PORT LAVACA DRAINAGE CRITERIA MANUAL

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CITY OF PORT LAVACA
DRAINAGE CRITERIA MANUAL

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SECTION 1 - INTRODUCTION

1.1.0 GENERAL

A long range, storm drainage master plan is necessary to insure sufficient drainage to new and vacant areas as well as to provide continued flood protection for existing urban areas. In preparation of the City of Port Lavaca Master Plan, only relatively large areas were considered. Generally only those greater than 250 acres. It was not the intent of the Master Plan to propose drainage improvements to serve every small parcel of land because it is simply not practical to do so. The proposed drainage improvements shown on the Master Plan should be considered a recommended "Frame Work" of the ultimate drainage system. It should be recognized that as time passes, and property develops, it may be necessary to modify the Master Plan in order to remain current with the changing conditions. It is also important to understand that this does not imply modifying the Master Plan simply to suit the needs of one individual or group. Proposed modifications shall be carefully studied by the City of Port Lavaca to insure that they are beneficial to the entire drainage area before such modifications are approved.

1.2.0 LEVEL OF FLOOD PROTECTION

It would be desirable if storm drainage improvements proposed by the Master Plan were designed so that all drainage facilities would handle a 100 year frequency rain. Unfortunately this is not economically feasible. The standard practice in most cities is to design storm drainage facilities so that during an extreme rainfall event, ponding of water in streets and yards will be of minimum depth and duration, and most importantly, that all new building slabs will be set above the anticipated ponding levels. If the design criteria contained in this manual is followed, and the improvements proposed by the Master Plan are implemented, in most cases this objective will be fulfilled.

1.3.0 STUDY AREA

The City of Port Lavaca Master Drainage Plan encompasses an overall drainage area of approximately 15,000 acres. The area studied in detail is bounded on the north by Maxwell Ditch Road, on the east by Lavaca Bay, on the south by Chocolate Bay and Chocolate Bayou, and on the west by a line extending through the intersection of FM 2433 and State Highway 35 to a point on Maxwell Ditch Road approximately 1200 feet east of the Calhoun County Airport.

The channels studied by detailed methods include,

(1) Barton Ditch
(2) Corporation Ditch
(3) Harbor of Refuge Ditch
(4) LaSalle Ditch
(5) Little Chocolate Bayou

(6) Lynn's Bayou
(7) Village Road Ditch
(8) West Branch of Lynn's Bayou (Lateral IVa)
(9) Wilson Ditch
*(10) Sand Crab Ditch
*(11) North Relief Channel (Proposed)
*(12) North Relief Channel Lateral 1 (Formerly the north portion of Lynn's Bayou)

* Indicates that a detailed study was performed for ultimate conditions only.

1.4.0 ENGINEERING METHODS

1.4.1 Hydrologic Analyses

Hydrological analyses were carried out to establish the expected flow rate from floods having a 25 and 100 year recurrence interval. The method of analysis was dependent upon the size of the drainage area. For areas less than 250 acres, the Rational Method was used to estimate peak flow rates. For areas greater than 250 acres, the analyses were based on the U.S. Geological Survey (USGS) open-file report 77-110 "Technique for Estimating the Magnitude and Frequency of Floods in Texas". The effects of urbanization on large watersheds were evaluated using a modification to the USGS technique which was developed by the Texas State Department of Highways and Public Transportation (TSDHPT).

1.4.2 Hydraulic Analyses

Cross sections for the backwater analyses were obtained from aerial photographic maps prepared by Williams-Stackhouse, Inc. in February of 1989. Dimensions and elevations for all bridges and culverts were determined by field measurements.

Water surface elevations for the selected recurrence intervals were computed using the U.S. Army Corps of Engineer's HEC-2 backwater computer program.

All elevations are referenced from the National Geodetic Vertical Datum (NGVD), formerly referred to as Mean Sea Level Datum of 1929. Elevation loops were performed using 3rd order field survey and radiated from National Geodetic Survey bench marks, B-593, D-580, F-580, and the U.S. Coast and Geodetic Survey triangulation station "DILLO". Additional bench marks were set at most bridges and culverts and are list in the Appendix of this manual.

