

STORMWATER MANAGEMENT

**City of
Dodge City**



DESIGN MANUAL

STORMWATER MANAGEMENT DESIGN MANUAL

CITY OF DODGE CITY, KANSAS

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PREFACE

Urbanization may be defined as the process of altering from rural to urban the regional land-use patterns. This transformation has had many impacts on our society. These impacts have technical, economic, social, and political implications which have resulted in a rapid shifting of public attitudes towards the identification of the problems of urbanization, and a growing emphasis in governmental attitudes towards the treatment of these problems.

Among these problems are the management of excess stormwater and the control of soil erosion and sedimentation. Stormwater management is necessary to preserve and promote the general health, welfare, and economic well-being of the region. Stormwater management is a regional feature that affects all governmental jurisdictions and all parcels of property. This characteristic makes it necessary to formulate a program that balances public and private involvement. Although the overall coordination and master planning should be provided by the governing units most directly involved, stormwater management planning and programming should also be regionally integrated to include state and national agency interests.

Concurrent with the management of excess stormwater is the problem of soil erosion and sedimentation in developing areas. As with stormwater management, soil erosion and sedimentation control are regional concerns which require a balanced effort between public and private entities if desired results are to be achieved.

The intent of this Manual is to present current design procedures and techniques for the management of urban stormwater drainage and control of erosion and sedimentation. The procedures and techniques presented herein are consistent with available data and present understanding of the hydrologic cycle, and have been either developed specifically for the Dodge City area, or taken from commonly used references where applicable.

Space requirements dictated that some less-frequently used charts and tables be omitted. The designer is encouraged to consult the original references for more comprehensive explanations, design charts, and tables. A Bibliography has been provided at the end of each section for this purpose.

It is not the intent of this manual to limit the designer to specific design procedures or techniques. Other more appropriate methodologies may be used where applicable, provided prior approval is obtained from the City Engineer of Dodge City.

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Section 1

Drainage Policy

1.10 Introduction

Definitions

Goals - Broad general purposes.

Objectives - Those specific things which are desired and striven for.

Policy - The outline of principles and truths which will be a motivating force for a plan or a course of action.

Flood Plain - The land area which is covered by overflowing water.

Development - A growth, improvement or expansion to the community.

Master Plan - A set of maps which outline ways to reduce stormwater drainage problems. The plan includes 100-year flood maps and profiles; and shows sizes, capacity and location of drainage facilities.

Design Criteria Manual - A set of standards, requirements and procedures used for planning, designing and reviewing stormwater drainage improvements.

Minor Drainage System - A system which consists of streets, storm sewers and smaller open drainageways. The minor system is that which is planned to accomodate the storm runoff having a ten percent chance of occurence in any year and which provides convenience drainage and reduces street maintenance costs.

Major Drainage System - A system which makes provisions to prevent major property damage and loss of life. The major system is that which is planned to accomodate the runoff from a storm having a one percent chance of occurence in any year.

Detention Storage Facility - A stormwater facility which temporarily stores runoff and releases it to the downstream drainway in a controlled manner.

Retention Storage Facility - A stormwater facility which does not have a positive outlet below a level and, therefore, maintains a fixed water elevation between runoff events except for losses due to evaporation and seepage.

Statement of Problems

Haphazard growth in the community can cause neglect of the urban stormwater system. Problems caused by this neglect include the flooding, soil erosion and pollution which result in the following:

1. Increased risk for loss of life.
2. Damage to personal and public property.
3. Maintenance demands upon municipalities.
4. Delays of emergency vehicles.
5. Loss of the natural beauty of the community.

Solutions to the problems are complex and difficult to achieve. While the engineering design of solutions may be clear other problems exist which hinder the solutions such as:

1. The unwillingness of landowners to contribute to problem solutions which do not directly benefit them.
2. Conflicting attitudes of various interest groups.
3. The lack of finances to solve all of the problems at once.

Thus it becomes necessary to develop a tool which will consider alternate solutions to the problems and aid in implementing and administering the solutions. The tool includes a Stormwater Design Manual which presents the goals and objectives of the City and provides technical design criteria and data to achieve the goals. The tool also includes a Stormwater Management Plan which outlines solutions in an effort to avoid a piecemeal approach to improvements.

Goals and Objectives

The goals and objectives of stormwater management in Dodge City are as follows:

1. Protect human life and health.
2. Minimize private and public property damage resulting from erosion, sedimentation, and flooding.
3. Provide an efficient use of public monies.
4. Develop a Drainage Design Manual that ensures that design of the drainage system are consistent with good engineering practice.
5. Encourage flood plain uses consistent with approved land use plans for the flood plain areas and coordinated with plans for the total community.
6. Provide a mechanism which allows development of areas with minimum adverse effects to the natural environment.
7. Develop a comprehensive drainage plan for the area to handle storm runoff safely and efficiently.

1.20 Planning Requirements

1. A Storm Drainage Master Plan and Drainage Design Criteria Manual.
2. Site Planning. All land development proposals should receive full site planning including consideration of flood hazards.
3. Multiple Use. Drainage facilities shall be coordinated with other facilities such as parks and streets.
4. Natural Channels. Natural channels shall be used for transport of runoff except as approved otherwise by the City. Natural drainways provide significant storage capacity and provide open spaces.
5. Detention and Retention Storage. Storage facilities such as ponds reduce the drainage capacity required downstream. The policy of Dodge City is to require storage as outlined in the Master Plan. Development of storage in other areas that become necessary or feasible shall be considered and approved by the City.
 - a. Upstream Storage. Storage of storm runoff close to the point of rainfall occurrence includes uses of parking lots, parks, road embankments, and ponds.
 - b. Downstream Storage. Storage downstream includes storage reservoirs within the channel and planned overflow in suitable areas.
6. Stormwater Transfer. The diversion of storm runoff from one basin to another shall be avoided unless specific justification is given and it is shown that no damage is caused to property in the receiving basin.
7. Operations and Maintenance. An important part of all storm drainage facilities is the continued maintenance of the facilities to insure they will function as designed. Maintenance of detention facilities involves removal of debris and sediment. Sediment and debris must also be periodically removed from channels and storm sewers. Trashracks and street inlets must be regularly cleared of debris to maintain system capacity. Channel bank erosion, damage to drop structures, crushing of pipe inlets and outlets, and deterioration to the facilities must be repaired to avoid reduced conveyance capability, unsightliness, and ultimate failure.

Developers of privately owned storage facilities shall give assurance to the City that the facility will be properly operated and maintained.
8. Jurisdictional Boundaries. Since drainage considerations and problems cross jurisdictional boundaries a successful drainage plan must emphasize cooperation between the City and others

including Ford County, the State of Kansas Department of Transportation, the Santa Fe Railroad and the U.S. Army Corps of Engineers.

1.30 Technical Services

Design Criteria

Storm drainage planning and design of new facilities and revision of old facilities shall adhere to the criteria developed and presented in the Dodge City Drainage Manual.

The design criteria represent sound engineering practice and shall be utilized in the City. The criteria are not intended to be an iron-clad set of rules within which the planner and designer must work; they are intended to serve as guidelines, standards, and methods for sound planning and design.

Criteria Updating

The design criteria shall be revised and updated as necessary to reflect advances in the fields of urban drainage engineering and water resources management.

Drainage Systems

Drainage facilities in the City shall provide for both a minor storm system and a major storm system.

Design Storm Frequencies

Design storm frequencies shall be as stated in the Dodge City Drainage Manual for various project components.

Major Drainage Channels

Open channels for conveyance of major storm runoff are desirable in urban areas, and use of such channels is recommended.

Optimum benefits from open channels can best be obtained by incorporating suitable areas and greenbelts with the channel layout. Open channels should follow the natural flowlines and should receive early attention in planning stages of a new development, along with other storm runoff features.

Natural watercourses, perhaps wet only during and after large rainstorms, should not be filled, straightened, or altered significantly. Such actions tend to reduce capacity and storage and increase the velocity to the detriment of those downstream as well as those adjacent to the channel work. Effort must be made to reduce flood peaks and control erosion so that the natural channel is preserved as much as possible.

Tailwater

The depth of flow in the receiving stream must be taken into consideration for hydraulic computations for both the minor or major storm runoff.

Storm Runoff

The determination of runoff magnitude shall be either the rational formula, or the Soil Conservation Service (SCS) Unit Hydrograph Procedure.

a. Areas under 200 Acres

The Rational Formula shall be the method used to compute the peak discharge of storm runoff for basins less than 200 acres in size. Since the design storm is different for different areas, peak discharges from sub-areas should not be added. The peak discharge is a new solution to the Rational Method at each point. The SCS Unit Hydrograph Procedure is useful for checking of the Rational Formula results to insure reasonableness. Storage considerations of storm runoff shall be analyzed using the SCS Unit Hydrograph Procedure for areas larger than 200 acres.

b. Areas over 200 Acres

The SCS Urban Hydrograph Procedure shall be used for computing the storm runoff hydrograph for drainage areas over 200 acres in size. However, the sub-basins making up the total area, when studied on an individual basis and less than 200 acres, may be studied using the Rational Formula.

Accuracy

The peak discharges determined by either of the two methods are approximations. Rarely will the drainage system operate at the design discharge. Flow will always be more or less in actual practice, merely passing through the design flow as it rises and falls. Thus, the engineer should not overemphasize the accuracy of his computed discharges. He should emphasize the design of a practical and hydraulically balanced system based on sound logic and engineering as well as on dependable hydrology as defined in the City's Stormwater Design Manual.

Use of Streets

Use of streets for urban drainage shall fully recognize that the primary use of streets is for traffic. Streets are also significant and important in urban drainage, and full use should be made of streets for storm runoff up to reasonable limits. Reasonable limits for the use of streets for conveying storm runoff shall be governed by the design criteria presented in the Dodge City Drainage Manual.

Storage Areas

Detention and retention storage areas shall have a release rate for the 10-year, 2-hour storm which does not exceed the natural runoff rate. The

design of all impoundments, whether temporary or permanent, shall be approved by the Chief Engineer of the Kansas Division of Water Resources. An ordinance has been adopted by the City to insure the proper functioning of these storage areas to avoid public nuisances and health and safety hazards.

Erosion and Sediment Control

The need for seidment and erosion control facilities, either permanent or temporary, shall be determined according to the standards for sediment and erosion control in developing areas, as stated in this manual.

A Temporary Erosion and Sediment Control Plan is required unless otherwise approved by the City Engineer.

The temporary erosion and sedimentation control facility shall be constructed prior to any grading or extensive land clearing, in accordance with the above plan. These facilities must be satisfactorily maintained until construction and landscaping are completed and the potential for on-site erosion has passed.

1.40 Submittals

Plat Submittals

Good-quality plan and profile drawings shall be presented to the City prior to receipt of final approval. The drawings shall remain the permanent property of the City. A copy of design computations shall be included with the drawings.

Plans for the proposed drainage system shall include property lines, lot and block numbers, dimensions, right-of-way and easement lines, floodplains, street names, paved surfaces (existing or proposed), location, size and type of inlets, manholes, culverts, pipes, channels, and related structures, contract limits, outfall details, miscellaneous riprap construction, contour lines, and title block.

Profiles shall indicate the proposed system (size and material) with elevations, flowlines, gradients, left and right bank channel profiles, station numbers, inlets, manholes, ground-line and curb-line elevations, typical sections, riprap construction, filling details, minimum permissible slab elevations within 100-year floodplains and adjacent to open drainage features, pipe crossings, design flow capacities, and title block.

Record Drawings

Upon project completion and final approval by the City the original mylar drawings depicting as-constructed conditions shall be presented to the City.

1.50 Implementation

In the future, amendments can be made to drainage policy, practice and procedure if a review of drainage problems encountered indicates the best interests of the public are not being served.

The financing of storm drainage improvements is fundamentally the responsibility of the affected property owners: Both the person directly affected by the water and the person from whose land the runoff originates.

Section 2

Storm Runoff

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Section 2

Storm Runoff

This section presents two methods for computing storm runoff in the Dodge City, Kansas area. The Rational Method is the primary tool for the determination of runoff from areas of 200 acres or less and is especially useful for the design of storm sewer systems. U.S. Department of Agriculture, Soil Conservation Service (SCS) has developed a synthetic unit hydrograph procedure for generating hydrographs from ungaged watersheds. The procedure has gained wide acceptance and is the standard hydrologic procedure for the design of earth dams in the State of Kansas (Engineering Guide-7). The presentation of these two methods is not intended to preclude the use of other methods; however, the designer is advised to secure approval from the City of Dodge City and any other appropriate reviewing agencies before utilizing different methods.

2.10 Rational Method

The Rational Method is an empirical runoff formula which has gained wide acceptance because of its simple, intuitive treatment of storm runoff. This method relates peak runoff to rainfall intensity, surface area, and surface characteristics by the formula:

$$Q = C_f C_i A \quad (2-1)$$

where:

- Q = peak runoff rate, in cubic feet per second (cfs);
- C = runoff coefficient representing a ratio of peak runoff rate to average rainfall intensity for a duration equal to the time of concentration;
- C_f = correction factor to adjust the runoff coefficient for less-frequent, high-intensity storms;
- i = average rainfall intensity, in inches per hour; and
- A = drainage area, in acres.

The Rational Method is based on the following assumptions:

1. The peak rate of runoff at any point is a direct function of the average uniform rainfall intensity during the time of concentration to that point.
2. The frequency of the peak discharge is the same as the frequency of the average rainfall intensity.

3. The time of concentration is the time required for the runoff to become established and flow from the most hydraulically remote part of the drainage area to the point under design. This assumption applies to the most remote in time, not necessarily in distance.

Practice generally limits use of the Rational Method to some maximum area. For larger areas, storage and subsurface drainage flow cause an attenuation of the runoff hydrograph so that the rates of flow tend to be overestimated by the Rational Method. In addition, the assumption of uniform rainfall distribution and intensity becomes less appropriate as drainage area increases. Because of the trend for overestimation of flows and the additional cost in drainage facilities associated with this overestimation, the application of a more sophisticated runoff computation technique is usually warranted on larger drainage areas. The designer should obtain permission from the City of Dodge City before applying the Rational Method to areas larger than 200 acres. Refer to Section 1.30 for more discussion of the design criteria.

Runoff Coefficient

The runoff coefficient, C , is a variable of the Rational Method which is least susceptible to a precise determination and provides the designer with a degree of latitude to exercise his independent judgment. The following discussion is intended to provide a guide to promote the uniform application of runoff coefficients.

The runoff coefficient accounts for abstractions for losses between rainfall and runoff which may vary with time for a given drainage area. These losses are caused by interception by vegetation, infiltration into permeable soils, retention in surface depressions, and evaporation and transpiration. In determining this coefficient, differing climatological and seasonal conditions, antecedent moisture conditions, and the intensity and frequency of the design storm should be considered.

Table 2-1 presents recommended C values. Where ranges are shown, adjustments should be made for level of development, surface type, soil type, and slope. It is often desirable to develop a composite runoff coefficient based in part on the percentage of different types of surfaces in the drainage area. This procedure can be applied to typical "sample" areas as a guide to the selection of usual values of the coefficient for the entire area. Suggested coefficients with respect to surface types are given in Table 2-2.

TABLE 2-1

Recommended Runoff Coefficients

<u>Land Use</u>	<u>Runoff Coefficient</u>	
Business		
Downtown	0.70 to	0.95
Neighborhood	0.50 to	0.70
Residential		
Single-family	0.30 to	0.50
Multi-units, detached	0.40 to	0.60
Multi-units, attached	0.60 to	0.75
Residential (ranch-type)	0.25 to	0.40
Apartment	0.50 to	0.70
Industrial		
Light	0.50 to	0.80
Heavy	0.60 to	0.90
Parks and Cemeteries	0.10 to	0.25
Playgrounds	0.20 to	0.35
Railroad Yard	0.20 to	0.35
Unimproved	0.10 to	0.30

TABLE 2-2

Suggested Runoff Coefficients for Surface Types

<u>Character of Surface</u>	<u>Runoff Coefficients</u>
Pavement	
Asphaltic and Concrete	0.95
Brick	0.85
Roofs	0.95
Turf	
Flat, 0 to 1%	0.25
Average, 1 to 3%	0.35
Hilly, 3 to 10%	0.40
Steep, 10%+	0.45
Cultivated Ground	
Flat, 0 to 1%	0.10
Average, 1 to 3%	0.20
Hilly, 3 to 10%	0.25
Steep, 10%+	0.30

The coefficients in these two tables are applicable for a 10-year storm. These coefficients are based on the assumption that the design storm does not occur when the ground surface is frozen or covered by melting snow. Table 2-3 presents correction factors to adjust the above runoff coefficient for the 100-year storm.

TABLE 2-3

Frequency Factors for the Rational Formula

<u>Recurrence Interval (years)</u>	<u>Adjustment Factor C_f</u>
10	1.00
100	1.25*

* C_f should not exceed 1.0

Rainfall Intensity

Rainfall intensity, i , is the average rate of rainfall, in inches per hour. Intensity is selected on the basis of design frequency of exceedence, a statistical parameter established by design criteria, and rainfall duration. For the rational method, the critical rainfall intensity is the rainfall having a duration equal to the time of concentration of the drainage basin.

Rainfall intensity can be determined for the 10-year and 100-year return periods from Figure 2-1. This figure was compiled from the National Oceanic and Atmospheric Administration (NOAA) Technical Memorandum NWS HYDRO-35 and the U.S. Weather Bureau Technical Paper No. 40. These curves are applicable for durations from 5 to 120 minutes.

Time of Concentration

One of the basic assumptions underlying the rational method is that runoff is a function of the average rainfall rate during the time required for water to flow from the most hydraulically remote point of the drainage basin to the point under consideration. Time of concentration is usually computed by determining the travel time through the watershed. Overland flow, storm sewer or road gutter flow, and channel flow are the three phases of direct flow commonly used in computing travel time.

Overland Flow

The travel time for overland flow consists of the time it takes water to travel from the uppermost part of the watershed to a defined channel or inlet of the storm sewer system. Overland flow is significant in small watersheds because a high proportion of travel time is due to overland flow. The velocity of overland flow can vary greatly with the surface cover and tillage. If the slope and land use of the overland flow segment are known, the travel time can be read from Figure 2-2 or calculated using the following equation:

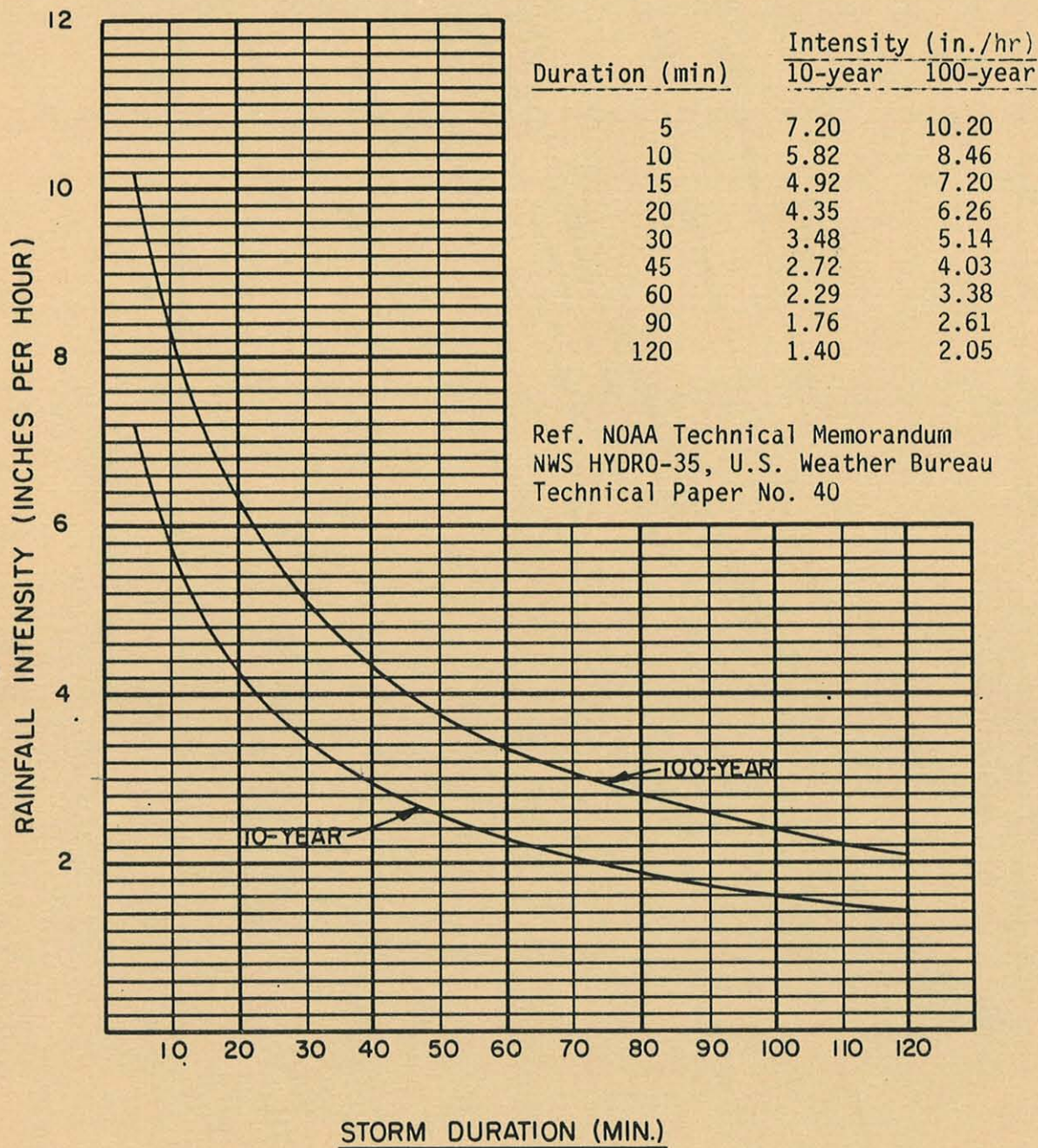


Figure 2-1, Rainfall Intensity Duration Curves

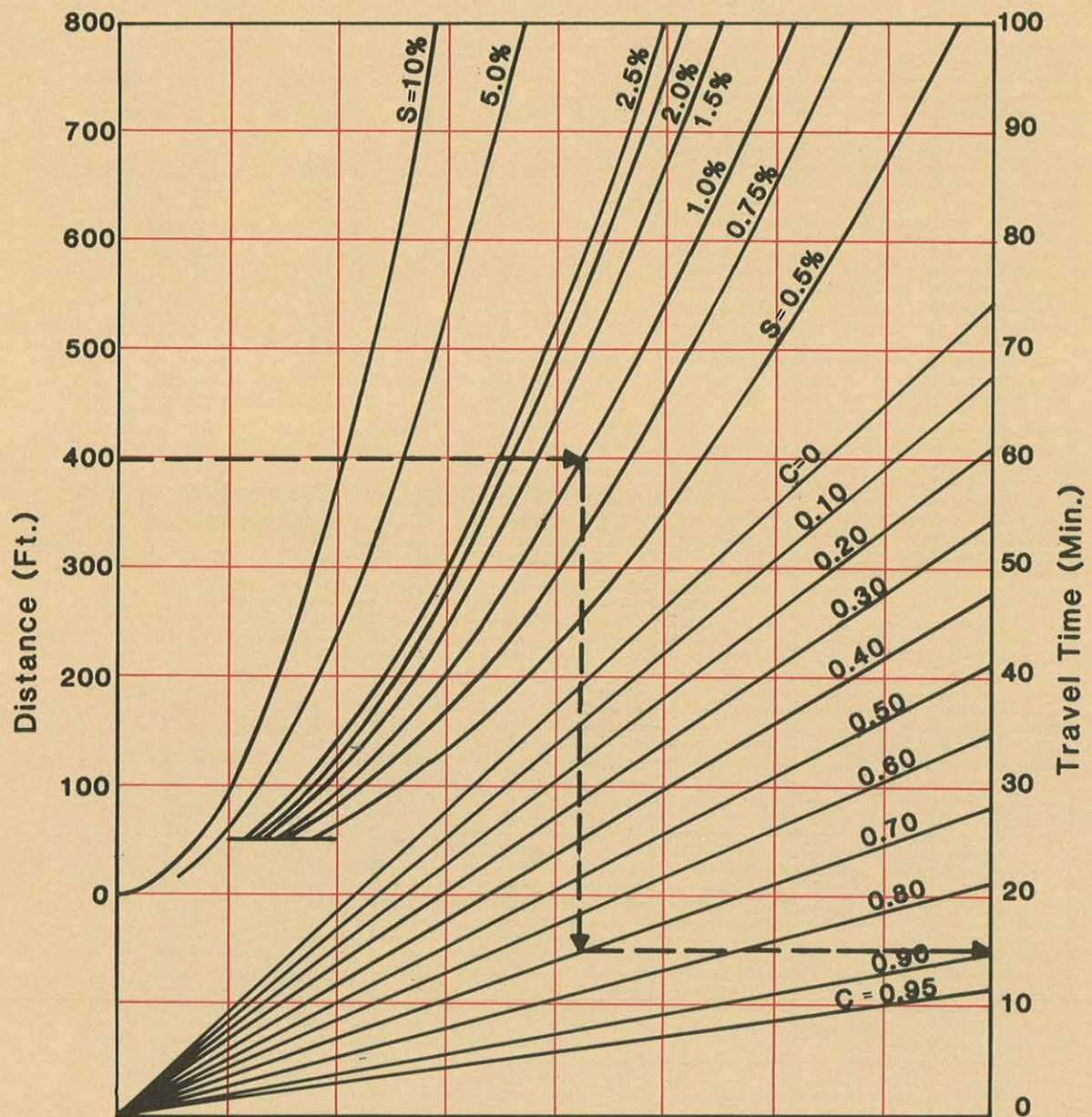


Figure 2-2

Overland Flow Curves

$$T = 1.87 (1.1 - CC_f)L^{0.5} S^{-0.33} \quad (2-2)$$

where:

T = travel time, in minutes;

CC_f = adjusted runoff coefficient;

L = length of overland flow, in feet; and

S = slope of flow path, in percent.

Times of concentration calculated for fully developed land use should not be less than 5 minutes to avoid the oversizing of inlets, storm sewers, and open channels.

Overland flow length should not exceed 300 feet for developed areas or 1,000 feet for undeveloped areas before being intercepted by a defined channel or storm sewer inlet. Beyond these distances use gutter flow or channel flow velocities based on Manning's Formula.

Storm Sewer or Road Gutter Flow

Travel time through the storm sewer or road gutter system to the main open channel is the sum of travel times in each individual component of the system between the uppermost inlet and the outlet. In most cases average velocities can be used without a significant loss of accuracy. During major storm events, the sewer system may be fully taxed and additional channel flow may occur, generally at a significantly lower velocity than the flow in the storm sewers. By using the average conduit size and the average slope (excluding any vertical drops in the system), the average velocity can be estimated using Manning's Formula. Refer to Sections 3 and 7 for further information on Manning's Formula.

Since the hydraulic radius of a pipe flowing half full is the same as when flowing full, the respective velocities are equal. Travel time may be based on the pipe flowing full or half full. The travel time through the storm sewers is computed by dividing the length of flow by the average velocity.

Channel Flow

The travel time for flow in an open channels can be determined by using Manning's formula to compute average velocities. Bankfull velocities should be used to compute these averages. Channels may be in either natural or improved condition.

Example 1

A model urbanized watershed is shown in Figure 2-3. Three types of flow exist from the furthestmost point of the watershed to the outlet. Compute time of concentration (T_C) based on the following data:

Reach	Description of Flow	Slope Percent	Length Feet
A to B	Overland (park, C = 0.2)	7	500
B to C	Overland (shallow gutter, C = 0.95)	2	900
C to D	Storm sewer (n = 0.015; diameter = 3 feet)	1.5	2,000
D to E	Open channel, gunite, trapezoidal (b = 5 ft; d = 3 ft; r = 1.78 ft; z = 1; n = 0.019 ft)	0.5	3,000

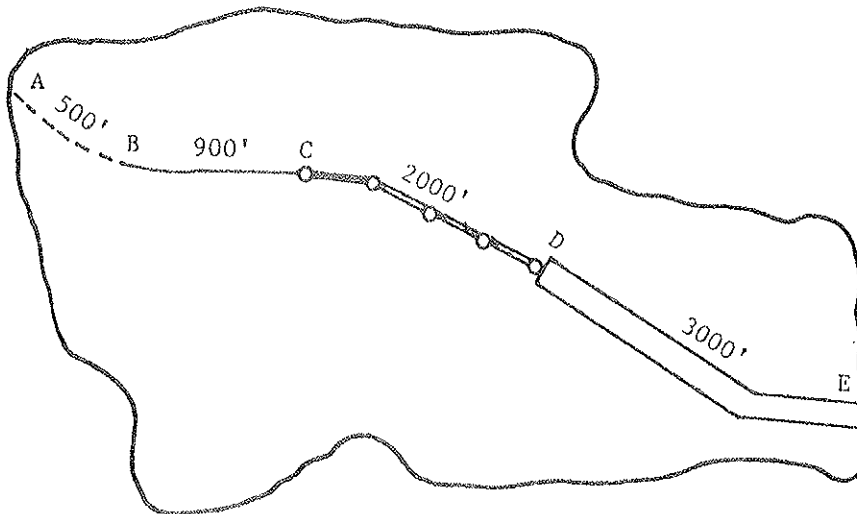


Figure 2-3 Urban Watershed

1. Compute the overland flow travel time.
Reach A to B (park). From Figure 2-2 for a slope of 7 percent, length of 500 feet, and C = 0.2, read T = 19.7 min = 1180 sec.

Reach B to C (street gutter). From Equation 2-2 for a slope of 2 percent, length of 900 feet, and C = 0.95, find T = 6.7 min = 400 sec.

2. Compute the storm sewer flow travel time.
Reach C to D. Use Manning's equation to compute full-pipe velocity.

$$v = \frac{1.49}{n} \frac{d^{2/3}}{4} s^{1/2}$$

$$v = \frac{1.49}{0.015} \frac{3^{2/3}}{4} (0.015)^{1/2} = 10 \text{ ft/sec}$$

$$T = \frac{\text{length}}{\text{velocity}} = \frac{2,000 \text{ ft}}{10 \text{ ft/sec}} = 200 \text{ sec}$$

3. Compute the open-channel flow travel time.
Reach D to E. Use Manning's formula to compute bankfull velocity.

$$v = \frac{1.49}{n} r^{2/3} s^{1/2}$$

$n = 0.019$ for gunite channel

$s = 0.005$

$$v = \frac{1.49}{0.019} (1.78)^{2/3} (0.005)^{1/2} = 8.14 \text{ ft/sec}$$

$$T = \frac{\text{length}}{\text{velocity}} = \frac{3,000 \text{ ft}}{8.14 \text{ ft/sec}} = 369 \text{ sec}$$

4. Summary

Reach	Description of flow	Length (ft)	Velocity (ft/sec)	Travel Time (sec)
A to B	Overland	500	0.42	1180
B to C	Overland	900	2.25	400
C to D	Storm Sewer	2,000	10.00	200
D to E	Open Channel	3,000	8.15	369
Total		6,400	2.98 (avg)	2149

$$T_c = \frac{2,149 \text{ sec}}{60 \text{ sec/min}} = 36 \text{ min}$$

2.20 Soil Conservation Service (SCS) Unit Hydrograph Procedure

Definition of Terms

The following definitions of terms are included to provide a better understanding of the SCS unit hydrograph method.

Actual Retention, F : Total of initial abstraction (I_a) and infiltration, in inches.

Duration, D : Unit hydrograph storm time length, in hours.

Concentration Time, T_c : Time for runoff to travel from the hydraulically most distant part of the watershed to the design point, in hours.

Design Storm: Selected storm duration and recurrence interval.

Dimensionless Unit Hydrograph: A hydrograph made to represent many unit hydrographs by using the time to peak and the peak discharge as basic units and plotting the hydrographs in ratios of these units.

Direct Runoff, Q : Volume of runoff resulting from precipitation (P) and actual retention, in inches.

Drainage Area, A : The total area that contributes directly to surface runoff, in square miles.

Hydraulic Length (e): Distance from the hydraulically most distant part of the watershed to the design point, in miles.

Impervious Area, I: The area within a basin through which water infiltrates with great difficulty. Typical impervious surfaces include parking lots, roads, and roofs.

Initial Abstraction, I_a : Volume of rainfall at the beginning of the storm which does not appear as runoff, in inches.

Lag Time, L: Time from the midpoint of rainfall excess to unit hydrograph peak, in hours.

Peak Discharge, q_p : The maximum rate of runoff created by Q inches of direct runoff, in cfs.

Potential Maximum Retention, S: Maximum volume of rainfall not contributing to runoff due to depression storage and infiltration, in inches.

Runoff Hydrograph: The product of the unit hydrograph ordinates and excess rainfall increments.

Slope, s: The average slope of the channel, in feet per mile. The following relation may be used to determine average slope.

$$s = \frac{H}{(0.8e)}$$

where:

H = the difference in elevation between the 0.8e elevation and the elevation at the point of study, in feet; and

e = length, in miles

Y: The slope in percent of the land of the watershed.

Storm Distribution: Rainfall time pattern for the selected storm duration.

Storm Duration: Actual duration of excess rainfall.

Time to Peak, T_p : Time from the beginning of rainfall excess to the peak rate of runoff, in hours.

Unit Hydrograph: The hydrograph having a volume of 1 inch of runoff from the corresponding unit hydrograph rainfall duration.

Design Storm

Selection of the design storm will depend on a number of factors including: 1) the type of structure (small dam, road culvert, storm drainage, etc.), 2) the particular component of a structure being designed (principal spillway, emergency spillway, etc.), and 3) the population and hazard potential. The design storm selection procedure contained in this manual represents a

general procedure applicable to the various hydraulic elements of the stormwater management system. Final design of structures may require adherence to special design procedures and standards. One example are the criteria set forth for small dams in Engineering Guide 1, Kansas State Board of Agriculture (1982).

The design storm duration is set equal to 6-hours. For the selected storm frequency, the rainfall depth is determined. Storm depths can be obtained for periods less than 2 hours from Figure 2-1. Additional rainfall depths are given in Table 2-4.

Table 2-4 Rainfall Depth Duration Frequency

Duration (hrs.)	Rainfall (inches)	
	Storm Frequency (Yr)	
	10	100
2	2.8	4.1
6	3.4	5.1
24	4.4	6.0

The recommended time distribution of storm rainfall is the Kent Six Hour Storm Distribution presented in the USGS Report 78-974 for Wichita, Kansas (Figure 2-4).

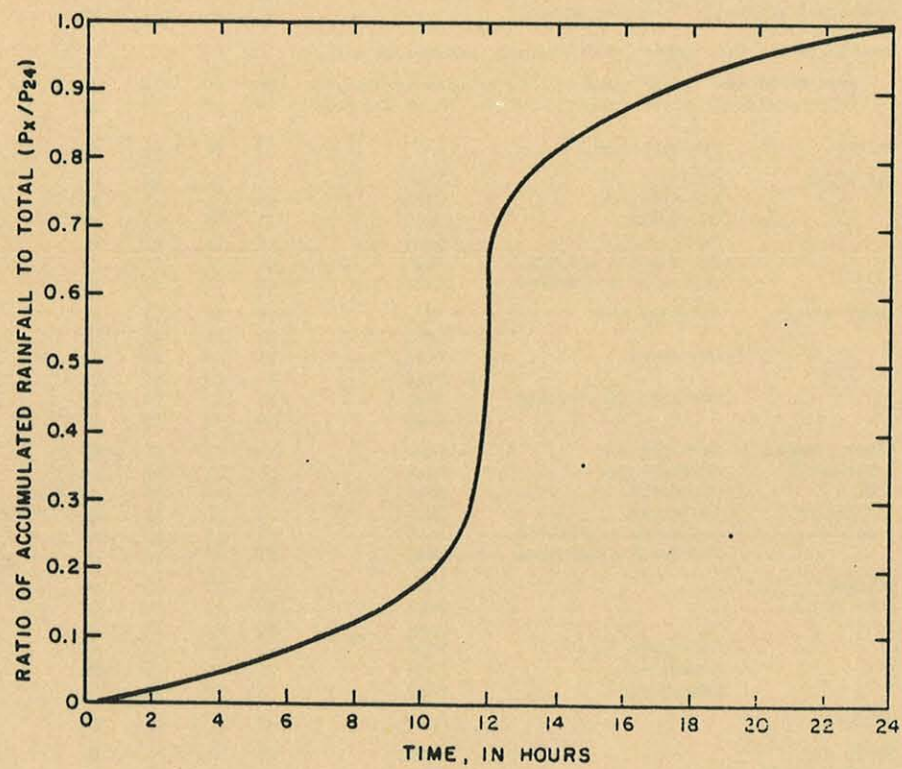


Figure 2-4 Kent Storm Distribution (1973)

Land Use Description/Treatment/Hydrologic Condition			Hydrologic Soil Group			
			A	B	C	D
Residential: ^{1/}						
	Average lot size	Average % Impervious ^{2/}				
	1/8 acre or less	65	77	85	90	92
	1/4 acre	38	61	75	83	87
	1/3 acre	30	57	72	81	86
	1/2 acre	25	54	70	80	85
	1 acre	20	51	68	79	84
Paved parking lots, roofs, driveways, etc. ^{3/}			98	98	98	98
Streets and roads:						
	paved with curbs and storm sewers ^{3/}		98	98	98	98
	gravel		76	85	89	91
	dirt		72	82	87	89
Commercial and business areas (85% impervious)			89	92	94	95
Industrial districts (72% impervious)			81	88	91	93
Open Spaces, lawns, parks, golf courses, cemeteries, etc.						
	good condition: grass cover on 75% or more of the area		39	61	74	80
	fair condition: grass cover on 50% to 75% of the area		49	69	79	84
Fallow	Straight row	---	77	86	91	94
Row crops	Straight row	Poor	72	81	88	91
	Straight row	Good	67	78	85	89
	Contoured	Poor	70	79	84	88
	Contoured	Good	65	75	82	86
	Contoured & terraced	Poor	66	74	80	82
	Contoured & terraced	Good	62	71	78	81
Small grain	Straight row	Poor	65	76	84	88
		Good	63	75	83	87
	Contoured	Poor	63	74	82	85
		Good	61	73	81	84
	Contoured & terraced	Poor	61	72	79	82
		Good	59	70	78	81
Close -seeded legumes ^{4/} or rotation meadow	Straight row	Poor	66	77	85	89
	Straight row	Good	58	72	81	85
	Contoured	Poor	64	75	83	85
	Contoured	Good	55	69	78	83
	Contoured & terraced	Poor	63	73	80	83
	Contoured & terraced	Good	51	67	76	80
Pasture or range		Poor	68	79	86	89
		Fair	49	69	79	84
		Good	39	61	74	80
	Contoured	Poor	47	67	81	88
	Contoured	Fair	25	59	75	83
	Contoured	Good	6	35	70	79
Meadow		Good	30	58	71	78
Woods or Forest land		Poor	45	66	77	83
		Fair	36	60	73	79
		Good	25	55	70	77
Farmsteads		---	59	74	82	86

^{1/} Curve numbers are computed assuming the runoff from the house and driveway is directed towards the street with a minimum of roof water directed to lawns where additional infiltration could occur.

^{2/} The remaining pervious areas (lawn) are considered to be in good pasture condition for these curve numbers.

^{3/} In some warmer climates of the country a curve number of 95 may be used.

^{4/} Close -drilled or broadcast.

Table 2-5

Runoff Curve Numbers for Hydrologic Soil-Cover Complexes
(Antecedent Moisture Condition II, and $I_a = 0.2 S$)

Runoff Potential - Curve Number (CN)

The SCS uses an index called the runoff curve number (CN) to represent the combined hydrologic effect of the soil type, land use, hydrologic condition of the soil cover, and the antecedent soil moisture. The CN indicates the runoff potential of soil which is not frozen. Higher CN's reflect a higher runoff potential.

The SCS soil classification system consists of four soil groups which are characterized as follows:

- Group A: deep sand, deep loess, aggregated silts
- Group B: shallow loess, sandy loam
- Group C: clay loams, shallow sandy loam, soils low in organic content, and soils usually high in clay
- Group D: Soils that swell significantly when wet, heavy plastic clays and certain saline soils

County soil surveys made by the SCS districts give a detailed description of the soils in a county. This is generally the best means of identifying the soil group.

The SCS cover classification includes three factors: 1) land use, 2) treatment, and 3) hydrologic condition. The land uses are subdivided by treatment practices (Table 2-5). The hydrologic condition reflects the level of land management which are given as poor, fair and good.

Antecedent soil moisture has a significant effect on runoff potential (CN). The SCS has developed three antecedent moisture conditions which are described below:

- Condition I: soils are dry but not to wilting point
- Condition II: average conditions
- Condition III: heavy rainfall, or light rainfall and low temperatures have occurred within the last 5 days; saturated soils

Table 2-6 gives the seasonal rainfall limits for the three soil moisture conditions.

Table 2-6 Seasonal Rainfall Limits and Antecedent Moisture Conditions

AMC group	Total 5-day antecedent rainfall	
	Dormant season Inches	Growing season Inches
I	Less than 0.5	Less than 1.4
II	0.5 to 1.1	1.4 to 2.1
III	Over 1.1	Over 2.1

Table 2-5 provides curve numbers for different land uses, treatment, and hydrologic conditions. The CN values are for soil moisture condition II. Curve Number adjustments for soil condition I or III can be made using Table 2-7.

Table 2-7 Curve Number Adjustment for Antecedent Moisture Conditions

CN for Condition II	Corresponding CN for Condition	
	I	III
100	100	100
95	87	99
90	78	98
85	70	97
80	63	94
75	57	91
70	51	87
65	45	83
60	40	79
55	35	75
50	31	70
45	27	65
40	23	60
35	19	55
30	15	50
25	12	45
20	9	39
15	7	33
10	4	26
5	2	17
0	0	0

The antecedent moisture condition selected shall produce reasonable runoff hydrographs for each specific design problem. The authors obtained acceptable results for watersheds in Dodge City using AMC II for both 100-year and 10-year frequency storms.

Excess Rainfall - Losses

The runoff volume (Q) depends on volume of precipitation (P) and the storage retention volume. The actual retention is the difference between the precipitation and runoff volumes. A volume of runoff, called the initial abstraction (I_a) is that volume of precipitation at the beginning of the storm which does not appear as runoff. The initial abstraction is a combination of interception, infiltration and depression storage.

The potential maximum retention (S) is the maximum retention volume of the soil. The SCS procedure assumes the initial abstraction is 20 percent of the potential maximum retention:

$$I_a = 0.2S \quad (2-4)$$

A second equation relates the potential maximum retention (S) to the curve number:

$$S = \frac{1000}{CN} - 10 \quad (2-5)$$

Empirical studies resulted in the following estimate of the runoff volume:

$$Q = \frac{(P-0.2S)^2}{P+0.8S} \quad (2-6)$$

The above relationships are used to construct Figure 2-5 for the determination of the direct runoff (Q). Note that two conditional relationships apply to the occurrence of direct runoff. The first states that no direct runoff occurs if the total rainfall does not exceed the initial abstraction:

$$P \geq I_a \quad \text{For direct runoff to occur} \quad (2-7)$$

Two additional relationships apply to occurrence of all rainfall contributing to direct runoff.

$$S \geq I_a + F \quad (2-8)$$

$$F = P - I_a - Q \quad (2-9)$$

Substituting Eq. 2-9 into Eq. 2-8 yields:

$$S \geq P - Q \quad \text{Infiltration of rainfall occurs} \quad (2-10)$$

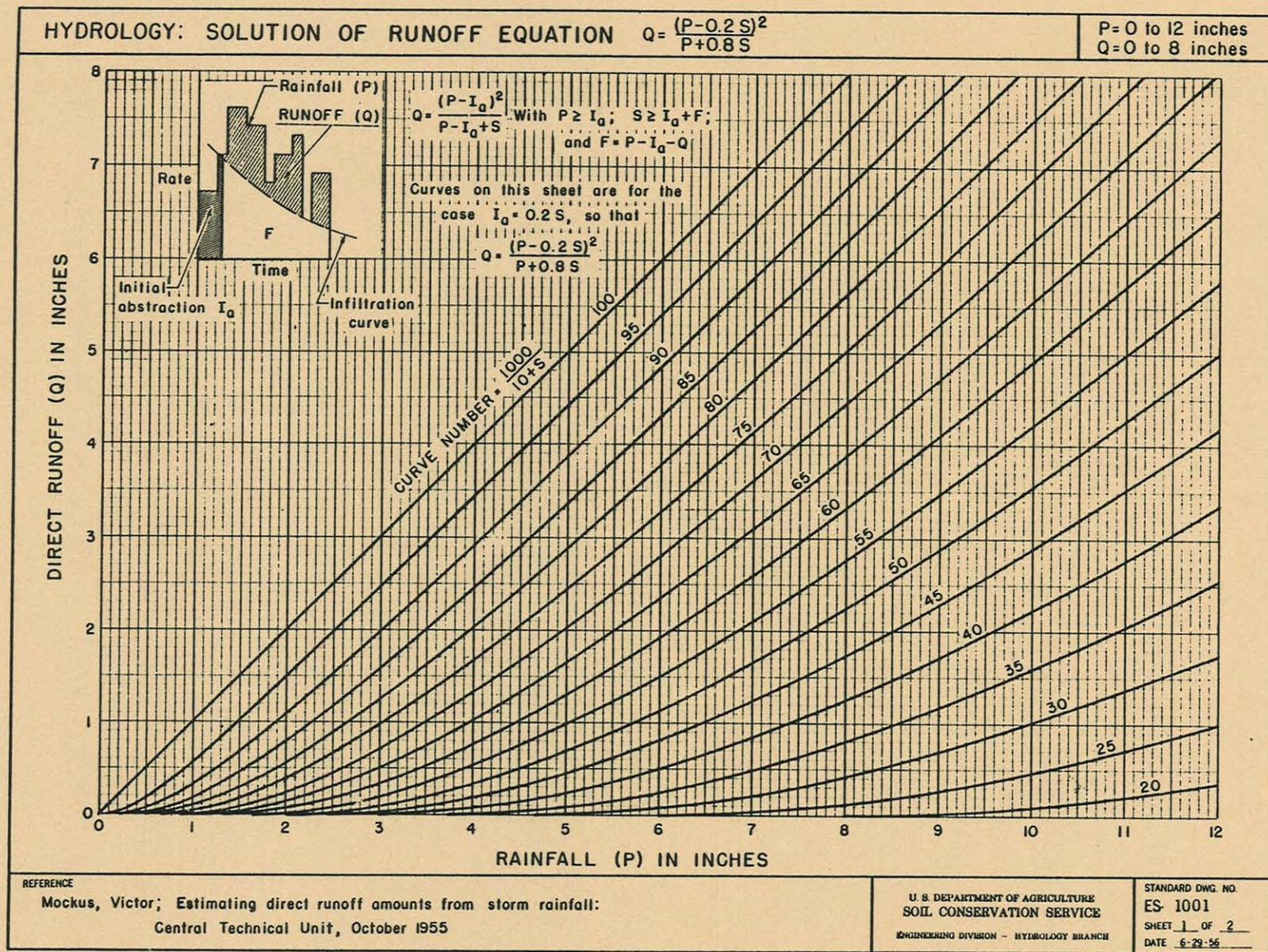
If Eq. 2-10 does not hold, the maximum potential retention has been equaled and all the rainfall will then occur as direct runoff.

Lag Time and Concentration Time

Two basic equations are used in defining the shape of the unit hydrograph. The first equation defines the lag time of the basin, L, or the time from the midpoint of unit excess rainfall to the unit hydrograph peak, t_p .

A United States Geological Survey Study (1978) evaluated the SCS synthetic hydrograph method for computing flood hydrographs for urbanized drainage basins in the Wichita, Kansas area. The study found that the calculated lag time (SCS equation for lag) consistently exceeded the observed lag times. Lag times calculated using an equation proposed by Putnam (1972) gave good results. The report concluded that with this lag time modification, the SCS method was found to be applicable to basins varying from slightly urbanized to fully urbanized in the Wichita area.

Figure 2-5 Solution of Rainfall Runoff Equation



The Putnam equation is used to calculate the lag time for urbanized basins, given by:

$$L = 0.49 \left(\frac{l}{\sqrt{s}} \right)^{0.5} (I)^{-0.57} \quad (2-11)$$

where:

L = lag time, in hours
 l = length of main water course, in miles
 s = channel slope, in feet per mile
 I = ratio of impervious area to total drainage area

The SCS equation is used for rural basins given by:

$$L = \frac{(l)^{0.8} (S+1)^{0.7}}{1900 (Y)^{0.5}} \quad (2-12)$$

where:

L = lag time, hours
 l = length of the mainstream to farthest divide in feet
 Y = average slope of watershed in percent
 $S = \frac{1000}{CN} - 10$

CN = SCS Curve Number

For basins which have a small amount of urbanization the SCS equation should be computed in addition to Putnam's equation and the smaller value of the two used to define the basin lag time.

A number of relationships are given for the dimensions of the SCS synthetic unit hydrograph (Figure 2-6). The concentration time is related to the lag by the equation:

$$T_c = \frac{5}{3} L \quad (2-13)$$

The time peak is related to the concentration time by:

$$T_p = \frac{2}{3} T_c \quad (2-14)$$

The time base of the unit hydrograph is given by:

$$T_b = 5 T_p \text{ (curvilinear unit graph)} \quad (2-15a)$$

$$T_b = \frac{8}{3} T_p \text{ (triangular unit graph)} \quad (2-15b)$$

The duration of the unit hydrograph is given by:

$$D = 0.133 T_C \quad (2-16)$$

Peak Discharge

The second equation which defines the shape of the unit hydrograph is the peak discharge, q_p . The equation for q_p , in cfs, is given by:

$$q_p = \frac{484AQ}{T_p} \quad (2-17)$$

where:

A = drainage area, in square miles
Q = direct runoff, in inches
 T_p = time to peak, in hours

Substituting Eq. 2-14 into Eq. 2-17 yields an alternate form of the peak discharge equation:

$$q_p = \frac{726 AQ}{T_C} \quad (2-18)$$

where:

T_C = concentration time, in hours

SCS Dimensionless Unit Hydrograph

The SCS dimensionless unit hydrograph was derived from a large number of natural unit hydrographs from watersheds varying widely in size and geographic locations. This dimensionless hydrograph (Figure 2-6) has its ordinate values expressed in a dimensionless ratio q/q_p or Q_a/Q and its abscissa values as t/T_p . By expressing the coordinates of the unit hydrograph as ratios of the peak and time to peak the dimensionless hydrograph can easily be modified by specific basin parameters to represent a unit hydrograph for the watershed. The ratios for the dimensionless unit hydrograph are given in Table 2-8. The curvilinear unit hydrograph has 37.5% of the total volume in the rising side which is represented by one unit of time and one unit of discharge. This unit hydrograph can be represented by an equivalent triangular hydrograph having the same units of time and discharge, thus having the same percent of volume in the rising side of the triangle. The SCS unit hydrograph procedure is demonstrated in Example 2.

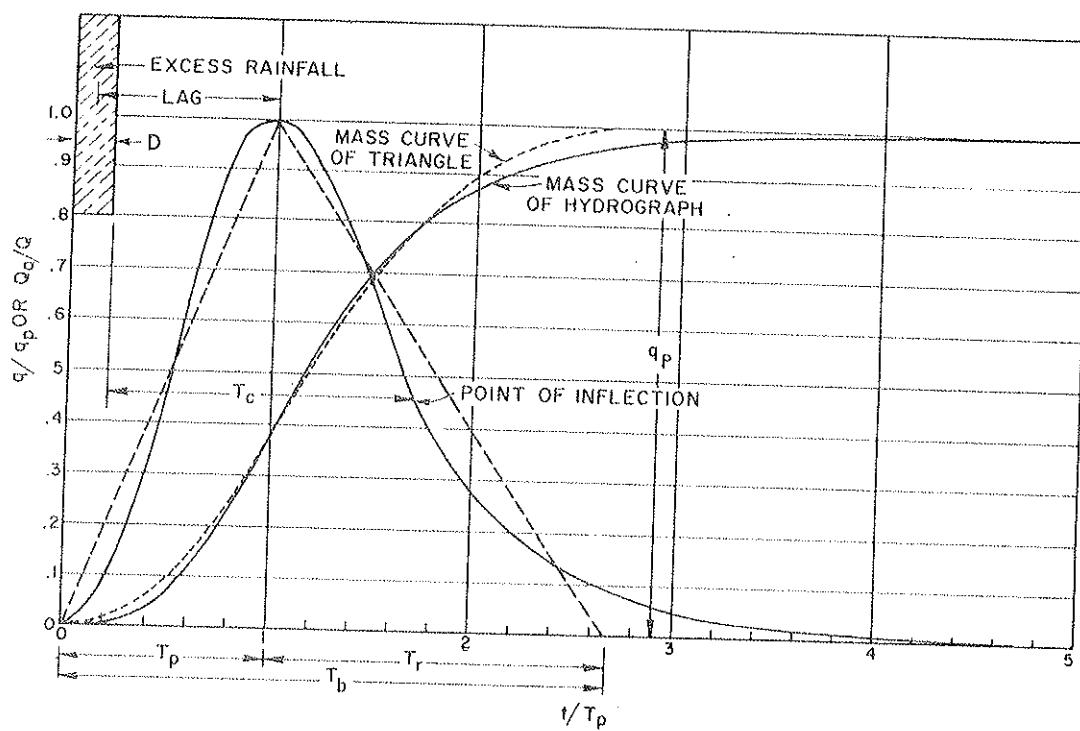


Figure 2-6 Dimensionless curvilinear unit hydrograph and equivalent triangular hydrograph

Example 2

Determine the peak discharge for a 1.94 square mile basin composed of seventy percent residential (1/4 acre lot size) and thirty percent open space (fair condition). Use the B Hydrologic Soil Group. Assume the basin to be composed of thirty three percent impervious area. The hydraulic length of the channel is 3.55 miles and has a slope of 57 feet per mile. Use the 100 year storm and a Type II soil moisture condition:

1. Compute the basin curve number. From Table 2-5 for Hydrologic Soil Group B:

Residential, average lot size 1/4 acre:	CN = 75
Open space, fair condition:	CN = 69

Weighted CN:

$$CN = 0.7 (75) + 0.3 (69) = 73.2 \text{ Use } 73$$

2. Compute the basin lag time and time of concentration:

$$\text{From Eq. 2-11: } L = (0.49) \left(\frac{3.55}{57} \right)^{0.5} (0.33)^{-0.57} = 0.63 \text{ hr.}$$

$$\text{From Eq. 2-13: } T_c = \frac{5}{3} (0.63) = 1.05 \text{ hr.} = 63 \text{ min.}$$

3. Compute the design storm total rainfall (P).

Using a design storm of 6 hours from table 2-4 the rainfall is 5.1 inches.

4. Compute the potential maximum retention (S) and initial abstraction (I_a):

$$\text{From Eq. 2-5: } S = \frac{1000}{73} - 10 = 3.7 \text{ in.}$$

$$\text{From Eq. 2-4: } I_a = (0.2)(3.7) = 0.74 \text{ in.}$$

5. Compute the SCS unit hydrograph duration (D), time base (T_b), and the time to peak (T_p).

$$\text{From Eq. 2-16: } D = (0.133)(1.05) = 0.14 = 8.4 \text{ min. (Use 10.0 min.)}$$

$$\text{From Eq. 2-14: } T_p = \frac{2}{3} (1.05) = 0.70 \text{ hr.} = 42 \text{ min.}$$

$$\text{From Eq. 2-15}_a: T_b = (5)(0.70) = 3.50 \text{ hr.} = 210 \text{ min.}$$

6. Compute the unit hydrograph peak discharge (q_p):

$$\text{From Eq. 2-17: } q_p = \frac{(484)(1.94)(1)}{0.70} = 1340 \text{ cfs/in. direct runoff}$$

7. Compute the design storm distribution.

Columns one through five of Table 2-9 are constructed from Figure, 2-4, Figure 2-5 and a total rainfall depth of 5.1 in.

Table 2-8 Ratios for Dimensionless Unit Hydrograph
and Mass Curve

Time Ratios (t/T_p)	Discharge Ratios (q/q_p)	Mass Curve Ratios (Q_a/Q)
0	.000	.000
.1	.030	.001
.2	.100	.006
.3	.190	.012
.4	.310	.035
.5	.470	.065
.6	.660	.107
.7	.820	.163
.8	.930	.228
.9	.990	.300
1.0	1.000	.375
1.1	.990	.450
1.2	.930	.522
1.3	.860	.589
1.4	.780	.650
1.5	.680	.700
1.6	.560	.751
1.7	.460	.790
1.8	.390	.822
1.9	.330	.849
2.0	.280	.871
2.2	.207	.908
2.4	.147	.934
2.6	.107	.953
2.8	.077	.967
3.0	.055	.977
3.2	.040	.984
3.4	.029	.989
3.6	.021	.993
3.8	.015	.995
4.0	.011	.997
4.5	.005	.999
5.0	.000	1.000

Table 2-9, Design Storm Construction

Cumulative ¹ Time (min.)	Rainfall: ² $\frac{P_x}{P_{24}}$	Cumulative ³ Rain (in.)	Cumulative ⁴ Direct ^t Runoff (in.)	Incremental Direct Runoff (in.)
10	0.000	0.0	0	0
20	0.014	0.07	0	0
30	0.022	0.11	0	0
40	0.031	0.16	0	0
50	0.040	0.20	0	0
60	0.048	0.24	0	0
70	0.056	0.29	0	0
80	0.068	0.35	0	0
90	0.080	0.41	0	0
100	0.093	0.47	0	0
110	0.107	0.55	0	0
120	0.120	0.61	0	0
130	0.138	0.70	0	0
140	0.158	0.81	0.001	0.001
150	0.181	0.92	0.01	0.01
160	0.217	1.11	0.03	0.02
170	0.311	1.59	0.16	0.13
180	0.663	3.38	1.10	0.94
190	0.736	3.75	1.35	0.25
200	0.788	4.02	1.54	0.19
210	0.820	4.18	1.66	0.12
220	0.843	4.30	1.75	0.09
230	0.863	4.40	1.82	0.07
240	0.880	4.49	1.89	0.07
250	0.895	4.56	1.94	0.05
260	0.908	4.63	1.99	0.05
270	0.921	4.70	2.05	0.06
280	0.927	4.73	2.07	0.02
290	0.937	4.78	2.11	0.04
300	0.952	4.86	2.17	0.06
310	0.963	4.91	2.21	0.04
320	0.972	4.96	2.25	0.04
330	0.980	5.00	2.28	0.03
340	0.987	5.03	2.30	0.02
350	0.994	5.07	2.34	0.04
360	1.000	5.10	2.36	0.02
				<u>2.36</u>

¹Storm duration of 6 hours²From Fig. 2-4³For rainfall of 5.1 inches⁴From Fig. 2-5 or Eq. 2-6

Note that runoff does not occur until the rainfall exceeds the initial abstraction ($P - I_a$). At 140 minutes, the rainfall ($P=0.81$ in.) just exceeds the initial abstraction ($I_a=0.74$) and the direct runoff can be calculated from Eq. 2-6.

8. Unit hydrograph construction

Using the values of $\frac{t}{T_p}$ and $\frac{q}{q_p}$ given in Table 2-8 and the computed values of T_p and q_p ($T_p = 42$ min., $q_p = 1340$ cfs per in. direct runoff), the unit hydrograph ordinates are computed (Table 2-10). These points are plotted on Figure 2-7.

Table 2-10, Ten Minute Unit Hydrograph

$\frac{t}{T_p}$	t (min)	$\frac{q}{q_p}$	q (cfs/in)
0	0	0	0
0.1	4	0.030	40
0.2	8	0.100	134
0.3	13	0.190	255
0.4	17	0.310	415
0.5	21	0.470	630
0.6	25	0.660	884
0.7	29	0.820	1099
0.8	34	0.920	1233
0.9	38	0.990	1327
1.0	42	1.000	1340
1.1	46	0.990	1327
1.2	50	0.930	1246
1.3	55	0.860	1152
1.4	61	0.780	1045
1.5	63	0.680	911
1.6	67	0.560	750
1.7	71	0.460	616
1.8	76	0.390	523
1.9	80	0.330	442
2.0	84	0.280	375
2.2	92	0.207	277
2.4	101	0.147	197
2.6	109	0.107	144
2.8	118	0.077	103
3.0	126	0.055	74
3.2	134	0.040	54
3.4	143	0.029	39
3.6	151	0.021	28
3.8	160	0.015	20
4.0	168	0.011	14
4.5	189	0.005	7
5.0	210	0.000	0

$T_p = 42$ min.

$q_p = 1340$ cfs/in.

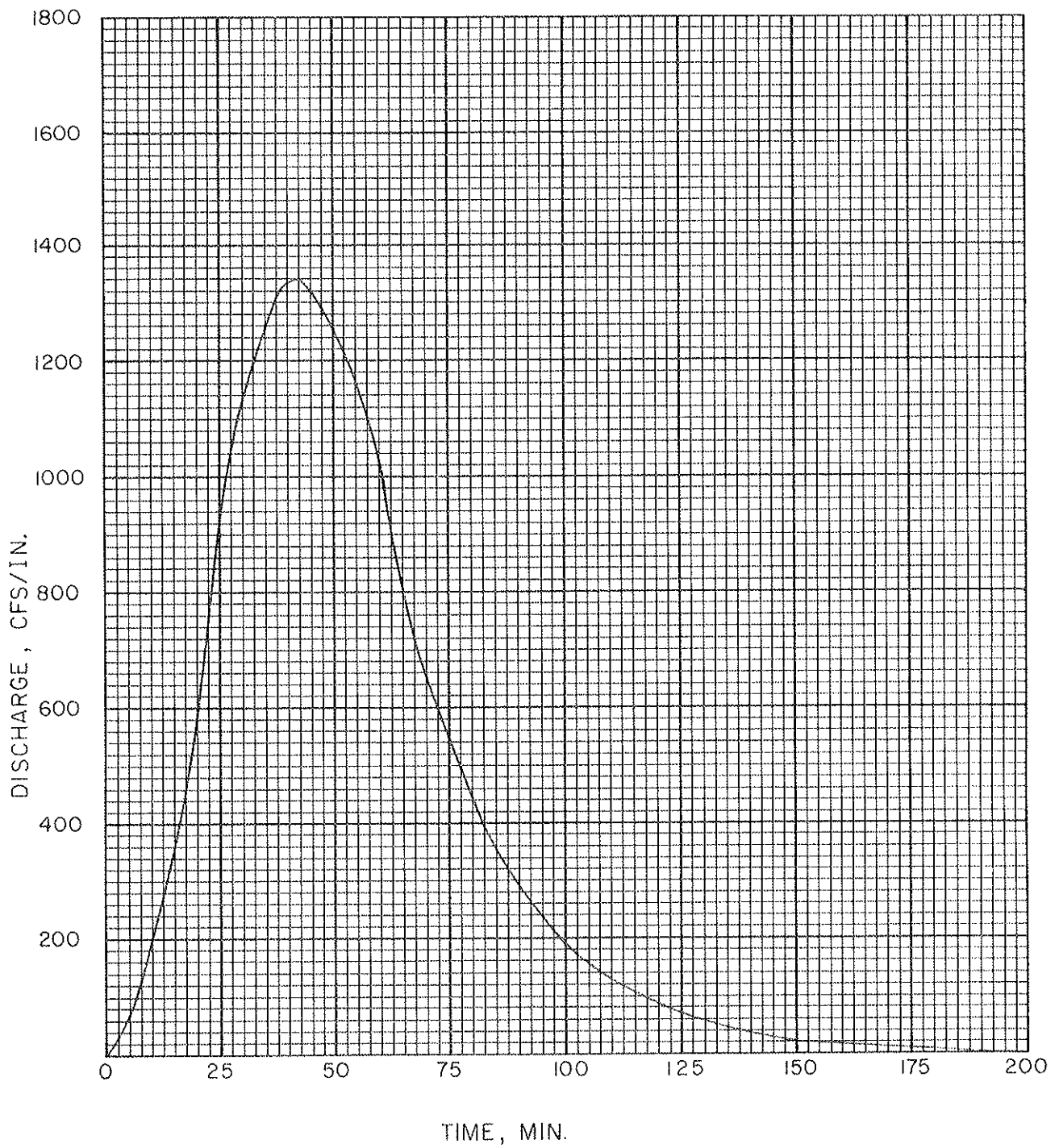


Figure 2-7 Unit Hydrograph -10 Minute Duration

The unit hydrograph ordinates of Figure 2-7 are read at ten minute intervals and the final ten minute unit hydrograph ordinates tabulated in Table 2-11.

Table 2-11, Ten Minute Unit Hydrograph Ordinates

<u>Time</u> <u>(min)</u>	<u>q</u> <u>(cfs/in)</u>
0	0
10	180
20	580
30	1130
40	1340
50	1250
60	1060
70	650
80	440
90	200
110	140
120	90
130	60
140	40
150	10
160	8
170	6
180	4
190	2
200	1
210	0

9. Hydrograph Construction

The final hydrograph is obtained by multiplying the ten minute duration unit hydrograph ordinates by the design storm rainfall values (direct runoff). The process of convoluting the runoff hydrograph is shown in Table 2-12. The peak discharge occurs at time of 230 minutes and is 1966 cfs.

Table 2-12, Runoff Hydrograph

Unit Hydrograph (cfs/in.)	Time (min.)	0.001	0.01	0.02	0.13	0.94	Direct Runoff (in.)										0.07	0.07	0.05	0.05	0.06	0.02	0.04	0.06	0.04	0.04	0.03	0.02	0.04	0.02	Runoff Hydro- graph (cfs)
0	140	0																													0
180	150	0																													0
580	160	0	2	0																											2
1130	170	1	6	4	0																										11
1340	180	1	11	12	23	0																									47
1250	190	1	13	23	75	169	0																								281
1060	200	1	12	27	147	545	45	0																							777
650	210	1	11	25	174	1062	145	34	0																						1452
440	220	0	6	21	163	1260	283	110	22	0																					1865
200	230	0	4	13	138	1175	335	215	70	16	0																				1966
140	240	0	2	9	85	996	313	255	136	52	13	0																			1861
90	250	0	1	4	57	611	265	238	161	102	41	13	0																		1493
60	260	0	1	3	26	414	163	201	150	121	79	41	9	0																	1208
40	270	0	0	2	18	188	110	124	127	113	94	79	29	9	0																893
10	280	0	0	1	12	132	50	84	78	95	88	94	57	29	11	0															731
8	290	0	0	1	8	85	35	38	53	59	74	88	67	57	35	4	0														604
6	300	0	0	0	5	56	23	27	24	40	46	74	63	67	68	12	7	0													512
4	310	0	0	0	1	38	15	17	17	18	31	46	53	63	80	23	23	11	0												436
2	320	0	0	0	1	9	10	11	11	13	14	31	33	53	75	27	45	35	68	23	7	0									375
1	330	0	0	0	1	8	3	8	7	8	10	14	22	33	64	25	54	68	45	23	7	0									358
0	340	0	0	0	0	6	2	2	5	5	6	10	10	22	39	21	50	80	45	23	5	0									332
						4	2	2	1	1	4	4	6	7	12	9	42	75	54	45	17	4									316
						2	1	1	1	1	3	4	5	10	26	13	26	64	50	54	17	4									293
						1	1	1	1	1	1	3	3	5	8	4	18	39	42	50	40	23									267
						0	0	0	0	0	0	0	0	0	1	2	2	26	26	42	38	27	45	12							240
							0	0	0	0	0	0	0	0	2	3	5	12	18	26	32	25	54	23							206
								0	0	0	0	0	0	0	1	2	4	8	8	18	20	21	50	27							160
									0	0	0	0	0	0	0	1	1	5	6	8	13	13	42	25							116
										0	0	0	0	0	0	0	2	4	6	6	6	9	26	21							78
											0	0	0	0	0	0	0	2	4	4	4	4	18	13							47
												0	0	0	0	0	0	0	2	4	2	3	8	9							28
													0	0	0	0	0	0	1	2	2	2	6	4							16
														0	0	0	0	0	0	2	2	2	4	3							9
															0	0	0	0	0	0	0	1	2	2							5
																0	0	0	0	0	0	0	2	1							3
																	0	0	0	0	0	0	0	1							1
																		0	0	0	0	0	0	0							0
																			0	0	0	0	0	0							0
																				0	0	0	0	0							0
																					0	0	0	0							0
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2.30 Bibliography

1. American Society of Civil Engineers, Design and Construction of Sanitary and Storm Sewers, Manuals and Reports on Engineering Practice, No. 37, New York, NY, 1969.
2. Aron, G. and E.L. White, Fitting a Gamma Distribution Over a Synthetic Unit Hydrograph, American Water Resources Association, Water Resources Bulletin, Vol. 18, No. 1, Minneapolis, MN, 1982.
3. Craig, G.S. and J.G. Rankl, Analysis of Runoff from Small Drainage Basins in Wyoming, U.S. Geological Survey Water-Supply Paper 2056, Washington, DC, 1978.
4. Federal Highway Administration, Design of Urban Highway Drainage - The State of the Art, Washington, D.C., 1979.
5. Henningson, Durham & Richardson, Inc., Omaha Metropolitan Area Stormwater Management Design Manual, City of Omaha, Douglas County, and Papio Natural Resources District, Omaha, NE, 1980.
6. Miller, J.F., R.H. Frederick, and R.J. Tracey, Precipitation - Frequency Atlas of the Western United States, NOAA Atlas 2, Vol. II - Wyoming, U.S. Weather Service, Washington, DC, 1973.
7. Peek, C.O., and P.R. Jordan, Determination of Peak Discharge from Rainfall Data for Urbanized Basins, Wichita, Kansas, U.S. Geological Survey, Report 78-974, Lawrence, Kansas, 1978.
8. Resource Consultants, Inc., Larimer County Stormwater Management Manual, Fort Collins, CO, 1979.
9. Soil Conservation Service, National Engineering Handbook, Section 4, Hydrology, Washington, DC, 1971.
10. Wright-McLaughlin Engineers, Urban Storm Drainage Criteria Manual, Vol 1, Denver Regional Council of Governments, Denver, CO, 1977.

Section 3

Street Drainage

- 3.10 Effects of Stormwater on Street Capacity
 - Interference Due to Sheet Flow Across Pavement
 - Hydroplaning
 - Splash
 - Interference Due to Gutter Flow
 - Interference Due to Ponding
 - Interference Due to Water Flowing Across Traffic Lane
 - Effect on Pedestrians
- 3.20 Design Criteria
 - Street Capacity for Minor Storms
 - Calculating Capacity
 - Street Capacity for Major Storm
 - Calculating Capacity
 - Ponding
 - Cross-Street Flow
- 3.30 Intersection Layout Criteria
 - Gutter Capacity, Minor Storm
 - Gutter Capacity, Major Storm
 - Ponding
 - Cross-Street Flow
- 3.40 Bibliography

Section 3

Street Drainage

Streets serve an important and necessary drainage service even though their primary function is for the movement of traffic. Traffic and drainage uses are compatible up to a point, beyond which drainage must be subservient to traffic needs.

Gutter flow in streets is necessary to transport runoff to storm inlets and to major drainage channels. Good planning of streets can substantially help in reducing the size of, and sometimes eliminating the need for, a storm sewer system in newly urbanized areas.

3.10 Effects of Stormwater on Street Capacity

The storm runoff which influences the traffic capacity of a street can be classified as follows.

- A. Sheet flow across the pavement as falling rain flows to the edge of the pavement.
- B. Runoff flowing adjacent to the curb.
- C. Stormwater ponded at low points.
- D. Flow across the traffic lane from external sources, or cross-street flow (as distinguished from water falling on the pavement surface).
- E. Splashing of any of the above types of flow on pedestrians.

Each of these types of storm runoff must be controlled within acceptable limits so that the street's main function as a traffic carrier will not be unduly restricted.

The effect of each of the above categories of runoff on traffic movement are discussed in the following sections.

Interference Due to Sheet Flow Across Pavement

Rainfall which falls upon the paved surface of a street or road must flow overland as sheet flow until it reaches a channel. Channels can be created either by curbs and gutters or by roadside ditches. The direction of flow on the street may be determined by the vector addition of the street grade and the crown slope, which is equivalent to drawing the perpendicular to a contour line on the road as shown in Figure 3-1. The depth of sheet flow will be essentially zero at the crown of the street and will increase as it proceeds towards the channel. Traffic interference due to sheet flow is essentially of two types: hydroplaning and splash.

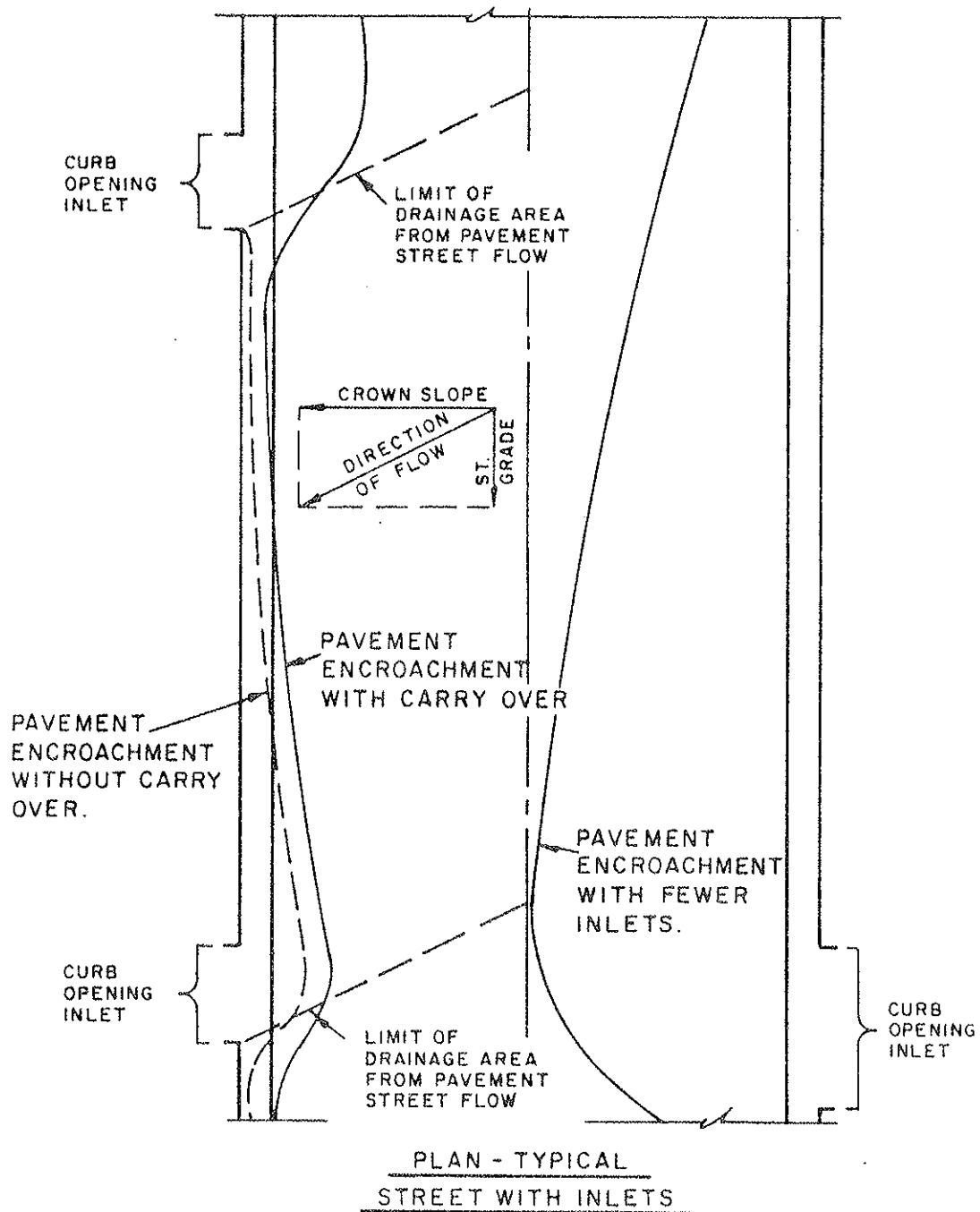


Figure 3-1 Gutter and Pavement Flow Patterns

Hydroplaning

Hydroplaning is the phenomenon of vehicle tires actually being supported by a film of water which acts as a lubricant between the pavement and the vehicle. It generally occurs at speeds commensurate with arterial streets and its effect can be minimized by achieving a relatively rough pavement which will allow water to escape from beneath the tires by pavement grooving to provide drainage, or by reducing travel speed.

Splash

Traffic interference due to splash results from sheet flow of excessive depth caused by water traveling a long distance or at a very low velocity before reaching a gutter. Increasing the street crown slope will decrease both the time and distance required for water to reach the gutter. The crown slope, however, must be kept within acceptable limits to prevent side-slipping of traffic during frozen surface conditions and to allow the opening of doors when parked adjacent to curbs. An exceedingly wide pavement section contributing flow to one curb will also affect the depth of sheet flow. This may be due to superelevation of a curve, off-setting of the street crown due to warping of curbs at intersections, or many traffic lanes between street crown and the gutter. Consideration should be given to all of these factors to maintain a depth of sheet flow within acceptable limits.

Interference Due to Gutter Flow

Water which enters a street, either sheet flow from the pavement surface or overland flow from adjacent land areas, will flow in the gutter of the street until it reaches an outlet, such as a storm sewer or a channel. Figure 3-1 shows the configuration of gutter flow moving down a street when there is a storm sewer system. As the flow progresses downhill and additional areas contribute to the runoff, the width of flow will increase and progressively infringe upon the traffic lane. If vehicles are parked adjacent to the curb, the width of spread will have little influence on traffic until it exceeds the width of the vehicle by several feet. However, on streets where parking is not permitted, as with many arterial streets, whenever the flow width exceeds a few feet it will significantly affect traffic. Field observations show that vehicles will crowd adjacent lanes to avoid curb flow.

As the width increases it becomes impossible for vehicles to operate without driving through water, and they again begin to use the inundated lane. At this point the traffic velocity will be significantly reduced as the vehicles begin to drive through the deeper water. Splash from vehicles traveling in the inundated lane obscures the vision of drivers of vehicles moving at a higher rate of speed on the open lane.

Eventually, if width and depth of flow become great enough, the street will become ineffective 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 street by moving along the crown of the roadway.

The street classification is also important when considering the degree of interference to traffic. A local street, and to a lesser extent a collector street, could be inundated with little effect upon vehicular travel. The small number of cars involved could move at a low rate of speed through the water even if the depth were four to six inches. However, reducing the speed of arterial traffic affects a greater number of private, commercial, and emergency vehicles.

Interference Due to Ponding

Storm runoff ponded on the street surface because of a change in grade or the crown slope of intersecting streets has a substantial effect on traffic. A major problem with ponding is that it may reach depths greater than the curb and remain on the street for long periods of time. Another problem is that ponding is localized in nature and vehicles may enter a pond moving at a high rate of speed.

The manner in which ponded water affects traffic is essentially the same as for curb flow; the width of spread onto the traffic lane is the critical parameter. Ponded water will often bring traffic on a street to a complete halt. In this case, incorrect design of only one facet of an entire street and storm drainage system will render the remainder of the street system useless during the runoff period.

Interference Due To Water Flowing Across Traffic Lane

Whenever storm runoff, other than sheet flow, moves across a traffic lane, a serious impediment to traffic flow occurs. The cross flow may be caused by superelevation of a curve or a street intersection exceeding the capacity of the higher gutter on a street with cross fall. The problem associated with this type of flow is the same as for ponding in that it is localized in nature and vehicles may be traveling at high speed when they reach the location. If the velocity of vehicles is naturally slow and use is light, such as on local streets, cross-street flow does not cause sufficient interference to be objectionable.

The depth and velocity of cross-street flow should always be maintained within such limits that it will not have sufficient force to affect moving traffic. If a vehicle which is hydroplaning enters an area of cross street flow, even a minor force could be sufficient to move it laterally towards the gutter.

At certain intersections the flow may be trapped between converging streets and must either flow over one street or be carried underground. If the vehicles crossing the intersection are required to stop, then very little hazard exists to the traveling public. This is the basis for the assumption that valley gutters are acceptable across a local street where it intersects another local or collector street. Another point in favor of the use of valley gutters is the continuation of the grade of the dominant street. If the crown of the local street is allowed to coincide with the crown of the major street, the outside traffic lanes of the major street will have a "hump" at the intersection.

Effect on Pedestrians

In areas where pedestrians frequently use sidewalks, splash due to vehicles moving through water adjacent to the curb is a serious problem. It must also be kept in mind that under certain circumstances, pedestrians will be required to cross ponded water adjacent to curbs.

Since the majority of pedestrian traffic will cease during the actual rainstorm, less consideration need be given to the problem while the rain is actually falling. Ponded water, however, remaining after the storm has passed, must be negotiated by pedestrians.

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

3.20 Design Criteria

Design criteria for the collection and transport of runoff on public streets is based on a reasonable frequency of traffic interference. That is, depending on the street classification, certain traffic lanes can be fully inundated once during the minor design storm return period. For example, a local street flow is allowed to cover the crown during a 10-year frequency storm. During the 10-year period, lesser storms will occur which will produce less runoff and will not inundate the entire street.

Planning and design for urban storm runoff must be considered from the viewpoint of both the regularly expected storm occurrence, that is, the minor storm, and the major storm occurrence. The minor storm will have a frequency of one in 10 years. The major storm will have a return period of 100 years. The objectives of the major storm runoff planning and design is to eliminate major damage and loss of life. The minor drainage system is necessary to eliminate inconvenience, frequently recurring minor damage, and high street maintenance costs.

Street Capacity for Minor storms

Determination of street capacity for the minor storm shall be based upon pavement encroachment. The pavement encroachment for the minor storm shall be limited as set forth in Table 3-1.

TABLE 3-1

Allowable Minor Storm Runoff Encroachment

<u>Street Classification</u>	<u>Minor Storm Frequency</u>	<u>Maximum Encroachment</u>
Local	10-year	No curb over-topping.* Flow may cover crown of street.
Collector, Arterial	10-year	No curb over-topping.* Flow spread must leave at least one lane (12 ft) free of water for a two lane roadway and two lanes for a four lane roadway.

*Where no curbing exists, encroachment shall not extend over property lines except at drainage easements. Where double curb exists, flow shall not overtop the lower curb.

The storm sewer system should begin at the point where the maximum encroachment is reached. Development of the major drainage system is encouraged so that the minor runoff is removed from the streets, thus moving the point at which the storm sewer system must begin further downstream.

Calculating Capacity

When the allowable encroachment has been determined, the gutter (that portion of the street used to convey runoff) capacity shall be computed using the modified Manning's formula.

Gutter cross sections usually resemble a triangular shape with the curb forming the vertical leg of the triangle. The gutter may have a straight cross slope, a cross slope composed of two straight lines (V-shape), composite cross slopes or parabolic sections for older pavements.

The modified Manning equation is utilized in triangular channels to better describe the hydraulic radius of a gutter section. The Manning equation is integrated across the section width. The equation in terms of cross slope and width of flow on the pavement is:

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

where: Q = flow rate ft³/sec (m³/s)

T = width of flow (spread) ft (m)

S_x = cross slope, ft/ft (m/m)

S = longitudinal slope, ft/ft (m/m)

The resistance of the curb face is neglected in the equation since the resistance is negligible when the cross slope is 10 percent or less.

Figure 3-2 can be used for a direct solution of the above equation using Manning's n value of 0.016. For other values of n divide the value of Q_n by n. Manning's n values for different street and gutter roughness conditions are presented in Table 3-2. For the discharge of a composite cross slope, use Figure 3-2 and Figure 3-3.

Table 3-2
Manning's Roughness Coefficients
for Streets and Gutters

a.	Concrete gutter troweled finish	0.012
b.	Asphalt pavement	
	(1) Smooth texture	0.013
	(2) Rough texture	0.016
c.	Concrete gutter with asphalt pavement	
	(1) Smooth	0.013
	(2) Rough	0.015
d.	Concrete pavement	
	(1) Float finish	0.014
	(2) Broom finish	0.016
e.	Brick	0.016

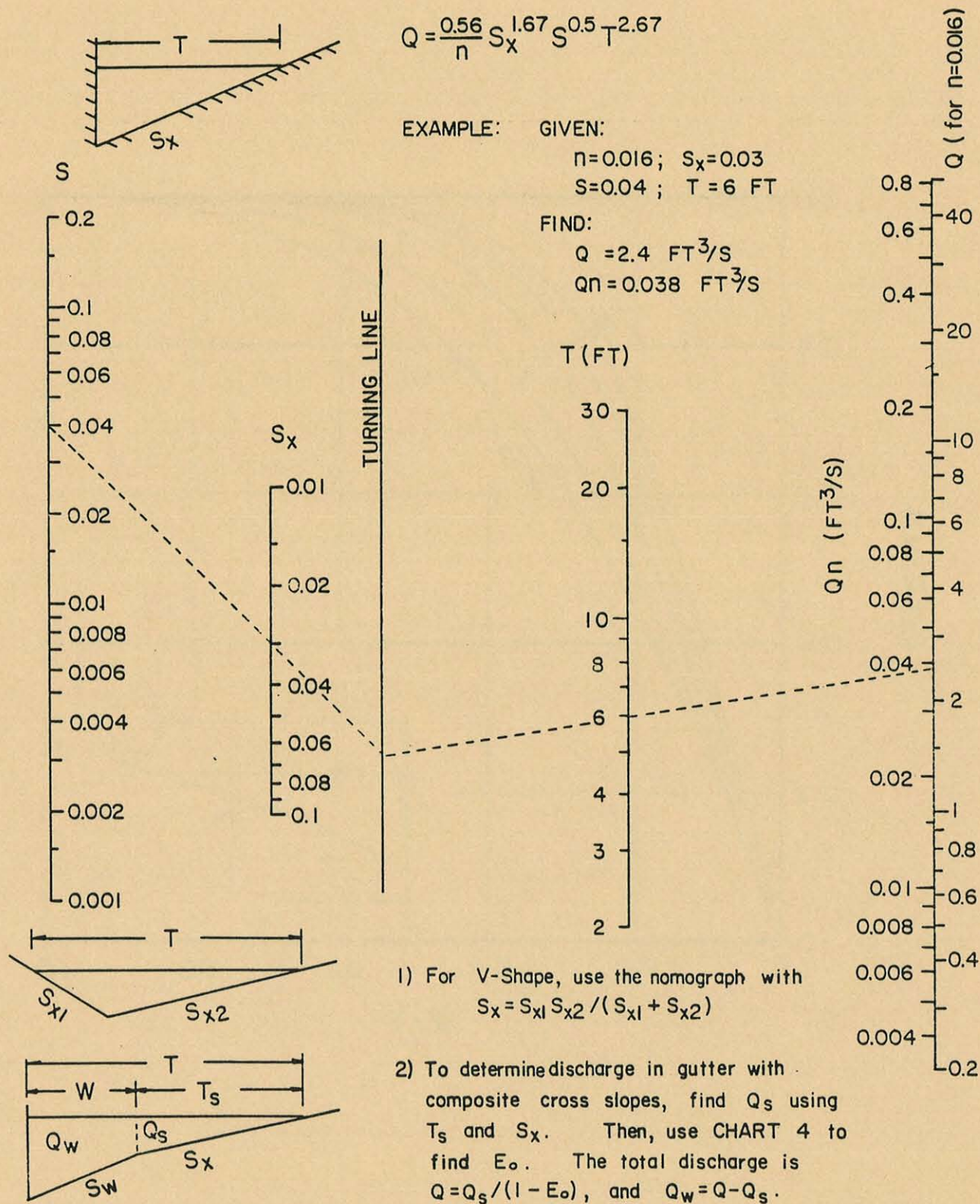


Figure 3-2 Flow in Triangular Gutter Section

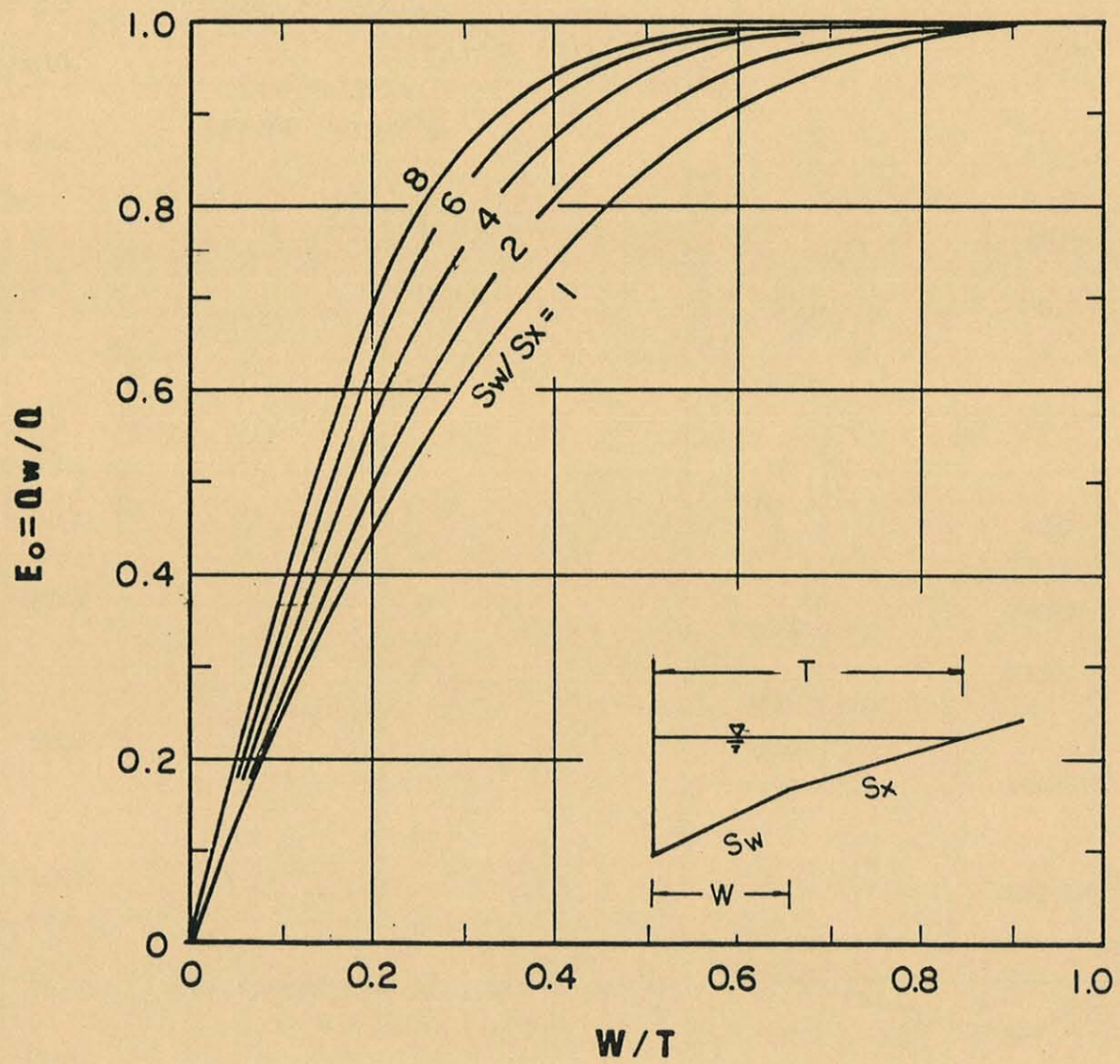


Figure 3-3 Ratio of Frontal Flow to Gutter Flow

Example

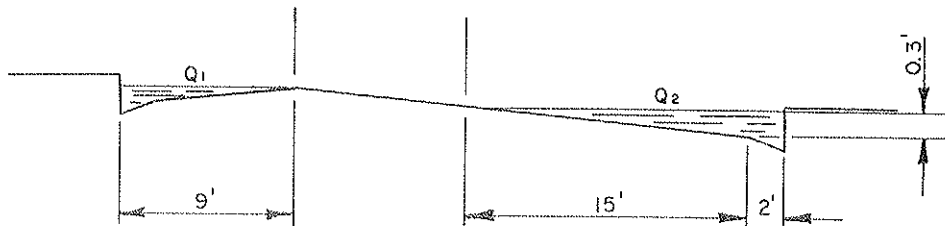
Gutter Carrying Capacity, Minor Storm

Given: 6" vertical curb
 2'-wide by 2"-deep gutter
 2% pavement crown slope
 36' street width, curb to curb
 Crown offset to 1/4 point for cross fall
 Collector street
 Street grade = 5%
 Assume $n=0.016$

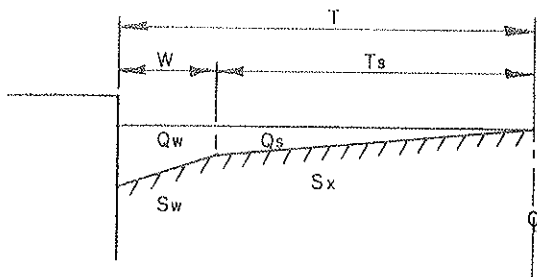
Find: Capacity, each gutter.

