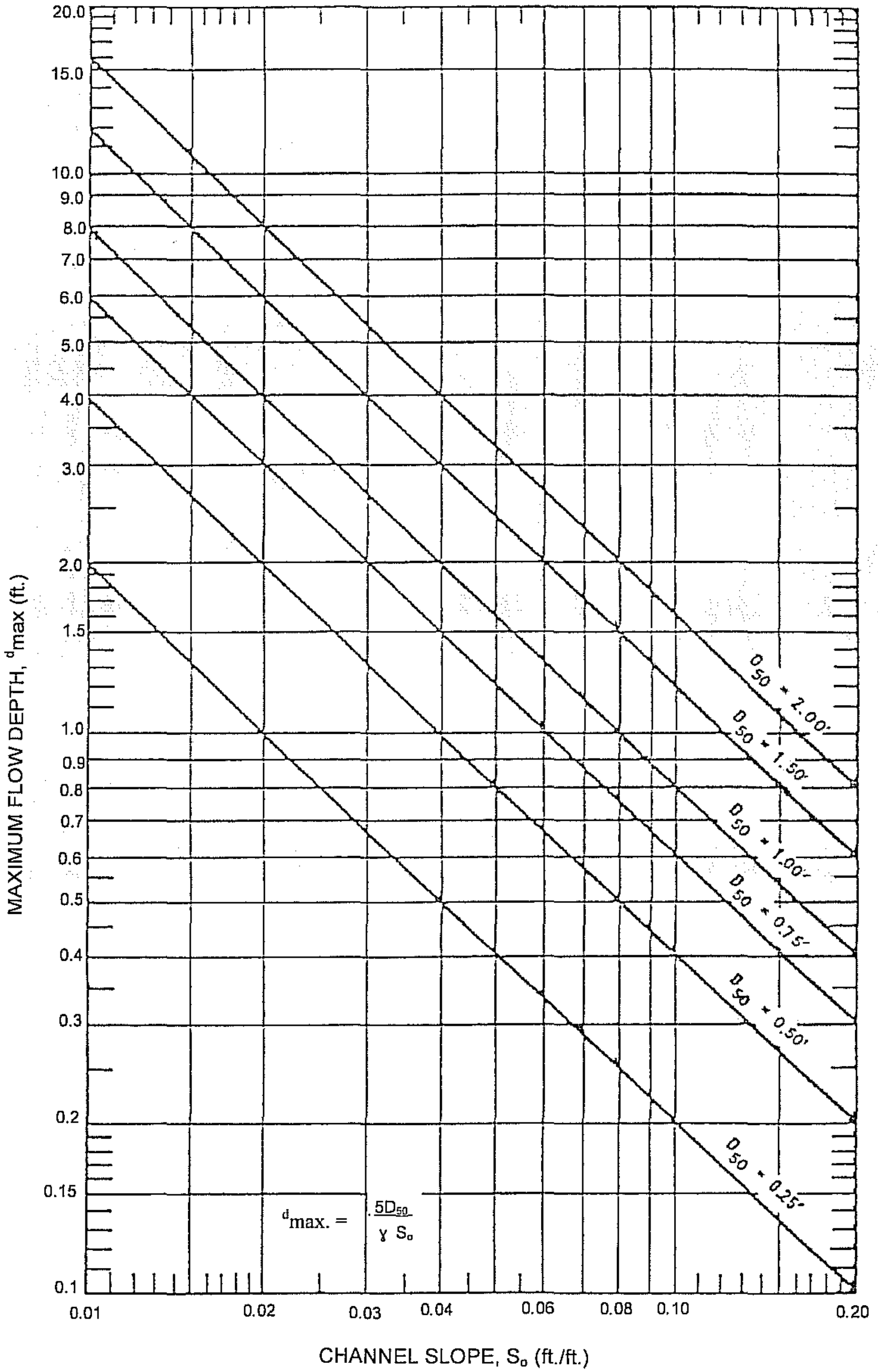


THE CITY OF BENTONVILLE

FLOW VELOCITY FOR CHANNELS LINED WITH VEGETATION OF RETARDANCE E

SOURCE: AHTD

Figure 9.21

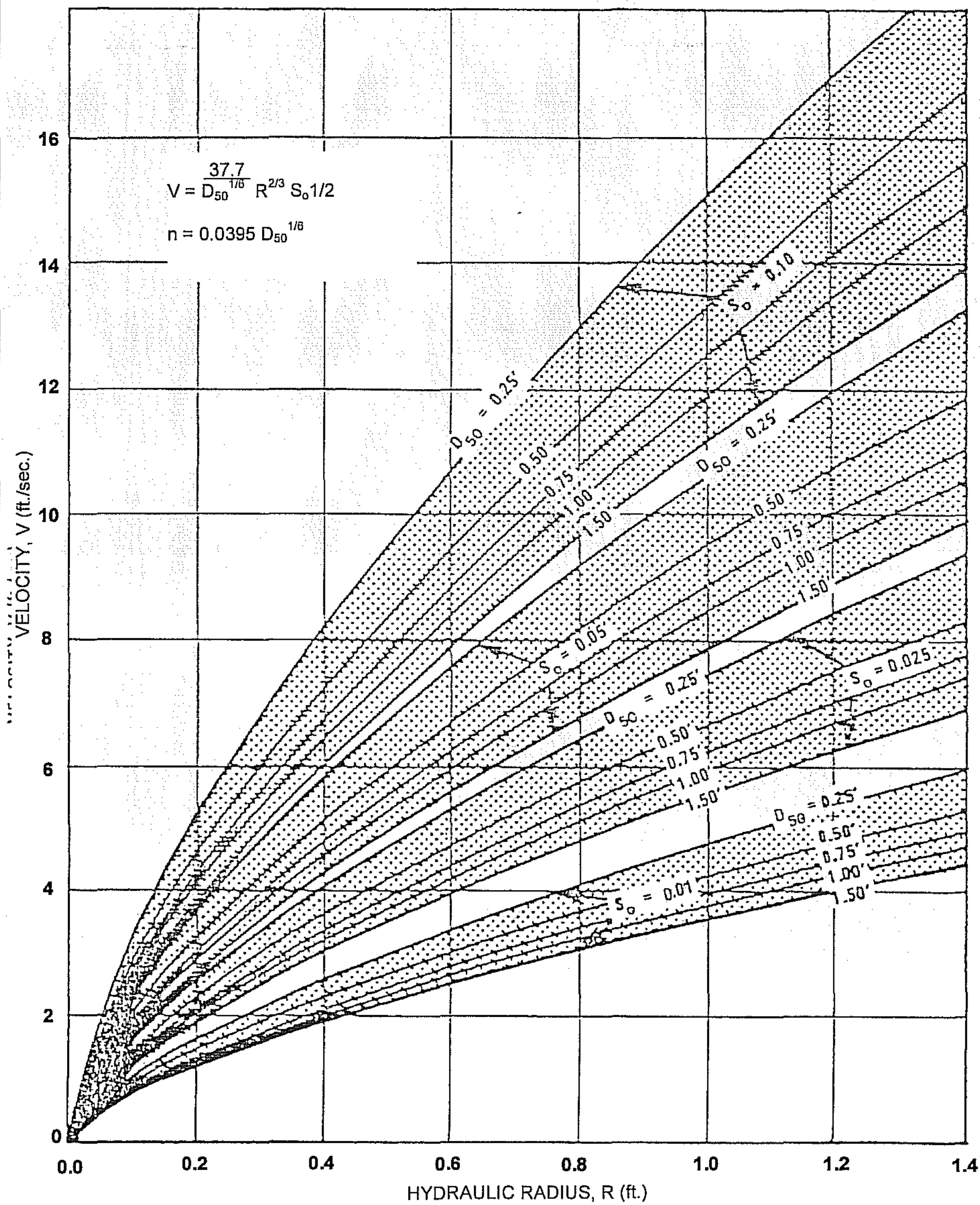


THE CITY OF BENTONVILLE

MAXIMUM PERMISSIBLE DEPTH OF FLOW (^{d}max.) FOR CHANNELS LINED WITH ROCK RIPRAP

SOURCE: AHTD