END OF SECTION

SECTION 2
DRAINAGE POLICY

CONTENTS

SECTION 2 - DRAINAGE POLICY

2.1.0 GENERAL

The City of Port Lavaca Drainage Criteria Manual represents a compilation of accepted drainage engineering principles and methods. It is intended to serve as a supplement to basic engineering knowledge obtainable from standard drainage handbooks and technical publications. The design factors, formulae, graphs, and procedures are to be used as guides in the solution of drainage problems involving the determination of quantity, rate of flow, method of collection, storage, and conveyance of storm water. Responsibility for actual design remains primarily with the design engineer, therefore, users of this manual should be knowledgeable and experienced in the theory and application of surface drainage engineering. The criteria presented address the most commonly encountered flood control design problems in Port Lavaca. There will occasionally be situations not covered by this manual or there may be, on occasion, cases which merit exception to the guidelines. It is recommended that when such situations arise, the City Engineer be consulted as early as possible to insure an acceptable and timely resolution to these special problems.

2.2.0 APPLICATION

The City of Port Lavaca drainage criteria manual shall govern the planning, design, and construction of storm drainage facilities within the City of Port Lavaca and within all areas subject to its extraterritorial jurisdiction. In order to receive a variance to any procedure of requirement contained in this manual, the applicant must receive approval from the City of Port Lavaca Planning Board and the City of Port Lavaca City Council.

The city ordinance empowering the drainage criteria and requirements presented in this manual is contained in Section VIII of the Subdivision Regulations for the City of Port Lavaca, adopted __________, __, 1989.

2.3.0 STORM DRAINAGE CALCULATIONS

Calculations to support all drainage designs shall be submitted to the City of Port Lavaca for review and approval. Construction shall not begin until approval is obtained. To facilitate an orderly and timely review by the City Staff all calculations shall be in a form similar to those shown in the Appendix of this manual. All calculations shall bear the seal of a Registered Professional Engineer, licensed in the State of Texas, and shall contain a statement by the Engineer that the design calculations have been prepared in full compliance with the requirements of this manual. All drainage calculations shall be made part of the City's permanent engineering records.

2.4.0 CONSTRUCTION PLANS AND SPECIFICATIONS

Construction drawing for all drainage facilities shall be submitted to the City of Port Lavaca for review and approval. Construction shall not begin until such approval is obtained.

2.4.1 Drawing Size

All drawings submitted to the City of Port Lavaca for review shall be on 24" by 36" paper.

2.4.2 Drawing Scales

A. Layout Sheets

Storm drainage system layout sheets shall be at a scale of 1" = 50' or 1" = 100'.

B. Plan-Profile Sheets

Plan-profile sheets shall have a horizontal scale of 1" = 50' in the plan view and 1" = 5' vertical scale in the profile view.

2.4.3 Storm Drainage Layout Sheets

The storm drainage layout sheets shall include the following minimum information.

(1) Property lines, lot and block numbers, dimensions, right-of-way and easements lines, and street names.

(2) Location, size, and type of inlets, manholes, pipes, headwalls, culverts, bridges, and channels.

(3) Proposed top and invert elevations of all inlets, manholes, etc.

(4) Existing contour lines and spot elevations at suitable intervals to indicate the slope of the existing ground.

(5) Suitable labeling to aid in relating the storm drainage layout to the plan-profile sheets and drainage calculations.

(6) Existing and proposed utilities where they cross the proposed improvements.

(7) Description and elevation of vertical bench marks.

(8) North arrow, scale, title block, etc.

2.4.4 Plan-Profile Sheets

A. Plan View

The plan view of the plan-profile sheets shall include the following minimum information.

(1) Property lines, lot and block numbers, dimensions, right-of-way and easements lines, and street names.

(2) Location, size, and type of inlets, manholes, pipes, headwalls, culverts, bridges, and channels.

(3) Proposed top and invert elevations of all inlets, manholes, etc.

(4) Existing and proposed utilities where they cross the proposed improvements.

(5) Existing and proposed improvements such as streets, sidewalks, poles, ditches, etc.

(6) North arrow, scale, title block, etc.

B. Profile View

The profile view of the plan-profile sheets shall include the following minimum information.

(1) Profile of the existing ground and proposed finished grades. For open channels show left bank, right bank, and center line.

(2) Profile of proposed storm drainage improvements including flowlines of open channels, flowline and top of pipes, inlets, manholes, etc.

(3) Size of all pipes, channels and structures.

(4) Proposed slopes and proposed flowline and invert elevations.