1. Determine allowable pavement encroachment.

From Table 3-1, one lane must remain open.



2. Calculate capacity for each gutter.



$$\begin{aligned} T &= 9 \text{ Ft.} & S_w &= 0.08 \\ W &= 2 \text{ Ft.} & S_x &= 0.02 \\ T_s &= 7 \text{ Ft.} & S &= 0.05 \end{aligned}$$

Using Nomograph, Figure 3-2.

$$T = T_s = 7 \text{ Ft.}$$

Therefore $Q_s = 2.1 \text{ cfs.}$

Using Figure 3-3

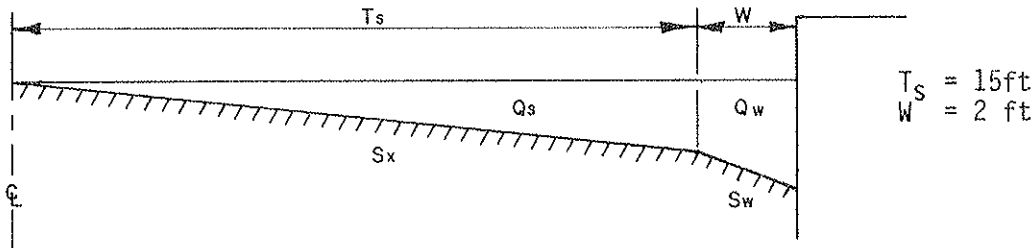
$$\frac{W}{T} = 0.22$$

$$\frac{S_w}{S_x} = 4.0$$

Therefore $E_0 = 0.62$

$$Q_t (\text{total}) = \frac{Q_s}{1 - E_0} = 5.53 \text{ cfs}$$

$$Q_w = Q_t - Q_s = 5.53 - 2.10 = 3.43 \text{ cfs}$$



Using nomograph, Figure 3-2

$$T = T_s = 15 \text{ Ft.}$$

Therefore $Q_s = 16 \text{ cfs}$

Using Figure 3-3

$$\frac{W}{T} = 0.12$$

$$\frac{S_w}{S_x} = 4.0$$

Therefore $E_0 = 0.37$

$$Q_t = 25.4 \text{ cfs}$$

$$Q_w = 9.4 \text{ cfs}$$

Street Capacity for Major Storm

Determination of the allowable capacity for the major storm shall be based upon allowable depth and inundated area. The allowable depth and inundated area for the major storm shall be limited as set forth in Table 3-3.

Calculating Capacity

When the allowable depth and inundated area as determined from Table 3-3, the street capacity shall be calculated using Manning's Formula with an "n" value applicable to the actual boundary conditions encountered.

TABLE 3-3

Allowable Major Storm Runoff Inundation

<u>Street Classification</u>	<u>Allowable Depth and Inundated Areas</u>
Local and Collector	Residential dwellings, public, commercial, and industrial buildings, shall not be inundated at the ground line, unless buildings are flood-proofed. The depth of water over the gutter flowline, or street centerline if crown is inverted, shall not exceed 18 inches.
Arterial	Residential dwellings, public, commercial, and industrial buildings, shall not be inundated at the ground line, unless buildings are flood-proofed. Depth of water at the street crown shall not exceed 6 inches to allow operation of emergency vehicles. The depth of water over the gutter flowline, or street centerline if the crown is inverted, shall not exceed 18 inches.

Ponding

The term ponding shall refer to areas where runoff is restricted to the street surface by sump inlets, street intersections, low points, intersections with drainage channels, or other reasons.

Minor storm

Limitations for pavement encroachment by ponding for the minor storm shall be those presented in Table 3-1, Allowable Minor Storm Runoff Encroachment. These limitations shall determine the allowable depth at inlets, gutter turnouts, culvert headwaters, etc.

Major Storm

Limitations for depth and inundated area for Major Storms shall be those presented in Table 3-3, Allowable Major Storm Runoff Inundation.

These limitations shall determine the allowable depth at inlets, gutter turnouts, culvert headwaters, etc.

Cross-Street Flow

Cross-street flow is classified into two general categories. The first type is runoff which has been flowing down the street in a gutter and when the gutter capacity is exceeded the flow crosses the street to the opposite gutter. The second type is flow from some external source, such a drainage way which is intersecting the street, which will flow across the crown of a street when the conduit capacity beneath the street is exceeded.

Depth

Cross-street flow depth shall be limited as set forth in Table 3-4.

TABLE 3-4

Allowable Cross-Street Flow

<u>Street Classification</u>	<u>Minor Design Runoff</u>	<u>Major Design Runoff</u>
Local	6-inch depth at crown	18 inches of depth above gutter flowline
Collector	Where valley gutters are allowed, depth of flow shall not exceed 6 inches.	18 inches of depth above gutter flowline
Arterial	None	6 inches or less over crown

Capacity

Based upon the limitations in Table 3-4 and other applicable limitations (such as ponding depth), the quantity of cross-street flow shall be calculated. Where allowable ponding depth would cause cross-street flow, the limitation shall be the minimum allowable of the two criteria.

3.30 Intersection Layout Criteria

The following design criteria are applicable at intersections of urban streets. Gutter capacity limitations covered above shall apply along the street, while this section shall govern at the intersection.

Gutter Capacity, Minor storm

Pavement Encroachment

Limitations at intersections for pavement encroachment shall be as given in Table 3-1, Allowable Minor Storm Runoff Encroachment.

Capacity

The capacity of each gutter approaching an intersection shall be calculated based upon the most critical cross section.

- a. Continuous Grade Across Intersection.
When the gutter flow will be continued across an intersection, the slope used for calculating capacity shall be that of the gutter flow line crossing the street. (Figure 3-4, Intersection Drainage).
- b. Flow Direction Change at Intersection.
When the gutter flow must undergo a direction change at the intersection greater than 45 degrees, the slope used for calculating capacity shall be the effective gutter slope, defined as the average of the gutter slopes at 0 ft, 25 ft, and 50 ft from the geometric point of intersection of the two gutters. (Figure 3-4, Intersection Drainage).
- c. Flow Interception by Inlet.
When gutter flow will be intercepted by an inlet on continuous grade at the intersection, the effective gutter slope shall be utilized for calculations. Under this condition, the points for averaging shall be 0 ft, 25 ft, and 50 ft upstream from the inlet (Figure 3-4, Intersection Drainage).

Gutter Capacity, Major Storm

Allowable Depth and Inundated Area

The allowable depth and inundated area for the major storm shall be limited as set forth in Table 3-3.

Capacity

The carrying capacity of each gutter approaching an intersection shall be calculated, based upon the most critical cross section.

The grade used for calculating capacity shall be as covered in Gutter Capacity, Minor storm.

Ponding

Minor storm

The allowable pavement encroachment for the initial storm shall be as presented in Table 3-1.

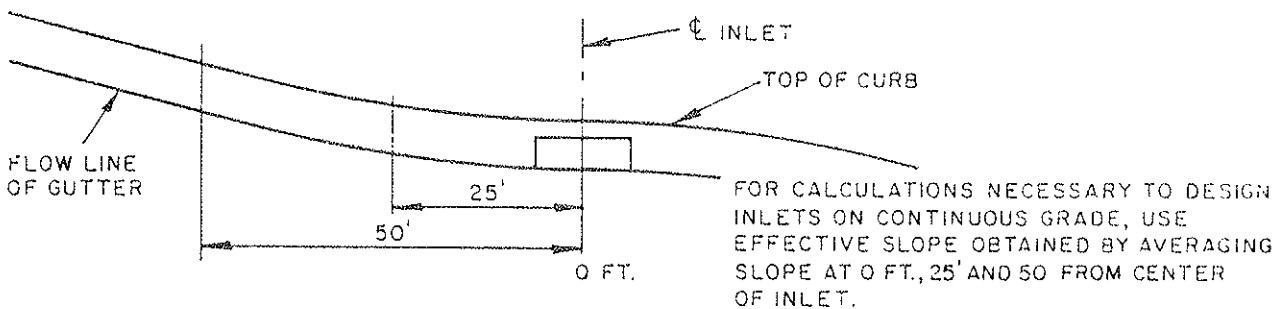
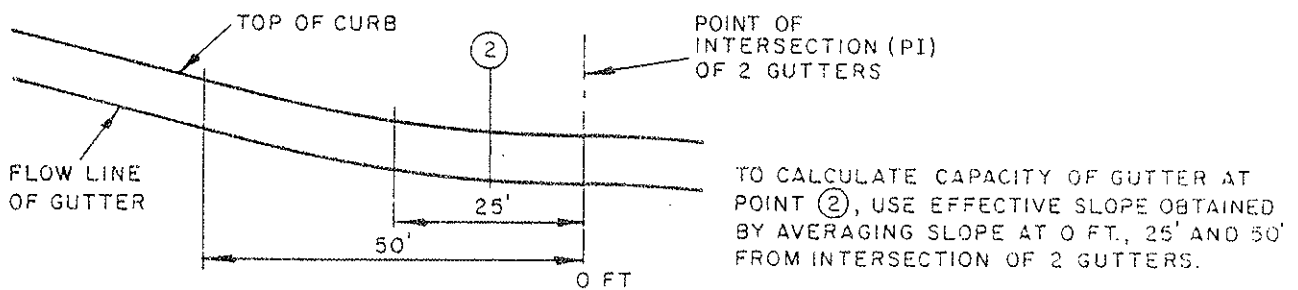
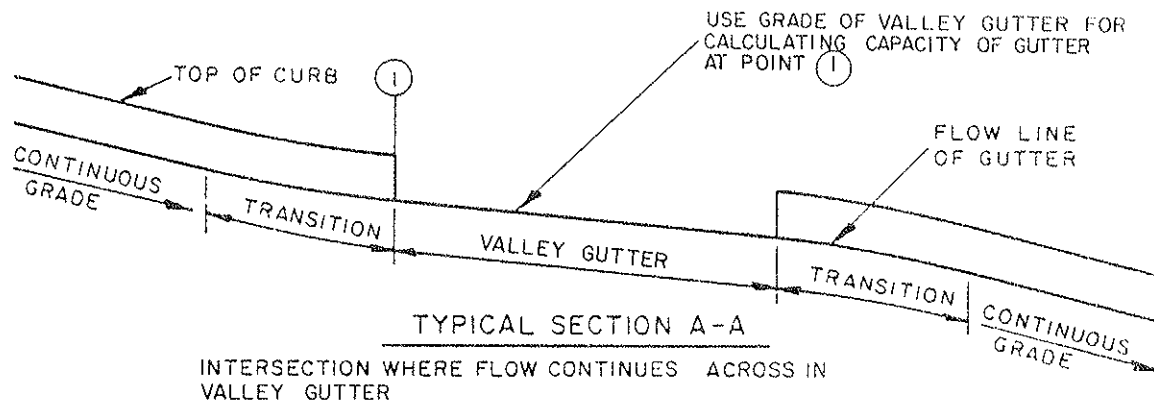
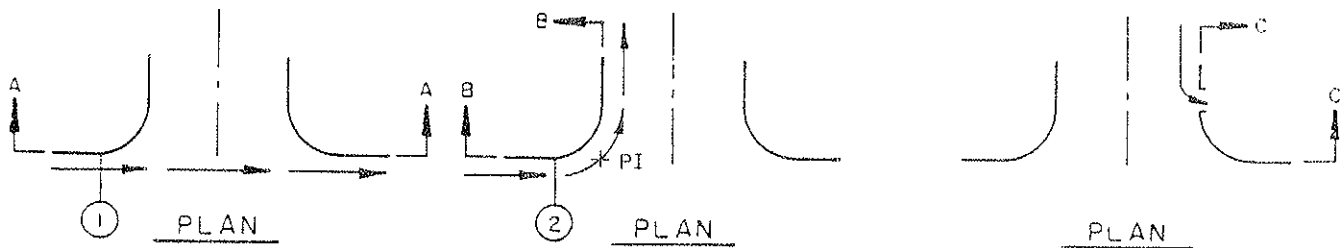


Figure 3-4 Intersection Drainage

Major Storm

The allowable depth and inundated area for the major storm shall be as presented in Table 3-3.

Cross-Street Flow

Depth

Cross-street flow depth at intersections shall be limited as set forth in Table 3-4.

Capacity

Where cross-street flow will be conveyed across a local or collector street, the cross-sectional area used for calculations shall be along the centerline of the local street. The slope shall be the slope of the valley gutter at that point.

3.40 Bibliography

1. Izzard, C.F., Hydraulics of Runoff from Developed Surfaces, Proc. Highway Reserach Board, Vol. 26, 1946.
2. Johnson, Frank L., Chang, Fred F.M., Drainage of Highway Pavements, HEC No. 12, Federal Highway Administration, March 1984.
3. National Research Council, Building Research Advisory Board, A Study of Invert Crown Residential Streets and Alleys, Federal Housing Administration, Washington, D.C., 1957.
4. Oglesby, Clarkson H., Highway Engineering, 3rd ed., John Wiley and Sons, Inc., New York, NY, 1975.
5. Searcy, J.K., Design of Roadside Drainage Channels, U.S. Bureau of Public Roads, Hydraulic Design Series No. 4, U.S. Government Printing Office, Washington, D.C., 1965.
6. Wright-McLaughlin Engineers, Urban Storm Drainage Criteria Manual, Denver Regional Council of Governments, CO, 1969.

4 Storm
Inlets

Section 4

Storm Inlets

- 4.10 Inlet Types
- 4.20 Inlets on a Grade
- 4.30 Comparison of Inlet Types
- 4.40 Grate Inlets
 - Capacity of Grate Inlets on a Continuous Grade
 - Capacity of Grate Inlets in a Sag
- 4.50 Curb-Opening Inlets
 - Capacity of Curb-Opening Inlets on a Continuous Grade
 - Composite Section
 - Explanation of Operation of Inlet
 - Computation by Electronic Calculator
 - Checking for Greater Storms
 - Capacity of Curb-Opening Inlets in a Sag
- 4.60 Combination Inlets
- 4.70 Inlet Location
 - Spacing of Inlets on a Continuous Grade
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- 4.80 Definition of Symbols for Section 3 and 4
- 4.90 Bibliography

Section 4

Storm Inlets

The hydraulic capacity of a gutter inlet depends upon its geometry and upon the characteristics of the gutter flow. The inlet capacity governs both the rate of water removal from the gutter and the amount of water that can enter the storm drain system. Many costly storm drains flow at less than design capacity because the storm runoff cannot get into the drains. Inadequate inlet capacity or poor inlet location may cause flooding on the traveled way which creates a safety hazard or at times interrupts traffic.

In addition to its hydraulic function, the inlet is frequently located in or near the path of vehicular traffic. Water-borne debris and trash may be deposited on the inlet causing complete or partial clogging. Often freedom from clogging and noninterference with traffic requires an inlet of a specific type rather than the most efficient inlet from a hydraulic point of view. For example, a curb-opening inlet might be used where a grate inlet would be more efficient.

See Section 4.8 for definitions of symbols.

4.10 Inlet Types

Gutter inlets can be divided into three major classes, each with many variations. These classes are (1) curb-opening inlets, (2) grate inlets, and (3) combination inlets (See Figure 4-1). Each type of inlet shall be installed with a depression of the gutter and may be a single or a multiple inlet (two or more closely spaced inlets acting as a unit). Two identical units placed end to end are called double inlets.

A brief description of the inlet types follow:

1. Curb-opening inlets. These inlets consist of a vertical opening in the curb through which the gutter flow passes.
2. Grate inlets. These inlets consist of an opening in the gutter covered by one or more grates.
3. Combination inlets. These units consist of both a curb-opening and a grate inlet acting as a unit.

4.20 Inlets on a Grade

The term inlet capacity is used here to mean the catch of the inlet under a given set of conditions rather than the maximum water that can be intercepted by the inlet if the discharge is increased without limit. The efficiency of an inlet is the discharge intercepted by the inlet, Q_1 , divided by the flow in the gutter, Q . The discharge that bypasses the inlet, Q_c , is termed carry-over.

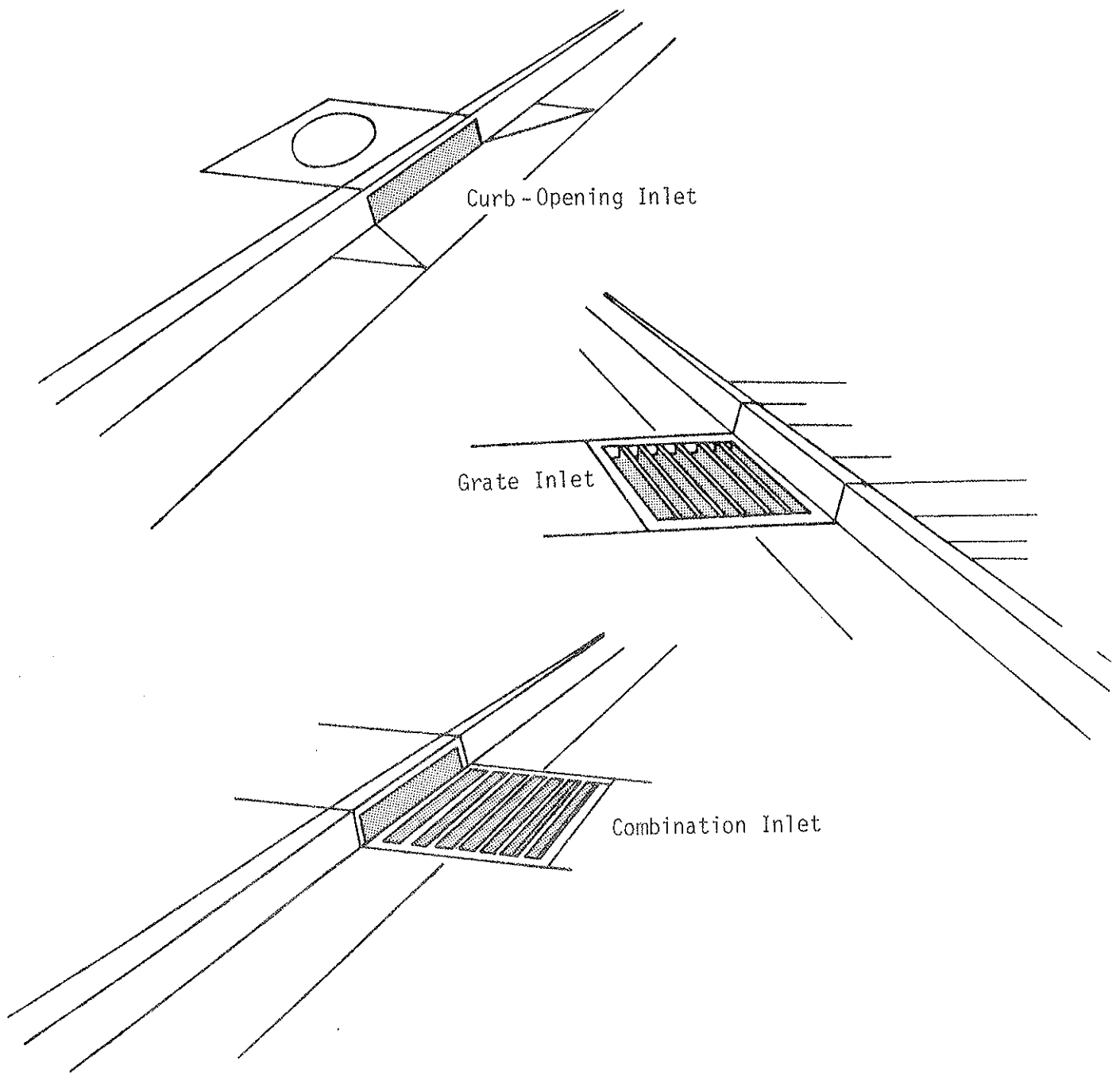


Figure 4-1 Inlet Types

A major factor in the capacity of a curb-opening inlet is the depth of water in the gutter immediately adjacent to the opening. The capacity of an efficient grate inlet depends principally upon the quantity of water flowing in the section formed by projecting the grate width upstream. Thus, an increase in longitudinal slope reduces inlet capacity (except for some grates or where deflectors are used) while an increase in transverse (cross) slope increases inlet capacity. Increase in length of a curb-opening inlet and increases in width of a grate opening increases the capacity of the inlet. For grate inlets, the efficiency of the grate opening is an important factor in inlet capacity.

For a curb-opening inlet, depressing the gutter increases the capacity of the inlet. The amount of the depression has more effect on the capacity than the arrangement of the depressed area with respect to the inlet.

Most investigators have pointed out that the capacity of an inlet is greatly increased by allowing a small percentage of the flow to bypass the inlet. For a given gutter discharge, the catch of each additional increment of width (grate inlets) or length (curb-opening inlets) becomes rapidly less. Thus, the cost of catching the small amount of flow near the thin edge of the triangular flow channel approaches the cost of catching the greater amount flowing nearer to the curb.

4.30 Comparison of Inlet Types

A curb-opening inlet generally requires a larger structure than a grate inlet of equal capacity but the curb opening is located back of the curb line and offers little interference with roadway traffic.

Figure 4-2, plotted from data in Reference 2, gives a comparison of the capacity of several inlet types at about the maximum cross slope recommended for low-type surfaces. These curves are for an inlet efficiency of 95 percent and apply to a particular condition. They are presented for reference in the discussion which follows and should not be used for design curves.

An undepressed curb-opening inlet (Curve 1) has less capacity than a depressed curb-opening inlet (Curve 2). Curb-opening inlets lose capacity rapidly with increase in longitudinal grade. Grate inlets generally lose capacity with increase in grade but to a lesser degree (Curve 5). A very efficient grate designed at Johns Hopkins University (Curve 4) gains in capacity with increase in grade. Deflector vanes (Curve 3) increase the capacity of curb-opening inlets with increase in grade. A combination inlet (Curve 6) without depression has slightly greater capacity than the grate inlet alone (Curve 4). Changes in cross slope affect the capacity of a curb-opening inlet much more than the capacity of a grate inlet.

The choice of inlet cannot always be made upon capacity alone. Debris carried by the gutter flow and interference with vehicular traffic must also be considered. Curb-opening inlets are relatively free of debris clogging while grate inlets have a tendency to clog and might clog completely where debris is a problem. Combination inlets are better than grate inlets alone

INTAKE CAPACITY AT 95 PERCENT CAPTURE OF GUTTER FLOW

Manning's $n = 0.013$

Cross slope = .0417 ft per ft

- | | |
|---------|--|
| Curve 1 | Curb Opening, no depression $L=10$ ft |
| Curve 2 | Curb Opening, 2-1/2-inch depression, $L=10$ ft |
| Curve 3 | Curb Opening, 3-ft-wide deflector, $L=8.33$ ft |
| Curve 4 | Grate, no depression, $W=2.5$ ft, $L=2.5$ ft |
| Curve 5 | Grate, 2-1/2-inch depression, $W=2.5$ ft, $L=2.5$ ft |
| Curve 6 | Combination, no depression |
| Curve 7 | Combination, 2-1/2-inch depression |

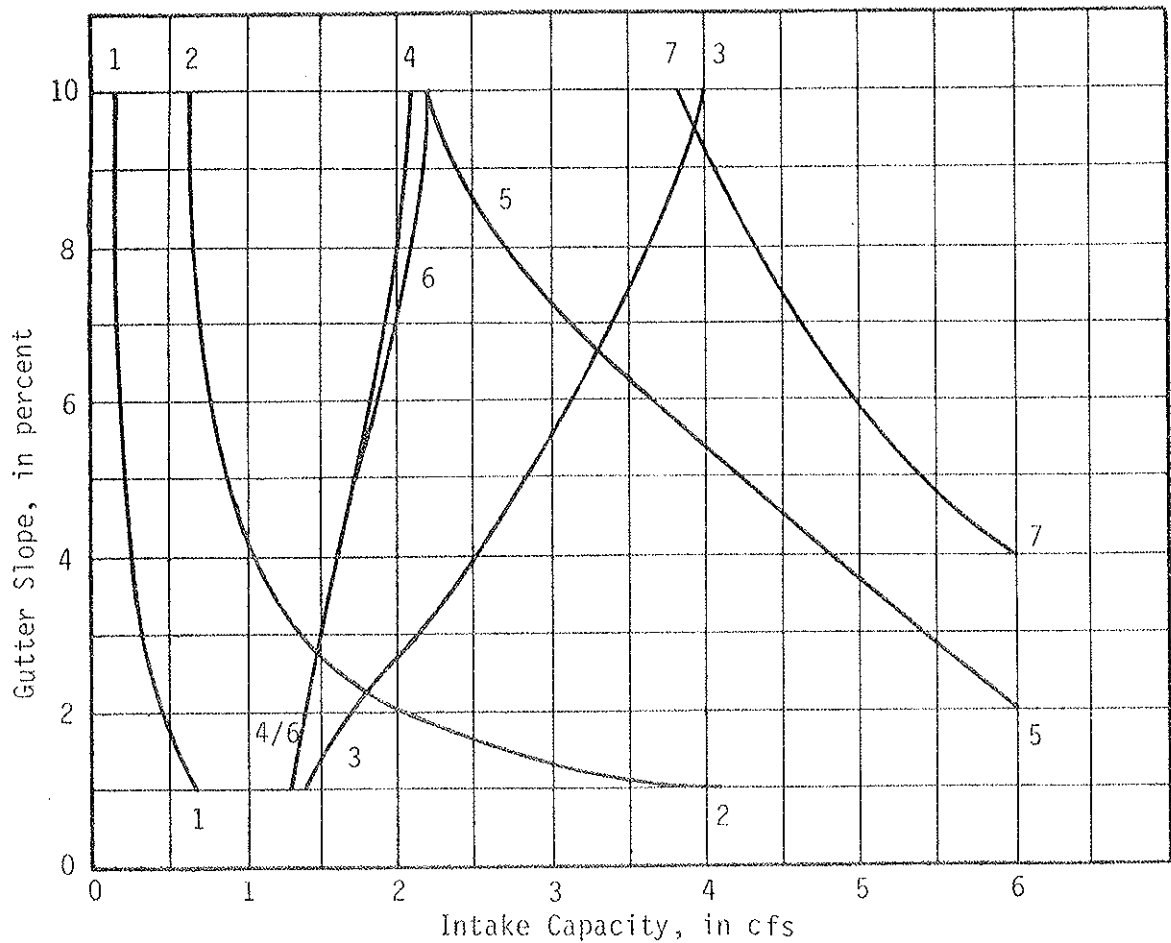


Figure 4-2 Comparison of Inlets

where debris is prevalent. From a water quality standpoint, the elimination of debris from the storm sewer system can be achieved by the use of grate inlets.

Grates with bars parallel to the flow, while much more efficient hydraulically, are hazardous to bicyclists if spacing between bars is wide. Curb-opening inlets with vertical openings greater than 5 inches create a potential hazard to pedestrians. The Johns Hopkins tests found that a few small rounded crossbars installed at the bottom of the longitudinal bars as stiffeners or as a safety stop for bicycle wheels, did not materially affect the capacity of the grate.

4.40 Grate Inlets

Grate inlets are best suited for areas where traffic does not travel close to the curb and where clogging with debris is not a problem. When the longitudinal grade exceeds about one percent, grates with rectangular longitudinal bars (parallel to flow) are much more efficient than grates with transverse (normal to flow) rectangular bars. As the grade increases, longitudinal bar grates become increasingly superior to transverse bar grates both in regard to interception of flow and decrease in tendency to catch debris. For example, the Johns Hopkins test found that on a 10 percent street grade with cross slope 1:18 and gutter flow of 3 cfs, a gutter inlet with longitudinal bars intercepted 2.7 cfs as compared with 1.9 cfs interception by a grate with transverse bars. However, some cross-bar grates of special design with bars curved in the vertical plane have been found in manufacturers tests to intercept more of the flow than longitudinal bar grates. A direct comparison between manufacturer's tests and those reported in the references cannot be made because of differences in testing conditions. In general, an unclogged, efficient, grate inlet on a continuous grade will intercept all water flowing within a width equal to that of the grate.

For an efficient grate inlet, all rectangular bars should be parallel with the flow and the openings should cover at least 50 percent of the width of the grate. The clear length of the opening should be sufficient to allow the water to fall through the openings without striking the far end of the grate. The required clear length of the bar (L_b) can be computed by the experimentally determined formula:

$$L_b = \frac{V}{2} (d + d_b)^{0.5} \quad (4-1)$$

where:

L_b = length of clear opening of grate, in feet;

V = mean approach velocity in the width of the grate opening, in feet per second;

d = depth of flow at the curb, in feet; and

d_b = depth of the bar, in feet.

Figure 4-3 is a graphical solution of the above equation for pavement with roughness coefficient $n = 0.015$ and depth of bar 3-1/2 inches. The effect of gutter slope on the length of clear opening needed for an efficient grate is shown by Example 1.

Example 1

Given: Pavement and gutter cross slope, $S_x = 1/4$ inch per foot (0.021 foot per foot); 2-foot concrete gutter; $n = 0.015$; allowable spread on pavement, 6 feet (total width of gutter, 8 feet); depth of grate bars, 3-1/2 inches.

Find: Length of clear opening of grate bars required for an efficient grate when longitudinal slope (S_o) is one percent; 10 percent.

Solution:

1. Use Figure 4-3. On upper left diagram for $T = 8$ feet move horizontally to S_x curve, 0.021.
2. From intersection in Step 1 move vertically downward to S_x curve, 0.021 in left center diagram.
3. From intersection in Step 2, move horizontally to the right diagram and read L_b on the bottom scale. At the intersection of the curve for $S_o = 0.01$, $L_b = 1.1$ feet, and at the intersection of curve for $S_o = 0.10$, $L_b = 3.0$ feet.

Example 1 shows that as the gutter slope varies so does the length of clear opening required for 100 percent interception of the water approaching the inlet within the width of the grate. A practical solution might be the adoption of several grates with different lengths of clear openings. Each length of grate would have a range of slopes for which it would be the most appropriate grate to use. If the length of clear opening is less than given by the above equation, some of the gutter flow will pass directly over the grate without being intercepted and add to the flow bypassing the grate.

Capacity of Grate Inlets on a Continuous Grade

The capacity of an undepressed efficient grate inlet can be determined by computing the flow in the section occupied by the grate width.

For straight-line sections, the flow intercepted by an undepressed efficient grate can be computed as explained in Figure 3-2. Example 2 illustrates the procedure.

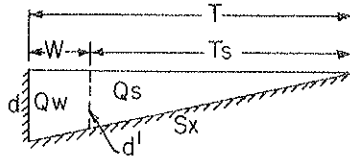
Example 2

Given: $Q = 1.5$ cfs; pavement cross slope, $S_x = 1/4$ inch per foot; $n = 0.016$; longitudinal slope = 2 percent; grate inlet, 30 inches wide by 24 inches long; depth of longitudinal bars, 3 inches; no cross bracing; 60 percent clear opening.

Find: Spread on the pavement and the discharge intercepted by the grate inlet.

Solution:

- Figure 3-2, lay a straight end on $S=0.02$ and $S_x=0.021$. Mark intersection on the turning line.



- Lay straight end on turning point and $Q=1.5$ cfs. Spread or $T = 7.2$ ft.
- Depth of flow at curb $= d = TS_x$
 $d = 0.021 (7.2) = 0.15$ ft
- Depth of flow at outward edge of grate ($W=2.5$ ft)
 $d_1 = (T-W)S_x ; (7.2-2.5) 0.021 = 0.10$ ft.
- From Figure 3-2 using composite sections in which $S_w=S_x=0.021$ and $T = T_s=4.7$ ft; read $Q_s = 0.48$ cfs
- $Q_w = Q - Q_s$
 $= 1.5 - 0.48 = 1.02$ cfs, provided that the grate is efficient.
- The grate has the necessary requirements for efficiency (Section 4.4) if the clear opening L_b is sufficient. The mean velocity in the 2.5-foot section over the grate equals the discharge over the grate divided by the area of the section (0.15 ft. deep at the curb and 0.10 ft. deep at the outer edge of the grate, and a width of 2.5 ft).

$$V = \frac{1.02 \text{ cfs}}{0.31 \text{ ft}^2} = 3.3 \text{ fps}$$

The clear opening required by equation (4-1) is

$$L_b = \frac{V}{2} (d+d_b) 0.5 = \frac{3.3}{2} (0.15+0.25) 0.5 = 1.0 \text{ ft}$$

The grate opening, 2.0 feet, exceeds that required; therefore, the grate is efficient.

The 2.5-foot grate intercepts 68 percent of the total flow, $1.02/1.5 = 0.68$, in Example 2. To intercept 100 percent of the flow a grate 7.2 feet wide is required, almost three times the width required for 67 percent interception.

Ordinarily, the width of the grate remains constant and the spacing of inlets are varied to limit the spread on the pavement to the desired quantity. The calculations for determining the discharge for 100 percent interception by a given grate are shown in Example 3.

Example 3

Total Interception by Undepressed Grate

Given: The same conditions as in Example 2.

Find: The gutter flow (Q) that would be totally intercepted by an efficient grate inlet, 30-inches wide.

Solution:

1. The spread on the pavement (T) is limited to the grate width (2.5 feet) for 100 percent interception.

$$d = TS_x = 2.5 (0.021) = 0.05 \text{ ft}$$

2. Using figure 3-2 and composite section flows with $T = W = 2.5$ ft., $S = 0.02$ and $S_x = 0.021$ the intersection of the flow line is off the graph. Using the equation:

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

$$Q_w = 0.09 \text{ cfs}$$

Examples 2 and 3 show the advantage of designing grate inlets for partial interception. For 100 percent interception under the conditions of these examples, the grate width would have to be tripled, or about 17 separate 30-inch grate inlets, each intercepting 0.09 cfs, would be required.

For composite straight-line gutter sections, the flow intercepted by an undepressed efficient grate is computed as explained in Figure 3-2.

Capacity of Grate Inlets in a Sag

A grate inlet in a sag operates first as a weir having a crest length roughly equal to the outside perimeter (P) along which the flow enters. Bars are disregarded and the side against the curb is not included in computing P. Weir operation continues to a depth (d) of about 0.4 feet

above the top of grate and the discharge intercepted by the grate is:

$$Q_i = 3.0 P d^{1.5} \quad (4-2)$$

where:

Q_i = rate of discharge into the grate opening, in cubic feet per second;

P = perimeter of grate opening, in feet, disregarding bars and neglecting the side against the curb; and

d = depth of water at grate, in feet.

When the depth at the grate exceeds about 1.4 feet, the grate begins to operate as an orifice and the discharge intercepted by the grate is:

$$Q_i = 0.67A (2gd)^{0.5} = 5.37Ad^{0.5} \quad (4-3)$$

where:

Q_i = rate of discharge into the grate opening, in cubic feet per second;

A = clear opening of the grate, in square feet;

g = acceleration of gravity, 32.2 feet per second²; and

d = depth of ponded water above top of grate, in feet.

Between depths over the grate of about 0.4 feet and about 1.4 feet the operation of the grate inlet is indefinite due to vortices and other disturbances. The capacity of the grate is somewhere between that given by the above equations.

Because of vortices and the tendency of trash to collect on the grate, the clear opening or perimeter of a grate inlet should be at least twice that required by the equations in order to remain below the design depth over the grate. Where the danger of clogging is slight, a factor of safety less than two might be used. If a combination inlet is used, the grate need only be as large as given by the equations because the curb opening provides the safety factor from clogging.

These equations are solved graphically with Figure 4-4. The dashed lines on this figure represent the range where neither weir nor orifice operation is fully effective. Example 4 illustrates the procedure.

Example 4

Given: Grate inlet, 2 feet by 2.5 feet with 50 percent clear opening, located in a sag with one 2-foot side against the curb.

Find:

1. Depth of water when $Q = 0.9$ cfs; and
2. Depth of water when $Q = 20$ cfs.

Solution:

1. Compute perimeter of grate opening (P) ignoring the bars and omitting any side over which the water does not enter, such as when one side is against the face of a curb. Divide the result by 2 to allow for partial clogging of the grate.

$$P = 2 + 2.5 + 2.5 = 7.0 \text{ feet}$$

$$\text{Effective } P = \frac{7}{2} = 3.5 \text{ feet}$$

2. Compute $\frac{Q}{P}$ ratio, using effective perimeter.

$$(1) \frac{0.9}{3.5} = 0.26$$

$$(2) \frac{20}{3.5} = 5.71$$

3. Compute the total area of clear opening (A), excluding area taken up by bars, and divide by 2 to allow for partial clogging of the grate.

$$\text{Clear opening} = .50 (2 \times 2.5) = 2.5 \text{ square feet}$$

$$\text{Effective area} = \frac{2.5}{2} = 1.25 \text{ square feet}$$

4. Compute $\frac{Q}{A}$ ratio, using effective area

$$(1) \frac{0.9}{1.25} = 0.72$$

$$(2) \frac{20}{1.25} = 16.0$$

5. Enter Figure 4-4 at the abscissa, using curve A with $\frac{Q}{P}$ values up to 3.0 feet and curve B with $\frac{Q}{A}$ values above 3.0 feet, and read the depth, in feet, of ponding over the grate.

(1) When $Q = 0.9$ cfs, $\frac{Q}{P} = 0.26$ and $\frac{Q}{A} = 0.72$,

using $\frac{Q}{P}$ and curve A, $d = 0.20$ feet.

(2) When $Q = 20$ cfs, $\frac{Q}{P} = 5.71$ and $\frac{Q}{A} = 16.0$

Using $\frac{Q}{A}$ and curve B, $d = 8.9$ feet

If the grate has appreciable cross slope so that the side away from the curb is higher than the curb side, the inflow over the side should be determined separately from that over the ends. In weir control, use the depth at the middle of the grates for end inflow and the depth away from the curb for side inflow.

4.50 Curb-Opening Inlets

Curb-opening inlets are used in many locations because they offer little interference to traffic and are relatively free from clogging by debris.

The curb-opening inlet discussed in this section is illustrated in Figure 4-5. It has a depression beginning W feet away from the curb and dropping 1 inch per foot below the plane of the pavement. Transitions at the two ends extend W feet from the end of the opening. The height of the opening need not be more than 4 inches since the water surface draws down as it accelerates on the depression apron. The opening should not exceed 5 inches in height in order to prevent a potential pedestrian hazard. The equations given apply only if the cross section of the street has a uniform slope to the face of the curb.

Capacity of Curb-Opening Inlets on a Continuous Grade

The operation of a curb-opening inlet on a grade is usually described in terms of the ratio of the flow intercepted Q_i to the approach flow Q which extends a distance T from the curb face. Q_i/Q can be defined in a dimensionless plot against $L_i/(F_w T)$ where L_i is the length of the inlet opening and F_w is the Froude Number related to the depth of the approach flow at a distance W from the curb (See Figure 4-6.) This is along a line at the outer edge of the inlet depression. The Froude Number is a ratio of the kinetic energy to the gravity force acting on the flow in the gutter.

Figure 4-6 is drawn for a cross slope, $S_x = 0.015$ and $W = 2$ ft. Note that $Q_i/Q = L_i/L_1$ up to the point where the parameter $L_i/F_w T = 0.4$. Beyond that point the relationship changes abruptly to a curved line for which $Q_i/Q = (L_i/L_3)^{0.4}$. L_1 is the value of L_i where the straight line intersects $Q_i/Q = 1.0$, while L_3 is the value of L_i where the curved line intersects $Q_i/Q = 1.0$. L_2 is the value of L_i at the breakpoint between the straight and curved lines.

From the diagram, it is apparent that if we know the value of $F_w T$ and L_i , the value of Q_i/Q can be found from the ordinate scale. Remember that this

diagram is for specific values of S_x and W . The position of the S_x line varies with these variables in accordance with the equations on Figure 4-6 while the position of the curved line remains fixed. Solutions for Q_i/Q may be read from Figure 4-7 for $W = 2$ feet or may be computed using an electronic calculator and the equations tabulated in the examples in Table 4-1. This Table also gives the equations for F_w and Q in terms of the cross-section variables.

Use of Figure 4-7

An example in dotted lines illustrates the use of Figure 4-7. The depth of flow d_w in the street section at a point W ft from the curb face is the starting point.

1. Enter with S_x (T-2) at top.
2. Follow down to line for n , normally 0.016.
3. Move across to longitudinal slope, S .
4. Follow down to spread, T ; this establishes a horizontal line.
5. If Q_i/Q is given, enter with Q_i/Q (upper right) follow across to Limit Line A, or line for S_x , whichever is intersected first.
6. Move down to lower margin ($Q_i/Q = 0.1$) and then diagonally to intersection with line from Step 4.
7. Follow down to length of inlet, L_i .
8. The horizontal line can be extended to line L_3 for 100% interception, then down to L_3 .
9. If length of inlet is given, enter with that length, move up to horizontal line established in Step 4, diagonally to $Q_i/Q = 0.1$, then vertically to S_x (or line A) and across to Q_i/Q .

Composite Section

It is a common practice to build gutters with steeper cross slopes than the pavement. This increases the depth of flow at the curb and the discharge for a given spread. Assuming that the width of the gutter is the same as the width W of the inlet depression, it is probable that the increment in flow caused by the steepened gutter cross slope will be picked up by the inlet. On that basis a family of curves has been calculated by the method given in Figure 3-2 to give the ratio of the increased flow to the flow in the straight section in Figure 4-8. To use this figure, first estimate Q by methods described previously, then, knowing S_x and T , read Q_i/Q on the ordinate scale and multiply by Q to obtain the increment in discharge to be added to Q_i .

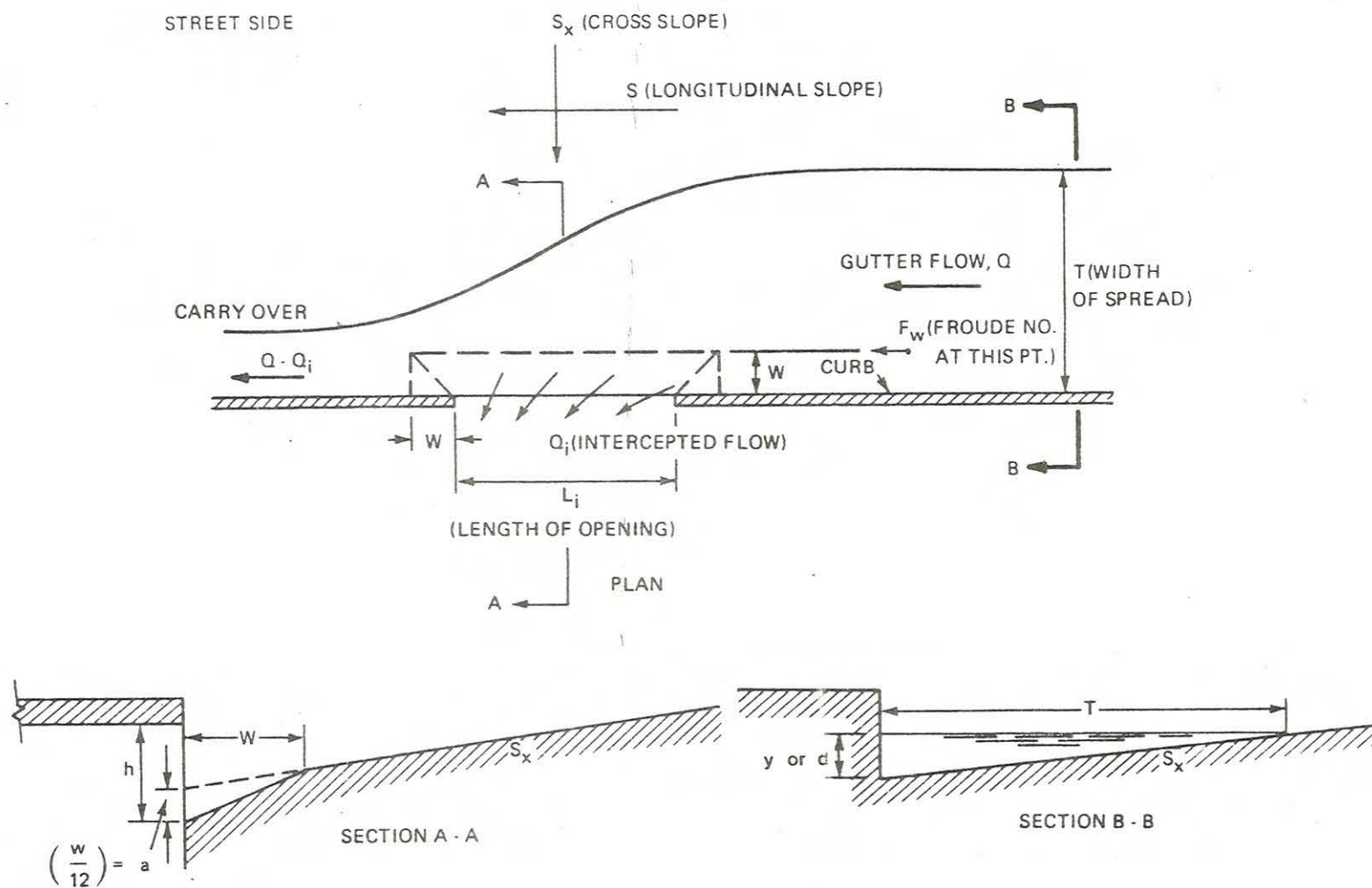


Figure 4-5 Graphical Definition of Symbols

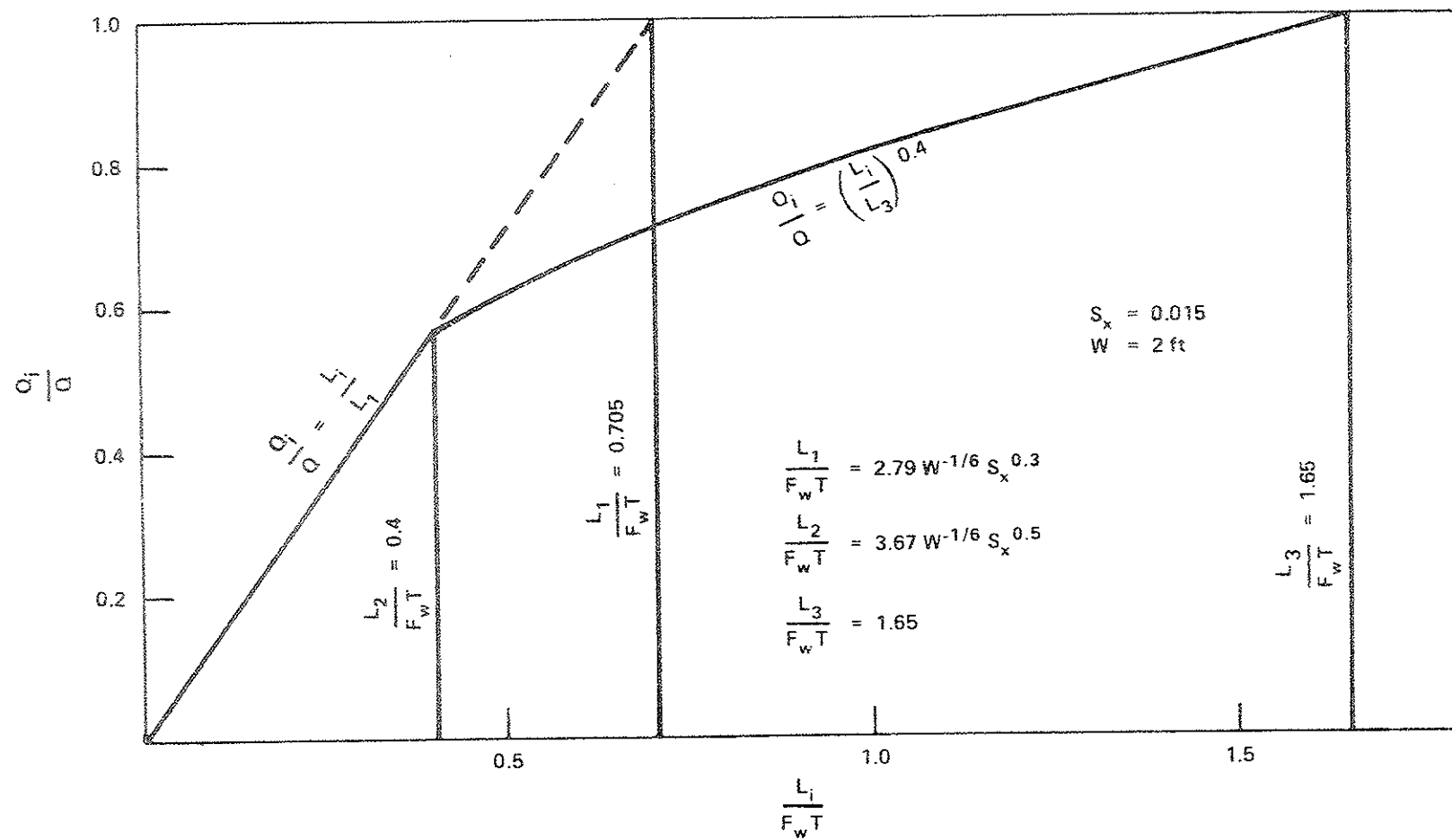


Figure 4-6 Dimensionless Graph of Q_i/Q vs. $L_i/F_w T$

Explanation of Operation of Inlet

It may be helpful to discuss the physical significance of the dimensionless curves in Figure 4-6. If we think of $F_W T$ as a constant for a given case of flow, then the abscissa scale is the length of inlet divided by that constant. We could recalibrate the scale to read directly in feet for inlet length. For short inlets, up to the length L_2 where the curve breaks, the inlet is acting as a weir. In fact, the flow intercepted is practically the same as would be intercepted by the same inlet at a sump, using the modified weir equation for that case. The major part of the flow is intercepted (60% or more depending on S_x) up to the length L_2 . For greater lengths of inlet, the remainder of the flow moves in gradually as indicated by the lesser increments of Q_i as length increases.

Computation by Electronic Calculator

For those who prefer using an electronic calculator, Table 4-1 illustrates the sequence of steps. As a rule, the designer will be working with a standard inlet and cross section for which S_x , n and W are fixed. The heading Equations (1), (2), and (3), taken from those in Figure 4-6, reduce to the numerical coefficients in the heading of Columns (9), (10), and (11).

On the first line, Inlet 1, the designer is to find the inlet length required for 100% interception on a 1% grade with $T = 10$ ft. The starred numbers represent the required criteria. Column 3 is used if there are a succession of grades for which Q is computed by Equation (5). Similarly Column 6 is for F_W computed by Equation (4). Multiplying $F_W T$ in Column 8 by the coefficients in the headings of Columns 9, 10, and 11 gives characteristic lengths L_1 , L_2 , and L_3 . As stated $Q_i/Q = 1$, so Q_i in Column 13 equals Q . Q_2/Q for the standard conditions is simply $0.462/0.770 = 0.600$, as recorded in the heading for Column 14. Either Column 15 or Column 16 is used to record L_i depending on whether Q_i , Q_2 or Q_2 (or in the case where L_i is given, L_1 , L_2 or L_2). In this case Q_i , Q_2 , so L_i is computed by Equation (8). Since $Q_i/Q = 1$, then $L_i/L_3 = 1$ and $L_i = 20$ as taken from Column 11. Actually, for 100% interception one may go directly from Column 11 to Column 16. Column 17 records the selected length L_1 , usually as a multiple of 2 feet depending on design standards. In the next example the computed length 11.4 becomes 12. If desired Q_i can then be recomputed by Equation (7) or (8) in Column 18 and subtracted from Q to give the carry-over discharge Q_c .

In the next three examples the independent variables are the same as the first example except that $Q_i/Q = 0.8$. For inlet 1, the required length reduced to 12 feet with $Q_c = 0.44$ cfs. Assuming the increment in runoff for the next subwatershed is the same as for inlet 1, the second inlet will then have $Q = 2.39 + 0.44 = 2.83$ cfs. This requires a recomputation of T as $(2.83/2.39)^{3/8} 10 = 10.7$ feet in accordance with Equation (5). L_3 changes slightly to 20.7. It is now assumed that the same size of inlet will be used again, so Q_i/Q is computed as $(12/20.1)^{0.4} = 0.81$ making $Q = 0.81 (2.83) = 2.30$ cfs and $Q_c = 0.53$ cfs. For inlet 3, again using 12 ft, the adjusted value of T becomes 10.8 and $L_3 = 20.3$ which leaves $Q_i/Q = 0.81$ and $Q_i = 2.37$ cfs. The flow intercepted has now become substantially equal to the increment in runoff for the intervening watershed.

Table 4-1
Computations for Curb - Opening Inlets

STANDARD SECTION
W=2 ft.
n=0.016
S_x=0.02

$$(1) L_1 = 2.49 S_x^{0.3} F_w T$$

$$(2) L_2 = 3.27 S_x^{0.5} F_w T$$

$$(3) L_3 = 1.65 F_w T$$

$$(4) F_w = 16.4 \left[(T-2) S_x \right]^{1/6} S_o^{1/2}$$

$$(5) Q = 35 S_x^{5/3} T^{8/3} S^{1/2} = .0515 T^{8/3} S_o^{1/2}$$

$$(6) T = \left[\frac{Q}{35 S^{1/2}} \right]^{3/8} S_x^{-5/8} = 3.04 \left(\frac{Q}{S_o^{1/2}} \right)^{3/8}$$

$$(7) \frac{Q_i}{Q} = \frac{L_i}{L_1}$$

$$(8) \frac{Q_i}{Q} = \left(\frac{L_i}{L_3} \right)^{0.4}$$

INLET NO.	S	$\frac{Q}{S^{1/2}}$ (cfs.)	Q (cfs.)	T (cfs.)	$\frac{F_w}{S^{1/2}}$	F _w	F _w T (ft.)	L ₁ .770 (ft.)	L ₂ .462 (ft.)	L ₃ 1.65 (ft.)	$\frac{Q_i}{Q}$	Q _i (cfs)	$\frac{Q_i}{Q}$.600 (cfs)	L _i (ft.)		USE L ₁ (ft.)	ACT. Q _i (cfs)	Q-Q _i =Q _c (cfs)
														$Q_i < Q_2$ L ₁ < L ₂	$Q_i > Q_2$ L ₁ > L ₂			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)
1	.01	23.9	2.39	10.0*	12.1	1.21	12.1	9.3	5.6	20.0	1.0 *	2.39	1.43		20.0	20.0	2.39	0
1	.01	23.9	2.39	10.0*	12.1	1.21	12.1	9.3	5.6	20.0	0.80*	1.91	1.43		11.4	12.0	1.95	0.44
2			2.83	10.7	12.2	1.22	12.2	9.4	5.6	20.1	0.81	2.26	1.70		11.9	12.0	2.30	0.53
3			2.92	10.8	12.3	1.23	12.3	9.5	5.7	20.3	0.81	2.37	1.75		11.9	12.0	2.37	0.55
3 alternate											0.99					20.0	2.90	0.02
4	.03	23.9	4.14	10.0	12.9	2.24	22.4	17.2	10.3	37.0	0.58	2.40	2.48	10.0*		10.0*	2.40	1.74
4 alternate 1											0.80*	3.31			21.1	22.0	3.36	0.78
4 alternate 2			0.65	5.0		2.05	10.25			16.9	0.80*	0.52	0.39		10.0*		0.53	0.12

(See text for sequence of computations)

* Required criteria

One foot is 0.3048m

One cubic foot is 0.0283m³

Supposing that Inlet 3 is just above an intersection making carry-over flow undesirable, the third inlet may be increased to 20 ft to intercept practically all the flow.

If 0.55 cfs can be allowed to pass by Inlet 3, then a cost saving with three 12-ft inlets can be realized by assuming $Q_i/Q = 0.80$.

In Inlet 4, it is assumed that a 10-ft inlet is to be used and Q_i/Q is to be found. In this case $L_i = L_2$ so Equation (7) is used. For alternate 1, the length for $Q_i/Q = 0.8$ is to be computed.

For Inlet 4, alternate 2, the problem is to find T which would enable the 10-ft inlet to intercept 80% of the flow. This would tell how far upstream the inlet would have to be moved to reduce Q to that amount. In this case $(10/L_3)^{0.4} = 0.80$ so $L_3 = (1/0.80)^{2.5} 10 = 17.5$ which must equal $1.65 F_w T$. Therefore $F_w T = 17.5/1.65 = 10.6$. As a first trial assume $F_w = 2.2$ making $T = 10.6/2.2 = 4.8$. Substituting this T in Equation (4) we find $F_w = 2.03$. A second trial with $F_w = 2.1$ yields $T = 5$ and gives a computed $F_w = 2.05$ which is close enough. Taking $T = 5$ by Equation (5), $Q = 0.515(5)^{8/3} 0.03^{1/2} = 0.65$ cfs. This is an absurdly small discharge, demonstrating that a 10-ft inlet is ineffective on a 0.03 grade. Rather than moving the inlet that far upgrade, it would obviously be more economical to go to a 22-ft inlet at the original location and save the extra length of pipe.

Checking for Greater Storms

In checking drainage systems for performance with storms greater than the design storm note that the spread on the pavement increases as $Q^{3/8}$, other variables remaining constant. Thus, if runoff is doubled, spread increases only $2^{3/8}$ or 1.3 times. Assuming the inlet has been designed for $Q_i/Q = 0.80$, this would reduce to about 0.7 but Q_i would increase about $(0.7/0.8)^2 = 1.75$ times. One would then have to check the pipe capacity and particularly the head loss entering the pipe to see if the greater Q_i could be accepted. Depending on consequences of street flooding, consideration might be given to increasing the pipe capacity.

Capacity of Curb-Opening in a Sag

The capacity of curb-opening inlets in a sag depends upon the depth of water at the inlet and the inlet geometry. The inlet operates as a weir until the water submerges the entrance. When the water depth exceeds about 1.4 times the height (h) of the curb-opening entrance, the inlet operates as an orifice. Between weir-type operation and orifice-type operation the capacity is indeterminate. Figure 4-9 gives the minimum height (h_m) of opening required for weir type operation. If the opening height (h) equals or exceeds h_m , Figures 4-10 to 4-12 will give the depth of ponding measured at the curb, just above the depressed area. The use of these charts is explained in Example 5.

Figures 4-10 thru 12 are based on experiments made at Colorado State University and apply to depressed curb-opening inlets with a height of opening equal or exceeding the appropriate h_m from Figure 4-9. When the inlet is not depressed, the approximate capacity can be computed by the weir equation:

$$Q_i = 3.0 L_i d_i^{1.5} \quad (4-4)$$

where:

Q_i = capacity of the inlet, in cubic feet per second;

d_i = depth of water above inlet lip, in feet; and

L_i = length of clear opening, in feet.

When the depth at the opening exceeds 1.4 h, the capacity may be computed by the equation:

$$Q_i = 0.67A \sqrt{2g \left(d_i - \frac{h}{2} \right)} \quad (4-5)$$

where:

A = area of opening, in square feet (hL_i); and

h = height of opening, in feet.

Example 5

Given: A curb-opening inlet in a sag; pavement cross slope 0.03; concrete, broom finish ($n = 0.016$); depression, width = one foot, amount one-inch; height of inlet opening = 0.50 feet; design discharge from both sides of the inlet, $Q_1 = 2$ cfs, $Q_2 = 8$ cfs; total $Q = 10$ cfs.

Find: Maximum depth of ponding (d_{max}) for $L_i = 5$ feet, 10 feet, and 15 feet.

Solution:

1. Use Figure 4-9 to check adequacy of the opening height to maintain free fall in the inlet. For $Q = 10$ cfs the requirements are: $L_i = 15$ feet, $h_m = 0.28$ feet; $L_i = 10$ feet, $h_m = 0.38$ feet; $L_i = 5$ feet, $h_m = 0.56$ feet. The opening height, 0.50 feet, exceeds the requirement for free fall for the 15' and 10' opening lengths and Figure 4-10 can be used to determine depth of ponding.

2. From Figure 4-10 the maximum ponding is:

L_i	15 feet	10 feet	5 feet
d_{max}	0.41 feet	0.52 feet	0.72 feet
$T = d_{max} S_x$	13.7 feet	17.3 feet	24.0 feet

3. The maximum depth of ponding at the curb opening may be exceeded

in the approach gutter, particularly on low flows. The depth of ponding in the gutter can be checked at the point where the gutter slope is 0.002 by using Figure 4-13.

For $L_i = 15$ feet, $Q_1 = 2$ cfs, $d_{max} = 0.41$ feet (Step 2), and $d = 0.3$ feet.

The gutter depth for Q_1 is less than the ponding depth at the inlet and water will back up in the gutter channel.

For $Q_2 = 8$ cfs, $d_{max} = 0.41$ feet (Step 2), and $d = 0.5$ feet (Figure 4-13).

The gutter depth for Q_2 is greater than the ponding depth at the inlet and the water profile tends to draw down on approaching the inlet.

For $L_i = 10$ and 5 feet, $d_{max} = 0.52$ or 0.72 feet (Step 2), and $d = 0.3$ or 0.5 feet.

The gutter depth for both Q_1 and Q_2 is less than the ponding depth for both 5- and 10-foot length inlets; therefore, water will back up in the gutter on both sides of the inlet.

In addition to illustrating the use of the sag curves, Example 5 shows the necessity of picking up most of the gutter flow before it reaches the low point of the sag vertical curve. Spreads on the pavement (T) and depths at the curb (d_{max}) noted in step 2 could not be tolerated on a high speed highway. The more common application of the sag curves would be in designing curb-opening inlets or their spacing to keep the depth of ponding and spread on the pavement within tolerable limits.

4.60 Combination Inlets

The capacity of an unclogged combination inlet on a continuous grade using an efficient grate, is not appreciably greater than that of the grate alone. Therefore, the capacity is computed by ignoring the curb-opening and computing the capacity of the grate opening alone.

Conner and Larson found that diagonal grating bars were superior to transverse bars on combination inlets. Neither investigation compared a diagonal bar grate with the hydraulically superior longitudinal bar grate of comparable size.

4.70 Inlet Location

In general, inlets should be placed at all low points in the gutter grade and at intersections to prevent the gutter flow from crossing traffic lanes of the intersecting road. In urban locations, inlets are normally placed upgrade from pedestrian crossings to intercept the gutter flow before it reaches the crosswalk. Where pavement surfaces are warped, as at cross streets, ramps, or in transitions between superelevated and normal sections, gutter flow should be picked up before the cross slope of the pavement begins to change in order to lessen water flowing across the roadway and to prevent icing.

In a sag vertical curve, three inlets are desirable. On major streets and arterials three inlets should be used; one at the low point and one on each side of this point where the grade elevation is at least 0.2 feet higher than that at the low point. The additional inlets furnish added capacity to allow for flow bypassing the upgrade inlets and provide a safety factor if the sag inlet becomes clogged. These inlets limit the deposition of sediment on the road in the sag, and they also reduce flow arriving at the low point and thereby prevent ponding which would flood the road.

Where a curbed roadway crosses a bridge the gutter flow should be intercepted and not be permitted to flow onto the bridge.

Spacing of Inlets on a Continuous Grade

Inlets should be spaced so as to limit the spread of the water on the pavement to the criterion outlined in Section 3.20.

With the maximum spread fixed and with a given pavement cross slope and longitudinal slope, the flow in the gutter is also fixed and can be calculated as explained in Section 3. The spacing of inlets is equal to the length of pavement needed to generate the discharge corresponding to the allowable spread on the pavement. The flow bypassing each inlet must be included in the flow arriving at the next inlet.

An example of the computations for inlet spacing will be given for a grate inlet (Example 6) and for a curb-opening inlet (Example 7).

Example 6

Grate Inlets

Given: A high-type arterial 4-lane section with 2' curb-and-gutter section ($n = 0.016$) (widths = $4(12) + 2(2) = 52'$). The pavement cross slope, $S_x = 0.02$ ft/ft, longitudinal slope, $S_o = 0.02$; composite C for pavement and shoulder = 0.6. The contributing area is 100' to each side of the centerline.

Find: Inlet spacing of 2-foot-wide efficient grate.

Solution:

1. Spread on pavement, $T = \frac{52}{2} - 12 = 14'$
2. Depth of flow at curb = $d = TS_x = 14 (0.02) = 0.28$ ft.
3. Compute the capacity for $S = S_x = 0.02$ ft/ft and $T = 14$ ft. Using Figure 3.2, $Q = 8.3$ cfs
4. The flow intercepted by a 2 ft. grate with efficient openings is determined from Figure 3.2 using composite sections.

$$T = T_s = 12 \text{ ft}, S = S_x = 0.02$$

$$Q_s = 5.5 \text{ cfs}$$

$$Q_w = 8.3 - 5.5 = 2.8 \text{ cfs}$$

5. The length of the roadway above the inlet should be sufficient to generate the gutter flow (8.3cfs) corresponding to the allowable spread. Use the rational method for successive lengths to determine the initial spacing.

$$Q = CiA$$

where:

$$Q = 8.3 \text{ cfs}$$

$$C = \text{composite for pavement and shoulder} = 0.60$$

$$T_c = 8 \text{ min. (Figure 2-2, distance} = 100 \text{ ft) overland flow}$$

$$V = Q/A = 8.3 \text{ cfs}/1.96 \text{ ft}^2 = 4.23 \text{ fps gutter flow}$$

$$A = \frac{200 \times L}{43560}$$

$$\text{Try } L = 400'$$

$$T_c = \frac{300}{4.23(60)} + 8.0 = 9.2 \text{ (use 10 min)}$$

$$i = 5.82 \text{ in/hr (Figure 2-1)}$$

$$A = \frac{200 \times 400}{43560} = 2.3 \text{ ac}$$

$$Q = CiA = 0.60 (5.82)(1.4) = 8.0 \text{ cfs} \quad 8.3 \text{ cfs}$$

Use $L = 400 \text{ ft}$ for first inlet

6. The flow bypassing the grate appears at the next grate and only the flow intercepted (2.8 cfs) must be supplied by the area between grates.

$$A_2 = \frac{2.8}{(0.6)(5.82)} = 0.80 \text{ ac or } 34928 \text{ sq. ft.}$$

$$L_2 = \frac{34928}{200} = 175 \text{ ft.}$$

Example 7

Curb-Opening Inlets

Given: Conditions as in Example 6.

Find: Inlet length for 50% interception and spacing between inlets
where $w = 2.0$ ft and $a = 2$ inches

Solution:

1. Spread on pavement = 14'
2. Discharge in gutter, from Example 6, $Q = 8.3$ cfs.
3. Length of roadway above first inlet = 400 feet (Step 5, Example 6).
4. Compute inlet size for 50% interception.
 $d = 0.28$ ft
 $n = 0.016$
 $S_0 = 0.02$ ft/ft
 $T = 14$ ft
 $Q_i/Q = 0.5$
 $S_x = 0.02$ ft/ft

From Figure 4-7, $L_i = 10$ ft

5. The flow intercepted by the inlet must be supplied by the intervening pavement if the design spread is obtained. The area of pavement need is:

$$A = \frac{4.15}{(0.6)(5.82)} = 1.19 \text{ ac or } 51,768 \text{ sq ft}$$

The spacing of successive inlets after the upstream inlet,

$$L = \frac{51768}{200} = 260 \text{ feet.}$$

Spacing of Inlets in a Sag

Three inlets should be placed in a sag vertical curve on all major streets, one at the low point and one on each side of this point, where the grade elevation is approximately 0.2 feet higher than that at the low point. The inlets should be spaced so as to limit the spread of water on the pavement to the criterion outlined in Section 3.20.

As a result of these criteria, the inlets in a sag of an expressway must, at times, be designed to remove the stormwater resulting from a 50-year storm over the contributing area minus the flow intercepted by the inlets on the grade, which are designed to limit the spread of water from a 10-year storm

to a tolerable limit. The inlets on the grade will intercept a greater quantity of water during the 50-year storm than the quantity (from a 10-year storm) used to determine their spacing, but the spread of water on the pavement will exceed the spread designated as the tolerable limit.

Because of the various combinations in which sag inlets are used, examples cannot be given to fit all problems encountered by the designer. Example 8 will illustrate the spacing of inlets in a sag which must be designed for a 50-year frequency when the inlets on the grade are spaced for a 10-year frequency. The problem of sag inlets designed for some other frequency can be solved with a slight modification of the procedure used in Example 8. The procedure can be used for other type inlets whose capacities are known.

Example 8

Design of Curb-Opening Inlets in a Sag

Given: Grades, -3 percent and +3 percent, each 2,170 feet long, intersecting at station 50. A 600-foot vertical curve connects the tangents. Conditions are the same as in Examples 6 and 7. The inlets on the grades are 10-foot curb-opening inlets, designed to limit the spread on the pavement to 6 feet at the 10-year frequency. The gutter is 2-feet wide and the depression is 2-feet wide and 2-inches deep.

Find: Size and location of the 3 curb-opening inlets in the sag. The inlets will be designed to limit the spread on the pavement for a 50-year frequency storm to 6 feet.

Solution:

1. The grades are symmetrical about the P.I. of the vertical curve and only a half section need be considered. The first inlet (see Example 7) is located 830 feet from the crest and successive inlets are spaced at 520 foot intervals. The computations for peak flow arriving at the sag inlet at station 50 are given in Table 4-2.
2. The 10-foot curb openings are spaced (Column 3) for a 10-year frequency storm as explained in Example 7. An inlet, of width opening to be determined, is placed at the P.V.I. station 50 + 00. The inlet at station 49 + 40 is placed where the grade elevation is about 0.2 feet higher than the grade elevation at the P.V.I. A 10-foot curb opening is tentatively placed here and the computations shown in Table 4-2 are made to determine the width of opening required in the sag. If the sag inlet opening is excessive, wider openings can be used at the 0.2 feet higher elevation point. The spacing and width of opening on the grades might require adjustment in some instances.
3. The runoff between inlets (column 11) is computed by the rational method based on the 50-year rainfall intensity (Column 10) during

Table 4-2 Computations for Sag Inlets

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
Inlet Station	Elev. (ft.)	Dist. (ft.)	Grade (%)	ΔA (acres)	C	ΔCA	ΔT_c (min.)	ΣT_c (min.)	i 50-yr.	ΔQ (cfs)	Q (cfs)	d (ft.)	T (ft.)	Q1/Q	Q1 (cfs)	Qc (cfs)
36 + 60	140.20	830	3.0	0.50	0.8	0.40	5	5	14.7	5.88	5.88	0.27	9.0	0.52	3.05	2.83
41 + 80	124.60	520	3.0	0.31	0.8	0.25	0.7	5.7	14.2	3.55	6.38	0.28	9.3	0.50	3.19	3.19
P.V.C. 47 + 00	109.00	520	3.0	0.31	0.8	0.25	0.7	6.4	13.8	3.45	6.64	0.28	9.3	0.50	3.32	3.32
49 + 40	104.69	240	1.8	0.14	0.8	0.11	0.5	6.9	13.4	1.47	4.79	0.25	8.3	0.58	2.78	2.01
P.V.I. 50 + 00	104.50	60	0.3	0.04	0.8	0.03	0.2	7.1	13.3	0.40	2.41					
Total at sta. 50 + 00											4.82	0.20	6.7	1.0	4.82	0

the accumulated time of concentration (Column 9). Column 12 is the Q arriving at the inlet and consists of the Q_c (Column 17) bypassing the last inlet plus the Q (Column 11) from the area between inlets. On the grade, the spread on the pavement, T (Column 14) exceeds the allowable spread, 8 feet, which was based on a 10-year rainfall intensity.

4. The discharge arriving at the sag inlet from both sides is 4.82 cfs (Column 12). From Figure 4-9, this Q would require the following height of opening; $L_i = 15$ feet, $h_m = 0.16$ feet; $L_i = 10$ feet, $h_m = 0.23$ feet; $L_i = 5$ feet, $h_m = 0.34$ feet.
5. The depth at the curb for an allowable spread of 6 feet on the traveled way is 0.24 feet. On Figure 4-10 for $w = 2$ feet, $a = 2$ inches, and $Q = 4.82$ cfs, a 10-foot opening will carry the flow with a depth of ponding in the gutter (d_{max}) = 0.25 feet. A 15-foot opening will carry the flow with a depth of ponding = 0.20 feet. The ponding with the 15-foot opening is less than the allowable (0.24 ft) and the 15-foot opening with a clear height of at least 0.16 ft (Step 4) is satisfactory.

4.80 Definition of Symbols for Sections 3 and 4

Symbol	Units	Description
A	sq-ft	Area of cross section
A	ac	Drainage area tributary to the point under design
a	in.	Depression of inlet lip below the extension of gutter flow line
C		Runoff coefficient in the rational formula
d or y	ft	Depth of gutter flow at the curb line
d_b	ft	Depth of grate bars
d_i	ft	Depth of water above inlet lip
F_w		Froude Number based on depths and velocity of uniform gutter flow at distance W from curb face
g	ft/sec ²	Acceleration of gravity = 32.2 ft/sec ²
h	ft	Height of curb-opening inlet
h_m	ft	Minimum height of curb-opening inlet for free fall into inlet
i	in./hr	Average rainfall intensity during the time of concentration
L	ft	Length of roadway between inlets
L_b	ft	Length of clear opening of grate bars (see Section 4.5)
L_i	ft	Length of curb-opening inlet
L_1	ft	Length of inlet when $Q_i/Q = 1$ on first section of curve for $Q_i/Q = \text{function } L_i/(F_w T)$
L_2	ft	Length of inlet at point of intersection of first and second sections of $Q_i/Q = \text{function } L_i/(F_w T)$
L_3	ft	Length of inlet when $Q_i/Q = 1$ on second section of $Q_i/Q = \text{function } L_i/(F_w T)$

n		Roughness coefficient in the modified Manning formula for triangular gutter flow
P	ft	Perimeter of grate opening, neglecting bars and side against curb
Q	cfs	Rate of discharge in gutter
Q _c	cfs	Rate of discharge bypassing the inlet (carry-over)
Q _i	cfs	Rate of discharge intercepted by inlet or total flow for sump inlets
Q ₂	cfs	Value of Q _i at L ₂
Q _i /Q		Interception rate of curb-opening inlet (also called efficiency of inlet)
S ₀ or S	ft/ft	Longitudinal slope of pavement
S _x	ft/ft	Cross slope of pavement
T	ft	Top width of water surface (spread on pavement)
T _c	min	Time of concentration of a watershed
V	ft/sec	Mean velocity of flow
W	ft	Width of depression for curb-opening inlets
X	ft	Distance from curb face in gutter flow computations

The following figures give the capacity of depressed curb-opening inlets for the range of conditions listed in Table 4-3. Instructions for using these figures with examples are given in the above section.

The capacity charts were developed for gutters with a roughness coefficient, $n = 0.016$, but they can be used with sufficient accuracy, for gutters with "n" values from 0.012 to 0.016. Figure 4-7 was developed with "n" values from 0.010 to 0.016.

Table 4-3

Limiting Conditions of Design Figures

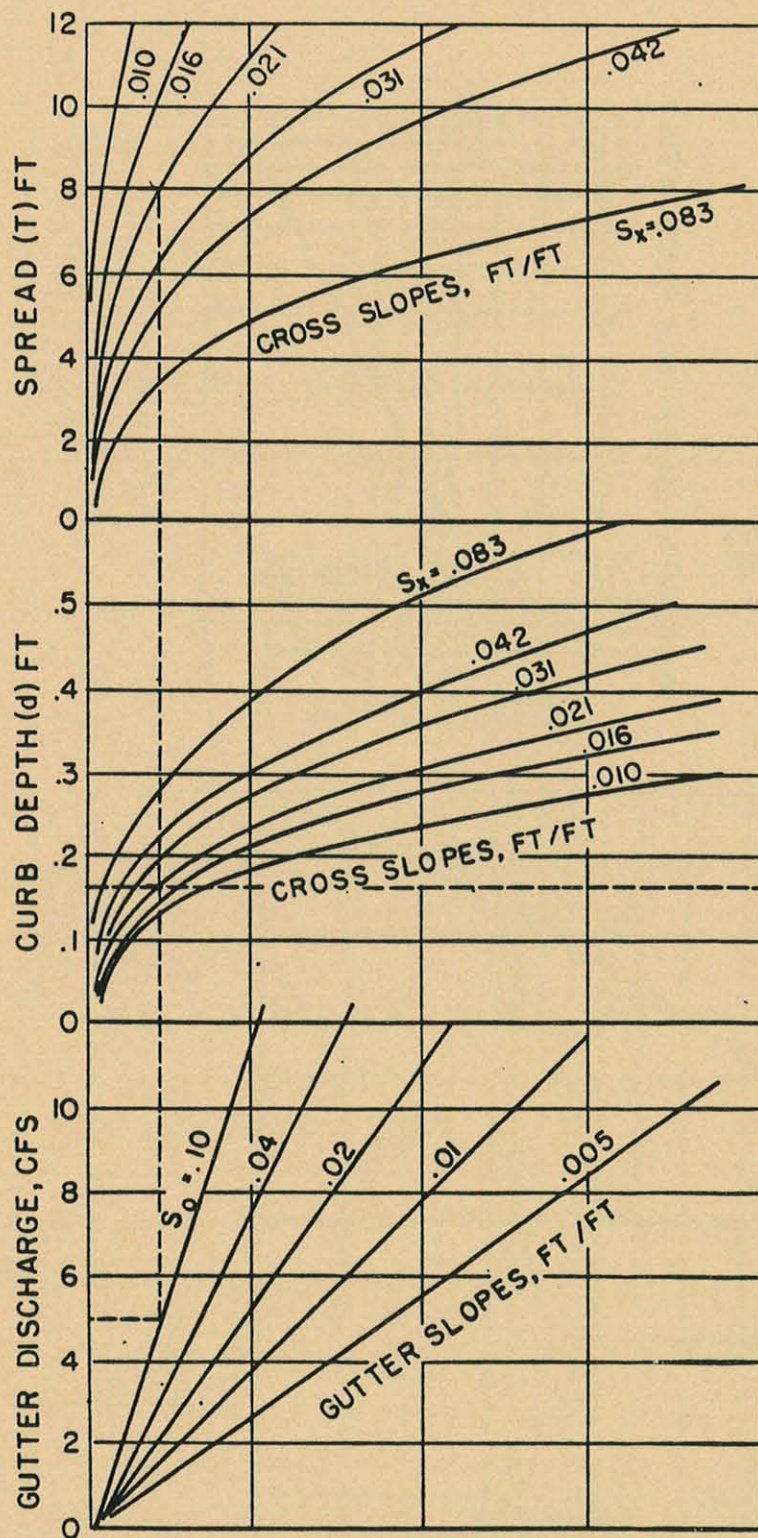
	<u>General Curves</u>		<u>Sump Curves</u>	
W, in ft	2	1	2	3
a, in in.	2	1	2	3
S ₀	0.004 - 0.15		0	
S _x	0.015 - 1.0		0.015 - 0.06	
L _j , in ft	1 - 200		5, 10, 15	
T, in ft	4 - 10		---	
Minimum Width from Curb to Crown of Roadway	--	12'	12'	18'
d _{max}	--	1'	1'	1'

where:

W = width of depression;
a = depression of inlet lip below the extension of gutter flow line;
S₀ = longitudinal slope of pavement;
S_x = cross slope of pavement;
L_j = length of curb-opening inlet;
T = top width of water surface (spread on pavement); and
d_{max} = maximum depth of ponding, curb-opening in sump.

4.90 Bibliography

1. Bauer, W.J., and Woo, D.C., Hydraulic Design of Depressed Curb-Opening Inlets, Highway Research Board Record No. 58, Washington, DC, 1964, p. 61-80.
2. Federal Highway Administration, Drainage of Highway Pavements, Hydraulic Engineering Circular No. 12, Washington, DC, 1969.
3. Izzard, Carl. F., Simplified Method For Design of Curb-Opening Inlets, Report presented at the Annual Meeting of the National Transportation Board, January 1977.
4. Karaki, S.S., and Haynie, R.M., Depressed Curb-Opening Inlets--Experimental Data, Colorado State University, Civil Engineering Section, Fort Collins, CO, 1961, CER61SSK34.



EXAMPLE (SEE DASHED LINES)

GIVEN:

$T = 8.0$ FT

$S_x = 0.021$

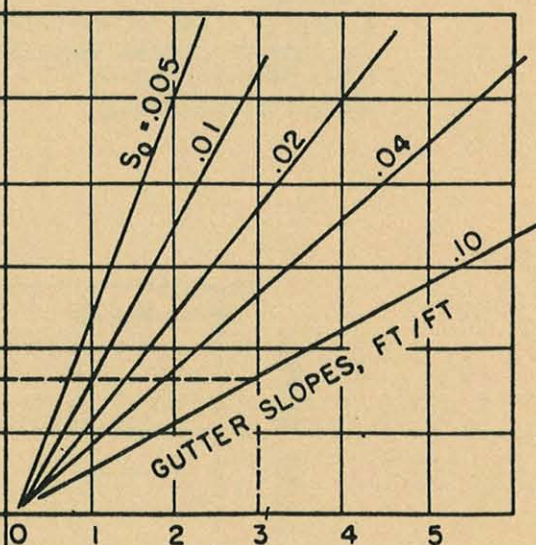
$S_o = 0.10$

FIND:

$Q = 5.0$ CFS

$d = 0.17$ FT

L_b FOR EFFICIENT GRATE
= 3.0 FT



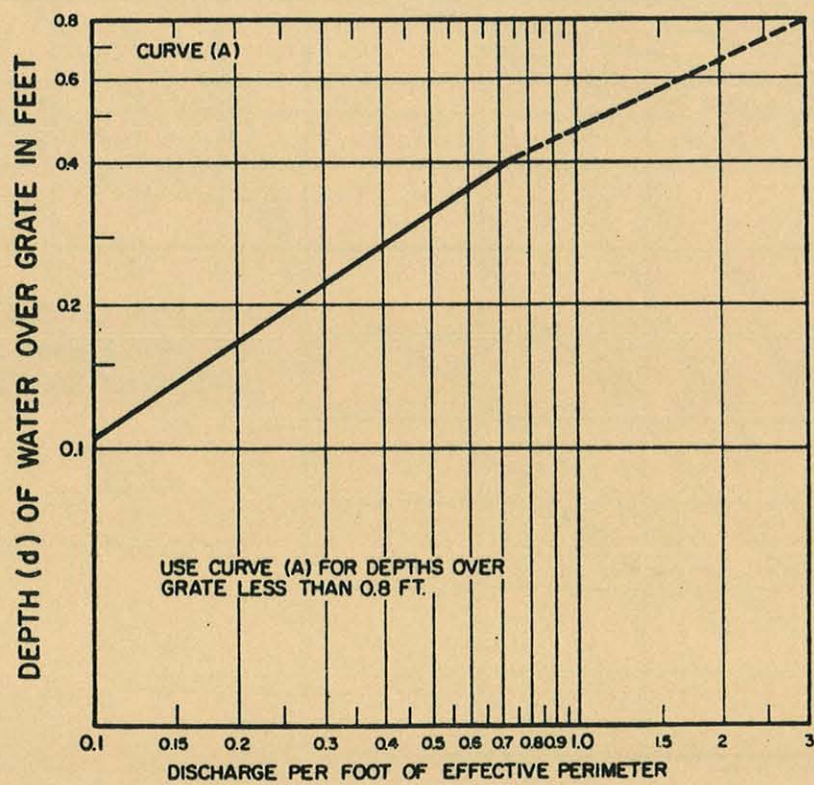
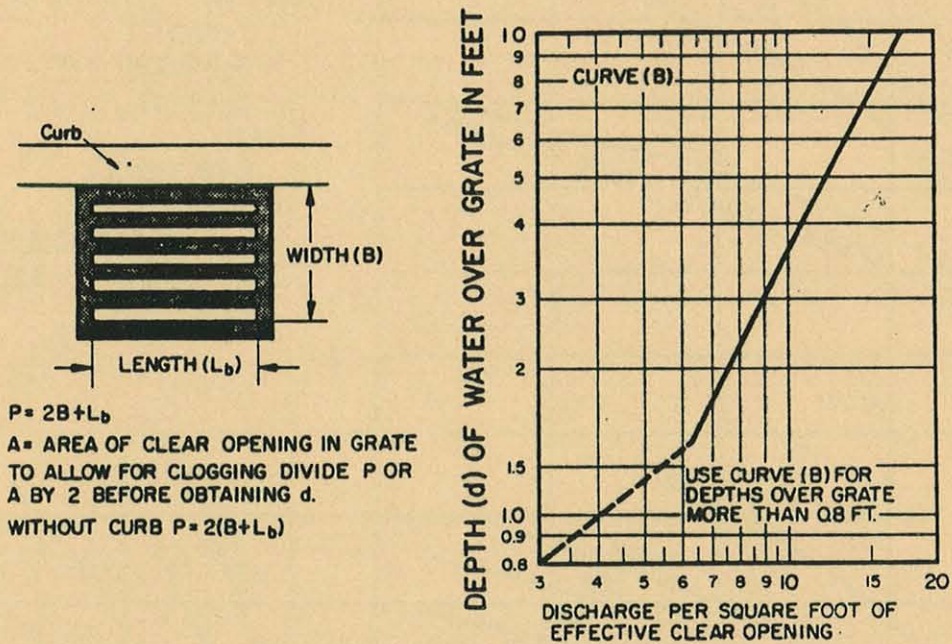
LENGTH OF CLEAR OPENING
OF GRATE BAR, L_b , IN FT, FOR
EFFICIENT GRATE

$n = 0.015$

$d_b = 3 \frac{1}{2}$ in.

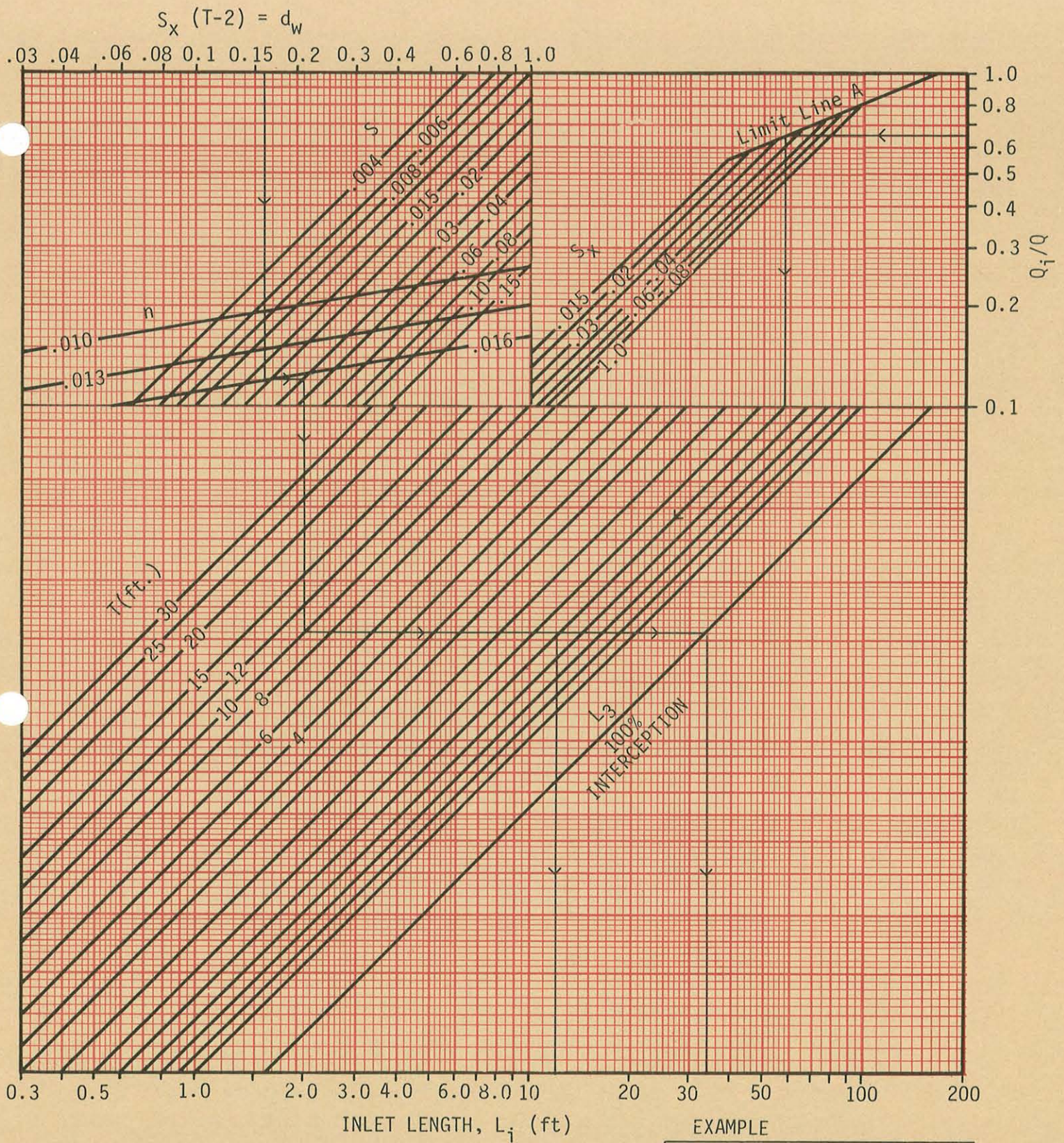
For other n 's use $Q_c = \frac{Q_n (0.015)}{n_c}$

Figure 4-3 Flow-Characteristics Curves for Grate Inlets



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Figure 4-4 Hydraulic Capacity of Grate Inlet in Sump



This chart assumes, $w=2$ ft, $a=2$ " and $h=6$ in.