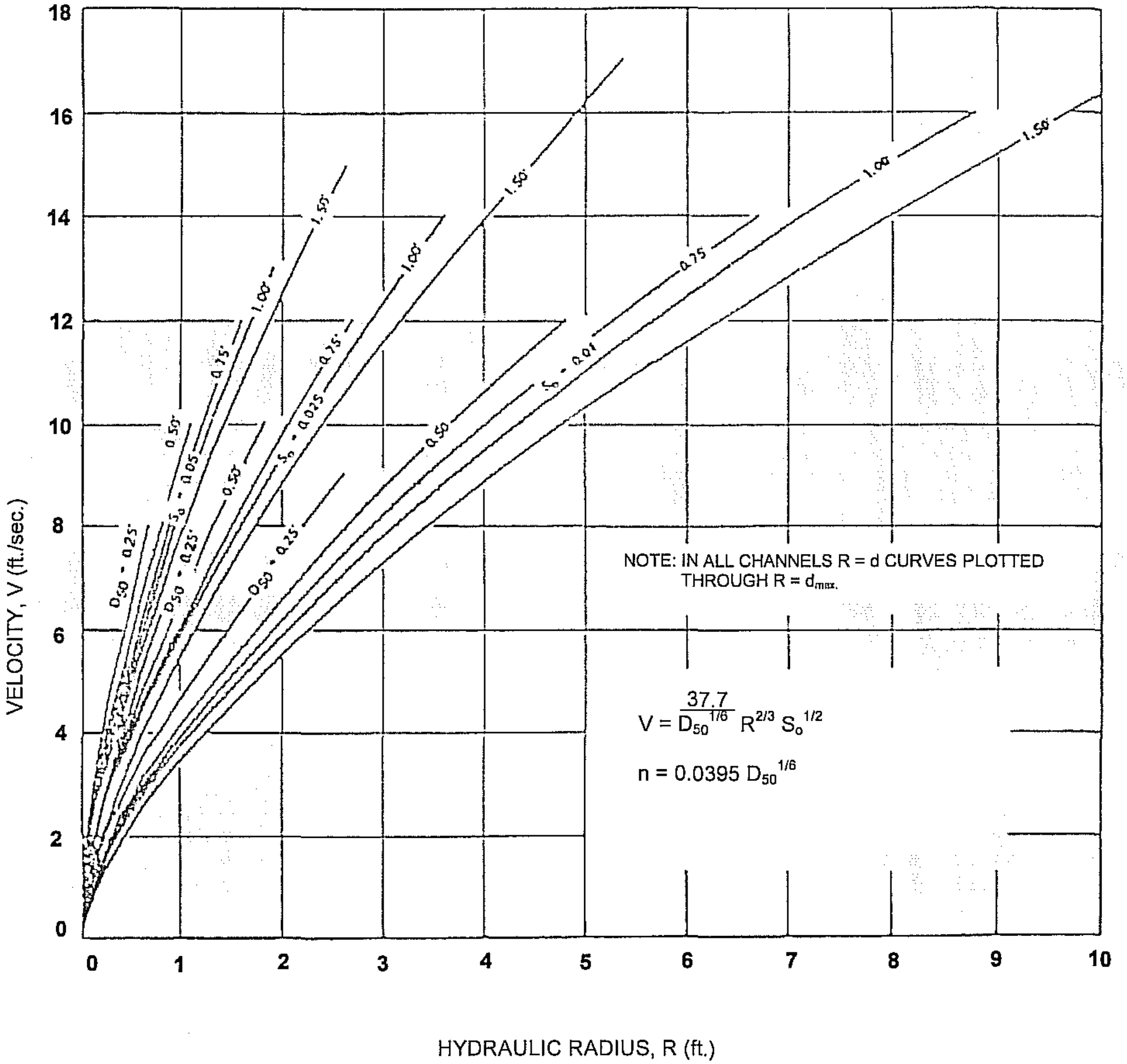
Figure 9.22



FLOW VELOCITY FOR CHANNELS LINED WITH ROCK RIPRAP
SLOPES = 0.01 TO 0.10, D_{50} = 0.25' TO 1.50'

SOURCE: AHTD

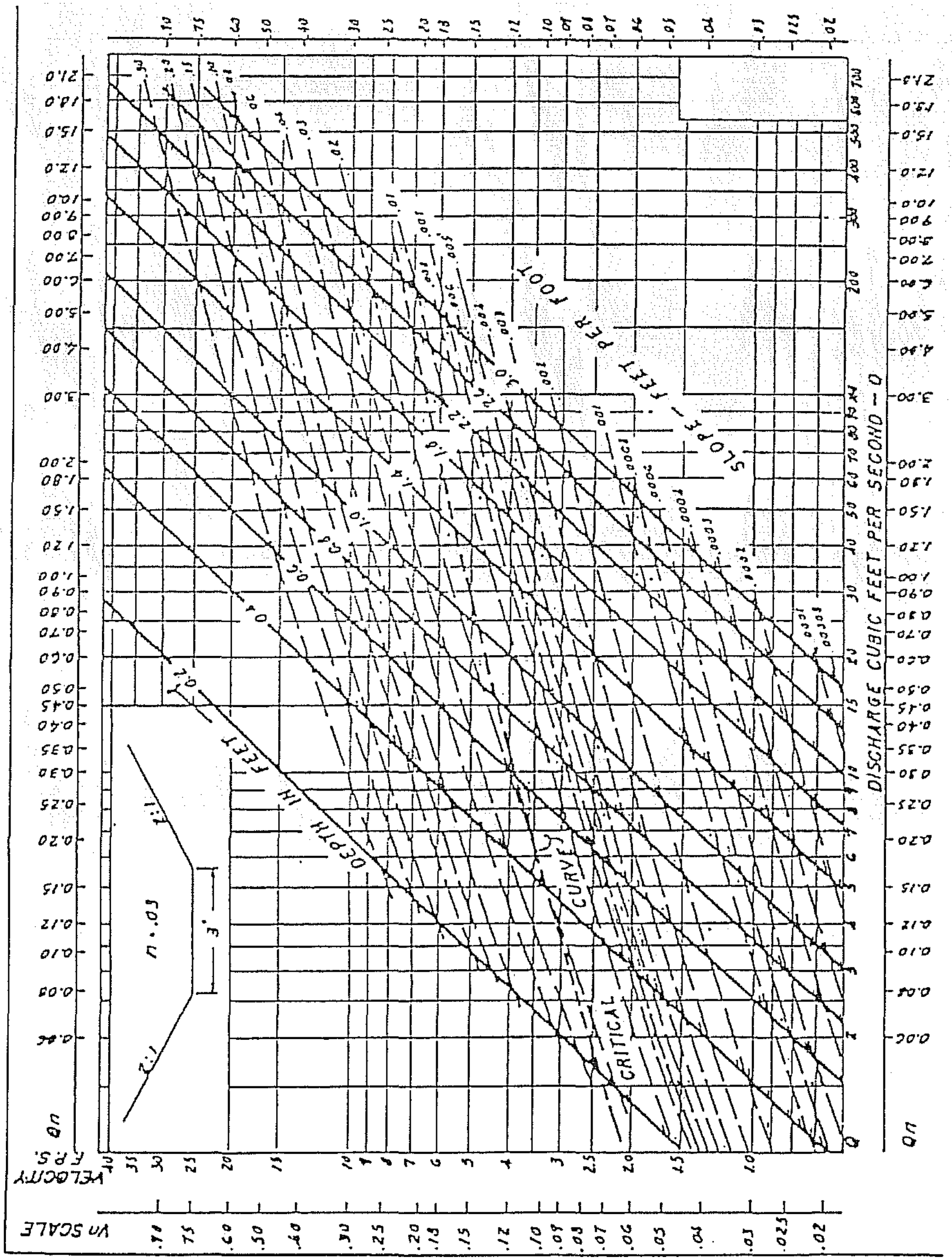
Figure 9.23



FLOW VELOCITY FOR CHANNELS LINED WITH ROCK RIPRAP
SLOPES = 0.01 TO 0.05, D_{50} = 0.25' TO 1.50'