(5) Proposed and existing utility crossings.

(6) Profile of the hydraulic gradient.

(7) Title block and scale.

2.4.5 Detail Sheets

Detail sheets shall include drawings of all inlets, manholes, headwalls, culverts, bridges, and riprap required to construct the proposed facilities. The detail sheet shall also include typical channel sections;

pipe embedment sections, backfill requirements, etc.

2.4.6 Specifications

Technical specifications shall include complete information on materials and construction methods that will govern the construction of the drainage improvements.

2.5.0 MASTER DRAINAGE PLAN REVISIONS

Two separate master drainage maps have been prepared for the City of Port Lavaca. The first, titled CURRENT CONDITIONS, provides 25 and 100 year water surface elevations along the areas major channels using existing structures, channel conditions, and drainage basin development. The second, titled ULTIMATE CONDITIONS, indicates the 25 and 100 year water surface elevations assuming 100 percent development, and the implementation of all of the recommended channel and structure improvements. It is essential that both of these maps remain up to date and therefore must be revised frequently in areas of development.

2.5.1 Map Revision Requirements

A. Map revisions shall be required when a developer or land owner:

(1) Wishes to place fill within the current or ultimate flood plain.

(2) Proposes a re-distribution of drainage areas, other than that shown on the Master Drainage Plan, which will produce a change in flow rate, (Q), equal to more than ten (10) percent of the total flow at any point in the system.

(3) Wishes to modify any of the recommended structure or channel improvements indicated on the Master Drainage Plan.

(4) Wishes to install additional structures in any of the studied channels.

(5) Constructs **any** of the recommended structure or channel improvements indicated on the Master Drainage Plan.

B. All map revisions shall be performed by a Registered Professional Engineer, approved by the City of Port Lavaca.

C. All revisions shall be accompanied by a report outlining **all** modifications which were made to the original HEC-2 data files.

D. The revised HEC-2 computer runs shall be supplied to the City on both paper printout and computer disk format.

E. Revised maps shall be supplied to the City on reproducible film.

F. Proposed modifications to the Master Drainage Plan shall be carefully studied by the design Engineer and the City of Port Lavaca to insure that they are **beneficial** to the entire drainage area before such modifications are approved. (i.e. Modifications which will raise the ultimate water surface elevation by more than 0.10 feet shall not be approved without the written consent of all affected land owners.)

G. All costs associated with the map revisions shall be paid for by the developer.

END OF SECTION

SECTION 3
STORM RUNOFF

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SECTION 3 - STORM RUNOFF

3.1.0 GENERAL

Prior to the design of any drainage facility, it is necessary for the engineer to estimate the rate of runoff which can be expected during the appropriate design storm. If the estimated discharge is incorrect, the facility may be either oversized or undersized. An improperly designed drainage system can be uneconomical, cause upstream flooding, interfere with traffic, disrupt commercial activities, and be a general nuisance in the effected area. However, due to the many complex physical and climatic factors involved, the actual rate of runoff for a particular recurrence interval storm can only be approximated.

Continuous long-term records of rainfall and the resulting storm runoff in a watershed would provide the most accurate means of estimating peak flows. Because sufficient records are unavailable for the Port Lavaca area, the accepted practice is to relate storm runoff to rainfall by use of empirical formulas and regional regression equations.

It is known that urbanization has a pronounced effect on the rate and volume of runoff from a given rainfall. Urban development alters the hydrology of a watershed by improving hydraulic efficiency, reducing surface infiltration, and reducing storage capacity. The reduction of surface infiltration and storage capacity is due to the elimination of porous surfaces and ponding areas by grading and paving building sites, streets, drives, parking lots, and sidewalks and by constructing buildings and other facilities characteristic of urban development. When selecting design runoff coefficients, the engineer should take into account the ultimate degree of urbanization expected for the watershed.

3.2.0 METHODS OF ANALYSIS

Numerous methods for computing storm runoff are available today. They range from the widely used rational method, developed in the late 1800's, to sophisticated computer watershed models. The method selected is based on the size of the drainage area, the available data, and the degree of sophistication required for the design. For the purposes of this manual, the methods to be used for determining peak flows shall vary depending on the size of the drainage basin as follows:

3.2.1 Drainage Areas Less Than 250 Acres

For drainage areas less than 250 acres, the engineer shall determine the peak runoff rate using the Rational Method, as described in Section 3.4.0.