Figure 4-7 Standard Curb-Opening Inlet Chart

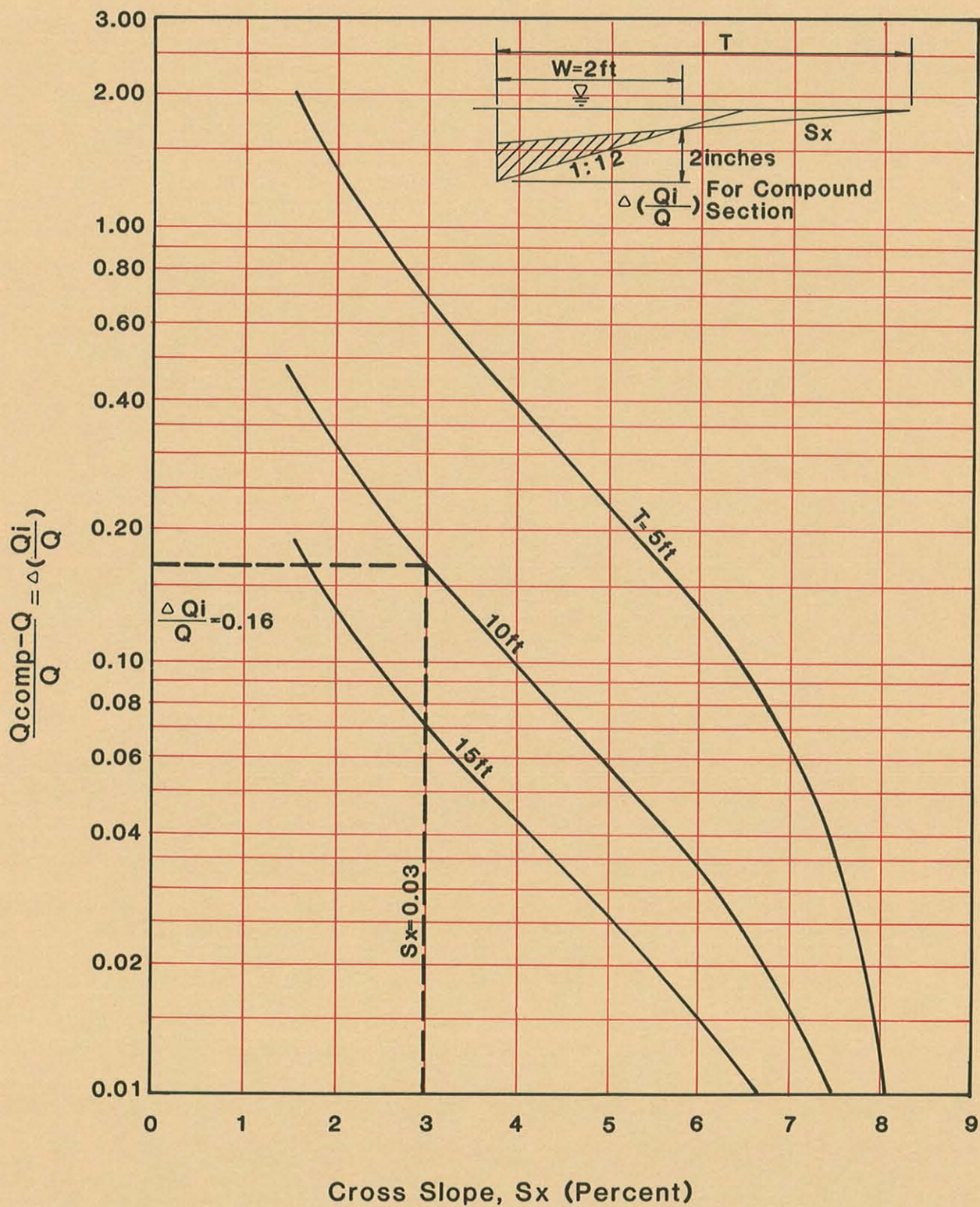


Figure 4-8 Additional Inlet Flow Due to Compound Section

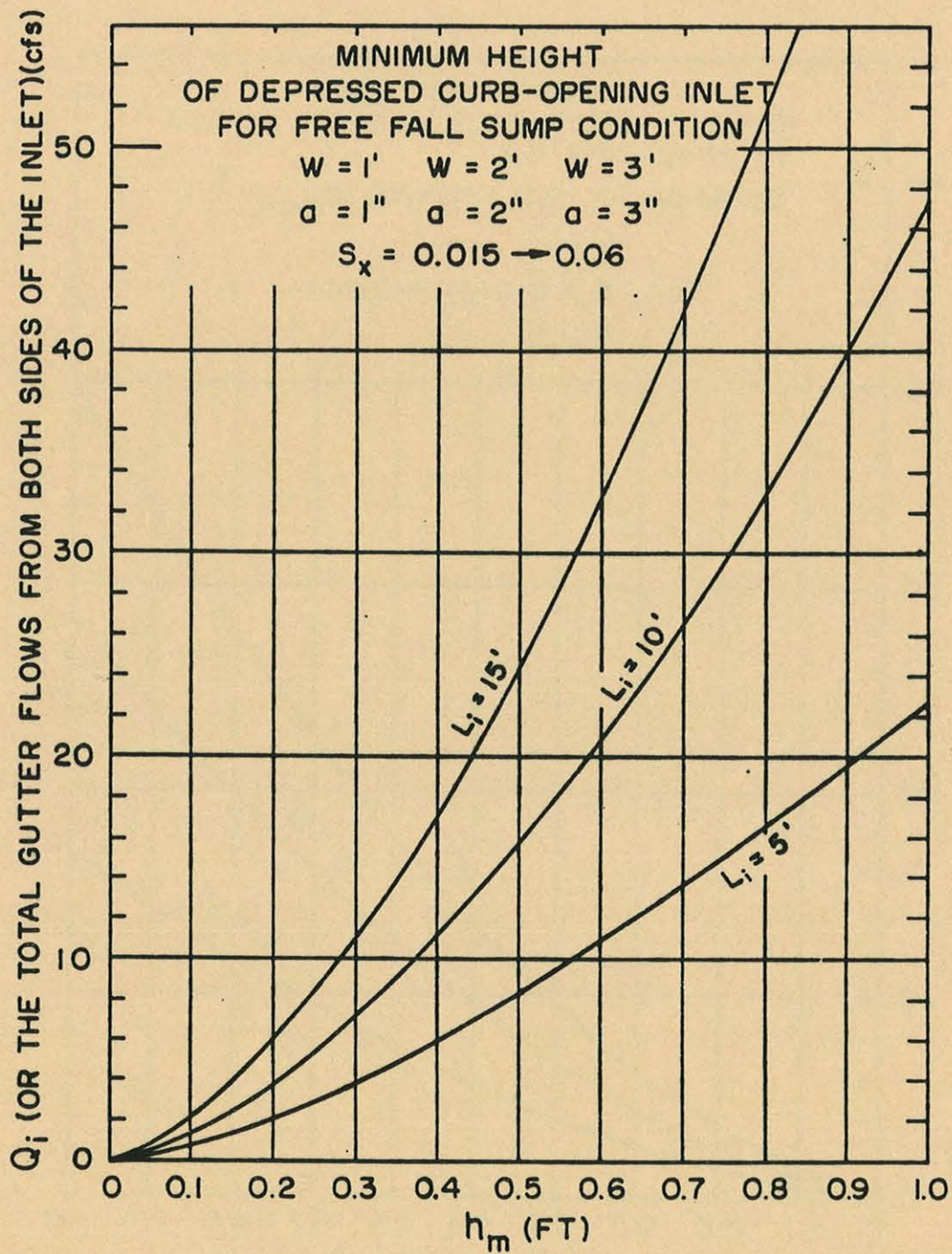


Figure 4-9 Sump Capacity for Curb-Opening Inlets

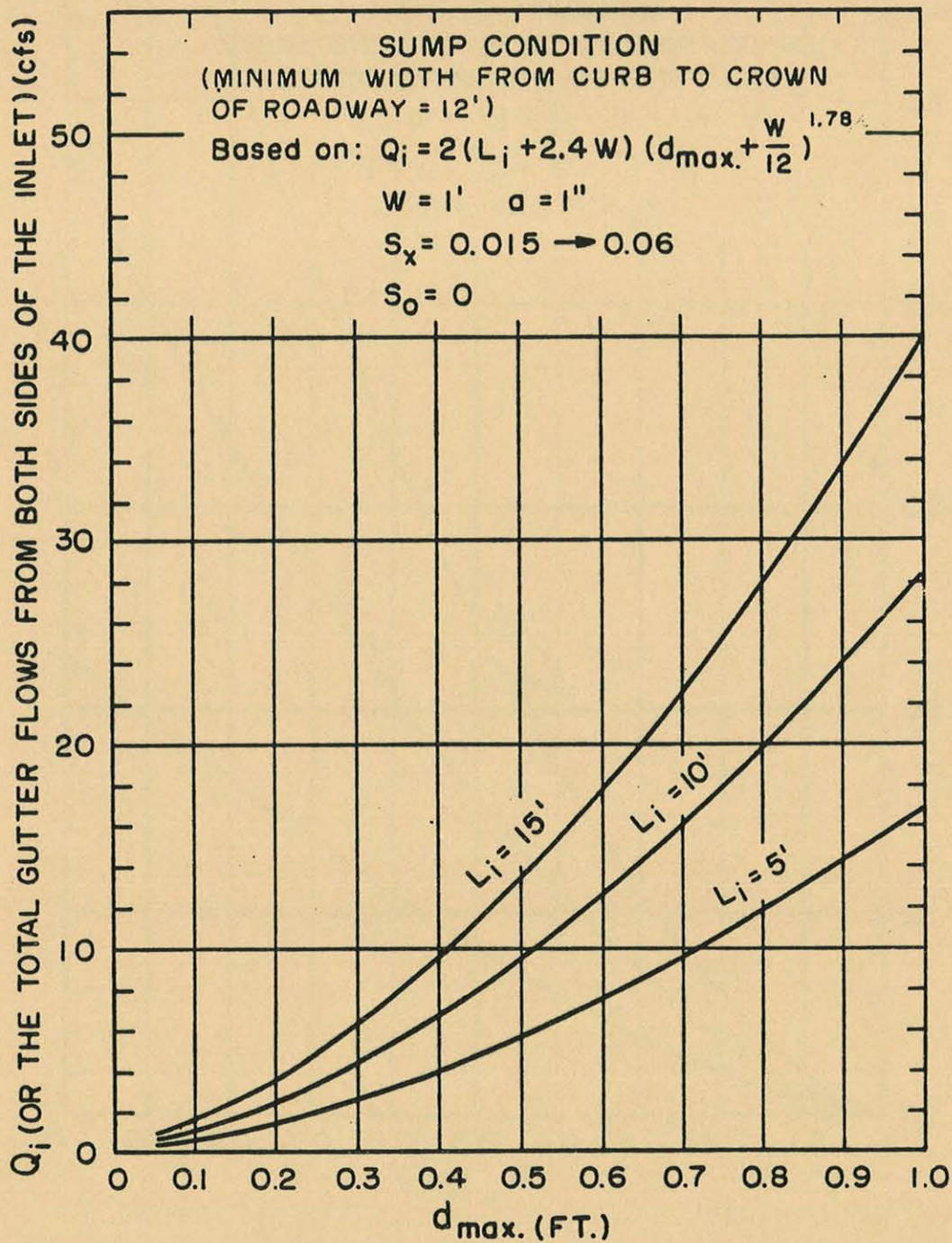


Figure 4-10 Sump Capacity for Curb-Opening Inlets

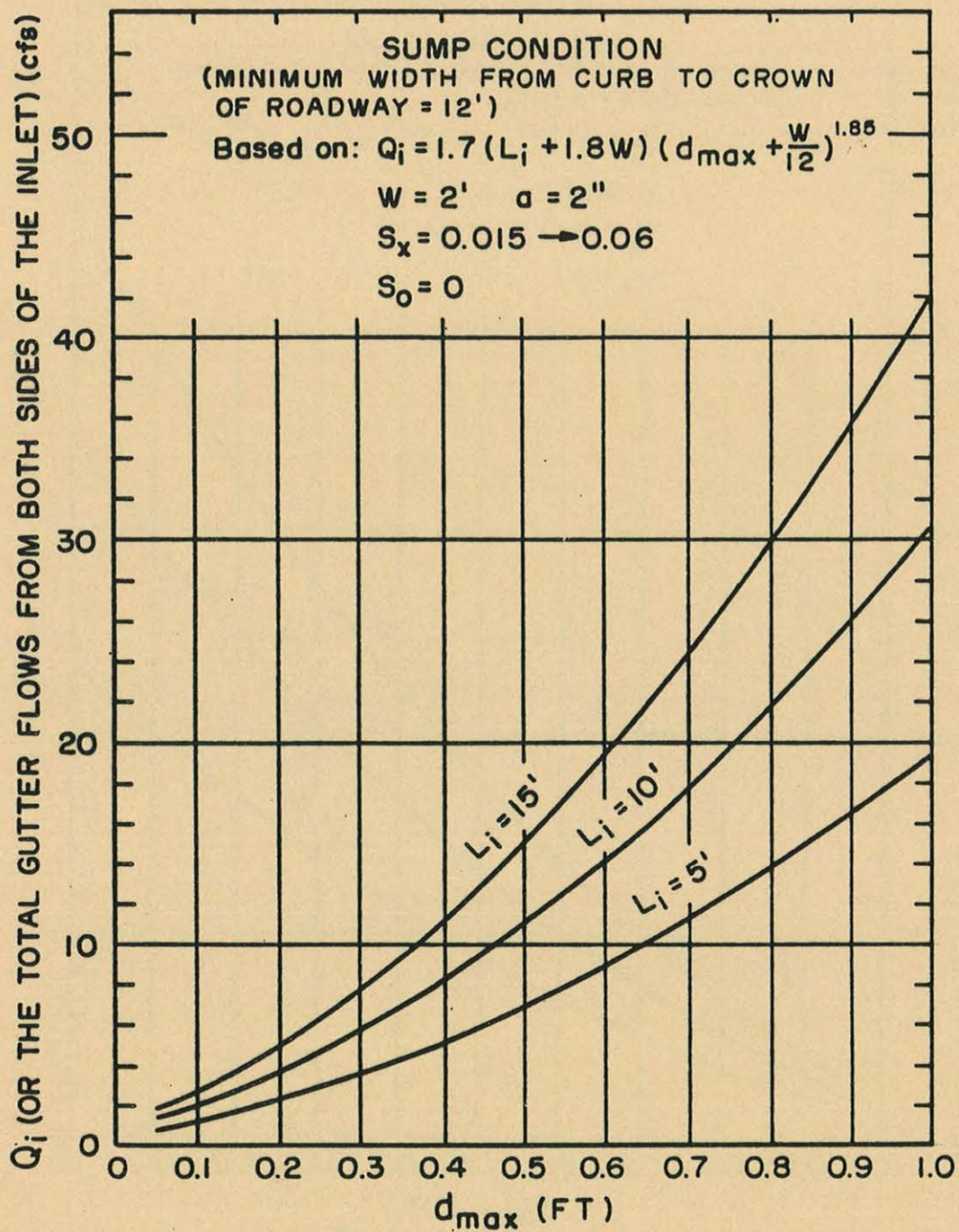


Figure 4-11 Sump Capacity for Curb-Opening Inlets

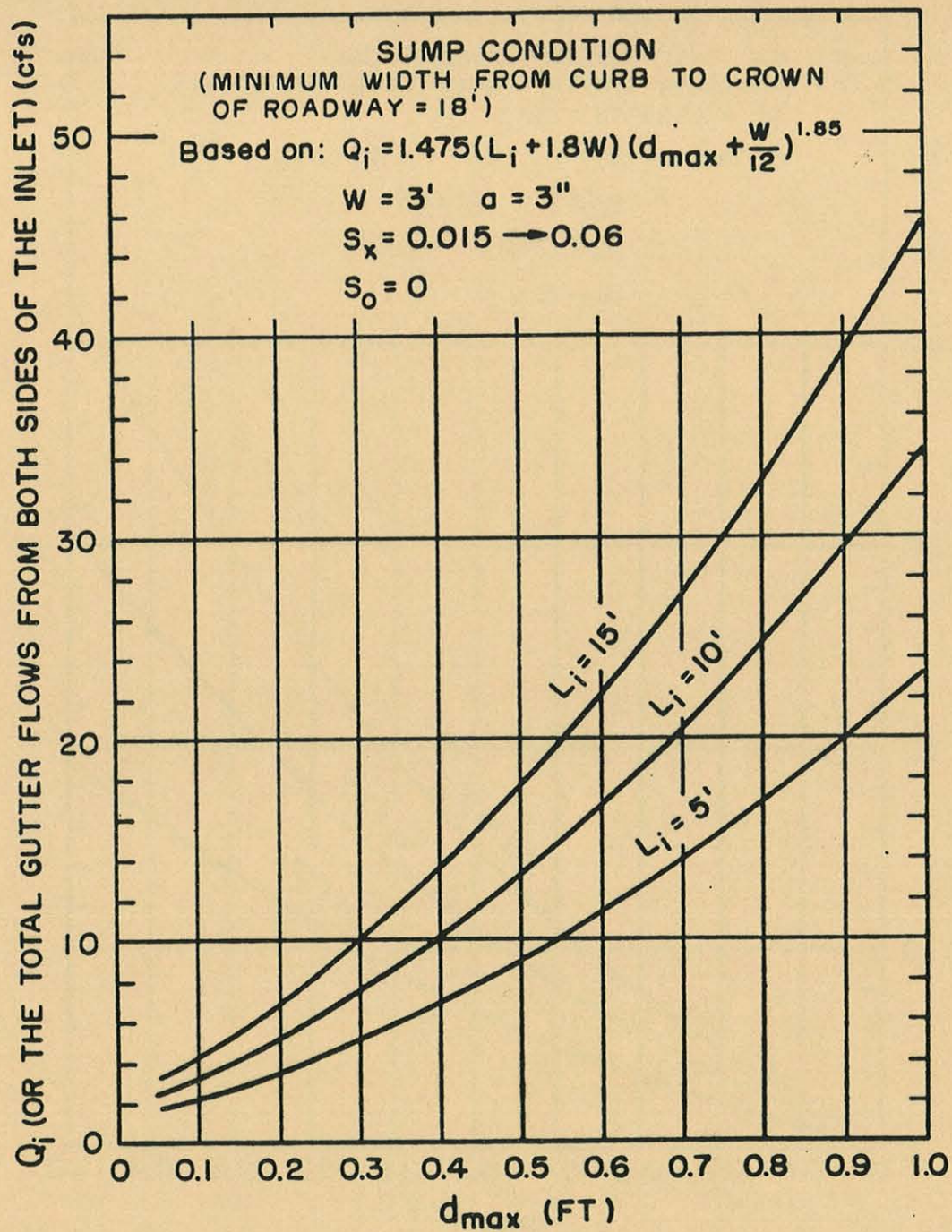


Figure 4-12 Sump Capacity for Curb-Opening Inlets

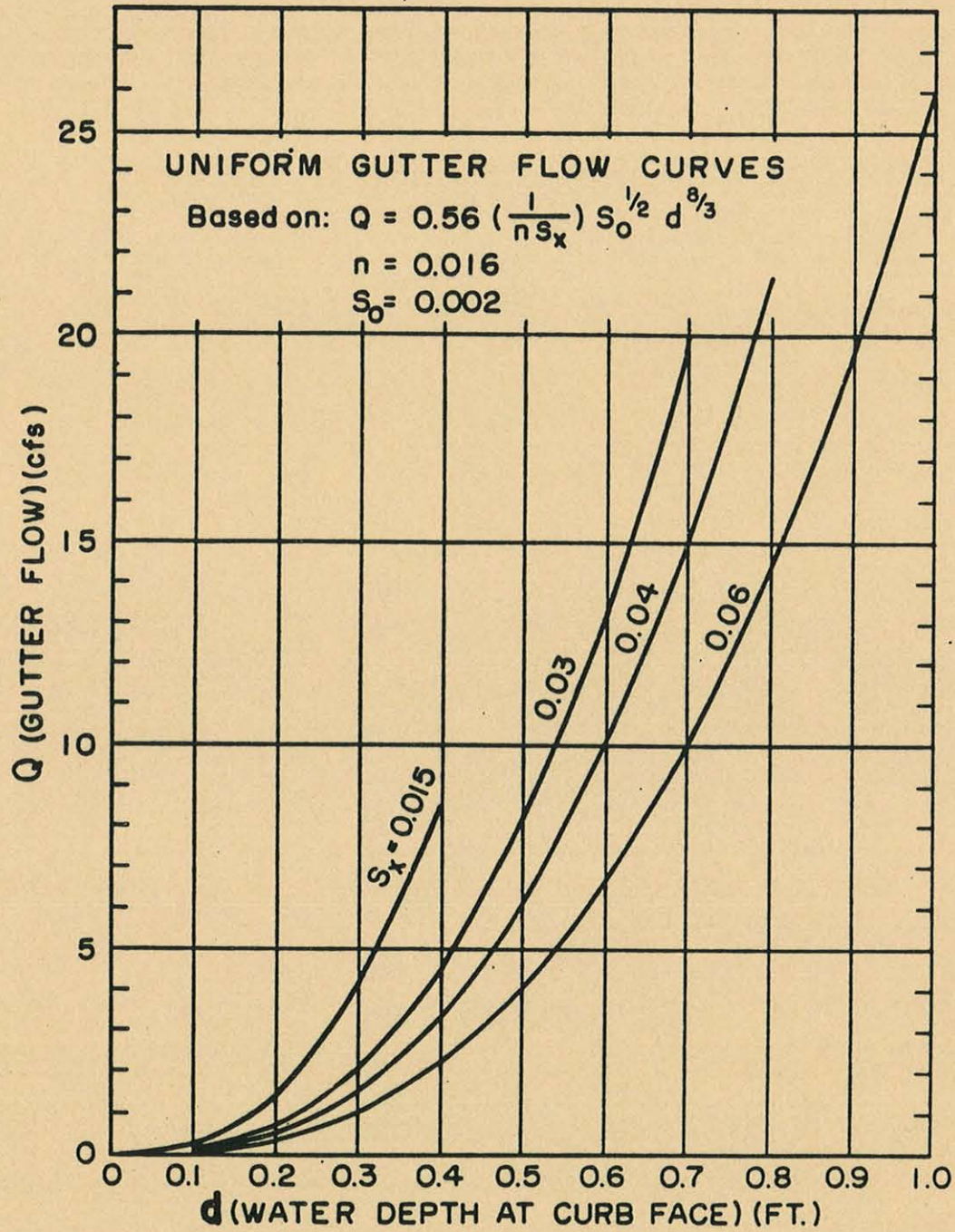


Figure 4-13 Uniform Gutter Flow Curves

5 Storm Drain & Appurtenances

Section 5

Storm Sewers and Appurtenances

5.10 General Criteria

Frequency of Design Runoff

Velocities and Grades

Materials

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Alignment

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Pipe Flow Charts

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Total Energy Losses at Structures

Minor Head Losses at Structures

Obstructions

Expansions and Contractions

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Preliminary Design Considerations

Inlet System

Storm Sewer System

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Section 5

Storm Sewers and Appurtenances

It is the purpose of this section to consider the significance of the hydraulic elements of storm sewers and their appurtenances to a storm drainage system. Hydraulically, storm drainage systems are conduits (open or enclosed) in which unsteady and non-uniform free flow exists. Storm sewers accordingly are designed for open-channel flow to satisfy as well as make possible the requirements for unsteady and non-uniform flow. Steady flow conditions may or may not be uniform.

All storm sewers 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 structures and their performance on the waterway in question.

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

1. The system must accommodate the initial surface runoff resulting from the selected design storm without serious damage to physical facilities or substantial interruption of normal traffic.
2. Runoff resulting from major storms exceeding the design storm must be anticipated and discharged 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 costs 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 minimum additional cost by the consideration of ultimate development on upstream or existing reaches.

5.10 General Criteria

Frequency of Design Runoff

The frequency of design runoff is a function of operational and economic criteria with a special emphasis on public safety. As discussed in other sections of this Manual some types of facilities do not require high levels of protection and periodic flooding is not objectionable. However, for all facilities the designer must consider the impact of a 100-year flood and provide for its passage without the

loss of life or major property damage.

Table 5-1 indicates the minimum acceptable frequencies of design runoff for storm sewers.

Table 5-1
Storm Sewer Design Storm Frequency

<u>Facility</u>	<u>Storm Return Period (Frequency)</u>
Streets and Gutters	10 years
Inlets	10 years
Storm Sewers	10 years
Major Drainage System	100 years

Velocities and Grades

Minimum Grades

Storm sewers should operate with velocities of flow sufficient to prevent excessive deposition of solid material; otherwise, objectionable clogging may result. The controlling velocity occurs near the bottom of the conduit and is considerably less than the mean velocity. Storm sewers shall be designed to have a minimum mean velocity flowing full of 2.5 fps. Table 5-2 indicates the grades for both concrete pipe ($n = 0.013$) 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. Any variance must be approved by the appropriate agency. Outlets on sewers of minimum grade should be designed to avoid sedimentation at the outfall.

Table 5-2

Minimum Slope Required For Scouring Velocity

<u>Pipe Size (inches)</u>	<u>Concrete Pipe Slope (ft/ft)</u>	<u>Corrugated Metal Pipe Slope (ft/ft)</u>
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

Maximum Velocities

Maximum velocities in conduits are important mainly because of the possibilities of excessive erosion on the storm drain inverts. Table 5-3 shows the limits of maximum velocity.

Table 5-3

Maximum Velocity in Storm Sewers

<u>Description</u>	<u>Maximum Permissible Velocity</u>
Culverts (all types)	15 fps
Storm Sewers (inlet laterals)	No Limit
Storm Sewers (collectors)	15 fps
Storm Sewers (mains)	12 fps

Minimum Diameter

Pipes which are to become an integral part of the public storm sewer system shall have a minimum diameter of 18 inches. If smaller diameters are required for utility clearance or special conditions contact the City of Dodge City for approval.

Materials

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

The coefficients of roughness listed in Table 5-4 are for use in the nomographs contained herein, or for direct solution of Manning's Equation.

Table 5-4

Roughness Coefficients "n" for Storm Sewers

<u>Materials of Construction</u>		<u>Design Coefficient¹</u>					
Concrete Pipe		0.013					
Corrugated-Metal Pipe*		2-2/3"x1/2" Corr. 3"x1" Corr.					
Unpaved		0.024		0.027			
25% Paved		0.021		0.023			
Structural Plate Pipe*							
		5 ft	7 ft	10 ft	15 ft		
Unpaved	0.033	0.032	0.030	0.028			
25% Paved	0.028	0.027	0.026	0.024			
Helically Corrugated Pipe*		2-2/3" x 1/2" corrugations					
		12"	18"	24"	36"	48"	
Unpaved	0.011	0.014	0.016	0.019	0.020		
25% Paved	--	--	0.015	0.017	0.020		
		3" x 1" corrugations					
		36"	48"	54"	60"	66"	72"
Unpaved	0.021	0.023	0.023	0.024	0.025	0.026	
25% Paved	0.019	0.020	0.020	0.021	0.022	0.022	
* Fully Paved All Types		0.012					

¹ Designer may select a single representative "n" for design purposes.

Manhole Location

Manholes shall be located at intervals not to exceed 450 feet for pipe 30 inches in diameter or smaller. Manholes shall be located at conduit junctions, changes in grade, changes in alignment, and ends of curved sections.

Manholes for pipe greater than 30 inches in diameter shall be located at points where design indicates entrance into the conduit is desirable; however, in no case shall the distance between openings or entrances be greater than 600 feet.

Pipe Connections

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

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, telephone, electrical, and sanitary sewer lines. When conflicts arise between a proposed drainage system and a utility system, the owner of the utility system shall be contacted.

Alignment

In general storm sewer alignment between manholes shall be straight. Long radius curves may be allowed to conform to street alignment. Short radius curves may be used on larger pipes in order to reduce head losses at junctions. Curves may be produced by angling the joints or by fabricating beveled ends. Angled joints shall be kept at a minimum to maintain a tight joint.

5.20 Flow in Storm Sewers

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

$$Q = AV; \text{ and} \quad \text{(Continuity)} \quad (5-1)$$

$$Q = \frac{1.49}{n} AR^{0.67} S_f^{0.5} \quad \text{(Manning's)} \quad (5-2)$$

where:

- Q = pipe flow in cfs;
- A = cross-sectional area of pipe, in square feet;
- V = velocity of flow, in feet per second;
- n = coefficient of roughness of pipe;
- R = hydraulic radius = A/W_p , in feet;
- S_f = friction slope in pipe, in feet per foot; and
- W_p = wetted perimeter, in feet.

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

1. Select pipe size and slope so that the velocity of flow will increase progressively, or at least will not appreciably decrease, at inlets, bends, or other changes in geometry or configuration.
2. Do not discharge the contents of a larger pipe into a smaller one, even though the capacity of the smaller pipe may be greater due to steeper slope.
3. At changes in pipe size match the soffits (inside top surface) of the two pipes at the same level rather than matching the flow lines. (When necessary for minimal fall, match the 0.8-diameter point of each pipe.)
4. Conduits are to be checked at the time of their design with reference to critical slope. If the slope of the line is greater than critical slope, the unit will likely be operating under entrance control instead of the originally assumed normal flow. Conduit slope should be kept below critical slope if at all possible. This also removes the possibility of a hydraulic jump within the line.

Pipe Flow Charts

Figures 5-1 through 5-3 are nomographs for determining flow properties in circular pipe. The nomographs are based upon a value of "n" of 0.012 for concrete and 0.024 for corrugated metal. The charts are self-explanatory, and their use is demonstrated by the example in Figure 5-1.

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

$$Q_C = \frac{Q_n(0.012)}{n_C} \quad (5-3)$$

where:

Q_C = flow based upon n_C ;

n_C = value of "n" other than 0.012; and

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 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 5-1 to read $Q_n = 34$ cfs. Using the formula, $Q_C = 34 (0.12/0.018) = 22.7$ 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 the nomograph can be used. Using the formula $Q_n = 22.7 (0.018/0.012) = 34$ cfs, enter Figure 5-1 to determine $d/D = 0.6$. Now enter Figure 5-3 to determine $V = 7.5$ fps (Answer).

5.30 Hydraulic Gradient and Profile Of Storm Sewer

When storm sewer systems are designed for full flow the designer shall establish the head losses caused by flow resistance in the conduit, changes of momentum and interference at junctions and structures. This information shall then be used to establish the design water surface elevation at each structure.

Determination of the hydraulic grade line is not required for all storm drainage design. It is not necessary to compute the hydraulic grade line of a conduit run if the slope and the pipe sizes are chosen so

that the slope is equal to or greater than friction slope, the inside top surfaces of successive pipes are lined up at changes in size, and the water surface at the point of discharge will not rise above the top of the outlet. In such cases the pipe will not operate under pressure and the slope of the water surface under capacity discharge will approximately parallel the slope of the invert of the pipe.

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

Total Energy Losses At Structures

The following total energy head losses at structures shall be determined for inlets, manholes, wye branches, or bends in the design of full flow closed conduits. Note total energy losses, h_e include minor losses, h_j and the change in velocity head, h_v . See Figures 5-4 and 5-4b for details of each case. Minimum head loss used at any structure shall be 0.10 feet, unless otherwise approved.

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

Minor Head Losses at Structures

The basic equations for minor head losses, where there is significant significant upstream and downstream velocity, takes the form as set forth below with the various conditions of the coefficient, k_m , shown in Tables 5-5, 5-6 and 5-7.

$$h_j = k_m \frac{V_2^2 - V_1^2}{2g} \quad (5-4)$$

where:

h_j = junction or structure minor 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_m = junction or structure coefficient of loss, in feet.

In the case where the initial velocity is negligible or when there is no velocity change the basic equation for head loss becomes:

$$h_j = k_m \frac{V_2^2}{2g} \quad (5-5)$$

Table 5-5

Junction or Structure

Minor Loss Coefficient k_m

Case No.	Reference Figure	Description of Condition	Coefficient k_m
VI	5-4b	Inlet or Manhole at Beginning of Line	1.25
VII	5-4b	Conduit on Curves for 90 degrees*	
		Curve radius = diameter	0.50
		Curve radius = 2 to 8 diameters	0.40
		Curve radius = 8 to 20 diameters	0.25
VIII	5-4b	Bends where Radius is Equal to Diameter	
		90-degree Bend	0.50
		60-degree Bend	0.43
		45-degree Bend	0.35
		22-1/2-degree Bend	0.20
		Manhole on Line with 60-degree Lateral	0.35
		Manhole on Line with 22-1/2-degree Lateral	0.75

* Where bends other than 90 degrees are used, the 90-degree bend coefficient can be used with the following percentage factor applied:

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

Obstructions

The values of the coefficient, k_m , for determining the loss of head due to obstructions in pipes are shown in Table 5-6, and the coefficients are used in the following equation to calculate the head loss at the obstruction:

$$h_j = k_m \frac{V^2}{2g} \quad (5-6)$$

Table 5-6
Head Loss Coefficients Due to Obstructions

$\frac{A^*}{A.}$	k_m	$\frac{A^*}{A.}$	k_m
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 area of opening
at obstruction

Expansions and Contractions

The values of the coefficient, k_m , for determining the loss of head due to sudden enlargements and sudden contractions in pipes are shown in Table 5-7. These coefficients are used in the following equation to calculate the head loss at the change in section:

$$h_j = k_m \frac{V_2^2}{2g} \quad (5-7)$$

where: V_2 = velocity in smaller pipe

Table 5-7

Head Loss Coefficients for Expansions And Contractions

$\frac{D_2^*}{D_1}$	Sudden Enlargements k_m	Sudden Contractions k_m
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.

5.40 Design Procedure For Storm Sewer Systems

Preliminary Design Procedure

- A. Prepare a drainage map of the entire area to be drained by proposed improvements. Contour maps serve as excellent drainage area maps when supplemented by field reconnaissance.
- B. Make a preliminary layout of the proposed storm drainage 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 a code identification number, the size of area, the direction of surface runoff by small arrows, and the coefficient of runoff for the area.
- E. Show all existing underground utilities.
- F. Establish design rainfall frequency.
- G. Establish inlet time of concentration.
- H. Establish the typical cross section of each street.
- I. Establish permissible spread of water on all streets within the drainage area.
- J. Include Steps A through I with plans submitted to the City of Dodge City for review. The drainage map submitted shall be suitable for permanent filing with the appropriate agency and shall be a good-quality reproducible copy.

Inlet System

Determining the size and location of inlets is largely a trial-and-error procedure. Using criteria outlined in Sections 2, 3, and 4 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.
- D. If an inlet is to be used to remove water from the street, size the inlet(s) and record the inlet size, 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 Steps 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 intercepted by the inlet. 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 used.

Storm Sewer System

After the computation of the quantity of storm runoff entering each inlet, the storm sewer system required to carry the runoff is designed. 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. This situation is due to the fact that as the time of concentration increases the rainfall intensity decreases.

A. Storm Sewer Pipe

The ground-line profile is now used in conjunction with the previous runoff calculations. The elevation of the hydraulic gradient is arbitrarily established approximately 2 feet below the ground surface. When this initial gradient is set and the design discharge is determined, a Manning's flow chart may be used to determine the pipe size and velocity.

It is probable that the initial gradient will have to be adjusted at this point since the intersection of the discharge and 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 2 foot criteria below the ground surface. If there is 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's Flow Chart based on a given discharge, pipe size, and slope.

B. Junctions, Inlets and Manholes

- A. Determine the hydraulic gradient elevations at the upstream end and downstream end of the pipe section in question. The elevation of the hydraulic gradient of the upstream end of pipe is equal to the elevation of the downstream end of pipe (hydraulic gradient) plus the product of the length of pipe and the pipe gradient.
- 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_j , at junction, inlet, or manhole at design point from Tables 5-5, 5-6, 5-7 or Figures 5-4a and 5-4b.
- G. Compute head loss at junction, inlet or manhole.

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

C. Major Storm System

Check the proposed system for the 100-year major storm event. Modify the proposed system or provide additional flow capacity as required to accommodate the major storm runoff according to the requirements stated in Sections 1, 3, 4, and 5.

5.50 Bibliography

1. The American Society of Civil Engineers and the Water Pollution Control Federation, Design and Construction of Sanitary and Storm Sewers, A.S.C.E. Manual of Engineering Practice, No. 37 (W.P.C.F. Manual of Practice, No. 9) American Society of Civil Engineers, New York, New York, 1960.
2. City of Austin, Texas, Engineering Department, et al, Drainage Criteria Manual, Austin, Texas, 1977.
3. Albertson, Maurice L., Barton, James R., and Simons, Daryl B., Fluid Mechanics for Engineers. Prentice-Hall, Inc., Englewood Cliffs, New Jersey, 1960.
4. Chow, Ven Te, Open-Channel Hydraulics, McGraw-Hill Book Company, Inc., New York, New York, 1959.
5. Federal Aviation Administration, Airport Drainage, Government Printing Office, Washington, D.C., 1966.
6. National Clay Pipe Institute, Clay Pipe Engineering Manual, National Clay Pipe Institute, Crystal Lake, Illinois, 1968.
7. Portland Cement Association, Design and Construction of Concrete Sewers, Portland Cement Association, Skokie, Illinois, 1968.
8. Portland Cement Association, Handbook of Concrete Culvert Pipe Hydraulics, Portland Cement Association, Chicago, Illinois, 1964.
9. Sangster, W.M., Wood, H.W., Smendon, E.T., and Bossy, H.G., Pressure Changes of Storm Drain Junctions, Engineering Series Bulletin No. 41, University of Missouri, Columbia, Missouri, 1958.
10. Steel, Ernst, W., Water Supply and Sewerage, McGraw-Hill Book Co., Inc., New York, New York, 1960.
11. Wright-McLaughlin Engineers, Urban Storm Drainage Criteria Manual, Denver Regional Council of Governments, Colorado, 1969.

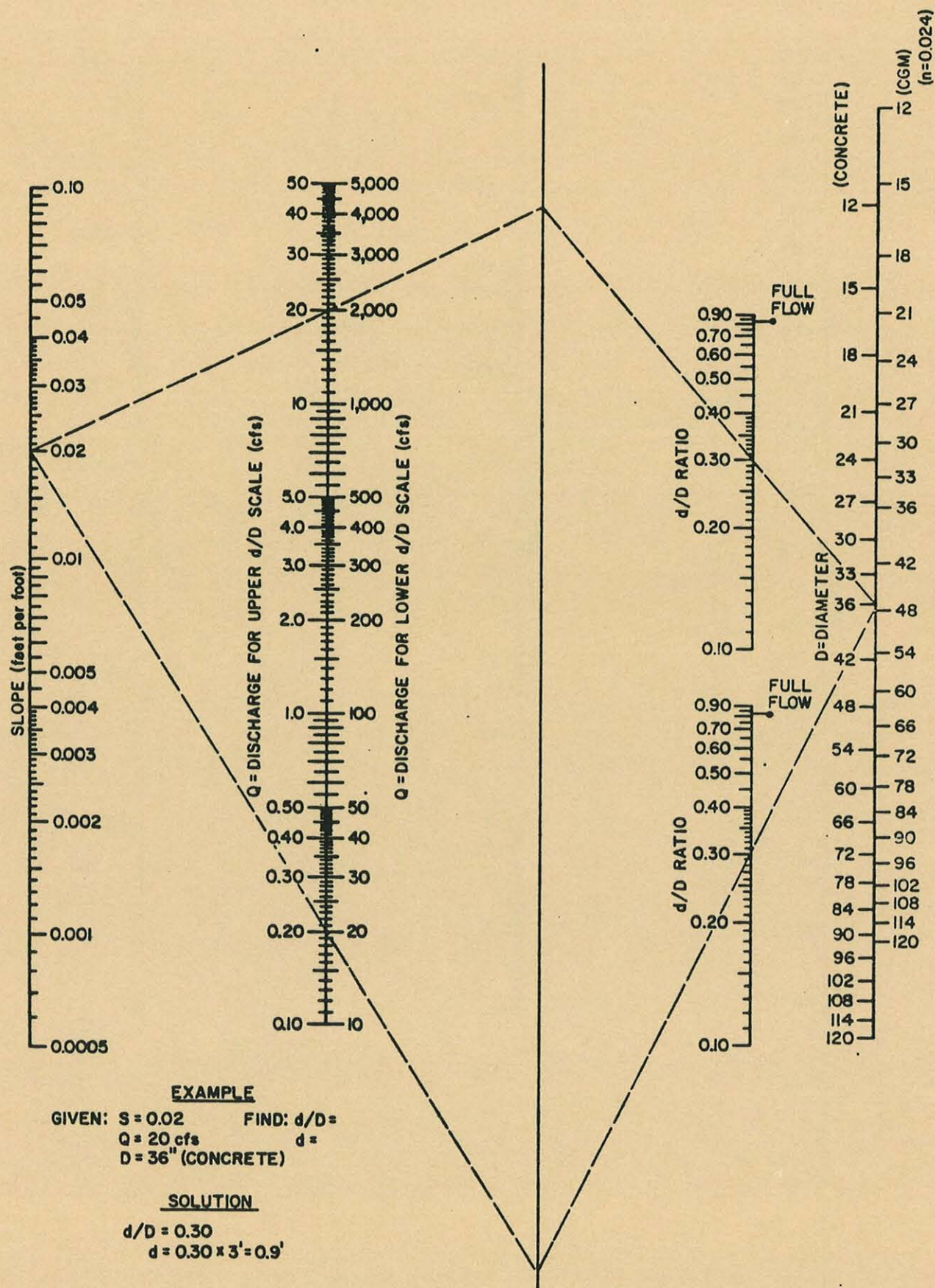


Figure 5-1 Uniform Flow for Pipe Culverts

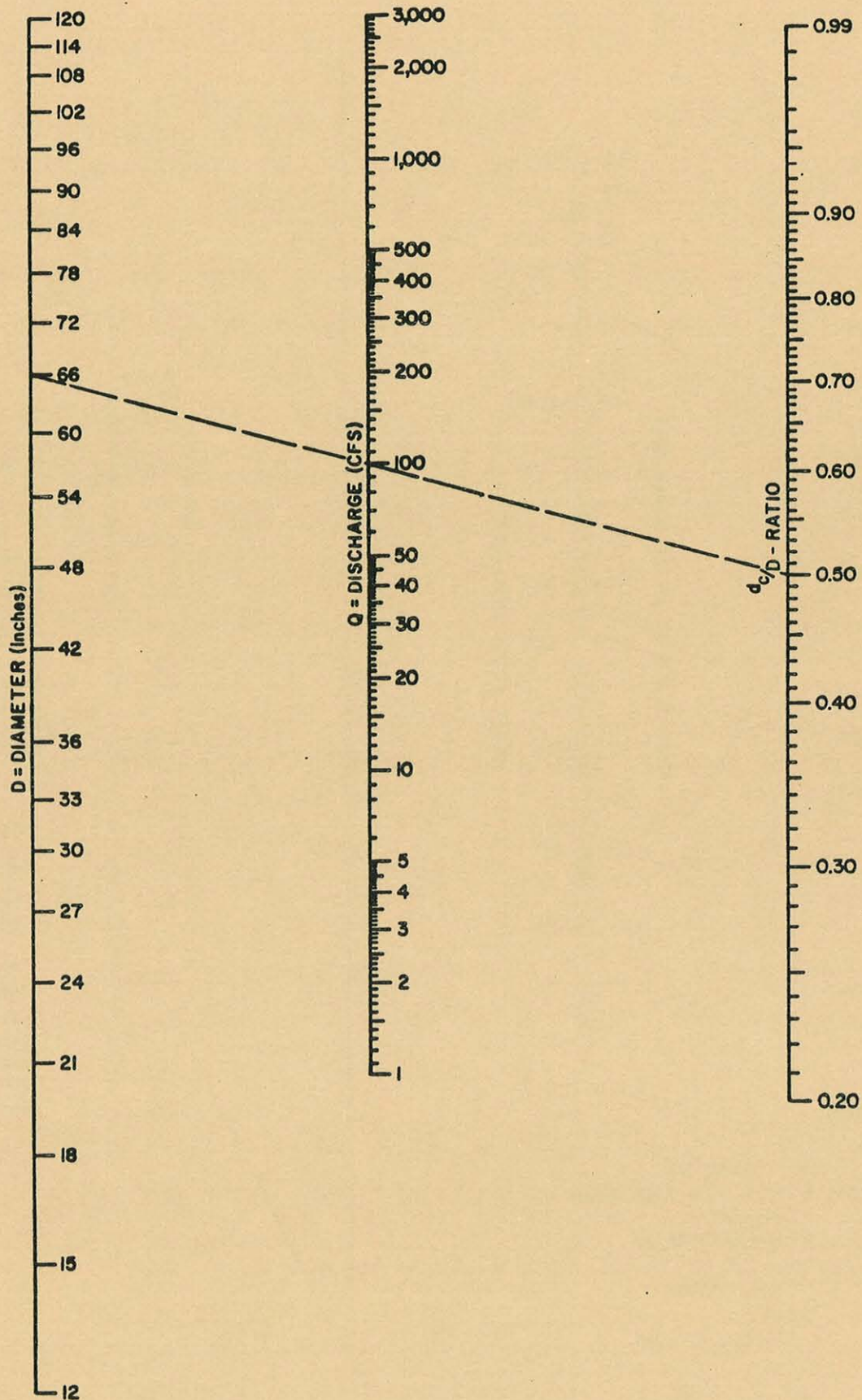


Figure 5-2 Critical Depth of Flow for Circular Conduits

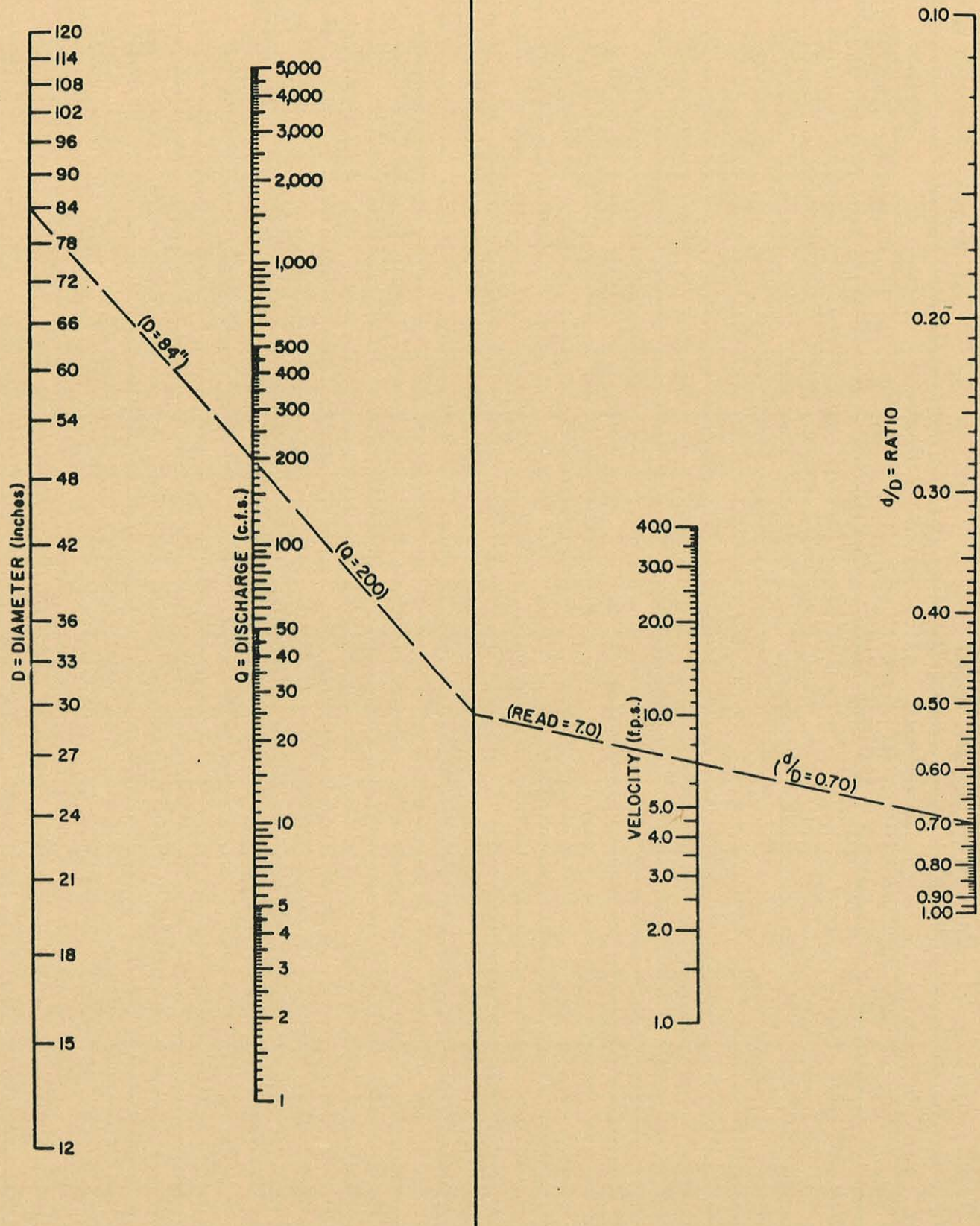


Figure 5-3 Velocity in Pipe Conduits

A - acres
i - inches/hr.
Q - cfs

COMPUTATION FORM
DRAINAGE & STORM SEWER DESIGN
RATIONAL METHOD

Comp. by _____ date _____

Clk by _____ date _____

[illegible]

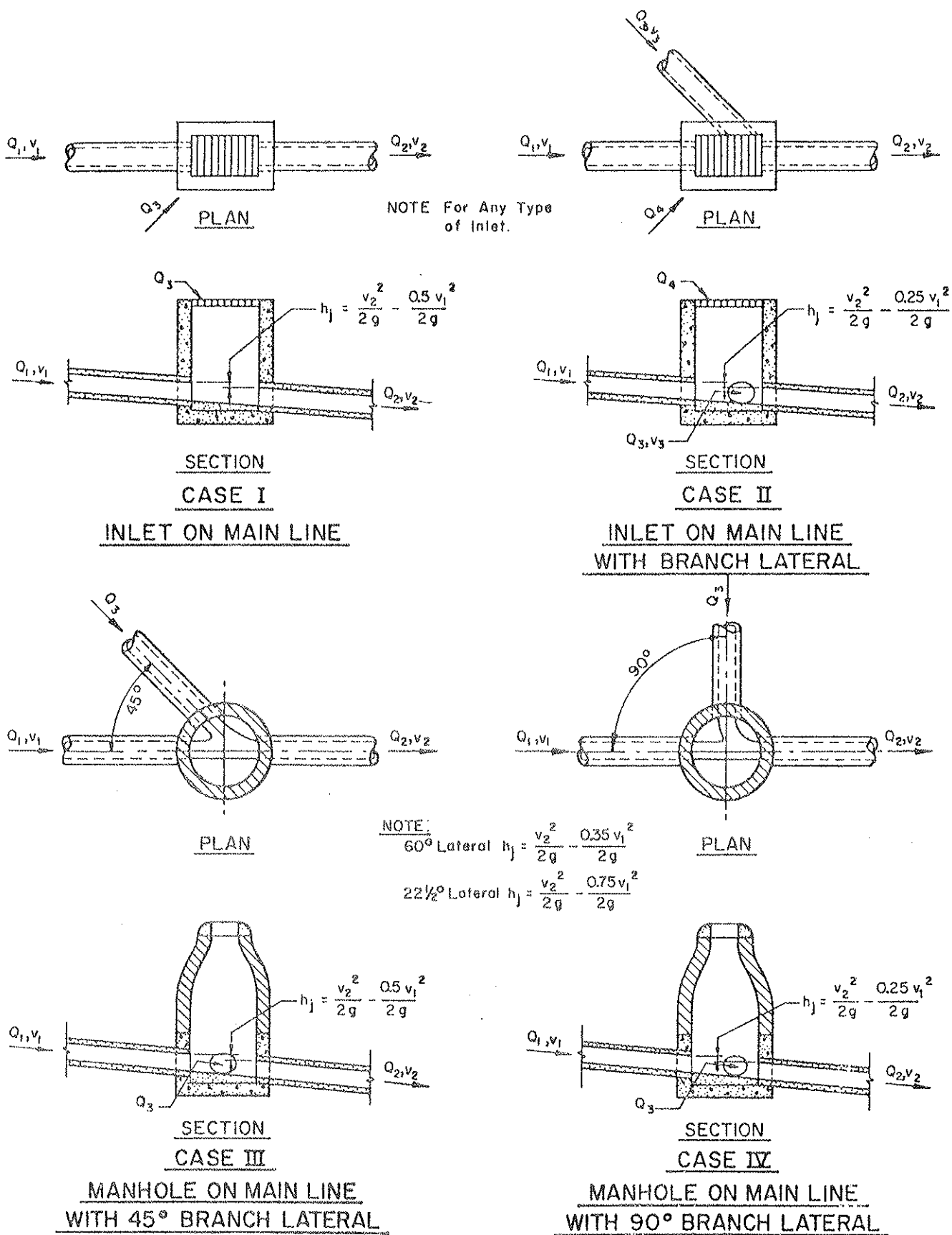
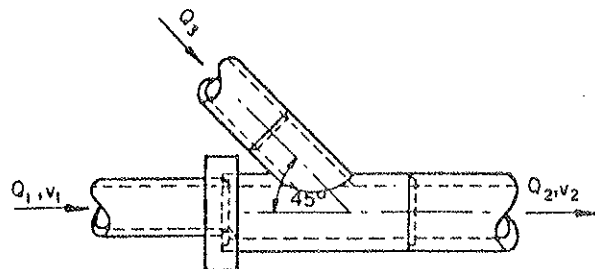
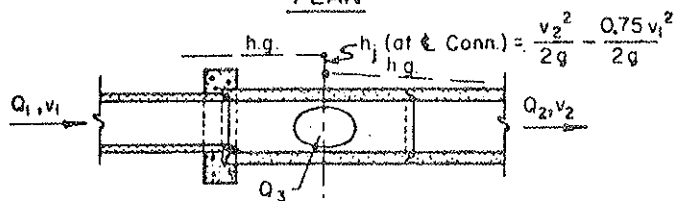


Figure 5-4a Minor Head Losses Due to Turbulence at Structures



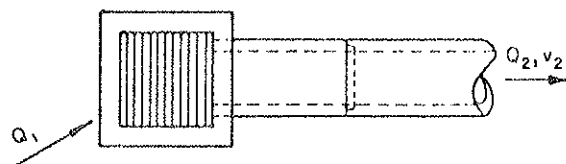
PLAN



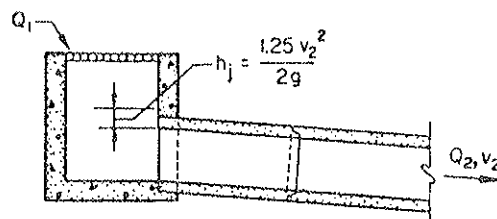
SECTION

CASE V

45° WYE CONNECTION OR CUT IN



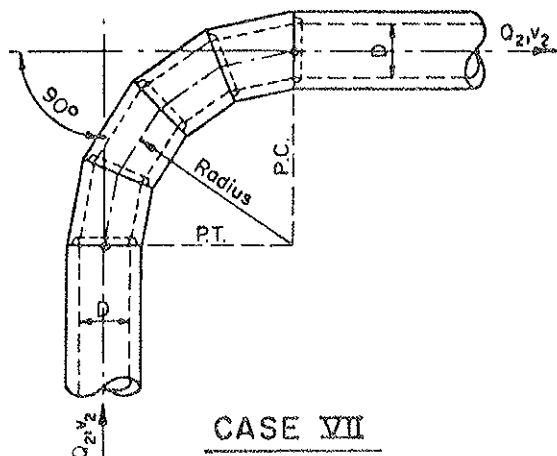
PLAN



SECTION

CASE VI

INLET OR MANHOLE AT BEGINNING OF LINE



CASE VII

CONDUIT ON 90° CURVES *

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

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

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

Radius = (8-20) Dia. of Pipe $h_j = 0.25 \frac{v_2^2}{2g}$

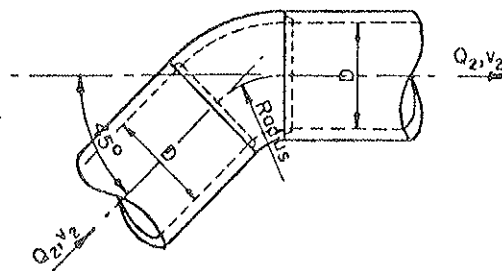
Radius = Greater than 20 Dia. of Pipe $h_j = 0$

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

60° curve 85%

45° curve 70%

22 1/2° curve 40%



CASE VIII

BENDS WHERE RADIUS IS EQUAL TO DIAMETER OF PIPE

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 $h_j = 0.20 \frac{v_2^2}{2g}$

Figure 5-4b Minor Head Losses Due To Turbulence at Structures

Section 6

Culvert Design

- 6.10 Design Criteria
 - Design Frequency
 - Culvert Discharge Velocities
- 6.20 Culvert Types
- 6.30 End Treatments
 - Conditions at Entrance
 - Parallel Headwall and Endwall
 - Flared Headwall and Endwall
 - Warped Headwall and Endwall
 - Improved Inlets
- 6.40 Culvert Design with Standard Inlets
 - Culvert Sizing
 - Design Procedure
 - Design Computation Forms
 - Invert Elevations
 - Culvert Diameter
 - Limited Headwater
 - Culvert Outlet
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- 6.50 Culvert Design with Improved Inlets
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 - Dimensional Limitations
- 6.60 Design Figures
- 6.70 List of Symbols
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Section 6

Culvert Design

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.

6.10 Design Criteria

The design flow shall be determined by the Rational Method or the SCS Unit Hydrograph Procedure as set forth in Section 2.

Design Frequency

Culverts shall be designed to pass the 10-year runoff with a two-foot freeboard and no flow over the roadway. The drainage system shall accommodate a 100-year flood, including provision for limited overflows at bridges and culverts without loss of life or major property damage. At the option of the appropriate agency the designer may design culverts as storm sewers with a 10-year design frequency. Under this option the proposed culvert shall be placed on line and grade to permit connection to the future storm sewer. The major storm impacts shall be investigated for both the culvert configuration and the proposed storm sewer configuration. The major storm analysis of the storm sewer shall include upstream and downstream reaches sufficient to show that provisions for the major storm can be made when the storm sewer is constructed.

In areas where an official floodway exists, increases in the 100-year water-surface elevation shall not be greater than 1 foot above the natural 100-year water-surface elevation.

Culvert Discharge Velocities

The velocity of discharge from culverts should be limited as shown in Table 6-1. Consideration must be given to the effect of high velocities, eddies or other turbulence on the natural channel, downstream property and roadway embankment.

TABLE 6-1

Culvert Discharge Velocity Limitations

<u>Downstream Condition</u>	<u>Maximum Allowable Discharge Velocity (fps)</u>
Earth	6 fps
Seeded or Sodded Earth	8 fps
Paved or Riprap Apron	15 fps

It is recommended that a minimum velocity of 2.5 feet per second at the design flow be maintained in all drainage structures to prevent siltation. Where doubt exists concerning silt or scour, protection commensurate with

the value of the structure and surrounding property shall be installed to insure that damage to or failure of the structure will not occur.

6.20 Culvert Types

Culverts shall be selected based on hydraulic principles, economy of size and shape, and with a resulting headwater depth which will not cause damage to adjacent property for the 100-year storm. It is essential to the proper design of a culvert that the conditions under which the culvert will operate are known. Five types of operating conditions are discussed below.

Type I Flowing Part Full with Outlet Control and Tailwater Depth Below Critical Depth.

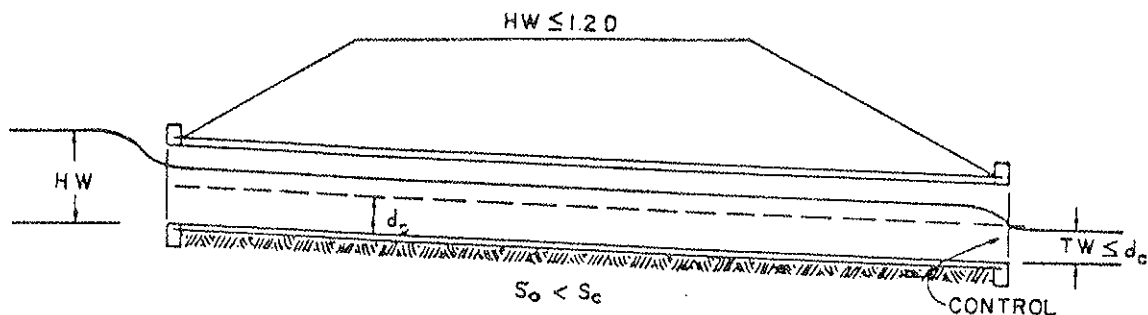


Figure 6-1

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 ($TW \leq d_c$).

The above condition is a common occurrence where the natural channels are on flat grades and have wide, flat floodplains. 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 is governed by the following equation when the approach velocity is considered zero.

$$HW = d_c + \frac{V_c^2}{2g} + h_e + h_f - S_0L \quad (6-1)$$

where:

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 (Figure 6-25 or 6-26);

D = diameter of pipe or height of box, in feet;

V_c = critical velocity, in feet per second, occurring at critical depth;

h_e = entrance head loss, in feet, where:

$$h_e = k_e \frac{V_c^2}{2g} \quad (6-2)$$

k_e = entrance loss coefficient (Table 6-2, page 6-54);

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 1.1 d_c ;

S_0 = slope of culvert, in feet per foot; and

L = length of culvert, in feet.

Type II Flowing Part Full with Outlet Control and Tailwater Depth Above Critical Depth.

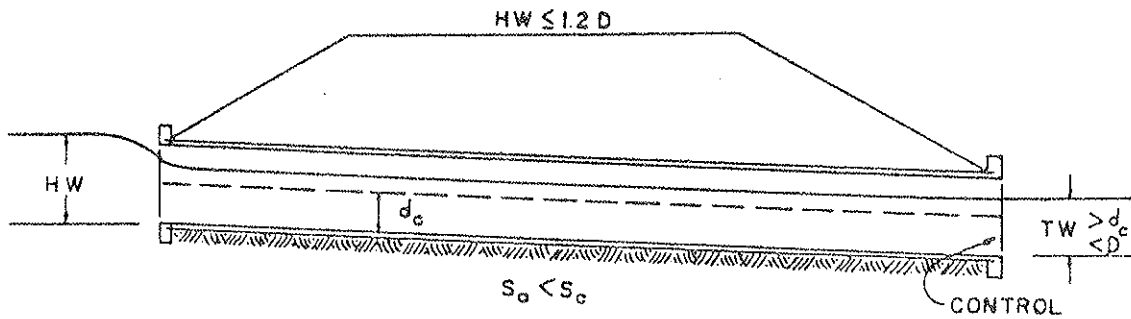


Figure 6-2

CONDITIONS

The entrance is unsubmerged ($HW < 1.2D$), the slope at design discharge is subcritical ($S_0 < S_c$), 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 design discharge is subcritical, and the tailwater depth is above critical depth, the control occurs at the outlet. The capacity of the culvert is governed by the following equation when the approach velocity is considered zero.

$$HW = TW + \frac{V_{TW}^2}{2g} + h_e + h_f - S_o L \quad (6-3)$$

where:

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, in feet;

V_{TW} = culvert discharge velocity, in feet per second, at tailwater depth;

h_e = entrance head loss, in feet, where;

$$h_e = k_e \frac{V_{TW}^2}{2g} \quad (6-4)$$

k_e = entrance loss coefficient (Table 6-2, page 6-54);

h_f = friction head loss, in feet = $S_f L$;

S_f = friction slope or slope that will produce uniform flow, in feet per foot. For Type II operation the friction slope is based upon TW depth;

S_o = slope of culvert, in feet per foot; and

L = length of culvert, in feet.

Type III Flowing Part Full with Inlet Control.

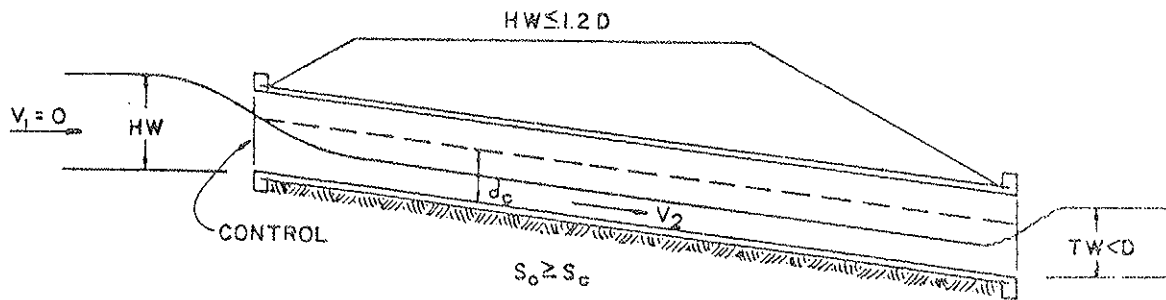


Figure 6-3

CONDITIONS

The entrance is unsubmerged ($HW < 1.2D$) and the slope at design discharge is equal to or greater than critical slope (supercritical) ($S_0 \geq S_c$).

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 operating under "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 is governed by the following equation when the approach velocity is considered zero.

$$HW = d_c + \frac{V_2^2}{2g} + k_e \frac{V_2^2}{2g} \quad (6-5)$$

where:

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 (Figure 6-25 or 6-26).

V_2 = critical velocity at entrance of 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 (Table 6-2).

Type IVA Flowing Full with Submerged Outlet.

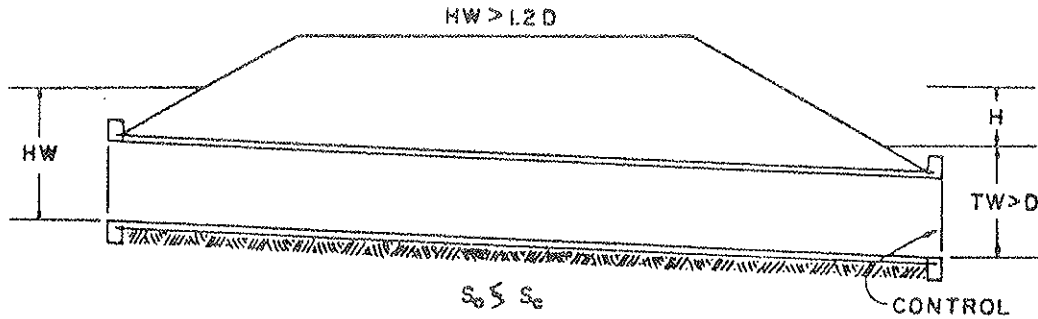


Figure 6-4

CONDITIONS

(Submerged Outlet)

The entrance is submerged ($HW \geq 1.2D$). The tailwater completely submerges the outlet.

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 installations. For an outlet to be submerged, the depth at the outlet must be equal to or greater than the diameter of pipe or height of box. The capacity of a culvert flowing full with a submerged outlet is governed by the following equation when the approach velocity is considered zero. Outlet velocity is based on full-pipe flow at the outlet.

$$HW = H + TW - S_0L \quad (6-6)$$

where:

HW = headwater depth, in feet, 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, in feet.

TW = tailwater depth, in feet.

S_0 = slope of culvert, in feet per foot.

L = length of culvert, in feet.

Type IVB Flowing Full with Partially Submerged Outlet.

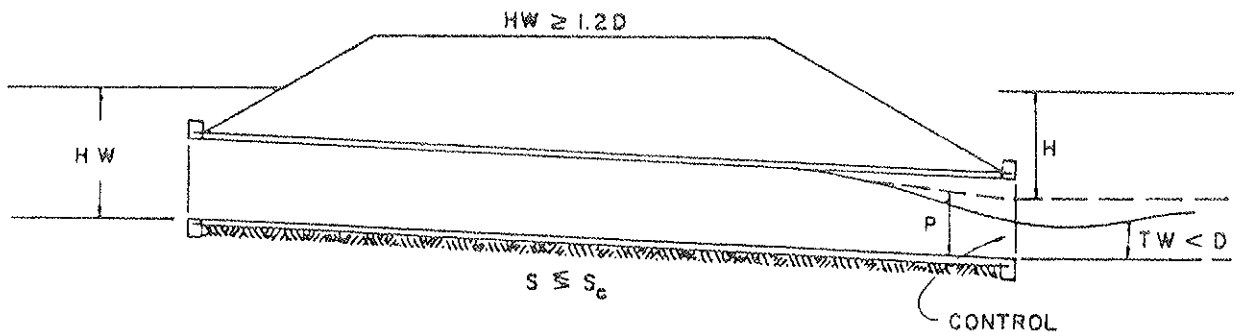


Figure 6-5

CONDITIONS

(Partially Submerged Outlet)

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

The capacity of a culvert flowing full with a partially submerged outlet is 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.

$$HW = H + P - S_0L \quad (6-7)$$

where:

HW = headwater Depth, in feet, 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 culverts flowing full, in feet.

P = pressure line height = $\frac{d_c + D}{2}$, in feet.

d_c = critical depth, in feet.

D = diameter or height of structure, in feet.

S_0 = slope of culvert, in feet per foot.

L = length of culvert, in feet.

6.30 End Treatments

The normal functions of properly designed headwalls and endwalls are to anchor the culvert to prevent movement, to control erosion and scour resulting from excessive velocities and turbulence, and to prevent adjacent soil from sloughing into the waterway opening. All headwalls shall be constructed of reinforced concrete and may be either straight parallel headwalls, flared headwalls, or warped headwalls with or without aprons as may be required by site conditions.

Conditions at Entrance

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

$$h_e = k_e \frac{V^2}{2g} \quad (6-8)$$

where:

h_e = entrance head loss, in feet.

V = velocity of flow in culvert, in feet per second.

k_e = entrance loss coefficient (Table 6-2, page 6-54).

In general the following guidelines should be used in the selection of the type of headwall or endwalls:

Parallel Headwall and Endwall

1. Approach velocities are low (below 6 fps).
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.

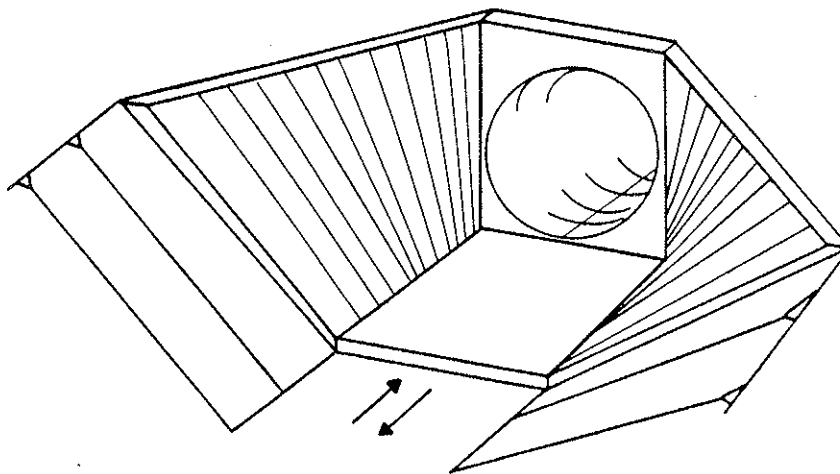
Flared Headwall and Endwall

1. Channel is well defined.
2. Approach velocities are between 6 and 10 fps.
3. Medium amounts of debris exist.

Warped (twisted) Headwall and Endwall

1. Channel is well defined and concrete lined.
2. Approach velocities are between 8 and 20 fps.
3. Medium amounts of debris exist.

These headwalls are effective with drop-down aprons to accelerate flow through culvert, and they are effective endwalls 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.



Warped (twisted) Headwall

Improved Inlets

Several types of improved inlets have been developed. The use of these inlets may provide substantial savings by a reduction in the barrel size of the proposed structure. The use of these inlets is optional and should be based on an economic analysis by the designer. For box culverts, reinforced concrete structures, and structures using headwalls the use of beveled inlets or tapered inlets is strongly recommended. For more information, the designer is referred to Hydraulic Design of Improvements for Culverts, Hydraulic Engineering Circular No. 13, August 1973, by the Federal Highway Administration.

6.40 Culvert Design with Standard Inlets

The information and publications necessary to design culverts according to the procedure given in this section can be found in Hydraulic Charts for the Selection of Highway Culverts, Hydraulic Engineering Circular No. 5, March 1965, and Capacity Charts for the Selection of Highway Culverts, Hydraulic Engineering Circular No. 10, March, 1965. Both of which are publications of the FHWA. Some of the charts and nomographs, from these publications, covering the more common requirements are located in the Design Charts at the end of this Section. For special cases and larger sizes, the FHWA publications should be used.

Culvert Sizing

Each chart, Figures 6-13 to 6-20, contains a series of curves which show the discharge capacity per barrel in cfs for each of several sizes of similar type culverts for various headwater depths in feet above the culvert invert at the inlet. The invert of the culvert is defined as the low point of its cross-section.

Each culvert size is described by two lines, one solid and one dashed. The numbers associated with each line are the ratio of the length, L , in feet, to the slope, $100 S_0$, in percent. The dashed lines represent the maximum $L/(100S_0)$ ratio for which the curves may be used without modification. The solid line represents the division between outlet and inlet control. For values of $L/(100S_0)$ less than that shown on the solid line, the culvert is operating under inlet control and the headwater depth is determined from the $L/(100S_0)$ value given on the solid line. The solid-line inlet-control curves are plotted from model test data. The dashed-line outlet-control curves were computed for culverts of various lengths with relatively flat slopes. Free outfall at the outlet was assumed, therefore, tailwater depth is assumed to not influence the culvert performance.

For culverts flowing under outlet control, the head loss at the entrance was computed, and the hydraulic roughness of the various materials used in culvert construction was taken into account in computing resistance loss for full or part-full flow. The Manning's "n" values used for each culvert type ranged from 0.012 to 0.032.

Except for large pipe sizes, headwater depths on the charts extend to three times the culvert height. Pipe arches and oval pipe show headwater up to 2.5 times their height since they are generally used in areas of low fill.

The dotted line, stepped across the charts, shows headwater depths of about twice the barrel height and indicates the upper limit of restricted use of the charts. Above this line the headwater elevation should be checked with the nomographs found in Figures 6-21 through 6-33.

The headwater depth given by the charts is actually the difference in elevation between the culvert invert at the entrance and the total head, that is depth plus velocity head, for flow in the approach channel. In most cases, the water surface upstream from the inlet is close to this level and the chart determination may be used as headwater depth for practical design purposes. Where the approach velocity is in excess of 3.0 feet per second, the velocity head must be subtracted from the curve determination of headwater to obtain the actual headwater depth.

Proper use of the capacity charts (Figures 6-13 thru 6-20) will minimize problems of scour or sedimentation. The procedure for sizing the culvert is summarized below. Data can be tabulated in the Design Computation Form, Figure 6-6.

1. List design data: Q (cfs), L (ft), allowable HW (ft), S_0 (ft/ft), type of culvert barrel and entrance.
2. Compute $L/(100S_0)$.
3. Enter the appropriate capacity chart with the design discharge, Q .
4. Find the $L/(100S_0)$ value for the smallest pipe that will pass the design discharge. If this value is above the dotted line in Figures 6-13 to 6-20, use the nomographs to check headwater conditions.
5. If the computed $L/(100S_0)$ is less than the value of $L/(100S_0)$ given for the solid line then the value of HW is the value obtained from the solid line curve. If the computed $L/(100S_0)$ is larger than the value for the dashed outlet-control curve, then special measures must be taken, and the reader is referred to the FHWA publications.

Check the HW value obtained with the allowable HW. If the indicated HW is greater than the allowable HW, then try the next larger pipe size.

The use of the nomographs on Figures 6-21 to 6-33 is limited to cases where tailwater depth is higher than the critical depth in the culvert. The advantage of the capacity charts over the nomographs is that the capacity charts are direct where the nomographs are trial and error. The capacity charts can be used only when the flow passes through critical at the outlet. When the critical depth at the outlet is less than the tailwater depth, the nomographs must be used; however, both give the same results where either of the two methods are applicable. The procedure for design requires the use of both nomographs and is as follows:

1. List design data: Q (cfs), L (ft), invert elevations at an inlet and outlet (ft), allowable HW(ft), mean and maximum flood velocities

in the natural stream (ft/sec), type of culvert, and entrance type for first selection.

2. Determine a trial size by assuming a maximum average velocity based on channel considerations to compute the area, $A = Q/V$.
3. Find HW for trial size culvert for inlet control and outlet control. For inlet control use Figures 6-27 to 6-33, connect a straight line through D and Q to Scale 1 of HW/D scales and project horizontally to the proper Scale (2 or 3). Compute HW and, if too large or too small, try another size before computing HW for outlet control.

Next, compute the HW for outlet control with Figures 6-21 to 6-24. Enter the graph with the length, the entrance coefficient for the entrance type, and the trial size. Connect the length scale and the culvert-size scale with a straight line, pivot on the turning line, and draw a straight line from the design discharge on the discharge scale through the turning point to the head scale (head loss, H). Compute HW from the equation:

$$HW = H + h_0 + LS_0$$

For TW greater than or equal to the top of the culvert, $h_0 = TW$, and for TW less than the top of the culvert:

$$h_0 = \frac{(d_c + D)}{2} \text{ or } TW$$

whichever is the greater. If TW is less than d_c , the nomographs cannot be used. See Figures 6-25 and 6-26 for critical-depth charts.

4. Compare the computed headwaters and use the higher HW to determine if the culvert is under inlet or outlet control. If outlet control governs and the HW is unacceptable, select a larger trial size and find another HW with the outlet-control nomographs. Since the smaller size of culvert had been selected for allowable HW by the inlet-control nomographs, the inlet control for the larger pipe need not be checked.

Design Procedure

Due to problems arising from topography and other considerations, the actual design of a culvert installation is more difficult than the simple process of sizing culverts. The procedure is a guide to design since the problems encountered are too varied and too numerous to be generalized. However, the actual process presented should be followed to insure that some special problem is not overlooked.

Design Computation Form

The use of a design computation form is a convenient method to use to obtain consistent designs with a minimum of culvert cost. An example of such a

Figure 6-6 Design Computation Form For Culverts

form is Figure 6-6.

Invert Elevations

After determining the allowable headwater elevation, the tailwater elevation, and the approximate length, invert elevations must be assumed. When considering ponded and non-ponded inlets, either for the design discharge or for some lesser storm which will not cause ponding, scour is not likely to occur in an artificial channel, such as a roadside ditch or a major drainage channel, when the culvert has the same slope as the channel. To reduce the chance of failure due to scour, invert elevations corresponding to the natural grade should be used as a first trial. For Figure 6-6 natural channels, the flow velocity in the channel upstream from the culvert should be investigated to determine if scour will occur.

Culvert Diameter

After the invert elevations have been assumed and using the design computation forms, the capacity charts, and the nomographs, the diameter of pipe that will meet the headwater requirements should be determined. The smallest diameter that appears in the nomographs and capacity charts is 12 inches. Since smaller diameter pipes are often closed by silt, it is recommended that pipe smaller than 18 inches not be used for any drainage where this manual applies. Since the pipe roughness influences the culvert diameter, both concrete and corrugated metal pipe should be considered in design, if both will satisfy the headwater requirements.

Limited Headwater

If there is insufficient headwater elevation to obtain the required discharge, it is necessary to oversize the culvert barrel, lower the inlet invert, use an irregular cross section, or use any combination of the preceding.

If the inlet invert is lowered, special consideration must be given to scour. The use of gabions or concrete drop structures, riprap, and headwalls with aprons and toe walls should be investigated and compared to obtain the proper design.

Culvert Outlet

The outlet velocity must be checked to determine if excessive scour will occur downstream. If scour is indicated, then riprap, an expanding end-section, or a more sophisticated energy dissipating structure should be used as discussed in Section 8 of this manual.

Minimum Slope

To prevent sediment from plugging the culvert, the culvert slope must be equal to or greater than the slope required to maintain a minimum velocity of 2.5 feet per second. The slope should be checked for each design, and if the proper minimum velocity is not obtained, the pipe diameter may be decreased, the slope steepened, a smoother pipe used, or a combination of these may be used.

6.50 Culvert Design with Improved Inlets

The objective of this design procedure is the hydraulic design of culverts, using improved inlets. This design procedure hinges on the selection of a culvert barrel based on its outlet-control performance curve, which is unique when based on elevation. The culvert inlet is then manipulated using edge improvements and adjustment of its elevation in order to achieve inlet-control performance compatible with the outlet-control performance. The resultant culvert design will best satisfy the criteria set by the designer and make optimum use of the barrel selected for the site.

Design Procedure

The flow chart shown in Figure 6-7 outlines the steps of the design procedure, and each step is discussed in detail below. Design calculation forms and design charts and tables that are included are located at the end of this section.

Step 1. Determine and Analyze Site Characteristics.

Step 2. Perform Hydrologic Analysis.

Step 3. Perform Outlet-Control Calculations and Select Culvert.

Outlet-control 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 (HW_0) exceeding the allowable headwater elevation (AHW EL.). For use in this procedure, the equation for headwater is in terms of elevation.

The full-flow outlet-control performance curve for a given culvert (size, inlet edge, shape, or material) defines its maximum performance. Therefore, inlet improvements beyond the beveled edge or changes in inlet invert elevation will not reduce the required outlet control headwater elevation. This makes the outlet-control performance curve an ideal limit for improved inlet design.

When the barrel size is increased, the outlet-control curve is shifted to the right, indicating a higher capacity for a given head (see Figure 6-8). Also, it may be generally stated that increased barrel size will flatten the slope of the outlet-control curve.

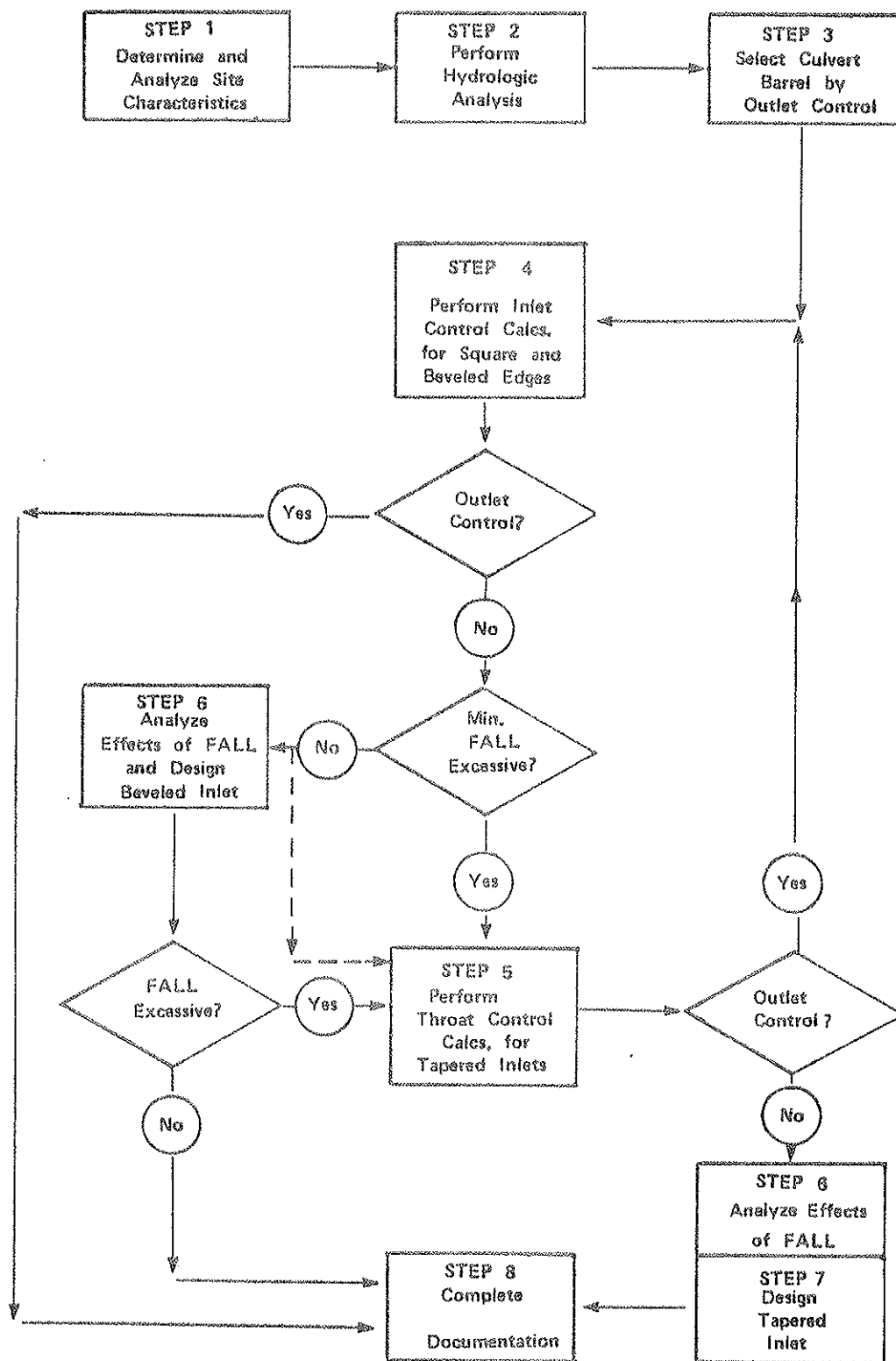


Figure 6-7 Culvert Design Flow Chart

The outlet-control curve passing closest to and below the AHW EL for the design Q, on the performance-curve graph defines the smallest possible barrel which will meet the hydraulic design criteria. However, that curve may be very steep (rapidly increasing headwater requirements for discharges higher than design), or the use of such a small barrel may not be practical. The proper culver selection procedure is:

- a. Calculate HW_0 at design discharge for trial culvert sizes, entrance conditions, shapes, and materials.
- b. Calculate headwater elevations at two additional discharge values in the vicinity of design Q in order to define outlet-control performance.
- c. Plot outlet-control performance curves for trial culvert sizes.
- d. Select barrel size, shape, and material.

This selection should not be based solely on calculations which indicate that the required headwater at the design discharge is near the AHW EL., but should also be based on outlet velocity as affected by material selection, the designer's evaluation of site characteristics, and the possible consequences of a flood occurrence in excess of the estimated design flood. A sharply rising outlet-control performance curve may be sufficient reason to select a culvert of different size, shape, or material.

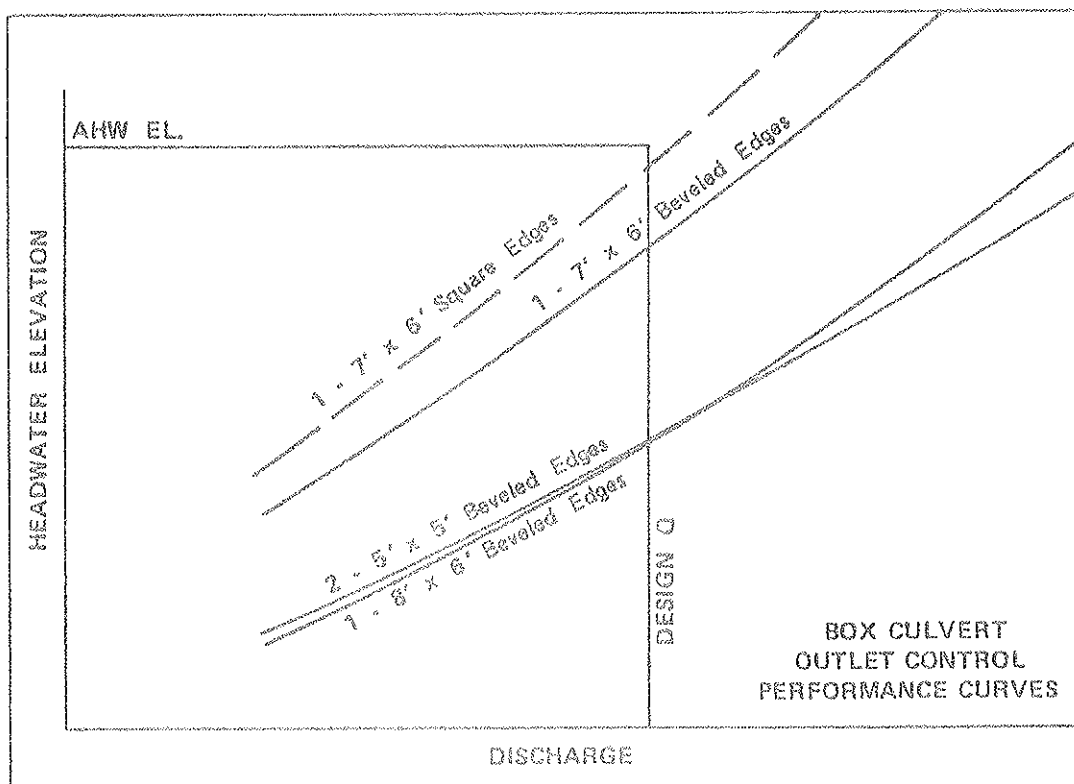


Figure 6-8

In order to calculate the barrel size required in outlet control, the applicable outlet-control nomograph may be used as follows:

- a. Intersect the "Turning Line" with a line drawn between Discharge and Head, H . To estimate H , use the following equation:
$$H = \text{AHW EL.} - \text{EL. Outlet Invert} - h_0$$

where h_0 may be selected as a culvert height. Accuracy is not critical at this point.

- b. Using the point on the "Turning Line", k_e , and the barrel length, draw a line defining the barrel size.

This size gives the designer a good first estimate of the barrel size and more precise sizing will follow rapidly.

Step 4. Perform Inlet-Control Calculations for Conventional and Beveled-Edge Culvert Inlets

The calculation procedure is similar to that used in HEC No. 5, except that headwater is defined as an elevation rather than a depth, a FALL may be incorporated upstream of the culvert face, and performance curves are an essential part of the procedure. The depression or FALL should have dimensions as described for side-tapered inlets.

- a. Calculate the required headwater depth, H_f , at the culvert face at design discharge for the culvert selected.
- b. Determine required face invert elevation to pass design discharge by subtracting H_f from the AHW EL.
- c. If this invert elevation is above the stream bed elevation at the face, the invert would generally be placed on the stream bed and the culvert will then have a capacity greater than design Q with headwater at the AHW EL.
- d. If this invert elevation is below the stream bed elevation at the face, the invert must be depressed, and the amount of depression is termed the FALL.
- e. Add H_f to the invert elevation to determine HW_f . If HW_f is lower than HW_0 , the barrel operates in outlet control at design Q . Proceed to Step 8, Figure 6-7.
- f. If the FALL is excessive in the designer's judgment from the standpoint of aesthetics, economy and other engineering reasons, a need for inlet geometry refinements is indicated. If square edges were used in Steps 3 and 4 above, repeat with beveled edges. If beveled edges were used, proceed to Step 5.

- g. If the FALL is within acceptable limits, determine the inlet-control performance by calculating required headwater elevation using the flow rates from Step 3 and the FALL determined above. $HW_f = H_f + EL. \text{ face invert.}$
- h. Plot the inlet-control performance curve with the outlet-control performance curve plotted in Step 3.
- i. Proceed to Step 6.

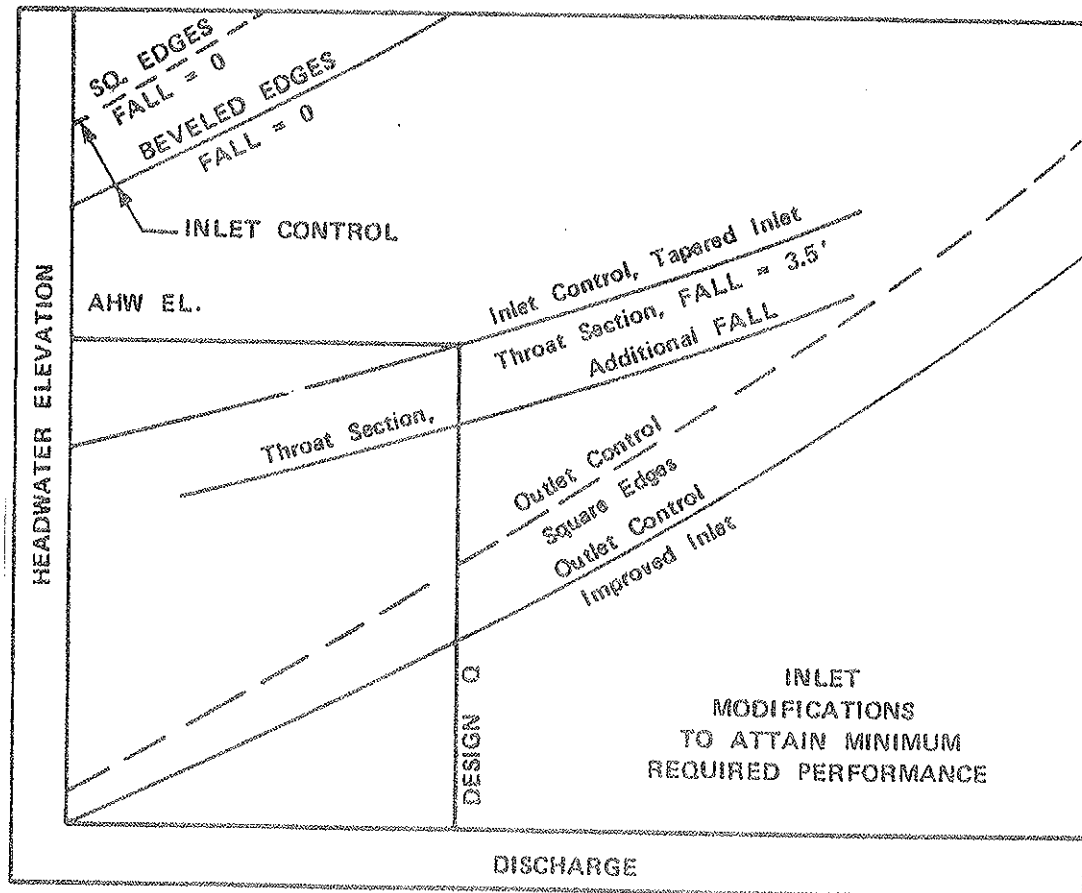


Figure 6-9

Step. 5 Perform Throat-Control Calculations for Side- and Slope-Tapered Inlets

The same concept is involved here as with conventional or beveled-edge culvert design.

- a. Calculate required headwater depth on the throat (H_t) at design Q for the culvert selected in Step 3.
- b. Determine required throat elevation to pass design discharge by subtracting H_t from the AHW EL.
- c. If this throat invert elevation is above the stream bed elevation, the invert would probably be placed on the stream bed and the culvert throat will have a capacity greater than the design Q with headwater at the AHW EL.
- d. If this throat invert elevation is below the stream bed elevation, the invert must be depressed, and the elevation difference between the stream bed at the face and the throat invert is termed the FALL. If the FALL is determined to be excessive, a larger barrel must be selected. Return to Step 5(a).
- e. Add H_t to the invert elevation to determine HW_t . If HW_t is lower than HW_o , the culvert operates in outlet control at design Q . In this case, adequate performance can probably be achieved by the use of beveled edges with a FALL. Return to Step 4.
- f. Define and plot the throat-control performance curve.