SOURCE: AHTD

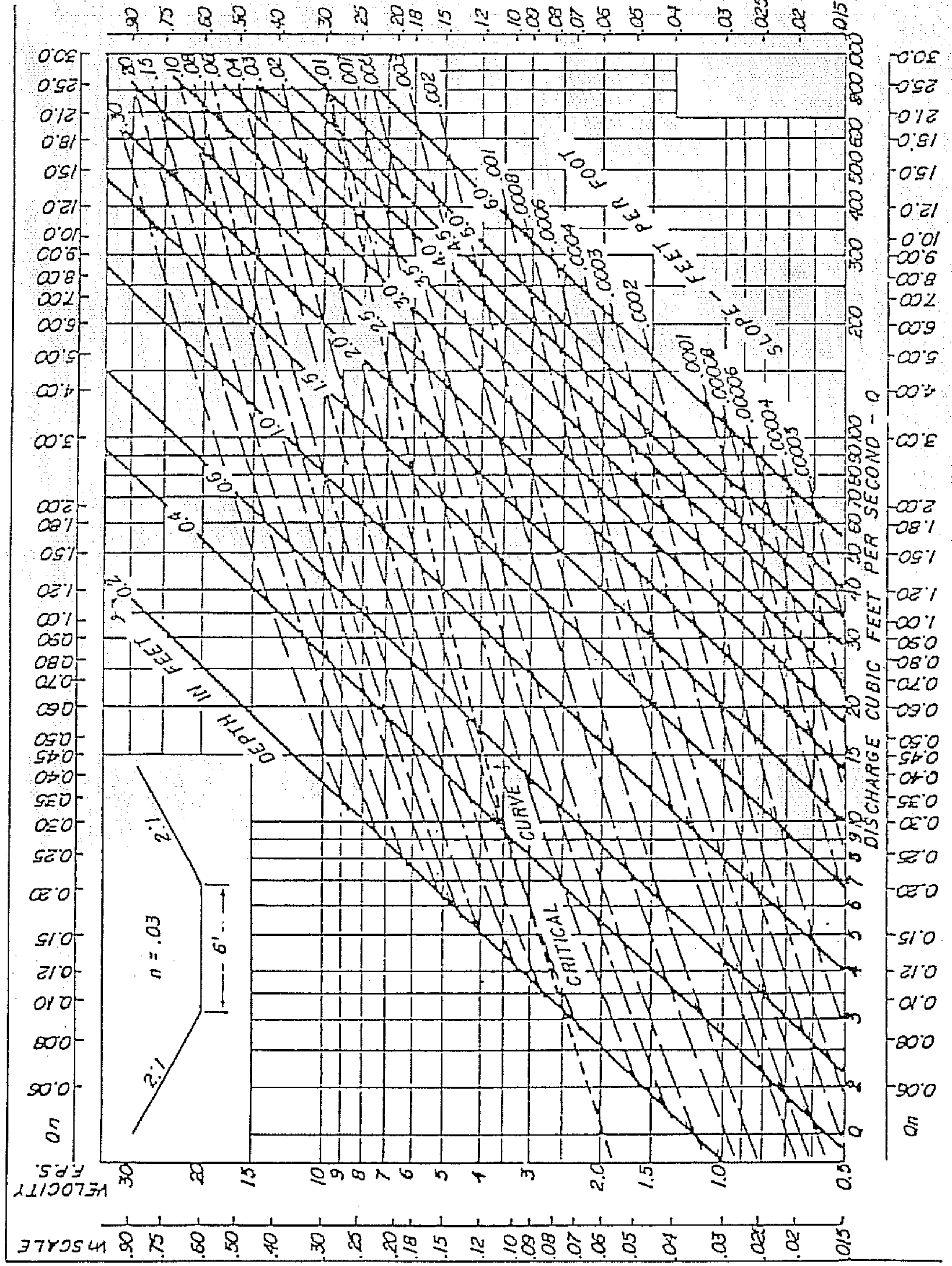
Figure 9.24



CHANNEL CHART
2:1 b = 3 Ft.