3.2.2 Drainage Areas Greater Than 250 Acres

Peak flows from drainage areas greater than 250 acres shall be calculated using the Regional Regression Equations described in Section 3.5.0.

3.3.0 DESIGN FREQUENCY

Planners and engineers often make reference to specific design frequency storms without understanding the actual flooding implications of such a storm. A five (5) year rain is not expected to occur at five (5) year intervals. Rather, it is the flood which has a 20 percent probability of being equaled or exceeded in any given year. For any five (5) successive years, the probability of exceeding a five (5) year design storm is 67 percent, not 100 percent, as one might expect. The mathematical relationships which are applicable for flood frequencies are:

The probability of exceeding the 'N-year' flood in any year = $1/N$.

The probability of not exceeding the 'N-year' flood in any year = $1-1/N$.

The probability of not exceeding the 'N-year' flood in x successive years = $(1-1/N)x$.

The probability of exceeding the 'N-year' flood once or more in x successive years = $1-(1-1/N)^x$.

3.3.1 5 Year Design Frequency

The following drainage facilities shall be designed based on a five (5) year design frequency:

(1) Streets and inlets.

(2) Underground storm sewer systems.

(3) Roadside ditches and driveway culverts.

3.3.2 25 Year Design Frequency

The following drainage facilities shall be designed based on a 25 year design frequency:

(1) Open channels serving a drainage area less than 250 acres. (Including culverts and bridges).

(2) Storm water retention/detention facilities with a total drainage area less than 250 acres.

3.3.3 100 Year Design Frequency

The following drainage facilities shall be designed based on a 100 year design frequency:

(1) Open channels serving a drainage area greater than 250 acres. (Including culverts and bridges).

(2) Storm water retention/detention facilities with a total drainage area greater than 250 acres.

3.4.0 RATIONAL METHOD

It is required that the rational method be used for estimating peak runoff from drainage areas less than 250 acres. This method will provide satisfactory results if understood and applied correctly. The rational method is based on the direct relationship between rainfall and runoff, and is expressed by the following equation:

$$Qp = CIA \qquad \text{(Eq. 3-1)}$$

where,

Qp = Peak runoff in cubic feet per second. Actually, Qp is in units of inches per hour per acre. Since the rate of in/hr/ac differs from cfs by less than one (1) percent the more common units of cfs are used.

C = Coefficient of runoff is the percentage of total rainfall on a given area that flows off as free water.

I = Average intensity of rainfall in inches per hour.

A = Area contributing runoff to the point of design in units of acres.

When using the rational method, the following basic assumptions are employed:

(1) The maximum rate of runoff occurs when the storm duration is equal to the time of concentration for the drainage area.

(2) The rainfall intensity is uniform over the entire drainage area during the entire storm duration.

(3) The runoff coefficient does not vary during the storm duration.

3.4.1 Runoff Coefficient

The runoff coefficient is the percentage of total rainfall on a given area that flows off as free water. The portion of the total rainfall that will reach the drainage system depends on many factors including the imperviousness of the surface, slope, ponding characteristics of the area, soil types, and vegetation. It should be noted that the runoff coefficient used in the rational formula is the variable least susceptible to precise determination. A reasonable coefficient must be selected to represent the overall effects of infiltration, ponding, and interception.

Recommended values for the runoff coefficient "C" for various surface types and storm frequencies are given in Table 3-1.

If the drainage area contributing runoff to a certain design point is composed of several surfaces for which different coefficients must be assigned, the coefficient used in the rational formula should be a weighted average based on the percentage of each surface type.

3.4.2 Rainfall Intensity

Rainfall intensity is the average rainfall rate expressed in inches of rainfall per hour, and varies with the storm duration and design frequency.

A. Time of Concentration

According to the underlying theory of the rational method, the maximum discharge at any point in a drainage system occurs when the entire area, tributary to that point, is contributing to the flow. The rainfall intensity is based on the rate of rainfall which can be expected to fall in the time required for water to flow from the most remote point of the area to the point being investigated. The "most remote point" is the point from which the time of flow is greatest, and may not be at the greatest linear distance from the point under investigation. The time at which maximum discharge occurs is referred to as the time of concentration. It is composed of two components: (1) "inlet time" and (2) "flow time". The inlet time is the time required for water to flow overland from the most remote point in the drainage sub-area to a drainage ditch or inlet. The flow time is the time during which water flows through the drainage system to any point being investigated.