Step 6. Analyze the effect of FALLS on Inlet-Control Section Performance

It is apparent from Figure 6-9 that either additional FALL or inlet improvements would increase the culvert capacity in inlet control by moving the inlet-control performance curve to the right toward the outlet-control performance curve. If the outlet-control performance curve of the selected culvert passes below the point defined by the AHW EL. and the design Q , there is an opportunity to optimize the culvert design by selecting the inlet so as to either increase its capacity to the maximum at the AHW EL. or to pass the design discharge at the lowest possible headwater elevation.

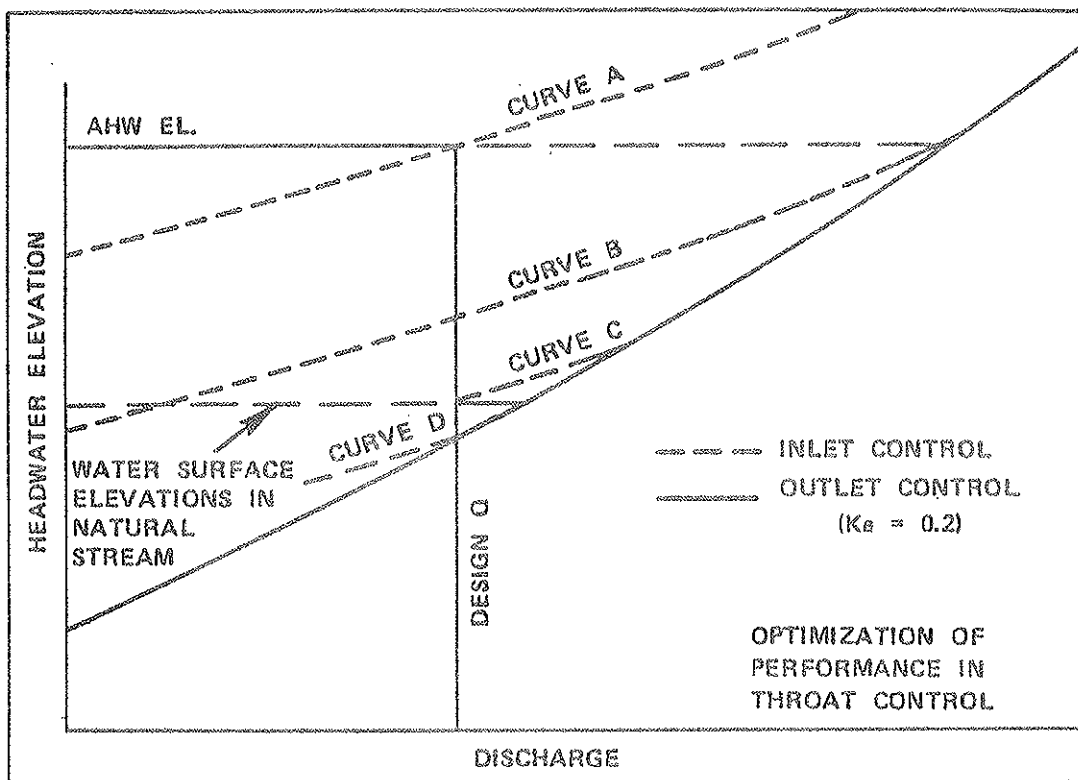


Figure 6-10

Some possibilities are illustrated in Figure 6-10. The minimum inlet-control performance which will meet the selected design criteria is illustrated by Curve A. This design has merit in that minimum expense for inlet improvements and/or FALL is incurred and the inlet will pass a flood in excess of design Q before performance is governed by outlet control. This performance is adequate in many locations, including those locations where headwaters in excess of the AHW EL. would be tolerable on the rare occasion of floods in excess of design Q.

Curve B illustrates the performance of a design which takes full advantage of the potential capacity of the selected culvert and the site to pass the maximum possible flow at the AHW EL. A safety factor in capacity is thereby incorporated in the design. This can be accomplished by the use of a FALL, by geometry improvements at the inlet or by a combination of the two. Additional inlet improvement and/or FALL will not increase the capacity at or above the AHW EL.

There may be reason to pass the design flow at the lowest possible headwater elevation even though the reasons are insufficient to cause the AHW EL. to be set at a lower elevation. The maximum possible reduction in headwater at design Q is illustrated by Curve C. Additional inlet improvement and/or FALL will not reduce the required headwater elevation at design Q.

The water-surface elevation in the natural stream may be a limiting factor in design, i.e., it is not productive to design for headwater at a lower elevation than natural streamflow elevations. The reduction in headwater elevation illustrated by Curve C is limited by natural water surface elevations in the stream. If the water surface elevations in the natural stream had fallen below Curve D, this curve would illustrate the maximum reduction in headwater elevation at design Q. Tailwater depths calculated by assuming normal depth in the stream channel may be used to estimate natural water-surface elevations in the stream at the culvert inlet. These may have been computed as a part of Step 3.

Curve A has been established in either Step 4 for conventional culverts or Step 5 for improved inlets. To define any other inlet-control performance curve such as B, C, or D for the same control section:

- a. Select a point on the outlet-control performance curve.
- b. Measure the vertical distance from this point to Curve A. This is the difference in FALL between Curve A and the curve to be established, e.g., the FALL on the control section for Curve A plus the distance between Curves A and B is the FALL on the control section for Curve B.

For conventional culverts only:

- c. Estimate and compare the costs incurred in FALLS (structural excavation and additional culvert length) to achieve various levels of inlet performance.
- d. Select design with increment in cost warranted by increased capacity and improved performance.
- e. If FALL required to achieve desired performance is excessive, proceed to Step 5.
- f. If FALL is acceptable and performance achieves the design objective, proceed to Step 8.

Step 7. Design Side- and/or Slope-Tapered Inlet (Figures 36 through 39).

Side-tapered inlets have an enlarged face area with a transition to the culvert barrel accomplished by tapering the sidewalls. The inlet has equal barrel and face heights except for the addition of a top bevel at the face. Slope-tapered inlets provide a greater head at the control section. The capacity depends largely upon the amount of FALL between the invert at the face and the invert at the throat section.

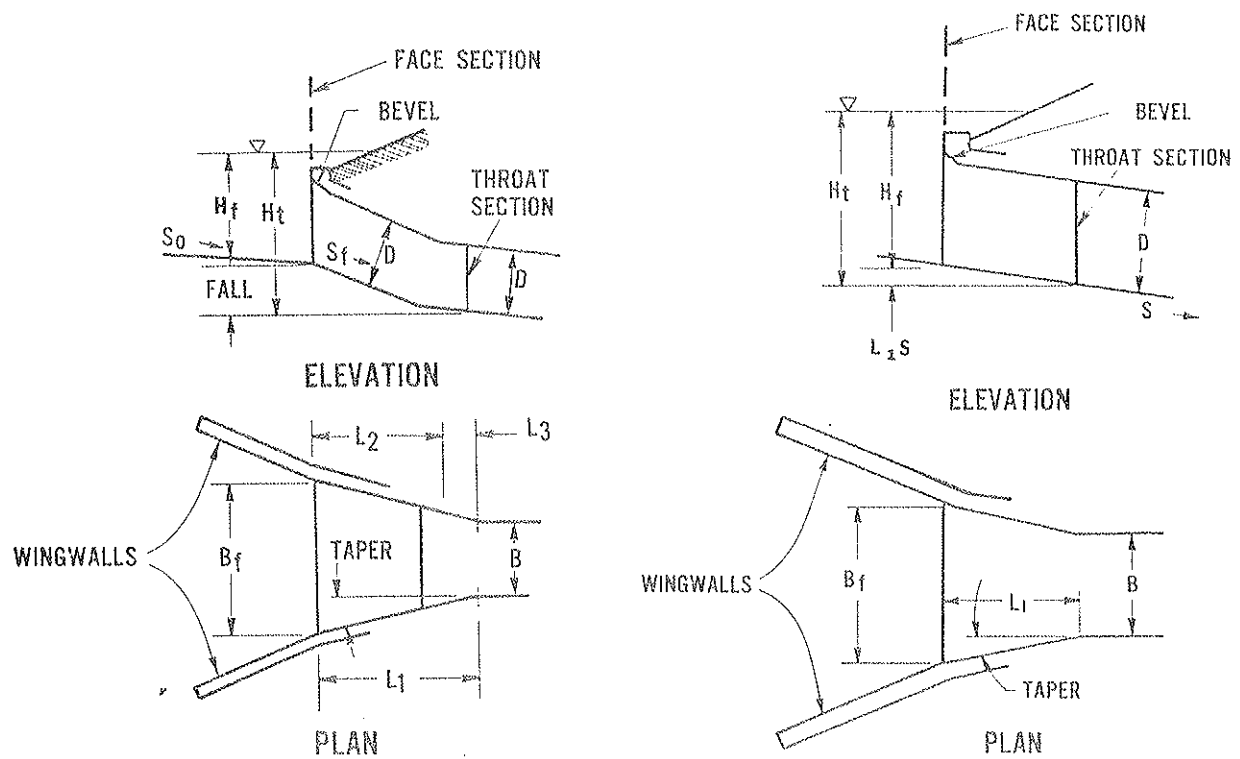


Figure 6-10a
Slope and Side Tapered Inlets

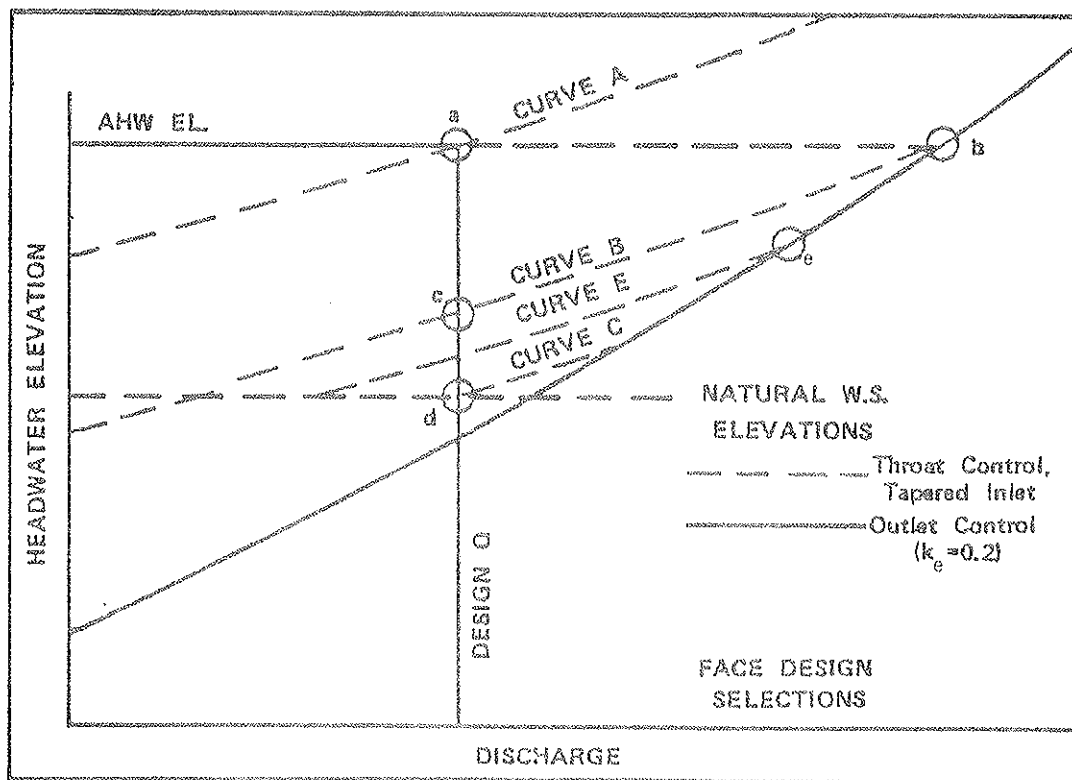


Figure 6-11

The face is the entrance of the barrel perpendicular to the flowline. The throat lies within the barrel section and is the intersection of the transition section and the barrel. The face or throat control is that section which governs the hydraulics of the structure.

Either a side- or slope-tapered inlet may be used if a FALL is required on the throat by use of a depression (FALL) upstream of the face of a side-tapered inlet or a FALL in the inlet of a slope-tapered inlet.

The face of the side- or the slope-tapered inlet should be designed to be compatible with the throat performance defined in Step 6. The basic principles of selecting the face design are illustrated in Figure 6-11.

The minimum face design is one whose performance curve does not exceed the AHW EL. at design Q. However, a "balanced" design requires that full advantage be taken of the increased capacity and/or lower headwater requirement gained through use of various FALLS. This suggests a face-control curve which intersects the throat-control curve: (1) at the AHW EL., (2) at design Q, (3) at its intersection with the outlet-control curve, or (4) other. These options are illustrated in Figure 6-11 by points a through e representing the intersections of face-control curves with the throat-control curves. The options are explained as follows: (1) Intersection of face- and throat-control curves at the AHW EL. (Point a or b): For the minimum acceptable throat-control performance (Curve A), this is the minimum face size that can be used without the required headwater elevation (HW_f) exceeding the AHW EL. at design Q (Point a). For throat-control performance greater than minimum but equal to or less than Curve B, this is the minimum face design which makes full use of the FALL placed on the throat to increase culvert capacity at the AHW EL. (Point b). (2) Intersection of face- and throat-control curves at design Q (Points a, c, or d): This face design results in throat control at discharges equal to or greater than design Q. It makes full use of the FALL to increase capacity and reduce headwater requirements at flows equal to or greater than the design Q. (3) Intersection of the face-control curve with throat-control curve at its intersection with the outlet-control curve (Points b or e): This option is the minimum face design which can be used to make full use of the increased capacity available from the FALL placed on the throat. It cannot be used where HW_f would exceed AHW EL. at design Q; e.g., with the minimum acceptable throat-control curve. (4) Variations in the above options available to the designer. The culvert face can be designed so that culvert performance will change from face control to throat control at any discharge at which inlet control governs. Options (1) through (3), however, fulfill design objectives of minimum face size to achieve the maximum increase in capacity possible for a given FALL, or the maximum possible decrease in the required headwater for a given FALL for any discharge equal to or greater than design Q.

Figure 6-12 illustrates the optional tapered-inlet designs possible. Note that the inlet dimensions for the side-tapered inlet are the same for all options. This is because performance of the side-tapered inlet nearly parallels the performance of the throat, and an increase in headwater on the throat by virtue of an increased FALL results in an almost equal increase in headwater on the face. Each foot of FALL on the throat of a culvert with a side-tapered inlet requires additional barrel length equal to the fill

slope; e.g., if the fill slope is 3:1, use of 4 ft of FALL rather than 3 ft results in a culvert barrel 3 ft longer as well as increased culvert capacity and/or reduced headwater requirements.

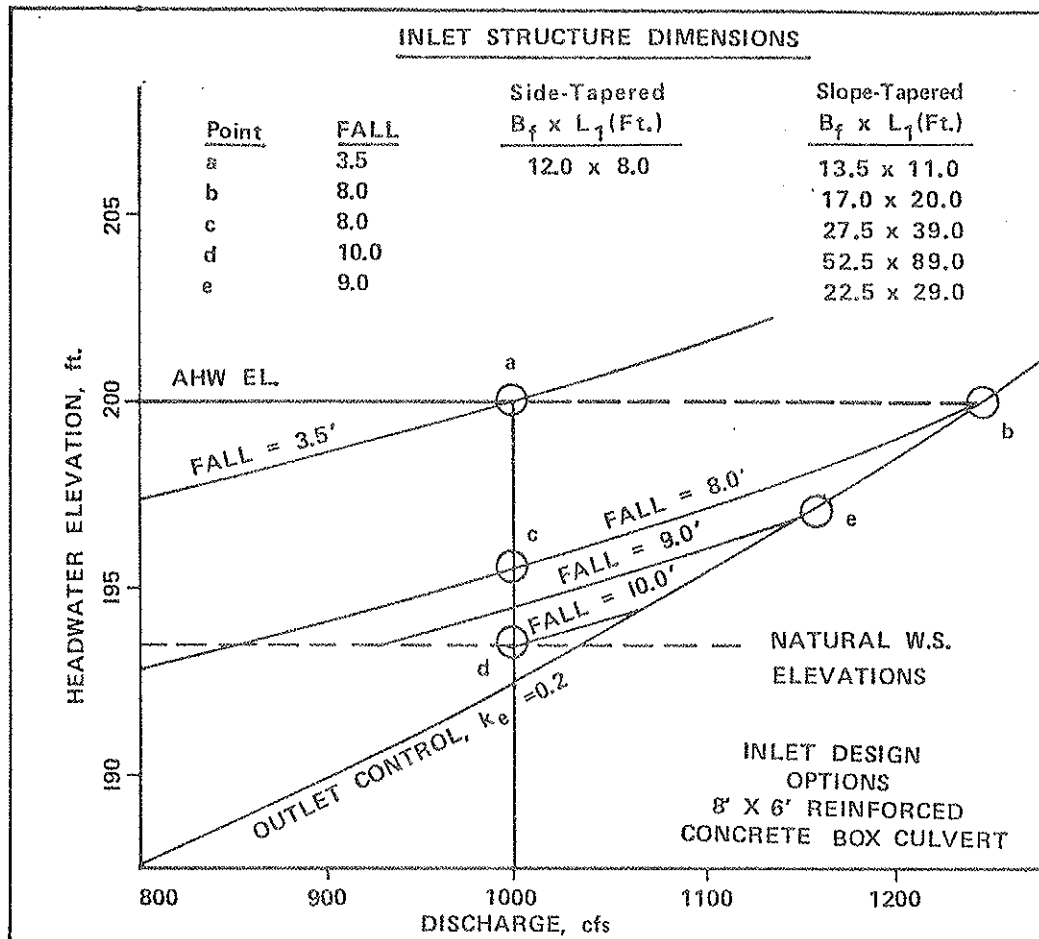


Figure 6-12

Face dimensions and inlet length increase for the slope-tapered inlet as the capacity of the culvert is increased by additional FALL on the throat. No additional head is created for the face by placing additional FALL on the throat. On the other hand, use of a greater FALL at the throat of a culvert with a slope-tapered inlet does not increase culvert length.

The steps followed in the tapered-inlet designs are:

- a. Compute H_f for side- and slope-tapered inlets for various FALLS at design Q and other discharges. Side-Tapered Inlet: $H_f = H_t - 1.0$ (approximate). Slope-Tapered Inlet: $H_f = \text{HW EL.} - \text{Streambed EL.}$

at face.

- b. Determine dimensions of side- and slope-tapered inlets for trial options.
- c. For slope-tapered inlets with mitered face, check for crest control.
- d. Compare construction costs for various options, including the cost of FALL on the throat.
- e. Select design with incremental cost warranted by increased capacity and improved performance.

From the above, it is apparent that in order to optimize culvert design, performance curves are an integral part of the design procedure. At many culvert sites, designers have valid reasons for providing a safety factor in designs. These reasons include uncertainty in the design discharge estimate, potentially disastrous results in property damage or damage to the highway from headwater elevations which exceed the allowable, the potential for development upstream of the culvert, and the chance that the design flood will be exceeded during the life of the installation.

Dimensional Limitations

Side-Tapered Inlets (See List of Symbols.)

1. $6:1 \geq \text{Taper} \geq 4:1$
Tapers greater than 6:1 may be used but performance will be underestimated.
2. Wingwall flare angle from 15 degrees to 26 degrees with top edge beveled or from 26 degrees to 90 degrees with or without bevels.
3. If FALL is used upstream of face, extend barrel slope upstream from face a distance of $D/2$ before sloping upward more steeply.
4. For pipe culverts, these additional requirements apply:
 - a. $D \leq E \leq 1.1D$
 - b. Length of square to round transition $\geq 0.5D$
 - c. $\text{FALL} \geq D/4$
 - d. $P \geq 3T$
 - e. $W_p = B_f + T$ or $4T$, whichever is larger.

Slope-Tapered Inlet (See List of Symbols.)

1. $6:1 \geq \text{Taper} \geq 4:1$

Tapers $> 6:1$ may be used, but performance will be underestimated.

2. $3:1 \geq S_f \geq 2:1$

If $S_f > 3:1$, use side-tapered design

3. Minimum $L_3 = 0.5B$

4. $1.5D \geq \text{FALL} \geq D/4$

For $\text{FALL} < D/4$, use side-tapered design

For $\text{FALL} > 1.5D$, estimate friction losses between face and throat.

5. Wingwall flare angle from 15 degrees to 26 degrees with top edge beveled or from 26 degrees to 90 degrees with or without bevels.
6. For pipe culvert, these additional requirements apply:
 - a. Square to circular transition length $> 0.5D$.
 - b. Square-throat dimension equal to barrel diameter. It is not necessary to check square-throat performance.

Example 1

Box Culvert

Given: Design Discharge, $Q = 1,000$ cfs, for a 10-year recurrence interval

Slope of stream bed, $S_0 = 0.05$ ft/ft

Allowable Headwater Elevation = 200.0 ft msl

Elevation Outlet Invert = 172.5 ft msl

Culvert Length, $L_a = 350$ ft

Downstream channel approximates an 8-ft wide trapezoidal channel with 2:1 side slopes and a Manning's "n" of 0.03.

Requirements: This box culvert will be located in a rural area where the Allowable Headwater Elevation is not too critical; that is, the damages are low due to exceeding that elevation at infrequent times. Thus, the culvert should have the smallest possible barrel to pass design Q without exceeding AHW EL. Use a reinforced concrete box with $n = 0.012$.

PROJECT: <u>Example No. 1</u>		OUTLET CONTROL DESIGN CALCULATIONS		DESIGNER: <u>JMN</u>	
STATION: _____		DATE: <u>12-10-73</u>			

INITIAL DATA:
 Q 10 = 1000 cfs
 AHW El. = 200 ft.
 S_o = 0.05
 L_a = 350 ft.
 El. Outlet Invert 172.5 ft.

Stream Data:

n = 0.03
 S_o = 0.05
 B = _____

Barrel Shape and Material Rect. Conc. Box Barrel n = 0.012

SKETCH

First Approximation
 Q = 1000 cfs, k_e = 0.5, L_a = 350 ft.
 H = AHW El. - El. Outlet Invert - h_o
 = 200 - 172.5 - 5 = 22.5
 A = 40 ft.² or D = _____ ft; Try 7' x 6'

Q	$\frac{Q}{N}$	H	$\frac{Q}{NB}$	(1) d_c	$\frac{d_c + D}{2}$	Qn	(2) TW	(3) h_o	(4) HW ₀	(5) V_o	COMMENTS
Trial No. <u>1</u> , N = <u>1</u> , B = <u>7</u> , D = <u>6</u> , k_e = <u>0.5</u> Square edges											
1000	1000	21	143	76	6.0		3.5	6.0	199.5	23.8	OK - Close to AHW El.
800	800	13.2	114	76	6.0		3.25	6.0	191.7		
1200	1200	30	172	76	6.0		3.8	6.0	208.5		
Trial No. <u>2</u> , N = <u>1</u> , B = <u>7</u> , D = <u>6</u> , k_e = <u>0.2</u> Beveled edges											
1000	1000	19	143	76	6.0		3.5	6.0	197.5	23.8	OK - Lowered HW ₀ 2'
800	800	12		Same as				6.0	190.5		Try 1-6' x 6'
1200	1200	27		38. edge				6.0	205.5		
Trial No. <u>3</u> , N = <u>1</u> , B = <u>6</u> , D = <u>6</u> , k_e = <u>0.2</u> Beveled edges											
1000	1000	26	167	76	6.0		3.5	6.0	204.5	27.8	No good - Does not meet design criteria exceeds AHW El.

Notes and Equations:
 (1) d_c cannot exceed D
 (2) TW based on d_n in natural channel, or other downstream control.
 (3) $h_o = \frac{d_c + D}{2}$ or TW, whichever is larger.
 (4) $HW_0 = H + h_o + \text{El. Outlet Invert}$.
 (5) Outlet Velocity (V_o) = Q/Area defined by d_c or TW, not greater than D . Do not compute until control section is known.

SELECTED DESIGN

N = 1 At Design Q :
 B = 7 ft.
 D = 6 ft. HW₀ = 197.5 ft.
 k_e = 0.2 V_o = 23.8 f/s

* $H = \left[1 + k_e + \frac{29n^2 \cdot L}{R^{1.33}} \right] \frac{V^2}{2g}$

PROJECT: <u>Example No. 1</u>		CULVERT INLET CONTROL SECTION DESIGN CALCULATIONS		DESIGNER: <u>JMN</u>							
STATION: _____				DATE: <u>12-10-73</u>							
INITIAL DATA: Q <u>10</u> = <u>1000</u> cfs AHW El. = <u>200</u> ft. S ₀ = <u>0.05</u> L ₀ = <u>350</u> ft. El. Stream Bed at Face <u>190</u> ft. Barrel Shape and Material <u>RCB</u> Barrel n = <u>0.012</u> N = <u>1</u> , B = <u>7</u> D = <u>6</u> , NBD ^{3/2} = <u>102.2</u> (Table 3) (Pipe) ND ^{3/2} = _____		CONVENTIONAL or BEVELED INLET: FACE CONTROL SECTION (Upper Headings)		TAPERED INLET THROAT CONTROL SECTION (Lower Headings)							
DEFINITIONS OF INLET CONTROL SECTION											
	Q NB	H _f / D	H _f	(1) El. Face Invert	El. Stream Bed At Face	(2)	(3) HW _f	(4)	(5)	Note: Use Upper Headings for Conventional or Beveled Face; Lower Headings for Tapered Inlet Throat.	
Q	Q/ NBD ^{3/2}	H _f / D	H _f	El. Throat Invert		FALL	HW _f	S	V ₀		
COMMENTS											
Trial No. <u>1</u> Inlet and Edge Description <u>Beveled-edged Inlet</u>											
1000	143	3.9	23.4	176.6	192*	15.4	200			FALL too large, try tapered	
* Adjusted upstream due to FALL										inlet - Do not use bevel- edged inlet	
Trial No. <u>2</u> Inlet and Edge Description <u>Tapered inlet throat</u>											
1000	9.72	2.65	15.9	184.1	190	5.9	200	0.033	34.2	Ok - Calc. Perf. Curves	
800	7.79	2.05	12.3				196.4			From plot, Opportunity to gain	
1200	11.48	3.4	20.4				204.5			(FALL = 1.3 + 5.9 = 7.2') Max. Capacity at AHW = 200	
Trial No. <u>3</u> Inlet and Edge Description <u>Tapered inlet throat, FALL = 7.2'</u>											
1000			15.9	182.8	190	7.2	198.7	0.029	33.3	Ok - Capacity at AHW = 200	
800			12.3				195.1			1062 c.f.s.	
1200			20.4				203.2				
Notes and Equations: (1) El. Face (or throat) invert = AHW El. - H _f (or H _t) (2) FALL = El. Stream Bed at Face - El. face (or throat) invert (3) HW _f (or HW _t) = H _f (or H _t) + El. face (or throat) invert, where El. face (or throat) invert should not exceed El. stream bed. (4) S ≈ S ₀ - FALL/L ₀ (5) Outlet Velocity = Q/Area defined by d ₀ at S										SELECTED DESIGN Inlet Description: FALL = <u>7.2</u> ft. Invert El. = <u>182.8</u> ft. Bevels: Angle = _____ b = _____ in., d = _____ in.	

PROJECT: Example No. 1DESIGNER: JMNSIDE-TAPERED INLET
DESIGN CALCULATIONS

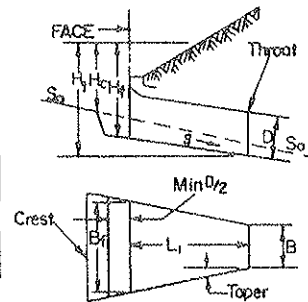
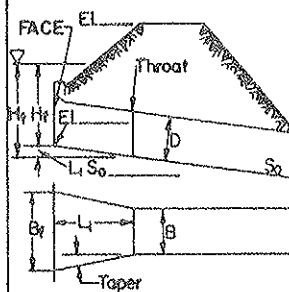
STATION: _____

DATE: 12-10-73

INITIAL DATA

Q 10 = 1000 cfsS₀ = 0.05AHW El. = 200 ft.L₀ = 350 ft.TAPER = 4 : 1Barrel Shape and Material RCB; n = 0.012Face Edge Description 45° Bevel'sN = 1, B = 7 ft, D = 6 ft

SKETCH



Q	El. Throat Invert	(1) $\frac{H_f}{D}$	(2) $\frac{Q}{B_f D^{3/2}}$	(3) $D^{3/2}$	(4) Min. B _f	(5) Min. A _f	(6) B _f	(7) L ₁	(8) S	(9) L ₁ S	(10) El. Face Invert	Upper Headings for Box Culverts, Lower Headings for Pipes COMMENTS
		$\frac{H_f}{D}$	$\frac{Q}{B_f D^{3/2}}$	$D^{3/2}$	Min. B _f	Min. A _f	B _f	L ₁	S	L ₁ S		

Trial No. 1, Q = 1000, HW_f = 200 (Min. required) FALL = 5.9'

1000	184.1	2.48	6.6	14.7	10.3	10.5	7.0	0.033	0.2	184.3	$B_f D^{3/2} [or A_f E^{1/2}] = 154$
900		2.14	5.84								$\frac{HW_f}{12.8} = 197.1$
1000		2.48	6.50								$\frac{HW_f}{14.5} = 198.8$
1100		2.77	7.14								$\frac{HW_f}{16.6} = 200.9$

Trial No. 2, Q = 1000, HW_f = 198.7 (FALL = 7.2')

1000	182.8	2.48	6.6	14.7	10.3	10.5	7.0	0.029	0.2	183.0	$B_f D^{3/2} [or A_f E^{1/2}] =$

Trial No. 3, Q = 1062, HW_f = 200 (FALL = 7.2')

1062	182.8	2.70	7.05	14.7	10.3	10.5	7.0	0.029	0.2	183.0	$B_f D^{3/2} [or A_f E^{1/2}] =$

Notes and Equations:

(1) $H_f/D [or H_f/E] = (HW_f - El. Throat Invert - 1)/D [or E]$

$D \leq E \leq 1.1D$

(2) Min. B_f = $Q / \left[\frac{D^{3/2}}{B_f D^{3/2}} \right]$

Min. A_f = $Q / \left[\frac{E^{1/2}}{A_f E^{1/2}} \right]$

(3) L₁ = $\left[\frac{B_f - NB}{2} \right]$ TAPER

(4) From throat design

(5) El. Face Invert - El. Throat Invert > 1 ft, recompute.

Face and Throat may be lowered to better fit site, but do not raise.

SELECTED DESIGN

B_f = 10.5 ft.

L₁ = 7.0 ft.

Bevels: Angle 45 °

d = 3 in., b = 5.3 in.

Crest Check:

HW_c = 198.7 ft.

H_c = 7.7 ft.

Q/W = 59 (Chart 17)

Min. W = 17 ft.

PROJECT: <u>Example No. 1</u>		SLOPE-TAPERED INLET DESIGN CALCULATIONS		DESIGNER: <u>JMN</u>	
STATION: _____				DATE: <u>12-10-73</u>	

INITIAL DATA: $Q = 1000$ cfs $S_o = 0.05$ AHW EL. <u>200</u> ft. $L_o = 350$ ft. El. Stream bed at crest <u>191</u> ft. El. stream bed at face <u>190</u> ft. TAPER = <u>4</u> : 1 (4:1 to 6:1) $S_f = 2$: 1 (2:1 to 3:1) Barrel Shape and Material <u>RCB</u>; $n = 0.012$ Inlet Edge Description <u>45° Bevels</u> N = <u>1</u>, B = <u>7</u> ft, D = <u>6</u> ft.	VERTICAL MITERED
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	Q	HWf	El. Throat Invert	(1) El. Face Invert	(2) Hf	$\frac{H_f}{D}$	$\frac{Q}{B_f D^{3/2}}$	$D^{3/2}$	(3) Min. Bf	Bf	S	Comments
Trial 1	1000	200	189.1	190	10	1.67	5.1	14.7	13.3	14	0.033	$B_f D^{3/2} =$ _____
												Vertical face point no. 1
Trial 2	1000	198.7	182.8	190	8.7	1.43	4.45	14.7	15.3	16	0.029	$B_f D^{3/2} =$ _____
												Vertical face point no. 2

Notes: Use only throat designs with FALL > 0.25D

(1) El. face Invert: Vertical = Approx. stream bed elevation
Mitered = El. Crest - y, where y = 0.4D (Approx.), but higher than throat invert elevation.

(2) $H_f = HW_f - \text{El. face invert}$

(3) Min. $B_f = Q / ((D^{3/2}) (Q / B_f D^{3/2}))$

(4) Min. L3	(5) L4	(6) L2	(7) Check L2	(8) Adj. L3	(9) Adj. TAPER	(10) L1	(11) W	$\frac{Q}{W}$	Hc	(12) Max. Crest El.	GEOMETRY
											$B_f =$ _____ ft. $L_3 =$ _____ ft. $L_1 =$ _____ ft. $L_4 =$ _____ ft. $L_2 =$ _____ ft. $d =$ _____ in. $b =$ _____ in. TAPER = _____ : 1
3.5	—	11.8	10.5	—	4.4:1	15.3					
3.5	—	14.4	14.4	3.6	—	18.0					

(4) Min. $L_3 = 0.5NB$ (5) $L_4 = S_f \times D / S_f$ (6) $L_2 = (\text{El. Face (Crest) Invert} - \text{El. Throat Invert}) S_f - L_4$ (7) Check $L_2 = \frac{B_f NB}{2}$ TAPER - L_3 (8) If (7) > (6), Adj. $L_3 = \frac{B_f - NB}{2}$ TAPER - L_2	(9) If (6) > (7) Adj. TAPER = $(L_2 + L_3) / \frac{B_f - NB}{2}$ (10) $L_1 = L_2 + L_3 + L_4$ (11) Mitered: $W = NB \times 2 \frac{L_1}{\text{TAPER}}$ (12) Max. Crest El. = $HW_f - H_c$
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PROJECT: Example No. 1

STATION: _____

SLOPE-TAPERED INLET
DESIGN CALCULATIONS

DESIGNER: JMN

DATE: 12-10-73

INITIAL DATA:

$Q = 1000$ cfs $S_o = 0.05$

AHW EL. 200 ft. $L_g = 350$ ft.

El. Stream bed at crest 191 ft.

El. stream bed at face 190 ft.

TAPER = 4 : 1 (4:1 to 6:1)

$S_f = 2$: 1 (2:1 to 3:1)

Barrel Shape and Material RCB $n = 0.012$

Inlet Edge Description 45° Bevels

N = 1, B = 7 ft., D = 6 ft.

	Q	HW _f	El. Throat Invert	(1) El. Face Invert	(2) H _f	H _f /D	$\frac{Q}{B_f D^{3/2}}$	D ^{3/2}	(3) Min. B _f	B _f	S	Comments
Trial 1	1062	200	182.8	190	10	1.67	5.1	19.7	14.2	15.0	0.029	B _f D ^{3/2} = _____
	Vertical face point no. 3											
Trial 2	1060	200	184.1	188.6	11.4	1.90	5.65	19.7	12.0	12.0	0.033	B _f D ^{3/2} = _____
	Mitered face point no. 1											

Note: Use only throat designs with FALL > 0.25D

(1) El. face invert: Vertical = Approx. stream bed elevation
Mitered = El. Crest - y, where y = 0.4D (Approx.), but higher than throat invert elevation.

(2) H_f = HW_f - El. face invert

(3) Min B_f = Q / ((D^{3/2}) (Q / (B_f D^{3/2})))

(4) Min. L ₃	(5) L ₄	(6) L ₂	(7) Check L ₂	(8) Adj. L ₃	(9) Adj. TAPER	(10) L _i	(11) W	$\frac{Q}{W}$	H _c	(12) Max. Crest El.	GEOMETRY B _f = ____ ft. L ₃ = ____ ft. L ₁ = ____ ft. L ₄ = ____ ft. L ₂ = ____ ft. d = ____ in. b = ____ in. TAPER = ____ : 1
3.5	—	14.4	12.5	—	3.5 : 1	17.9					
3.5	7.8	6.0	6.5	4.0	—	17.8	15.9	63.0	8.0	192.0	

(4) Min. L₃ = 0.5NB

(5) L₄ = S_f + D/S_f

(6) L₂ = (El. Face (Crest) Invert - El. Throat invert) S_f - L₄

(7) Check L₂ = $\frac{B_f - NB}{2}$ TAPER - L₃

(8) If (7) > (6), Adj. L₃ = $\frac{B_f - NB}{2}$ TAPER - L₂

(9) If (6) > (7) Adj. TAPER = (L₂ - L₃) / $\frac{B_f - NB}{2}$

(10) L_i = L₂ + L₃ + L₄

(11) Mitered: W = NB + 2 $\frac{L_i}{TAPER}$

(12) Max. Crest El. = HW_f - H_c

PROJECT: Example No. 1 SLOPE-TAPERED INLET DESIGN CALCULATIONS DESIGNER: J.M.N.
STATION: _____ DATE: 12-10-78

INITIAL DATA:
 $Q_{10} = 1000$ cfs $S_o = 0.05$
AHW EL. 200 ft. $L_o = 350$ ft.
El. Stream bed at crest 191 ft.
El. stream bed at face 190 ft.
TAPER = 4:1 (4:1 to 6:1)
 $S_f = 2$:1 (2:1 to 3:1)
Barrel Shape and Material RCB; $n = 0.012$
Inlet Edge Description 45° Bevels
 $N = 1$, $B = 7$ ft., $D = 6$ ft.

VERTICAL

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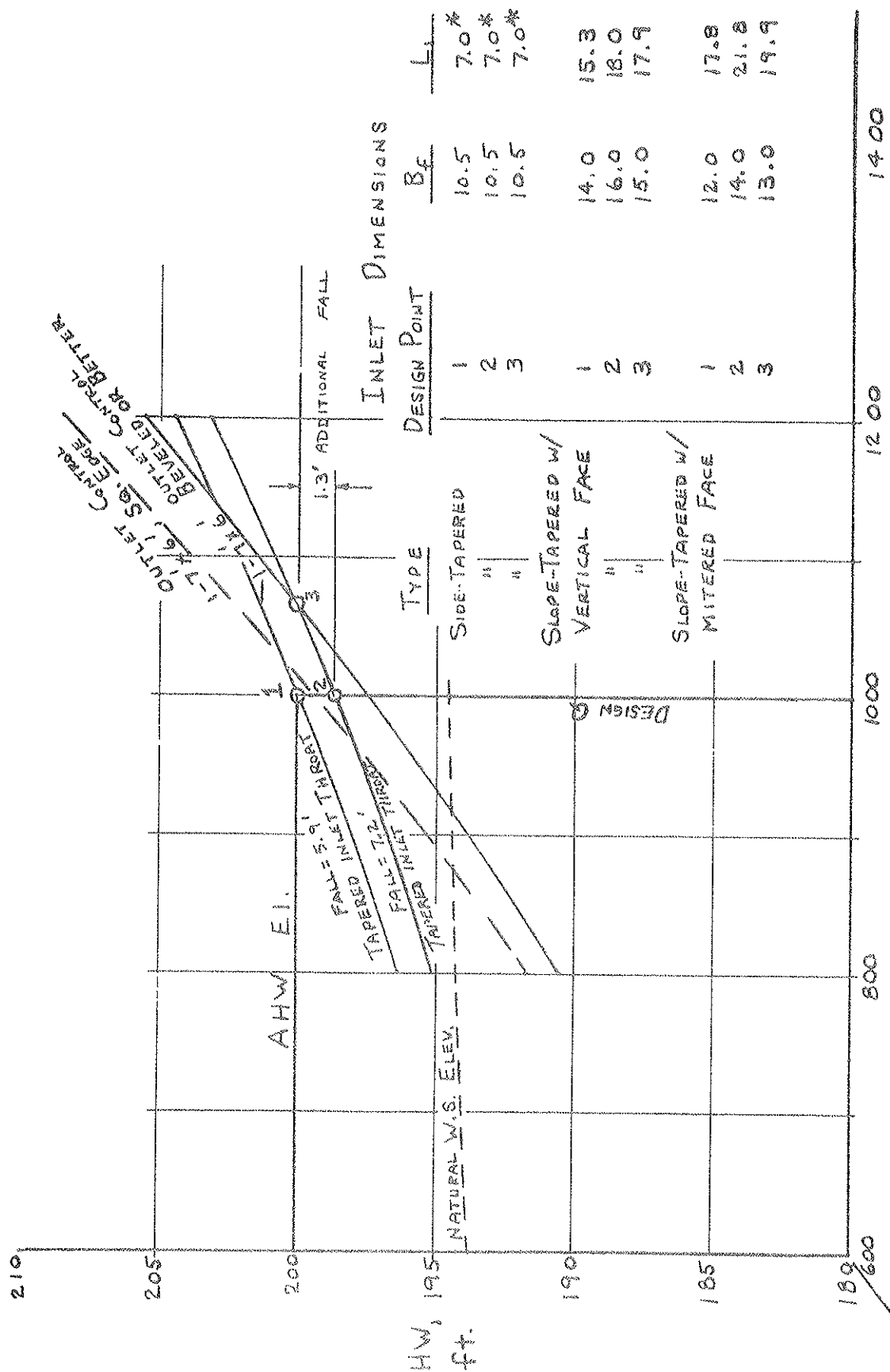
	Q	HW _f	El. Throat Invert	(1) El. Face Invert	(2) H _f	H _f /D	Q/B _f D ^{3/2}	D ^{3/2}	(3) Min. B _f	B _f	S	Comments
Trial 1	1000	198.7	182.8	188.6	10.1	1.68	5.15	14.7	13.5	14.0	0.029	$B_f D^{3/2} =$ _____
												Mitered face point no. 2
Trial 2	1062	200	182.8	188.6	11.4	1.94	5.65	14.7	12.8	13.0	0.029	$B_f D^{3/2} =$ _____
												Mitered face point no. 3

Note: Use only throat designs with FALL > 0.25D

- (1) El. face Invert: Vertical = Approx. stream bed elevation
Mitered = El. Crest - y, where y = 0.4D (Approx.), but higher than throat invert elevation.
(2) $H_f = HW_f - \text{El. face invert}$
(3) $\text{Min. } B_f = Q / (D^{3/2}) \cdot Q / (B_f D^{3/2})$

(4) Min. L ₃	(5) L ₄	(6) L ₂	(7) Check L ₂	(8) Adj. L ₃	(9) Adj. TAPER	(10) L ₁	(11) W	Q/W	H _c	(12) Max. Crest El.	GEOMETRY B _f = ____ ft. L ₃ = ____ ft. L ₁ = ____ ft. L ₄ = ____ ft. L ₂ = ____ ft. d = ____ in. b = ____ in. TAPER = ____:1
3.5	7.8	8.6	10.5	5.4	—	21.8	17.9	55.9	7.4	191.3	
3.5	7.8	8.6	8.5	—	4.03:1	19.9	16.8	63.3	7.9	192.1	

- (4.) $\text{Min. } L_3 = 0.5NB$
(5) $L_4 = S_f \cdot D / S_f$
(6.) $L_2 = (\text{El. Face (Crest) Invert} - \text{El. Throat Invert}) S_f - L_4$
(7.) Check $L_2 = \frac{B_f - NB}{2}$ TAPER - L_3
(8.) If (7) > (6), Adj. $L_3 = \frac{B_f - NB}{2}$ TAPER - L_2
(9.) If (6) > (7) Adj. TAPER = $(L_2 - L_3) / \frac{B_f - NB}{2}$
(10.) $L_1 = L_2 + L_3 + L_4$
(11.) Mitered: $W = NB \cdot 2 \cdot \frac{L_1}{\text{TAPER}}$
(12.) Max. Crest El. = $HW_f - H_c$



Q, cfs
DESIGN PERFORMANCE CURVES-1-7'x6' RCB

Example 1

Conclusion

Since the requirements called for the smallest possible reinforced concrete box culvert, the barrel should be a single 7 ft x 6 ft.

Selection of the inlet would be based on cost. The additional 1.3 ft of FALL gains 62 cfs at AHW EL. = 200.0, but this is not significant at this site. It appears that a side- or slope-tapered design meeting the Q and HW requirements of point 1 would be adequate and the least expensive.

Examination of the outlet control curve shows that a discharge of 1,200 cfs (20% above design) results in an AHW EL. 5.5 ft above design. At this site, no serious flooding of upstream property or the roadway will be caused by such a headwater, and no larger barrel is required.

The dimensions of several alternate inlet structure designs are presented, based on points 1, 2, and 3 on the culvert performance curves. Note that the side-tapered inlets remain about the same size for all FALL values, while the slope-tapered inlets increase in size as FALL increases. However, the side-tapered inlets require an increasingly larger upstream sump as FALL increases. Which design will be more favorable will be a matter of economics and site considerations.

Example 2

Pipe Culvert

Given: Design Discharge, Q_{10} = 150 cfs
 Allowable Headwater Elevation = 100.0 ft msl
 Elevation Outlet Invert = 75.0 ft msl
 Culvert Length, L_a = 350 ft
 Downstream channel approximates a 5-ft-wide trapezoidal
 channel with 2:1 side slopes, a Manning's "n" of 0.03, and
 S_0 = 0.05.

Requirements: This pipe culvert is located in a suburban area where the AHW
EL. may be exceeded by 2 to 3 ft without extreme damage.
However, headwater elevations greater than 103.0 ft should be
avoided for flows significantly higher than the design Q of
150 cfs.

PROJECT: <u>Example No. 2</u>		OUTLET CONTROL DESIGN CALCULATIONS		DESIGNER: <u>JMN</u>	
STATION: _____		DATE: <u>12-10-73</u>			

INITIAL DATA:

Q 0 = 150 cfs

AHW El. = 100 ft.

So = 0.05

La = 350 ft.

El. Outlet Invert 75 ft.

Stream Data:

Barrel Shape and Material Circular CMP Barrel n = 0.024

SKETCH

First Approximation
 Q = 150 cfs, $k_e = 0.25$, $L_d = 350$ ft.
 $H = \text{AHW El.} - \text{El. Outlet Invert} - h_o$
 $= 100' - 75' - 5' = 20'$
 $\therefore A = \text{_____ ft}^2$ or $D = 46''$ ft; Try 42''

Q	$\frac{Q}{N}$	* H	$\frac{Q}{NB}$	(1) d_c	$\frac{d_c \cdot D}{2}$	Qn	(2) TW	(3) h_o	(4) HW ₀	(5) V_o	COMMENTS
Trial No. <u>1</u> , N = <u>1</u> , B = <u>—</u> , D = <u>3.5'</u> , $k_e = 0.25$											
150	150	31	150	> 3.5	3.5	—	1.6	3.5	109.5		75 + 31 × 3.5 = 109.5
											HW ₀ > AHW El. Try 48"
Trial No. <u>2</u> , N = <u>1</u> , B = <u>—</u> , D = <u>4'</u> , $k_e = 0.25$											
150	150	15.6	150	3.6	3.8	—	1.6	3.8	94.4		OK - Check square edge
100	100	7.0	100	3.1	3.5	—	1.4	3.5	85.5		
200	200	27.8	200	> 4	4.0	—	1.9	4.0	106.8		
Trial No. <u>3</u> , N = <u>1</u> , B = <u>—</u> , D = <u>4'</u> , $k_e = 0.5$											
150	150	16.2					1.6	3.8	95.0		From inlet control section
100	100	7.2					1.4	3.5	85.7		Calculations, FALL req'd
200	200	28.8					1.9	4.0	107.8		$\therefore$ Use improved inlet

Notes and Equations:

(1) d_c cannot exceed D

(2) TW based on d_n in natural channel, or other downstream control.

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

(4) $HW_0 = H + h_o + \text{El. Outlet Invert.}$

(5) Outlet Velocity (V_o) = Q/Area defined by d_o or TW, not greater than D. Do not compute until control section is known.

SELECTED DESIGN

N = 1 At Design Q:

B = — ft.

D = 4 ft. HW₀ = 94.4 ft.

$k_e = 0.25$ or 0.5 $V_o = \text{_____}$ f/s

$$H = \left[1 + k_e + \frac{29n^2 \cdot L}{R^{1.33}} \right] \frac{V^2}{2g}$$

PROJECT: Example No. 2

STATION: _____

CULVERT INLET CONTROL SECTION

DESIGN CALCULATIONS

DESIGNER: JAN

DATE: 12-10-75

INITIAL DATA:

Q 0 = 150 cfs

AHW El. = 100 ft.

S₀ = 0.05

L₀ = 350 ft.

El. Stream Bed at Face 92.5 ft.

Barrel Shape and Material Circ. CNP Barrel n = 0.024

N = 1, B = _____

D = 4', NBD^{3/2} = _____

(Pipe) ND^{5/2} = 32 (table 5)

CONVENTIONAL or BEVELED
INLET: FACE CONTROL SECTION
(Upper Headings)

TAPERED INLET
THROAT CONTROL SECTION
(Lower Headings)

DEFINITIONS OF INLET CONTROL SECTION

Q	Q/NB	H _f /D	H _f	(1)	El. Stream Bed At Face	(2)	(3)	(4)	(5)	Note: Use Upper Headings for Conventional or Beveled Face; Lower Headings for Tapered Inlet Throat.
				El. Face Invert						
Q	Q/NBD ^{3/2}	H _f /D	H _f	El. Throat Invert		FALL	HW _f	S	V ₀	COMMENTS
Trial No. <u>1</u> Inlet and Edge Description <u>Square edges</u>										
150	150	2.07	8.3	91.7	92.5	0.8	100.0	0.048		FALL required, use bevels
Trial No. <u>2</u> Inlet and Edge Description <u>Beveled edges</u>										
150	150	1.92	7.7	92.3	92.5	~0	100	0.05	16	Check tapered inlet throat
100	100	1.25	5.0				97.3			
200	200	2.90	11.6				103.9			
Trial No. <u>3</u> Inlet and Edge Description <u>Tapered inlet throat, rough</u>										
150	4.7	1.65	6.6	92.5	92.5	0	99.1	0.05	16	Increases Q at AHW
100	3.1	1.21	4.8				97.3			El. from 150 to 170 c.f.s.
200	6.2	2.22	8.9				101.4			

Notes and Equations:

(1) El. Face (or throat) invert = AHW El. - H_f (or H_t)

(2) FALL = El. Stream Bed at Face - El. face (or throat) invert

(3) HW_f (or HW_t) = H_f (or H_t) + El. face (or throat) invert, where El. face (or throat) invert should not exceed El. stream bed.

(4) S ≈ S₀ - FALL/L₀

(5) Outlet Velocity = Q/Area defined by d_n at S

SELECTED DESIGN

Inlet Description: Beveled edges

FALL = 0 ft.

Invert El. = 92.5 ft.

Bevels:

Angle = 45°

b = _____ in., d = 2 in.

PROJECT: Example No. 4 SIDE-TAPERED INLET DESIGN CALCULATIONS DESIGNER: JMA

STATION: _____ DATE: 12-10-73

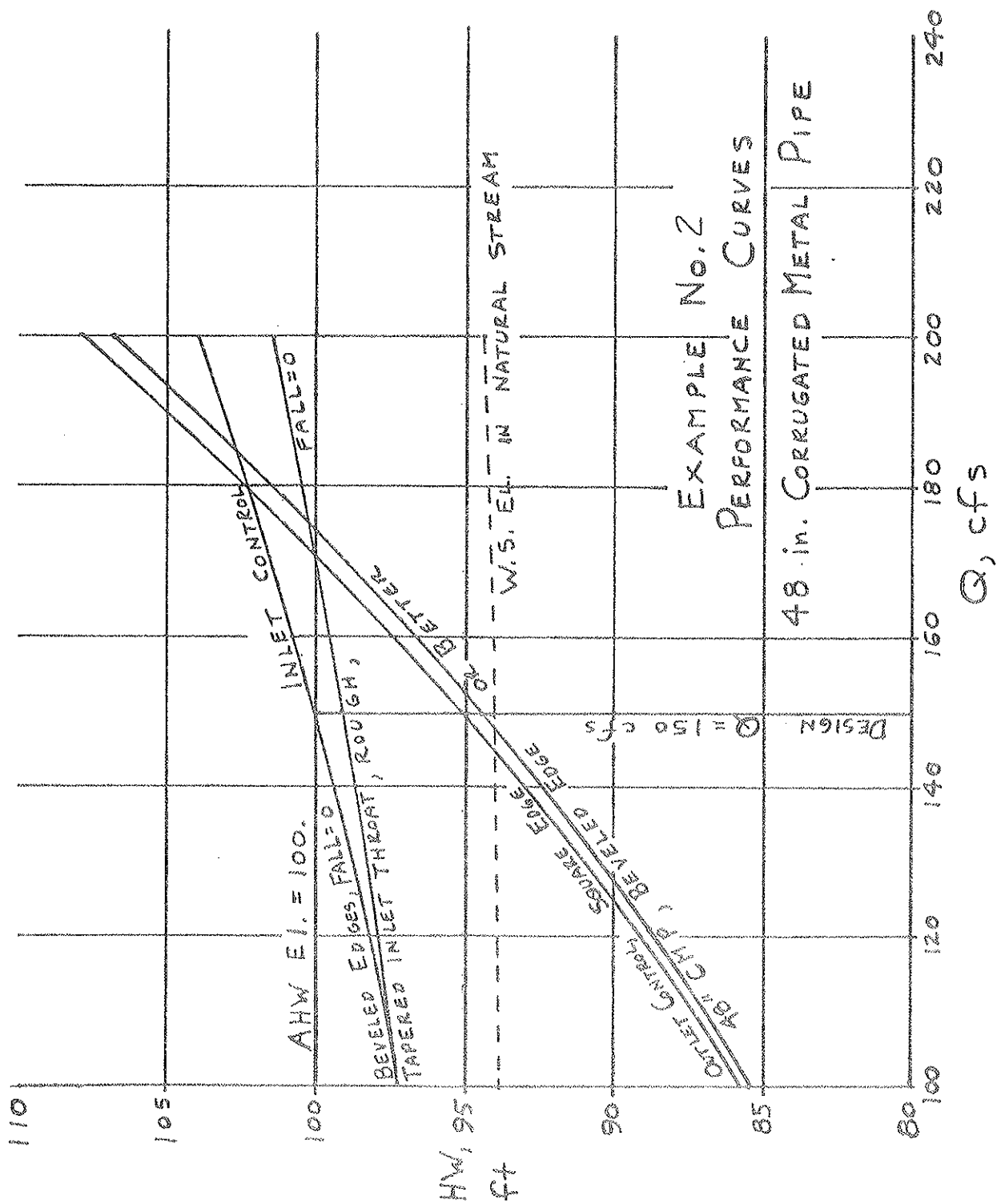
INITIAL DATA
 $Q = 0 = 150$ cfs $S_o = 0.05$
 AHW El. = 100 ft. $L_a = 350$ ft.
 TAPER = 4 : 1
 Barrel Shape and Material Circular CMP
 Face Edge Description 45° Bevels
 N = 1, B = _____ ft, D = 4' ft

SKETCH

Q	El. Throat Invert	(1)	(2)	(3)	(4)	Bf	L1	S	L1S	(5) El. Face Invert	Upper Headings for Box Culverts, Lower Headings for Pipes COMMENTS
		$\frac{H_f}{E}$	$\frac{Q}{B_f D^{3/2}}$	$\frac{Q}{A_f E^{1/2}}$	Min. B_f						
Trial No. <u>1</u> , Q = <u>150</u> , HWf = <u>99.1</u> (Use lower column headings)											
150	92.5	1.4	4.0	2.0	18.8	6.0	4.0	0.05	0.2	92.7	$B_f D^{3/2} [or A_f E^{1/2}] = 18.85$ Std. design: $B_f = 1.5D$ = 6' ∴ std. design O.K.
Trial No. _____, Q = _____, HWf = _____											
											$B_f D^{3/2} [or A_f E^{1/2}] =$ _____
Trial No. _____, Q = _____, HWf = _____											
											$B_f D^{3/2} [or A_f E^{1/2}] =$ _____

Notes and Equations: $(99.1 - 92.5 - 1) / 4 = 1.4$
 (1) $H_f / D [or H_f / E] \approx (HW_f - El. Throat Invert - 1) / D [or E]$
 $0 \leq E \leq 1.10$
 (2) Min. $B_f = Q / [D^{3/2} \frac{Q}{B_f D^{3/2}}]$
 $Min. A_f = Q / [E^{1/2} \frac{Q}{A_f E^{1/2}}]$
 (3) $L_1 = \left[\frac{B_f - NB}{2} \right] TAPER = \left[\frac{6.0 - 4.0}{2} \right] 4 = 4.0'$
 (4) From throat design
 (5) El. Face Invert - El. Throat Invert > 1 ft., recompute.
 Face and Throat may be lowered to better fit site, but do not raise.

SELECTED DESIGN
 $B_f = 6.0$ ft.
 $L_1 = 4.0$ ft.
 Bevels: Angle 45°
 $d =$ _____ in., $b = 3$ in.
 Crest Check:
 $HW_c = 99.1$ ft. 99.1
 $H_c = 6.1$ ft. 93.0
 $Q/W = 44.0$ (Chart 17)
 Min. W = 3.4 ft.



Example 2

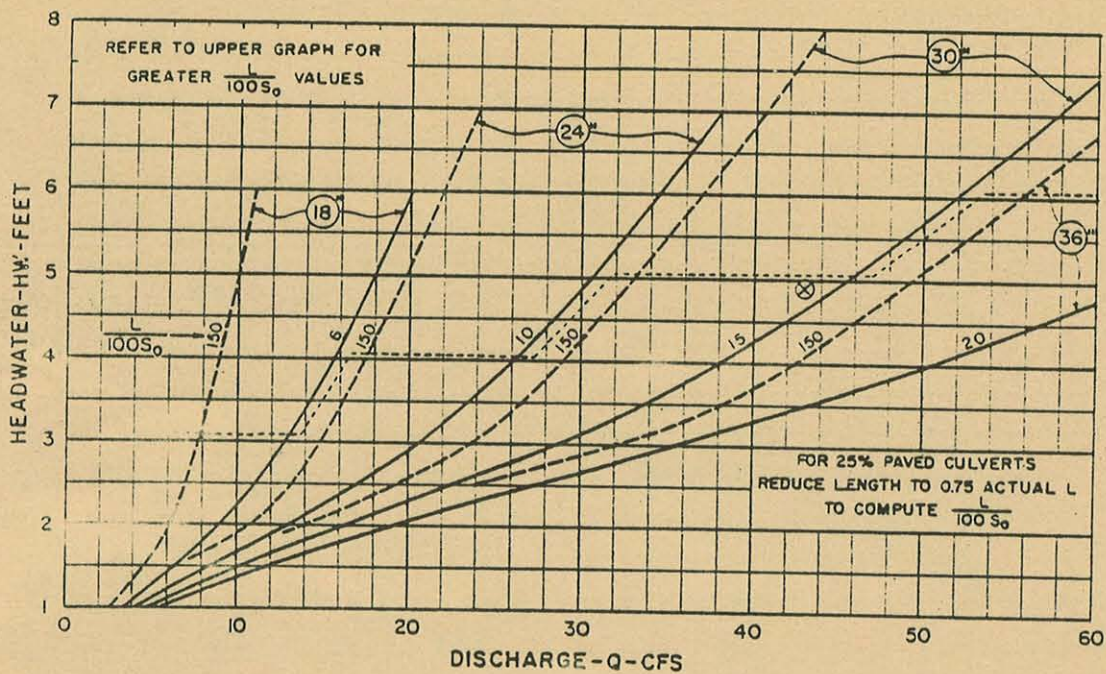
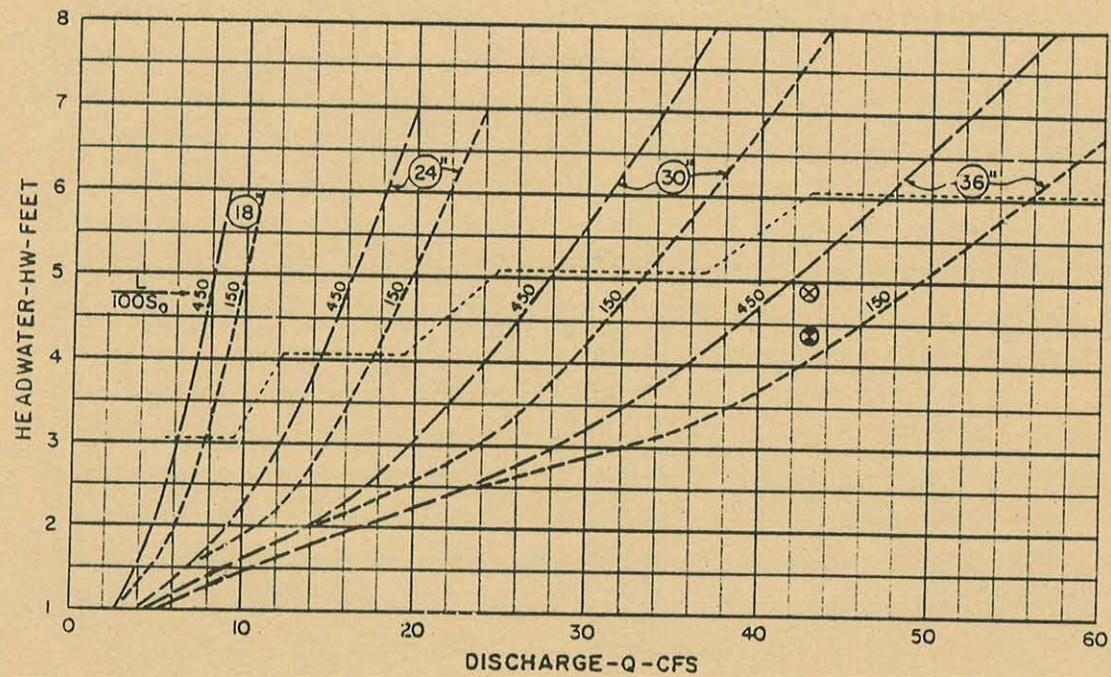
Conclusion

From the performance curves, beveled edges meet the AHW EL. of 100 ft and $Q = 150$ cfs, while the use of a side-tapered inlet would increase Q to 170 cfs at AHW EL. = 100 ft. In both cases, the FALL = 0. It appears that the beveled-edge inlet would be sufficient and the least costly in this case, since the culvert performance curve does not exceed 103.0 ft until Q is 186 cfs.

For additional examples consult Appendix A of Reference 4.

Design Figures

Capacity charts and nomographs covering the range of applications commonly encountered in urban drainage are contained in this section. These charts are from Federal Highway Administration publications. For situations beyond the range covered by these charts reference should be made to the original publications.

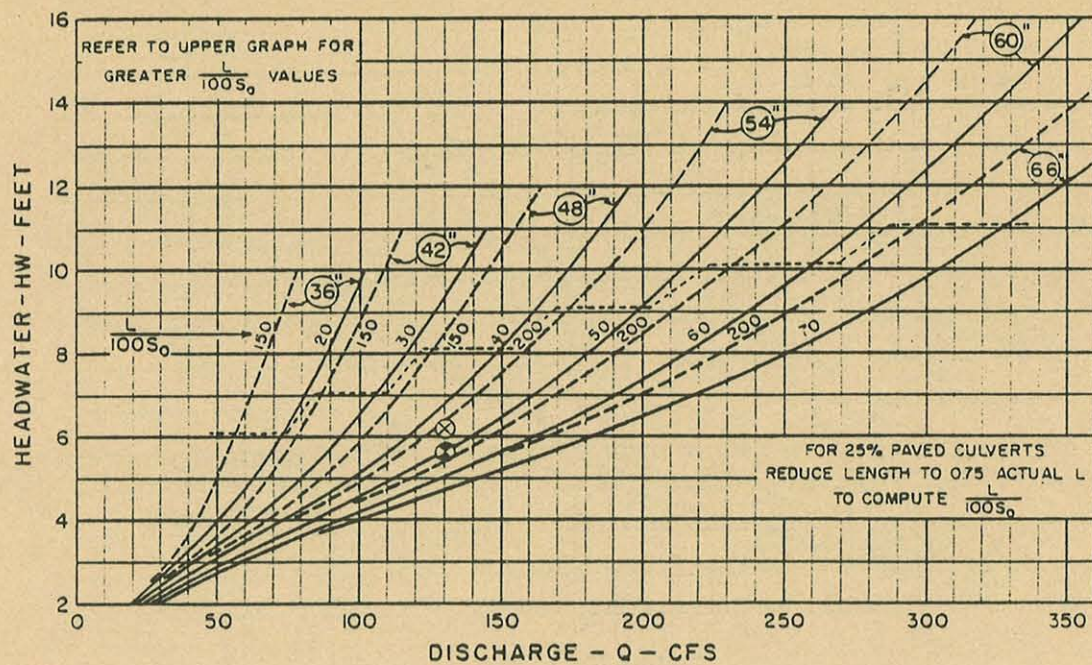
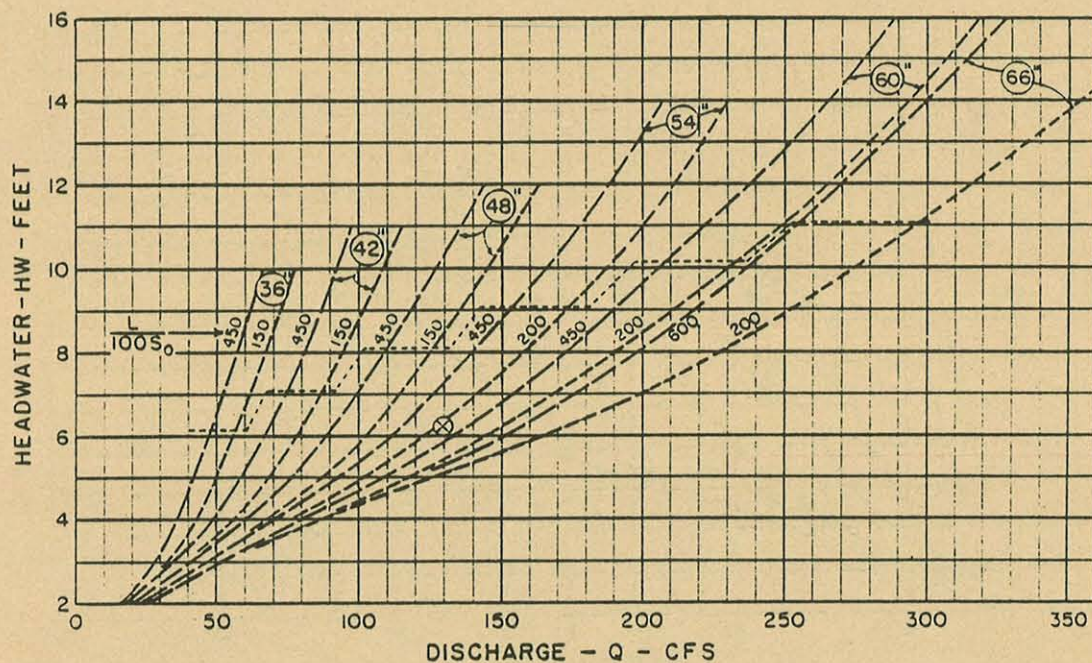


EXAMPLE

- ⊗ GIVEN:
43 CFS; AHW = 4.9 FT.
L = 72 FT.; $S_0 = 0.003$
- ⊗ SELECT 36" UNPAVED
HW = 4.4 FT.

STANDARD
CIRCULAR CORR. METAL PIPE
HEADWALL ENTRANCE
18" TO 36" ○

Figure 6-13 Culvert Capacity

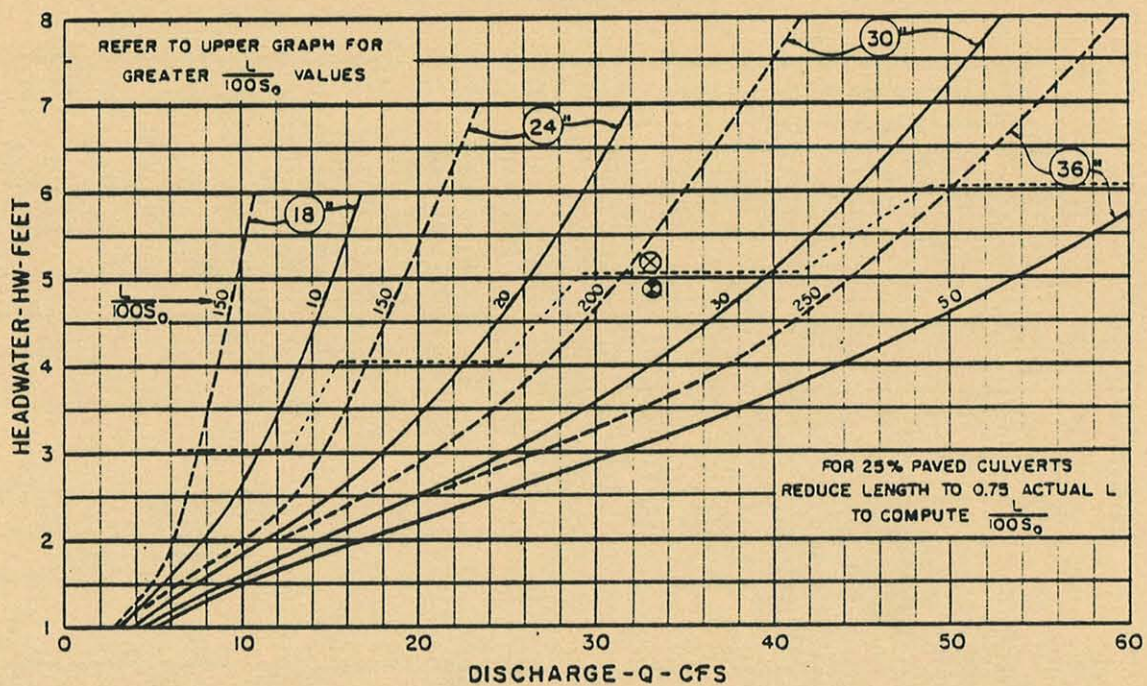
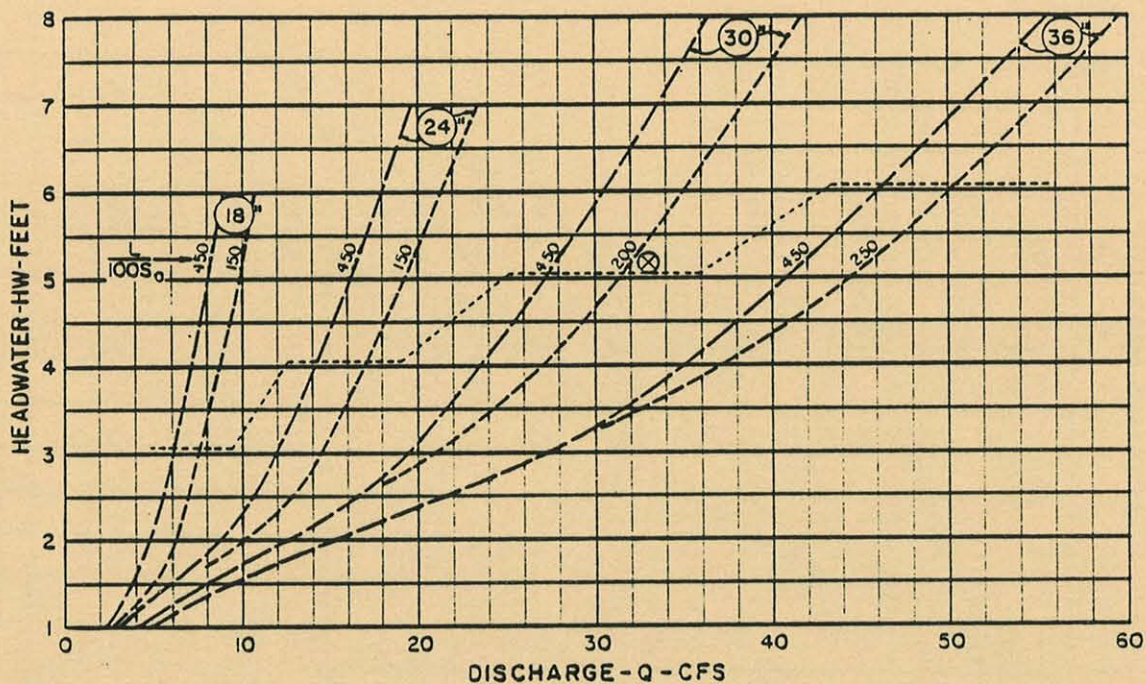


EXAMPLE

- ⊗ GIVEN:
130 CFS; AHW = 6.2 FT.
L = 120 FT.; $S_0 = 0.025$
- ⊗ SELECT 54" UNPAVED
HW = 5.6 FT.

STANDARD
CIRCULAR CORR. METAL PIPE
HEADWALL ENTRANCE
36" TO 66" ○

Figure 6-14 Culvert Capacity

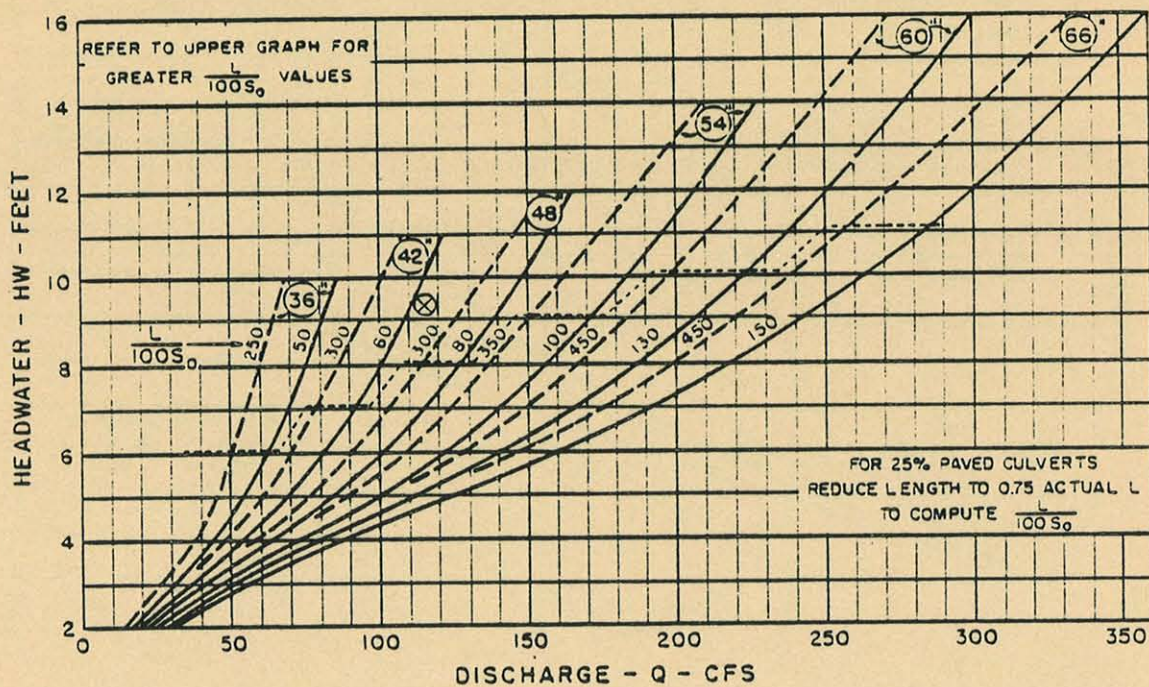
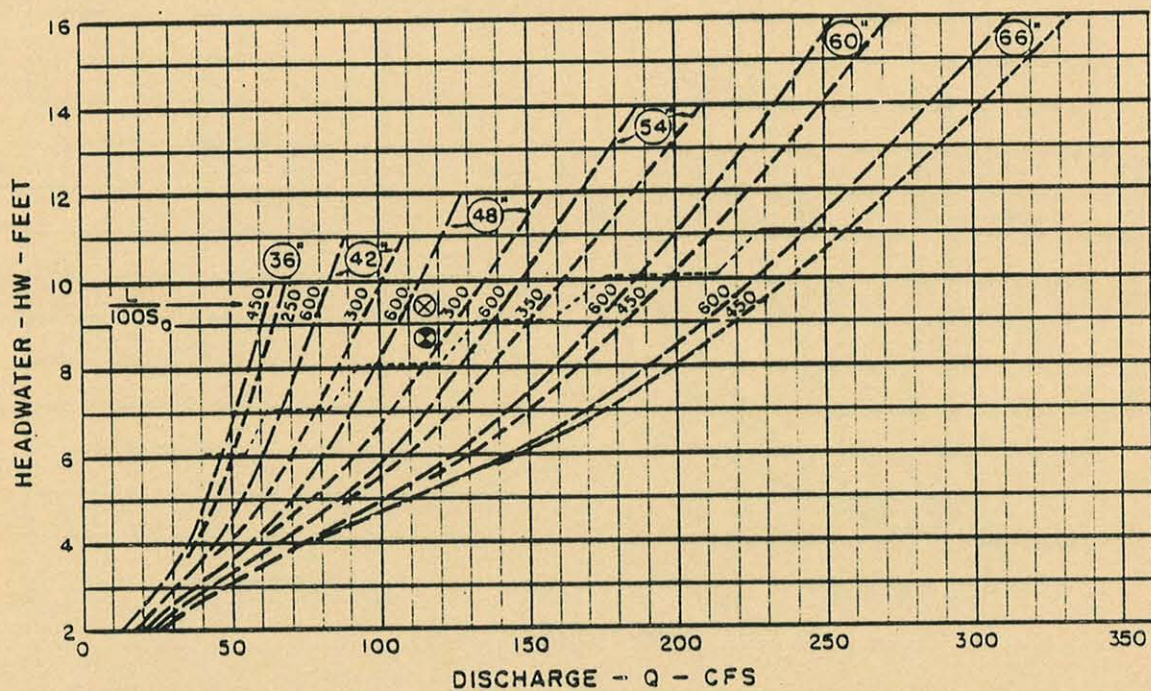


EXAMPLE

- ⊗ GIVEN:
33 CFS; AHW = 5.2 FT.
L = 70 FT.; $S_0 = 0.005$
- ⊗ SELECT 30" UNPAVED
HW = 4.9 FT.

STANDARD
CIRCULAR CORR. METAL PIPE
PROJECTING ENTRANCE
18" TO 36" ○

Figure 6-15 Culvert Capacity

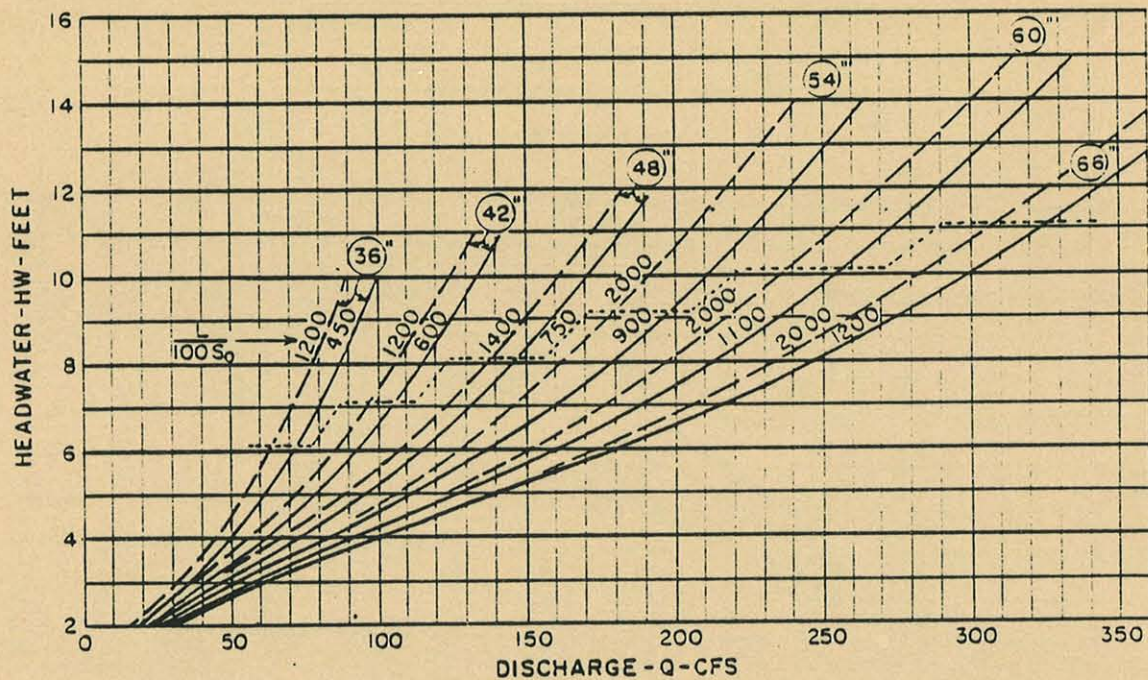
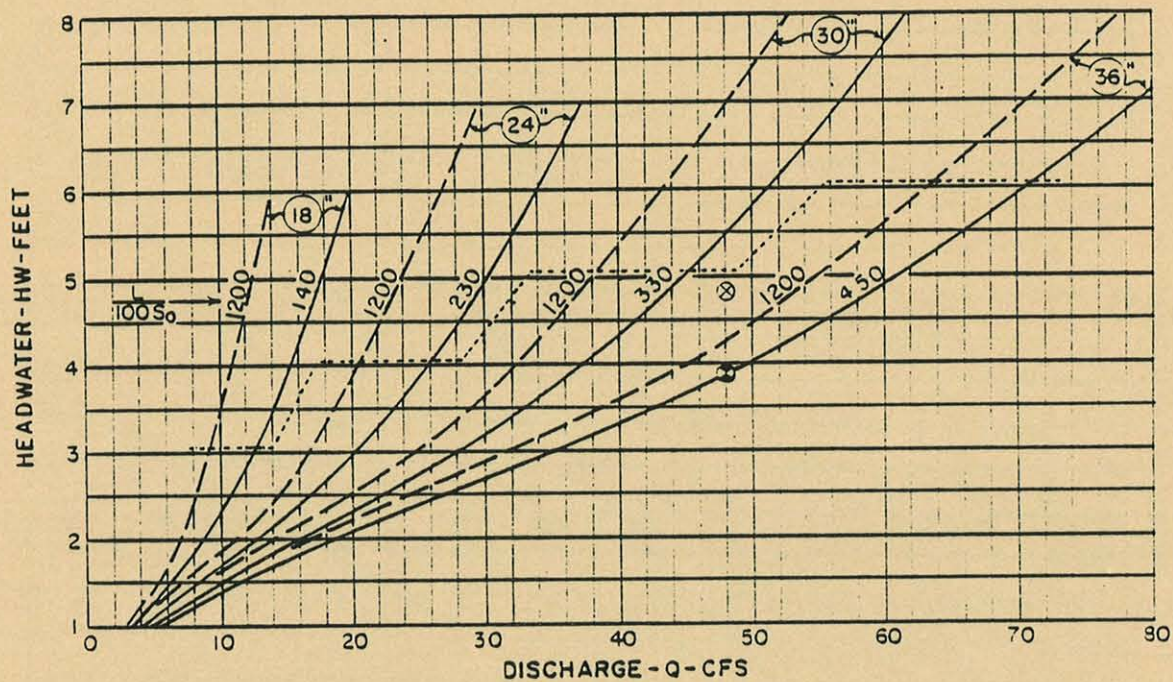


EXAMPLE

- ⊗ GIVEN:
115 CFS; AHW = 9.4 FT.
L = 135 FT.; $S_0 = 0.0034$
- ⊗ SELECT 48" UNPAVED
HW = 8.6 FT.

STANDARD
CIRCULAR CORR. METAL PIPE
PROJECTING ENTRANCE
36" TO 66" ○

Figure 6-16 Culvert Capacity



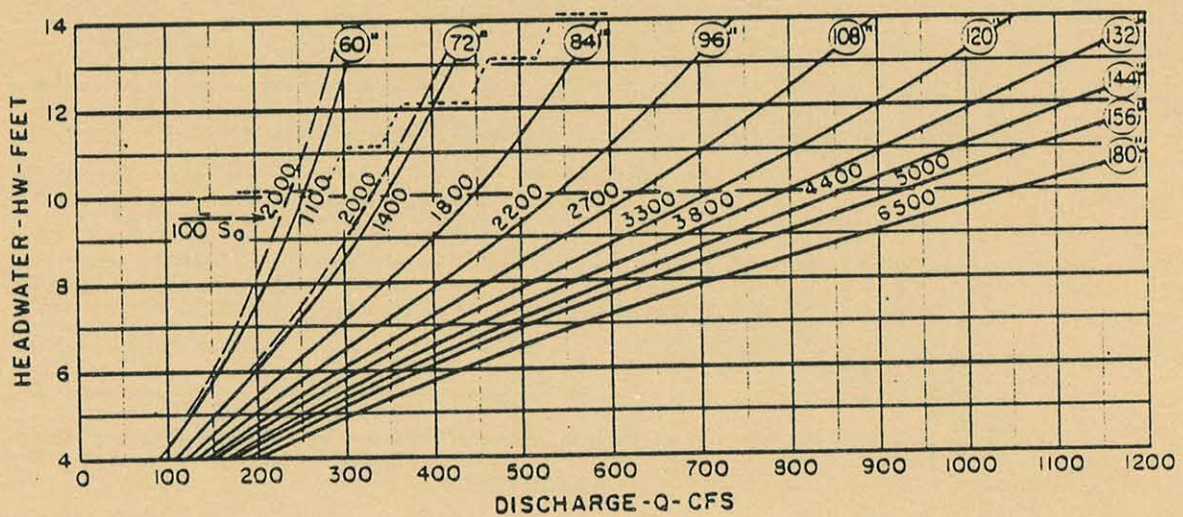
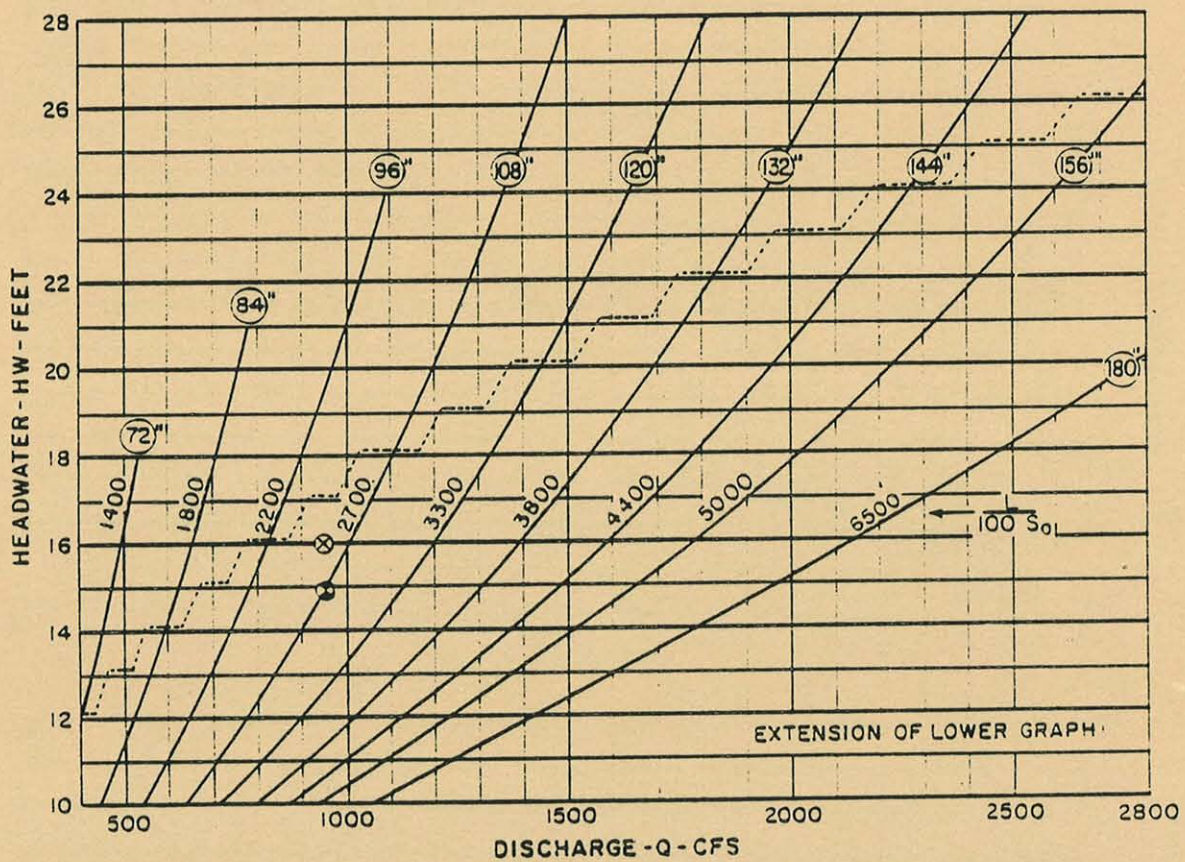
EXAMPLE

⊗ GIVEN:
48 CFS; AHW = 4.8 FT
L = 60 FT; $S_0 = 0.003$

⊗ SELECT 36"
HW = 3.9 FT

CIRCULAR CONCRETE PIPE
SQUARE-EDGED ENTRANCE
18" TO 66" ○

Figure 6-17 Culvert Capacity

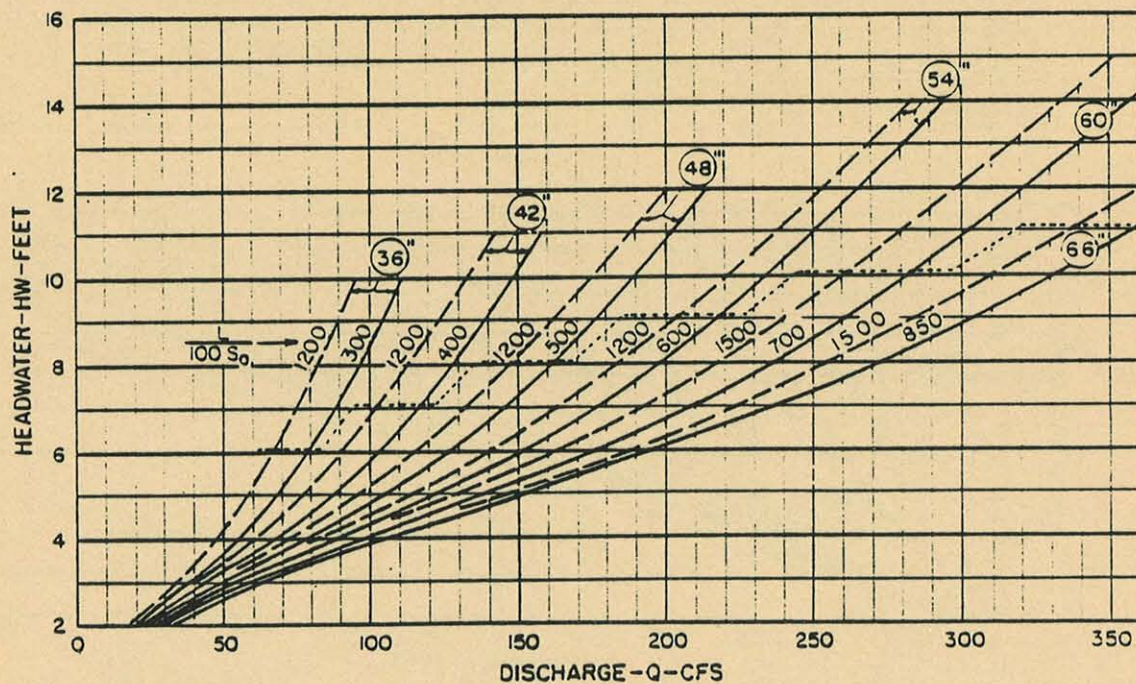
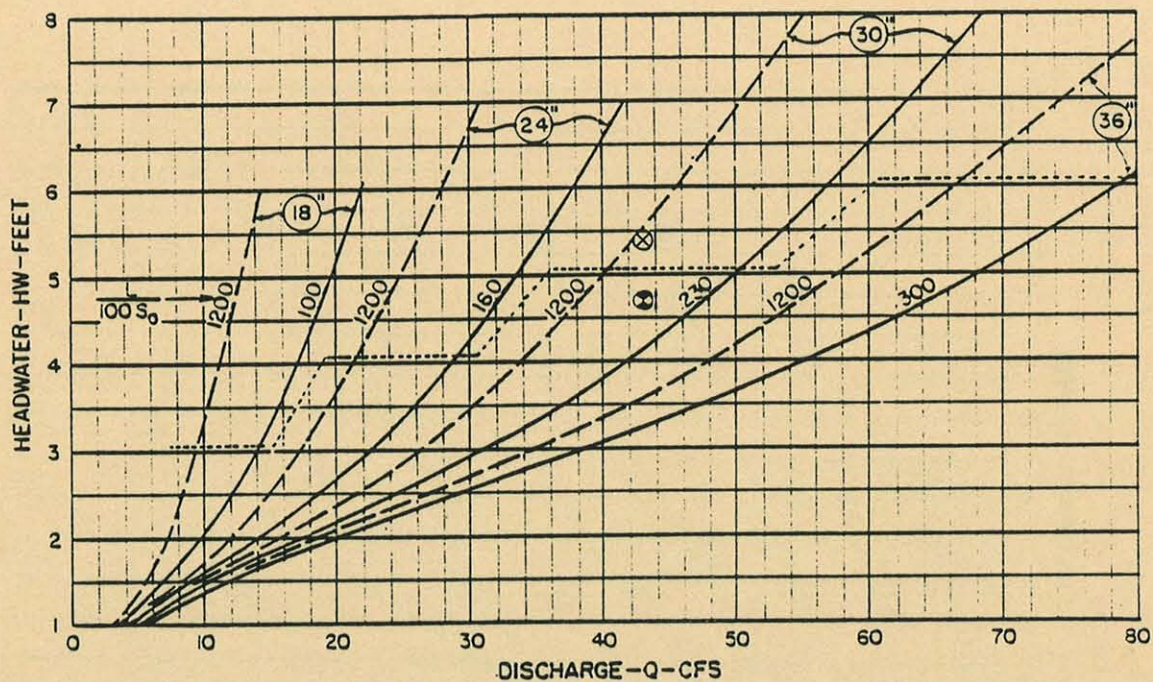


EXAMPLE

- ⊗ GIVEN:
950 CFS; AHW = 16 FT
L = 480 FT; $S_0 = 0.040$
- ⊕ SELECT 108"
HW = 15.0 FT

CIRCULAR CONCRETE PIPE
SQUARE-EDGED ENTRANCE
60" TO 180" ○

Figure 6-18 Culvert Capacity



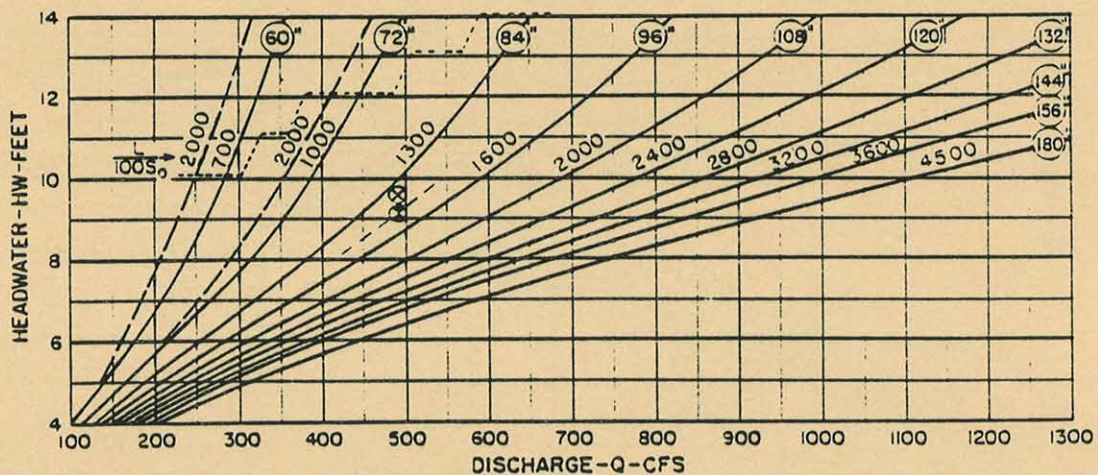
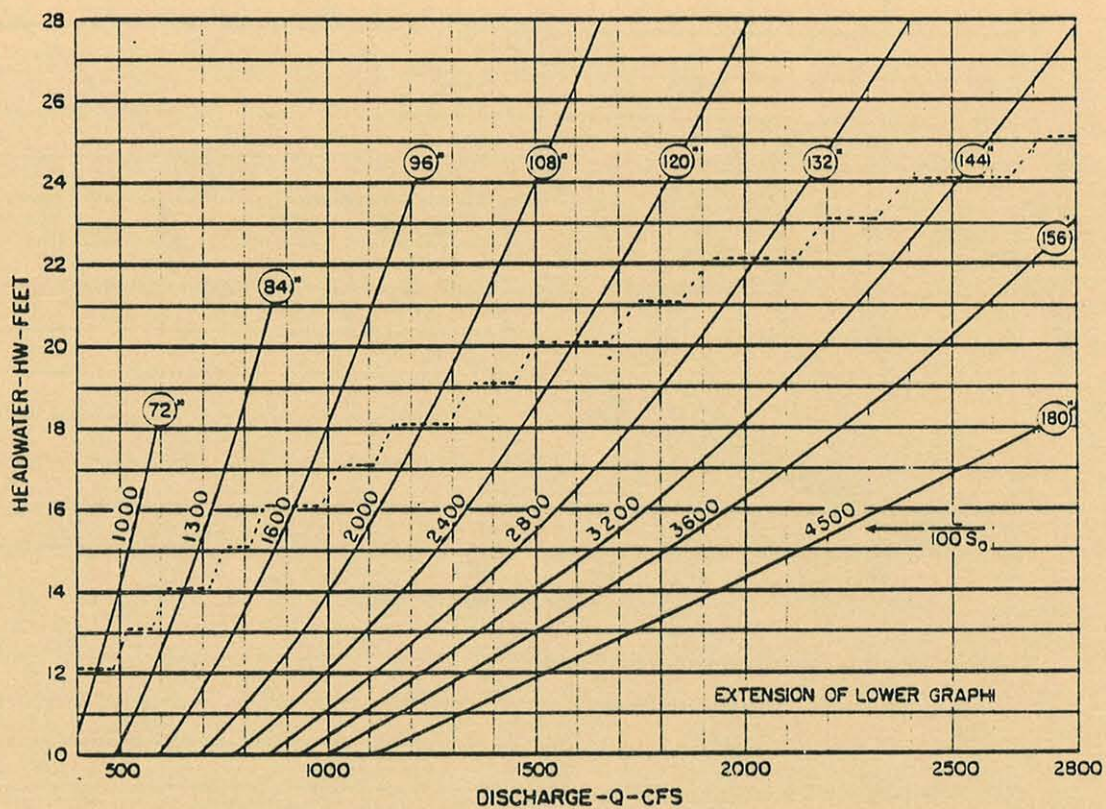
EXAMPLE

⊗ GIVEN:
43 CFS ; AHW = 5.4 FT.
L = 120 FT. ; $S_0 = 0.002$

⊗ SELECT 30"
HW = 4.7 FT.