Source: AHTD

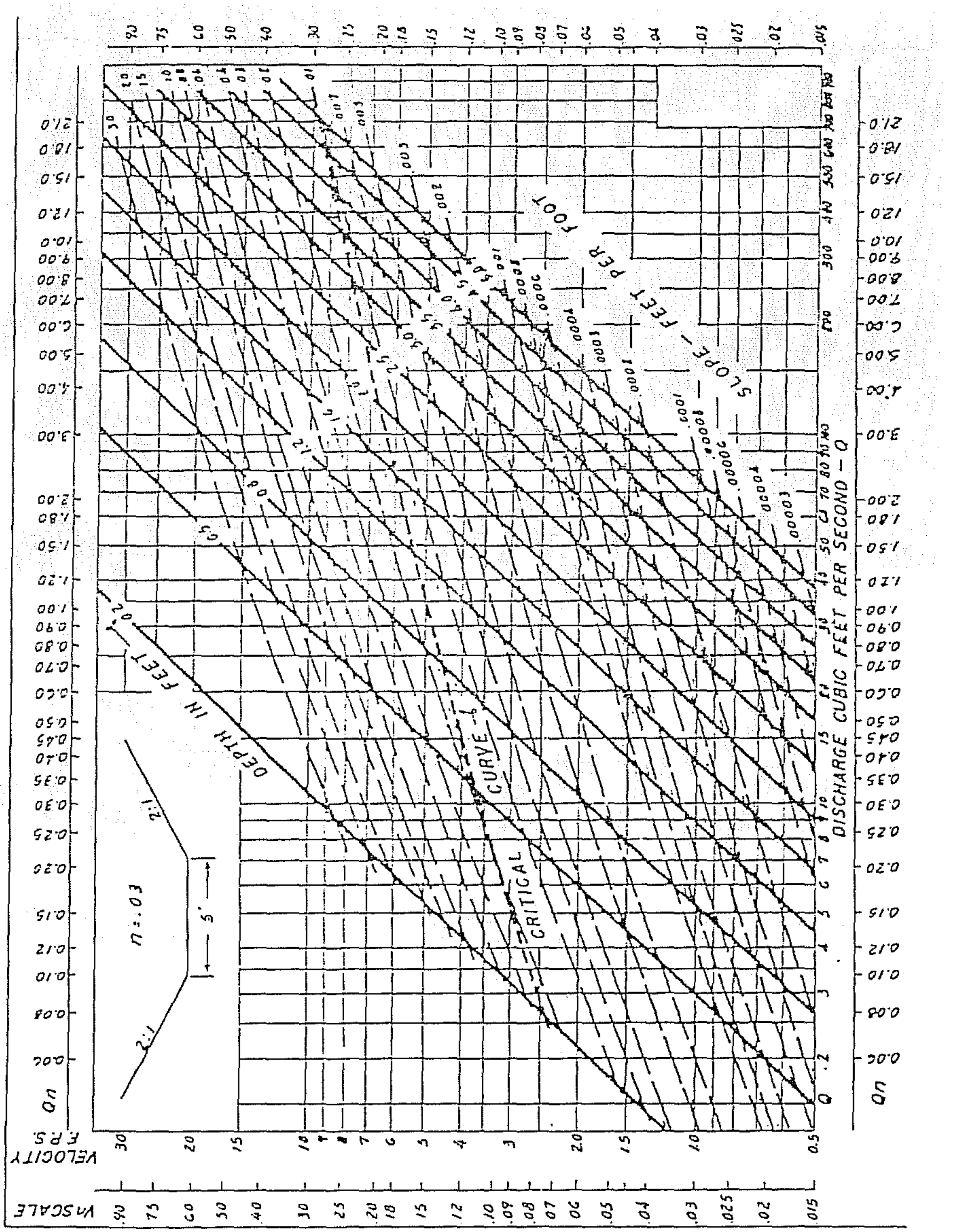
Figure 9.25



CHANNEL CHART
2:1 b = 6 Ft.

Source: AHTD

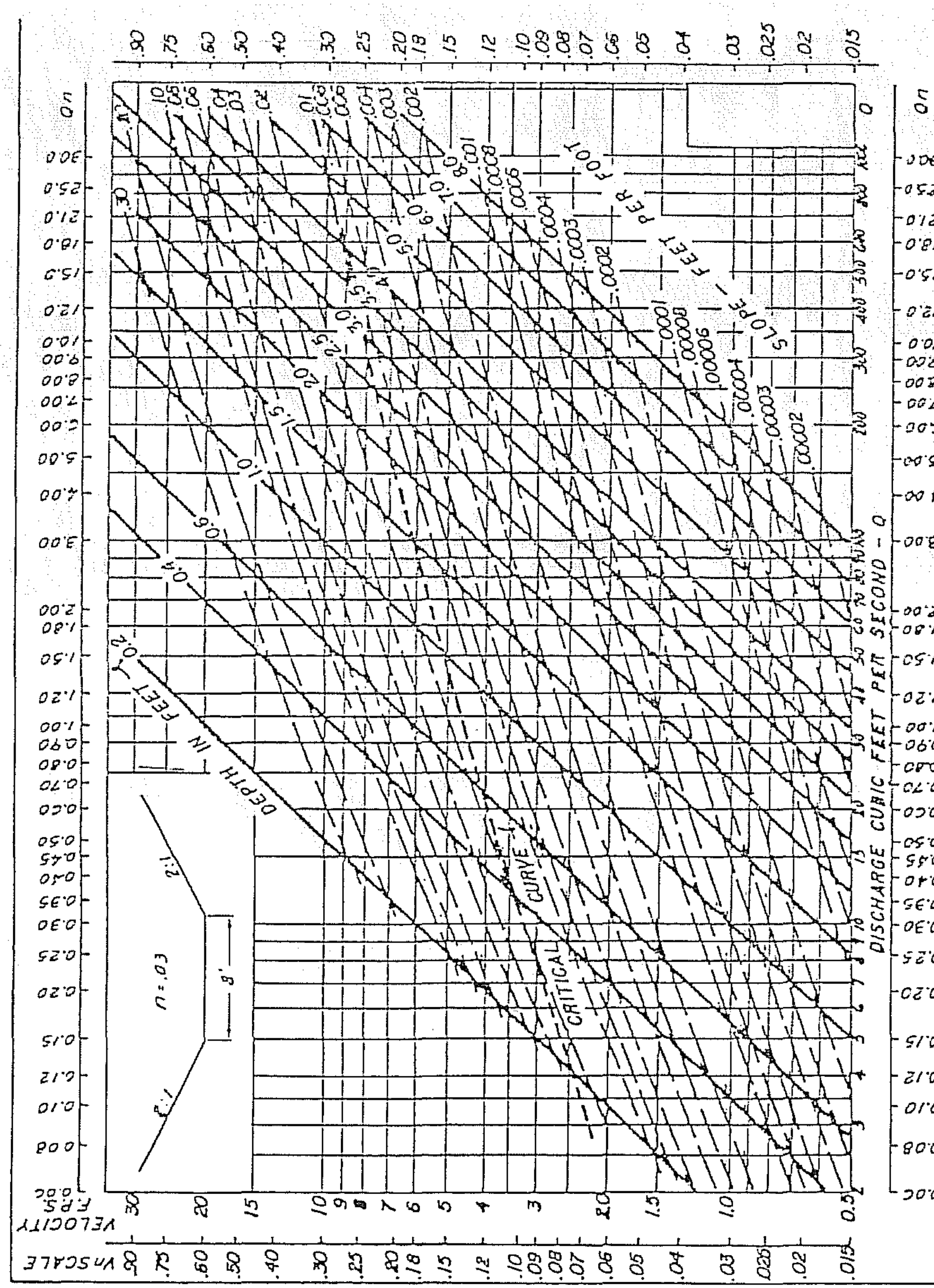
Figure 9.26



CHANNEL CHART
2:1 b = 5 Ft.