It should be noted that for some irregularly shaped drainage areas, the runoff may reach a peak prior to the time that all areas are contributing to the flow. In such situations sound engineering judgment is required to determine the time of concentration which will produce the highest rate of runoff.

(1) Inlet Time

The inlet time will vary with the surface characteristics and slope of the drainage area. It is characterized by relatively shallow, slow moving, unconcentrated flow, which occurs overland in shallow swales or in street gutters. The nomographs, Figure 3-1 "Time of Concentration for Overland Flow" and Figure 3-2 "Time of Concentration for Gutter Flow", will provide adequate estimates of inlet time in most situations.

(2) Flow Time

The flow time is the time required for the water to flow from an inlet or collection point, through pipes or channels, to the point being investigated. It is characterized by swiftly moving, concentrated flow in a defined channel of conduit. The average flow velocity in an open channel can be determined by using Manning's equation or backwater profiles as discussed in Sections 6 and 7 of this manual. For storm sewer pipe flowing full, (pressure flow conditions), the average velocity in the pipe is computed using the relationship,

$$V = Q/A \qquad \text{(Eq. 3-2)}$$

where,

V = Average velocity in feet per second.
Q = Design discharge in cubic feet per second.
A = Cross-sectional area of pipe in square feet.

The flow time may now be determined by direct computation using the following formula:

$$T_f = L/(60\ V \qquad \text{(Eq. 3-3)}$$

where,

T_f = Time of flow in minutes.
L = Flow distance in feet.
V = Average flow velocity in feet per second.

B. Intensity Equations

The following equation may be used to determine the peak rainfall intensity for storm durations ranging from 5 minutes to 24 hours.

$$I_{2\ year} = 70/(T_c+8.9)^{0.810} \qquad \text{(Eq. 3-4)}$$

$$* \ \mathbf{I_{5\ year} = 73/(T_c+8.4)^{0.772}} \qquad \textbf{(Eq. 3-5)}$$

$$I_{10\ year} = 81/(T_c+8.4)^{0.760} \qquad \text{(Eq. 3-6)}$$

$$* \ \mathbf{I_{25\ year} = 88/(T_c+8.4)^{0.745}} \qquad \textbf{(Eq. 3-7)}$$

$$I_{50\ year} = 95/(T_c+8.4)^{0.737} \qquad \text{(Eq. 3-8)}$$

$$I_{100\ year} = 96/(T_c+8.9)^{0.720} \qquad \text{(Eq. 3-9)}$$

where,

I_f = Peak rainfall intensity in inches per hour for a storm with a return frequency f.

T_c= Time of concentration or storm duration in minutes.

* 5 and 25 year equations are used for drainage areas less than 250 acres. Other equations are shown for reference only.

Figure 3-3 shows the intensity-duration-frequency curves for Calhoun County and may be used to provide a graphical solution to Equations 3-4 through 3-9.

The rainfall intensity equations above are based on Technical Paper No. 40, prepared by the U.S. Department of Commerce and the Weather Bureau.

Tables 3-2 and 3-3 show the total runoff per acre for a 5 and 25 year frequency rain assuming a range of runoff coefficients (C) and times of concentration (T_c).

3.4.3 Drainage Area

The drainage area used in the rational formula is the portion of the watershed, in acres, that contributes flow to the point being studied. A map shall be prepared for each project showing the ultimate area which will contribute flow to the drainage system being designed. The portion of the drainage area being developed shall be further subdivided to show the drainage sub-areas contributing to each inlet. Division lines between drainage areas and sub-areas are determined by topography, street layout, lot grading, and inlet location.

3.5.0 REGIONAL REGRESSION EQUATIONS

The U.S. Geological Survey, in cooperation with the Texas State Department of Highways and Public Transportation, has developed statewide regression equations which relate peak flow rates to the physiographic, hydrologic, and meteorologic characteristics of watersheds in Texas. The results of the USGS study were published in USGS open-file report 77-110 "Technique for Estimating the Magnitude and Frequency of Floods in Texas". In this report the state is divided into six regions and mathematical equations are given for estimating the peak flow rate from storms with recurrence intervals of 2, 5, 10, 25, 50, and 100 years. The equations applicable for undeveloped watersheds in Calhoun County are discussed in Section 3.5.1. These equations were later modified by the TSDHPT in order to account for the effect urbanization has on a watershed. The equations applicable for urbanized watersheds in Calhoun County are discussed in Section 3.5.2 and Section 3.5.3.