**CIRCULAR CONCRETE PIPE
GROOVE-EDGED ENTRANCE
18" TO 66" ○**

Figure 6-19 Culvert Capacity



EXAMPLE

- ⊗ GIVEN:
490 CFS ; ΔHW = 9.6 FT.
L = 60 FT. ; S₀ = 0.000
- ⊙ SELECT 90" ($\frac{L}{D} = 8$)
HW = 9.2 FT.

CIRCULAR CONCRETE PIPE
GROOVE-EDGED ENTRANCE
60" TO 180" ⊙

Figure 6-20 Culvert Capacity

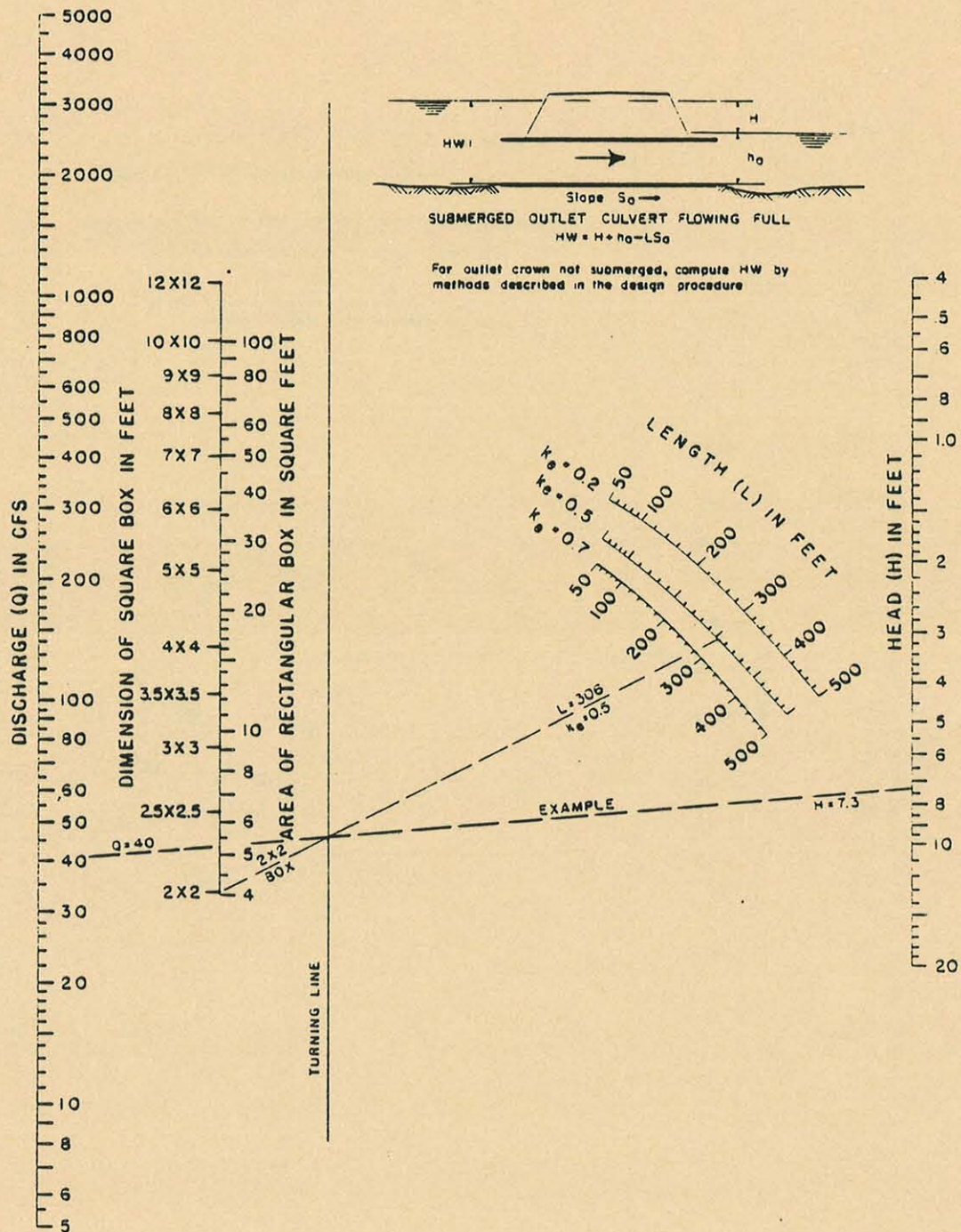
TABLE 6-2

Entrance Loss Coefficients

Outlet Control, Full or Partly Full

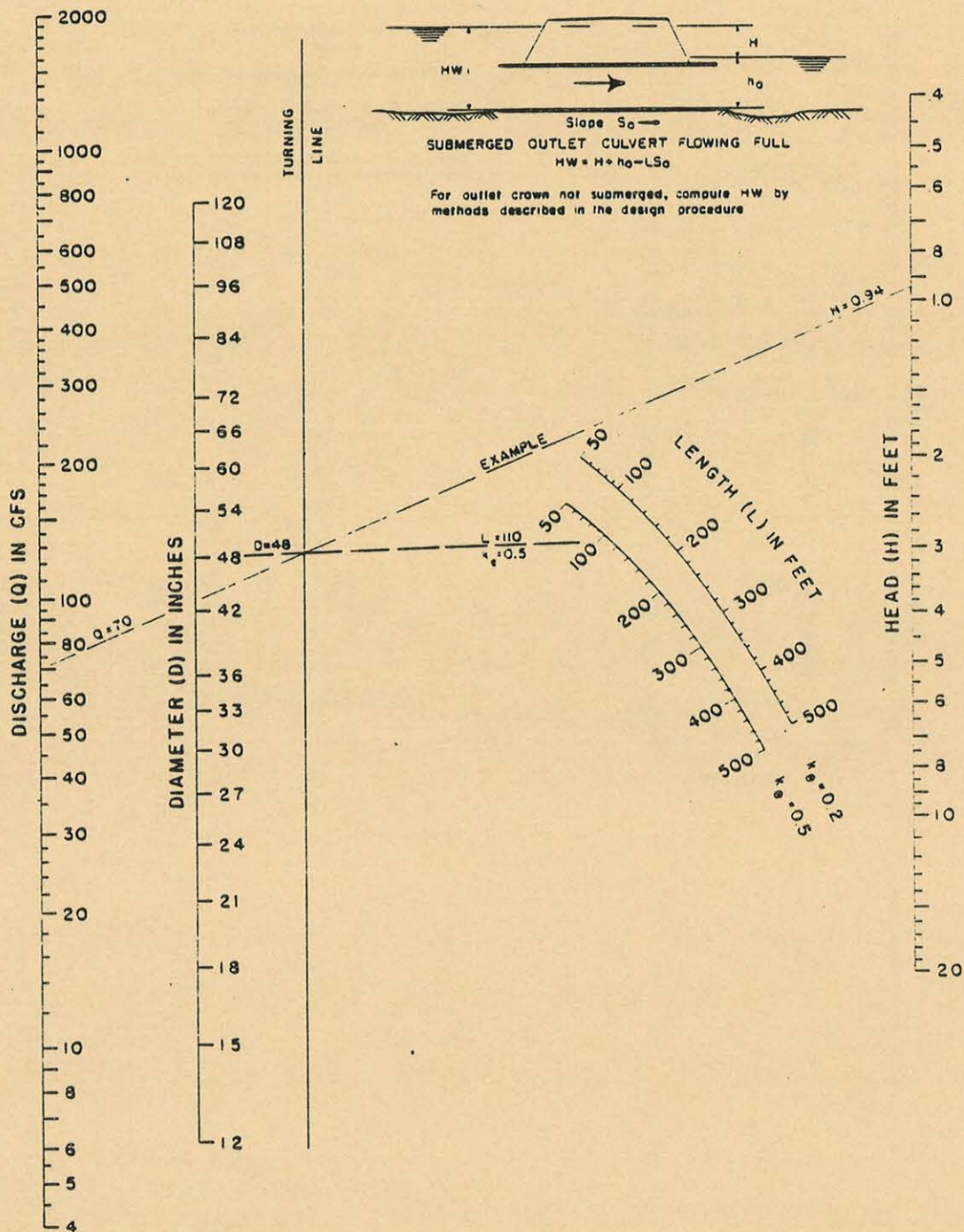
<u>Type of Structure and Design of Entrance</u>	<u>Coefficient k_e</u>
Pipe, Concrete	
Projecting from fill, socket end (groove end)	0.2
Projecting from fill, square-cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove end)	0.2
Square-edged	0.5
Rounded (radius = $1/12D$)	0.2
Mitered to conform to fill slope	0.7
*End section conforming to fill slope	0.5
Beveled edges, 33.7-degree to 45-degree bevels	0.2
Side- or slope-tapered inlet	0.2
Pipe, or Pipe-Arch, Corrugated Metal	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls square-edge	0.5
Mitered to conform to fill slope, paved or unpaved slope	0.7
*End section conforming to fill slope	0.5
Beveled edges, 33.7-degree to 45-degree bevels	0.2
Side- or slope-tapered inlet	0.2
Box, Reinforced Concrete	
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of $1/12$ barrel dimension, or beveled edges on 3 sides	0.2
Wingwalls at 30 degrees to 75 degrees to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of $1/12$ barrel dimension, or beveled top edge	0.2
Wingwall at 10 degrees to 25 degrees to barrel	
Square-edged at crown	0.5
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7
Side- or slope-tapered inlet	0.2

*Note: End sections conforming to fill slope are the sections commonly available from manufacturers. From limited hydraulic tests they are equivalent in operation to a headwall in both inlet and outlet control. Some end sections incorporating a closed taper in their design have a superior hydraulic performance.



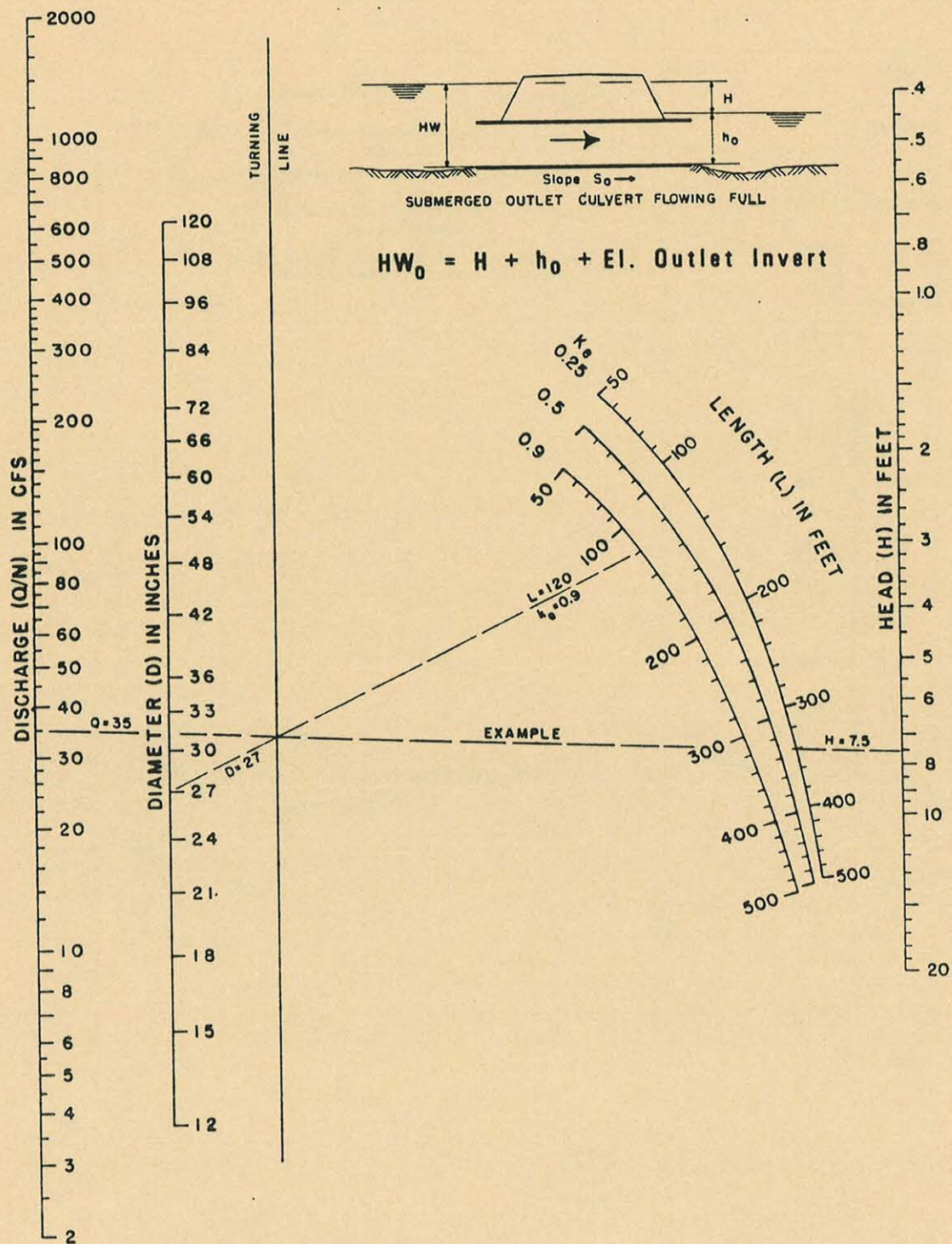
HEAD FOR
CONCRETE BOX CULVERTS
FLOWING FULL
 $n = 0.012$

Figure 6-21 Outlet-Control Nomograph



HEAD FOR
CONCRETE PIPE CULVERT
FLOWING FULL
 $n = 0.012$

Figure 6-22 Outlet-Control Nomograph



HEAD FOR
STANDARD
C. M. PIPE CULVERTS
FLOWING FULL
 $n = 0.024$

Figure 6-23 Outlet-Control Nomograph

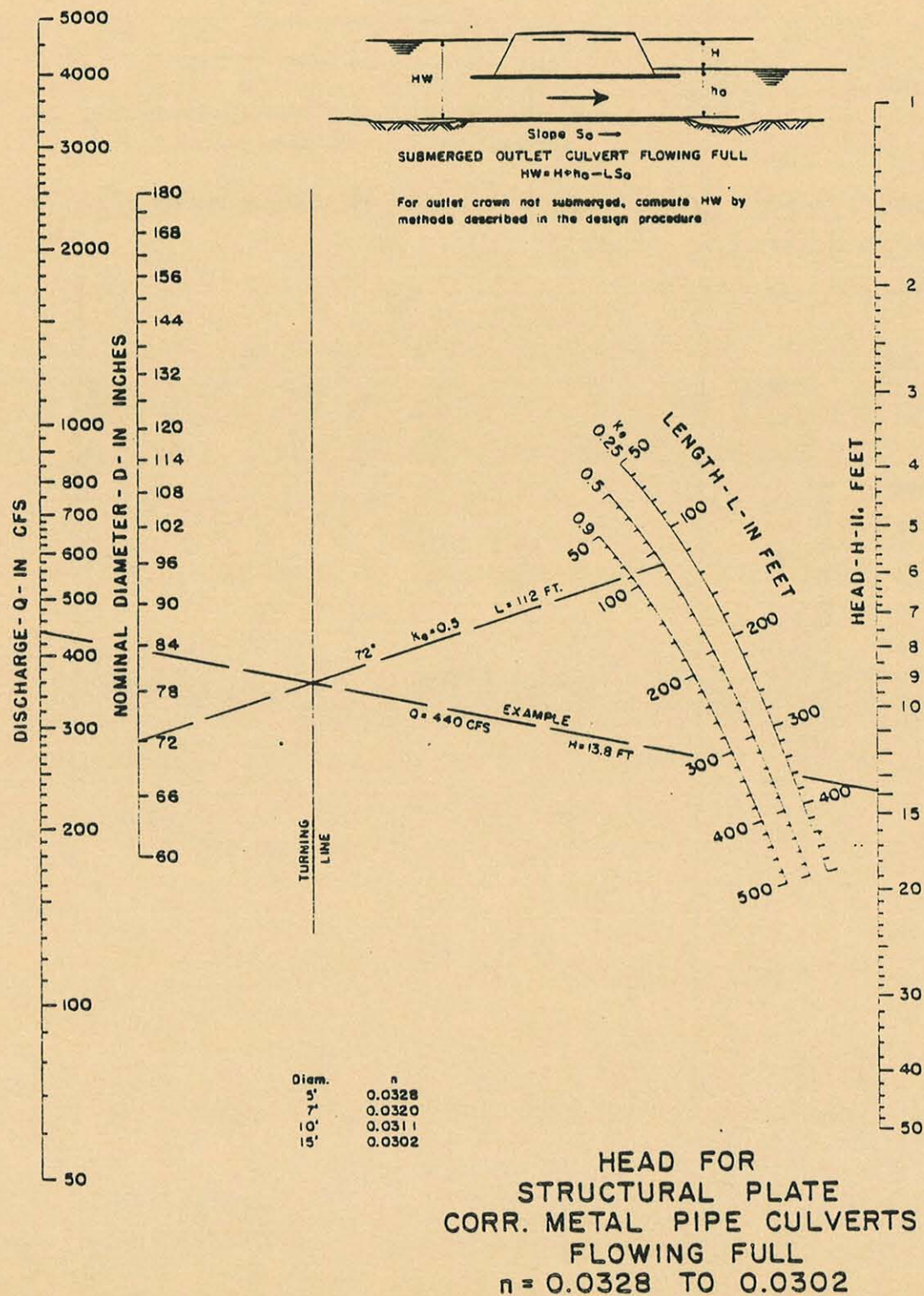
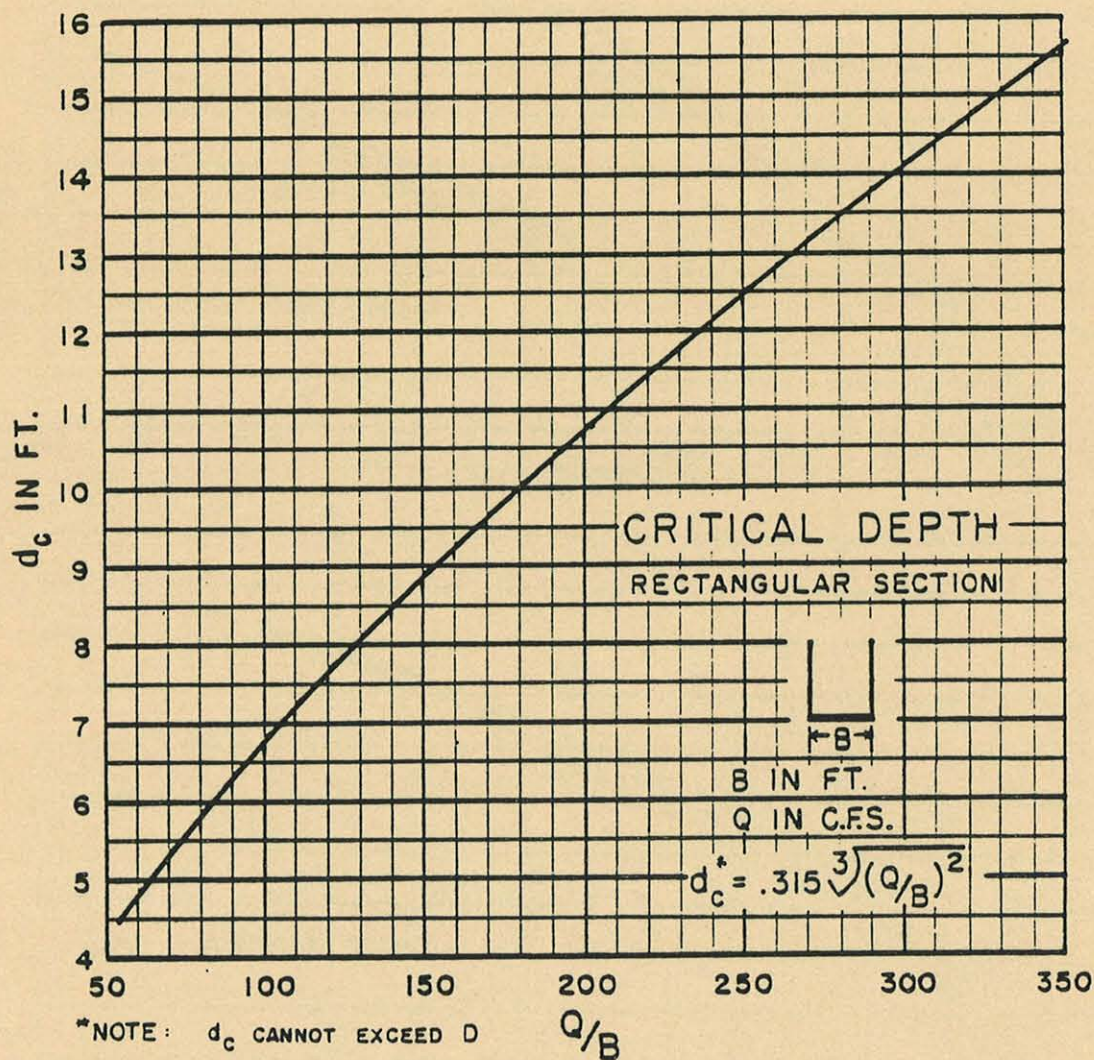
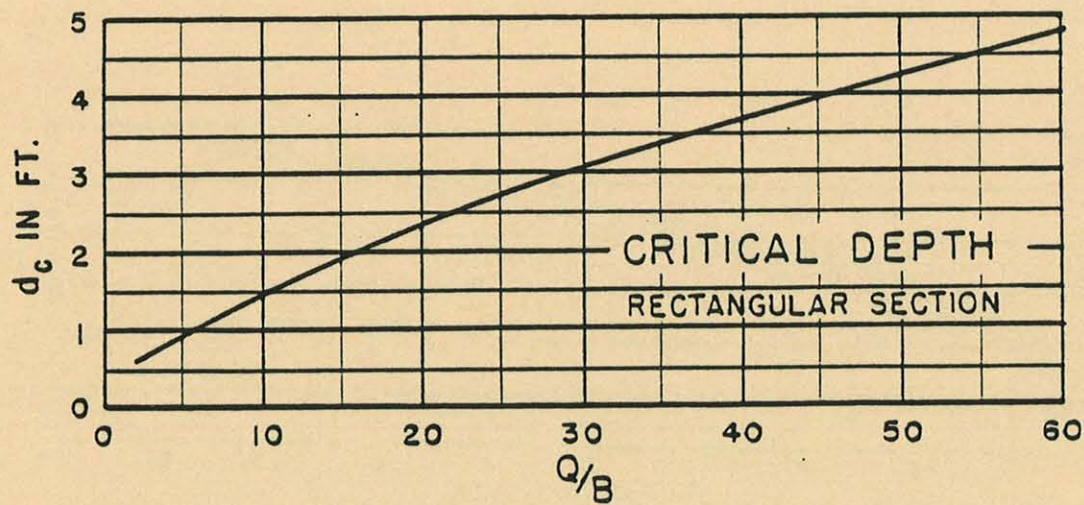


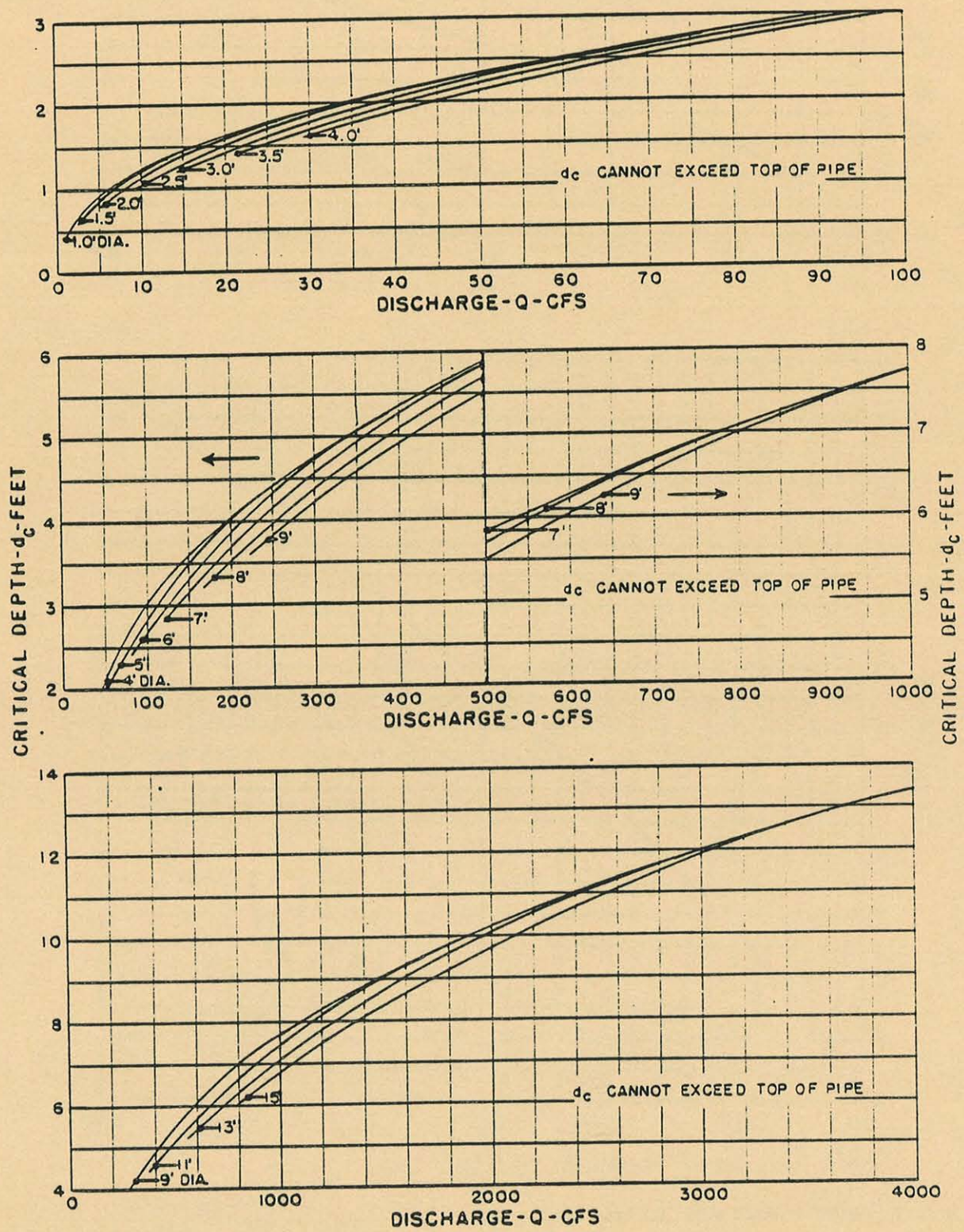
Figure 6-24 Outlet-Control Nomograph



BUREAU OF PUBLIC ROADS JAN. 1963

RECTANGULAR SECTION

Figure 6-25 Critical-Depth Chart



CIRCULAR PIPE

Figure 6-26 Critical-Depth Chart

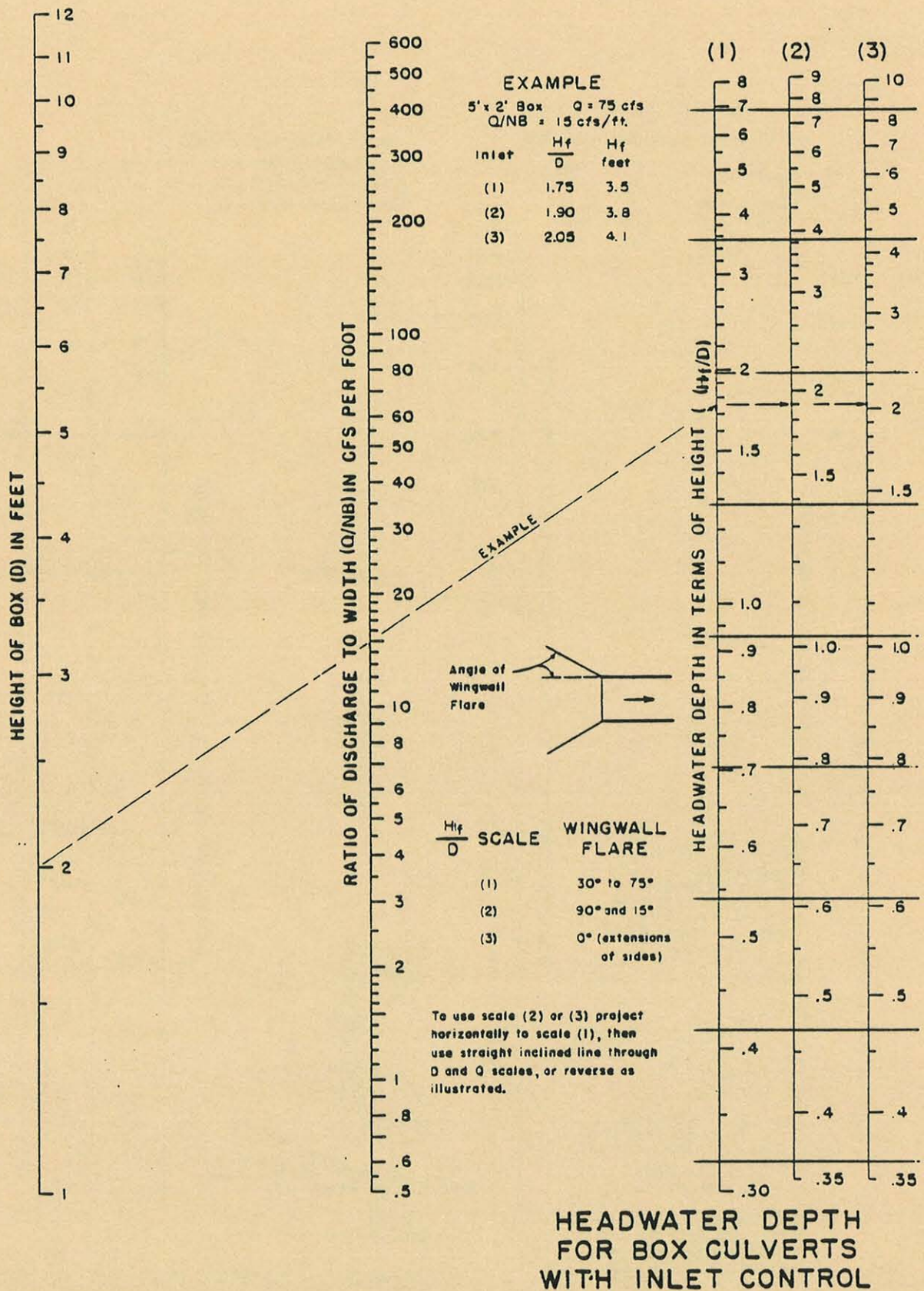


Figure 6-27 Inlet-Control Nomograph

EXAMPLE

B = 7 FT. D = 5 FT. Q = 500 CFS Q/NB = 71.5

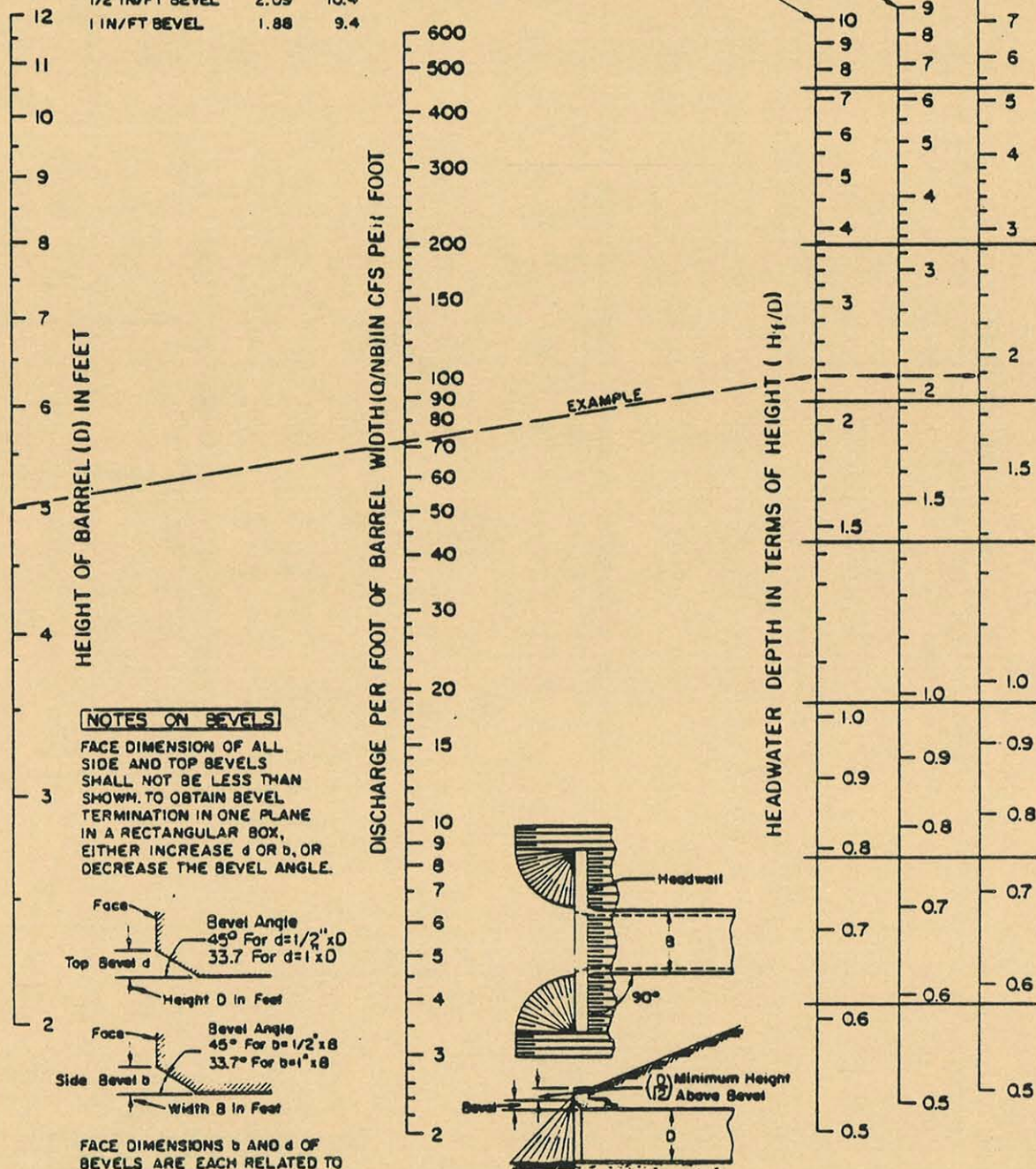
ALL EDGES	$\frac{H_f}{D}$	H_f feet
CHAMFER 3/4"	2.31	11.5
1/2 IN/FT BEVEL	2.09	10.4
1 IN/FT BEVEL	1.88	9.4

INLET FACE-ALL EDGES:

1 IN/FT. BEVELS 33.7° (1:1.5)

1/2 IN/FT BEVELS 45° (1:1)

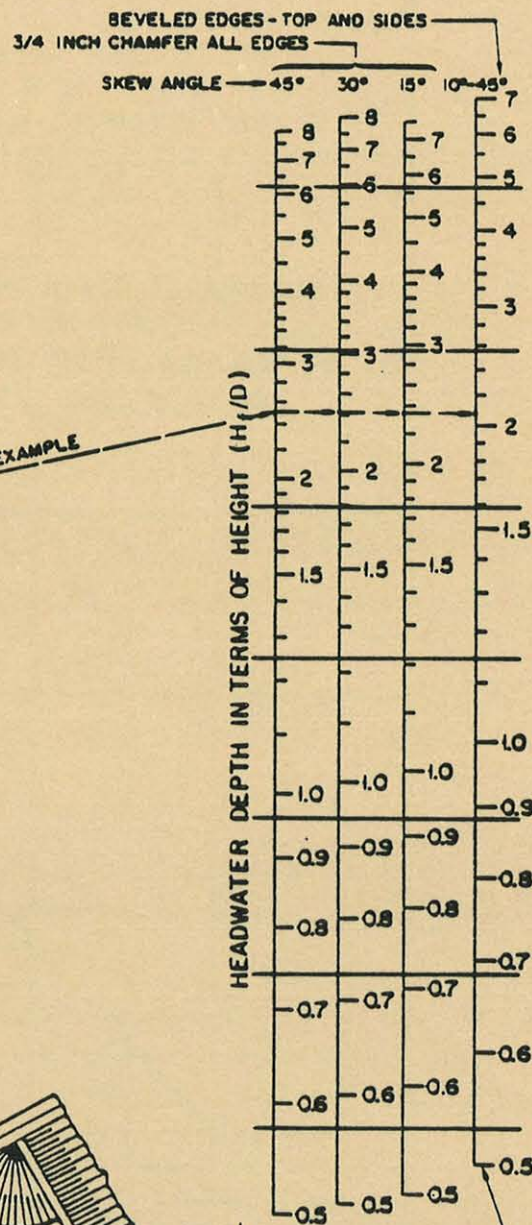
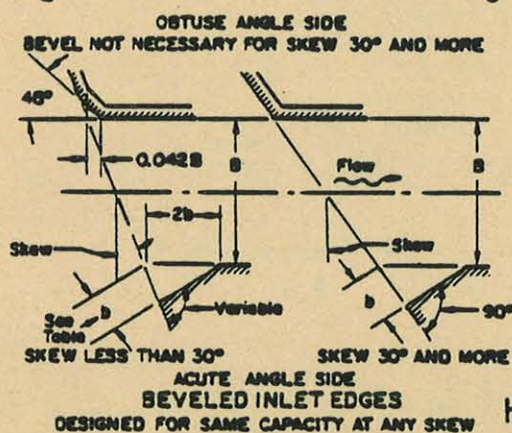
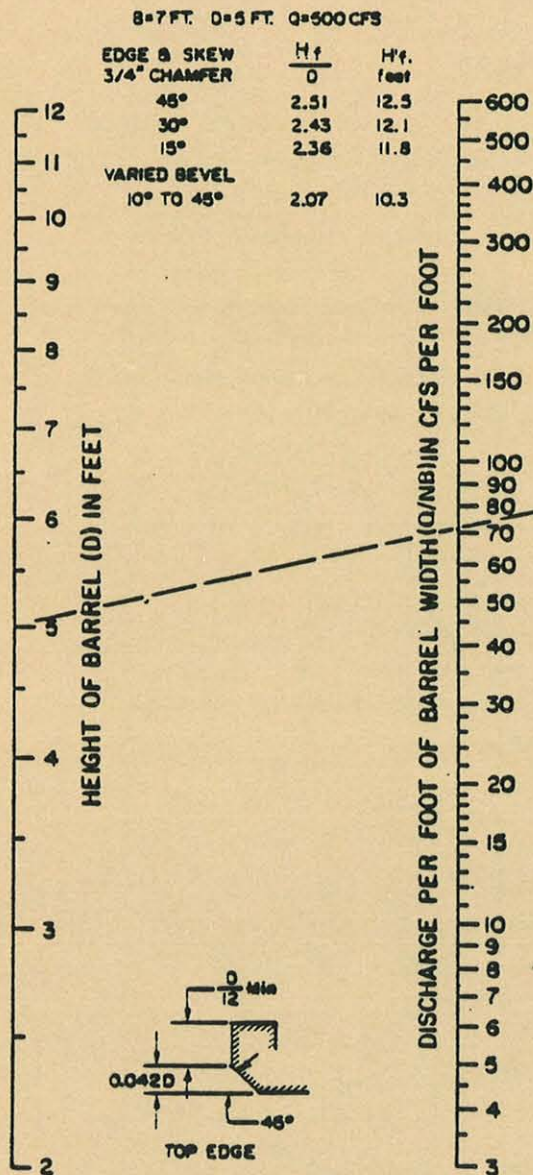
3/4 INCH CHAMFERS



HEADWATER DEPTH FOR INLET CONTROL RECTANGULAR BOX CULVERTS 90° HEADWALL CHAMFERED OR BEVELED INLET EDGES

Figure 6-28 Inlet-Control
Nomograph

EXAMPLE



BEVELED EDGES AS DETAILED

SKEW ANGLE	SIDE BEVEL b
10°	3/4" x 8 (11)
15°	1" x 8
22-1/2°	1-1/4" x 8
30°	1-1/2" x 8
37-1/2°	2" x 8
45°	2-1/2" x 8

HEADWATER DEPTH FOR INLET CONTROL
SINGLE BARREL BOX CULVERTS
SKEWED HEADWALLS
CHAMFERED OR BEVELED INLET EDGES

Figure 6-29 Inlet-Control
Nomograph

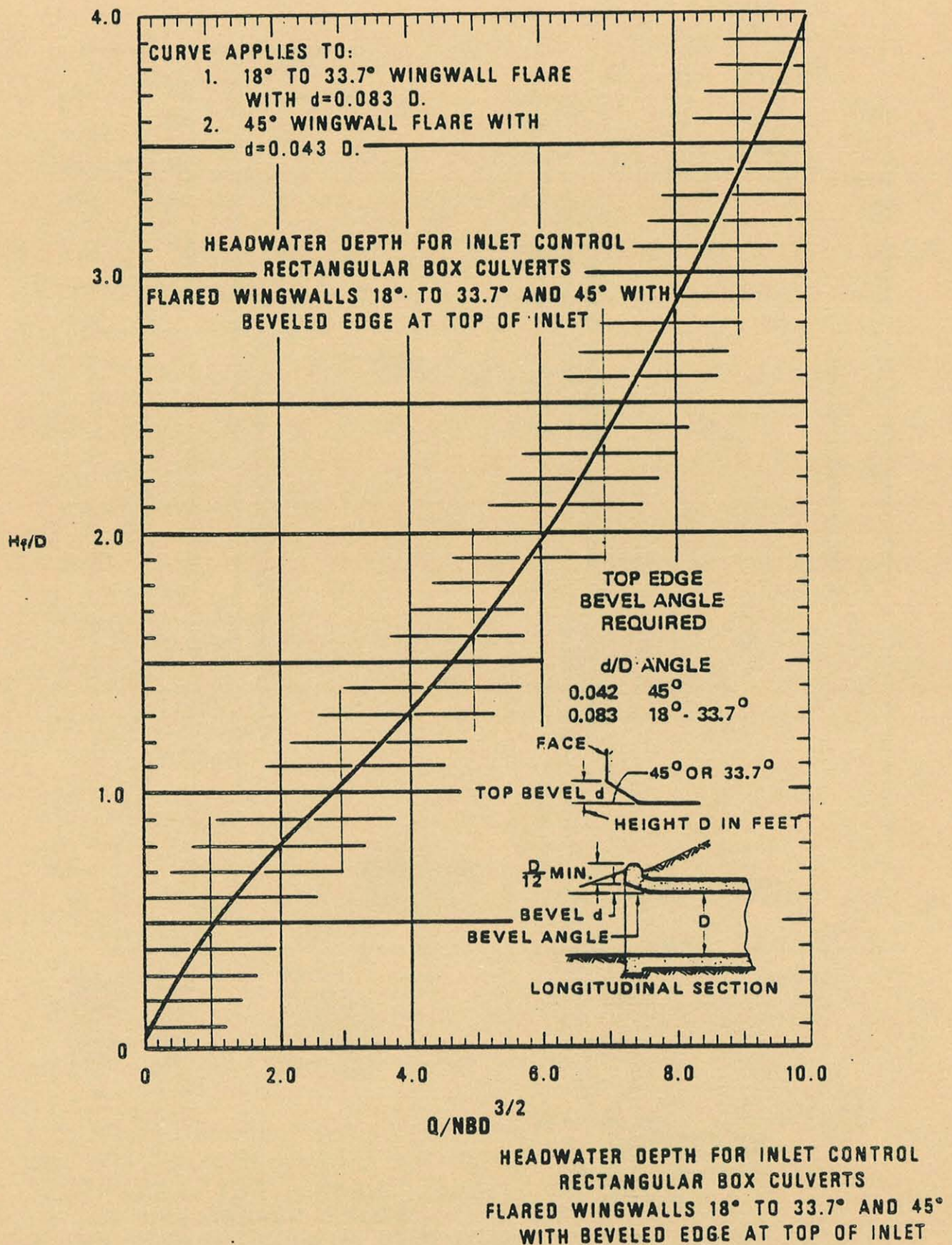


Figure 6-30 Inlet-Control Nomograph

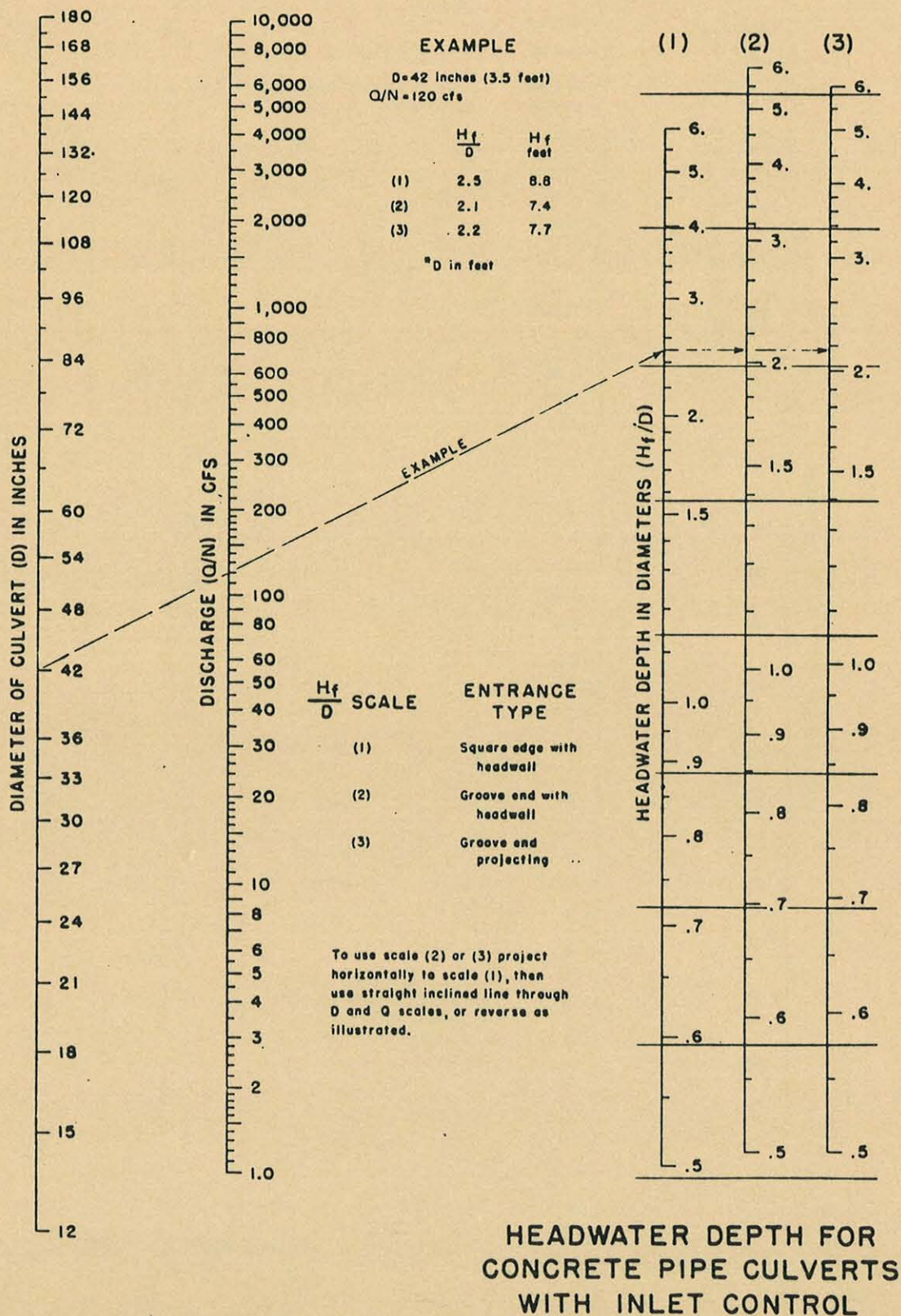


Figure 6-31 Inlet-Control Nomograph

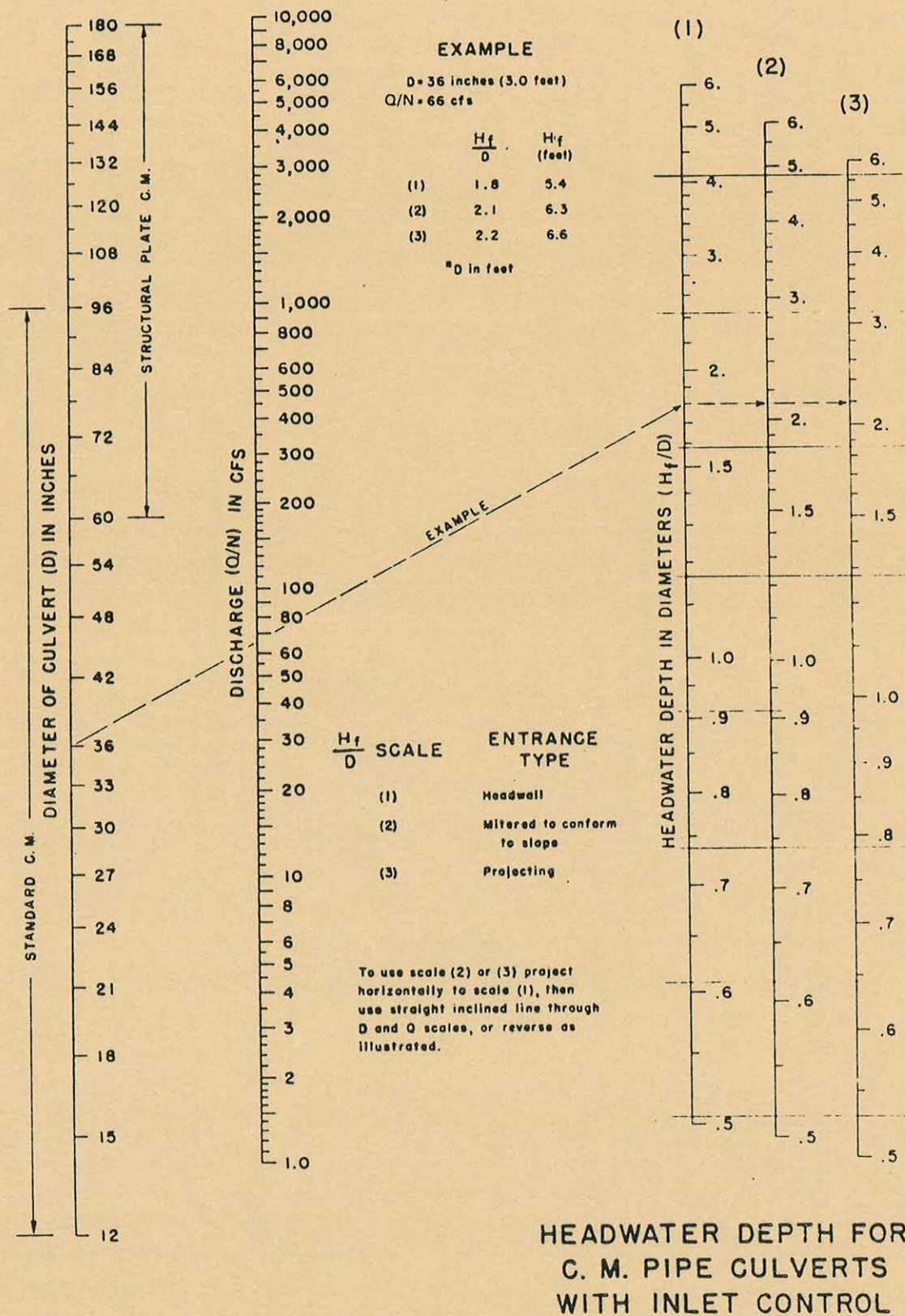


Figure 6-32 Inlet-Control Nomograph

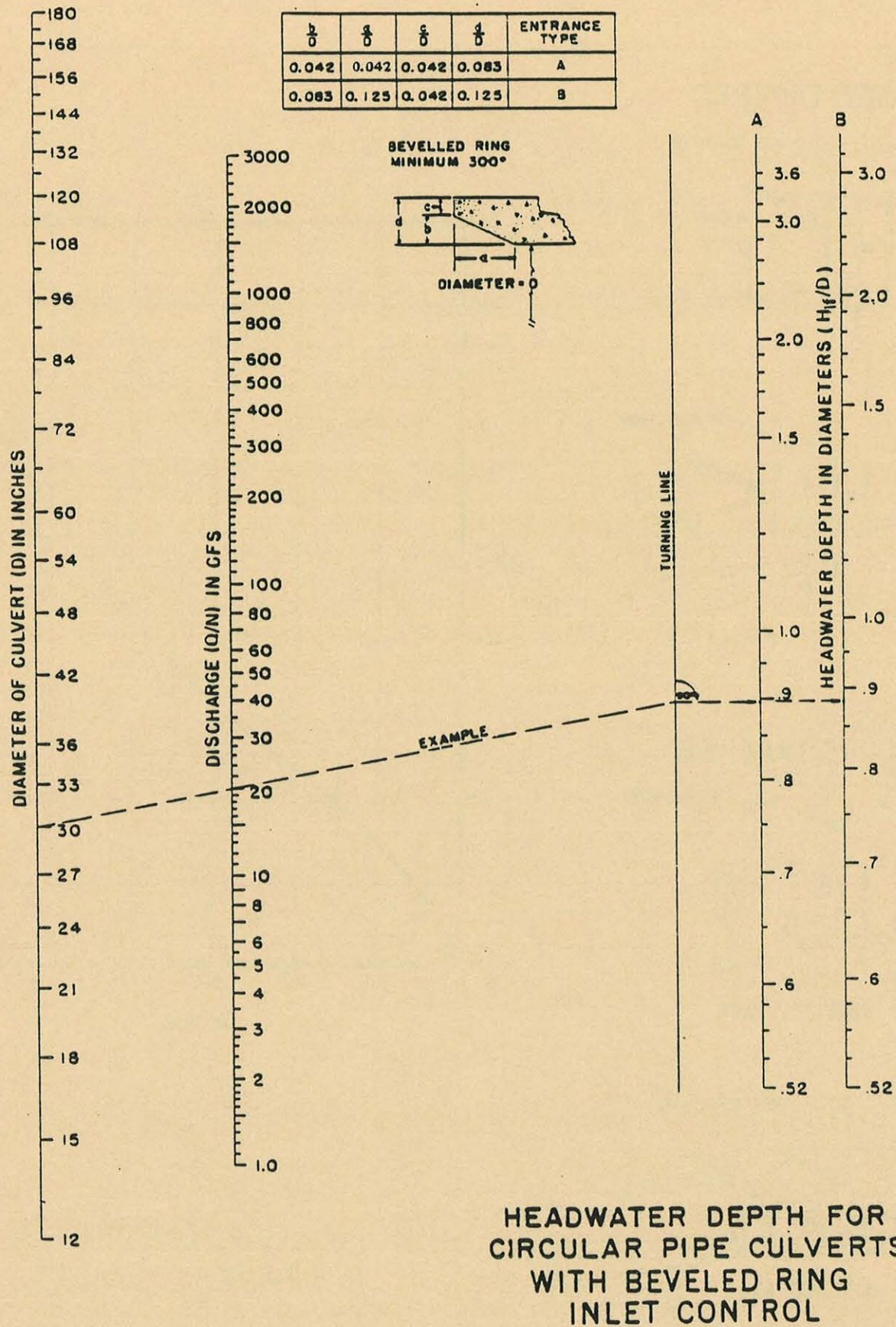
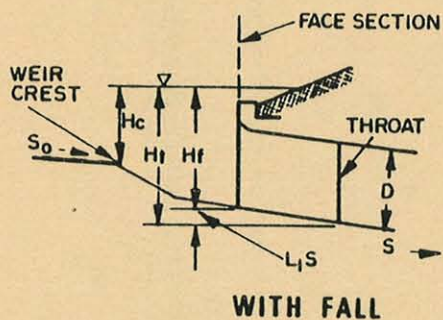
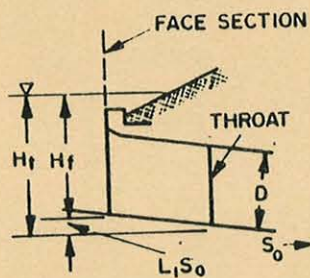


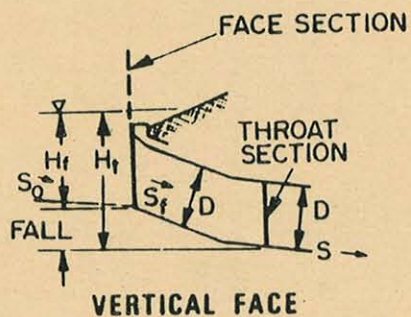
Figure 6-33 Inlet-Control Nomograph

SIDE-TAPERED

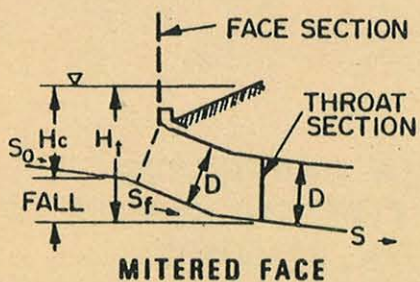


WITH FALL

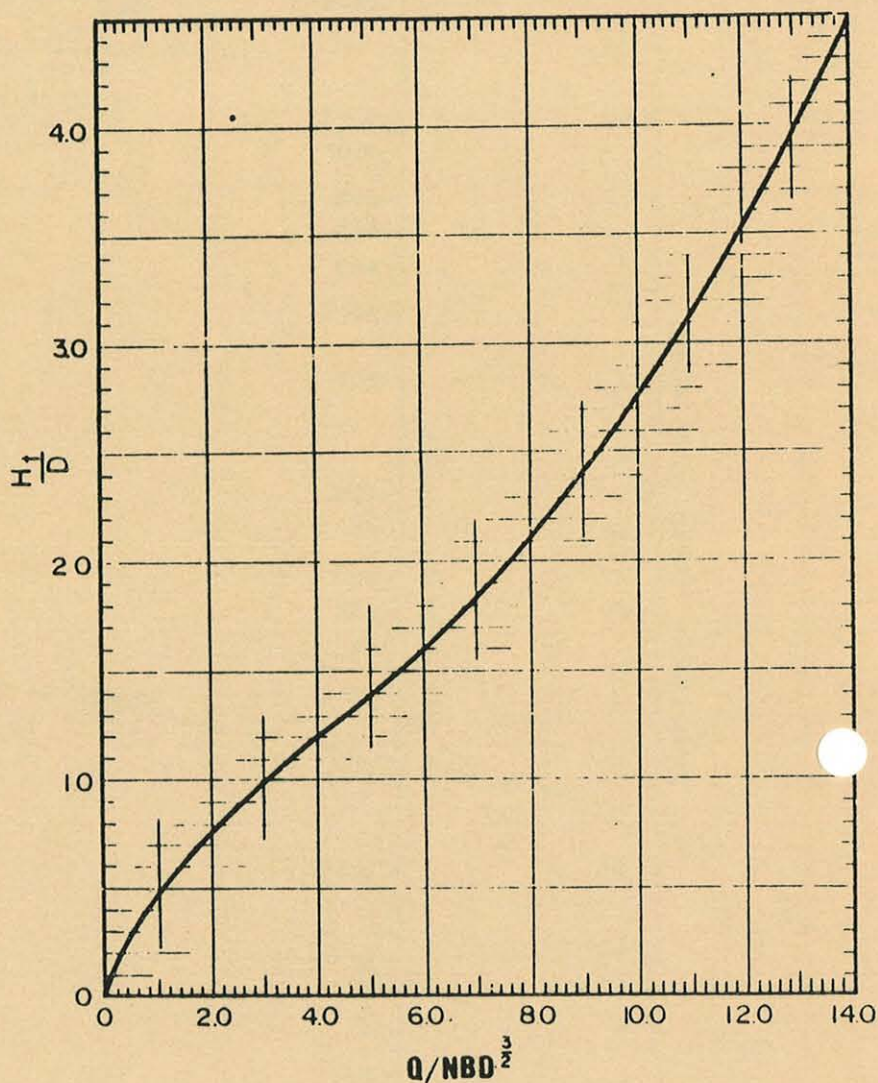
SLOPE-TAPERED



VERTICAL FACE

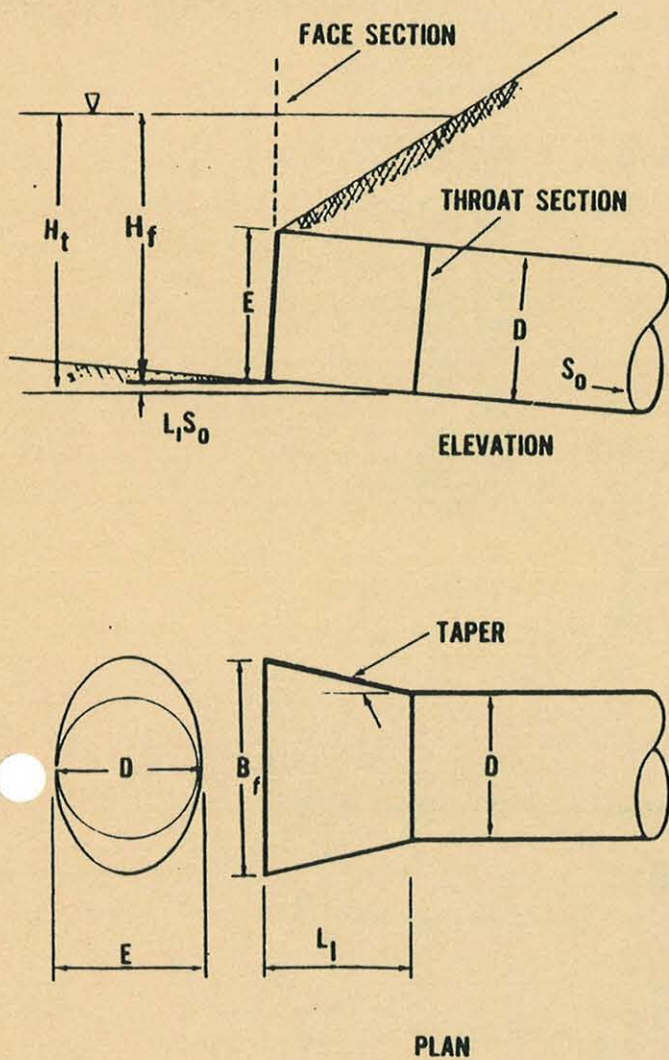


MITERED FACE

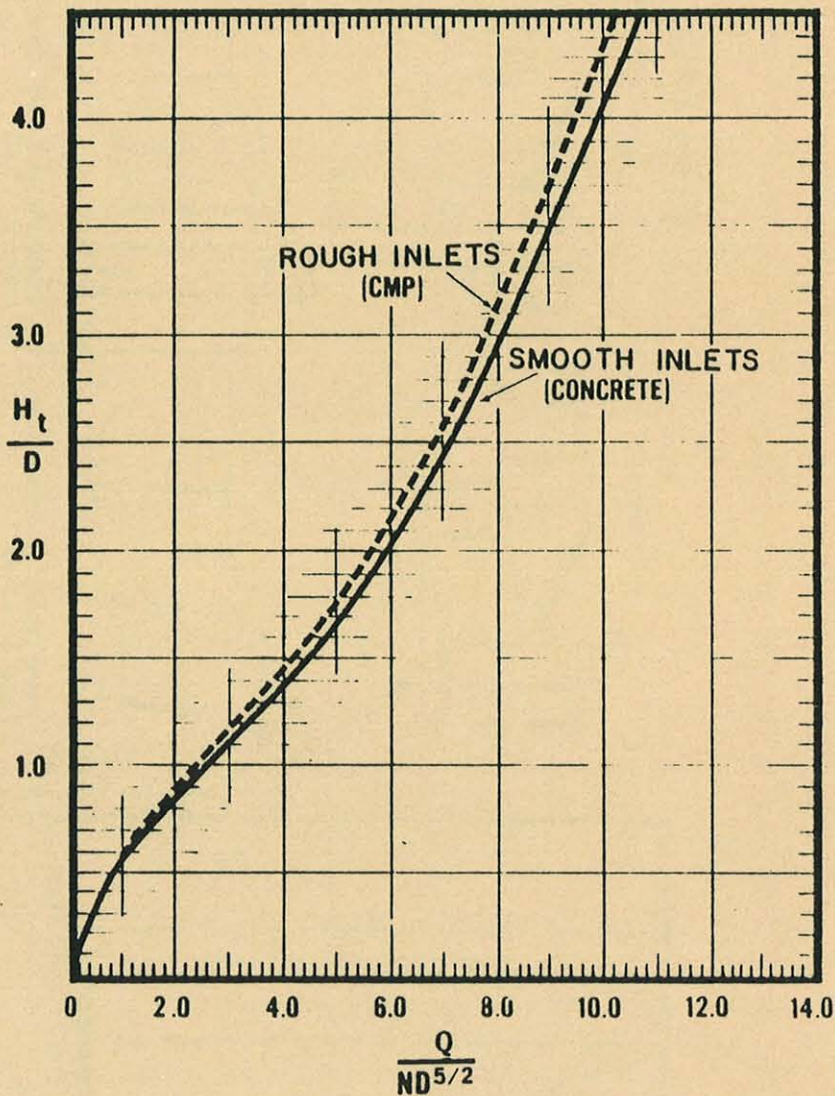


BOX CULVERTS
TAPERED INLETS

Figure 6-34 Throat-Control Curve

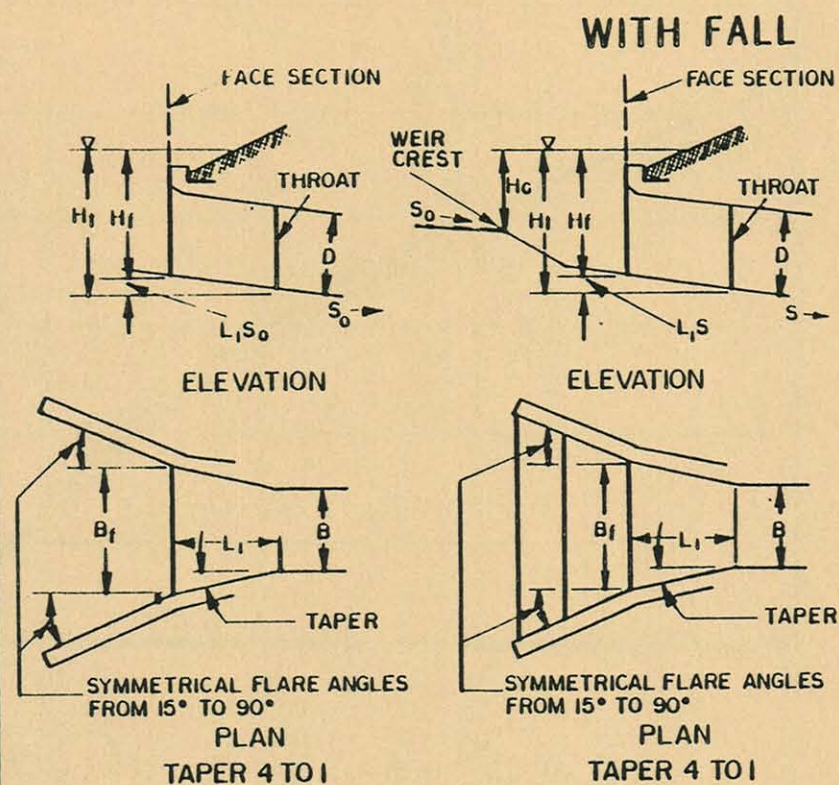
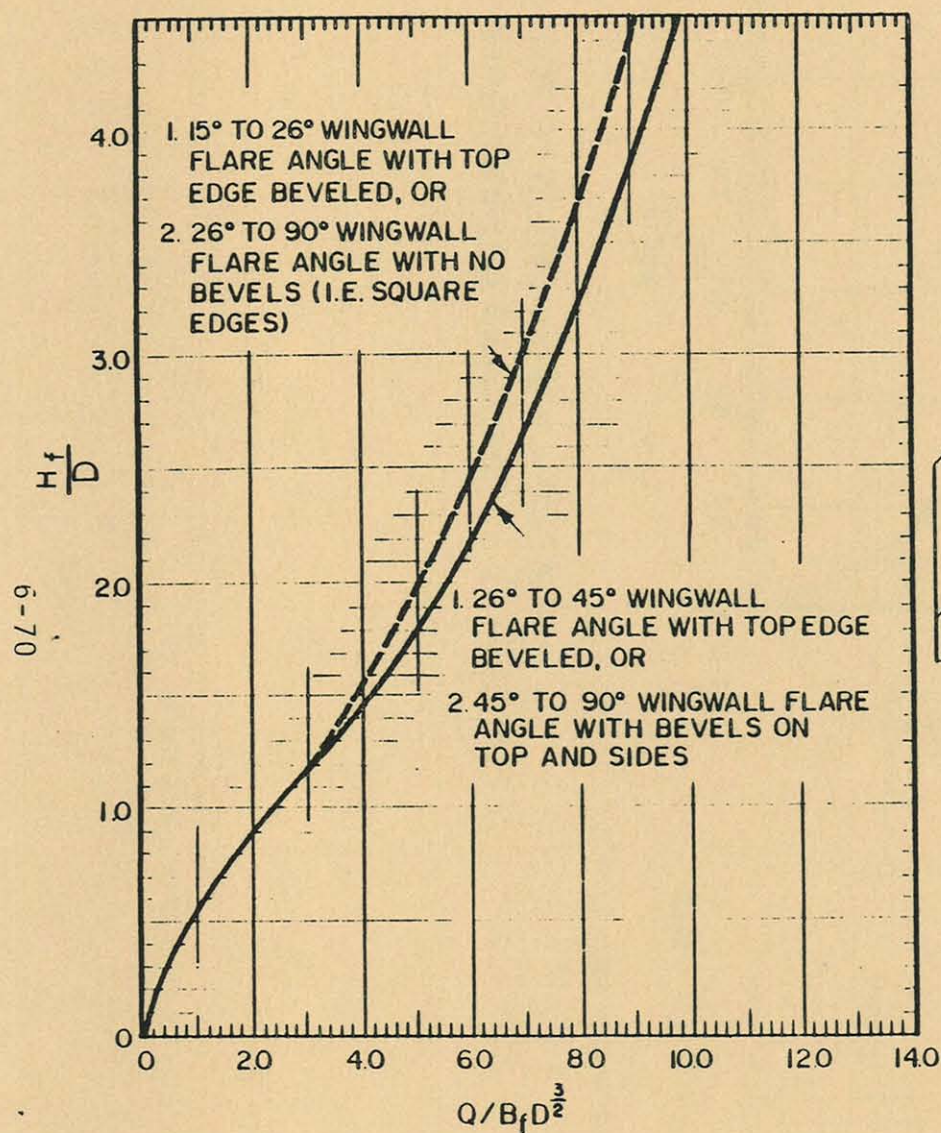


TAPER MAY VARY FROM 4:1 TO 6:1
 $D \leq E \leq 1.1 D$

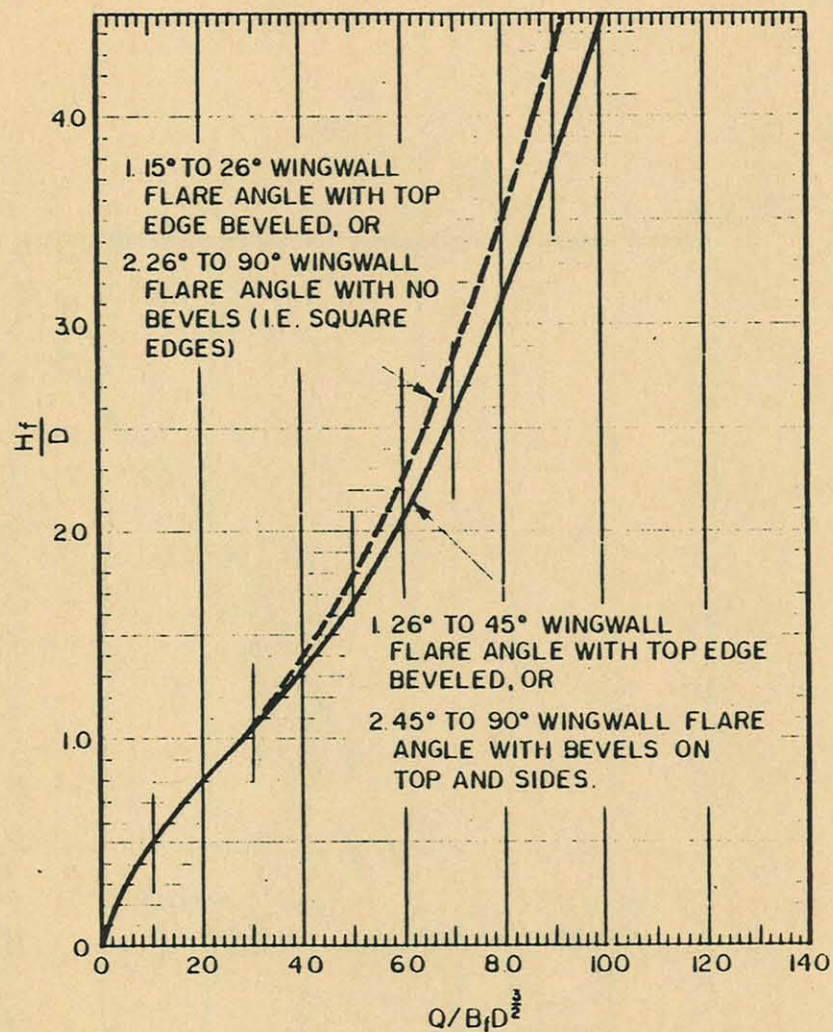


FOR SIDE-TAPERED INLETS TO PIPE CULVERT
 (CIRCULAR SECTIONS ONLY)

Figure 6-35 Throat-Control Curve

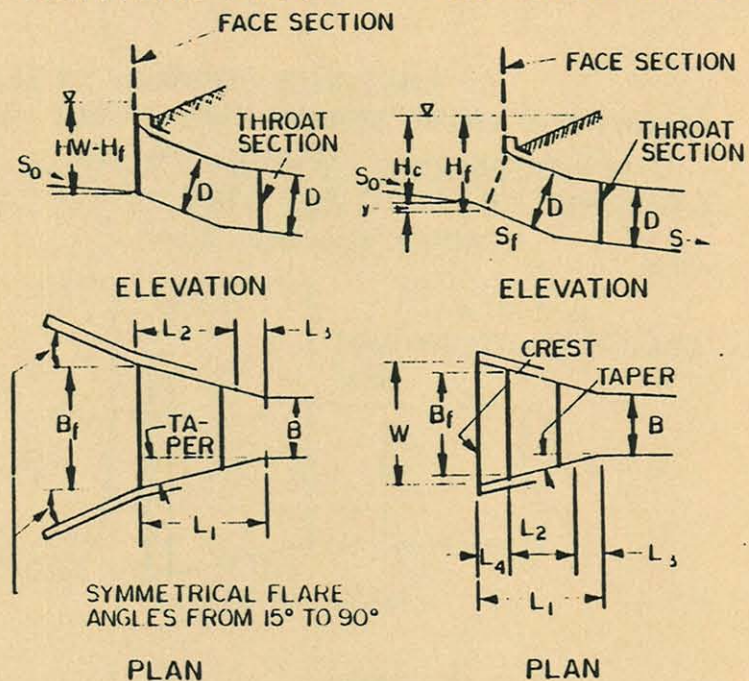


BOX CULVERTS SIDE-TAPERED INLETS



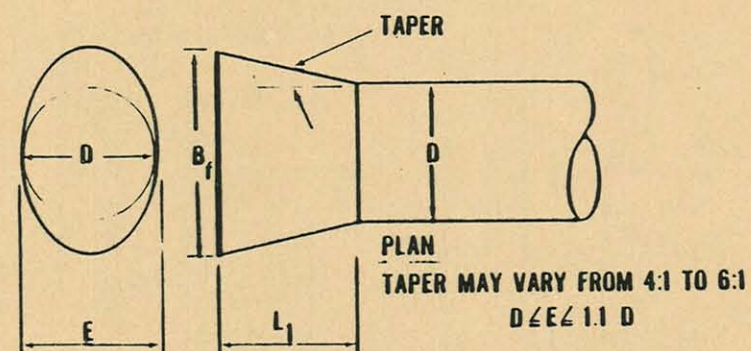
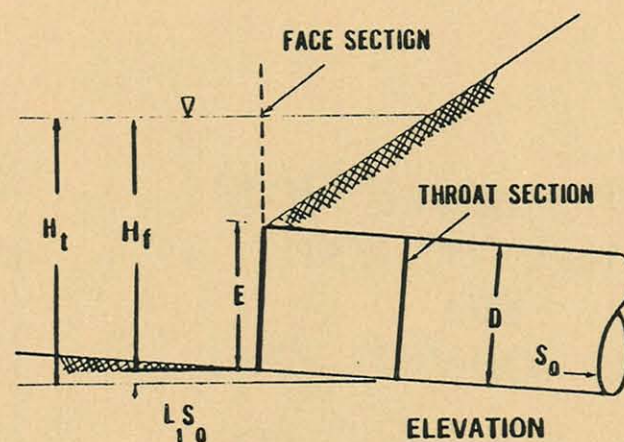
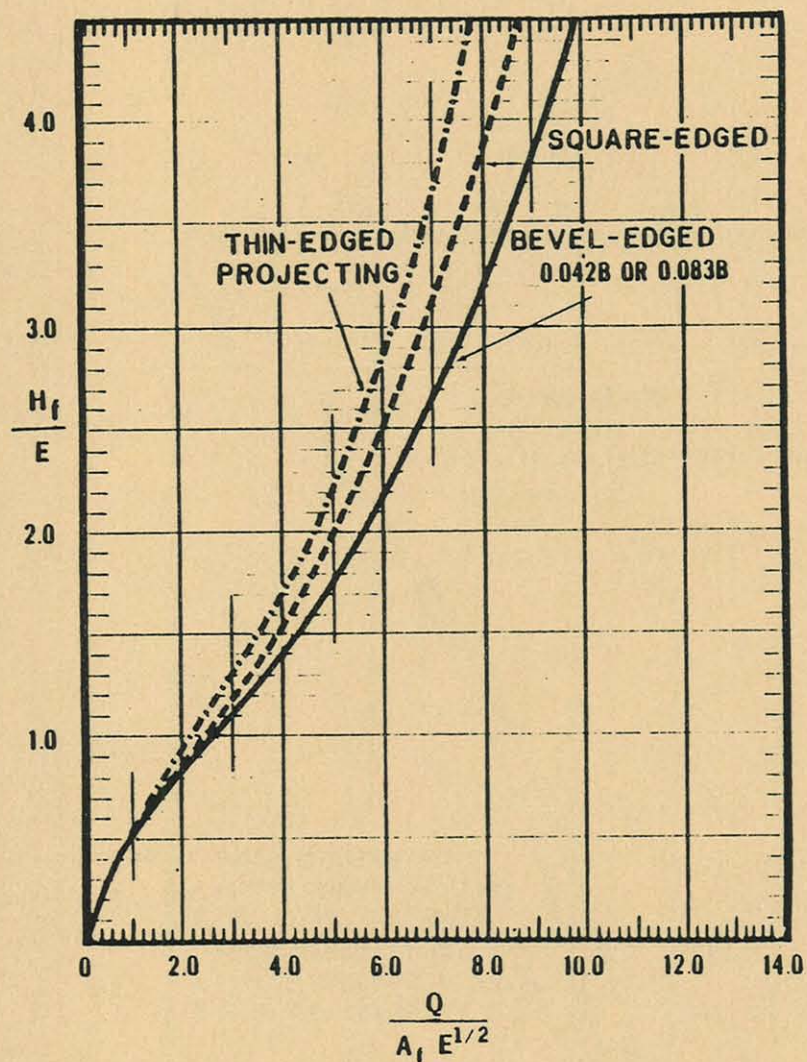
VERTICAL FACE

MITERED FACE



FACE CONTROL CURVES
FOR
BOX CULVERTS
SLOPE-TAPERED INLETS

Figure 6-37 Face-Control Curve



**FACE CONTROL CURVES
FOR SIDE-TAPERED INLETS TO PIPE CULVERTS
(NON-RECTANGULAR SECTIONS ONLY)**

**NOTE: FOR MULTIPLE BARRELS, DESIGN SIDE-TAPERED
INLETS AS INDIVIDUAL STRUCTURES**

Figure 6-38 Face-Control Curve

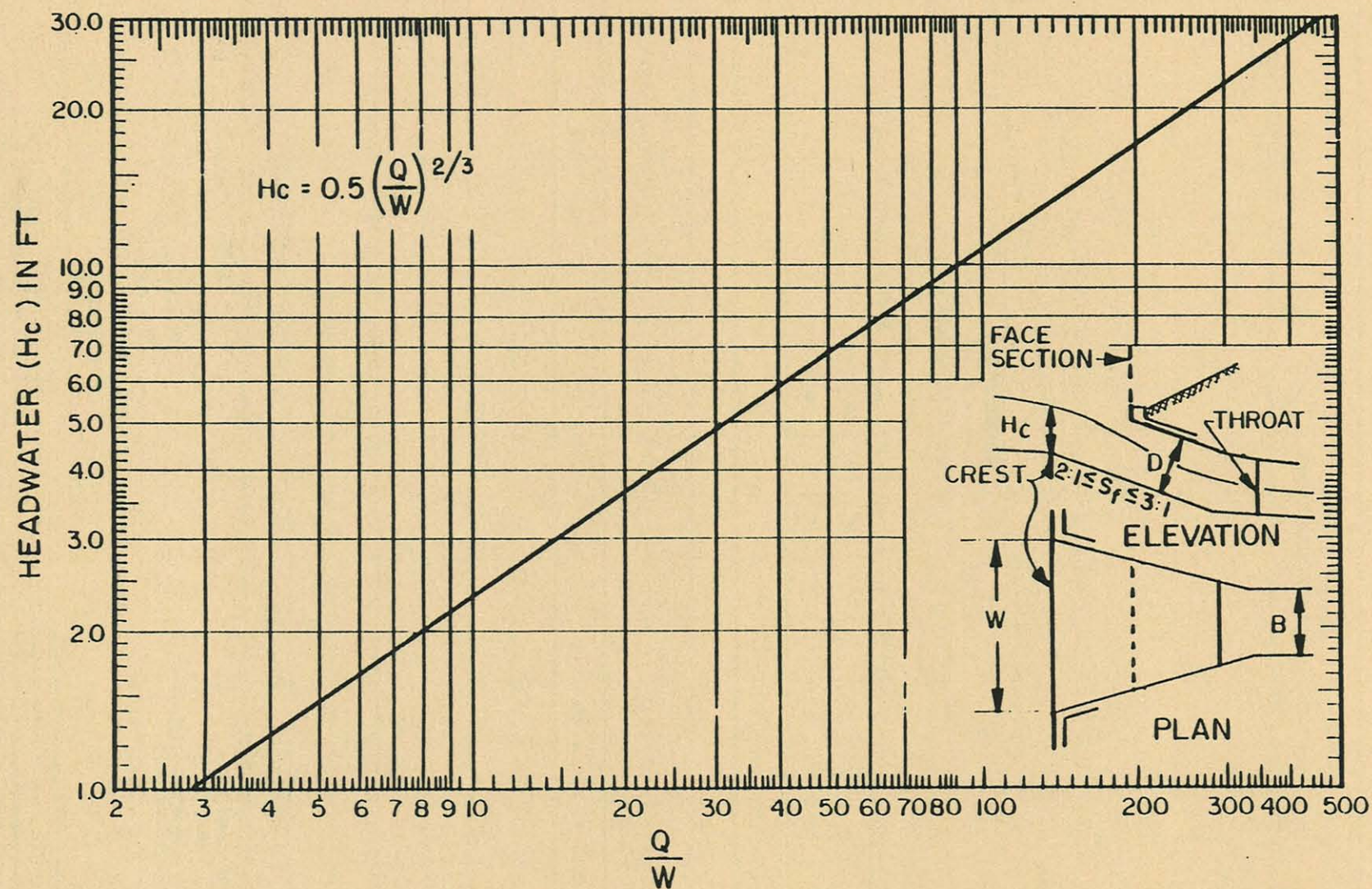


Figure 6-39 Headwater Required for Crest Control

PROJECT: _____	OUTLET CONTROL DESIGN CALCULATIONS	DESIGNER: _____
STATION: _____		DATE: _____

INITIAL DATA:

Q = _____ cfs

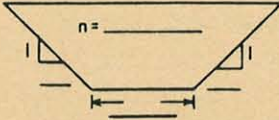
AHW El. = _____ ft.

S_o = _____

L_a = _____ ft.

El. Outlet Invert = _____ ft.

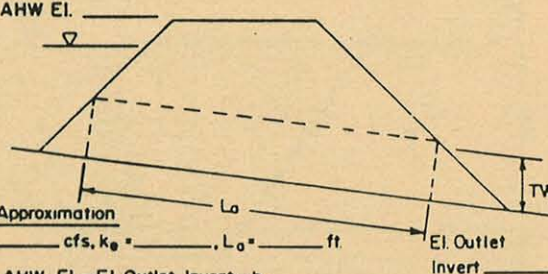
Stream Data:



n = _____

Barrel Shape and Material _____ Barrel n = _____

SKETCH



First Approximation
Q = _____ cfs, k_e = _____, L_o = _____ ft.
H = AHW El. - El. Outlet Invert - h_o
= _____ - _____ - _____ = _____
∴ A = _____ ft.² or D = _____ ft.; Try _____

Q	Q/N	* H	Q/NB	(1) d _c	d _c +D 2	Qn	TW	(3) h _o	(4) HW _o	(5) V _o	COMMENTS
Trial No. _____ , N = _____ , B = _____ , D = _____ , k _e = _____											
Trial No. _____ , N = _____ , B = _____ , D = _____ , k _e = _____											
Trial No. _____ , N = _____ , B = _____ , D = _____ , k _e = _____											

Notes and Equations:

(1) d_c cannot exceed D

(2) TW based on d_n in natural channel, or other downstream control.