Source: AHTD

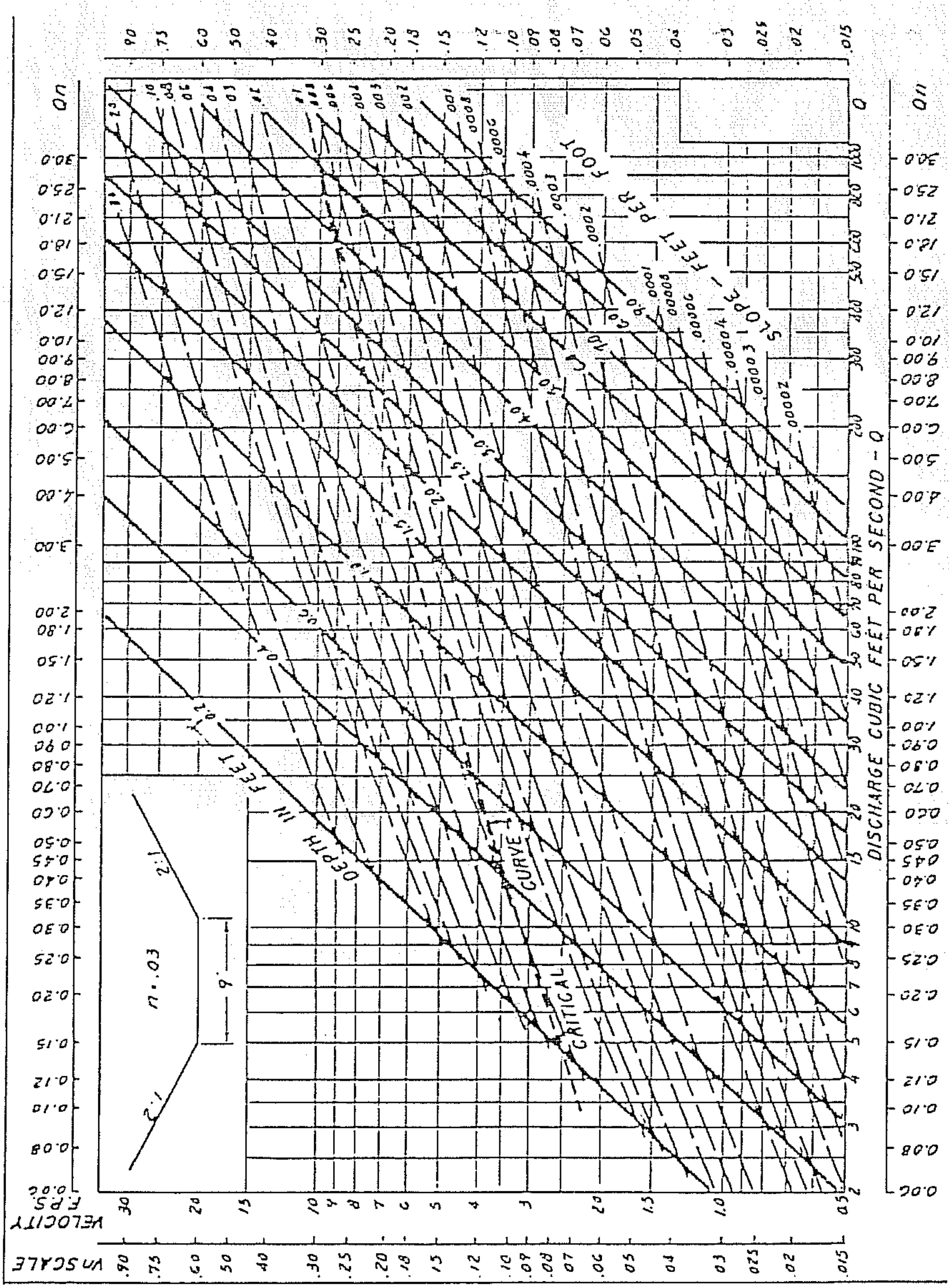
Figure 9.27



CHANNEL CHART
2:1 b = 8 Ft.

Source: AHTD

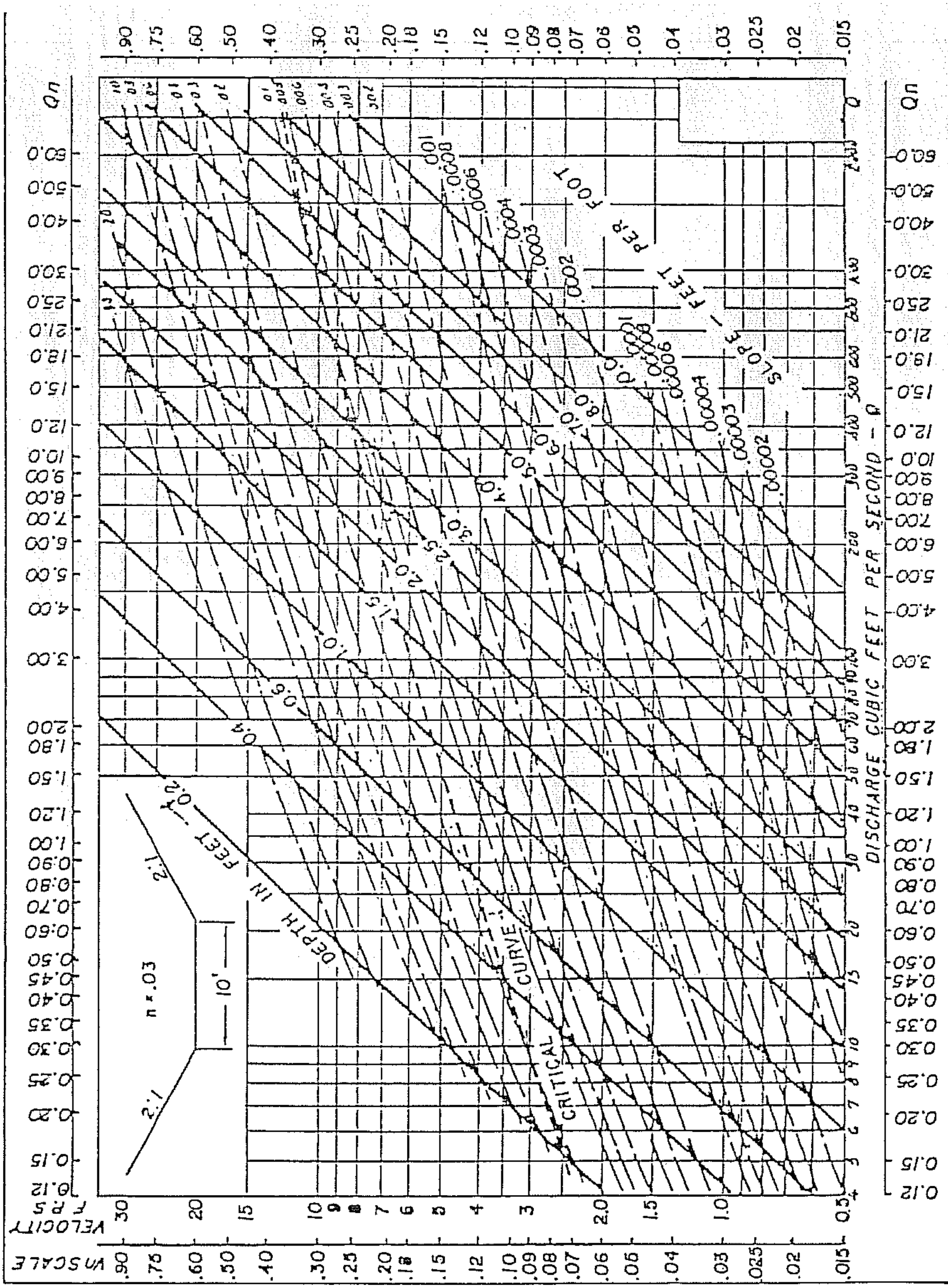
Figure 9.29



CHANNEL CHART
2:1 b = 9 Ft.