3.5.1 Rural Watersheds

For undeveloped watersheds in Calhoun County, with an area greater than 250 acres, the equations used for determining peak flows are as follows:

$$Q_{2\ year} = 2.586\ A^{0.629}\ S^{0.130} \qquad \text{(Eq. 3-10)}$$

$$Q_{5\ year} = 3.833\ A^{0.685}\ S^{0.254} \qquad \text{(Eq. 3-11)}$$

$$Q_{10\ year} = 4.568\ A^{0.714}\ S^{0.317} \qquad \text{(Eq. 3-12)}$$

$$Q_{25\ year} = 5.334\ A^{0.747}\ S^{0.386} \qquad \text{(Eq. 3-13)}$$

$$Q_{50\ year} = 5.839\ A^{0.769}\ S^{0.431} \qquad \text{(Eq. 3-14)}$$

$$* \ \mathbf{Q_{100\ year} = 6.202\ A^{0.788}\ S^{0.469}} \qquad \textbf{(Eq. 3-15)}$$

where,

Q_f = Peak runoff in cubic feet per second from a storm with a return frequency f.

A = Area in acres which contributes runoff to the point being studied.

S = Average slope of the drainage basin, expressed in feet per 100 feet (percent).

* 100 year flow rate is used for all areas greater than 25 acres. Other equations are shown for reference only.

3.5.2 Urban Watersheds

For urbanized watersheds in Calhoun County, with an area greater than 250 acres, the equations used for determining peak flows are as follows:

$$Q_{2\ year} = 6.800\ A^{0.669}\ S^{0.095}\ (13-BDF)^{-0.43} \qquad \text{(Eq. 3-16)}$$

$$Q_{5\ year} = 10.215\ A^{0.711}\ S^{0.201}\ (13-BDF)^{-0.39} \qquad \text{(Eq. 3-17)}$$

$$Q_{10\ year} = 11.230\ A^{0.724}\ S^{0.250}\ (13-BDF)^{-0.36} \qquad \text{(Eq. 3-18)}$$

$$Q_{25\ year} = 12.567\ A^{0.748}\ S^{0.309}\ (13-BDF)^{-0.34} \qquad \text{(Eq. 3-19)}$$

$$Q_{50\ year} = 12.964\ A^{0.781}\ S^{0.353}\ (13-BDF)^{-0.32} \qquad \text{(Eq. 3-20)}$$

$$* \ \mathbf{Q_{100\ year} = 13.045\ A^{0.796}\ S^{0.385}\ (13-BDF)^{-0.32}} \qquad \textbf{(Eq. 3-21)}$$

where,

Q_f = Peak runoff in cubic feet per second from a storm with a return frequency f.

A = Area in acres which contributes runoff to the point being studied.

S = Average slope of the drainage basin, expressed in feet per 100 feet (percent).

BDF = Basin Development Factor. See Section 3.5.3 below for discussion.

* 100 year flow rate is used for all areas greater than 250 acres. Other equations are shown for reference only.

3.5.3 Basin Development Factor For Urban Watersheds

The basin development factor (BDF) is an index of urbanization which provides a measure of the drainage efficiency for an urbanized watershed. The following discussion outlines the procedure for determining the BDF of an urbanized watershed.

The basin is divided into three sub-areas. Each section contains approximately one-third of the watershed area, and the division lines are made so that the travel distances along the major watercourses within each sub-area are about equal.

Within each of the three sub-areas, several aspects of the drainage system are evaluated and assigned a factor as follows:

A. Channel Improvement Factor

If more than 50 percent of the main channel and principal tributaries have been improved by straightening, deepening, clearing, etc., the channel improvement factor is assigned a value of one (1). If such improvements do not exist, the channel improvement factor is zero (0).

B. Channel Lining Factor

If more than 50 percent of the main channel and principal tributaries have been improved by lining with an impervious material such as concrete, the channel lining factor is assigned a value of one (1). If such improvements do not exist, the channel lining factor is zero (0).