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

(4) HW_o = H + h_o + El. Outlet Invert.

(5) Outlet Velocity (V_o) = Q/Area defined by d_c or TW, not greater than D. Do not compute until control section is known.

SELECTED DESIGN

N = _____ At Design Q :

B = _____ ft.

D = _____ ft. HW_o = _____ ft.

k_e = _____ V_o = _____ f/s

* $H = \left[1 + k_e + \frac{29n^2 \cdot L}{R^{1.33}} \right] \frac{v^2}{2g}$

Figure 6-40 Design Computation Form for Improved Culverts

PROJECT: _____		CULVERT INLET CONTROL SECTION DESIGN CALCULATIONS		DESIGNER: _____	
STATION: _____				DATE: _____	

INITIAL DATA:

Q _____ = _____ cfs

AHW El. = _____ ft.

S₀ = _____

L₀ = _____ ft.

El. Stream Bed at Face _____ ft.

Barrel Shape and Material _____ Barrel n = _____

N = _____, B = _____

D = _____, NBD^{3/2} = _____

(Pipe) ND^{3/2} = _____

CONVENTIONAL or BEVELED
INLET: FACE CONTROL SECTION
(Upper Headings)

TAPERED INLET
THROAT CONTROL SECTION
(Lower Headings)

DEFINITIONS OF INLET CONTROL SECTION										
	$\frac{Q}{NB}$	$\frac{H_f}{D}$	H_f	(1) El. Face Invert	El. Stream Bed At Face	(2) FALL	(3) HW _f	(4) S	(5) V ₀	
Q	$\frac{Q}{NBD^{3/2}}$	$\frac{H_f}{D}$	H_f	El. Throat Invert			HW _f			COMMENTS

Trial No. _____ Inlet and Edge Description _____

Trial No. _____ Inlet and Edge Description _____

Trial No. _____ Inlet and Edge Description _____

Notes and Equations:

(1) El. Face (or throat) invert = AHW El. - H_f (or H_t)

(2) FALL = El. Stream Bed at Face - El. face (or throat) invert

(3) HW_f (or HW_t) = H_f (or H_t) + El. face (or throat) invert, where El. face (or throat) invert should not exceed El. stream bed.

(4) S ≈ S₀ - FALL/L₀

(5) Outlet Velocity = Q/Area defined by d_n at S

SELECTED DESIGN

Inlet Description:

FALL = _____ ft.

Invert El. = _____ ft.

Bevels:

Angle = _____

b = _____ in., d = _____ in.

Figure 6-41 Design Computation Form for Improved Culverts

PROJECT: _____		SIDE-TAPERED INLET DESIGN CALCULATIONS		DESIGNER: _____							
STATION: _____		DATE: _____									
INITIAL DATA Q = _____ cfs $S_o =$ _____ AHW El. = _____ ft. $L_a =$ _____ ft. TAPER = _____ : 1 Barrel Shape and Material _____ Face Edge Description _____ N = _____, B = _____ ft, D = _____ ft			SKETCH 								
	Q	El. Throat Invert	$\frac{H_f}{D}$ $\frac{H_f}{E}$	$\frac{Q}{B_f D^{3/2}}$ $\frac{Q}{A_f E^{1/2}}$	$D^{3/2}$ $E^{1/2}$	(2) Min. B_f Min. A_f	B_f	(3) L_1	(4) S	(5) $L_1 S$	Upper Headings for Box Culverts, Lower Headings for Pipes COMMENTS
Trial No. _____, Q = _____, HW _f = _____											
											$B_f D^{3/2} [\text{or } A_f E^{1/2}] =$ _____
Trial No. _____, Q = _____, HW _f = _____											
											$B_f D^{3/2} [\text{or } A_f E^{1/2}] =$ _____
Trial No. _____, Q = _____, HW _f = _____											
											$B_f D^{3/2} [\text{or } A_f E^{1/2}] =$ _____
Notes and Equations: (1) $H_f / D [\text{or } H_f / E] = (HW_f - \text{El. Throat Invert} - 1) / D [\text{or } E]$ $D \leq E \leq 1.1 D$ (2) Min. $B_f = \frac{Q}{(D^{3/2})(Q / B_f D^{3/2})}$ $\text{Min. } A_f = \frac{Q}{(E^{1/2})(Q / A_f E^{1/2})}$ (3) $L_1 = \left[\frac{B_f - NB}{2} \right] \text{ TAPER}$ (4) From throat design (5) El. Face Invert - El. Throat Invert > 1 ft., recompute. Face and Throat may be lowered to better fit site, but do not raise.											SELECTED DESIGN $B_f =$ _____ ft. $L_1 =$ _____ ft. Bevels: Angle _____ ° $d =$ _____ in., $b =$ _____ in. Crest Check: $HW_c =$ _____ ft. $H_c =$ _____ ft. $Q/W =$ _____ (Chart I7) Min. W = _____ ft.

Figure 6-42 Design Computation Form for Improved Culverts

PROJECT: _____		SLOPE-TAPERED INLET		DESIGNER: _____	
STATION: _____		DESIGN CALCULATIONS		DATE: _____	

INITIAL DATA:

Q = _____ cfs S_o = _____

AHW EL. _____ ft. L_d = _____ ft.

El. Stream bed at crest _____ ft.

El. stream bed at face _____ ft.

TAPER = _____ : 1 (4:1 to 6:1)

S_f = _____ : 1 (2:1 to 3:1)

Barrel Shape and Material _____

Inlet Edge Description _____

N = _____, B = _____ ft., D = _____ ft.

VERTICAL **MITERED**

	Q	HW _f	El. Throat Invert	(1) El. Face Invert	(2) H _f	$\frac{H_f}{D}$	$\frac{Q}{B_f D^{3/2}}$	$D^{3/2}$	(3) Min. B _f	B _f	S	Comments
Trial 1												$B_f D^{3/2} =$ _____
Trial 2												$B_f D^{3/2} =$ _____

Note: Use only throat designs with FALL > 0.25D

(1) El. face invert = Vertical = Approx. stream bed elevation at face
Mitered = El. Crest - y, where y = 0.4D (Approx.), but higher than throat invert elevation.

(2) $H_f = HW_f - \text{El. face invert}$

(3) Min. $B_f = \frac{Q}{(D^{3/2})(Q / B_f D^{3/2})}$

(4) Min. L ₃	(5) L ₄	(6) L ₂	(7) Check L ₂	(8) Adj. L ₃	(9) Adj. TAPER	(10) L ₁	(11) W	$\frac{Q}{W}$	H _c	(12) Max. Crest El.	GEOMETRY B _f = _____ ft. L ₃ = _____ ft. L ₁ = _____ ft. L ₄ = _____ ft. L ₂ = _____ ft. d = _____ in. b = _____ in. TAPER = _____ : 1

(4.) Min. $L_3 = 0.5NB$

(5.) $L_4 = S_f + D/S_f$ (Mitered only)

(6.) $L_2 = (\text{El. Face (Crest) Invert} - \text{El. Throat Invert}) S_f - L_4$

(7.) Check $L = \frac{B_f - NB}{2}$ TAPER - L_3

(8.) If (7) > (6), Adj. $L_3 = \frac{B_f - NB}{2}$ TAPER - L_2

(9.) If (6) > (7) Adj. TAPER = $(L_2 + L_3) / \left[\frac{B_f - NB}{2} \right]$

(10.) $L_1 = L_2 + L_3 + L_4$

(11.) Mitered: $W = NB + 2 \left[\frac{L_1}{\text{TAPER}} \right]$

(12.) Max. Crest El. = $HW_f - H_c$

Figure 6-43 Design Computation Form for Improved Culverts

6.70 List of Symbols

<u>Symbol</u>	<u>Units</u>	<u>Description</u>
A_b	sq ft	Area of bend section of slope-tapered inlets
A_f	sq ft	Area of inlet face section
A_t	sq ft	Area of inlet throat section
AHW EL.	ft	Allowable headwater elevation at culvert entrance
B	ft	Width of culvert barrel or diameter of pipe culvert
b	in.	Dimension of side be bevel
B_b	ft	Width of bend section of slope-tapered inlets
B_f	ft	Width of face section of improved inlets
C_d		Discharge coefficient based on bend section control
C_f		Discharge coefficient based on face section control
C_t		Discharge coefficient based on throat section control
D	ft	Height of box culvert or diameter of pipe culvert
d	in.	Dimension of top bevel
d_c	ft	Critical depth of flow
E	ft	Height of side-tapered pipe-culvert face section, excluding bevel dimension
FALL	ft	Approximate depression of control section below the stream bed
g	ft/sec ²	Acceleration of gravity = 32.2
H	ft	Head or energy required to pass a given quantity of water through a culvert flowing in outlet control
H_b	ft	Depth of pool, or head, above the bend section invert
H_c	ft	Depth of pool, or head, above the crest
H_f	ft	Depth of pool, or head, above the face section invert
H_t	ft	Depth of pool, or head, above the throat section invert
HG Line	ft	Hydraulic grade line

<u>Symbol</u>	<u>Units</u>	<u>Descriptions</u>
HW	ft	Headwater elevation; subscript indicates control section (HW, as used in HEC #5, is a depth and is equivalent to H_f in this Manual)
HW _c	ft	Headwater elevation required for flow to pass crest in crest control
HW _f	ft	Headwater elevation required for flow to pass face section in face control
HW _o	ft	Headwater elevation required for culvert to pass flow in outlet control
HW _t	ft	Headwater elevation required for flow to pass throat section in throat control
h _o	ft	Elevation of equivalent hydraulic grade line referenced to the outlet invert
k _e		Entrance energy loss coefficient
k _b		A dimensionless effective pressure term for bend section control
k _t		A dimensionless effective pressure term for inlet throat control
L _a	ft	Approximate total length of culvert, including inlet face section control
L ₁ , L ₂ , L ₃ , L ₄	ft	Dimensions relating to the improved inlet as shown in sketches of the different types of inlets
N		Number of barrels
n		Manning's roughness coefficient
P	ft	Length of depression
Q	cfs	Volume rate of flow
R	ft	Hydraulic radius = $\frac{\text{Area}}{\text{Wetted Perimeter}}$
S	ft/ft	Slope of culvert barrel
S _e	ft/ft	Slope of embankment
S _f	ft/ft	Slope of FALL for slope-tapered inlets (a ratio of horizontal to vertical)

<u>Symbol</u>	<u>Units</u>	<u>Description</u>
S_0	ft/ft	Slope of natural channel
T	ft	Depth of the depression
Taper	ft/ft	Sidewall flare angle (also expressed as the cotangent of the flare angle)
TW	ft	Tailwater depth at outlet of culvert referenced to outlet invert elevation
V	ft/sec	Mean velocity of flow
W	ft	Width of weir crest for slope-tapered inlet with mitered face
W	ft	Top width of depression
y	ft	Difference in elevation between crest and face section of a slope-tapered inlet with mitered face

6.80 Bibliography

1. City of Austin, Texas, Engineering Department et al, Drainage Criteria Manual, Austin, Texas, 1977.
2. Federal Highway Administration, Hydraulic Charts for the Selection of Highway Culverts, U.S. Government Printing Office, Washington, D.C., 1965.
3. Federal Highway Administration, Capacity Charts for the Hydraulic Design of Highway Culverts, U.S. Government Printing Office, Washington, D.C., 1965.
4. Federal Highway Administration, Hydraulic Design of Improved Inlets for Culverts, U.S. Government Printing Office, Washington, D.C., 1973.

7 Open
Channel Flow

Section 7

Open-Channel Flow

7.10 Channel Discharge

Manning's Equation

Uniform Flow

Normal Depth

7.20 Design Considerations

7.30 Channel Cross Sections

Side Slope

Depth

Bottom Width

Trickle Channels

Freeboard

7.40 Channel Drops

7.50 Supercritical Flow

7.60 Bibliography

Section 7

Open-Channel Flow

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.

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

Velocities are usually low, resulting in a longer time of concentration and lower downstream peak flows.

Channel storage tends to decrease peak flows.

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

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 a natural channel, is the most efficient and the most desirable.

In many areas facing urbanization, the runoff has been so minimal that natural channels do not exist. However, small trickle paths nearly always exist which provide an excellent basis for location and construction of channels. Good land planning should reflect even these minimal trickle channels to reduce development costs and minimize drainage problems. In some cases the prudent utilization of natural water routes in the development of a 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 increase in low flows and peak storm runoff rates. A natural channel must be studied to determine the measures needed to avoid future bottom scour and bank cutting. Erosion control measures can be taken at reasonable cost which will preserve the natural appearance without sacrificing hydraulic efficiency.

7.10 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 erosion and silting. The hydraulic characteristics of channels shall be determined by Manning's equation.

$$Q = \frac{1.49}{n} A R^{0.67} S^{0.5} \quad (7-1)$$

where:

Q = discharge in cfs

n = coefficient of roughness

A = cross-sectional area of channel in sq. ft.

R = hydraulic radius of channel, $\frac{A}{P}$, in feet;

P = wetted perimeter, in feet; and

S = slope of the frictional gradient, in feet per foot.

Uniform Flow

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 and 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 the assumption that they will carry uniform flow at the normal depths, 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.

Normal Depth

The normal depth is computed so frequently that it is convenient to use nomographs for various types of cross sections to eliminate the need for trial and error solutions, which are time consuming. Nomographs for uniform flow are given in Figures 7-1 and 7-2.

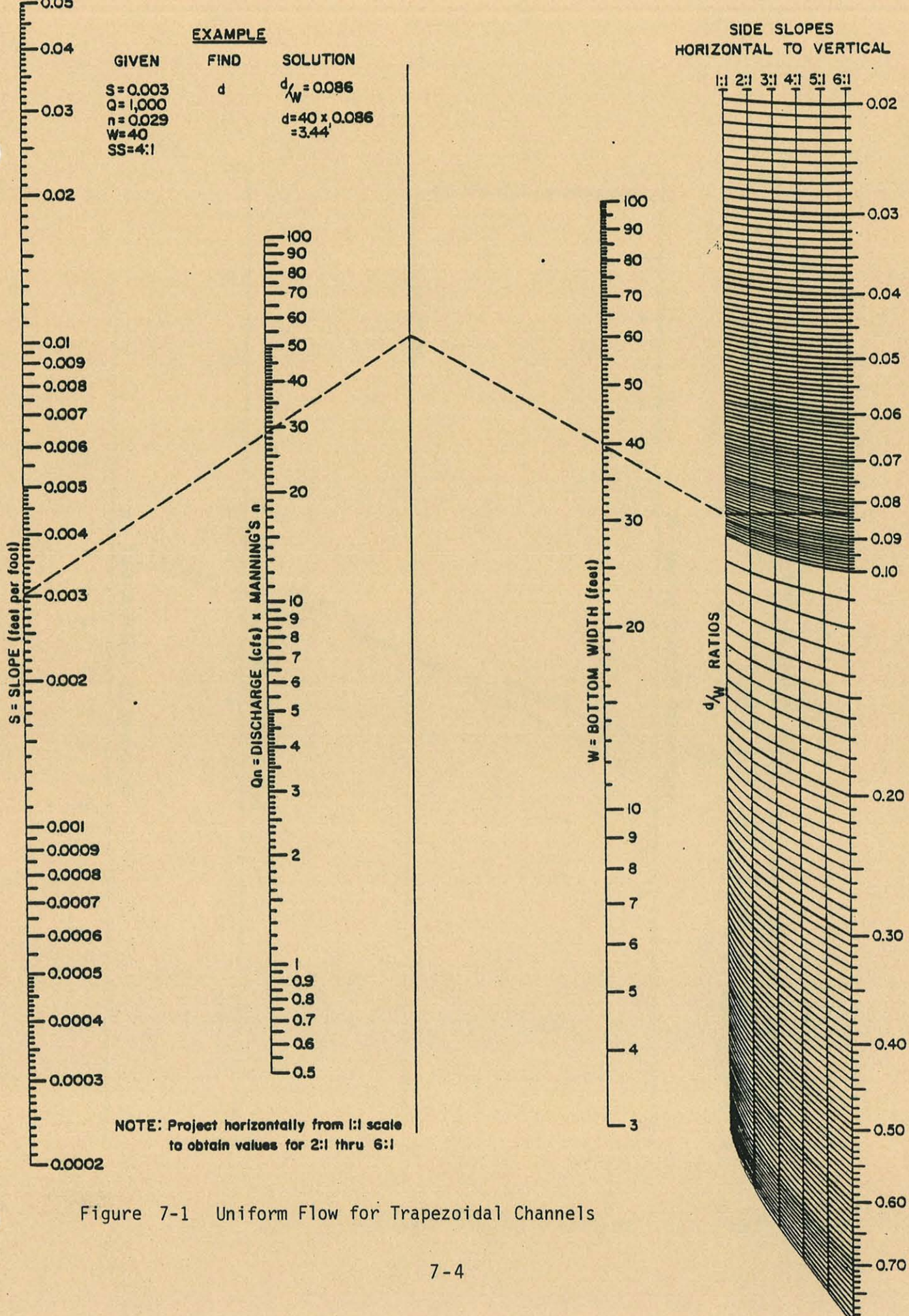


Figure 7-1 Uniform Flow for Trapezoidal Channels

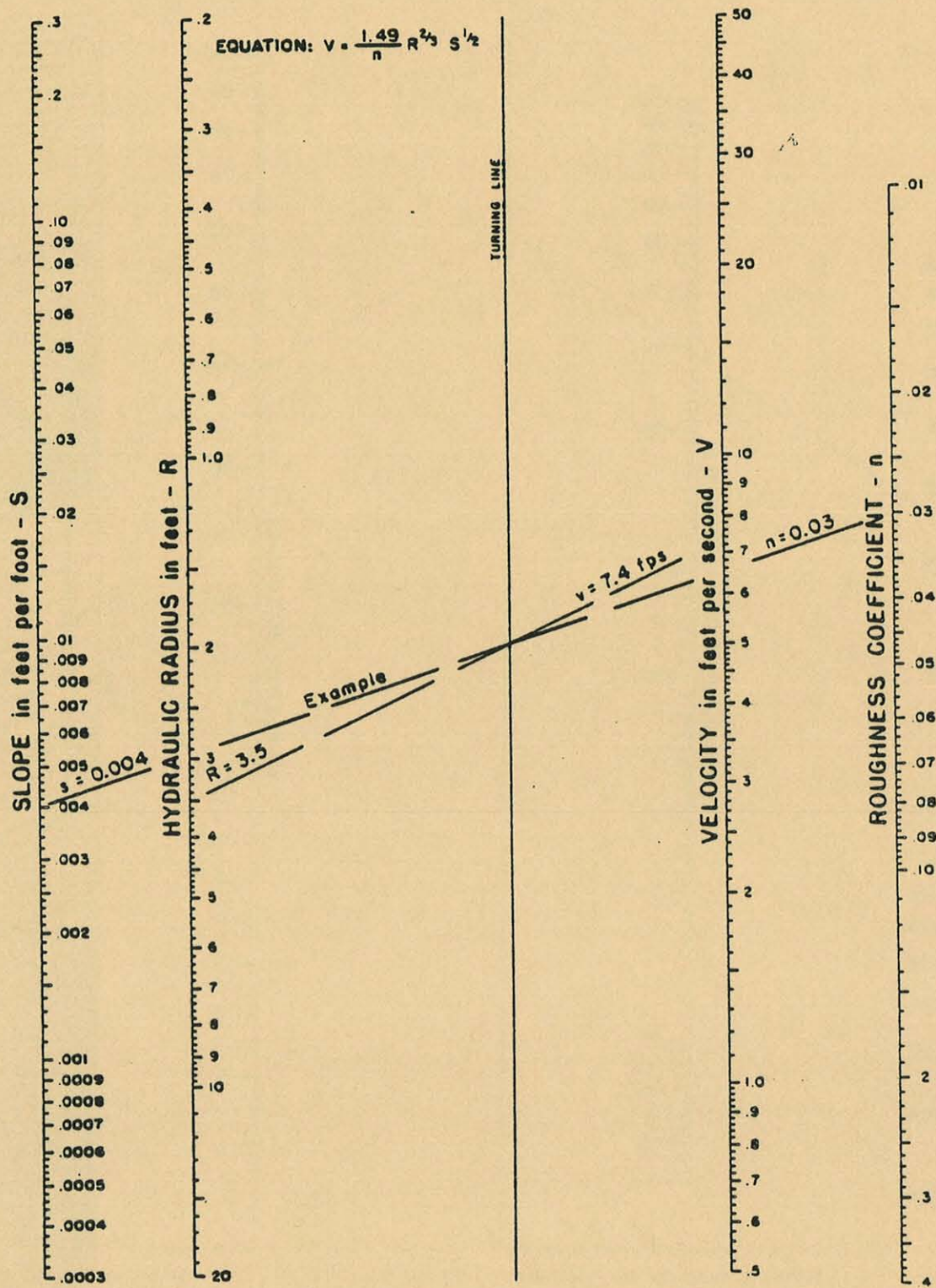


Figure 7-2 Manning's Formula Nomograph

7.20 Design Considerations

Man-made 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. Figure 7-3 shows several typical cross sections used for urban drainage channels with grass cover.

Accurate determination of the "n" value is critical in the analysis of the hydraulic characteristics of a channel. The "n" value for each channel reach should be based on experience and judgment with regard to the individual channel characteristics. Table 7-1 gives a method of determining the composite roughness coefficient based on actual channel conditions.

Where practicable, 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 appropriate agency, channel velocities in man-made, unlined channels should not exceed 6 fps.

Maximum permissible design velocities are shown in Table 7-3 for earth channels with various vegetative cover.

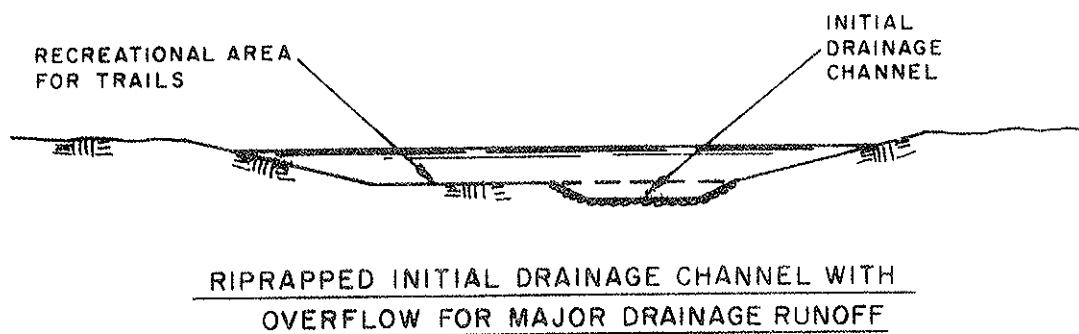
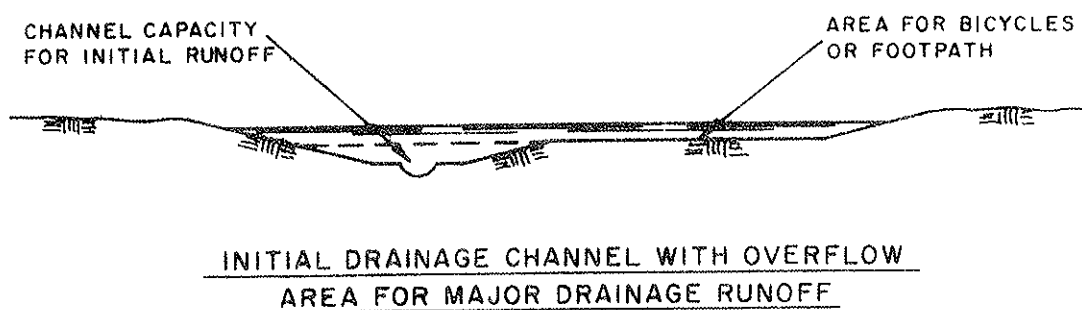
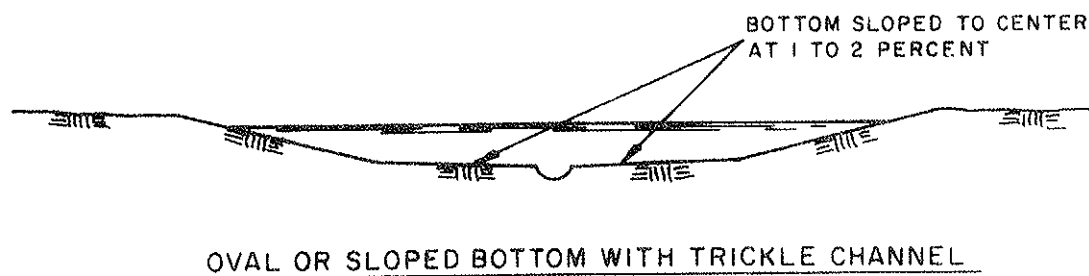
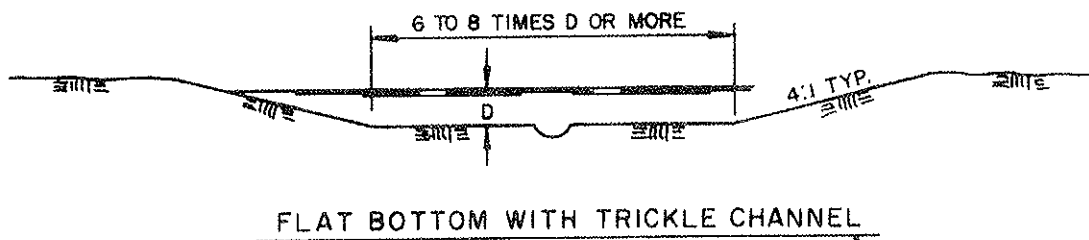


Figure 7-3 Typical Channel Sections

TABLE 7-1
Composite Roughness Coefficients for Channels

$$n = (n_0 + n_1 + n_2 + n_3 + n_4) m \quad (7-2)$$

	<u>Channel Conditions</u>	<u>Value</u>
Material Involved n_0	Earth	0.020
	Fine 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	Negligible	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

TABLE 7-2
Roughness Coefficients for Channels

<u>Type of channel and description</u>	<u>Minimum</u>	<u>Normal</u>	<u>Maximum</u>
EXCAVATED OR DREDGED			
a. Earth, straight and uniform			
1. Clean, recently completed	0.016	0.018	0.020
2. Clean, after weathering	0.018	0.022	0.025
3. Gravel, uniform section, clean	0.022	0.025	0.030
4. With short grass, few weeds	0.022	0.027	0.033
b. Earth, winding and sluggish			
1. No vegetation	0.023	0.025	0.030
2. Grass, some weeds	0.025	0.030	0.033
3. Dense weeds or aquatic plants in deep channels	0.030	0.035	0.040
4. Earth bottom and rubble sides	0.028	0.030	0.035
5. Stony bottom and weedy banks	0.025	0.035	0.040
6. Cobble bottom and clean sides	0.030	0.040	0.050
c. Dragline-excavated or dredged			
1. No vegetation	0.025	0.028	0.033
2. Light brush on banks	0.035	0.050	0.060
d. Rock cuts			
1. Smooth and uniform	0.025	0.035	0.040
2. Jagged and irregular	0.035	0.040	0.050
e. Channels not maintained, weeds and brush uncut			
1. Dense weeds, high as flow depth	0.050	0.080	0.120
2. Clean bottom, brush on sides	0.040	0.050	0.080
3. Same, highest stage of flow	0.045	0.070	0.110
4. Dense brush, high stage	0.080	0.100	0.140

TABLE 7-2 (continued)
Roughness Coefficients for Channels

<u>Type of channel and description</u>	<u>Minimum</u>	<u>Normal</u>	<u>Maximum</u>
NATURAL STREAMS			
Minor streams (top width at flood stage 100 ft)			
a. Streams on plain			
1. Clean, straight, full stage, no rifts or deep pools	0.025	0.030	0.033
2. Same as above, but more stones and weeds	0.030	0.035	0.040
3. Clean, winding, some pools and shoals	0.033	0.040	0.045
4. Same as above, but some weeds and stones	0.035	0.045	0.050
5. Same as above, lower stages, more ineffective slopes and sections	0.040	0.048	0.055
6. Same as 4, but more stones	0.045	0.050	0.060
7. Sluggish reaches, weedy, deep pools	0.050	0.070	0.080
8. Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.075	0.100	0.150
LINED OR BUILT-UP CHANNELS			
a. Corrugated Metal	0.021	0.025	0.030
b. Concrete			
1. Trowel finish	0.011	0.013	0.015
2. Float finish	0.013	0.015	0.016
3. Finished, with gravel on bottom	0.015	0.017	0.020
4. Unfinished	0.014	0.017	0.020
5. Gunite, good section	0.016	0.019	0.023
6. Gunite, wavy section	0.018	0.022	0.025
7. On good excavated rock	0.017	0.020	
8. On irregular excavated rock	0.022	0.027	

TABLE 7-2 (continued)

Roughness Coefficients for Channels

Type of channel and description	Minimum	Normal	Maximum
c. Concrete bottom float finished with sides of			
1. Dressed stone in mortar	0.015	0.017	0.020
2. Random stone in mortar	0.017	0.020	0.024
3. Cement rubble masonry, plastered	0.016	0.020	0.024
4. Cement rubble masonry	0.020	0.025	0.030
5. Dry rubble or riprap	0.020	0.030	0.035
d. Gravel bottom with sides of			
1. Formed concrete	0.017	0.020	0.025
2. Random stone in mortar	0.020	0.023	0.026
3. Dry rubble or riprap	0.023	0.033	0.036
e. Asphalt			
1. Smooth		0.013	
2. Rough		0.016	
f. Grassed	0.030	0.040	0.050

TABLE 7-3

Maximum Permissible Design Velocities¹

Cover	Slope (percent)		
	0-5	5-10	10
Kentucky bluegrass	6	5	4
Wheatgrass	6	5	4
Smooth brome	6	5	4
Tall fescue	6	5	4
Reed canarygrass	4.5	3.5	NR
Annuals ²	2.5	NR	NR
Rye			
Oats			
Ryegrass			

¹ Velocity in fps.² Annuals: use only as temporary protection until permanent vegetation is established.

NR - Not recommended.

7.30 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.

Side Slope

Except in horizontal curves the flatter the side slope, the better. Normally slopes shall be no steeper than 3:1, which is also the practical limit for mowing equipment. Rock or concrete-lined channels or those which for other reasons do not require slope maintenance may have slopes as steep as 1.5:1.

Depth

Deep channels are difficult to maintain and can be hazardous. Constructed channels should therefore be as shallow as practical.

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 with 6 to 8 times the depth desirable.

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 riprapped trickle channel or concrete low flow liners which have a capacity of 5 percent of the design peak flow. Care must be taken to insure that low flows do not create an erosion problem.

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 shall be a minimum of one foot, or provide an additional capacity of approximately one-third of the design flow. For deep flows with high velocities one may use the formula:

$$\text{Freeboard (in feet)} = 1.0 + 0.025 \, v d^{0.33} \quad (7-3)$$

where:

v = velocity of flow, in fps; and
d = depth of flow, in ft.

For the freeboard of a channel on a sharp curve, extra height must be added to the outside bank or wall in the amount:

$$H' = \frac{v^2 (T + B)}{2gR} \quad (7-4)$$

where:

H' = additional height on outside edge of channel, in ft;

v = velocity of flow in channel, in fps;

T = width of flow at water surface, in ft;

B = bottom width of channel, in ft;

R = centerline radius of turn, in ft; and

g = acceleration of gravity, 32.2 ft/sec².

If R is equal or greater than 3B additional freeboard is not required.

7.40 Channel Drops

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. Special attention must be given to protecting the channel from erosion in the area of channel drops.

The Structures section of the Design Manual should be consulted when considering drops in either open-channel or closed-conduit flow.

7.50 Supercritical Flow

The specific energy for open-channel flow may be expressed by the following equation:

$$H = d + \frac{v^2}{2g} \quad (7-5)$$

where:

H = total energy head, in feet;

d = depth of flow, in feet;

v = velocity of flow, in fps; and

g = acceleration of gravity, in feet/sec².

When depth of flow is plotted against specific energy for a given channel and discharge, the resulting curve shows that, at a given specific energy, there are two possible depths. At minimum energy, only one depth of flow

exists, known as the critical depth. At critical depth, the following relationship applies:

$$d = \frac{v^2}{g} \quad (7-6)$$

The Froude Number, F, is defined as:

$$F = \sqrt{\frac{v}{gd}} \quad (7-7)$$

It can be shown that $F = 1$ for critical flow. If the Froude Number is greater than 1, the flow is subcritical, but when the Froude Number is less than 1, the flow is supercritical.

Supercritical flow in an open channel in an urbanized area creates certain hazards which the designer must take into consideration. From a practical standpoint it is generally not possible to have any curvature in such a channel. Careful attention must be taken to insure against excessive oscillatory waves which may extend down the entire length of the channel from only minor obstructions upstream. Imperfections at joints of lined channels may rapidly cause a deterioration of the joints, in which case a complete failure of the channel can rapidly occur. In addition, high-velocity flow entering cracks or joints creates an uplift force by the conversion of velocity head to pressure head which can damage the channel lining. It is evident that when designing a lined channel with supercritical flow the designer must use utmost care and consider all relevant factors. Section 8, Structures, should be consulted when designing channels for supercritical flow.

7.70 Bibliography

1. City of Austin, Texas, Engineering Department et al, Drainage Criteria Manual, Austin, TX, 1977.
2. Chow, Ven Te. Open-Channel Hydraulics, McGraw-Hill Book Company, Inc., New York, NY, 1959.
3. Resource Consultants, Inc., Larimer County Stormwater Management Manual, Fort Collins, CO, 1979.
4. Wright-McLaughlin Engineers, Urban Storm Drainage Criteria Manual, Denver Regional Council of Governments, CO, 1969.

Section 8

Structures

8.10 Energy Dissipators

- Impact Stilling Basin

- Plunge Pools

- Drops

8.20 Flow Transitions

8.30 Riprap

8.40 Bibliography

Section 8

Structures

Hydraulic structures include energy dissipators, channel drops and transitions, baffle chutes, riprap, and many other specific drainage works. Their shape, size, and other features vary widely depending upon the function to be served.

8.10 Energy Dissipators

Energy dissipators are often necessary at the end of outfall sewers, culverts or channels. Stilling basins, a type of energy dissipator, are useful at locations where the designer wants to convert supercritical flow to subcritical flow downstream from a high velocity channel or conduit.

Impact Stilling Basin

Generally, this type of basin lends itself to use with pipes. It is an effective stilling device even with deficient tailwater where the discharge is relatively small. This basin can be used with either an open chute or a closed conduit structure. The design shown in Figure 8-1 has been used for discharges up to about 400 cfs, for larger discharges multiple basins could be placed side-by-side.

The general arrangement of the basin and the dimensional requirements for various discharges are shown on Figure 8-1 and Table 8-1. This type of basin is subject to large dynamic forces and turbulences which must be considered in the structural design. The structure must be made sufficiently stable to resist sliding against the impact load on the baffle wall. The entire structure must resist the severe vibrations inherent in this type of device, and the individual structural members must be sufficiently strong to withstand the large dynamic loads.

Riprap should be provided along the bottom and sides adjacent to the structure to avoid the tendency for scour of the outlet channel downstream from the end sill when a shallow tailwater exists. Downstream wingwalls placed at 45 degrees may also be effective in reducing scouring tendencies and flow concentrations downstream.

Plunge Pools

The plunge pool is a free-falling overflow which drops vertically into a pool.

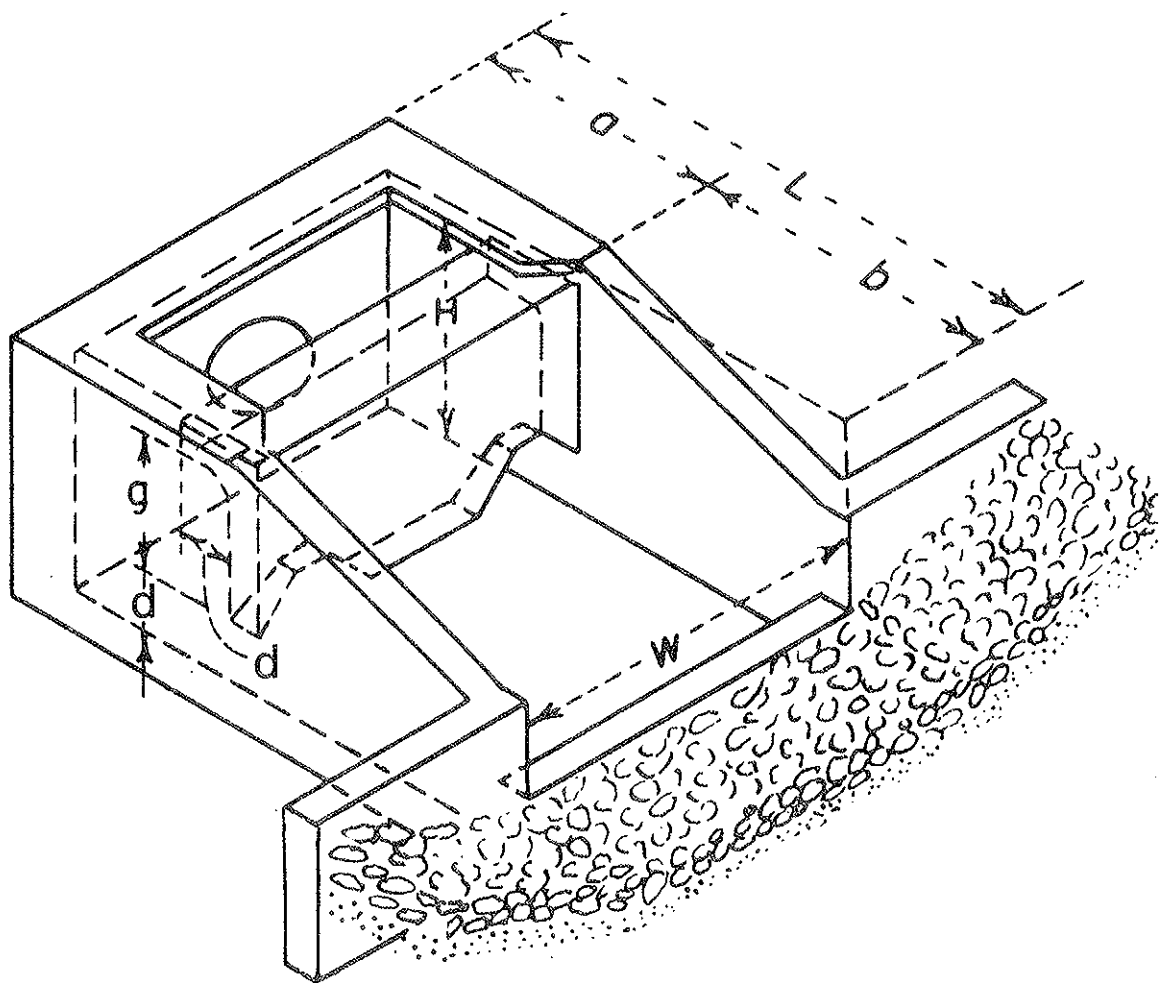


Figure 8-1a Dimensional Criteria for Impact-Type Stilling Basins

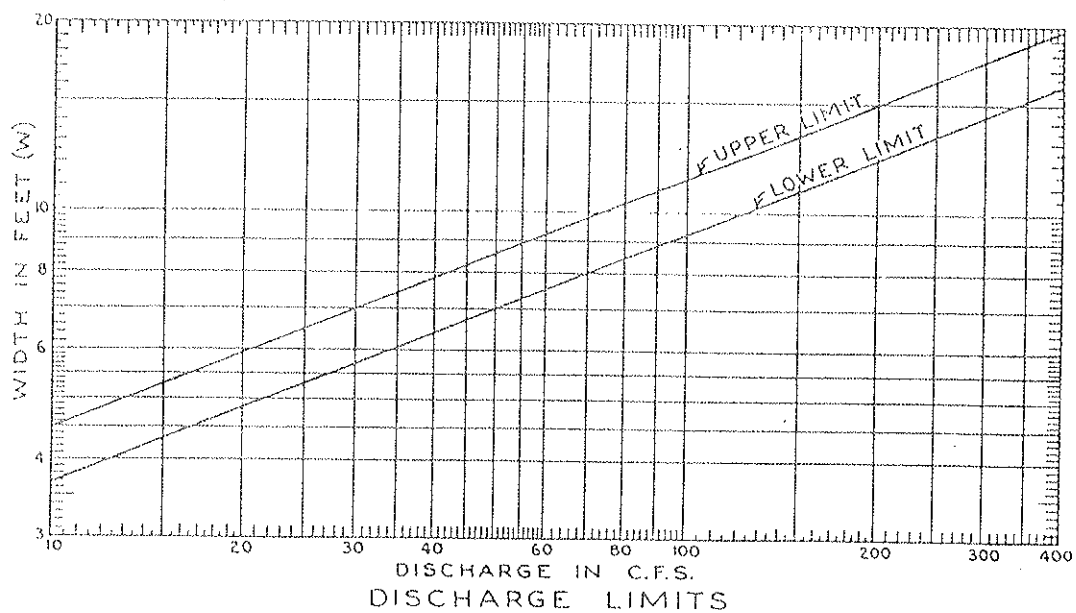
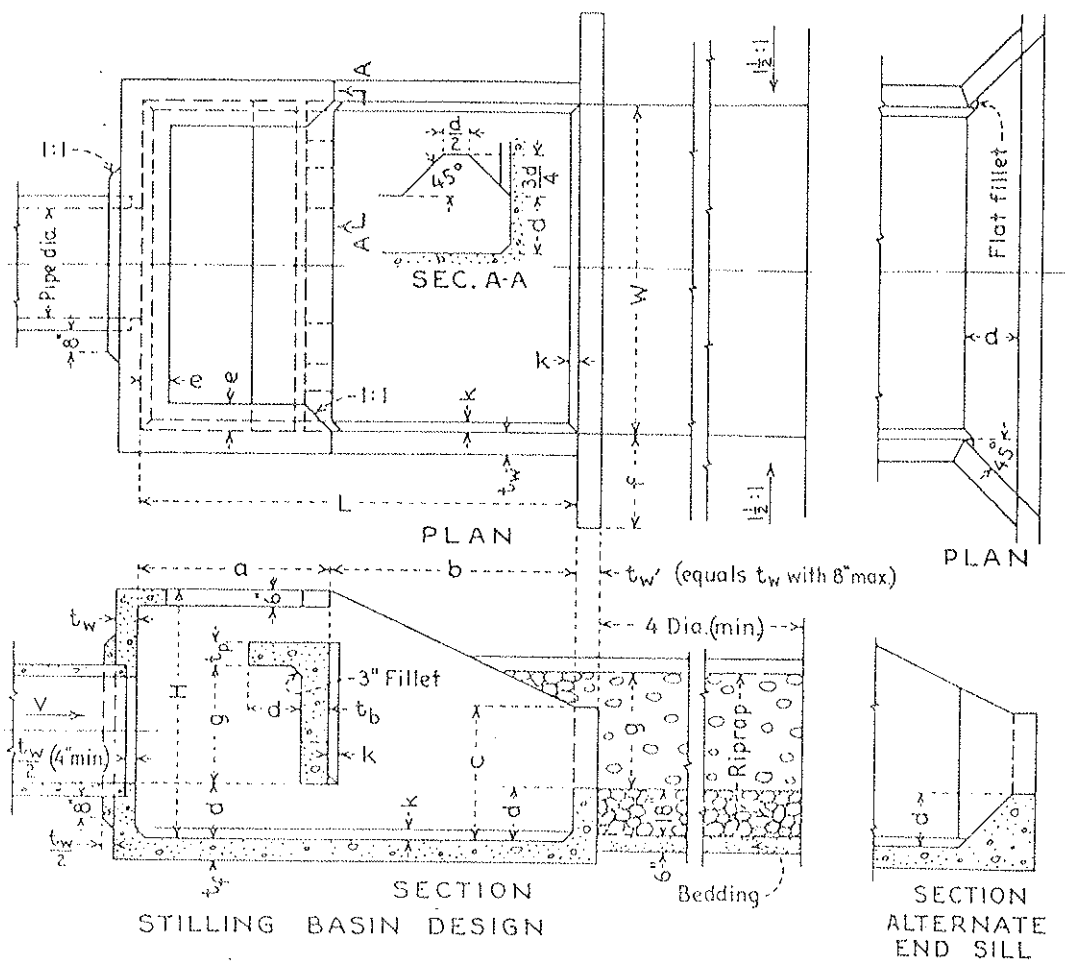


Figure 8-1b Dimensional Criteria for Impact-Type Stilling Basins, Continued

Table 8-1 Dimensions for Impact-Type Stilling Basins

Suggested pipe size ¹		Max discharge Q	Feet and Inches										Inches					
Dis in.	Area (sq ft)		W	H	L	a	b	c	d	e	f	g	t _w	t _f	t _s	t _p	K	Suggested riprap size
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19) ²
18	1.77	21	5-6	4-3	7-4	3-3	4-1	2-4	0-11	0-6	1-6	2-1	6	6½	6	6	3	4.0
24	3.14	38	6-9	5-3	9-0	3-11	5-1	2-10	1-2	0-6	2-0	2-6	6	6½	6	6	3	7.0
30	4.91	59	8-0	6-3	10-8	4-7	6-1	3-4	1-4	0-8	2-6	3-0	6	6½	7	7	3	8.5
36	7.07	85	9-3	7-3	12-4	5-3	7-1	3-10	1-7	0-8	3-0	3-6	7	7½	8	8	3	9.0
42	9.62	115	10-6	8-0	14-0	6-0	8-0	4-5	1-9	0-10	3-0	3-11	8	8½	9	8	4	9.5
48	12.57	151	11-9	9-0	15-8	6-9	8-11	4-11	2-0	0-10	3-0	4-5	9	9½	10	8	4	10.5
54	15.90	191	13-0	9-9	17-4	7-4	10-6	5-5	2-2	1-0	3-0	4-11	10	10½	10	8	4	12.0
60	19.63	236	14-3	10-9	19-0	8-0	11-0	5-11	2-5	1-0	3-0	5-4	11	11½	11	8	6	13.0
72	28.27	339	16-6	12-3	22-0	9-3	12-9	6-11	2-9	1-3	3-0	6-2	12	12½	12	8	6	14.0

¹Suggested pipe will run full when velocity is 12 feet per second or half full when velocity is 24 feet per second. Size may be modified for other velocities by $Q=AV$, but relation between Q and basin dimensions shown must be maintained.

²For discharges less than 21 second-feet, obtain basin width from curve of Fig. 8-1b. Other dimensions proportional to W ; $H=\frac{3W}{4}$, $L=\frac{4W}{3}$, $d=\frac{W}{6}$, etc.

³ Minimum size of individual stones which will resist movement when critical velocity occurs over the end sill.

See Reference 1 for additional information.

The pool must be heavily protected with large riprap or reinforced concrete. The approximate pool depth is given by the following equation:

$$d_s = 1.32 H_T^{0.225} q^{0.54} \quad (8-1)$$

where:

- d_s = the maximum depth of scour below tailwater level, in feet;
 H_T = the head from the reservoir to tailwater levels, in feet; and
 q = the unit discharge, in cfs per foot of stilling basin width.

A plunge pool may only be used with a continuous low flow present in the channel because of the health and safety hazards which could be created by a stagnant pool.

Drops

The function of drop structures is to convey water from a higher to a lower elevation and to dissipate excess energy resulting from its drop. A canal along this same terrain would ordinarily be steep enough to cause severe erosion in earth canals or disruptive flow in lined canals. The water must therefore be conveyed with a drop structure designed to safely dissipate the excess energy. Different kinds of drops that may be used are vertical, baffled apron, rectangular inclined, and pipe drops.

Vertical drops are often the most economical for small drops of less than three feet. They consist of a simple weir above a vertical retaining wall structure and a splash-pool-type energy dissipator that are consolidated in a single inexpensive structure. These structures can be constructed from sheet pile and riprap, gabion retaining walls and channel mats, or reinforced concrete.

Baffled apron drops may be used for nearly any decrease in water-surface elevation where the horizontal distance to accomplish the drops is relatively short. They are particularly adaptable to the situation where the downstream water-surface elevation may vary because of such things as degradation or an uncontrolled water surface. A further discussion on baffled aprons may be found in References 1 and 2.

Rectangular inclined (RI) drops and pipe drops are used where the decrease in elevation is in the range of 3 to 15 feet over a relatively short distance. Economics dictate if it is more practicable to use a pipe drop or an RI drop. Usually a pipe drop will be selected for the smaller flow and an RI drop will be selected for the larger flows. If the drop crosses another waterway or a roadway it will probably be more economical to use a pipe drop.

Chutes are usually used where the drop in elevation is greater than 15 feet and where the water is conveyed over long distances and along grades that may be flatter than those for drops but yet steep enough to maintain supercritical velocities. The decision as to whether to use a chute or a series of drops should be based upon a hydraulic and economic study of the

two alternatives. Drops should not be so closely spaced as to possibly preclude uniform flow between outlet and inlet of consecutive structures, particularly where checks or control notches are not used at the inlets. The danger is that sufficient tailwater depths may not exist to produce hydraulic jumps in the pools, and thus shooting flow may develop through the series of drops and possibly damage the canal. Also, with drops too closely spaced on a steep slope, problems of excavation and backfill may make such construction undesirable or prohibitive. About 200 feet of canal should be the minimum distance between the inlet and outlet of consecutive drop structures. The economic study should compare costs of a series of drops and a chute considering advantages and disadvantages pertinent to the specific conditions. Since the maintenance costs for a series of drops is usually considerably more than for a chute accomplishing the function, it is sometimes economically justifiable to spend considerably more for a chute than for a series of drops. A more complete discussion on chutes will be found in References 1, 2, and 4.

8.20 Flow Transitions

A flow transition is a change of cross section designed to be accomplished in a short distance with a minimum amount of flow disturbance. The types of transitions are shown in Figure 8-2. Of these the abrupt (headwall) and the straight line (wingwall) are the most common.

Special inlet transitions are useful when the conservation of flow energy is essential because of allowable headwater considerations.

Outlet transitions (expansions) must be considered in the design of all culverts, channel protection, and energy dissipators. The standard wingwall-apron combinations and expansions upstream of dissipator basins are most common.

8.30 Riprap

Preventing bank damage caused by surges from a stilling basin and forestalling possible undermining of the structure caused by erosive currents passing over the end sill usually requires placing riprap on the channel bottom and banks downstream.

Experience has shown that the primary reason for riprap failure is undersized individual stones in the maximum size range. Failure has occurred because of an underestimation of the required stone size, and a general tendency for the riprap in place to be smaller than specified, despite quality-control procedures.

The curve in Figure 8-3 gives the individual minimum stone size (diameter and weight of a spherical specimen) for a range of bottom velocities up to 17 feet per second.

A well-graded riprap layer containing about 40 percent of the rock pieces smaller than the required size is as stable, or more stable, than a uniformly-graded layer consisting entirely of stones of the required size.

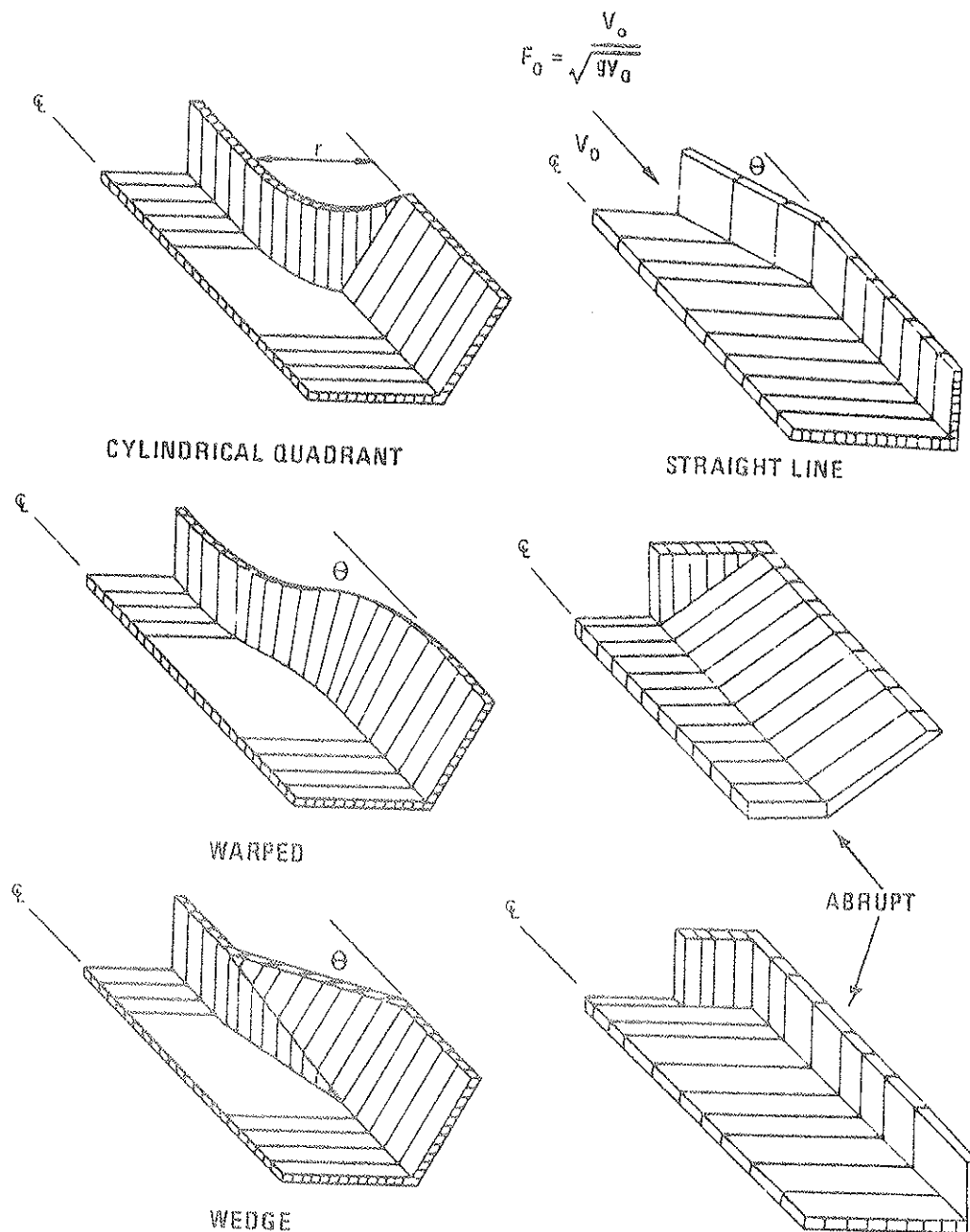


Figure 8-2 Transition Types.

Quality

Stone for riprap shall consist of field stone or rough, unhewn quarry stone of approximately rectangular shape. The stone shall be hard and angular and of such quality that it will not disintegrate on exposure to water or weathering, and it shall be suitable in all other respects for the purpose intended. The specific gravity of the individual stones shall be at least 2.5.

Rubble concrete may be used provided it has a density of at least 150 pounds per cubic foot and otherwise meets the requirements of this Standard. The concrete must be broken such that large flat slabs are eliminated. The largest dimension shall not be greater than three times the smallest dimension (i.e., if a concrete slab is 6 inches thick, the slab would be broken up so that it is no wider than 18 inches in any direction).

The riprap layer should be a minimum of 1.5 times as thick as the dimension of the large stones (curve size) and should be placed over a layer of plastic filter cloth manufactured for that purpose or a minimum of 12 inches of properly grade sand, gravel or stone.

For noncohesive soils the 15-percent size (d_{15}) of the granular bedding or filter material should be less than 4 to 5 times the 85-percent size (d_{85}) of the adjacent soil layer. The 15-percent size of the filter material should also be at least 4 to 5 times the 15-percent size of the protected soil. For cohesive soils such as highly plastic clays, the 15-percent size of the filter material may be as great as 0.4 mm so long as the material is well graded with the ratio of the 60-percent size to the 10-percent size (coefficient of uniformity) not exceeding 20. Section 11 should also be consulted regarding erosion control practices utilizing riprap lining.

Filter

Riprap that is 12 inches in diameter or larger shall not be dumped directly onto the plastic filter cloth since it may tear or displace the filter cloth. Instead, at least a 4-inch blanket of gravel shall be placed over the filter cloth, or riprap shall be placed directly on the filter cloth by hand or by the equipment bucket. Side slopes shall be 2:1 or flatter in order for the gravel not to slide down the filter cloth before placing the riprap.

8.40 Bibliography

1. Federal Highway Administration, Hydraulic Design of Energy Dissipators for Culverts and Channels, Hydraulic Engineering Circular No. 14, Government Printing Office, Washington, DC, 1975.
2. Cedergren, H.R., Seepage, Drainage, and Flow Nets, 2nd ed., John Wiley & Sons, New York, NY, 1977.
3. Peterka, A.J., Hydraulic Design of Stilling Basins and Energy Dissipators. U.S. Department of the Interior, Bureau of Reclamation, Denver, CO 1978.

4. Wright-McLaughlin Engineering, Urban Storm Drainage Criteria Manual,
Denver Regional Council of Governments, CO, 1969.
5. U.S. Department of the Interior, Bureau of Reclamation Design of Small
Canal Structures, Government Printing Office, Washington, DC, 1978.

SIZE OF RIPRAP TO BE USED DOWNSTREAM FROM STILLING BASINS

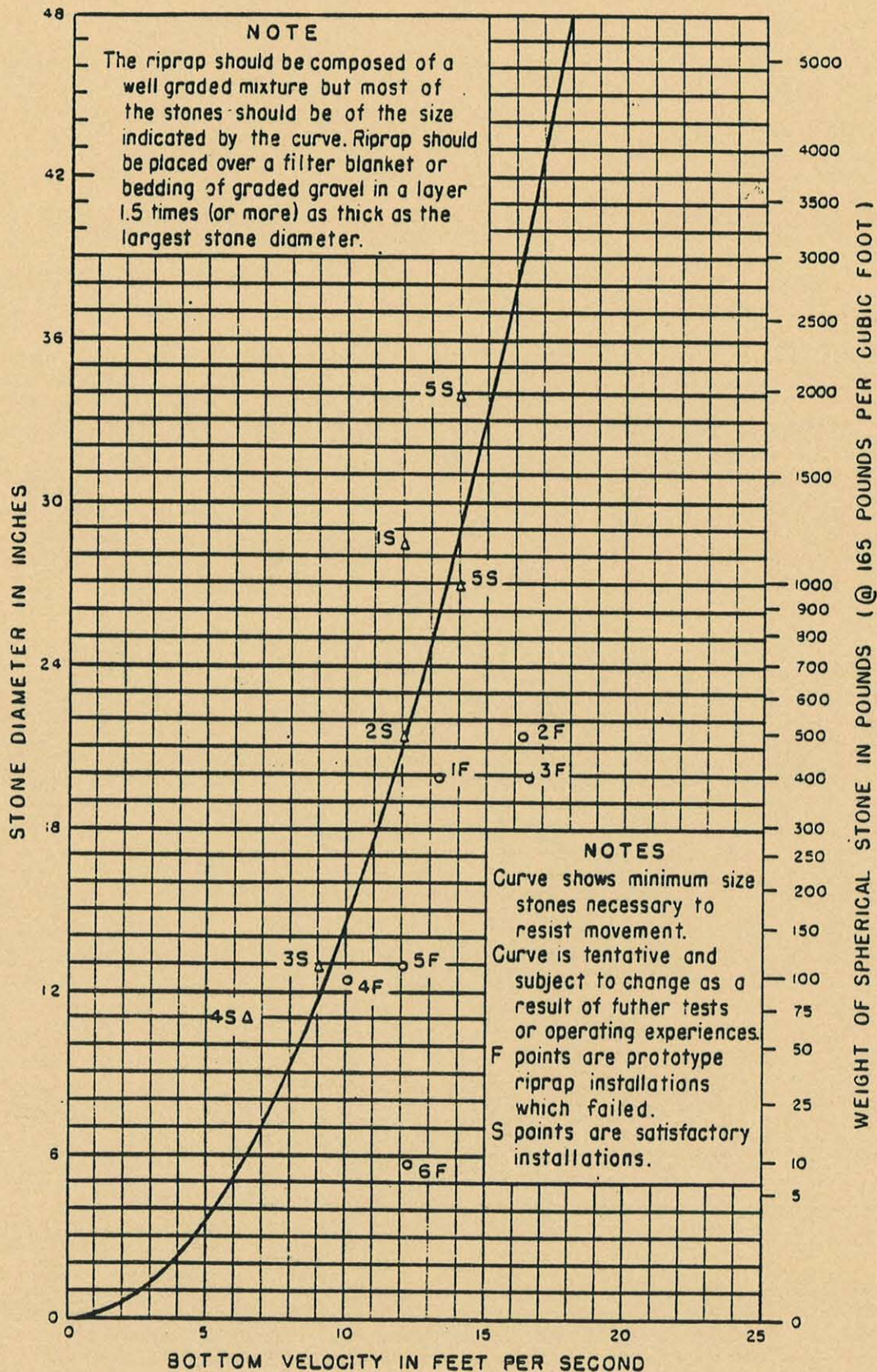


Figure 8-3 Maximum Stone Size for Riprap

Section 9

Storage

- 9.10 Upstream Storage
 - Rooftop Ponding
 - Parking Lots
 - Recreation Areas
 - Road Embankments
 - On-Site Ponds
 - Combinations
- 9.20 Design Criteria
 - Design Storms
 - Principal Outlets
 - Emergency Spillways
 - Site Conditions
 - Embankments
- 9.30 Hydraulic Design Methods
 - Modified Rational Method
 - The SCS Unit Hydrograph Procedure for Storage Analysis
- 9.40 Bibliography

Section 9

Storage

On-site detention of runoff is an alternative to other methods of urban stormwater management. Storage, which involves collecting excess runoff before it enters the main drainage system, can often be an effective and economical means of reducing peak flowrates and mitigating problems of flooding, pollution, soil erosion, and siltation.

The purpose of this Section is to introduce the design procedures and methods of application of on-site detention facilities. This includes rooftop storage, parking lot storage, recreational area storage, and small detention basins and ponds constructed within the limits of development areas.

On-site detention of stormwater generally refers to storage of excess runoff on the site of a development prior to its entry into a sewer system and the gradual release of the stored runoff after the peak flow has passed. Generally, detention facilities will not reduce the total volume of runoff, but will redistribute the rate of runoff over a longer time period.

Embankment type storage areas which impound more than 30 acre-feet of water at an elevation of the top of the embankment are required by Kansas law to be submitted for permit and approval to the Chief Engineer of the Division of Water Resources, Kansas State Board of Agriculture. The designer of such structures shall refer to engineering design guides EG-1 and EG-2, Division of Water Resources, for design criteria beyond the scope of this manual.

9.10 Upstream Storage

Upstream storage utilization is usually controlled by the planner in the early stages of development. However, architects, engineers, home builders, land developers, and governmental officials all have a responsibility to work towards more upstream storage. It is with upstream storage that the greatest potential exists for reducing the cost of urban drainage.

Rooftop Ponding

Building codes require the design of roofs for a snowload. Thus, many existing buildings are already designed to carry the load which would be imposed by rooftop rainfall ponding.

The drainage outlet from the rooftop storage should be sized to release approximately 0.5 inches per hour. During most storms, no water will be stored because it will runoff as quickly as it falls. Only in the larger storms will water accumulate. For instance, in a heavy thunderstorm lasting 30 minutes having a precipitation rate of 2 inches per hour, the maximum depth of ponding will be about 0.75 inches. About 90 minutes after the storm all water will have drained off the roof.

Parking Lots

There are two general forms of stormwater detention on parking lot surfaces. One form involves the storage of runoff in depressions constructed at drain locations. The stored water is drained into the sewer system slowly, using restrictions, such as orifice plates, in the drain. Proper design of such paved areas would restrict ponding to areas which would cause the least amount of inconvenience to the users of the parking areas. For example, the parking lot of a shopping center would have the ponding areas located in the least-used portions of the lot, allowing customers to walk to their vehicles in areas of no ponding, except when the entire lot is filled with vehicles. Drainage of ponded water would be fairly rapid as compared to rooftop ponding, to prevent customer inconvenience. In most cases, the water would pond to a depth not to exceed 12 inches and the ponding area would most likely be drained within 30 minutes or less after the rainfall. Computation of the amount of storage needed would be similar to the analysis used in designing detention basins on ground surfaces.

Another type of stormwater detention on parking lots consists of using the paved areas of the lot to channel the runoff to grassed areas or gravel-filled seepage pits. The flow then infiltrates into the ground. Soil conditions and the effects of siltation in reducing infiltration must be considered.

Minimum slopes of 1.0 percent are recommended in parking lot detention areas. Maximum slopes should not exceed 4 percent to avoid gasoline spillage from tanks and to minimize vehicle traction problems on icy paving.

The use of parking lots for the storage of rainfall, if done properly, can be accomplished without inconvenience to the parking lot users and without interfering with the layout and functioning of the parking lot. Furthermore, controlled rates of release are generally large enough so that water is ponded only during the larger storms.

Recreational Areas

Recreational areas generally have a substantial area of grass cover which often has a high infiltration rate. Storm runoff from such fields is generally minimal. The best use of such recreational fields can be made by providing for the ponding of runoff from adjacent areas. Grassed recreational fields can be utilized for the temporary detention of the storm runoff without adversely affecting their primary function.

Most urban areas contain parks, both the neighborhood type and the large central type. Parks, like recreational fields, create little runoff of their own; however, parks provide excellent detention storage potential for the storage of runoff from adjacent areas.

Road Embankments

The temporary detention of storm runoff behind road embankments is an acceptable practice. The reader is referred to Sections 5 and 6 of this Manual dealing with inlets and culverts for design criteria relative to the use of ponding behind road embankments. It is, of course, necessary to

review the damage potential to upstream property, and to the embankment.

The design criteria to be used for the temporary detention of water behind road embankments should include consideration of the major storm runoff, i.e., the runoff to be expected once each 100 years.

On-Site Ponds

The construction of on-site ponds which have recreational benefits provides significant detention benefits when properly planned and designed. The use of such ponds is particularly encouraged in planned unit developments where large areas of grass and open space are common.

Controlled outlets for the surcharge storage can be used, and it is suggested that such outlets be designed to release at a rate that does not exceed the rate estimated for natural conditions for the 10-year storm.

Combinations

In many instances, one on-site detention method cannot conveniently or economically satisfy the required amount of stormwater storage. Limitations in storage capacities, site development conditions, soils limitations, and other related constraints may require that more than one method be utilized. For example, rooftop, parking lot, and surface pond storage might all be required to compensate for increases in runoff due to development of a particular site. Whichever combinations are suitable should be incorporated into the site development plan.

9.20 Design Criteria

Design Storms

Design storms shall be as outlined in Table 9-1.

TABLE 9-1 DESIGN STORM CRITERIA

<u>Classification</u>	<u>Principal Spillway Storm</u>	<u>Emergency Spillway Storm</u>
Class 1 - All embankment type structures where failure may cause loss of life, serious damage to homes, industrial and commercial facilities, public utilities and railroads.	50-yr, 6-hr. (4.6 in.)	0.40 PMP*-, (10.6 in.)
Class 2 - All other structures not class 1 such as rooftops, parking lots and small detention ponds.	10-yr, 6-hr. (3.4 in.)	100-yr., 6 hr. (5.1 in.)

*PMP is the probable maximum storm which is equal to 26.4 inches of rainfall for Dodge City, Kansas.

The principal spillway storm (or in the case of the Modified Rational

Method, the 10-yr or 50-yr. storm of the critical duration) shall be routed through the storage area with a maximum release rate which does not exceed the peak discharge for the same storm under natural conditions. The routing shall begin at the elevation of the lowest ungated outlet. The structure shall be sized to handle this storm without any emergency spillway flow.

The emergency spillway storm is then routed through the storage area with a starting water surface elevation equal to the elevation of the lowest ungated outlet. Site conditions will establish whether a riser pipe or excavated spillway is selected to handle the emergency spillway storm. The emergency spillway shall be designed to handle the emergency spillway storm without causing overtopping and failure of the storage structure.

Procedures for developing the design storms shall be those outlined in Section 2.

Principal Outlets

Either corrugated metal or reinforced concrete pipes may be utilized for the principal outlet. The minimum pipe diameter shall be 18 inches. The principal outlet shall be able to completely drain the storage area in a reasonable amount of time.

To prevent the formation of vortices at the inlet, a hood or anti-vortex baffle shall be installed. Vortices can significantly reduce the discharge for a given headwater because of energy losses.

Depending on the geometry of the outlet structure (either drop-inlet riser or hood-inlet pipe) discharge for various headwater depths can be controlled by the inlet crest (weir control), the riser or barrel opening (orifice control), or the riser or barrel pipe (pipe control). Each of these flow controls shall be evaluated when determining the rating curve of the principal outlet. In general, the riser pipe diameter is at least 6 inches greater than the barrel pipe diameter. Therefore, the minimum riser pipe diameter shall be 24 inches.

Weir flow may be computed by the following equation:

$$Q = CLH^{1.5} \quad (9-1)$$

where:

Q = discharge, in cfs;

C = weir coefficient. For riser pipes, C = 3.1 may be used;

L = length of the weir, in feet. For circular riser pipes, L is the pipe circumference; and

H = the depth of flow over the pipe crest, in feet.

Orifice flow may be computed by the following equation:

$$Q = CA(2gH)^{0.5} \quad (9-2)$$

where:

C = orifice coefficient. For sharp-edged orifices, C = 0.6;
A = cross-sectional area of the pipe, in ft²;
g = gravitational acceleration, 32.2 ft/sec²; and
H = head above the centerline of the pipe, in feet.

Pipe flow may be computed by the following equation:

$$Q = A \sqrt{2g(H - S_f L) / (1 + k_b + k_e)} \quad (9-3)$$

where:

H = the difference between headwater and tailwater elevations, in feet;
k_b = bend loss coefficient;
k_e = entrance loss coefficient;
S_f = friction slope, in feet per foot; and
L = length of pipe, in feet.

Emergency Spillways

The position, profile, and length of the spillway are influenced by geologic and topographic features of the site. The cross-section dimensions are governed by hydraulic elements and are determined by acceptable reservoir routing of the design storm. Spillway velocities shall be limited by the criteria listed in Table 9-2.

Provisions shall be made for additional safety by providing freeboard above the maximum water surface elevation of the emergency spillway storm of 3 feet for Class 1 structures and 2 feet for Class 2 structures. In addition the minimum elevation difference between the crest of the spillway and the top of embankment shall be 4 feet.

Discharge from emergency spillway shall be so directed that flows return to the channel without causing erosion along the downstream toe of the dam. Emergency spillways proposed for the protection of earthen embankments shall be in full cut, if possible, to avoid flows against the constructed fill. The side slopes of the excavated channel in earth shall be no steeper than 3:1 for ease of maintenance. Where the site limitations prevent a full channel cut, a wing dike shall be provided to direct spillway flows away from the downstream toe of the dam.

The configuration of the entrance channel from the reservoir to the control section of the emergency spillway shall be a smooth transition to avoid turbulent flow over the spillway crest. The outlet channel of the emergency spillway shall convey flow to the channel below the structure with a minimum of erosion. The slope of the exit channel usually follows the configuration of the abutment, unless exit velocities are excessive. A flatter slope with a widening of the bottom width, should reduce exit velocities to a level where erosion will not pose a problem. In cases of highly erodible soils it may be necessary to use other means of protection such as riprap, grouted

TABLE 9-2
PERMISSIBLE VELOCITIES FOR VEGETATED SPILLWAYS

	Permissible velocity feet per second			
	Erosion resistant soils		Easily erodible soils	
	Slope of exit channel Percent		Slope of exit channel Percent	
	0 to 5	5 thru 10	0 to 5	5 thru 10
Bermudagrass	8	7	6	5
Bahiagrass				
Buffalograss	7	6	5	4
Kentucky bluegrass				
Smooth brome				
Tall fescue				
Reed Canarygrass				
Sod forming grass-legume mixtures	5	4	4	3
Lespedeza sericea	3.5	3.5	2.5	2.5
Weeping lovegrass				
Yellow bluestem				
Native grass mixtures				

rock or concrete paving to form the exit channel. Especially adapted grasses provide a stabilizing effect and reduce erosion in the exit channel. Detention storage can be increased to reduce the frequency and/or duration of use of the emergency spillway and thereby reduce erosion problems.

The outlet channel slope should be greater than critical slope for one-fourth of the peak outflow to assure that flow will be controlled at the spillway section. Otherwise, both weir control and open-channel control should be evaluated over the range of headwater depths.

Site Conditions

The geologic conditions of the site shall be investigated to determine the suitability for impoundment of surface water. Groundwater level increases downstream of the storage area are to be avoided. The intensity and detail of the geologic investigation should be consistent with the class of structure and the complexity of the site geology.

Embankments

The width of the top of the embankment shall not be less than 10 feet for embankments less than 20 feet in height or less than 12 feet for embankments between 20 and 25 feet in height. Special consideration must be given to width for embankments used as a public road. The use of guardrails and other safety measures will depend upon the amount of vehicular traffic and the authority which has responsibility for the road.

Side slopes on embankment structures will vary with materials used and shall be designed to produce a stable and easily maintained structure. Soil mechanics and slope stability analysis shall be required on all Class I structures. A guide to the steepest allowable slopes is presented in Table 9-3.

TABLE 9-3 ALLOWABLE SLOPE EMBANKMENTS

<u>Height of Structure (feet)</u>	<u>Upstream Slope</u>	<u>Downstream Slope</u>
Less than 12	3:1	2:1
12 or greater	3:1	2½:1

An allowance for settlement shall be made for embankment structures.

Consideration shall be given to preventing seepage through the embankment and foundation from causing embankment erosion and structure instability for all Class I structures.

Cutoff Collars

All pipes through material subject to saturation within the embankment shall be fitted with cutoff collars. The collars shall be of sufficient size and number to increase the length of the seepage path by at least 15 percent. The collars shall be spaced at no more than 25-foot centers. Generally, the collars project a minimum of two feet beyond the outside of the pipe and should not be closer than two feet from a field joint. The number of

collars can be determined from the following equation.

$$N = \frac{(25+2.5h)(.15)}{2p}$$

N = number of collars

h = maximum height of embankment, feet

p = projected height of collar

The saturation zone is defined as $25+2.5h$.

9.30 Hydraulic Design Methods

The two methods suggested for predicting the volume of runoff over time and the peak flow are the Modified Rational Method and the SCS Unit Hydrograph Procedure.

Modified Rational Method Analysis

The term Modified Rational Method Analysis refers to a procedure for manipulating the basic Rational Method to reflect the fact that storms with durations greater than the normal time of concentration for a basin will result in a larger volume of runoff even though the peak discharge is reduced. This greater volume of runoff produced by longer storm durations must be analyzed to determine the correct sizing for detention facilities.

The approach becomes more valid on progressively smaller basins, eventually reaching a size so small that watershed modeling is approached. The procedure should, therefore, be limited to relatively small areas such as rooftops, parking lots, or other upstream areas with tributary basins less than 20 acres. This would minimize major damage which could result from overtopping or failure of the proposed detention facility.

Figure 9-1, Modified Rational Method Hydrographs, presents a family of curves for a theoretical basin. These hydrographs are developed by using the basic Rational Method assumptions of constant rainfall intensity, time of concentration for the longest flow path, and the coefficient of runoff. The typical Rational Method hydrograph with the peak discharge coinciding with the time of concentration for the basin (T_c) is first calculated using the normal formula $Q = CiA$. Following this, a family of hydrographs representing storms of greater duration are developed. The peak runoff rate for each hydrograph is equal to CiA where i is the rainfall intensity for the storm duration in question. The rising limb and falling limb of the hydrograph are, in each case, equal to T_c for the basin. The area under the hydrograph is also equal to the peak discharge rate for that particular rainfall multiplied by the duration of the rainfall.

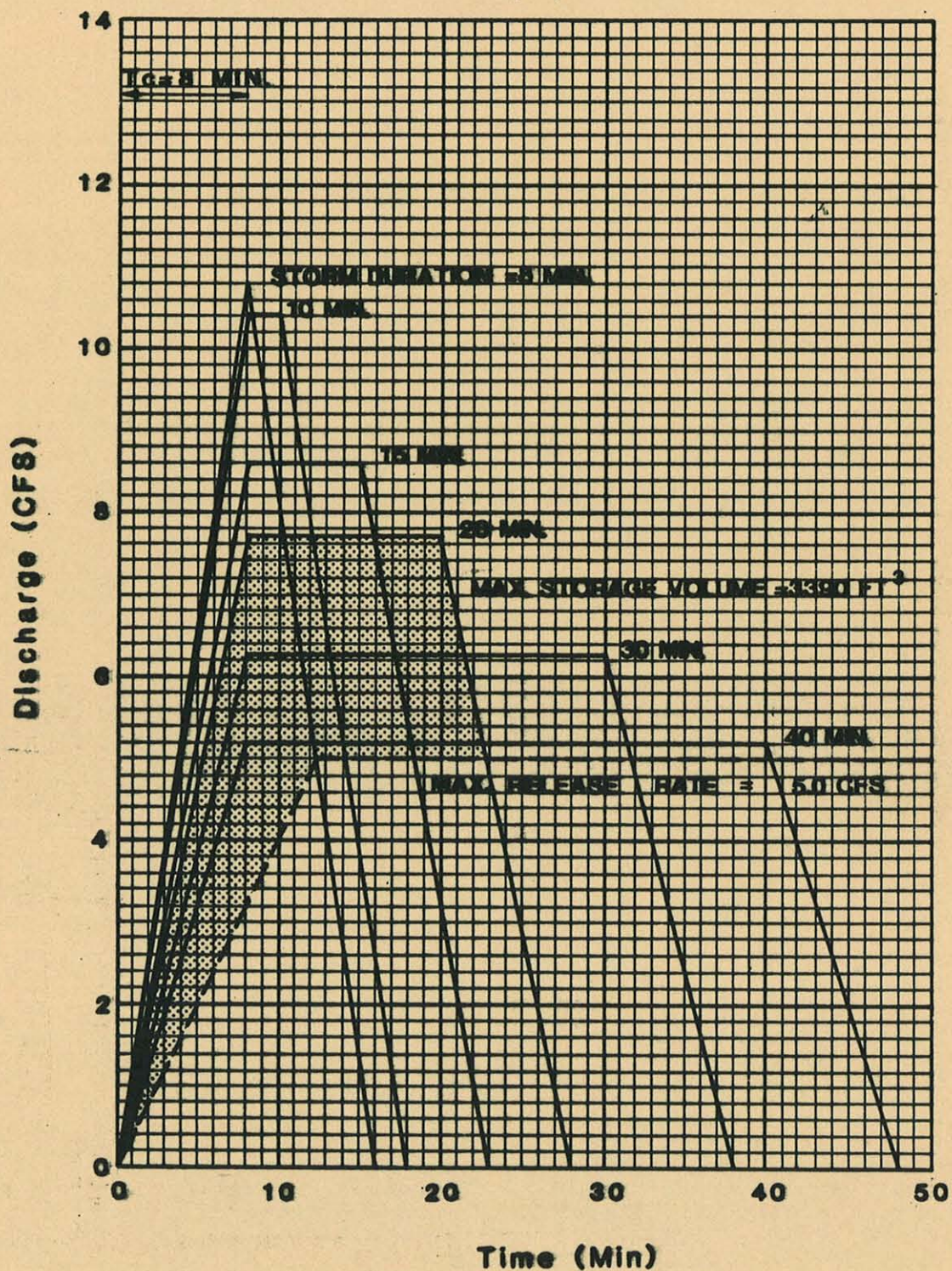


Figure 9-1 Modified Rational Method Hydrographs

The following example presents the calculation method for a typical two-acre basin.

Example 1

Given: Area: $A = 2.0$ acres
Type of development: commercial parking lot, fully paved, $C = 0.9$
Time of Concentration: $T_C = 8$ minutes
Design Frequency = 10 years
Use Rainfall-Intensity Duration Curves in Figure 2-1.

Required: Develop family of curves representing Modified Rational Method hydrographs for the 8-, 10-, 15-, 20-, 30- and 40-minute rainfall durations.

Rainfall Duration (min)	Rainfall Intensity (in./hr)	Peak Runoff Rate (cfs)
8	6.0	10.8
10	5.8	10.4
15	4.8	8.6
20	4.3	7.7
30	3.5	6.3
40	2.9	5.2

Answer: The resulting storm hydrographs are depicted in Figure 9-1.

It is recommended that a coefficient be added to the Rational Method to account for antecedent precipitation conditions for major storms with recurrence intervals greater than 10 years. These coefficients are presented in Table 2-3. Under these conditions, the Rational Formula becomes $Q = C_f C_i A$. Although this approach does not totally reconcile the difficulties in representing volume of runoff by the Rational Method, it does attempt to predict more realistic hydrograph volumes characteristic of high-frequency storms.

The next step in determining the necessary storage volume for the detention facility is to set a release rate and determine the volume of storage necessary to accomplish this release rate.

To determine the storage volume required, a reservoir routing procedure should be accomplished for each of the hydrographs, with the critical storm duration and required volume being determined. The importance of the particular project should govern the type of routing utilized. For small areas requiring repetitive calculations, such as in bays of a parking lot, an assumed release curve is normally satisfactory. For larger areas, such as a pond in a small park with 20 acres or more of tributary area, a reservoir routing procedure would be in order.

Figure 9-1, Modified Rational Method Hydrographs, represents a method for small area detention analyses. The assumed release curve approximates a formal reservoir routing in much the same way the Rational Method Hydrograph approximates a true storm hydrograph. The curve allows for the low release

rate at the beginning of a storm and an increasing release rate as the storage volume increases.

In normal flood routing, the maximum release rate will always occur at the point where the outflow hydrograph crosses the receding limb of the inflow hydrograph. For this reason, the design release rate is forced to coincide with that point on the falling limb of the hydrograph resulting from the storm of duration equal to the time of concentration for the basin. The release rate is held constant past this point. The storage volume is then found by determining the area between the inflow and release hydrographs. Example 2 continues the calculations initiated in Example 1 to determine the required storage volume.

Example 2

Given: Drainage basin and other hydrologic information presented in Example 1.

Allowable release rate: $Q_0 = 5.0$ cfs

Required: Determine the critical storage volume.

Storm Duration (min)	Required Storage Volume (ft ³)
8	2780
10	3240
15	3315
20	3390
30	2640
40	1005

The critical storage volume is then 3,390 ft³ occurring for a 20-minute rainfall duration.

The limitations in the assumptions behind this method are evident. The approach becomes more valid on progressively smaller basins. The procedures should, therefore, be limited to relatively small areas where no major damage would result from over-topping or failure of the proposed detention facility. Care should be used when applying this method to areas in excess of 20 acres.

The SCS Unit Hydrograph Procedure for Storage Analysis

The SCS Unit Hydrograph Procedure develops a hydrograph which provides a reliable solution for detention storage effects. Procedures provide the designer greater flexibility for the representation of actual conditions to be modeled.

The development of the runoff hydrograph is presented in Section 2 of this Manual. A flood routing procedure may be used to determine required volume

of the detention basin. Several flood routing procedures are available in published texts. The data needed for the routing computations are the inflow hydrograph, the physical dimensions of the storage basin, the maximum outflow allowed, and the hydraulic characteristics of the outlet structure or spillway. This method is referred to as the Modified Puls routing procedure.

After the inflow hydrograph, depth-storage relationship, and depth-outflow relationship have been determined, they are combined in a routing routine. The results of the routing are the ordinates of the outflow hydrograph, the depths of storage, and the volumes of storage at each point in time of the flood duration.

The routing period, or time interval, Δt , is selected small enough so that there is a good definition of the hydrograph and the variation in the hydrograph during the period Δt is approximately linear. This can be accomplished by setting $\Delta t = 5$ minutes, or the same time interval as in the SCS Unit Hydrograph procedure.

Several assumptions are made in this procedure and include the following:

1. The entire inflow hydrograph is known.
2. The storage volume is known at the beginning of the routing.
3. The outflow rate is known at the beginning of the routing.
4. The outlet structures are such that the outflow is uncontrolled and the outflow rate is dependent only on the headwater.

The derivation of the routing equation begins with the conservation of mass which states that the difference between the average inflow and average outflow during some time period Δt , is equal to the change in storage during that time period. This can be written in equation form

as:

$$\bar{I} - \bar{O} = \Delta S / \Delta t \quad (9-5)$$

If inflow during the period is greater than outflow, then ΔS is positive and the pond gets deeper. If inflow is less than outflow during the period, then ΔS is negative and the pond gets shallower. Using the assumptions made previously, this equation can be rewritten as:

$$\frac{I_1 + I_2}{2} - \frac{O_1 + O_2}{2} = \frac{S_2 - S_1}{\Delta t} \quad (9-6)$$

Multiplying both sides by two and separating the right-hand side yields:

$$I_1 + I_2 - O_1 - O_2 = \frac{2S_2}{\Delta t} - \frac{2S_1}{\Delta t} \quad (9-7)$$

Rearranging so that all the known terms are on the left-hand side and all the unknown terms on the right-hand side yields the final routing equation:

$$(I_1 + I_2) + \frac{(2S_1 - O_1)}{\Delta t} = \frac{(2S_2 + O_2)}{\Delta t} \quad (9-8)$$

However, the last equation has two unknowns, S_2 and O_2 . We need a second equation which relates storage and outflow. If outflow is a direct function of reservoir depth (as it is with uncontrolled outflow), there is a direct relationship that exists between reservoir elevation, reservoir storage, and outflow. Therefore, for a particular elevation, we have an answer for storage and outflow (S and O). A relationship between O and $(2S/\Delta t) + O$ is determined for several elevations and plotted on logarithmic graph paper. The routing equation is solved by adding all the known terms on the left-hand side. This yields a value for $(2S/\Delta t) + O$. This value is found on the log-log plot of $(2S/\Delta t) + O$ versus O and the value of O_2 is obtained from the graph.

The $(2S/\Delta t) + O$ versus O relationship is derived by combining the depth-storage relationship and the depth-outflow relationship, as previously discussed. This is shown in Table 9-4. Columns 1, 2, and 3 are tabulations of the depth-storage and depth-outflow relationships for a specific detention facility.

In column 4, the units of $2S/\Delta t$ and O must be the same. If O is in cfs then $2S/\Delta t$ must be changed to cfs. For a routing time interval of 5 minutes:

$$\frac{2S \text{ ac-ft}}{5 \text{ min}} \times \frac{1 \text{ cfs-day}}{1.98 \text{ ac-ft}} \times \frac{1440 \text{ min}}{1 \text{ day}} = 291S$$

$$\text{Thus, } \frac{2S + O}{\Delta t} = 291S + O \text{ for } \Delta t = 5 \text{ min.}$$

where: S has units of acre-feet, O has units of cfs, and $2S/\Delta t$ has units of cfs.

TABLE 9-4

Development of a $(2S/\Delta t) + O$ vs. O Relationship

Depth (ft) (1)	Storage, S (ac-ft) (2)	Outflow, O (cfs) (3)	$(2S/\Delta t) + O$ (cfs) (4)
0	0.0	0	0
2	0.1	40	69
4	0.6	138	313
6	3.0	274	1,147
8	11.0	426	3,627
10	32.0	560	9,872
12	72.0	671	21,623
14	131.0	765	38,886

The sizing of the outlet works for detention ponding is a matter of judgment depending upon the actual conditions for the specific case. The designer may approach the sizing of the outlet works on a trial- and-error basis with the objective being the optimum use of the available storage capacity of the ponding area. In addition all ponding areas must be carefully analyzed in regard to the major storm runoff conditions. In many cases it will be found that the initial storm runoff should be routed through the outlet works with only minimal ponding. The storage would then be utilized to reduce the major runoff. If other provisions are made for the major runoff the designer may be primarily concerned with the initial drainage system. The outlet capacity would then be substantially less than the inflow from the initial runoff. It cannot be overemphasized, however, that the use of downstream channel storage requires competent planning and design to avoid the creation of an unnecessary hazard.

9.40 Bibliography

1. City of Austin, Texas, Engineering Department et al., Drainage Criteria Manual, Austin, TX, 1977.
2. Division of Water Resources, Kansas State Board of Agriculture, Engineering Guide-1, EG-1, October 19, 1982.
3. Division of Water Resources, Kansas State Board of Agriculture, Engineering Guide-2, EG-2, November 1, 1983.
4. Henningson, Durham & Richardson, P.S., City of Yakima, Washington, Stormwater Drainage Control Manual, Yakima, WA, 1982.
5. Poertner, Herbert G., Practices in Detention of Urban Stormwater Runoff, American Public Works Association Special Report No. 43. Chicago, IL, 1974.
6. Ward, Andy; Haan, Tom; and Tapp, John, The DEPOSITS Sedimentation Pond Design Manual, University of Kentucky Institute for Mining and Minerals Research, Lexington, KY, 1979.

7. Wright-McLaughlin Engineering, Urban Storm Drainage Criteria Manual,
Denver Regional Council of Governments, CO, 1969.

Section 10

Flood Proofing

- 10.10 Flood-Proofing Requirements
- 10.20 Types of Flood Proofing
- 10.30 Procedures
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 - Elevated Structures
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Section 10

Flood Proofing

Flood proofing is defined by Federal Insurance Administration as any combination of structural and nonstructural additions, changes or adjustments to structures which reduce or eliminate flood damage to real estate or improved real property and sanitary facilities, structures, and their contents. The Federal Insurance Administration has published several references to provide detailed criteria and design procedures for flood-proofing structures. These references are listed in the bibliography of this Section and should be consulted for detailed design information.

10.10 Flood-Proofing Requirements

Any proposed non-residential structures located within the 100-year floodplain should either have the lowest floor elevated to one foot above the 100-year flood elevation or should be flood-proofed to that elevation. The 100-year floodplain and corresponding elevations of the 100-year flood can be determined by reviewing the Flood Hazard Boundary Maps and Flood Insurance Rate Maps published by the Federal Insurance Administration and available for review in the office of the appropriate agency. Flood plain regulations which further define flood-proofing requirements can be found in Part 5, Article 4, of the Dodge City - Ford County Zoning Regulations.

10.20 Types of Flood Proofing

There are two basic systems for flood proofing for residential and commercial structures. One system consists of the use of conventional construction methods along with a total drain, sump, and pump operation to keep water away from the lower portions of the structure. This type of system is feasible in regions with soils having low coefficients of permeability (clay soils). This system is termed the Drain or Sump System and is the more economical method. This system accepts the inevitability of some infiltration from a high head of water into a reasonably economical basement construction via the sump. The sump type basement protection system is adaptable to low and medium volumes of inflow of flood waters within the capability of its drainage and pumping system. This system is only applicable to structures having a grade line above the 100-year flood elevation on all sides.

The other system utilizes a watertight wall and slab treatment for all portions of the structure beneath the 100-year flood elevation. This type of system is required for soils having a high coefficient of permeability (sandy soils). This is termed an Undrained or Barge System and is required where inflow is beyond the capacity of a sump drain system or where the grade line of the proposed structure cannot be located above the 100-year flood elevation. An undrained structure is designed to be watertight. All openings below the 100-year flood elevation are equipped with flood-proofed, watertight closures.

Both these systems will allow flood proofing of structures up to 5 feet above the bottom of the lowest floor slab. This height restriction is due mainly to buoyancy considerations. Special design features will allow flood proofing to higher elevations.

10.30 Procedures

Several general procedures to achieve flood proofing are discussed in this Section. Some of these procedures, such as laying out sites and raising buildings, are intended primarily for new construction which would represent a proper use of floodplain sites. Other procedures such as those to keep the water out or those to minimize losses if the water gets in, would apply to both new and existing structures.

Site Layout

The practice of clustering buildings is prevalent in planned unit developments. This clustering permits buildings to be attractively grouped on parts of a site which are above flood levels and reserves the low-lying sections as landscaped green areas and parking facilities.

Where natural high ground does not exist, sites can be raised by filling, providing the fill does not interfere with the flow of flood waters (i.e., the structure is not located within the "floodway").

Flood-proofing measures can be designed to blend with the overall appearance of a structure. When this is done, a structure's appearance can be preserved and in some cases even enhanced by flood-proofing.

Elevated Structures

The practice of elevating a building on "stilts" to provide an "open" effect at ground level can also reduce the flood hazard. If some means of access is maintained and utilities can continue to function, activities would not be interrupted during floods.

Where land is at a premium, as in central business districts, buildings are often placed on stilts with parking facilities on the ground level.

Waterproofing Structures

The design techniques discussed above either achieve flood-proofing through site planning and development or incorporate flood-proofing in the initial construction of the buildings. The location and environment of structures in urban areas may make these solutions impractical. In these circumstances the building owner, architect, or engineer is faced with a job of designing flood-proofing measures for existing conventional buildings which are exposed to flood water. These flood proofed buildings can incorporate many contemporary design features such as large window areas, pedestrian arcades, open floor space, and curtain wall panels.