Source: AHTD

Figure 9.30

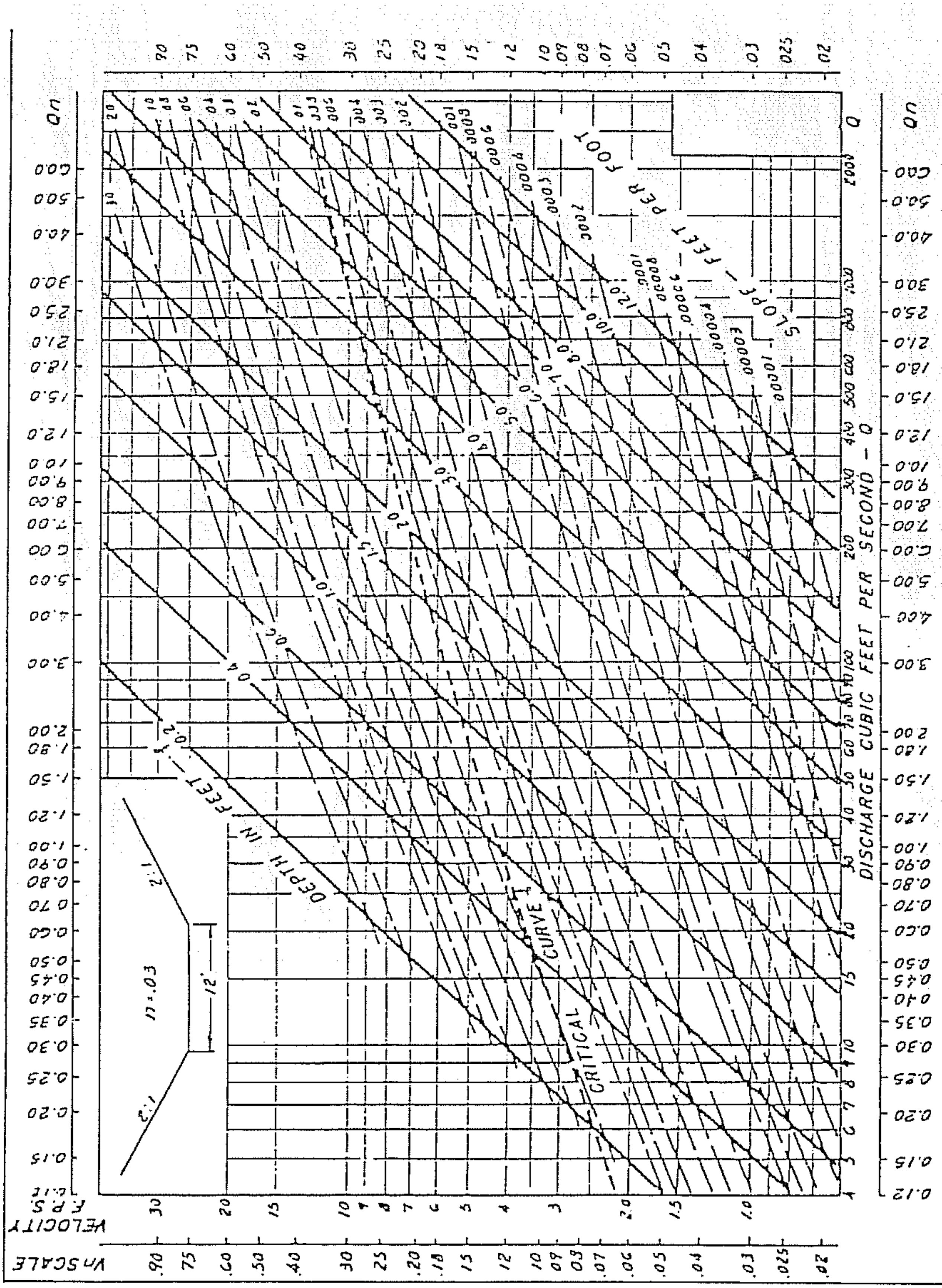


THE CITY OF B BENTONVILLE

CHANNEL CHART
2:1 b = 10 Ft.

Source: AHTD

Figure 9.31

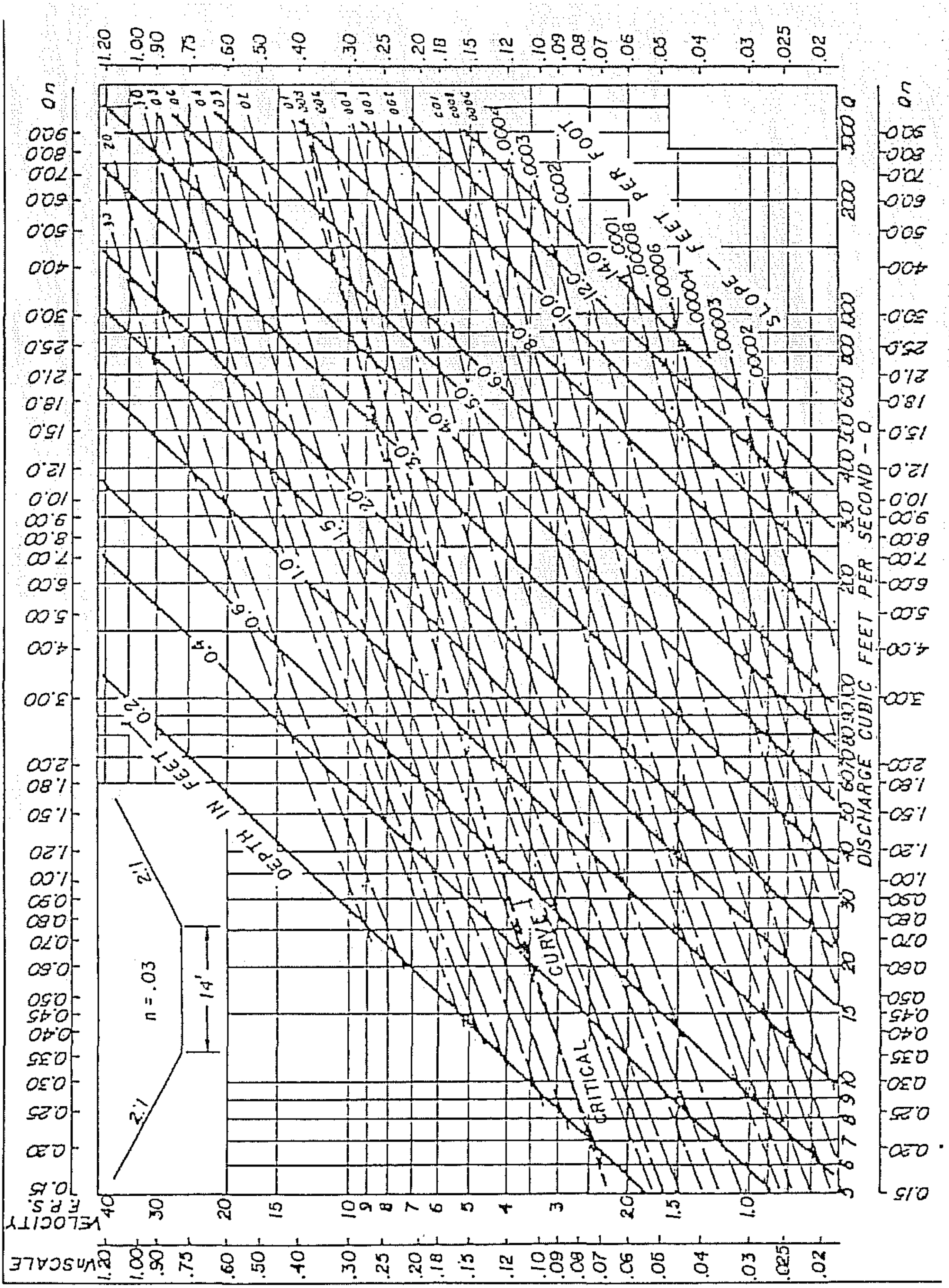


THE CITY OF BENTONVILLE

CHANNEL CHART
2:1 b = 12 Ft.