In designing new structures, or in altering existing ones, thought should be given to the use of recessed flood shields which are normally hidden from view, but can be easily lowered or slid into place upon the receipt of a flood warning. Such shields also escape the danger of being misplaced.

When flood shields must be mounted on the street side of an opening, the brackets to which they would be bolted can be concealed with easily removed aluminum strips or "skins."

Internal Flood-Proofing Measures

Owners of buildings which are subject to flood but cannot be easily altered to keep the water out can consider the use of water-resistant construction materials to reduce flood damage. Even owners of flood-proofed structures generally able to withstand flooding would also be well advised to consider the use of such materials to reduce losses when flooding exceeds the protection level.

10.40 Engineering Aspects

Flood-proofing to keep the water out of buildings falls, in part, within the province of the structural engineer. When flood waters surround a building they impose loads on the structure and substructure beyond those it normally is designed to withstand. A determination of these loads is a prerequisite of flood-proofing efforts.

The following paragraphs discuss some of the more common structural problems that could be encountered. Because of the complexity of these problems, building owners who are contemplating flood-proofing should engage the service of a professional engineer who has a working knowledge of structures and who has had experience with hydraulic structures or flood-proofing. This is necessary to insure that the flood-proofing does not worsen the problem by creating structural damages such as ruptured walls and floors in addition to the damages resulting from water contact and disruption.

Structural Problems

The forces which would act upon a typical building under conditions varying from normal (nonflood) to partial submergence (flood with and without subsurface or foundation drainage) are graphically presented in Figures 10-1, 10-2, and 10-3. The building cross-section shown is considered representative of that which would be commonly encountered in a flood-proofing program.

Loading from Structure and Contents

The weight of the building itself (masonry, concrete, steel, wood, etc.), known as dead load, together with the weight of its live load (furniture, machinery, merchandise, occupants, etc.) will normally be transmitted through the roof and floor systems to supporting columns and walls and thence to the foundations. These loads generally are transmitted directly to the supporting soil or bedrock under the foundation. Loads of this type will normally be unaffected by flooding and will have the same value for both flood and nonflood conditions.

Restraint from Floor and Roof Systems

Flooding produces large lateral forces on the structure. These forces will be resisted by the building walls, floor, and roof systems. Many commercial and industrial buildings are designed and constructed in a manner to provide adequate connection and anchorage between these systems for support and structural unity, but each building must be individually evaluated and strengthened where necessary.

Most residential and many light commercial and industrial buildings, however, do not have the necessary anchorage and would require modification to provide it. This would involve adequate transverse bridging in addition to anchorage into the walls around the entire perimeter. Steel angles bolted into both the floor system and the walls at their juncture would be one method of anchorage.

Resultant of Nonflood and Flood Loading

The nonflood loading is the force exerted by the soil backfill upon the wall. These pressures depend upon the physical characteristics of the soil particles, the degree of compaction, the moisture content, and the movement of the wall caused by the backfill and foundation deformation, if any. Where the top of the zone of saturation (water table) is at an elevation above the base of the foundation, the pressure on the wall and floor slab is due to the buoyant weight of the soil plus the full hydrostatic pressure of the water. When a workable subdrainage system is provided to lower the elevation of the water table, the pressure on the wall will be reduced. The degree to which the water table can be lowered will depend upon the permeability of the soil and the efficiency of the subdrainage system.

Flood loading without subdrainage is the force of the full hydrostatic pressure of the water above as well as below the ground line plus the buoyant weight of the soil. As schematically illustrated in Figure 10-2, the magnitude of this force can be considerably larger than the force developed under nonflood conditions (shown in Figure 10-1 for comparison).

When subdrainage is provided, the flood loading is reduced. However, in the case of an existing building with an existing unmodified subdrainage system, prudence would dictate that no load reduction be assumed. Subdrains, where already installed, are generally provided only to intercept seepage and control uplift on basement floors due to groundwater. If such a subdrainage system were to be modified to attain a known degree of effectiveness, a load reduction could be determined. Obviously, for new construction the subdrainage system can be designed and constructed to afford a predetermined degree of reduction of flood loads.

The magnitude of the flood-induced forces that will be encountered is indicated by the fact that a one-story brick building (3-5/8 inches of brick over wood frame) can be expected to withstand no more than two feet of water above the ground line providing the wall is in good condition. For brick with concrete-block backup, this height would be somewhat greater.

Subsurface Drainage

Groundwater conditions may adversely affect the stability of a building or structure either through uplift which tends to "float" the building or by erosion which can undermine the support. Investigation and analysis of the factors involved at any specific building and the design of control or corrective measures are endeavors requiring the attention of a professional engineer.

Groundwater problems can be controlled by the installation of subdrainage systems (Figures 10-1 and 10-3) to reduce the lateral forces on the

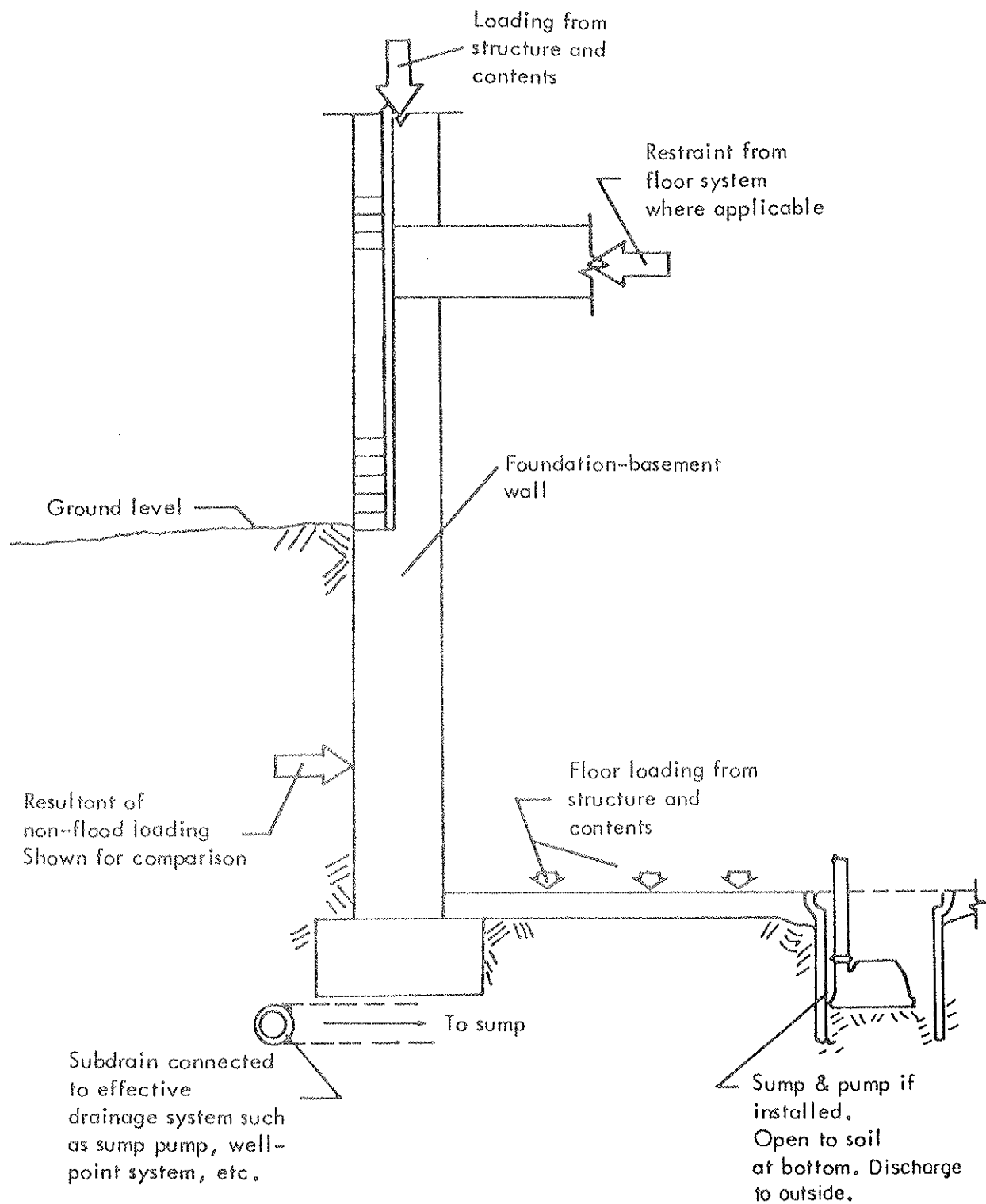


Figure 10-1 Building Loads Under Nonflood Conditions

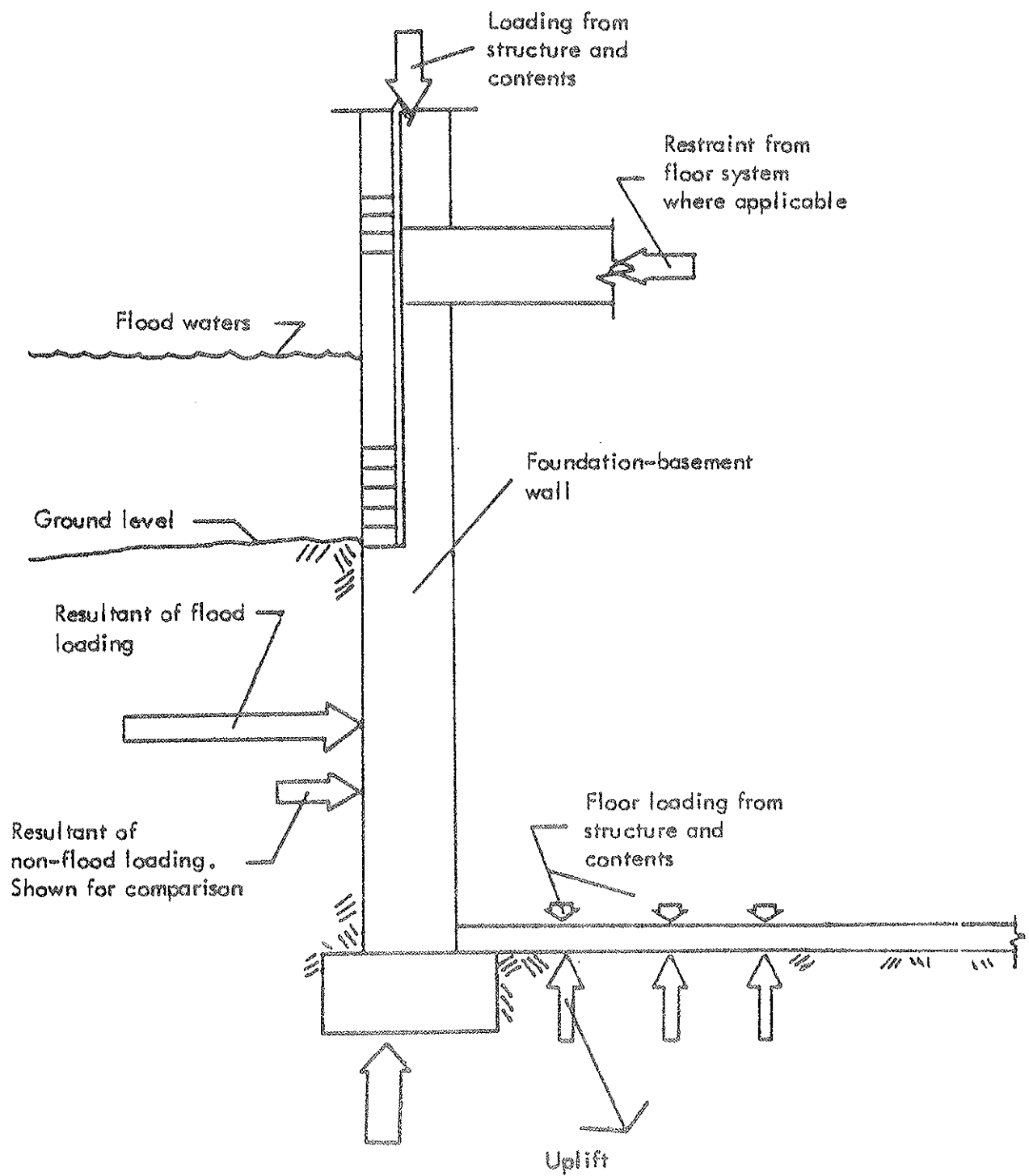


Figure 10-2 Building Loads Without Subsurface Drainage

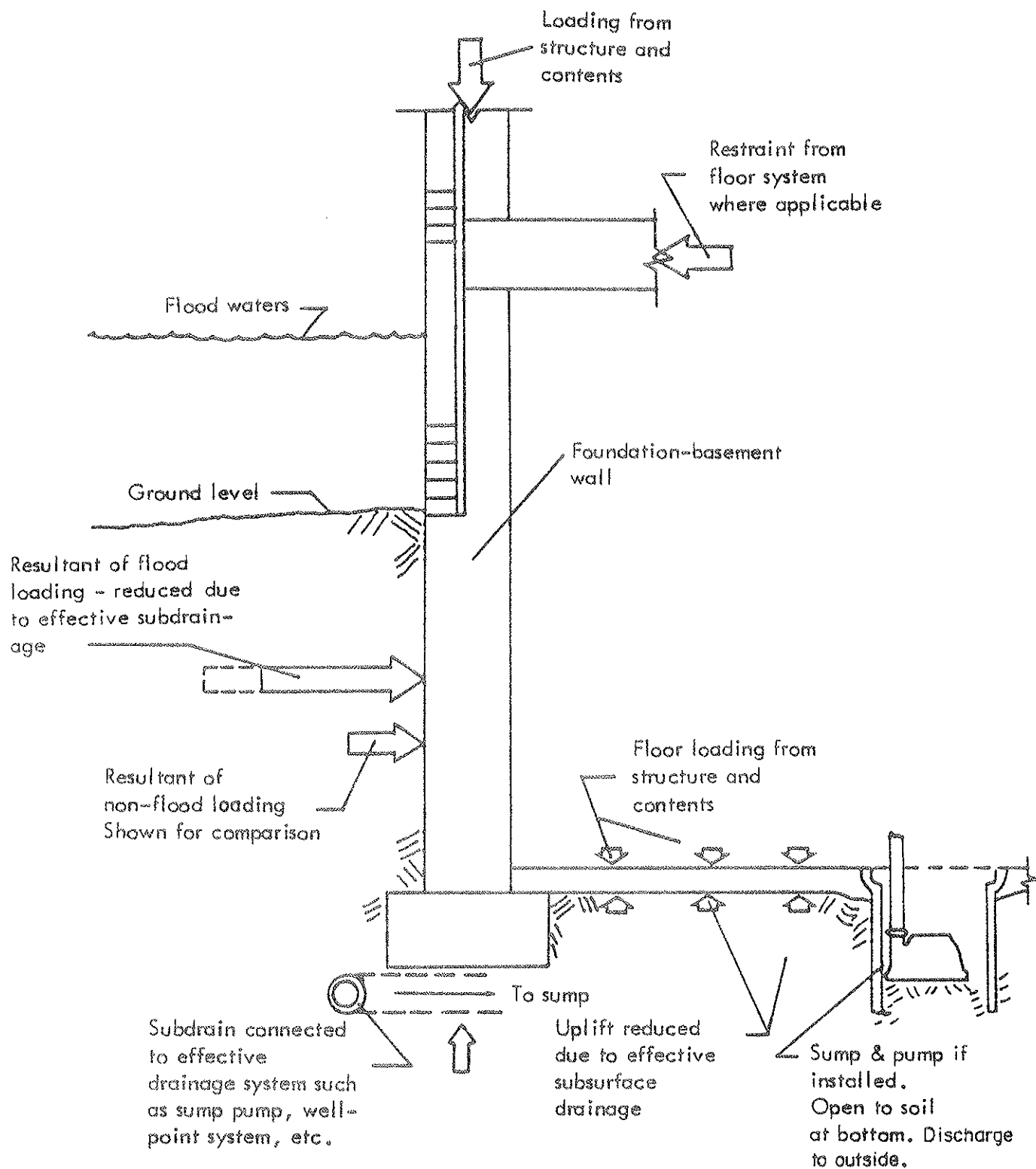


Figure 10-3 Building Loads with Subsurface Drainage

foundation walls and floor slabs. Experience has shown that the composition of soils in a particular area can vary widely with extreme ranges of permeability existing in areas of similar geological origin. Such ranges in permeability call for careful investigation and analysis. The design of a subsurface drainage system must be based on the results of soil investigations of permeability and analyses of structural strength.

A sump and pump system can be employed to help protect the subsurface part of a building. The pump could be designed to accept storm and seepage flows and pump them to a point above the flood waters. The sump should be open to the soil at the bottom and to atmospheric pressure at the top within the basement. This would provide a fail-safe feature, in that power or pump failure would allow water to flood the basement and thereby tend to balance the outside flood-induced pressures upon the basement walls and floor slab. As an alternative, a prearranged program of deliberate flooding with clean water could be employed to minimize the cost of clean up after a flood.

Seepage Control

Foundation walls can be made watertight to minimize water infiltration through cracks and crevices in the walls. In buildings under construction, this can be accomplished through the use of waterproof membranes and seals. Construction joints can be protected by the use of a neoprene or other similar waterstop. Existing masonry or stone foundations are more difficult to waterproof, particularly if the mortar joints have deteriorated with age. Sealing of walls to prevent seepage can be accomplished in many cases by coating them, preferably on the exterior, with hydraulic cement, epoxy paint, or other similar waterproofing materials.

It must be recognized that sealing and waterproofing of walls increases the hydraulic forces acting on the walls unless the drainage through the walls which is afforded by the cracks and crevices prior to sealing is provided by other means. Sometimes the wisest course would be to permit the seepage through the wall and then control it by a floor drain and sump pump. Existing cracks and leaks in walls sometimes can be the most practical form of drainage to relieve pressure. In some cases this drainage can be supplemented by holes drilled through the walls. Structural and hydraulic analyses of alternative designs and associated cost estimates will enable the designer to choose the most suitable means of controlling seepage at a given building.

Sewage Backup

Most existing subdrains, whether connected to sewerage systems or not, are subject to backflow and high pressures during floods. Since these high pressures could burst the usually encountered clay pipe subdrains and endanger basement walls and floors, some device such as a gate valve must be provided for isolating the subdrains around the building from these high pressures.

There are several alternative methods for controlling backflow through sewers. One method would be to install a main valve at a location where the sewer is strong enough to resist the flood-induced pressure and where all possible reverse flows can be stopped. See locations "A" and "B" in Figure 10-4. This valve should be designed to accommodate grit and other

materials which could lodge in it.

If the pipe is of sufficient strength, an alternative would be to install separate valves on all basement fixtures and floor drains (Figure 10-4). These valves could be inflatable rubber plugs or a similar type of mechanically expandable rubber plug. Valves designed for low pressure (20 psi or less) could be installed in drain lines of fixtures which are below design water levels. In either of the above alternatives, it would be necessary to provide adequate sump pumps to handle any leakage.

Figure 10-5 presents another alternative for controlling sewer backup. This alternative provides for conveying all floor drainage, appliance drainage, drain-tile flow, and any seepage that might enter the building to a sump pump. The pump would lift the drainage up to an elevation above the design flood on a permanent basis. By thus eliminating all gravity sewer drains, the problem of flooding backflow can be eliminated and a subsurface area permitted to function during floods.

Structural Engineering

The highly technical and thorough nature of the investigations and analyses required in the design of effective, safe, and reliable flood-proofing measures for both new and existing facilities cannot be overstressed. Construction or modification of subdrainage systems without such investigation and analysis can result in a situation potentially more dangerous to life and property than no flood-proofing program at all.

The large number of factors and the potential magnitude of the forces involved make it impossible to design flood-proofing measures by intuition. Such an approach can lead to loss of property and even life during a flood.

10.50 Flood-proofing Operations

Any individual or organization undertaking a contingent or emergency protection program must have a standard operating procedure to carry out the flood-proofing measures when the need arises. Some buildings can be secured in a short time while others may take considerably longer.

The flood-proofing system should be designed so that it may be put into operation as quickly and as simply as possible. Flood shields, doors and hatches may have to be handled during the most adverse weather conditions (perhaps during the storm which causes the flood), so lightweight metals should be used wherever possible.

Flood-proofing items including bolts, gaskets, caulking, timbers, and flood shields should be stored for easy access. The larger items should be stored close to the point of insertion and in such a manner that they can be easily slid or dropped into position. One lost or improperly mounted flood shield, or the failure to isolate a sewer can defeat the best flood-proofing system.

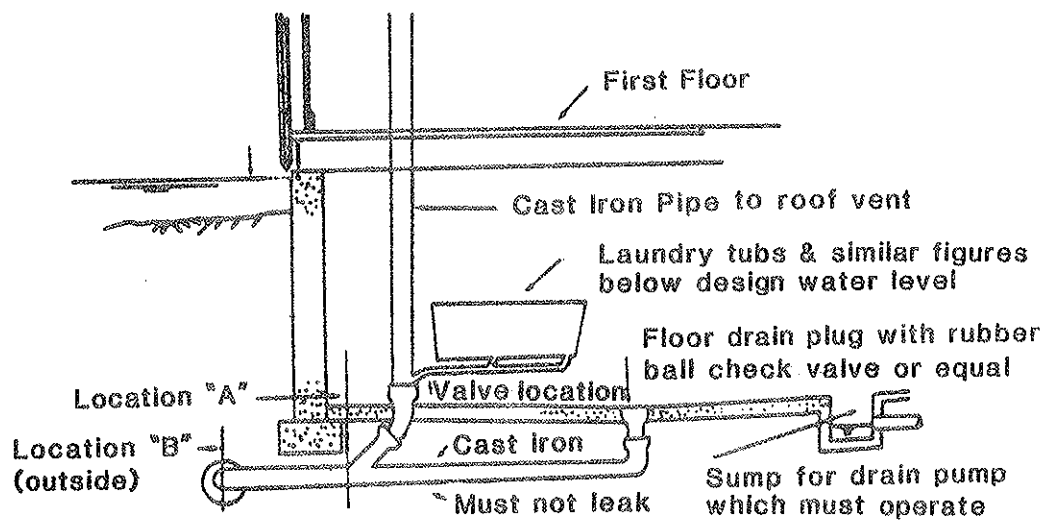


Figure 10-4 Locations for Cutoff Valves on Sewer Lines

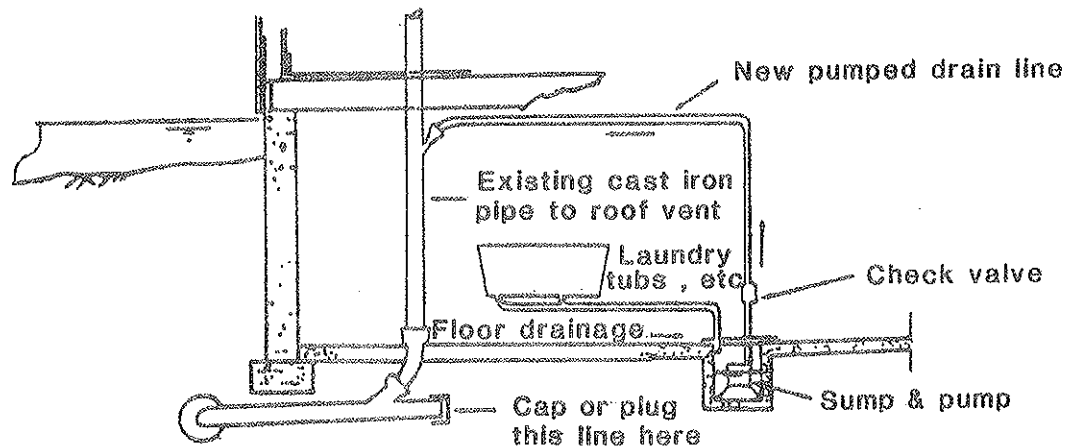


Figure 10-5 Elimination of Gravity-Flow Basement Drains

Basement Rooms

Newly designed buildings may have machinery located on upper floors but most older structures have the electrical machinery and the heating and pumping equipment on lower floors. It is these lower, subsurface floors which first experience flooding problems from seepage and sewer backup. Flood-proofing systems requiring pumps, electrical equipment, and emergency generators as an essential part of the operation must be kept in working condition throughout the crisis.

When buildings have entrances to subsurface levels from ground level, these entrances should be adjusted to prevent the entry of overland flow.

Utilities

When flood-proofing a building, provision should be made to eliminate the threat of flooding by way of gas mains, sewers, conveyor systems, and water pipes or drain tiles which enter the building. Check valves can be installed in utility pipes to protect against this source of flooding.

If sewers are to be isolated, they should be constructed of pressure pipe. Otherwise the pressure in the line could cause a rupture.

Fuses and circuit breakers servicing flooded areas should be clearly marked and easily accessible. Electrical circuits serving lower levels should be designed or modified so that they can be cut off if flooding begins. This will protect against fires and loss of life due to electrical shocks. As another precaution, valuable electrical appliances which cannot be moved should be disconnected at the unit to prevent short circuiting and damage to their power components.

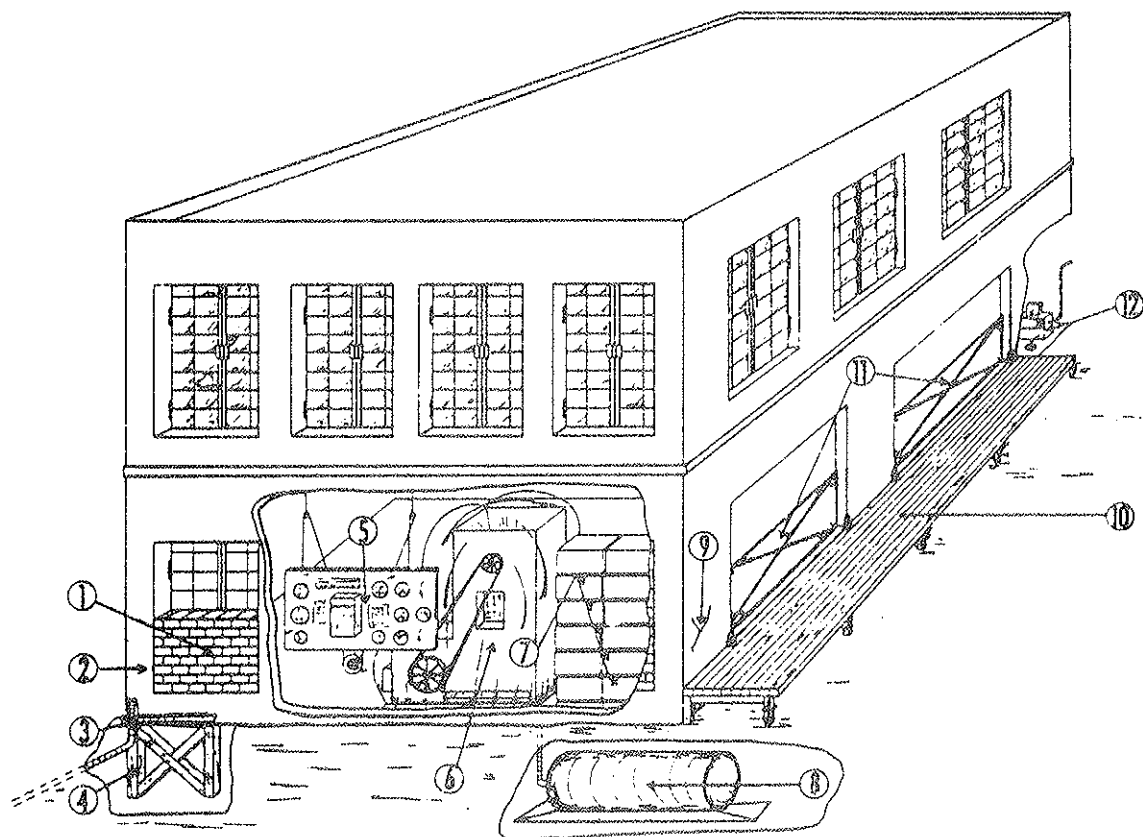
Wall Openings

Windows and vents both above and below the surface should be sealed to prevent the entry of flood waters. They may also need to be reinforced. The walls should be strong enough to support the pressures added by vent and window reinforcements. The wall should also be treated or constructed to prevent large amounts of water from passing through it.

Residential Homes

Residential construction does not lend itself readily to flood-proofing because of the extensive use of materials that do not impede the passage of water. Moreover, houses are seldom designed to withstand any significant horizontal pressures.

In most cases in which an owner has purchased a finished house in an area subject to flooding, his success with flood-proofing will depend on whether flood stages are low on his property and whether the outer walls of the structure are reasonably impervious. Under these conditions flood shields can be designed to restrict the entry of water through openings in the walls, providing the walls are strong enough to resist flood-induced pressures. An effective flood-proofing program must also include measures to cope with sewer backup and groundwater seepage.



EXPLANATION

1. Permanent closure of opening with masonry
2. Thoroseal coating to reduce seepage
3. Valve on sewer line
4. Underpinning
5. Instrument panel raised above expected flood level
6. Machinery protected with polyethylene covering
7. Strips of polyethylene between layers of cartons
8. Underground storage tank properly anchored
9. Cracks sealed with hydraulic cement
10. Rescheduling has emptied the loading dock
11. Steel bulkheads for doorways
12. Sump pump and drain to eject seepage

Figure 10-6 Flood-Proofed Building

Homeowners who have suffered severe basement flooding should consider the relocation of furnaces, hot water heaters, washers, dryers, air conditioners, freezers, refrigerators, power shop equipment, and other appliances as a permanent flood-proofing measure.

The flood-proofing of a structure is analogous in many respects to making a ship watertight and seaworthy. Flood-proofing involves not only adjustments to the foundation and substructure but also modifications of those parts of the superstructure that are below anticipated flood levels (Figure 10-6).

10.60 Bibliography

1. NAHB Research Foundation, Inc., Manual for the Construction of Residential Basements in Non-Coastal Flood Environs, Department of Housing and Urban Development Federal Insurance Administration, Washington, DC, 1977.
2. Office of the Chief of Engineers, U.S. Army, Flood-Proofing Regulations, Washington, DC, 1972.
3. Wright-McLaughlin Engineers, Urban Storm Drainage Criteria Manual, Denver Regional Council of Governments, CO, 1969.
4. Dodge City - Ford County Zoning Regulations, City of Dodge City, Kansas.

Section 11

Sediment and Erosion Control

11.10 Understanding the Problem

Erosion

Sedimentation

Sediment Damage

11.20 Erosion and Sediment Hazards Associated with Developing Areas

11.30 Principles of Reducing Erosion and Sedimentation from Developing Areas

11.40 Standards for Erosion and Sediment Control in Developing Areas

11.50 The Sediment Control Plan

Sample Requirements for Sediment Control Plans

Sediment Control Plan

Sediment Control Practices

Sediment Basins

Street Profiles

Drainage Plan

11.60 Standards for Temporary Structural Practices

Diversion Dike

Interceptor Dike

Perimeter Dike

Straw Bale Dike

Silt Fence

Interceptor Swale

Perimeter Swale

Stabilized Construction Entrance

Stone Outlet Structure

Grade Stabilization Structure (Paved Chute or Flume)

Grade Stabilization Structure (Pipe Slope Drain)

Sediment Basin

Sediment Trap

11.70 Standards for Permanent Structural Practices

Diversion

Grassed Waterway

Level Spreader

Riprap

Land Grading

11.80 Bibliography

SECTION 11

Sediment and Erosion Control

11.10 Understanding the Problem

Sedimentation involves three basic processes: erosion, transportation, and deposition. These are natural geologic phenomena that have been in continuous operation for millions of years. Man's land development activities, however, have initiated severe, highly undesirable, and damaging alterations in the natural sedimentation cycle by drastically accelerating the erosion-sedimentation process.

Erosion

The process of soil erosion by water on upland areas involves detachment from the soil mass, transportation primarily by flowing water, and eventual deposition of at least the larger particles of the sediment. Soil is detached by raindrop impact and by runoff shear forces, but man's activities often loosen and pulverize soil, thereby making it more easily moved by rainfall and runoff. Downslope transportation of soil particles is primarily by channelized runoff, although raindrop splash causes some net downslope movement and increases the transport capability of unchannelized overland flow. Significant runoff does not occur until the rainfall intensity is greater than the soil infiltration rate, but once runoff begins, the quantity and size of material that can be transported is a function of runoff velocity and turbulence. Deposition of eroded sediment may occur when the velocity or turbulence of runoff decreases significantly. Deposition is normally a quite selective process, with the largest and densest particles settling out first, and the finer particles being carried farther. Therefore, the size distribution of the eroded material is important in determining the portion of the sediment load deposited and the composition of the material remaining in the runoff. Dispersed colloidal-sized particles remain in suspension for extended periods of time.

Soil eroded from upland areas comes from locations of channelized flow called rills and from sheet erosion above and between rills. Sheet erosion is not greatly influenced by slope steepness or location on a slope. Rill erosion primarily results from the effects of runoff and is greatly influenced by slope and flowrate.

The soil type and the soil-cover conditions greatly influence soil susceptibility to detachment and transport by rainfall and runoff.

The principal effect land development activities have on the erosion process is that of exposing disturbed soils to precipitation and to surface runoff. Shaping of land for construction or development purposes alters the soil cover and the soil in many ways, often detrimentally affecting on-site drainage and runoff patterns and, eventually, off-site stream and streamflow characteristics. Protective vegetation is reduced or removed, excavations are made, topography is altered, the removed soil material is stockpiled (often without protective cover) and the physical properties of the soil are changed. The energy responsible for erosion is provided by falling rain and by the movement of surface runoff.

Sedimentation

Surface runoff moves as sheet flow or becomes concentrated in rills and gullies. It is dynamic in that it has energy to erode and transport soil particles or sediment. If the available energy is greater than the burden of the sediment load being transported, the moving water will erode the soil to obtain additional sediment; if the load is greater, deposition of some of the transported material will occur. Normally, runoff builds up rapidly to a peak and then diminishes. Excessive quantities of sediment are derived by erosion principally during the higher flows. During lower flows, as the velocity of runoff decreases, the transported materials are deposited to be picked up by later peak flows. In this way, sediments are carried downslope, or downstream, intermittently and progressively from their source.

A study of sedimentation due to highway construction and land development in Virginia indicated that 99 percent of the sediment discharge occurred during periods of high flow which took place during only 3 percent of the period of measurement.

Sediment Damage

Over four billion tons of sediment are estimated to reach the ponds, rivers, and lakes of our country each year; approximately one billion tons of this sediment is actually carried all the way to the ocean. Approximately 10 percent of this amount is contributed by erosion from land undergoing highway construction or land development. Although these latter quantities may appear to be small compared to the total, they could represent more than one-half of the sediment load carried by many streams draining small watersheds undergoing development.

Sediment yields in streams flowing from already urbanized drainage basins vary from approximately 200 to 500 tons per square mile per year. In contrast, areas undergoing urbanization often have a sediment yield of from 1,000 to 100,000 tons. It is easy to comprehend the tremendous quantity of sediment reaching our streams and rivers annually since an estimated 3,000 to 4,000 acres of land in the U.S. are undergoing development for housing, industrial sites, and highway construction every day. For very small areas, where construction activities have drastically altered or destroyed vegetative cover and the soil mantle, sediment derived from one acre of land may exceed 20,000 to 40,000 times that obtained from adjacent undeveloped woodland in an equivalent period of time.

Deposition tends to occur as the velocity of sediment-transporting streams decreases. Excessive quantities of sediment cause costly damage to water areas and to private and public lands. Obstruction of stream channels and navigable rivers by masses of deposited sediment reduces their hydraulic capacity which, in turn, causes an increase in subsequent flood crests and a consequent increase in flood damages.

Sediment fills drainage channels, especially along highways and railroads, and plugs culverts and storm drainage systems, thus necessitating frequent and costly maintenance. Municipal and industrial water supply reservoirs lose storage capacity; the usefulness of recreational impoundments is

impaired or destroyed; navigable channels must continually be dredged; and the cost of filtering muddy water preparatory to domestic or industrial use becomes excessive. The added expenses of water purification in the United States amounts to millions of dollars each year.

The general effect of fine-grained sediments, such as clays, silts, and fine sands, in an aquatic environment is to reduce drastically both the kinds and the amounts of organisms present. Sediments alter the existing aquatic environment by screening out sunlight and by changing the rate and the amount of heat radiation. Particles of silt settling on stream and lake bottoms form a blanket which creates a hostile environment for the organisms living there and literally smothers many of them and their eggs.

Coarser-grained materials also blanket bottom areas to suppress aquatic life found in these areas. Where currents are sufficiently strong to move the bedload, the abrasive action of these materials in motion accelerates channel scour and has an even more severely deleterious effect upon aquatic life. The aesthetic attraction of many streams, lakes, and reservoirs used for swimming, boating, fishing, and other activities has been seriously impaired by bank cutting and channel scour caused by higher flood stages induced by sedimentation.

11.20 Erosion and Sediment Hazards Associated with Developing Areas

The development process is such that many people may be adversely affected even by development of areas of land of only limited size. Uncontrolled erosion and sediment from these areas often cause considerable economic damage to individuals and to society in general. Surface-water pollution (streams, rivers, and lakes), channel and reservoir siltation, and damage to public facilities and private property, are some of many examples of problems caused by uncontrolled erosion and sedimentation.

Hazards associated with development include:

1. A large increase in areas exposed to storm runoff and soil erosion.
2. Increased volumes of storm runoff, accelerated soil erosion and sediment yield, and higher peak flows caused by:
 - a. Removal of existing protective vegetative cover.
 - b. Exposure of underlying soil or geologic formations less pervious and/or more erodible than original soil surface.
 - c. Reduced capacity of exposed soils to absorb rainfall due to compaction caused by heavy equipment.
 - d. Enlarged drainage areas caused by grading operations, diversions, and street construction.
 - e. Prolonged exposure of unprotected disturbed areas due to scheduling problems and/or delayed construction.

- f. Shortened times of concentration of surface runoff caused by altering steepness, distance, and surface roughness and through installation of "improved" storm drainage facilities.
 - g. Increased impervious surfaces associated with the construction of streets, buildings, sidewalks, and paved driveways and parking lots.
3. Alteration of the groundwater regime that may adversely affect drainage systems, slope stability, and survival of existing and newly established vegetation.
 4. Creation of exposures facing south and west that may hinder plant growth due to adverse temperature and moisture conditions.
 5. Exposure of subsurface materials that are rocky, acid, or otherwise unfavorable to the establishment of vegetation.
 6. Adverse alteration of surface runoff patterns by construction and development.

11.30 Principles of Reducing Erosion and Sedimentation from Developing Areas

The principles of reducing erosion and sedimentation from developing areas are:

1. Plan the development to fit the particular topography, soils, waterways, and natural vegetation at the site.

Initially, this is best achieved through adoption of a general land-use plan based upon a comprehensive inventory of soil, water, and related resources.

Slope length and gradient are key elements in determining the volume and velocity of the runoff and its associated erosion. As both slope length and steepness increase, the rate of runoff increases and the potential for erosion is magnified. Where possible, steep slopes should be left undisturbed. By limiting the length and steepness of the designed slopes, runoff volumes and velocities can be reduced and erosion hazards minimized.

Soils which contain a high proportion of silt and very fine sand are generally the most erodible. The erodibility of these soils is decreased as the percentage of clay or organic matter content increases. Well-drained and well-graded gravel-sand mixtures with little silt are the least erodible soils. By reducing the length and steepness of a given slope, even a highly erodible soil may show little evidence of erosion. Long steep slopes should be broken by benching, terracing, or constructing diversion structures.

The nearly level area adjacent to either side of the channel of streams is known as the floodplain. Floodplains are important because they store excess runoff temporarily, thus helping to avoid erosion and

flooding problems further downstream. Because of this, floodplains should be preserved in their natural state. Construction and filling activities should be minimized in these areas.

The natural vegetative cover is extremely important in controlling erosion since it: (a) shields the soil surface from the impact of falling rain; (b) increases infiltration of water into the soil; (c) reduces the velocity of the runoff water; and (d) holds soil particles in place as well as filters surface runoff.

2. Expose the smallest practical area of land for the shortest possible time.

When earthwork is required and the natural vegetation is removed, keep the area and the duration of exposure to a minimum. Plan the phases or stages of development so that only the areas which are actively being developed are exposed. All other areas should have a good cover of temporary or permanent vegetation or mulch. Grading should be completed as soon as possible after it is begun. Immediately after grading is complete, permanent vegetative cover should be established in the area. As cut slopes are made and as fill slopes are brought up to grade, these areas should be revegetated as the work progresses. This is known as staged seeding. Minimizing grading of large or critical areas during the season of maximum erosion potential (May 1 through September 30) reduces the risk of erosion.

After the best decision has been made regarding land use and the development process begins, effective erosion control and sediment reduction depends upon careful site planning, judicious selection of conservation practices, adequate design, accurate installation in a timely fashion, and sufficient maintenance to ensure the intended results.

3. Apply "Erosion Control" practices as a first line of defense against on-site damage.

This third principle relates to using practices that control erosion on a site to prevent excessive sediment from being produced. Keep soil covered as much as possible with temporary or permanent vegetation or with various mulch materials. Special grading methods such as roughening a slope on the contour or tracking with a cleated dozer may be used. Other practices include diversion structures to divert surface runoff from exposed soils and grade stabilization structures to control surface water.

"Gross" erosion in the form of gullies must be prevented by these control devices. Lesser types of erosion such as sheet and rill erosion should be prevented, but often scheduling or the large number of practices makes this impractical. However, when erosion is not adequately controlled, sediment control is more difficult and expensive.

4. Apply "Sediment Control" practices as a perimeter protection to prevent off-site damage.

This principle relates to using practices that control sediment once it is produced and preventing it from getting off-site. Diversion ditches, sediment traps, vegetative filters, and sediment basins are examples of practices to control sediment. Vegetative and structural sediment control measures can be classified as either temporary or permanent depending on whether or not they will remain in use after development is complete. Generally, sediment can be retained by two methods: (a) filtering runoff as it flows through an area and (b) impounding the sediment-laden runoff for a period of time so that the soil particles are deposited. The best way to control sediment, however, is to prevent erosion.

5. Implement a thorough maintenance and follow-up operation.

This fifth principle is vital to the success of the four others. A site cannot be effectively controlled without thorough, periodic checks of the erosion and sediment control practices. These practices must be maintained just as construction equipment must be maintained and materials checked and inventoried. An example of applying this principle would be to start a routine "end-of-day check" to make sure that all control practices are working properly.

Usually, these five principles are integrated into a system of vegetative and structural measures, along with management techniques, to develop a plan to prevent erosion and provide sediment control. In most cases, a combination of limited grading, limited time of exposure, and a judicious selection of erosion control practices and sediment-trapping facilities will prove to be the most practical method of controlling erosion and the associated production and transport of sediment.

11.40 Standards for Erosion and Sediment Control in Developing Areas

The purpose of these Standards is to establish uniform criteria for the design, review, approval, installation, and maintenance of erosion control and sediment reduction practices on land undergoing grading, construction, or development. Those responsible for selection and design or for review and approval of these practices should evaluate the conditions existing at the specific site and determine if the minimum criteria contained herein are adequate or if more stringent criteria should be employed.

It is intended that these Standards will be used as the approved erosion and sediment control practices. However, innovative methods are encouraged, provided that they are approved by the City Engineer.

These Standards were written to be generally applicable to all development and construction sites. However, there may be many specific problems or sites not appropriately covered by these standards. It is intended that modifications to these Standards may be made by the City of Dodge City in order to improve or update them.

It is not the intent of this manual to preclude the use of other techniques or to override the designer's ability or responsibility to use the most appropriate methods of analysis or practices. When conditions warrant

special consideration or when more sophisticated methods are justified by engineering or economic considerations, the designer is encouraged to use these methods. Approval to use such methods shall be obtained from the City Engineer before proceeding with the design.

These Standards have been organized into two general areas:

1. Temporary Structural Practices
2. Permanent Structural Practices

Structural practices are constructed for the purpose of controlling the flow of water, preventing erosion by flowing water, or for trapping sediment.

Temporary practices are those used for relatively short periods of time, from a three-month limit on a straw bale dike to a 36-month limit on a sediment basin, to accomplish erosion or sediment control during some stage of construction. Requirements for temporary practices in the Standards have been streamlined to accomplish the intended purpose with a minimum of cost. These practices must not be used for longer than the periods of time prescribed. Also, individual designs are not required for temporary structural practices in order to economize as much as possible on the designer's time. Permanent practices are to be used for permanent features, for an indefinite period of time, or when the temporary practices are not adequate.

It is the intent that practices described in Section 11.6 be temporary and that the removal of these practices be the responsibility of the developer. Removal should be performed when the need for the practice has been eliminated by permanent stabilization.

The Standards are such that the permanent practices will accomplish their intended function with a minimum of maintenance over long periods of time, such as an underground storm drainage system would. Permanent practices require individual designs in order to fit the individual situations.

11.50 The Sediment Control Plan

The required Sediment Control Plan is a plan for controlling erosion and sediment during construction in compliance with the laws, ordinances, and these Standards. This plan shall be a part of the total site development plan and prescribes all the steps necessary, including scheduling, to assure erosion and sediment control during all phases of construction including final stabilization.

Planning for sediment control should begin with the conceptual plan and its preparation. Such features as soils and topography should be considered for the conceptual plan as well as any requirements for sediment control or stormwater management.

Planning for sediment control should also begin with first-hand knowledge of the site by the designer. The plan shall be based on a sufficiently accurate topographic map that reflects the existing topography and site conditions. Adjacent areas affecting the site or affected by the site and its development shall be shown on the plans in sufficient detail to

accomplish the need. Examples of this would be areas draining onto the site or areas where storm runoff leaves the site and travels to a stream or drainage system.

The Sediment Control Plan will consist of the best selection of erosion control practices and sediment-trapping facilities, in conjunction with an appropriate schedule, to accomplish an adequate level of control. Particular attention must be given to concentrated flows of water, either to prevent its occurrence or to provide conveyance devices according to the Standards to prevent "major" or "gross" erosion. Sediment-trapping devices will usually be required at all points of egress of sediment-laden water. The plan must include permanent structures for conveying storm runoff, final site stabilization, removal of temporary sediment control features such as sediment basins, and finally, stabilization of the sites where temporary features were removed. Plans showing improvements or construction to be done outside the property line for the site will generally not be approved unless the plan is accompanied by an appropriate legal easement for the area in which the work is to be done.

The standardization of sediment control plans makes them easier to study and review. The List of Standard Symbols (Figure 11-1) was developed to facilitate plan review and to be easy to apply to the drawings by drafting or by the use of standard symbols. The symbols should be bold and easily identified on the plans.

Unless otherwise approved, one of the following scales shall be used for the detailed sediment control plans for urban development sites: 1" = 20', 1" = 30', 1" = 40', or 1" = 50'. The contour interval for these plans shall be 2 feet or smaller.

Sample Requirements for Sediment Control Plans

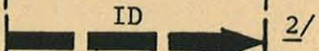
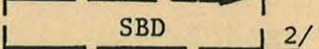
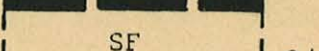
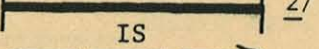
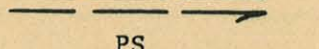
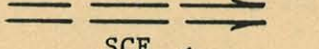
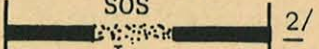
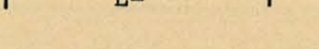
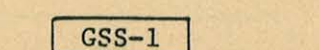
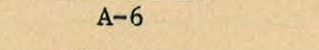
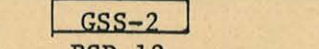
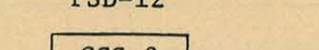
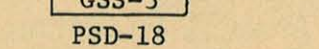
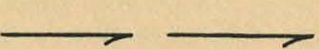
The area covered by the Sediment Control Plan must be adequately located on a vicinity map. It must further be delineated and identified by project name as shown on the record plat. All plans submitted for approval must be accompanied by a completed Project Information Sheet (Figure 11-2). Information required on this form will be furnished by the developer and/or his engineer.

Sediment Control Plan

The Sediment Control Plan shall include the existing and proposed topography. Existing topography can be either from actual field survey obtained from approved photogrametric methods or from information obtained from responsible agencies. No proposed slopes will exceed 2:1. All slopes steeper than 3:1 will require low-maintenance stabilization.

The existing and proposed improvements shall be shown on the sediment control plan and will include all buildings, roads, storm drains, etc. Proposed removal or alterations of existing facilities shall be indicated on the plan.

Figure 11-1
LIST OF STANDARD SYMBOLS

Diversion Dike	
Interceptor Dike.....	
Perimeter Dike	
Straw Bale Dike.....	
Silt Fence	
Interceptor Swale.....	
Perimeter Swale.....	
Stabilized Construction Entrance.....	
Stone Outlet Structure.....	
Grade Stabilization Structure.....	
Chute or Flume (Example for size group..... A and six foot bottom width)	
Rigid Pipe Slope Drain (Example for..... 12" diameter pipe)	
Flexible Pipe Slope Drain (Example for..... 18" diameter pipe)	
Diversion	
Grassed Waterway.....	
Level Spreader.....	
Subsurface Drain.....	

Notes:

1. Arrows point downslope or to the outlet.
2. Dike and diversion symbols, shall have a cross mark to clearly show where the structure begins and ends.

Figure 11-2

SAMPLE PROJECT INFORMATION SHEET

To: The City of Dodge City

_____ Date _____

Gentlemen:

The enclosed Sediment Control Plan, supporting documents, drawings and computations are respectfully submitted for your approval.

Record Plat Title: _____

Scheduled Start of Grading: _____

Type Zoning: _____

Scheduled Completion of Storm Drainage, Paving and/or Stabilization: _____

Size of Tract: _____

Sediment Control Practices Planned:

Preliminary Plan (date of approval) _____

Grading options:

Owner(s): _____

_____ Limited initial grading (designate areas)

Address: _____

_____ Delayed site (or lot) grading

Phone: _____

_____ Delayed finish grading

_____ Total site grading initially

Soil Information:

Locate property boundaries on Soil Survey Sheet and provide copy, OR, indicate Sheet Number and list symbols: _____

Erosion Control Devices (use proper nomenclature - permanent structures to be so indicated on drawings):

_____ Diversion Dikes

_____ Interceptor Dikes

_____ Grade Stabilization Structure(s)

_____ (specify flume, flexible down drain, etc.)

_____ Level Spreaders

_____ Grassed Waterways (or ditches)

_____ Diversions

_____ Sediment Traps

_____ Sediment Basin (specify number _____)

_____ Other _____

Jurisdiction (check one):

County _____

City _____

Other (specify) _____

Utilities (check appropriate space):

Sewer (_____ public or _____ private)

Water (_____ public or _____ private)

Critical Area Stabilization (Identify areas on plan):

_____ Structural (crushed stone, paving, etc.)

Plans Prepared by:

(Phone No.)

(Address)

Sediment Control Practices

All sediment control practices must be identified on the Sediment Control Plan. These practices will be shown in sufficient detail to facilitate implementation. All permanent sediment control structures will be labeled on the plan as PERMANENT. All temporary stabilization practices will be labeled on the plan as TEMPORARY. The location and methods of stabilization will be indicated on the Plan.

A schedule, or sequence, of operations will be included on the Sediment Control Plan. Special emphasis will be placed on the scheduled start of clearing and/or grading, sequence of installation of sediment control and stormwater management facilities, duration of exposure, and the scheduled start and completion dates of stabilization measures (both temporary and permanent).

Sediment Basins

On projects incorporating sediment basins, the information requested in the Standards shall be submitted for approval. Supporting computations will be furnished on a design data sheet similar to that shown in Figure 11-3 for sediment basin design. Additionally, state on the Plan the provisions for stabilization of the structure and the elevation at which cleanout is required.

Street Profiles

On projects where streets are to be graded, street profiles must be furnished. Include the location and spacing of interceptor dikes, the location of diversion dikes, and the location of outlets for the dikes. Provide information describing the condition below the outlets, including existing vegetation, slope, and the method of stabilizing the outfall area, where necessary.

Drainage Plan

A Drainage Plan shall be provided as per Sections 2 and 3. Based on this Plan, indicate the velocity for (1) pipe outfall, (2) outfall structure, and (3) natural or designed channel below outfall structures to point to entry into existing system or natural stream. Show on Sediment Control Plan the proposed method of stabilizing the outfall, consistent with computed velocities.

11.60 Standards for Temporary Structural Practices

Diversion Dike

Definition

A temporary ridge of compacted soil immediately above cut or fill slopes and constructed with sufficient grade to provide drainage.

Figure 11-3

SEDIMENT BASIN DESIGN DATA SHEET

Computed by _____ Date _____
 Checked by _____ Date _____

Project _____
 Basin _____
 # _____ Location _____
 Total Area draining to basin, _____ Acres.

BASIN VOLUME DESIGN

1. Min. required vol. = 67 cu. yds. x _____ ac. drainage = _____ cu. yds.
2. Vol. of basin = _____ cu. yds.
3. Excavate _____ cu yds. to obtain required capacity.
 Min. vol. before cleanout = 27 cu. yds. x _____ ac. drainage = _____ cu. yds.
 Elevation corresponding to scheduled time to clean out _____
 Distance below top of riser _____.

DESIGN OF SPILLWAYRunoff

4. Q_p = _____ cfs (See Chapter 2, Storm Runoff)

Pipe Spillway (Q_{ps})

5. Min. pipe spillway capacity, Q_{ps} = 0.2 x _____ ac. drainage = _____ cfs.
 Note: If there is no emergency spillway, then req'd. Q_{ps} = Q_p = _____ cfs.
6. H = _____ ft. Barrel length = _____ ft.
7. Barrel: Diam. _____ inches;
8. Riser: Diam. _____ inches; Length _____ ft.; h = _____ ft.

Emergency Spillway Design

10. Emergency Spillway Flow, Q_{es} = Q_p - Q_{ps} = _____ - _____ = _____ cfs.
11. Width _____ ft. H _____ ft.
 Entrance Channel Slope _____ %
 Exit Channel Slope _____ %

ANTI-SEEP COLLAR DESIGN

12. y = _____ ft.; z = _____ :1; pipe slope = _____ %, L_s = _____ ft.
 Use _____ collars, _____' - _____" square; projection = _____ ft.

DESIGN ELEVATIONS

13. Riser Crest = _____ Design High Water = _____
 Em. Spwy. Crest = _____ Top of Dam = _____

Purpose

The purpose of a diversion dike is to intercept storm runoff from small upland areas and divert it from exposed slopes to an acceptable outlet.

Application

The diversion dike is used for the period of construction at the top of newly constructed slopes to prevent excessive erosion until permanent drainage features are installed and/or slopes are stabilized:

Design Criteria

Table 11-1 Diversion Dike Design Criteria

Drainage Area:	Less than 5 acres (for larger drainage areas see Standards for Diversion).
Top Width:	2 feet minimum.
Height (Compacted):	18 inches minimum height measured from the existing ground at the upslope toe to top of the dike.
Side Slopes:	2:1 or flatter.
Grade:	Dependent upon topography, but must have positive drainage.
Stabilization:	Where slope of channel (flow area) is: 0 - 5%: Stabilization may be required by the designer according to the needs of the site. Over 5%: Stabilization shall be required. Stabilization shall be: (1) in accordance with Standards for Diversion, or (2) by lining the flow area with stone that meets AASHTO M43, Size No. 2 or 24 in a layer at least 3 inches in thickness and extending up the upslope side of the dike at a height of at least 8 inches, measured vertically from the upslope toe, and extending at least 7 feet upslope from the upslope toe.

Outlet

- A. Diverted runoff from a protected or stabilized area shall discharge directly to an undisturbed stabilized area or into a level spreader or grade stabilization structure.
- B. Diverted runoff from a disturbed or exposed upland area shall be conveyed to a sediment trap or sediment basin, or to an area protected by any of these practices.

Interceptor Dike

Definition

A temporary ridge of compacted soil located across disturbed areas or rights-of-way.

Purpose

The purpose of an interceptor dike is to shorten the length of exposed slopes, thereby reducing the potential for erosion by intercepting storm runoff and diverting it to a stabilized outlet or sediment-trapping device.

Application

Interceptor dikes are constructed across disturbed rights-of-way such as for pipelines and streets or disturbed areas such as graded parking lots or landfills. The dikes shall remain in place until the disturbed areas are permanently stabilized.

Design Criteria

Table 11-2 Interceptor Dike Design Criteria

A design is not required for interceptor dikes; however, the following criteria shall be used:

Drainage Area:	Less than 5 acres (for larger drainage areas see Standards for Diversion).
Top Width:	2 feet minimum.
Height:	18 inches minimum height measured from the existing or graded ground at the upslope toe to top of the dike.
Side Slopes:	2:1 or flatter (flat enough to allow construction traffic to cross if desired).
Grade:	0.5 to 1.0 percent and must have positive drainage to an acceptable outlet.
Stabilization:	Not required by this Standard but may be required by the designer according to the needs of the site.

Stabilization, when required, shall be: (1) in accordance with Standards for Diversion, or (2) by lining the flow area with stone that meets AASHTO M43, Size No. 2 or 24 in a layer at least 3 inches in thickness and pressed into the soil. The lining shall extend up the upslope side of the dike a height of at least 8 inches measured vertically from the upslope toe

shall extend at least 7 feet upslope from the upslope toe.

Spacing:

Slope of right-of-way or disturbed areas above dike	Greater than 10%	5-10%	Less than 5%
Maximum distance between dikes	100 feet	200 feet	300 feet

Outlet

Interceptor dikes shall have an outlet that functions with a minimum of erosion.

Runoff shall be conveyed to a sediment trap or sediment basin when either the dike channel or the drainage area above the dike is not adequately stabilized.

The on-site location may need to be adjusted to meet field conditions in order to utilize the most suitable outlet.

Perimeter Dike

Definition

A temporary ridge of compacted soil located along the perimeter of the site or disturbed areas.

Purpose

The purpose of a perimeter dike is to prevent off-site storm runoff from entering the disturbed area and to prevent sediment-laden storm runoff from leaving the construction site or disturbed area.

Application

The perimeter dike is used at the perimeter of the disturbed area during the period of construction to transport sediment-laden water to a sediment-trapping device. This dike shall remain in place until the disturbed area is permanently stabilized. The storm runoff prevented from entering the disturbed area by the perimeter dike shall be adequately handled to prevent damage to adjacent property by flooding or erosion.

Design Criteria

The perimeter dike shall not be constructed outside the property lines without obtaining legal easements from affected adjacent property owners.

A design is not required for perimeter dikes; however the following criteria shall be used:

Table 11-3 Perimeter Dike Design Criteria

Drainage Area:	Less than 5 acres (for larger drainage areas see Standards for Diversion).
Top Width:	2 feet minimum.
Height:	18 inches minimum height measured from the existing or graded ground at the upslope toe to top of the dike.
Side Slopes:	2:1 or flatter (flat enough to allow construction traffic to cross if desired).
Grade:	Dependent upon topography, but shall have positive drainage to an adequate outlet.
Stabilization:	Where slope of the channel (flow area) is: 0 - 5%: Stabilization may be required by the designer according to the needs of the site. Over 5%: Stabilization shall be required. Shall be: (1) in accordance with Standards for Diversion, and the area to be stabilized shall be the channel (flow area); or (2) the flow area shall be lined with stone that meets AASHTO M43, Size No. 2 or 24 which is placed in a 3-inch-thick layer and pressed into the soil.

Outlet

- A. Diverted runoff from a protected or stabilized upland area shall outlet directly onto an undisturbed stabilized area or into a level spreader or grade stabilization structure.
- B. Diverted runoff from a disturbed or exposed upland area shall be conveyed to a sediment trap or a sediment basin, or to an area protected by any of these practices.

Straw Bale Dike

Definition

A temporary barrier with a life expectancy of 3 months or less, installed across or at the toe of a slope.

Purpose

The purpose of a straw bale dike is to intercept and detain small amounts of sediment from unprotected areas of limited extent.

Application

The straw bale dike is used where:

- A. No other practice is feasible, and
- B. There is no concentration of water in a channel or other drainageway above the barrier, and
- C. Erosion would occur in the form of sheet and rill erosion, and
- D. Contributing drainage area is less than 0.5 acres and the length of slope above the dike is less than 100 feet. The practice may also be used for a long single-family lot if the slope is less than 15 percent. The contributing drainage area in this instance shall be less than 1 acre and the length of slope above the dike shall be less than 200 feet.

Design Criteria

A design is not required. All bales shall be placed on the contour and shall be either wire bound or nylon-string tied.

Silt Fence

Definition

Temporary barrier fence made of burlap or polypropylene material which is water-permeable but will trap water-borne sediment.

Purpose

To intercept and detain water-borne sediment from unprotected areas of limited extent.

Application

A silt fence is used during the period of construction near the perimeter of a disturbed area to intercept sediment while allowing water to percolate through. This fence shall remain in place until the disturbed area is permanently stabilized. Silt fence should not be used where there is a concentration of water in a channel or other drainageway.

Design Criteria

A silt fence is constructed near the perimeter of a disturbed site within the developing area. It is not to be constructed outside the property lines without obtaining a legal easement from the affected adjacent property owners.

A design is not required for the installation of a silt fence; however, the following criteria shall be observed:

Table 11-4 Silt Fence Design Criteria

Drainage Area:	Less than 2 acres.
Height:	30-inch minimum height measured from the existing or graded ground.
Material:	Burlap, polypropylene fabric, or nylon reinforced with polyester netting. The Mullen burst strength shall be greater than 150 psi. The edges shall be treated to prevent unraveling.
Support:	Steel fence posts spaced a maximum 8 feet apart. Woven wire shall be required to support the material unless the manufacturers recommendations exclude its use.

Outlet

The silt fence shall be placed and constructed in such a manner that runoff from a disturbed upland area shall be intercepted, the sediment trapped, and the surface runoff allowed to percolate through the structure.

The silt fence shall be placed in such a manner that surface runoff which percolates through will flow onto an undisturbed stabilized area or stabilized outlet.

Interceptor Swale

Definition

A temporary excavated drainageway located across disturbed areas or rights-of-way.

Purpose

The purpose of an interceptor swale is to shorten the length of exposed slopes, thereby reducing the potential for erosion by intercepting storm runoff and diverting it to a stabilized outlet or sediment-trapping device.

Application

Interceptor swales are constructed across disturbed rights-of-way such as for pipelines and streets or disturbed areas such as graded parking lots or landfills. The swale shall remain in place until the disturbed areas are permanently stabilized.

Design Criteria

A design is not required for interceptor swales; however the following criteria shall be used:

Table 11-5 Interceptor Swale Design Criteria

Drainage Area:	Less than 5 acres (for larger drainage areas see Standards for Grassed Waterway).		
Bottom Width:	7 feet minimum and the bottom shall be level.		
Depth:	1 foot minimum.		
Side Slopes:	2:1 or flatter (flat enough to allow construction traffic to cross if desired).		
Grade:	1 to 3 percent; must have positive drainage to an adequate outlet.		
Stabilization:	Not required by this Standard but may be required by the designer according to the needs of the site. Stabilization shall be: (1) in accordance with the Standards for Grassed Waterway, or (2) by lining the flow area with stone that meets AASHTO M43, Size No. 2 or 24 in a layer at least 3 inches in thickness and pressed into the soil. The lining shall extend across the bottom and up both sides of the channel a height of at least 8 inches vertically above the bottom.		
Traffic Crossings:	At all points where several or more vehicle crossings per day will be made, the swale shall be stabilized according to Stabilization Method No. 2, except the stone lining shall be at least 6 inches in thickness for the full width of the traffic crossing roadway.		
Spacing:			
Slope of right-of-way or disturbed area	Greater than 10%	5-10%	Less than 5%
Maximum distance between swales	100 feet	200 feet	300 feet

Outlet

An interceptor swale shall have an outlet that functions with a minimum of erosion.

Runoff shall be conveyed to a sediment trap or sediment basin.

The on-site location may need to be adjusted to meet field conditions in order to utilize the most suitable outlet.

Perimeter Swale

Definition

A temporary excavated drainageway located along the perimeter of the site or disturbed areas.

Purpose

The purpose of a perimeter swale is to prevent off-site storm runoff from entering the disturbed area and to prevent sediment-laden storm runoff from leaving the construction site or disturbed area.

Application

The perimeter swale is used for the period of construction at the perimeter of the disturbed area to transport sediment-laden water to a sediment trap or sediment basin. This swale shall remain in place until the disturbed area is permanently stabilized. The perimeter swale also is used to prevent storm runoff from entering the disturbed area. This runoff shall be adequately handled to prevent damage to adjacent property from flooding or erosion.

Design Criteria

The perimeter swale shall not be constructed outside the property lines without obtaining legal easements from affected adjacent property owners. A design is not required for perimeter swale; however the following criteria shall be used:

Table 11-6 Perimeter Swale Design Criteria

Drainage Area:	Less than 5 acres (for larger drainage areas, see Standards for Diversion or Grassed Waterway).
Bottom Width:	7 feet minimum and the bottom shall be level.
Depth:	1 foot minimum.
Side Slope:	2:1 or flatter (flat enough to allow construction traffic to cross if desired).
Grade:	Dependent upon topography, but shall have a minimum grade of 1 percent to an adequate outlet.
Stabilization:	Where slope of the channel (flow area) is: 1 - 5%: Stabilization may be required by the designer according to the needs of the site.

Over 5%: Stabilization shall be required.

Stabilization shall be: (1) in accordance with the Standards for Grassed Waterway, or (2) by lining the flow area with stone that meets AASHTO M43, Size No. 2 or 24 in a layer at least 3 inches in thickness and pressed into the soil. The lining shall extend across the bottom and up both sides of the channel a height of at least 8 inches vertically above the bottom.

Traffic Crossings:

At all points where several vehicle crossings per day will be made, the swale shall be stabilized according to No. 2 above, except the stone lining shall be at least 6 inches in thickness for the full width of the traffic crossing roadway.

Outlet

- A. Diverted runoff from a protected or stabilized upland area shall outlet directly onto an undisturbed stabilized area, level spreader, or into a grade stabilization structure.
- B. Diverted runoff from a disturbed or exposed upland area shall be conveyed to a sediment trap or a sediment basin, or within an area protected by any of these practices.

Stabilized Construction Entrance

Definition

A stabilized pad of crushed stone located at any point where traffic will be entering or leaving a construction site to or from a public right-of-way, street, alley, sidewalk, or parking area.

Purpose

The purpose of a stabilized construction entrance is to reduce or eliminate the tracking or flowing of sediment onto public rights-of-way.

Application

A stabilized construction entrance applies to all points of construction ingress and egress.

Design Criteria

Table 11-7 Stabilized Construction Entrance Design Criteria

Stone Size: Use AASHTO Designation M43, Size No. 2 (2.5 inches to 1.5 inches).

Thickness: Not less than 8 inches.

Width: Not less than full width of all points of ingress and egress.

Length: As required, but not less than 50 feet.

Maintenance

The entrance shall be maintained in a condition which will prevent tracking or flowing of sediment onto public rights-of-way. This may require periodic top dressing with additional stone as conditions demand and repair and/or cleanout of any measures used to trap sediment. All sediment spilled, dropped, washed, or tracked onto public rights-of-way must be removed immediately.

When necessary, wheels must be cleaned to remove sediment prior to entrance onto public right-of-way. When washing is required, it shall be done on an area stabilized with crushed stone which drains into an approved sediment trap or sediment basin. All sediment shall be prevented from entering any storm drain, ditch, or watercourse through use of sand bags, gravel, boards, or other approved methods.

Stone Outlet Structure

Definition

A temporary crushed-stone dike installed in conjunction with, and as a part of, a diversion dike, interceptor dike, or perimeter dike.

Purpose

The purpose of the stone outlet structure is to provide a protected outlet for a diversion dike, interceptor dike, or perimeter dike, to provide for diffusion of concentrated flow, and to allow the area behind the dike to dewater.

Application

Stone outlet structures apply to any point of discharge where there is need to dispose of runoff at a protected outlet or to diffuse concentrated flow for the duration of the period of construction. When the entire drainage area to the structure is not stabilized, a sediment trap must be provided in conjunction with the stone outlet structure. (See Standards for Sediment Trap.)

Design Criteria

A stone outlet structure shall be used only where the contributing watershed is less than five acres. The minimum length, in feet, of the crest of the stone outlet structure shall be equal to six times the number of acres of contributing drainage area. The crest of the stone dike shall be at least six inches lower than the lowest elevation of the top of the earth dike and shall be level. The stone shall be crushed stone. Gravel may be used only

if crushed stone is not available. The stone shall meet AASHTO M43, Size No. 2 or 24.

Outlet

The stone outlet structure shall be located so as to discharge onto an already stabilized area or into a stable watercourse. Stabilization shall consist of complete vegetal cover, paving, etc., sufficiently established to be erosion resistant.

Grade Stabilization Structure (Paved Chute or Flume)

Definition

A temporary channel lined with bituminous concrete, Portland cement concrete, or comparable non-erodible material placed to extend from the top of a slope to the bottom of a slope.

Purpose

The purpose of the paved chute or flume is to convey surface runoff safely down slopes without causing erosion.

Application

A paved chute or flume is to be used where concentrated flow of surface runoff must be conveyed down a slope in order to prevent erosion. The maximum allowable drainage area shall be 36 acres.

Design Criteria

Size Group A

- A. The height (H) of the dike at the entrance is at least 1.5 feet.
- B. The depth (d) of the chute down the slope is at least 8 inches.
- C. The length (L) of the inlet and outlet sections is 5 feet.

Size Group B

- A. The height (H) of the dike at the entrance is at least 2 feet.
- B. The depth (d) of the chute down the slope is at least 10 inches.
- C. The length (L) of the inlet and outlet section is 6 feet.

Each size group has various bottom widths and allowable drainage areas as shown below:

Table 11-8 Paved Chute Or Flume Design Criteria

Size ¹	Bottom Width, b (feet)	Maximum Drainage Area (acres)	Size ¹	Bottom Width, b (feet)	Maximum Drainage Area (acres)
A-2	2	5	B-4	4	14
A-4	4	8	B-6	6	20
A-6	6	11	B-8	8	25
A-8	8	14	B-10	10	31
A-10	10	18	B-12	12	36

1. The size is designated with a letter and a number, such as A-6, which means a chute or flume in Size Group A with a 6-foot bottom width. The selected size shall be shown on the plans.

If a minimum of 75 percent of the drainage area will have a good grass or woodland cover throughout the life of the structure, the drainage areas listed above may be increased by 50 percent. If a minimum of 75 percent of the drainage area will have a good mulch cover throughout the life of the structure, the drainage areas listed above may be increased by 25 percent.

Outlet

When a paved chute or flume of size Group B is used, the velocity at its outfall shall be checked for erosion potential downstream.

Grade Stabilization Structure (Pipe Slope Drain)

Definition

A flexible tubing and/or rigid pipe with prefabricated entrance section temporarily placed to extend from the top of a slope to the bottom of a slope.

Purpose

The purpose of the pipe slope drain is to convey surface runoff safely down slopes without causing erosion.

Application

Pipe slope drains are to be used where concentrated flow of surface runoff must be conveyed down a slope in order to prevent erosion. The maximum allowable drainage area shall be 5 acres.

Design Criteria

Pipe slope drains are to be sized as follows:

Table 11-9 Pipe Slope Drain Design Criteria

<u>Size</u>	<u>Pipe or Tubing Diameter, D (in.)</u>	<u>Maximum Drainage Area (acres)</u>
PSD-12	12	0.5
PSD-18	18	1.5
PSD-21	21	2.5
PSD-24	24	3.5
PSD-30	30	5.0

Inlet

The height of the earth dike at the entrance to the pipe slope drain shall be equal to or greater than the diameter of the pipe, D, plus 12 inches.

Outlet

The pipe slope drain shall outlet onto a riprap apron and then into a stabilized area or stable watercourse. A sediment-trapping device shall be used to trap sediment from any sediment-laden water conveyed by the pipe slope drain.

Sediment Basin

Definition

A temporary barrier or dam constructed across a waterway or at other suitable locations to intercept sediment-laden runoff and to trap and retain the sediment.

Scope

This standard applies to the installation of temporary sediment basins on sites where: (1) failure of the structure would not result in loss of life, damage to homes or buildings, or interruption of use or service of public roads or utilities; (2) the drainage area does not exceed 100 acres, and (3) the basin is to be removed within 36 months after the beginning of construction of the basin.

Purpose

The purpose of a sediment basin is to intercept sediment-laden runoff and reduce the amount of sediment leaving the disturbed area in order to protect drainageways, properties, and rights-of-way below the sediment basin.

Application

A sediment basin applies where physical site conditions or land ownership restrictions preclude the installation of erosion control measures to

adequately control runoff, erosion, and sedimentation. It may be used below construction operations which expose critical areas to soil erosion. It remains in effect until the disturbed area is protected against erosion by permanent stabilization.

Design Criteria for Temporary Sediment Basins

Compliance with Laws and Regulations

Design and construction shall comply with state and local laws, ordinances, rules, and regulations.

Location

The sediment basin should be located to obtain the maximum storage benefit from the terrain and for ease of cleanout of the trapped sediment. It should be located to minimize interference with construction activities and construction of utilities.

Size of the Basin

The volume of the sediment basin, as measured from the bottom of the basin to the elevation of the crest of the principal spillway shall be at least 67 cubic yards per acre of total drainage area (0.5 inches).

Sediment basin shall be cleaned out when the volume as described above is reduced by sedimentation to 27 cubic yards per acre of drainage area (0.2 inches), except in no case shall the sediment level be permitted to build up higher than 1 foot below the principal spillway crest. This cleanout shall restore the original design volume to the sediment basin. The elevation corresponding to the maximum allowable sediment level shall be determined and shall be stated in the design data as a distance below the top of riser and shall be clearly marked on the riser.

The basin dimensions necessary to obtain the required basin volume as stated above shall be clearly shown on the plans to facilitate plan review, construction, and inspection.

Entrance of Runoff into Basin

Points of entrance of surface runoff into excavated sediment basins shall be protected to prevent erosion. Diversions, grade stabilization structures or other water control devices shall be installed as necessary to ensure direction of runoff and protect points of entry into the basin.

Disposal

The sediment basin plans shall indicate the method(s) of disposing of the sediment removed from the basin. The sediment shall be placed in such a manner that it will not erode from the site. The sediment shall not be deposited downstream from the basin or in or adjacent to a stream or floodplain.

The sediment basin plans shall also show the method of disposing of the sediment basin after the drainage area is stabilized, and shall include the stabilizing of the sediment basin site. Water lying over the trapped sediment shall be removed from the basin by pumping, cutting the top of the riser, or other appropriate method prior to removing or breaching the embankment. Sediment shall not be allowed to flush into the stream or drainageway.

Safety

Sediment basins are attractive to children and can be very dangerous. Therefore, they shall be fenced or otherwise secured unless this is deemed unnecessary due to the remoteness of the site or other circumstances. In any case, local ordinances and regulations regarding health and safety must be adhered to.

Spillways

Sediment basins shall conform to the hydrologic, hydraulic and other design criteria for storage structures as given in Section 9.20.

Sediment Trap

Definition

A small temporary basin formed by excavation and/or an embankment to intercept sediment-laden runoff and to trap and retain the sediment.

Purpose

The purpose of a sediment trap is to intercept sediment-laden runoff and trap the sediment in order to protect drainageways, properties, and rights-of-way below the sediment trap from sedimentation.

Application

A sediment trap is usually installed in a drainageway, at a storm drain inlet, or at other points of discharge from a disturbed area.

Design Criteria

If any of the design criteria presented here cannot be met, see Standards for Sediment Basin.

Drainage Area

The drainage area for a sediment trap shall be less than 5 acres.

Location

The sediment trap should be located to obtain the maximum storage benefit from the terrain, for ease of cleanout and disposal of the trapped sediment and to minimize interference with construction activities.

Trap Size

The volume of a sediment trap as measured at the elevation of the crest of the outlet shall be at least 1800 cubic feet per acre of drainage area. The volume of the trap shall be calculated using standard mathematical procedures. The volume of a natural basin may be approximated by the equation: $\text{volume (ft}^3\text{)} = 0.4 \times \text{surface area (ft}^2\text{)} \times \text{maximum depth (ft)}$.

Trap Cleanout

Sediment shall be removed and the trap restored to its original dimensions when the sediment has accumulated to one-half of the design depth of the trap. Sediment removed from the trap shall be deposited in a suitable area and in such a manner that it will not erode.

Embankment

All embankments for sediment traps shall not exceed 5 feet in height as measured at the low point of the original ground along the centerline of the embankment. Embankments shall have a minimum 4-foot-wide top and side slopes of 2:1 or flatter. The embankment shall be compacted by traversing with equipment while it is being constructed.

Excavation

All excavation operations shall be carried out in such a manner that erosion and water pollution shall be minimal. Any excavated portion of sediment trap shall have 2:1 or flatter slopes.

Outlets

There are four types of outlets for sediment traps. Each sediment trap is named according to the type of outlet that it has. Each type has different design criteria and will be discussed separately. The outlets shall be designed, constructed, and maintained in such a manner that sediment does not leave the trap and that erosion of the outlet does not occur. A trap may have several different outlets, with each outlet conveying part of the flow, based on the criteria below. The combined outlet capacity shall meet that criteria. For example, a 12-foot earth outlet (adequate for 2 acres) and a 12-inch pipe outlet (adequate for 1 acre) could be used for a 3-acre drainage area.

1. An earth-outlet sediment trap consists of a basin formed by excavation and/or an embankment. The trap has a discharge point over or cut into natural ground. The outlet width, in feet, shall be equal to six times the drainage area, in acres. If an embankment is used, the outlet crest shall be at least 1 foot below the top of the embankment. The outlet shall be free of any restriction to flow.
2. A pipe-outlet sediment trap consists of a basin formed by excavated embankments. The outlet for the trap is through a perforated riser and a pipe through the embankment. The outlet

pipe and riser shall be made of corrugated metal. The riser diameter shall be of the same or larger diameter than the pipe. The top of the embankment shall be at least 1.5 feet above the crest of the riser. At least the top two-thirds of the riser shall be perforated with 0.5-inch-diameter holes spaced 8 inches vertically and 10 to 12 inches horizontally. All pipe connections shall be watertight.

Select pipe diameter from the following table:

Table 11-10 Sediment Trap Outlets

<u>Minimum Pipe Diameter (in.)</u>	<u>Maximum Drainage Area (ac)</u>
12	1
18	2
21	3
24	4
30	5

3. A stone-outlet sediment trap consists of a basin formed by an embankment or excavation and an embankment. The outlet for the trap is over a level stone section. The stone outlet for a sediment trap differs from that for a stone outlet structure because of the intentional ponding of water behind the stone. To provide for a ponding area, a relatively impervious core (e.g., timber, concrete block, or straw bales) is placed in the stone. The core shall be covered by 6 inches of stone.

The minimum length, in feet, of the outlet shall be equal to six times the drainage area, in acres. The crest of the outlet (top of stone) shall be at least 1 foot below the top of the embankment. The crushed stone used in the outlet shall meet AASHTO M43, Size No. 2 or 24, or its equivalent. Gravel meeting the above gradation may be used if crushed stone is not available.

4. A storm-inlet sediment trap consists of a basin formed by excavation or natural ground that discharges through an opening in a storm drain inlet structure. This opening can either be the inlet opening or a temporary opening made by omitting bricks or blocks in the inlet.

A yard drain inlet or an inlet in the median strip of a dual highway would use the inlet opening for an outlet. A curb inlet would require a temporary opening. The trap should be out of the roadway so as not to interfere with construction. Placing the trap on the opposite side of the opening and diverting water from the roadway to the trap is one means of doing this.

Sediment Plan Details

There is no standard symbol for a sediment trap. Each trap shall be delineated on the plans in such a manner that it will not be confused with

any other features. Each trap on a plan shall have a number and the numbers shall be consecutive. The following information shall be shown for each trap in a summary table form on the same sheet that the trap is on:

1. Type of trap
2. Size of outlet
3. Trap dimensions
4. Embankment height and excavation depth
5. Drainage area

11.70 Standards For Permanent Structural Practices

Diversion

Definition

A drainageway of parabolic or trapezoidal cross section with a supporting ridge on the lower side that is constructed across the slope.

Purpose

The purpose of a diversion is to intercept and convey runoff to stable outlets at nonerosive velocities.

Application

Diversions are used where:

1. Runoff from higher areas is or has potential for damaging property, causing erosion, or interfering with or preventing the establishment of vegetation on lower areas.
2. The length of slopes needs to be reduced so that soil loss will be reduced to a minimum.

Diversions are only applicable below stabilized or protected areas. Avoid establishment on slopes greater than 15 percent.

Design Criteria

The design procedures for trapezoidal channels are provided in Section 7.

Location

Diversion location shall be determined by considering outlet conditions, topography, land use, soil type, length of slope, and the layout of the proposed development.

Capacity

The constructed diversion shall have capacity to carry, as a minimum, the peak discharge from a 10-year frequency rainfall event with freeboard of not less than 1 foot.

Diversions designed to protect homes, schools, industrial buildings, roads, parking lots, and comparable high risk areas, and those designed to function in connection with other structures, shall have sufficient capacity to carry peak runoff expected from a storm frequency consistent with the hazard involved.

Velocity and Grade

The permissible velocity for the specified method of stabilization will determine the maximum grade. Maximum permissible velocities of flow for the stated conditions of stabilization are shown in Table 7-3, Maximum Permissible Design Velocities.

Cross Section

The diversion channel shall be parabolic or trapezoidal in shape.

The diversion shall be designed to have stable side slopes. The side slopes shall not be steeper than 2:1 and shall be flat enough to ensure ease of maintenance of the structure and its protective vegetative cover.

The ridge shall have a minimum width of 4 feet at the design water-surface elevation, a minimum of 1 foot of freeboard, and a reasonable settlement factor shall be provided.

Outlets

Each diversion shall have a stable outlet. The outlet may be a constructed or natural waterway, a stabilized open channel, grade stabilization structure, etc. In all cases, the outlet must discharge in such a manner as not to cause erosion. Outlets shall be constructed and stabilized prior to the operation of the diversion.

Grassed Waterway

Definition

A natural or man-made drainageway of parabolic or trapezoidal cross section that is below adjacent ground level and is stabilized by suitable vegetation. The flow is normally wide and shallow, and conveys the runoff down the slope.

Purpose

The purpose of a grassed waterway is to convey runoff without causing damage by erosion.

Application

Grassed waterways are used where added channel capacity or stabilization is required to control erosion resulting from concentrated runoff and where such control can be achieved by this practice along or in combination with others.

Design Criteria

Capacity

The minimum capacity shall be that required to confine the peak rate of runoff expected from a 10-year frequency rainfall event or a higher frequency corresponding to the hazard involved. This requirement for confinement may be waived on slopes of less than 1 percent where out-of-bank flow will not cause erosion or property damage.

Where there is base flow, it shall be handled by a subsurface drain or a concrete, stone, or gabion mattress-lined low-flow channel. The capacity of the subsurface drain or low-flow channel shall be 5 percent of the design peak flow or as determined by actual measurement of the maximum base flow.

The design procedure for trapezoidal channels is provided in Section 7.

Velocity

Maximum permissible velocities of flow for the stated conditions of stabilization are shown in Table 7-3, Maximum Permissible Design Velocities.

Cross Section

The design water-surface elevation of a waterway downstream of diversions or other tributary channels shall be equal to or less than the design water-surface elevation in the diversion or other tributary channels.

Waterways shall have a minimum topwidth of 15 feet and a minimum bottom width of 10 feet.

Outlets

Each waterway shall have a stable outlet. The outlet may be another waterway, a stabilized open channel, grade stabilization structure, etc.

In all cases, the outlet must discharge in such a manner as not to cause erosion. Outlets shall be constructed and stabilized prior to the operation of the waterway.

Drainage

Subsurface drainage measures shall be provided for sites having high water tables or seepage problems, except where water-tolerant vegetation such as reed canarygrass can be used.

Where there is base flow, a subsurface drain or concrete low-flow channel shall be required.

Level Spreader

Definition

An outlet constructed at zero grade across the slope, whereby concentrated runoff may be discharged at nonerosive velocities onto undisturbed areas stabilized by existing vegetation.

Purpose

The purpose of the level spreader is to convert a concentrated flow of sediment-free runoff (e.g., diversion outlets) into sheet flow and to discharge it into areas stabilized by existing vegetation without causing erosion.

Application

The level spreader is used only in those situations where the spreader can be constructed on undisturbed soil, where the area directly below the level lip is stabilized by existing vegetation, where the drainage area above the spreader is stabilized by existing vegetation, and where the water will not be reconcentrated immediately below the point of discharge.

Design Criteria

The design criteria for level spreader shall be a maximum of 1 cfs per foot of length based on the peak flowrate from a 10-year frequency rainfall event.

For situations exceeding this criteria, see Standards for Grade Stabilization Structure, Grassed Waterway, etc.

Final discharge will be over the level lip, protected with fiberglass matting erosion stops and jute or excelsior protective material, to an existing stabilized area. The stabilized area shall have a complete vegetative cover sufficiently established to be erosion resistant.

Riprap

Definition

A layer of loose rock or aggregate placed over an erodible soil surface.

Purpose

The purpose of riprap is to protect the soil surface from the erosive forces of water.

Application

This practice applies to soil-water interfaces where the soil conditions, water turbulence and velocity, expected vegetative cover, and groundwater conditions are such that the soil may erode under the design flow conditions. Riprap may be used, as appropriate, at such places as storm

drain outlets, channel banks and/or bottoms, roadside ditches, drop structures, and shorelines.

Design Criteria

The minimum design discharge for channels and ditches shall be the peak discharge from a 10-year frequency rainfall event. The roughness coefficient, n , to be used for determining flow on the constructed riprap surface shall be equal to 0.022. See paragraph 8.30 for further design criteria.

Land Grading

Definition

Reshaping of the existing topography in accordance with a plan as determined by engineering survey and layout.

Purpose

The purpose of land grading is to provide for erosion control and vegetative establishment on those areas where the existing topography is to be reshaped by grading according to plan.

Design Criteria

The grading plan should be based upon the incorporation of building designs and street layouts that fit and utilize existing topography and desirable natural surroundings to avoid extreme grade modifications. Information submitted will provide sufficient topographic surveys and soil investigations to determine limitations that must be imposed on the grading operation related to slope stability, effect on adjacent properties and drainage patterns, and measures for drainage, and vegetative treatment.

The plan must show existing and proposed contours of the areas to be graded. The plan shall also include practices for erosion control, slope stabilization, safe disposal of runoff water, and drainage, such as waterways, lined ditches, reverse slope benches (include grade and cross section), grade stabilization structures, retaining walls, and surface and subsurface drains. The plan shall also include scheduling and phasing of these practices. The following shall be incorporated into the plan:

1. Provisions shall be made to safely conduct surface runoff to storm drains, protected outlets, or to stable water courses to ensure that surface runoff will not damage slopes or other graded areas. (See Standards for Grassed Waterway, Diversion, and Grade Stabilization Structure.)
2. Cut-and-fill slopes shall not be steeper than 2:1. Where the slope is to be mowed, the slope shall be no steeper than 3:1 (4:1 is preferred because of safety factors related to mowing steep slopes).
3. Reverse slope benches or diversions shall be provided whenever the vertical interval (height) of any 2:1 through 5:1 slope exceeds 15

feet. Benches shall be located so as to divide the slope face as equally as possible and shall convey the water to a stable outlet. Soils, seeps, rock outcrops, etc., shall also be taken into consideration when designing benches.

- a. Benches shall be wide enough to accommodate the construction equipment in use and provide for ease of maintenance.
 - b. Benches shall be designed with a reverse slope of 5:1 or flatter to the toe of the upper slope and with a minimum of 1 foot in depth. Bench gradient to the outlet shall be between 1 and 2 percent.
 - c. The flow length within a bench shall not exceed 800 feet unless accompanied by appropriate design and computations. (See Standards for Diversion.)
4. Surface water shall be diverted from the face of all cut and fill slopes by the use of diversions, ditches, and swales, or conveyed downslope by the use of a designed structure, except where:
- a. The length of overland flow, in feet, to the crest of the slope shall not exceed the distance "A" given in the following diagram for any combination of side slopes and vertical intervals;
 - b. The face of the slope is or shall be stabilized and the face of all graded slopes shall be protected from surface runoff until they are stabilized; and,
 - c. The face of the slope shall not be subjected to any concentrated flows of surface water from natural drainageways, graded swales, downspouts, etc.

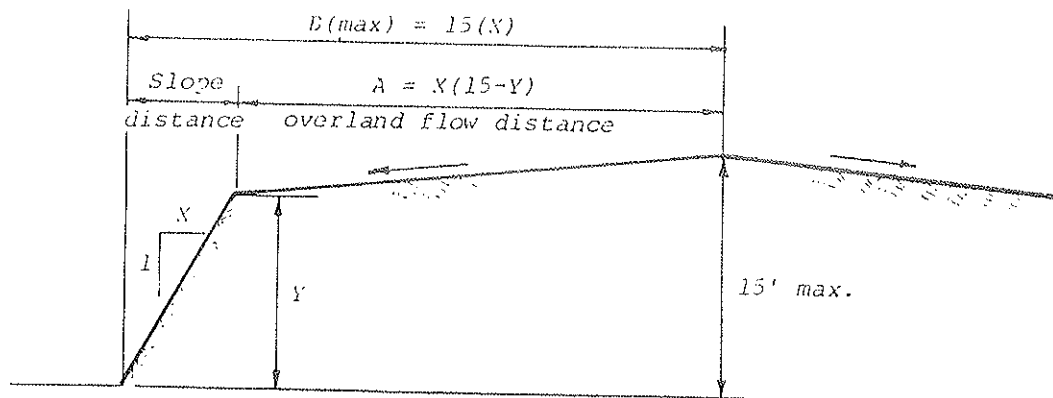
The maximum total horizontal overland flow and slope distance (B) shall not exceed 15 times the side slope (X) of the cut or fill slope. Maximum allowable overland flow distance, in feet, to the top of the slope with no diversion of surface water will be determined by use of the formula $A = X(15 - Y)$, where:

A = maximum overland flow distance, in feet, to slope crest;

B = maximum horizontal distance, in feet (shall not exceed 15X);

X = side slope, or the horizontal distance in feet to 1 foot vertical; and

Y = vertical interval, or the height of cut or fill slope in feet measured vertically from bottom elevation of slope to slope crest.



TYPICAL SECTION OF A SLOPE

FIGURE 11-4

Example: Determine the maximum allowable overland flow distance, A, for a 3:1 side slope with a vertical interval of 7 feet.

Given: $X = 3$; $Y = 7$

$$A = X(15 - Y)$$

$$A = 3(15 - 7)$$

$$A = 24 \text{ feet}$$

If maximum allowable overland flow is exceeded, surface water shall be diverted from the slope face and carried to a stable outlet, or conveyed downslope with a designed structure.

5. Serrated cut slopes shall be constructed so as to facilitate long-lasting vegetative stabilization. These serrations shall be made in rippable rock with conventional equipment as the excavation is made. Each step or serrate shall be constructed on the contour and will have steps cut at nominal 2-foot intervals with nominal 3-foot horizontal shelves. These steps will vary depending on the slope ratio of the cut slope. The normal slopeline is 1.5:1. These steps will weather and act to hold moisture, lime, fertilizer, and seed, and to produce a much quicker and longer-lived vegetative cover and slope stabilization. Overland flow shall be diverted from the top of all serrated cut slopes and carried to a suitable outlet.

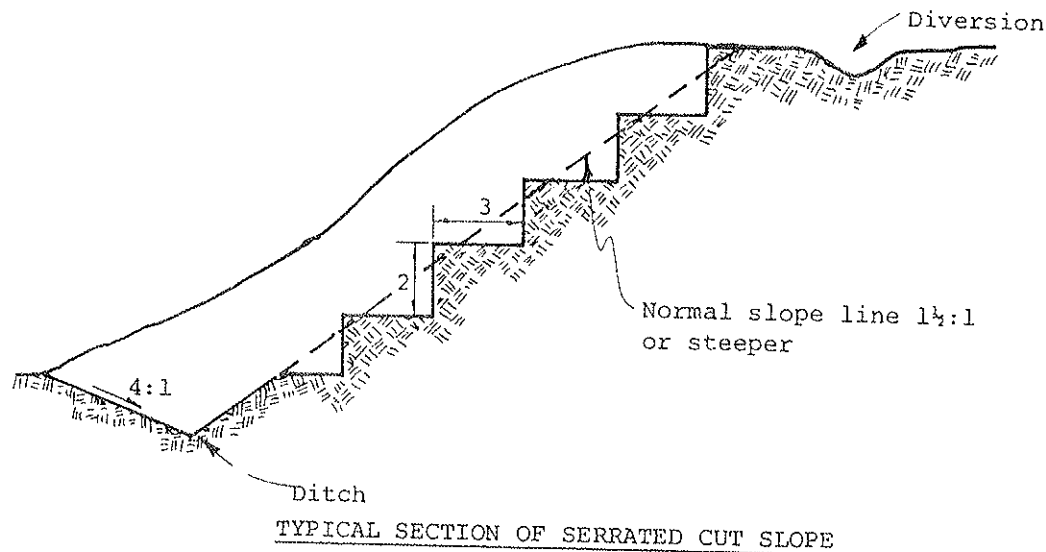


FIGURE 11-5

6. Subsurface drainage shall be provided where necessary to intercept seepage that would otherwise adversely affect slope stability or create excessively wet site conditions that would hinder or prohibit vegetative establishment.
7. Slopes shall not be created so close to property lines as to endanger adjoining properties without adequately protecting such properties against erosion, slippage, settlement, subsidence, or other related damages.
8. Material for earth fills shall be obtained from designated areas. Except for approved landfills, the fill material shall be free of brush, rubbish, rocks, logs, stumps, building debris, and other objectionable material that would interfere with or prevent construction of satisfactory fills. It should be free of stones over 2 inches in diameter where compacted by hand or mechanical tampers, or over 6 inches in diameter where compacted by rollers or other equipment. Frozen material shall not be placed in the fill nor shall the fill material be placed on a frozen foundation.
9. Stockpiles, borrow areas, and spoil areas shall be shown on the plans and shall be subject to the provisions of this Standard.

11.80 Bibliography

1. Meyer, L.D., "Unpublished Notes," Agricultural Engineer, Agricultural Research Service, USDA, Sedimentation Laboratory, Oxford, MS.

2. Interstate Commission on the Potomac River Basin, "Urban Sediment Can be Controlled." Proceedings of Winter Meeting, February 17-18, 1966.
3. U.S. Department of Interior, Geological Survey. "Sediment Movement in an Area of Suburban Highway Construction, Scotts Run Basin, Fairfax County, VA, 1961-64 Water Supply Paper No. 1541-E 1960.
4. U.S. Department of Housing and Urban Development. "Proceedings of the National Conference on Sediment Control, Washington, DC, September 14-16, 1969." May 1970.
5. E.P.A., Office of Water Programs, Washington, DC, "Control of Sediments Resulting from Highway Construction and Land Development."
6. U.S. Department of Interior, Geological Survey, "Hydrology for Urban Land Planning - A Guidebook on Hydrologic Effects of Urban Land Use." Circular 554, 1968.
7. USDA, Agricultural Research Service, "Predicting Rainfall-Erosion Losses from Cropland East of the Rocky Mountains," Agriculture Handbook No. 282, 1965.
8. American Association of State Highway and Transportation Officials, "Standard Specifications for Highway Materials," Washington, DC, 1970.
9. DA-SCS, "Standards and Specifications No. 378, Ponds," SCS Field Office Technical Guide.
10. Brune, Gunnar M., "Trap Efficiency of Reservoirs," Transactions of the American Geophysical Union, Vol. 34, No. 3, pp. 407-418, June 1953.
11. Boysen, Stephen M., "Predicting Sediment Yield in Urban Areas," Proceedings, National Symposium on Urban Rainfall and Runoff and Sediment Control, University of Kentucky, College of Engineering, Lexington, KY, 1974.
12. USDA-SCS, "Engineering Field Manual for Conservation Practices," 1969.
13. USDA-SCS "Handbook of Channel Design for Soil and Water Conservation," Technical Paper No. 61, June 1954.
14. Fletcher, B.P. and Grace, J.L., Jr., "Practical Guidance for Estimating and Controlling Erosion at Culvert Outlets," U.S. Army Corps of Engineers, Waterways Experiment Station, Misc. Paper No. H-72-5, May 1972.
15. National Academy of Sciences, Highway Research Board, "Tentative Design Procedure for Riprap-Lined Channels," National Cooperative Highway Research Program Report No. 108, 1970.
16. USDA, Soil Conservation Service, Field Office Technical Guides.
17. Mulches for Wind and Water Erosion Control, USDA, ARS 41-84, July 1963.

18. Applied Mulches and Mulching, USDA, ARS Special Report, ARS 22-71, August 1971.
19. USDA, Soil Conservation Service Agriculture Handbook 296, "Land Resource Regions and Major Land Resource Areas of the United States."
20. Guideline Specifications for Sodding. American Sod Producers Association, Inc., New Brunswick, NJ.
21. Growing Ground Covers, USDA Home and Garden Bulletin No. 175.
22. State of New York, "The Conservationist," April-May 1971, pp. 22-27.
23. Agricultural Information Bulletin 285, "Protecting Trees Against Damage from Construction Work," Superintendent of Documents, U.S. Government Printing Office, Washington, DC. 20402.
24. Guidelines for the Control of Erosion and Sediment in Urban Areas of the Northeast, USDA, Soil Conservation Service, Upper Darby, Pennsylvania, 1970.
25. Jones, J.N., Armiger, W.H., and Bennett, O.L., "A Two-Step System for Revegetation of Surface Mine Spoils," Journal of Environmental Quality, Vol. 4, No. 2, 1975, pp. 233-235.
26. Agriculture Handbook No. 60, "Diagnosis and Improvement of Saline and Alkali Soils," by United States Salinity Laboratory Staff.
27. "How to Treat Salt-Damaged Land," University of Maryland, Agronomy Department, Agronomy Mimeo No. 8, Rev., May 1962.
28. "Wind Erosion Forces in the United States and Their Use in Predicting Soil Loss," Agricultural Handbook 346.
29. "How to Control Wind Erosion," USDA-ARS, Agriculture Information Bulletin 354.
30. Soil Conservation Society of America, "Resource Conservation Glossary," 1970.
31. "Hydraulic Design of Stilling Basins and Energy Dissipators," Engineering Monograph No. 25, United States Department of Interior, Bureau of Reclamation, U.S. Government Printing Office, Denver, CO.

Section 12

Design Examples

- 12.10 General
- 12.20 Street Drainage
 - Gutter Flow
 - Minor Storm
 - Major Storm
- 12.30 Inlet Design
 - Inlets on a Continuous Grade
 - Curb-Opening Inlet
 - Grate Inlet
 - Inlet Spacing
 - Inlets in a Sump Condition
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- 12.40 Storm Sewer Design by the Rational Method
- 12.50 Detention Basin Design by the SCS Unit Hydrograph Procedure
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 - Standard Culvert
 - Inlet Control
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Section 12

Design Examples

The following examples represent a simplified version of a typical Subdivision Drainage Design. The examples are intended to demonstrate the specific design method presented in Sections 2 through 11. These examples also show the interrelationship between the various methods in design applications. For a detailed discussion of these methods, consult the appropriate sections of this manual and their respective bibliographies.

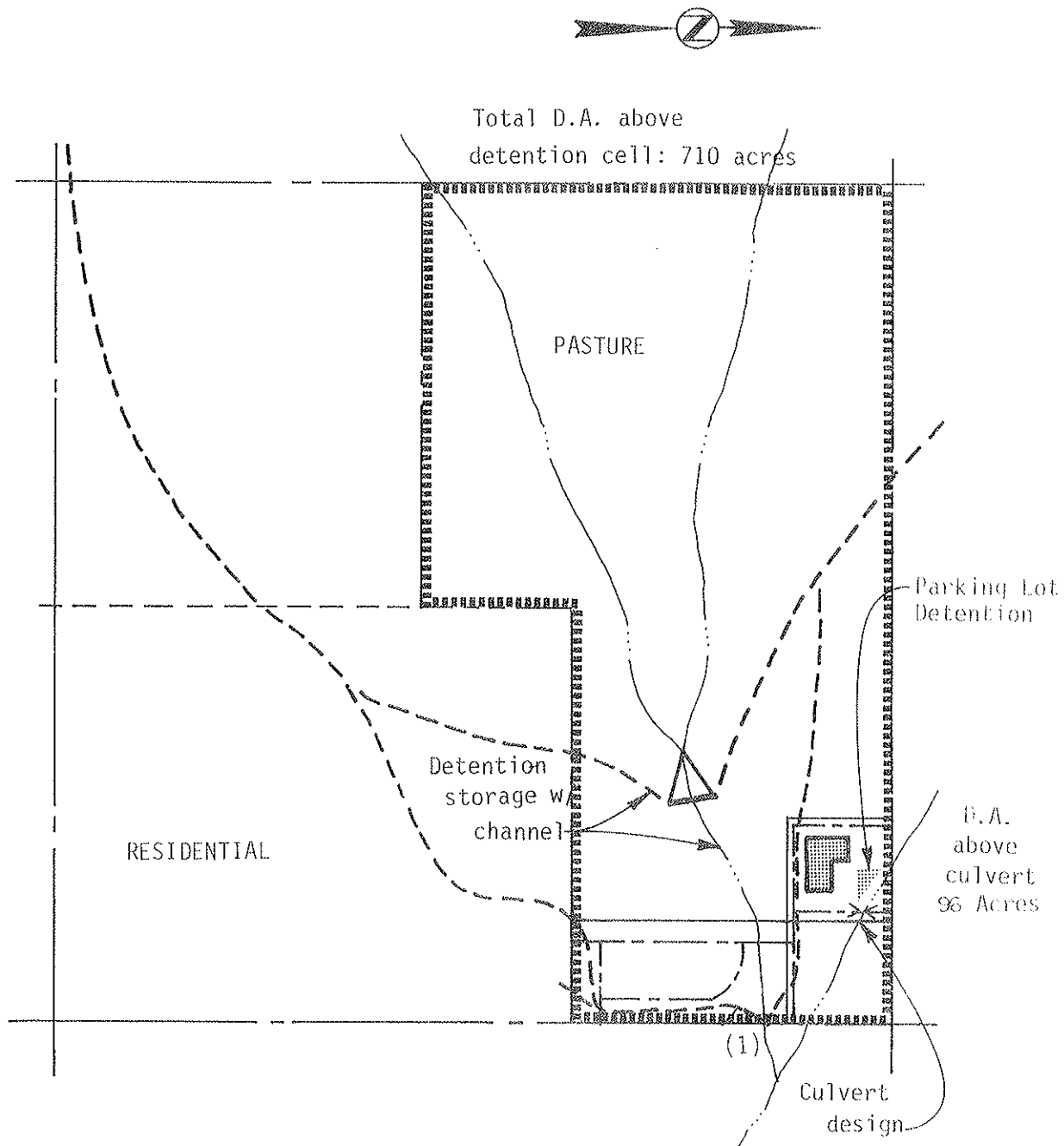
12.10 General

A developer owns 300 acres which he intends to develop in segments over the next 10 years. The first area to be developed will be a 30 acre tract containing 20 acres of single family residential lots of approximately 11,250 square feet and a 10 acre shopping center.

Due to limited resources, the developer intends to use stormwater management techniques to limit runoff and minimize his overall expenditures for drainage facilities.

Figure 12-1 shows the drainage basin, the proposed development, and drainage facilities. The drainage plan utilizes a detention pond to reduce the peak runoff from the majority of the basin, thus reducing the size of required trunk sewer through the proposed development. The proposed shopping center will utilize parking lot detention and roof storage to reduce peak runoff.

Figure 12-2 shows the proposed residential development layout, with appropriate dimensions and design street grades.



D.A. above pt (1) = 810 acres

Figure 12-1 Overall Site Plan

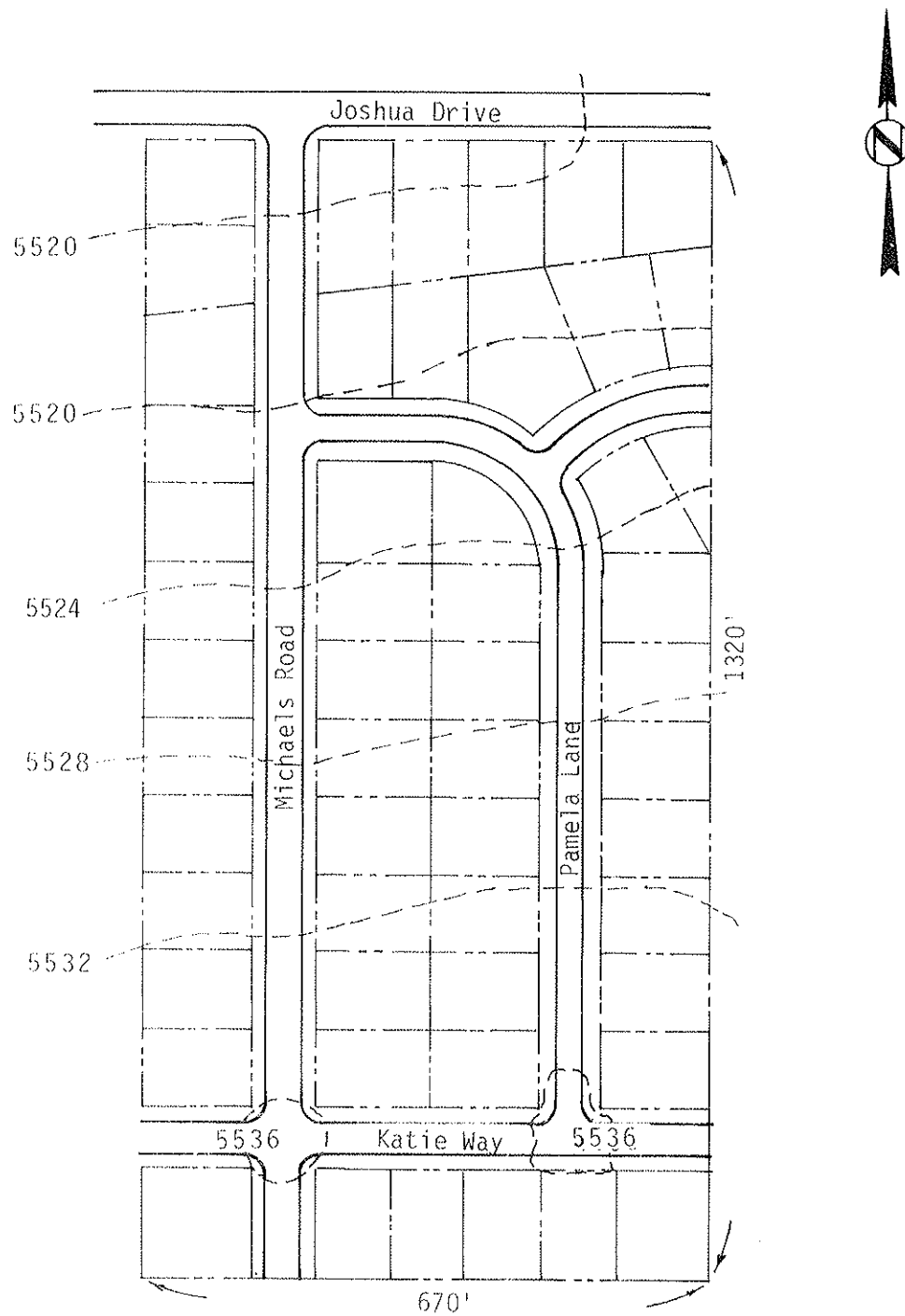


Figure 12-2 Sub-division Layout

12.20 Street drainage

Michaels Road is a 36' wide collector street on a 2% grade, and drains a uniform width of 330' along its length. Katie Way is also a 36' wide collector, but is graded at 0.8% and -0.8%, creating a sump curve in the middle of the block. It drains a total of 1.82 acres, with 60% contributing to the east inlet. Pamela Lane is a 25' wide local street, and drains a width of 280' through its length.

Using the criteria of Section 2 and 3, design the surface drainage system for these streets.

Gutter flow

Minor Storm

Michaels Road

Given: 6" vertical curb
2% pavement cross slope, S_x
36' street width
2% longitudinal slope, S_o

Find: Gutter flow for the minor storm of 10 year frequency.

1. Determine the allowable pavement encroachment
Use Table 3-1: one 12' lane must remain free
 $36' - 12' = 24'$

$\therefore$ encroachment = 12' per side

2. Calculate the capacity for each gutter.
Use Figure 3-2

$$n = 0.016$$

$$S_x = S_y = 0.02$$

$$T = 12 \text{ ft}$$

$$d = TS_x = 12(0.02) = 0.24 \text{ ft}$$

$$\therefore \underline{Q = 5.5 \text{ cfs}}$$

3. Using the same procedure for Katie Way and Figure 3-2

$$n = 0.016$$

$$S_x = 0.02$$

$$S_o = 0.008$$

$$T = 12 \text{ ft}$$

$$d = TS_x = 12(0.02) = 0.24 \text{ ft}$$

$$\therefore \underline{Q = 3.5 \text{ cfs}}$$

Pamela Lane

Given: 6" vertical curb
2% pavement cross slope, S_x
25' wide street
2% longitudinal slope, S_o

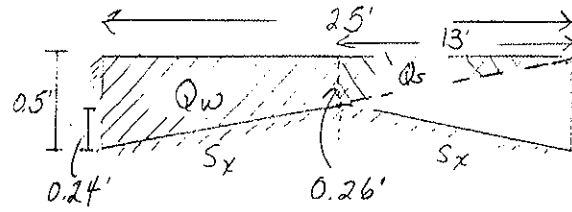
Find: Gutter flow for the minor storm

1. Determine pavement encroachment

Use Table 3-1: Flow may not overtop curb
(25'-1') $\div$ 2 = 12' encroachment

2. Calculate the capacity for each gutter. - Composite sections
Use Figure 3-2 and Figure 3-3

$$\begin{aligned}
 n &= 0.016 \\
 S_o &= S_x = 0.02 \\
 y_{max} &= 0.50 \text{ ft} \\
 d &= TS_x = 0.02(12) = 0.24
 \end{aligned}$$



$$\begin{aligned}
 \text{When: } T &= 13 \text{ ft} \\
 S_o &= S_x = 0.02
 \end{aligned}$$

$$Q_t = Q_w + Q_s$$

$$\text{Then: } Q_s = 6.8 \text{ cfs}$$

$$\text{Figure 3-3; } \frac{W}{T} = \frac{12}{25} = 0.48$$

$$\text{Then } E_o = 0.83$$

$$Q_t = \frac{Q_s}{1 - E_o} = \frac{6.8}{1 - 0.83} = 40 \text{ cfs}$$

$$Q_w = Q_t - Q_s$$

$$Q_w = 40 - 6.8 = \underline{\underline{33.2 \text{ cfs}}}$$

Major Storm

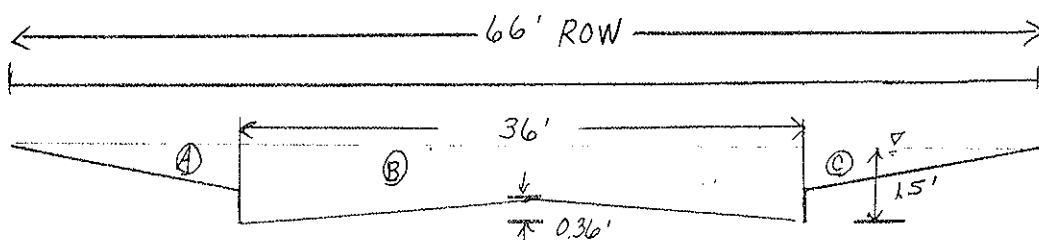
Given: 8 Acres residential development
 Property lines of 66' R.O.W. are one foot
 higher than curb; minimum along all streets

Find: Encroachment due to major storm

1. Allowable depth of flow

Use Table 3-2 : 18" above gutter flowline for
 local and collector streets

2. Street capacity: collectors
 Use Mannings



3. Assume areas (A) and (C) (parking) are non-effective
 for flow, due to obstructions

4. Major storm discharge

$$Q = \frac{1.49}{n} A R^{2/3} S_0^{1/2} \quad (\text{eq. 7-1})$$

$$A = 36(1.5) - \frac{1}{2}(0.36)(18) \times 2 = 47.52 \text{ ft}^2$$

$$R = \frac{A}{P} = \frac{47.52 \text{ ft}^2}{[(1.5)^2 + 36]} = 1.22 \text{ ft}$$

$$Q = \frac{1.49}{0.016} (47.52) (1.22)^{2/3} (0.02)^{1/2}$$

$$Q = \underline{714.5 \text{ cfs}}$$

5. Area needed to exceed major storm discharge
100 yr, 5 min storm, $t_c = 5 \text{ min.}$

$$i = 10.20 \text{ in/hr (Figure 2-1)}$$

$$C = 0.52 \quad (\text{Table 2-1})$$

$$Q = C_f C i A \quad (\text{Eq 2-1})$$

$$A = \frac{714.5 \text{ cfs}}{(1.25)(0.52)(10.2 \text{ in/hr})}$$

$$A = \underline{107.8 \text{ acres}}$$

But DA above streets (8a) < 108 acres; therefore,
street has sufficient capacity

12.30 Inlet design

Using the gutter capacities for Michaels Road, Katie Way, and Pamela Lane developed previously, determine inlet locations for continuous grade and sag conditions.

Use the criteria presented in Sections 2 and 4.

Inlets on a Continuous Grade

Curb-Opening Inlet

Given: Michaels Road gutter flow 5.5 cfs / gutter

$$n = 0.016$$

$$S_o = 0.02$$

$$S_x = 0.02$$

$$T = 12 \text{ ft}$$

$$W = 2 \text{ ft}$$

Find: Inlet length for curb-opening inlet for various interception rates

Use equations listed in table 4-1

$$\begin{aligned} 1. F_w &= 16.4 [(T-2)S_x]^{1/6} S_o^{1/2} \\ &= 16.4 [(10)(0.02)]^{1/6} (0.02)^{1/2} \\ &= 1.77 \end{aligned}$$

$$2. F_w T = 21.28$$

$$3. L_1 = 2.49 S_x^{0.3} F_w T = 2.49 (0.02)^{0.3} (21.28) = 16.39$$

$$L_2 = 3.27 S_x^{0.5} F_w T = 3.27 (0.02)^{0.5} (21.28) = 9.84$$

$$L_3 = 1.65 F_w T = 1.65 (21.28) = 35.11$$

4. $Q_2 \cdot \frac{L_2}{L_1} Q_1 = 3.30$

5. Try $Q_i/Q = 1.00, 0.8, 0.6$

$$\frac{Q_i}{Q} = \left(\frac{L_i}{L_3} \right)^{0.4} \quad \text{for } Q_i > Q_c, L_i > L_2$$

rewritten as;

$$\frac{L_i}{L_3} = \left(\frac{Q_i}{Q} \right)^{2.5}$$

$\frac{Q_i}{Q}$	$\frac{L_i}{L_3}$	L_i
1.0	1.0	35.11
0.8	0.572	20.1
0.6	0.279	9.79 < L_2
0.605	0.285	10.0
0.65	0.342	12.0

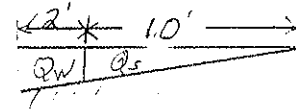
A 9.84' inlet would provide the most efficient inlet. However use standard 10' inlet for economy.

6. For $L_1 = 10.1$ $\frac{Q_i}{Q} = 0.605$

$$\therefore Q_i = 0.605(5.5) = \underline{\underline{3.33 \text{ cfs}}}$$

Grate Inlet

Check grate inlet for same location
 Use 24" wide by 48" grates
 60% clear opening and 3" bar depth



a) Flow depth at curve
 $d = TS_x = 0.24 \text{ ft}$

b) Flow depth at outside edge of grate
 $d' = 10(0.02) = 0.20 \text{ ft}$

c) Gutter flow

Figure 2-2

$T = 12 \text{ ft}$ then $Q_t = 5.5 \text{ cfs}$

$T = 10 \text{ ft}$ then $Q_s = 3.4 \text{ cfs}$

$$Q_w = Q_t - Q_s$$

$$= 5.5 - 3.4$$

$$Q_w = 2.1 \text{ cfs} ; \text{ gutter discharge}$$

d) Grate efficiency

$$\frac{2.1 \text{ cfs}}{5.5 \text{ cfs}} = 0.38 = \underline{\underline{38\%}}$$

e) Since curb-opening inlet captures 61% of flow, compared to 38% of flow by grate, the curb-opening is more efficient.
Assuming costs are equal, use curb-opening.

Inlet Spacing

check inlet spacing for curb-opening - Michaels Road
Inlet design using Rational Formula

Assume $t_c = 5 \text{ min}$ $i = 7.2 \text{ in/hr}$ (Figure 2-1)
 $L = 330'$ 10 yr storm

First Inlet

$$A = \frac{Q}{C i} \quad Q \text{ is max allowable gutter flow}$$

$$A = \frac{5.5(2)}{7.2(0.52)} = 2.94 \text{ acres} = 127,980 \text{ ft}^2$$

$$\text{Spacing} = \frac{\text{Area}}{\text{width}} = \frac{127,980 \text{ ft}^2}{330 \text{ ft}} = 388 \text{ ft} \quad \text{use } \underline{\underline{400 \text{ ft}}}$$

Second and following inlet spacing

$$A = \frac{Q_i}{C i} \quad Q_i \text{ is max amount to pick-up between inlets (p. 12-12-C)}$$

$$A = \frac{2.1(2)}{(7.2)(0.52)} = 1.12 \text{ acres} = 48,865 \text{ ft}^2$$

$$\text{spacing} = \frac{\text{Area}}{\text{width}} = \frac{48,865}{330} = 148 \text{ ft} \quad \text{use } \underline{\underline{150 \text{ ft}}}$$

Inlet in a Sag Condition

Minor Storm Grate Inlet
Katie Way

Given: Allowable curb depth for minor storm 0.31'
Total drainage area = 1.82 acres
60% to first inlet
 $t_c = 5$ min.

Find: Inlet type and size

1. Total runoff

$$\begin{aligned} Q &= CiA \\ &= 0.52 (7.2) (1.82) \\ &= 6.8 \text{ cfs or } \underline{\underline{4.1 \text{ cfs for first inlet}}} \end{aligned}$$

2. Capacity of (24" x 48") grate inlet

$$Q_i = 3.0 P d^{1.5} \quad (\text{Egn 4-2})$$

$$Q_i = 3.0 \left(\frac{24(48)}{144} \right) (0.31)^{1.5}$$

$$Q_i = 4.14 \text{ cfs} > 4.1 \text{ cfs} \quad \text{OK}$$

Minor storm curb opening Inlet

1. Capacity of curb opening inlet $W = 2'$, $a = 2''$

Try 8' Inlet

$$Q = 3.0 K_i d_i^{1.5} (E_g - 4) d_i = 0.48 \quad (0.31' + 2'')$$

$$Q = 3.0(8)(0.48)^{1.5}$$

$$= \underline{\underline{8.0 \text{ cfs}}} \quad \text{OK}$$

Major Storm

1. Provide for the 100 yr flood from sag area

$$Q_{100} = C_i A c f$$

$$Q_{100} = 0.52(1.25)(10.2)(1.82) = 12.1 \text{ cfs}$$

2. Subtract storm water design capacity

$$12.1 \text{ cfs} - 6.8 \text{ cfs} = 5.30 \text{ cfs} \quad \text{must go overland}$$

3. Try Property line swale with 2' bottom and 3 to 1 side slopes and 1 foot depth

$$\text{For: } y = 0.75 \text{ ft}, S = 0.01, n = 0.027, z = 3, b = 2$$

$$A = by + zy^2 = 3.19 \text{ ft}^2$$

$$P = b + 2y\sqrt{1+z^2} = 6.74 \quad R = \frac{A}{P} = 0.47$$

Manning - Egn. (7-1)

$$Q = \frac{1.49}{n} A R^{2/3} S^{1/2} = \frac{1.49}{0.027} (3.19)(0.47)^{2/3} (0.01)^{1/2} = \underline{\underline{10.7 \text{ cfs}}}$$

channel OK

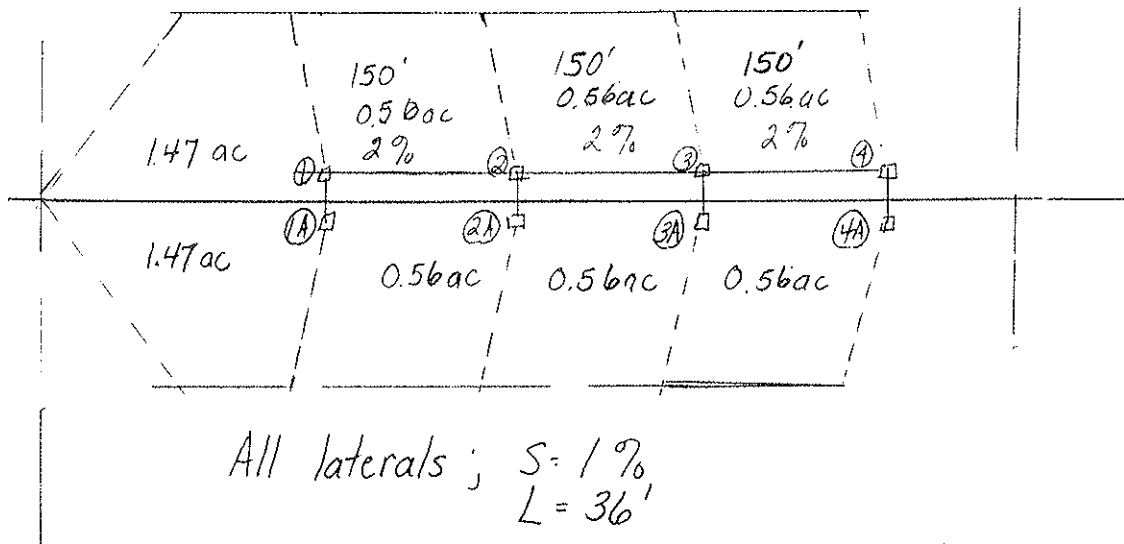
Project _____ Computed _____

Subject _____ Date _____ Sht. 12-16 of _____

12-40 Storm sewer design

Given: Inlet locations as previously determined
Runoff coefficient 0.52, $n=0.013$
Drainage areas as shown below
12" minimum pipe size
10 year design frequency

Find: Storm sewer sizes for Michaels Road



$$Q = ACi$$

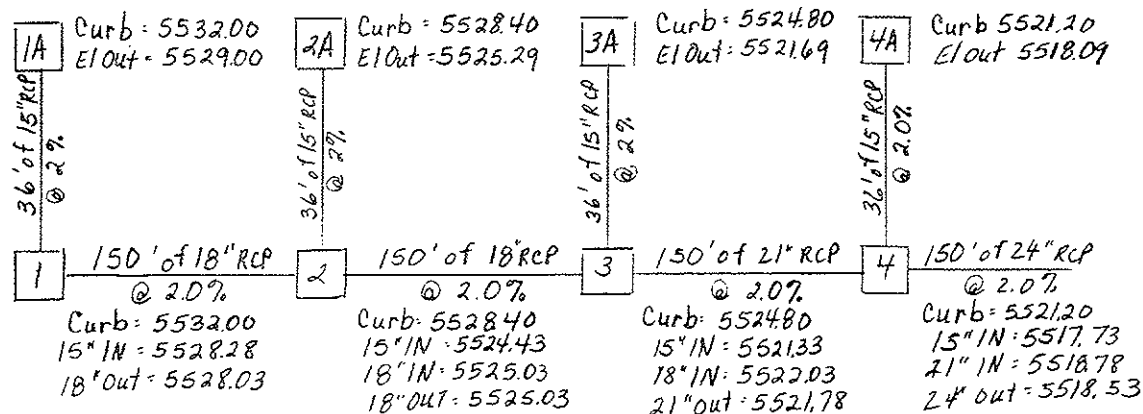
A - acres
I - min
I - inches/hr.
Q - cfs

COMPUTATION FORM DRAINAGE & STORM SEWER DESIGN RATIONAL METHOD

Project Michaels Road 12-17
Comp. by _____ date _____
Ck by _____ date _____

LOCATION			A		C	TIME OF CONCENTRATION		i	Q	DESIGN					PROFILE				
STREET	FROM	TO	INCRE- MENT	TOTAL		TO INLET	PIPE OR CHANNEL			SIZE	SLOPE %	n	CAP. FULL	V	LENGTH FT.	FALL FT.	OTHER LOSSES	UPPER INV EL.	LOWER INV EL.
Main	1A	1	1.47	1.47	0.52	5	0	7.2	5.5	15	2	0.013	9.1	7.5	36	0.72	-	5529.00	5528.28
Main	1	2	1.47	2.94	0.52	5	0.1	7.2	11.0	18	2	0.013	14.8	8.4	150	3.0	0.25	5528.03	5525.03
Main	2	3	1.12	4.06	0.52	5	0.4	7.1	14.2	18	2	0.013	14.8	8.4	150	3.0	-	5525.03	5522.03
Main	3	4	1.12	5.18	0.52	5	0.7	7.1	19.1	21	2	0.013	22.3	9.3	150	3.0	0.25	5521.78	5518.78
Main	4	5	1.12	6.30	0.52	5	1.0	6.9	22.7	24	2	0.013	32.0	10.2	150	3.0	0.25	5518.53	5515.53
Lateral	2A	2	0.56		0.52	5	-	7.2	2.1	15	1	0.013	9.1	7.5	36	0.36	-	5525.29	5524.43
Lateral	3A	3	0.56		0.52	5	-	7.2	2.1	15	1	0.013	9.1	7.5	36	0.36	-	5521.69	5521.33
Lateral	4A	4	0.56		0.52	5	-	7.2	2.1	15	1	0.013	9.1	7.5	36	0.36	-	5518.09	5517.73

Final design for sewer system on Michaels Road



Outfall EL = 5515.53

12.50 Detention Basin Design by the SCS based Hydrograph Procedure

In order to limit the amount of runoff flowing through the planned development, thus minimizing the size of the trunk sewer, the developer has decided to build a small detention pond above the subdivision.

The developer owns approximately 200 acres of the 710 acre drainage area controlled by the detention pond. The area is currently used for grazing purposes. Within the next 10 years, the developer will be constructing addition subdivisions, but will attempt to maintain the existing drainage channel in their natural conditions.

Minor channel improvements (realignment and cleaning) and planting of moderate channel vegetation are anticipated.

However, 510 acres of the drainage area are not owned by the developer. Current and future zoning maps are consulted to determine the development within the region. The zoning maps show 510 acres are residential areas, similar to the present residential area. Thus the entire area is considered for future residential development conditions.

Given: Drainage area = 710 acres = 1.11 mi^2
Channel length = 1.94 miles
Slope = $0.021 \text{ ft/ft} = 112.9 \text{ ft/mile}$

Land use within planned development of 200 acres

- a) Residential 55% ($\frac{1}{3}$ acre avg size)
- b) Commercial 5%
- c) Streets 15%
- d) Open space 25%

Use the "B" Hydrologic Soil Group and Type II soil moisture condition

Find: Development of imperviousness
Natural and ultimate 10 year peak discharge
Ultimate 100-year peak storage
Unit and storm hydrograph
Required storage

1) Imperviousness, I

Use Table 2-6

Residential - $\frac{1}{3}$ acres	30%	$\times 0.55 =$	16.5%
Commercial	85%	$\times 0.05 =$	4.3%
Streets	100%	$\times 0.15 =$	15.0%
Open Space	0%	$\times 0.25 =$	0%
			<u>35.8%</u>

2) Curve Number (CN)

1. Use Table 2-5 "B" group - Type II

Residential - 1/3 acre	72
Commercial	92
Streets	98
Open Space - "good"	69

2. A weighted CN value is obtained

$$CN = 72(0.55) + 92(0.05) + 98(0.15) + 69(0.25)$$

$$CN = 76.2 \text{ - Use } 76$$

3) Excess Rainfall - Losses

$$S = \frac{1000}{CN} - 10 \quad \text{potential maximum retention}$$

$$S = \frac{1000}{76} - 10 = 3.2$$

$$I_a = 0.25 = 0.2(3.2) = 0.63 \quad \text{Initial abstraction}$$

4) Lag time and time of concentration

$$L = 0.49 \left(\frac{l}{\sqrt{S}} \right)^{0.5} (I)^{-0.57} \quad \text{Putnam Eq (2-11)}$$

$$L = 0.49 \left(\frac{1.94}{\sqrt{1129}} \right)^{0.5} (0.358)^{-0.57}$$

$$L = 0.3760 \text{ hrs}$$

$$T_c = \frac{5}{3} L = \frac{5}{3} (0.376) = 0.63 \text{ hr} = 37.6 \text{ min} \quad \text{Eq (2-13)}$$

- 5) Compute the design storm rainfall (P)
 Use a design storm of 6 hours from Table 2-4
 the rainfall is 5.1 inches

- 6) Compute the SCS unit hydrograph duration (D),
 time base (T_b) and the time to peak (T_p)

$$D = 0.133 T_c = 5.0 \text{ min} \quad \text{Eq (2-16)}$$

$$T_p = \frac{2}{3} T_c = 25.07 \text{ min} = 0.42 \text{ hr} \quad \text{Eq (2-14)}$$

$$T_b = 5 T_p = 125 \text{ min} \quad \text{Eq (2-15a)}$$

- 7) Compute the unit hydrograph discharge (q_p).

$$q_p = \frac{484A @}{T_p} = \frac{484 (1.11) (1)}{0.42} = 1280 \text{ cfs/in} - \text{direct runoff}$$

- 8) Compute the design storm distribution (See Table 12-1)
 10 yr storm $P = 3.4$ inches

Runoff does not begin until 160 minutes and
 the direct runoff is greater than the initial
 abstraction (0.63 in)

Project _____ Computed _____

Subject _____ Date _____ Sht. 12-23 of _____

TABLE 12-1

ULTIMATE 10 YR DESIGN STORM CONSTRUCTION - KENT

TIME ' (MINUTES)	P_x/P_{24}^2	P_x^3 (in)	CUMULATIVE DIRECT RUNOFF(in)	INCREMENTAL DIRECT RUNOFF(in)
0	0	0	0	0
10	0	0	0	0
20	0.014	0.05	0	0
30	0.022	0.07	0	0
40	0.031	0.11	0	0
50	0.040	0.14	0	0
60	0.048	0.16	0	0
70	0.056	0.19	0	0
80	0.068	0.23	0	0
90	0.080	0.27	0	0
100	0.093	0.32	0	0
110	0.107	0.36	0	0
120	0.120	0.41	0	0
130	0.138	0.47	0	0
140	0.158	0.54	0	0
150	0.181	0.62	0	0
160	0.217	0.74	.003	.003
170	0.311	1.06	.05	.05
180	0.663	2.25	.55	.50
190	0.736	2.50	.69	.14
200	0.788	2.68	.80	.11
210	0.820	2.79	.87	.07
220	0.843	2.87	.92	.05
230	0.863	2.93	.96	.04
240	0.880	2.99	1.00	.04
250	0.895	3.04	1.04	.04
260	0.908	3.09	1.07	.03
270	0.921	3.13	1.10	.03
280	0.927	3.15	1.11	.01
290	0.937	3.19	1.14	.03
300	0.952	3.24	1.17	.03

310	0.963	3.27	1.20	.03
320	0.972	3.30	1.22	.02
330	0.980	3.33	1.24	.02
340	0.987	3.36	1.26	.02
350	0.994	3.38	1.27	.01
360	1.000	3.40	1.29	.02

¹Storm duration of 6 hr

²From Fig 2-4

³For rainfall of 34 inches

⁴From Fig 2-5 or Eq 2-6

TABLE 12-2

ULTIMATE 100 YR DESIGN STORM CONSTRUCTION - KENT

TIME ¹ (minutes)	P_x/P_{24}^2	P_x^3 (in)	CUMULATIVE DIRECT RUNOFF(in)	INCREMENTAL DIRECT RUNOFF(in)
0	0	0	0	0
10	0	0	0	0
20	0.014	0.07	0	0
30	0.022	0.11	0	0
40	0.031	0.16	0	0
50	0.040	0.20	0	0
60	0.048	0.24	0	0
70	0.056	0.29	0	0
80	0.068	0.35	0	0
90	0.080	0.41	0	0
100	0.093	0.47	0	0
110	0.107	0.55	0	0
120	0.120	0.61	0	0
130	0.138	0.70	.002	.002
140	0.158	0.81	.01	.01
150	0.181	0.92	.02	.01
160	0.217	1.11	.06	.04
170	0.311	1.59	.22	.16
180	0.663	3.38	1.27	1.05
190	0.736	3.75	1.54	.27
200	0.788	4.02	1.75	.21
210	0.820	4.18	1.87	.12
220	0.843	4.30	1.96	.09
230	0.863	4.40	2.04	.08
240	0.880	4.49	2.11	.07
250	0.895	4.56	2.17	.06
260	0.908	4.63	2.23	.05
270	0.921	4.70	2.28	.05
280	0.927	4.73	2.31	.03
290	0.937	4.78	2.35	.04
300	0.952	4.86	2.41	.06

Project _____ Computed _____

Subject _____ Date _____ Sht. 7-25 of _____

310	0.963	4.91	2.45	.04
320	0.972	4.96	2.49	.04
330	0.980	5.00	2.53	.04
340	0.987	5.03	2.55	.02
350	0.994	5.07	2.58	.03
360	1.000	5.10	2.61	.03

¹ STORM DURATION OF 6 HOURS.

² FROM FIG. 2-4

³ FOR RAINFALL OF 5.1 INCHES.

⁴ FROM FIG. 2-5 OR EQ. 2-6

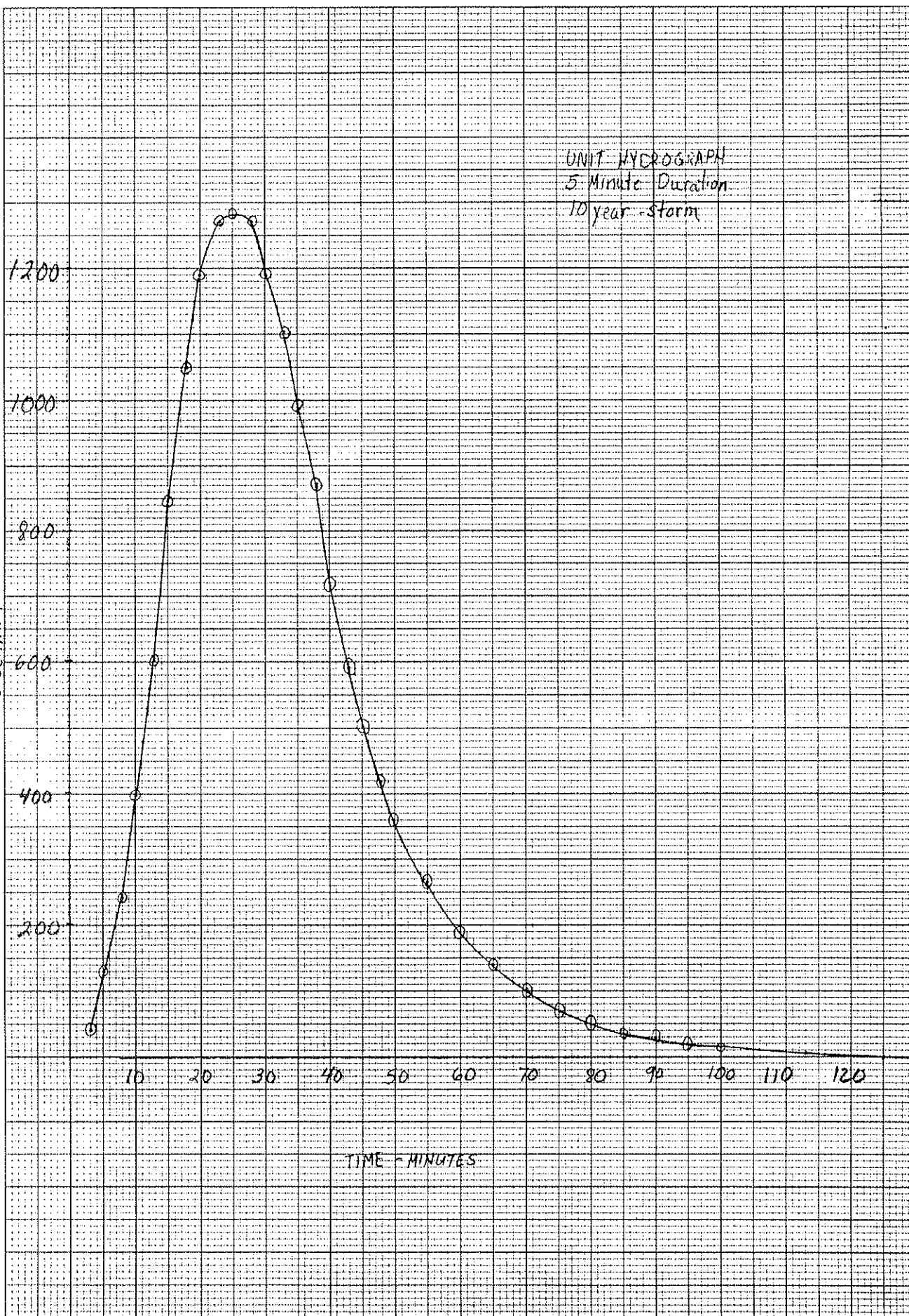
9) Unit hydrograph construction

Using the values of t/T_p and g/g_p in Table 2-8
 and the computed value of T_p and g_p ($T_p = 25 \text{ min}$, $g_p = 1280 \text{ cfs/in}$)
 5 minute Unit hydrograph

t/T_p	t (min)	g/g_p	g cfs/in
0.0	0	0.00	0
0.1	3	0.03	38
0.2	5	0.10	128
0.3	8	0.19	243
0.4	10	0.31	397
0.5	13	0.47	602
0.6	15	0.66	845
0.7	18	0.82	1050
0.8	20	0.93	1190
0.9	23	0.99	1267
1.0	25	1.00	1280
1.1	28	0.99	1267
1.2	30	0.93	1190
1.3	33	0.86	1101
1.4	35	0.78	998
1.5	38	0.68	870
1.6	40	0.56	717
1.7	43	0.46	589
1.8	45	0.39	499
1.9	48	0.33	422
2.0	50	0.28	358
2.2	55	0.207	265
2.4	60	0.147	188
2.6	65	0.107	137
2.8	70	0.077	99
3.0	75	0.055	70
3.2	80	0.040	51
3.4	85	0.029	37
3.6	90	0.021	27
3.8	95	0.015	19
4.0	100	0.011	14
4.5	113	0.005	6

DISCHARGE - CFS/IN

UNIT HYDROGRAPH
5 Minute Duration
10 year storm



TIME - MINUTES

10) The unit hydrograph ordinates are read in 10 minute intervals

<u>Time (min)</u>	<u>q (cfs/in)</u>
0	0
10	397
20	1190
30	1190
40	717
50	358
60	188
70	99
80	51
90	27
100	14
110	10
120	2

A direct runoff table is generated using the unit hydrograph ordinates. A runoff hydrograph resulted with a peak discharge of 843 cfs at 210 minutes. The computer hydrograph resulted in a peak discharge of 880 cfs at 205 minutes.

To _____ From _____ 12-30

Subject _____ Date _____

** HYDROGRAPH GENERATION USING 'SCS' METHOD **

WATERSHED NAME: KENT10
CURVE NUMBER = 76
WATERSHED AREA= 1.110 SQ. MILES
LENGTH OF THE MAIN CHANNEL= 10243 FEET
SLOPE OF THE WATERSHED= 2.10 PERCENT
IMPERVIOUS AREA= 35.80 PERCENT
HYDRAULIC LENGTH MODIFIED= 0.00 PERCENT
STORM RETURN PERIOD= 10 YEARS

LAG TIME= 0.3777 HOURS
TIME TO PEAK= 0.4197 HOURS
UNIT HYDROGRAPH PEAK= 1280.10 CFS/IN
UNIT HYDROGRAPH DURATION= 0.0839 HRS
TIME BASE OF UNIT GRAPH= 2.0984 HOURS

*** RUNOFF PARAMETERS ***

TIME TO PEAK= 205.00 MIN
PEAK DISCHARGE= 880.60 CFS
RUNOFF VOLUME= 76.56 ACRE-FT
= 1.29 INCHES

RUNOFF HYDROGRAPH

TIME	HYDROGRAPH		
0	0.00		
10	0.00	230	463.48
20	0.00	240	328.23
30	0.00	250	243.89
40	0.00	260	196.05
50	0.00	270	167.05
60	0.00	280	142.80
70	0.00	290	115.88
80	0.00	300	105.47
90	0.00	310	112.27
100	0.00	320	115.01
110	0.00	330	110.45
120	0.00	340	103.01
130	0.00	350	94.46
140	0.00	360	82.95
150	0.00	370	69.72
160	0.47	380	49.12
170	7.98	390	28.22
180	101.12	400	13.80
190	539.26	410	5.99
200	857.70	420	2.37
210	833.09		
220	646.95		

12-31

To _____ From _____

Subject _____ Date _____

** HYDROGRAPH GENERATION USING 'SCS' METHOD **

WATERSHED NAME: 100YR
 CURVE NUMBER = 76
 WATERSHED AREA= 1.110 SQ. MILES
 LENGTH OF THE MAIN CHANNEL= 10243 FEET
 SLOPE OF THE WATERSHED= 2.10 PERCENT
 IMPERVIOUS AREA= 35.80 PERCENT
 HYDRAULIC LENGTH MODIFIED= 0.00 PERCENT
 STORM RETURN PERIOD= 100 YEARS

LAG TIME= 0.3777 HOURS
 TIME TO PEAK= 0.4197 HOURS
 UNIT HYDROGRAPH PEAK= 1280.10 CFS/IN
 UNIT HYDROGRAPH DURATION= 0.0839 HRS
 TIME BASE OF UNIT GRAPH= 2.0984 HOURS

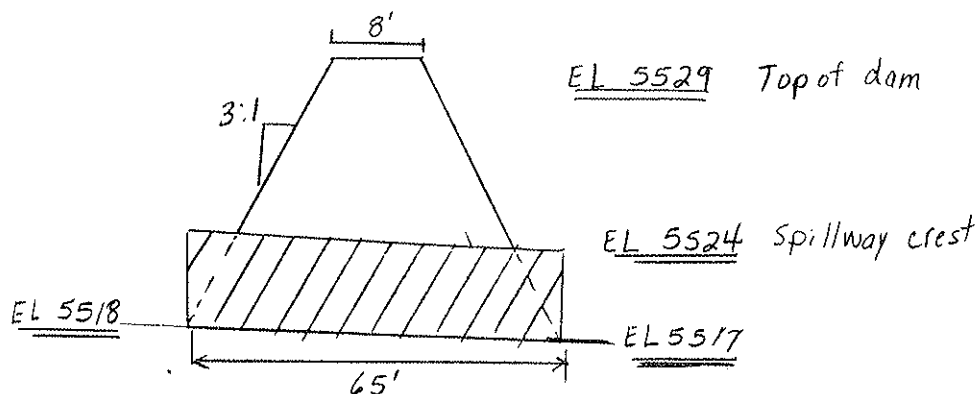
*** RUNOFF PARAMETERS ***

TIME TO PEAK= 205.00 MIN
 PEAK DISCHARGE= 1855.81 CFS
 RUNOFF VOLUME= 154.99 ACRE-FT
 = 2.62 INCHES

RUNOFF HYDROGRAPH

TIME	HYDROGRAPH		
0	0.00	230	888.85
10	0.00	240	618.47
20	0.00	250	451.54
30	0.00	260	351.51
40	0.00	270	296.21
50	0.00	280	257.02
60	0.00	290	208.77
70	0.00	300	186.21
80	0.00	310	198.79
90	0.00	320	206.12
100	0.00	330	198.11
110	0.00	340	178.51
120	0.00	350	154.68
130	0.17	360	137.03
140	2.74	370	119.87
150	12.85	380	87.58
160	33.99	390	51.51
170	82.41	400	25.58
180	332.85	410	11.21
190	1253.21	420	4.48
200	1848.10	430	1.66
210	1722.59		
220	1287.54		

Detention Basin Outlet



1. Principal outlet: 54" CMP

$n = 0.013$

EL = 5518

Weir length = 300'

2. Emergency spillway:

$b = 12'$

EL = 5524

$z = 3$

$n = 0.04$ Table 7-2 poorly maintained
 weir length = 100' grass channel

3. Stage-storage data

El (ft msl)	Area (ac)	Avg Area (ac)	Depth (ft)	Vol (ac-ft)	Cum Volume (ac-ft)
5518	1				0
		1.5	2	3	
5520	2				3
		3.5	2	7	
5522	5				10
		7.5	2	15	
5524	10				25
		15	2	30	
5526	20				55
		35.0	2	70	
5528	50				125

Project _____ Computed _____
Subject _____ Date _____ Sht. 12-33 of _____

Computer equivalent surface area calculated
by trial and error for X

$$X^2 - 6XW + 18W^2 - \frac{\text{Vol}(43560)}{W} = 0$$

X = top width Vol = cum volume at depth (w) ac-ft
W = depth

$$SA = X^2 / 43560 = (ac)$$

Volume at 11 ft should correspond to 125 ac-ft

The 100 year discharge (708 cfs) is less than
10 year hydrograph with no improvements (880 cfs)

** RETRIEVED HYDROGRAPH DAM10

**ROUTING THROUGH RESERVOIR **

** RESERVOIR PARAMETERS **

SURFACE AREA OF RESERVOIR= 12.80 AC.
 DEPTH OF RESERVOIR = 12.90 FT.
 VOLUME OF STORAGE IN RESERVOIR = 148.89 AC-FT.
 VOLUME OF INFLOW = 76.32 AC-FT.

** RESERVOIR CONTROL STRUCTURES **

**PARAMETERS FOR SUBMERGED ORIFICE **

HEIGHT OF PIPE ABOVE BOTTOM OF POND= 0.00 FT.
 DIAMETER OF PIPE = 4.50 FT.
 NUMBER OF PIPES= 1

** PARAMETERS FOR WEIR **

HEIGHT OF WEIR ABOVE BOTTOM OF POND= 6.00 FT.
 LENGTH OF WEIR = 100.00 FT.

** PARAMETERS FOR WEIR **

HEIGHT OF WEIR ABOVE BOTTOM OF POND= 11.00 FT.
 LENGTH OF WEIR = 300.00 FT.

STAGE STORAGE DISCHARGE CURVE

STAGE FT.	STORAGE AC-FT	DISCHARGE CFS
0.00	0.00	0.00
0.65	6.67	16.41
1.29	13.42	32.83
1.94	20.25	49.24
2.58	27.15	65.65
3.23	34.14	82.07
3.87	41.20	98.48
4.52	48.35	114.89
5.16	55.58	130.23
5.81	62.89	143.94
6.45	70.28	231.92
7.10	77.75	454.49
7.74	85.31	752.68
8.39	92.96	1109.90
9.03	100.69	1517.35
9.68	108.50	1969.29
10.32	116.40	2461.61
10.97	124.39	2991.16
11.61	132.47	3912.76
12.26	140.64	5206.86

HYDROGRAPH IDENTIFICATION:

TIME TO PEAK= 290.00 MINS.
 PEAK DISCHARGE= 119.40 CFS.
 PEAK STORAGE= 50.47 AC-FT.
 PEAK STAGE= 4.70 FT.

DAM10

** INFLOW/OUTFLOW HYDROGRAPH **

T(MINS) INFLOW(CFS) OUTFLOW(CFS)

0	0.00	0.00	550	0.00	67.55
10	0.00	0.00	560	0.00	65.39
20	0.00	0.00	570	0.00	63.29
30	0.00	0.00	580	0.00	61.25
40	0.00	0.00	590	0.00	59.27
50	0.00	0.00	600	0.00	57.37
60	0.00	0.00	610	0.00	55.52
70	0.00	0.00	620	0.00	53.73
80	0.00	0.00	630	0.00	52.00
90	0.00	0.00	640	0.00	50.32
100	0.00	0.00	650	0.00	48.70
110	0.00	0.00	660	0.00	47.11
120	0.00	0.00	670	0.00	45.58
130	0.00	0.00	680	0.00	44.09
140	0.00	0.00	690	0.00	42.66
150	0.00	0.00	700	0.00	41.27
160	0.47	0.00	710	0.00	39.92
170	7.88	0.11	720	0.00	38.62
180	101.12	1.51	730	0.00	37.36
190	539.26	11.70	740	0.00	36.15
200	857.70	35.10	750	0.00	34.97
210	833.09	61.92	760	0.00	33.83
220	646.95	83.64	770	0.00	32.73
230	463.48	98.39	780	0.00	31.65
240	328.23	107.55	790	0.00	30.61
250	243.88	113.01	800	0.00	29.60
260	196.05	116.18	810	0.00	28.62
270	167.05	118.04	820	0.00	27.68
280	142.80	119.10	830	0.00	26.77
290	115.88	119.40	840	0.00	25.89
300	105.47	119.10	850	0.00	25.03
310	112.27	118.80	860	0.00	24.21
320	115.01	118.66	870	0.00	23.41
330	110.45	118.50	880	0.00	22.64
340	103.01	118.16	890	0.00	21.89
350	94.46	117.61	900	0.00	21.17
360	82.95	116.78	910	0.00	20.48
370	69.72	115.62	920	0.00	19.80
380	49.12	113.94	930	0.00	19.15
390	28.22	111.58	940	0.00	18.52
400	13.80	108.75	950	0.00	17.91
410	5.99	105.66	960	0.00	17.32
420	2.37	102.49	970	0.00	16.75
430	0.88	99.35	980	0.00	16.19
440	0.31	96.25	990	0.00	15.65
450	0.10	93.22	1000	0.00	15.13
460	0.03	90.29	1010	0.00	14.63
470	0.01	87.44	1020	0.00	14.14
480	0.00	84.69	1030	0.00	13.67
490	0.00	82.02	1040	0.00	13.21
500	0.00	79.41	1050	0.00	12.77
510	0.00	76.88	1060	0.00	12.35
520	0.00	74.43	1070	0.00	11.94
530	0.00	72.06	1080	0.00	11.54
540	0.00	69.77	1090	0.00	11.15
			1100	0.00	10.78
			1110	0.00	10.42
			1120	0.00	10.08

** RETRIEVED HYDROGRAPH 100YR

**ROUTING THROUGH RESERVOIR **

** RESERVOIR PARAMETERS **

SURFACE AREA OF RESERVOIR= 12.80 AC.

DEPTH OF RESERVOIR = 12.90 FT.

VOLUME OF STORAGE IN RESERVOIR = 148.89 AC-FT.

VOLUME OF INFLOW = 154.51 AC-FT.

WARNING!! TOTAL INFLOW VOLUME EXCEEDS RESERVOIR DESIGN CAPACITY

** RESERVOIR CONTROL STRUCTURES **

**PARAMETERS FOR SUBMERGED ORIFICE **

HEIGHT OF PIPE ABOVE BOTTOM OF POND= 0.00 FT.

DIAMETER OF PIPE = 4.50 FT.

NUMBER OF PIPES= 1

** PARAMETERS FOR WEIR **

HEIGHT OF WEIR ABOVE BOTTOM OF POND= 6.00 FT.

LENGTH OF WEIR = 100.00 FT.

** PARAMETERS FOR WEIR **

HEIGHT OF WEIR ABOVE BOTTOM OF POND= 11.00 FT.

LENGTH OF WEIR = 300.00 FT.

STAGE STORAGE DISCHARGE CURVE

STAGE FT.	STORAGE AC-FT	DISCHARGE CFS
0.00	0.00	0.00
0.65	6.67	16.41
1.29	13.42	32.83
1.94	20.25	49.24
2.58	27.15	65.65
3.23	34.14	82.07
3.87	41.20	98.48
4.52	48.35	114.89
5.16	55.58	130.23
5.81	62.89	143.94
6.45	70.28	231.92
7.10	77.75	454.49
7.74	85.31	752.68
8.39	92.96	1109.90
9.03	100.69	1517.35
9.68	108.50	1969.29
10.32	116.40	2461.61
10.97	124.39	2991.16
11.61	132.47	3912.76
12.26	140.64	5206.86

HYDROGRAPH IDENTIFICATION:

TIME TO PEAK= 235.00 MINS.

PEAK DISCHARGE= 707.69 CFS.

PEAK STORAGE= 84.17 AC-FT.

PEAK STAGE= 7.64 FT.

100 YR

** INFLOW/OUTFLOW HYDROGRAPH **

T(MINS)	INFLOW(CFS)	OUTFLOW(CFS)			
0	0.00	0.00	550	0.00	93.07
10	0.00	0.00	560	0.00	90.14
20	0.00	0.00	570	0.00	87.30
30	0.00	0.00	580	0.00	84.55
40	0.00	0.00	590	0.00	81.89
50	0.00	0.00	600	0.00	79.28
60	0.00	0.00	610	0.00	76.75
70	0.00	0.00	620	0.00	74.31
80	0.00	0.00	630	0.00	71.94
90	0.00	0.00	640	0.00	69.65
100	0.00	0.00	650	0.00	67.43
110	0.00	0.00	660	0.00	65.28
120	0.00	0.00	670	0.00	63.18
130	0.17	0.00	680	0.00	61.14
140	2.74	0.04	690	0.00	59.17
150	12.85	0.28	700	0.00	57.27
160	33.99	1.02	710	0.00	55.42
170	82.41	2.83	720	0.00	53.64
180	332.85	8.78	730	0.00	51.91
190	1253.21	34.00	740	0.00	50.24
200	1848.10	84.38	750	0.00	48.61
210	1722.59	136.05	760	0.00	47.03
220	1287.54	406.79	770	0.00	45.50
230	888.85	674.46	780	0.00	44.02
240	618.47	700.64	790	0.00	42.58
250	451.54	626.45	800	0.00	41.20
260	351.51	528.76	810	0.00	39.85
270	296.21	443.82	820	0.00	38.56
280	257.02	387.22	830	0.00	37.30
290	208.77	334.82	840	0.00	36.08
300	186.21	287.26	850	0.00	34.91
310	198.79	255.22	860	0.00	33.77
320	206.12	237.72	870	0.00	32.67
330	198.11	229.40	880	0.00	31.59
340	178.51	223.21	890	0.00	30.55
350	154.68	214.59	900	0.00	29.55
360	137.03	204.12	910	0.00	28.57
370	119.87	192.74	920	0.00	27.63
380	87.58	179.35	930	0.00	26.72
390	51.51	162.63	940	0.00	25.84
400	25.58	143.90	950	0.00	24.99
410	11.21	140.68	960	0.00	24.17
420	4.48	137.28	970	0.00	23.37
430	1.66	133.86	980	0.00	22.60
440	0.58	130.47	990	0.00	21.86
450	0.19	126.75	1000	0.00	21.14
460	0.06	123.10	1010	0.00	20.44
470	0.01	119.56	1020	0.00	19.77
480	0.00	116.12	1030	0.00	19.12
490	0.00	112.60	1040	0.00	18.49
500	0.00	109.09	1050	0.00	17.88
510	0.00	105.69	1060	0.00	17.29
520	0.00	102.40	1070	0.00	16.72
530	0.00	99.21	1080	0.00	16.16
540	0.00	96.10	1090	0.00	15.63
			1100	0.00	15.11

12.60 Open channel design

Design a trapezoidal channel to convey a discharge from a detention cell.

Given: $Q_{100} = 301$ cfs (hypothetical discharge)
 Channel slope = 0.9% or 0.009 ft/ft
 Side Slopes = 3 horizontal to 1 vertical ($z=3$)

Find: Dimensions of trapezoidal channel to convey 100-year discharge from cell.

1. $S = 0.009$ ft/ft
 $n = 0.030$ from Table 7-2 for well-maintained grass channel

$$Qn = 301(0.03) = 9.0$$

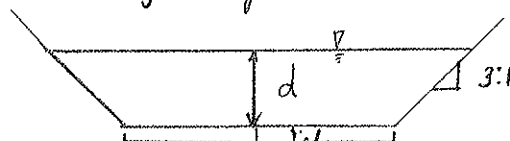
2. From Figure 7-1 Find bottom width for channel such that $w/d \geq 6$

$$\therefore d/w \leq 0.17$$

3. Try $W = 25$ ft
 $d/w = 0.068$

$$d = \frac{d}{w}(w) \\ = 0.068(25) = 1.70 \text{ ft}$$

4. Check Manning's Equation



$$n = 0.03 \\ S = 0.9\%$$

Project _____ Computed _____

Subject _____ Date _____ Sht. 12-39 of _____

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

$$A = by + zy^2$$

$$P = b + 2y\sqrt{1+z^2}$$

$$\frac{n Q}{1.49 S^{1/2}} = \frac{A^{5/3}}{P^{2/3}} = \frac{(by + zy^2)^{5/3}}{(b + 2y\sqrt{1+z^2})^{2/3}}$$

$$\frac{0.03(301)}{1.49(0.009)^{1/2}} = 63.88 = \frac{(25y + 3y^2)^{5/3}}{(25 + 2y\sqrt{1+9})^{2/3}}$$

Trial and error solution

$$y = \underline{\underline{1.68 \text{ ft}}}$$

$$V = \frac{1.49}{n} R^{2/3} S^{1/2}$$

$$R = \frac{A}{P} = \frac{by + zy^2}{b + 2y\sqrt{1+z^2}} = \frac{25(1.68) + 3(1.68)^2}{25 + 2(1.68)\sqrt{10}}$$

$$= \frac{1.49}{0.03} (1.42)^{2/3} (0.009)^{1/2}$$

$$R = 1.42 \text{ ft}$$

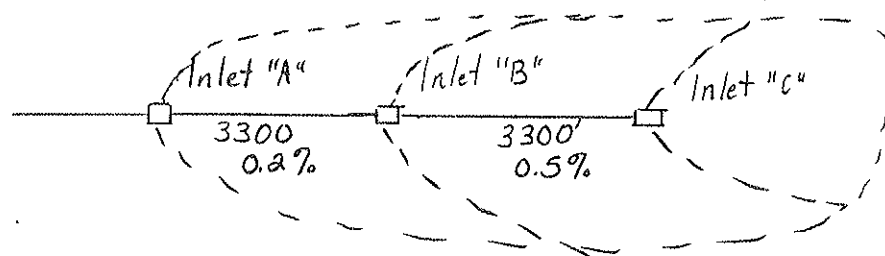
$$\underline{\underline{V = 5.94 \text{ fps (OK)}}}$$

5. Minimum freeboard = 1 ft

$$\therefore \text{ Use } \begin{aligned} d &= 3' \\ b &= 25' \\ z &= 3 \\ S &= 0.009 \text{ ft/ft} \end{aligned}$$

12.70 Trunk Sewer Design by the SCS hydrograph Procedure

A major trunk sewer will be required as a result of this project. Since the drainage area exceeds 200 acres, the SCS hydrograph procedure will be used to determine runoff.



Basin Characteristics

	"A"	"B"	"C"
Area	100ac	150ac	150ac
Length	0.55 mi	0.68 mi	0.68 mi
Slope	1.5%	2.0%	2.0%
I	60%	60%	60%
CN	76	76	76

1. Develop a 10-year ultimate hydrograph for each sub-basin (See computer run for discharges)

2. Size pipe from "C" to "B"

Grade = 0.005 ft/ft

$n = 0.013$ (smooth concrete pipe)

$Q_{10c} = 310 \text{ cfs}$

78" ϕ RCP, capacity 370 cfs

Project _____ Computed _____

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3. Add Area "B" hydrograph
 $Q_{10} = 590 \text{ cfs}$
C+B

4. Size pipe from "B" to "A"

$$\begin{aligned}\text{Grade} &= 0.002 \text{ ft/ft} \\ n &= 0.013 \text{ (RCP)} \\ Q_{10 \text{ C+B}} &= 590 \text{ cfs}\end{aligned}$$

114" ϕ RCP ; capacity 645 cfs

5. Add Area "C" hydrograph

$$Q_{10 \text{ a+b+c}} = 750 \text{ cfs}$$

6. Size pipe below "A"

$$\begin{aligned}\text{sewer grade} &= 0.002 \text{ ft/ft} \\ n &= 0.013 \\ Q_{10 \text{ a+b+c}} &= 750 \text{ cfs}\end{aligned}$$

132" ϕ pipe ; capacity 954 cfs

Capacity Manning's Equation

$$Q = \frac{1.49}{n} A R^{2/3} S^{1/2} ; \frac{1.49(\pi)(D^{8/3})}{n(4^{5/3})} (S)^{1/2}$$

$$Q = \frac{0.46}{n} D^{8/3} S^{1/2} ; \text{ From Fig 5-1 + 5-3 } \frac{d}{D} \text{ ratio}$$

Check velocity table 6-1 for permissible velocities

12-42

** HYDROGRAPH GENERATION USING 'SCS' METHOD **

WATERSHED NAME: A-AREA
 CURVE NUMBER = 76
 WATERSHED AREA= 0.160 SQ. MILES
 LENGTH OF THE MAIN CHANNEL= 2900 FEET
 SLOPE OF THE WATERSHED= 1.50 PERCENT
 IMPERVIOUS AREA= 60.00 PERCENT
 HYDRAULIC LENGTH MODIFIED= 0.00 PERCENT
 STORM RETURN PERIOD= 100 YEARS

LAG TIME= 0.1629 HOURS
 TIME TO PEAK= 0.1810 HOURS
 UNIT HYDROGRAPH PEAK= 427.91 CFS/IN
 UNIT HYDROGRAPH DURATION= 0.0362 HRS
 TIME BASE OF UNIT GRAPH= 0.9049 HOURS

*** RUNOFF PARAMETERS ***

TIME TO PEAK= 190.00 MIN
 PEAK DISCHARGE= 217.55 CFS
 RUNOFF VOLUME= 11.04 ACRE-FT
 = 1.29 INCHES

RUNOFF HYDROGRAPH

TIME	HYDROGRAPH		
0	0.00	190	217.55
10	0.00	200	121.27
20	0.00	210	62.42
30	0.00	220	42.43
40	0.00	230	32.88
50	0.00	240	24.75
60	0.00	250	23.84
70	0.00	260	21.73
80	0.00	270	18.75
90	0.00	280	14.00
100	0.00	290	11.06
110	0.00	300	17.98
120	0.00	310	17.64
130	0.00	320	15.79
140	0.00	330	13.83
150	0.00	340	13.34
160	0.57	350	10.69
170	7.49	360	9.11
180	88.30	370	5.48
		380	1.12

STORING AAREA HYDROGRAPH ON DISK.

** HYDROGRAPH GENERATION USING 'SCS' METHOD **

WATERSHED NAME: B-AREA
 CURVE NUMBER = 76
 WATERSHED AREA= 0.230 SQ. MILES
 LENGTH OF THE MAIN CHANNEL= 3590 FEET
 SLOPE OF THE WATERSHED= 2.00 PERCENT
 IMPERVIOUS AREA= 60.00 PERCENT
 HYDRAULIC LENGTH MODIFIED= 0.00 PERCENT
 STORM RETURN PERIOD= 100 YEARS

LAG TIME= 0.1686 HOURS
 TIME TO PEAK= 0.1874 HOURS
 UNIT HYDROGRAPH PEAK= 594.09 CFS/IN
 UNIT HYDROGRAPH DURATION= 0.0374 HRS
 TIME BASE OF UNIT GRAPH= 0.9369 HOURS

*** RUNOFF PARAMETERS ***

TIME TO PEAK= 190.00 MIN
 PEAK DISCHARGE= 308.41 CFS
 RUNOFF VOLUME= 15.86 ACRE-FT
 = 1.29 INCHES

RUNOFF HYDROGRAPH

TIME	HYDROGRAPH		
0	0.00	220	62.14
10	0.00	230	47.88
20	0.00	240	36.05
30	0.00	250	34.37
40	0.00	260	31.37
50	0.00	270	27.18
60	0.00	280	20.49
70	0.00	290	15.93
80	0.00	300	25.40
90	0.00	310	25.50
100	0.00	320	22.78
110	0.00	330	20.01
120	0.00	340	19.21
130	0.00	350	15.58
140	0.00	360	13.17
150	0.00	370	8.19
160	0.75	380	1.84
170	10.03		
180	118.80		
190	308.41		
200	179.94		
210	93.16		

STORING BAREA HYDROGRAPH ON DISK.

** HYDROGRAPH GENERATION USING 'SCS' METHOD **

WATERSHED NAME: C-AREA
 CURVE NUMBER = 76
 WATERSHED AREA= 0.230 SQ. MILES
 LENGTH OF THE MAIN CHANNEL= 3590 FEET
 SLOPE OF THE WATERSHED= 2.00 PERCENT
 IMPERVIOUS AREA= 60.00 PERCENT
 HYDRAULIC LENGTH MODIFIED= 0.00 PERCENT
 STORM RETURN PERIOD= 100 YEARS

LAG TIME= 0.1686 HOURS
 TIME TO PEAK= 0.1874 HOURS
 UNIT HYDROGRAPH PEAK= 594.09 CFS/IN
 UNIT HYDROGRAPH DURATION= 0.0374 HRS
 TIME BASE OF UNIT GRAPH= 0.9369 HOURS

*** RUNOFF PARAMETERS ***

TIME TO PEAK= 190.00 MIN
 PEAK DISCHARGE= 308.41 CFS
 RUNOFF VOLUME= 15.86 ACRE-FT
 = 1.29 INCHES

** INFLOW/OUTFLOW HYDROGRAPH **

T(MINS)	INFLOW(CFS)	OUTFLOW(CFS)			
0	0.00	0.00	220	0.00	62.14
10	501.90	0.00	230	0.00	47.88
20	342.83	0.00	240	0.00	36.05
30	63.07	0.00	250	0.00	34.37
40	7.38	0.00	260	0.00	31.37
50	0.61	0.00	270	0.00	27.18
60	0.00	0.00	280	0.00	20.49
70	0.00	0.00	290	0.00	15.93
80	0.00	0.00	300	0.00	25.40
90	0.00	0.00	310	0.00	25.50
100	0.00	0.00	320	0.00	22.78
110	0.00	0.00	330	0.00	20.01
120	0.00	0.00	340	0.00	19.21
130	0.00	0.00	350	0.00	15.58
140	0.00	0.00	360	0.00	13.17
150	0.00	0.00			
160	0.00	0.75			
170	0.00	10.03			
180	0.00	118.80			
190	0.00	308.41			
200	0.00	179.94			
210	0.00	93.16			

STORING CAREA HYDROGRAPH ON DISK.

** RETRIEVED HYDROGRAPH CAREA

HYDROGRAPH IDENTIFICATION:

TIME TO PEAK= 190.00 MINS.
PEAK DISCHARGE= 308.41 CFS.

** ROUTING THROUGH PIPE USING DESIGN MODE.

SLOPE OF PIPE = 0.0050 (FT/FT).
LENGTH OF PIPE = 3300.00 FT.
PIPE SELECTED IS A SMOOTH CONCRETE PIPE
MANNING'S 'n' = .013
DESIGN PIPE SIZE IS 78 INCHES.

HYDROGRAPH IDENTIFICATION:

TIME TO PEAK= 195.00 MINS.
PEAK DISCHARGE= 308.41 CFS.

** ADDING THE LAST HYDROGRAPH TO BAREA

HYDROGRAPH IDENTIFICATION:

TIME TO PEAK= 190.00 MINS.
PEAK DISCHARGE= 586.36 CFS.
PEAK STAGE= 0.00 FT.

** ROUTING THROUGH PIPE USING DESIGN MODE.

SLOPE OF PIPE = 0.0020 (FT/FT).
LENGTH OF PIPE = 3300.00 FT.
PIPE SELECTED IS A SMOOTH CONCRETE PIPE
MANNING'S 'n' = .013
DESIGN PIPE SIZE IS 114 INCHES.

HYDROGRAPH IDENTIFICATION:

TIME TO PEAK= 195.00 MINS.
PEAK DISCHARGE= 586.36 CFS.

** ADDING THE LAST HYDROGRAPH TO AAREA

HYDROGRAPH IDENTIFICATION:

TIME TO PEAK= 195.00 MINS.
PEAK DISCHARGE= 753.41 CFS.
PEAK STAGE= 0.00 FT.

** ROUTING THROUGH PIPE USING DESIGN MODE.

SLOPE OF PIPE = 0.0020 (FT/FT).
LENGTH OF PIPE = 3300.00 FT.
PIPE SELECTED IS A SMOOTH CONCRETE PIPE
MANNING'S 'n' = .013
DESIGN PIPE SIZE IS 132 INCHES.

HYDROGRAPH IDENTIFICATION:

TIME TO PEAK= 200.00 MINS.
PEAK DISCHARGE= 753.41 CFS.

12.80 Detention basin design by Rational Formula

Determine the required volume to control runoff of the 10 year storm from an 8 acre parking lot. Assume the lot is divided into 4 cells of 2 acres each, and that the slopes are 2%. The inlet time for each cell is 5 minutes.

Given : Area = 2 acres

$C = 0.9$

$T_c = 5 \text{ min}$

Freq = 10 yr

Find : Family of curves for Modified Rational Formula ; critical volume

1. Curve family

Duration (min)	i (in/hr)*	Q_p (cfs) **
5	7.20	12.96
10	5.82	10.48
15	4.92	8.86
20	4.35	7.83
30	3.48	6.26
40	2.85	5.13
50	2.59	4.66
60	2.29	4.12

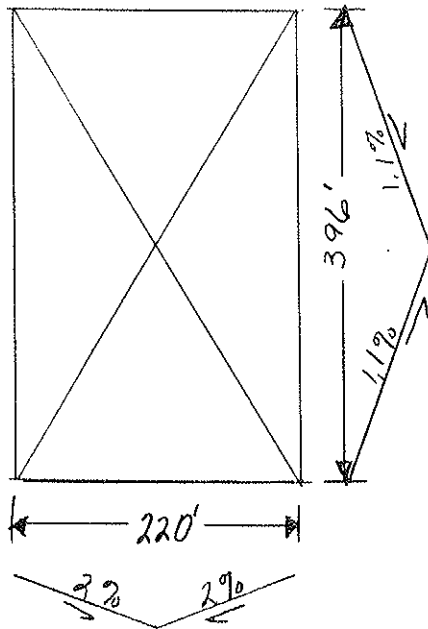
* intensity - Figure 2-1

** $Q_p = C i A$

Project _____ Computed _____

Subject _____ Date _____ Sht. 12 of 70

2. Cell dimensions



$$\text{Cell volume} = \frac{1}{3} (220) (396) (2.2) \\ = 63888 \text{ ft}^3$$

3. Critical volume

Assume release rate = 3.5 cfs

$$\text{Vol} = Q_p \left(60 \frac{\text{sec}}{\text{min}} \right) (D)$$

D	St. Vol (ft ³)	Rel. Vol (ft ³)	Storage (ft ³)
5	3888	1050	2838
10	6288	2100	4188
15	7974	3150	4824
20	9396	4200	5196
30	11268	6300	4968
40	12312	8400	3912
50	13980	10500	3480
60	14832	12600	2232

← maximum storage

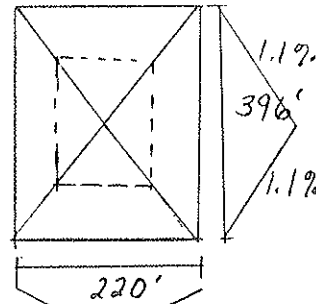
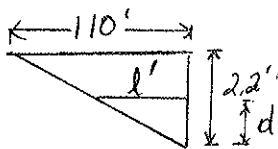
Project _____ Computed _____

Subject _____ Date _____ Sht. 12-48 of _____

4. Check ponding depth

Critical volume at 5196 ft³

$$V = \frac{1}{3} A d$$



$$\frac{110}{l'} = \frac{2.2}{d}$$

$$\frac{198}{w'} = \frac{2.2}{d}$$

$$l' = \frac{110d}{2.2}$$

$$w' = \frac{198d}{2.2}$$

$$A = lw = 2l'(2w')$$

$$A = \frac{(2)^2(110)(198)d^2}{(2.2)^2}$$

$$V = \frac{1}{3} A d = \frac{(2)^2(110)(198)d^3}{3(2.2)^2}$$

$$d = \sqrt[3]{\frac{V(3)(2.2)^2}{(110)(198)(2)^2}} ; V = 5196 \text{ ft}^3$$

$d = 0.95'$ - minimum requirement is 12"

$$A = 18,000 \text{ ft}^2 ; V = 6000 \text{ ft}^3$$

Project _____ Computed _____

Subject _____ Date _____ Sht. 12-49 of _____

5. Inlet design

$$Q_i = 5.37 A d^{0.5} \quad \text{or} \quad Q_i = 3.0 P d^{1.5}$$

Solve for A

$$P = \frac{3.5 \text{ cfs}}{3.0 (1.0)^{1.5}} = 1.17 \text{ ft}$$

$$\frac{3.5 \text{ cfs}}{5.37 (1)^{0.5}} = 0.65 \text{ ft}^2$$

Small grate is need - 12" ϕ pipe

12.90 Culvert Design

A new culvert will be needed under the arterial street which borders the proposed shopping center on the south side. The total drainage area above the culvert is 96 acres, and the time of concentration is 15 minutes. The composite runoff coefficient is 0.8.

Downstream of the culvert, the channel has been regraded and aligned with a channel slope of 1%. The outlet elevation will be elevation 5510. The natural stream slope through the area was 2%. The proposed culvert will be 150 feet long, and the natural stream bed elevation at the inlet is elevation 5576. The top of the road elevation is elevation 5524.

Design a culvert which will pass the 100 year flood without exceeding the top of the road elevation. Determine the culvert size needed assuming inlet control. Also determine the culvert size needed using an improved inlet.

Use the forms and charts presented in Section 6 of the manual.

1. Discharge - use Rational equation

$$i = 4.17 \text{ in/hr (hypothetical intensity)}$$

$$Q = ciA = 0.8(4.17)(96)(1.25) = 400 \text{ cfs}$$

Use 400 cfs

PROJECT:

DESIGNER:

DATE:

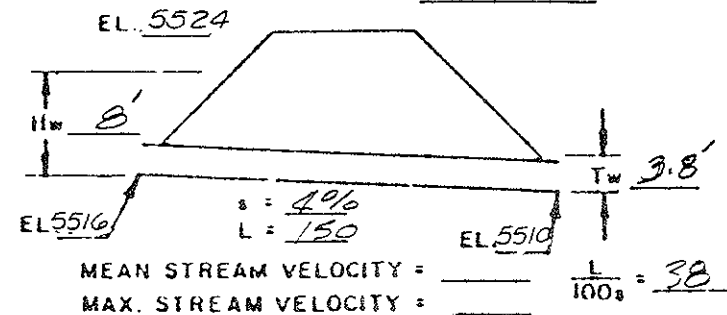
HYDROLOGIC AND CHANNEL INFORMATION

 $Q_1 = 400$ TAILWATER ELEVATION = 5513.8 $Q_2 =$ TAILWATER ELEVATION =

(Q_1 = DESIGN DISCHARGE, SAY Q_{25}
 Q_2 = CHECK DISCHARGE, SAY Q_{50} OR Q_{100})

SKETCH

STATION:



CULVERT DESCRIPTION (ENTRANCE TYPE)	Q	SIZE	HEADWATER COMPUTATION												CONTROLLING HW	OUTLET VELOCITY	COST	COMMENTS
			INLET CONT.		OUTLET CONTROL HW = $H + h_o - L_s$								CHART					
			$\frac{H_w}{D}$	Hw	K_e	H	d_c	$\frac{d_c + D}{2}$	T_w	h_o	L_s	Hw	No.	Hw				
groove edge proj	400	96"	.93	7.44	0.2	1.35	4.9	6.45		6.45	6	1.9	$\frac{6.31}{6.22}$	7.4	7.4	13.8		riprap apron

SUMMARY & RECOMENDATIONS:

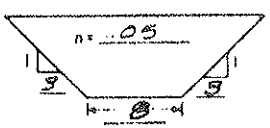
Inlet control governs; use 96" RCP

12-51

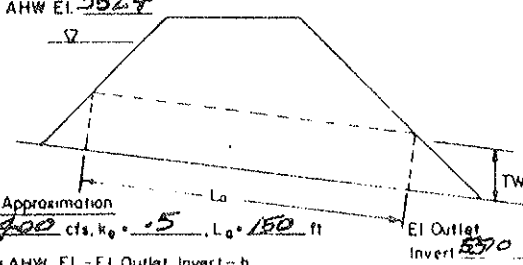
PROJECT: _____		OUTLET CONTROL DESIGN CALCULATIONS		DESIGNER: _____	
STATION: _____				DATE: _____	

INITIAL DATA:
 $Q = 100 = 400$ cfs
 AHW El. = 5524 ft.
 $S_o = .02$ (.01 d/s)
 $L_o = 150$ ft.
 El. Outlet Invert = 5510 ft.

 Stream Data:



SKETCH



First Approximation
 $Q = 400$ cfs, $k_o = .5$, $L_o = 150$ ft
 $H = \text{AHW El.} - \text{El. Outlet Invert} - h_o$
 $= 5524 - 5510 - 5 = 9$
 $\therefore A = \text{ft}^2$ or $D = \text{ft}$; Try _____

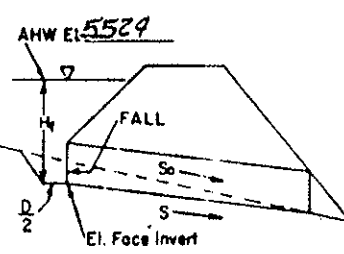
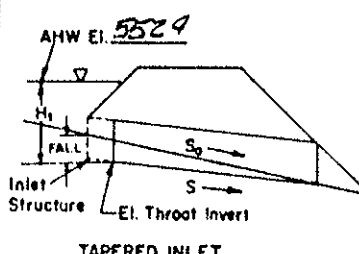
Q	$\frac{Q}{N}$	H	$\frac{Q}{NB}$	(1) d_c	$\frac{d_c D}{2}$	Qn	(2) TW	(3) h_o	(4) HW	(5) V_o	COMMENTS
Trial No <u>1</u> , N = <u>1</u> , B = <u>5</u> , D = <u>5</u> , $k_o = .5$ 60" RCP w/ square edge											
400	400	⁶⁻²² 13	400	7.5	5	20	3.8	5	5528		exceeds allowable HW
Trial No <u>2</u> , N = <u>1</u> , B = <u>-</u> , D = <u>66"</u> , $k_o = .2$ larger pipe w/ beveled edge											
400	400	7	400	5.3	5.4	20	3.8	5.5	5524	16.8	HW is OK, but $V_o > 15$ fps
350	350	5.3	350	5.1	5.3			5.1	5520.9		$Q_{100} - 50$ cfs
450	450	8.8	450	7.5	5.5			5.5	5524.3		$Q_{100} + 50$ cfs
Trial No _____, N = _____, B = _____, D = _____, $k_o = _____$											

Notes and Equations:
 (1) d_c cannot exceed D
 (2) TW based on d_c in natural channel, or other downstream control.
 (3) $h_o = \frac{d_c D}{2}$ or TW, whichever is larger.
 (4) $HW = H + h_o + \text{El. Outlet Invert.}$
 (5) Outlet Velocity ($V_o = Q/\text{Area}$) defined by d_c or TW, not greater than D. Do not compute until control section is known.

SELECTED DESIGN

N = 1 At Design Q:
 B = _____ ft.
 D = 5.5 ft. HW = 5524 ft.
 $k_o = .2$ $V_o = 16.8$ f/s

$H = \left[1 + k_o + \frac{29n^2 \cdot L}{R^{1.33}} \right] \frac{V^2}{2g}$

PROJECT: _____		CULVERT INLET CONTROL SECTION DESIGN CALCULATIONS		DESIGNER: _____	
STATION: _____				DATE: _____	
INITIAL DATA: $Q = 100 = 400$ cfs AHW El. = <u>552.9</u> ft. $S_o = .027$ $L_o = 1.50$ ft. El. Stream Bed at Face <u>551.6</u> ft. Barrel Shape and Material <u>RCP</u> Barrel In. = <u>0.012</u> $N = 1$, $B =$ _____ $D = 66"$, $NBD^{3/2} =$ _____ (Pipe) $N/D^{3/2} = 70.9$		 CONVENTIONAL or BEVELED INLET: FACE CONTROL SECTION (Upper Headings)		 TAPERED INLET THROAT CONTROL SECTION (Lower Headings)	
DEFINITIONS OF INLET CONTROL SECTION					
	$\frac{Q}{NB}$	$\frac{H_f}{D}$	H_f	(1) El. Face Invert	El. Stream Bed At Face
	$\frac{Q}{NBD^{3/2}}$	$\frac{H_1}{D}$	H_1	El Throat Invert	FALL
					(2) (3) (4) (5)
					HW _f HW _t S V _o
Note: Use Upper Headings for Conventional or Beveled Face; Lower Headings for Tapered Inlet Throat.					
COMMENTS					
Trial No. <u>1</u> Inlet and Edge Description <u>Beveled inlet, Type B</u>					
400	400	$\frac{6.33}{2.1}$	11.6	552.4	551.6
				3.6	552.9
					16.8
fall too large; try tapered inlet					
Trial No. <u>2</u> Inlet and Edge Description <u>tapered throat, smooth</u>					
400	5.69	$\frac{6.33}{1.8}$	9.9	551.1	551.6
350	4.94	$\frac{1.65}{9.1}$	9.1	552.1	552.6
450	6.35	$\frac{2.20}{12.1}$	12.1	552.1	552.6
Trial No. _____ Inlet and Edge Description _____					
Notes and Equations:					
(1) El Face (or throat) invert = AHW El - H_f (or H_t) (2) FALL = El Stream Bed at Face - El. face (or throat) invert (3) HW _f (or HW _t) = H_f (or H_t) - El. face (or throat) invert, where El. face (or throat) invert should not exceed El. stream bed. (4) $S \approx S_o - \text{FALL} / L_o$ (5) Outlet Velocity = $Q / \text{Area defined by } d_n \text{ at } S$					
SELECTED DESIGN					
Inlet Description: FALL = <u>1.9</u> ft Invert El = <u>552.1</u> ft. Bevels: Angle = <u>N/A</u> $b =$ _____ in., $d =$ _____ in.					

PROJECT: _____		SIDE-TAPERED INLET DESIGN CALCULATIONS		DESIGNER: _____	
STATION: _____		DATE: _____			

INITIAL DATA Q/W = <u>400</u> cfs $S_o = .02$ AHW EI = <u>5524</u> ft. $L_o = 150$ ft. TAPER = <u>4</u> : 1 Barrel Shape and Material <u>66" RCP</u> Face Edge Description <u>45° bevels</u> $N = 1$, $B =$ _____ ft., $D = 5.5$ ft.	SKETCH
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Q	EI, Throat Invert	(1)	(2)	(3)	(4)	B ₁	L ₁	S	L ₁ S	(5) EI, Face Invert	Upper Headings for Box Culverts, Lower Headings for Pipes COMMENTS
		$\frac{H_f}{D}$	$\frac{Q}{B_1 D^{3/2}}$	$\frac{Q}{A_1 E^{1/2}}$	$\frac{E^{1/2}}{A_1}$						

Trial No. 1, Q = 400, HW₁ = 5524.0

400	5514.1	1.62	6.38 4.6	2.35	37	10	9	.027	0.29	5514.34	8.53 [or $A_1 E^{1/2}$] = 43.20
350		1.20	3.45							6.6	HW ₁ = 5520.94
400		1.38	3.94							7.6	5521.94

Trial No. 1, Q = 400, HW₁ = 5524.0

450		1.55	4.43								8.53 [or $A_1 E^{1/2}$] = 43.20
											8.53 5522.87

Trial No. _____, Q = _____, HW₁ = _____

											$B_1 D^{3/2}$ [or $A_1 E^{1/2}$] = _____

Notes and Equations: (1) H_f/D [or H_f/E] = (HW ₁ - EI, Throat Invert - 1)/D [or E] $D \leq E \leq 1.1D$ (2) Min. $B_1 = \frac{Q}{(D^{3/2})(Q/B_1 D^{3/2})}$ $\text{Min. } A_1 = \frac{Q}{(E^{1/2})(Q/A_1 E^{1/2})}$ (3) $L_1 = \left[\frac{B_1 - NB}{2} \right]$ TAPER (4) From throat design (5) EI, Face Invert - EI, Throat Invert > 1 ft., recompute. Face and Throat may be lowered to better fit site, but do not raise	SELECTED DESIGN $B_1 = 10'$ ft. $L_1 = 9'$ ft. Bevels: Angle <u>45°</u> $d = 5.5$ in., $b = 2.8$ in. Crest Check: HW _c = <u>5524</u> ft. $H_c = 8$ ft. $Q/W = 69$ (Fig. 6-39) Min. W = <u>6.25</u> ft. < B_1 (OK)
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