Source: AHTD

Figure 9.32

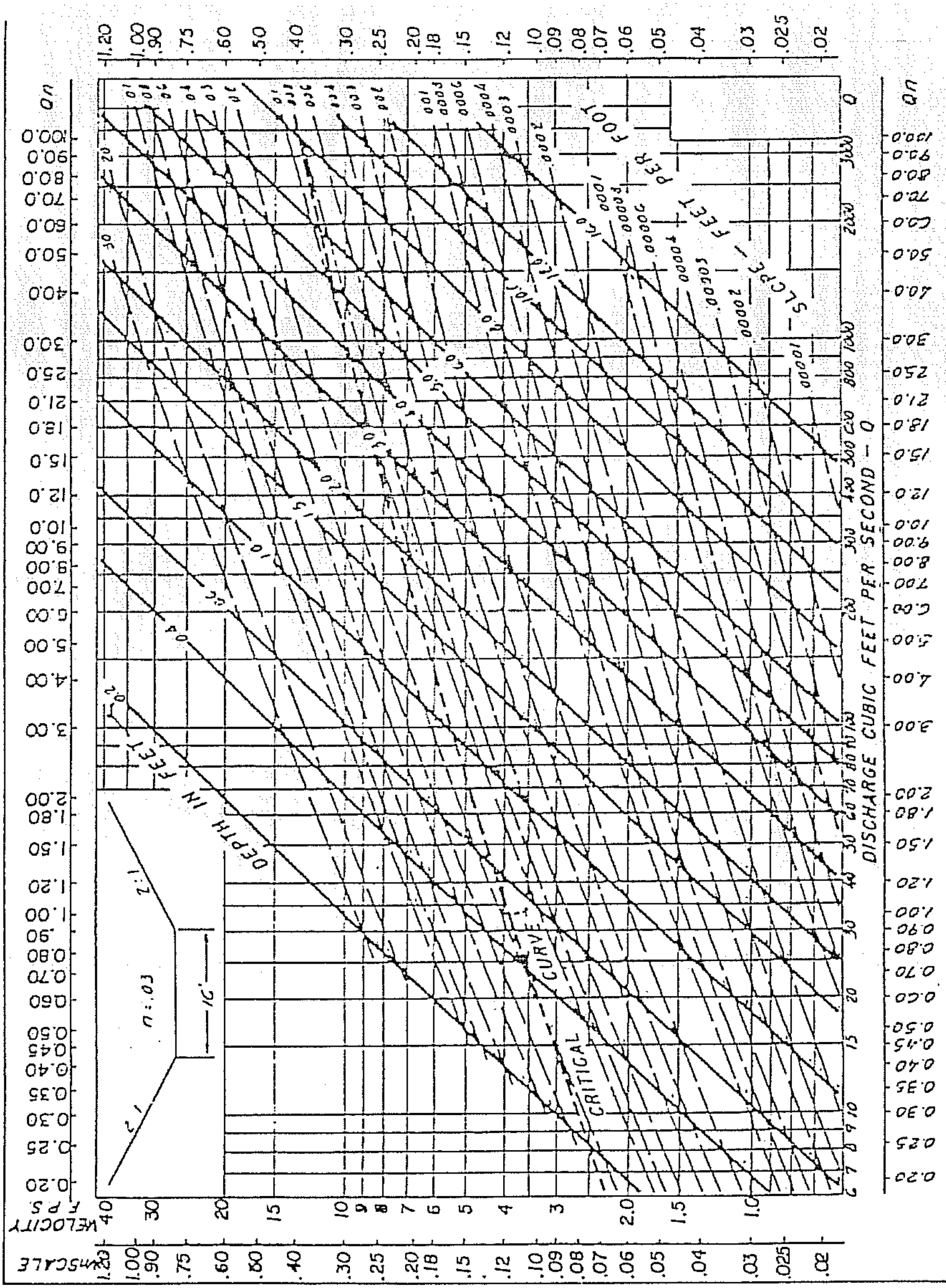


THE CITY OF BENTONVILLE

CHANNEL CHART
2:1 b = 14 Ft.

Source: AHTD

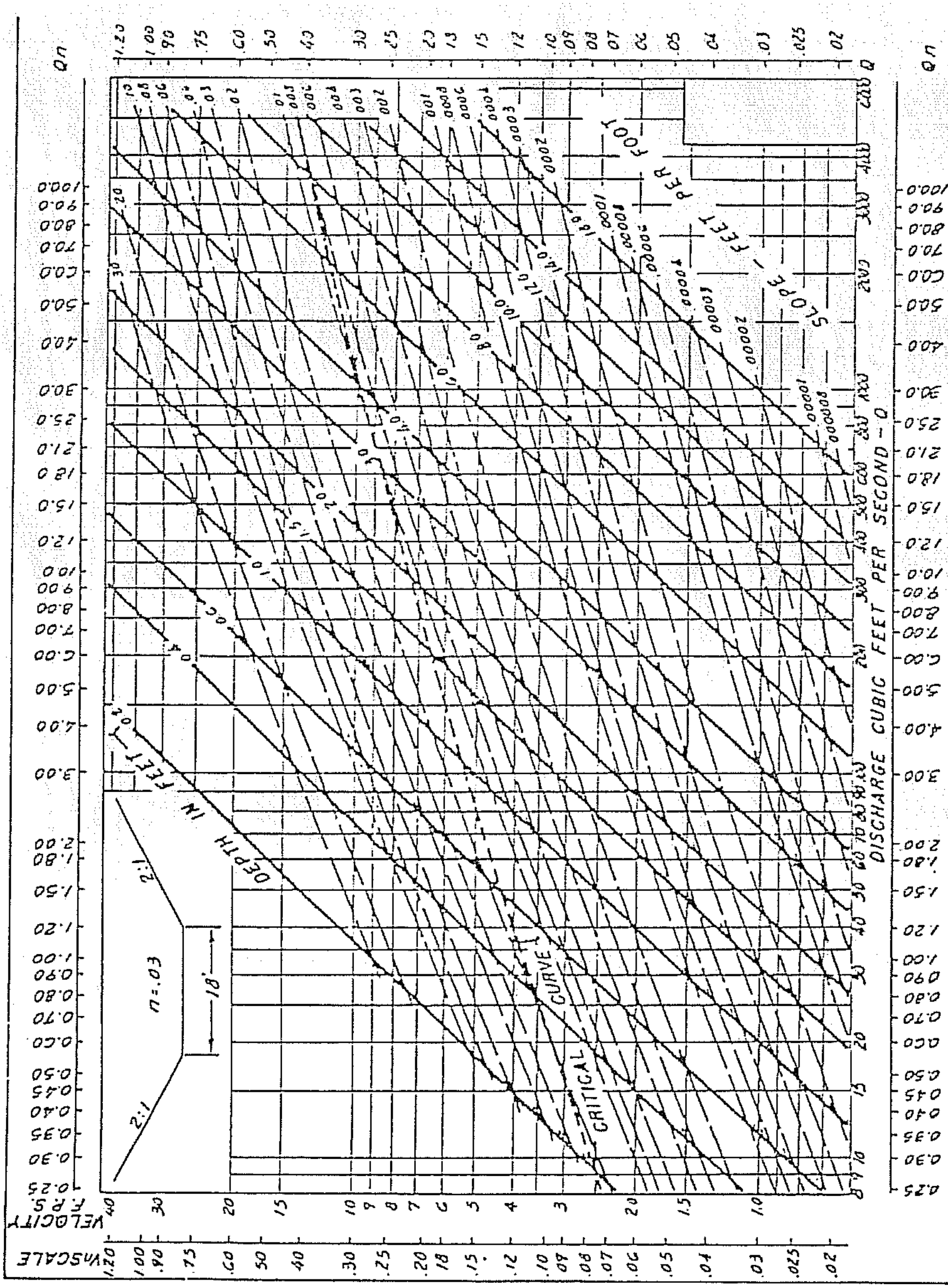
Figure 9.33



CHANNEL CHART
2:1 b = 16 Ft.

Source: AHTD

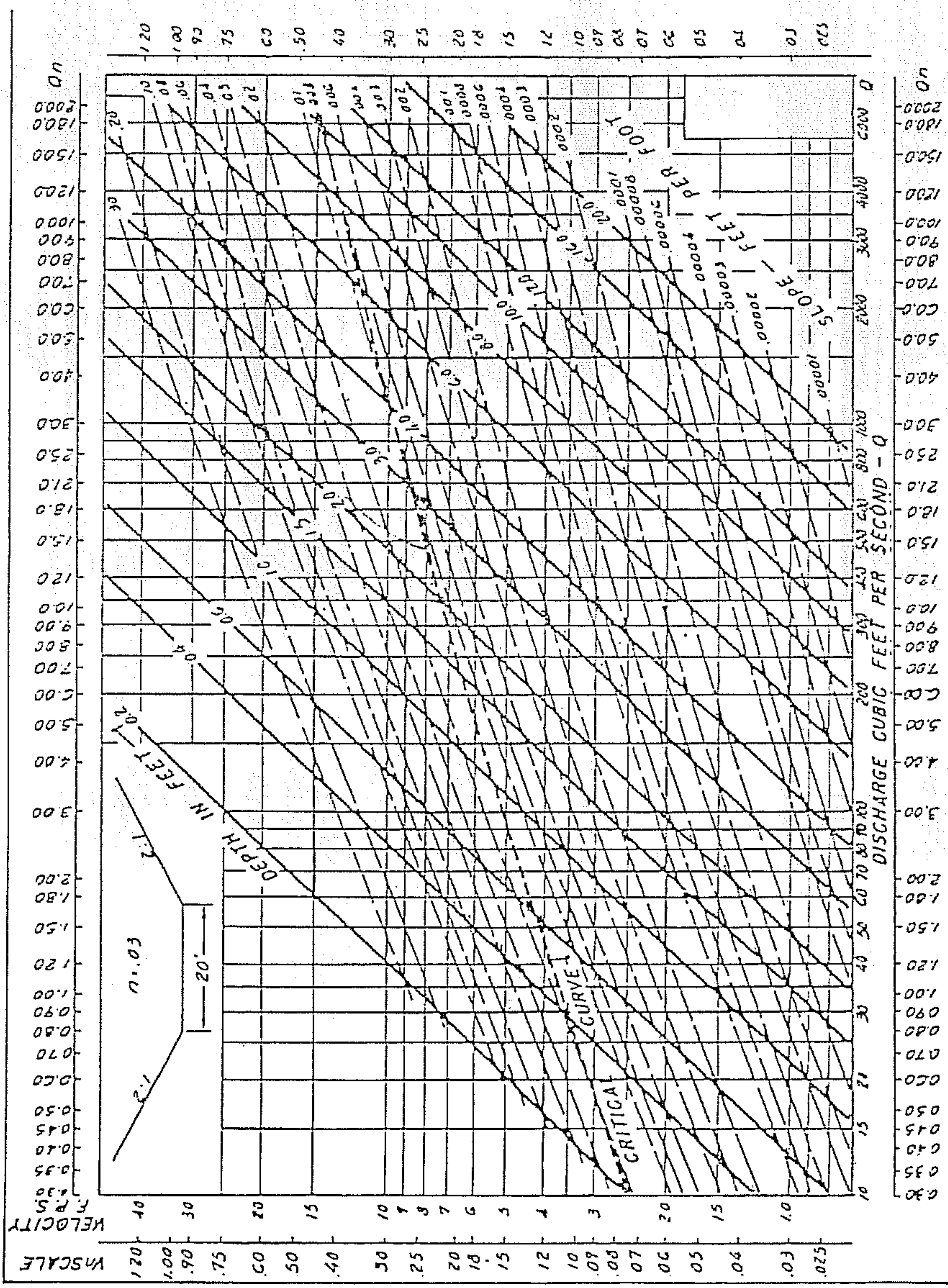
Figure 9.34



CHANNEL CHART
2:1 b = 18 Ft.

Source: AHTD

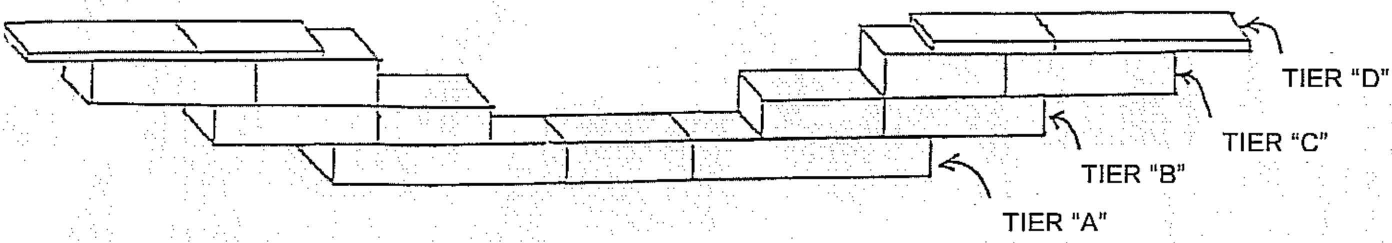
Figure 9.35



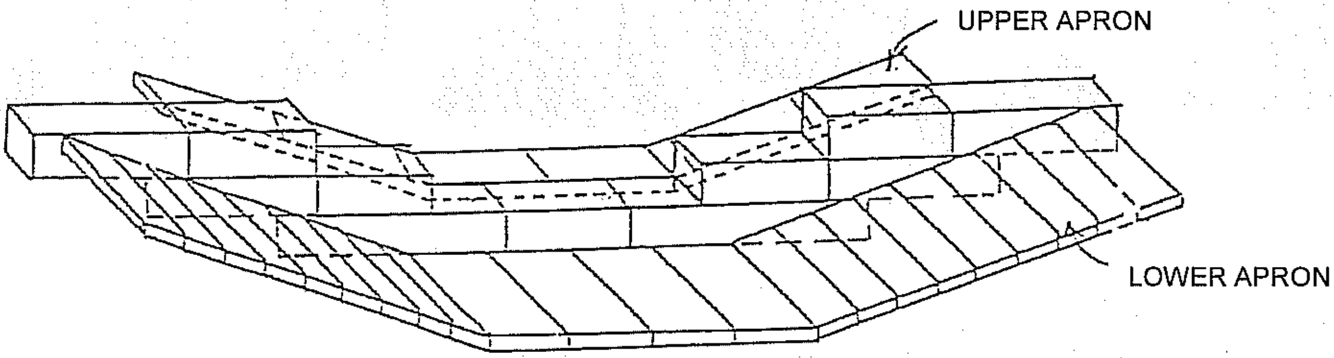
CHANNEL CHART
2:1 b = 20 Ft.

Source: AHTD

Figure 9.36



STEP - 1



STEP - 2

THE CITY OF B BENTONVILLE

GABION DROP STRUCTURE
CONSTRUCTION DETAIL

Figure 9.37

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Modern Sewer Design, American Iron and Steel Institute

Sewer Design and Construction, American Society of Civil Engineers and the Water Pollution Control Federation.

Hydrology for Engineers, McGraw-Hill Book Company, Inc.

Handbook of Concrete Culvert Pipe Hydraulics, Portland Cement Association.

Concrete Pipe Design Manual, American Concrete Pipe Association

Handbook of Steel Drainage & Highway Construction Products, American Iron and Steel Institute.

Flood Insurance Study, Benton County, Arkansas and incorporated areas, Federal Emergency Management Agency

Flood Hydrograph Package, Users Manual, US Army Corps of Engineers, revised March 1987

Hydraulic Design of Energy Dissipators for Culverts and Channels, U.S. Department of Transportation

Hydraulic Design of Improved Inlets for Culverts, U.S. Department of Transportation

Capacity Charts for the Hydraulic Design of Highway Culverts, U.S. Department of Commerce

Design of Roadside Drainage Channels, U.S. Department of Commerce

Design Charts for Open-Channel Flow, U.S. Department of Commerce

Drainage Manual, Arkansas State Highway and Transportation Department

Introduction to Hydrology, Second Edition, Harper & Row, Publishers

Stormwater Management and Drainage Manual, City of Little Rock, Arkansas, 1985

Design of Urban Highway Drainage, U.S. Department of Transportation, Federal Highway Administration, August 1979

Urban Hydrology for Small Watersheds, U.S. Department of Agriculture, Soil Conservation Service, June 1986

Design Manual for Storm Drainage Facilities, City of Dallas, Texas

Hydraulic Engineering Circular No. 12, U.S. Department Transportation - Federal Highway Administration

Drainage Manual, The City of Austin, Texas

Stormwater Management and Drainage Manual, The City of Rogers, Arkansas