C. Storm Drain Factor

If more than 50 percent of the secondary tributaries within a sub-area consist of storm drains, the storm drain factor is assigned a value of one (1). If such improvements do not exist, the storm drain factor is zero (0).

D. Curb and Gutter Factor

If more than 50 percent of a sub-area is urbanized, and more then 50 percent of the streets are constructed with curb and gutter, the curb and gutter factor is assigned a value of one (1). If such improvements do not exist, the curb and gutter factor is zero (0).

The Basin Development Factor (BDF) is calculated as the sum of the above factors for each of the three sub-areas. The maximum value of the BDF is twelve (12) and the minimum value is zero (0).

When forecasting the effects that urbanization will have on a drainage basin, the value used for the Basin Development Factor must be given careful consideration. It is recommended that in such instances a BDF of ten (10) or twelve (12) be assumed.

It should be noted that the peak flows calculated using a BDF of zero (0) will be larger than those calculated using the equations in Section 3.5.1 for rural watersheds. This is because the urban equations allow for the effects of urban development even if the extent of that development falls below the 50 percent level.

END OF SECTION

TABLE 3-1

RATIONAL METHOD RUNOFF COEFFICIENTS
(For Use With Equation 3-1)

Character of Surface	Runoff Coefficients (C) Return Frequency					
	2 year	5 year	10 year	25 year	50 year	100 year
Streets						
Asphalt	0.73	0.78	0.82	0.88	0.92	0.95
Concrete	0.79	0.84	0.87	0.93	0.96	0.99
Drives & Walks	0.79	0.84	0.87	0.93	0.96	0.99
Roofs	0.79	0.84	0.87	0.93	0.96	0.99
Lawns & Parks						
Sandy Soil, < 2% Slope	0.06	0.07	0.07	0.08	0.09	0.09
Sandy Soil, > 2% Slope	0.10	0.11	0.12	0.13	0.14	0.15
Clay Soil, < 2% Slope	0.16	0.18	0.19	0.20	0.21	0.22
Clay Soil, > 2% Slope	0.20	0.21	0.23	0.24	0.26	0.27
Cultivated & Pasture Land						
Sandy Soil, < 2% Slope	0.10	0.11	0.12	0.13	0.14	0.15
Sandy Soil, > 2% Slope	0.18	0.19	0.21	0.22	0.24	0.25
Clay Soil, < 2% Slope	0.27	0.29	0.31	0.33	0.35	0.37
Clay Soil, > 2% Slope	0.35	0.38	0.41	0.44	0.47	0.50

Description of Area	Typical Runoff Coefficients (C) Return Frequency					
	2 year	5 year	10 year	25 year	50 year	100 year
Residential						
Average Lot Size						
> 1/2 Acre	0.27	0.29	0.30	0.33	0.36	0.38
1/4 to 1/2 Acre	0.36	0.38	0.40	0.44	0.48	0.50
1/4 to 1/8 Acre	0.45	0.48	0.50	0.55	0.60	0.62
< 1/8 Acre	0.51	0.54	0.57	0.63	0.68	0.71
Apartments	0.55	0.58	0.61	0.67	0.73	0.76
Commercial						
Light	0.55	0.58	0.61	0.67	0.73	0.76
Heavy	0.67	0.70	0.74	0.81	0.89	0.93
Industrial						
Light	0.48	0.50	0.53	0.58	0.64	0.66
Heavy	0.57	0.60	0.63	0.69	0.76	0.79

TABLE 3-2

RATE OF RUNOFF PER ACRE FOR 5 YEAR DESIGN FREQUENCY
(Product of C x I, For Use With Equation 3-1)

Time of Concentration (Tc)	Runoff Coefficients (C)								
	0.10	0.20	0.30	0.40	0.50	0.60	0.70	0.80	0.90
5 min.	0.98	1.97	2.95	3.94	4.92	5.91	6.89	7.88	8.86
10	0.77	1.54	2.31	3.08	3.85	4.62	5.39	6.17	6.94
15	0.64	1.28	1.92	2.56	3.20	3.84	4.48	5.12	5.76
20	0.55	1.10	1.65	2.21	2.76	3.31	3.86	4.41	4.96
25	0.49	0.97	1.46	1.95	2.43	2.92	3.40	3.89	4.38
30	0.44	0.87	1.31	1.75	2.18	2.62	3.06	3.49	3.93
35	0.40	0.79	1.19	1.59	1.99	2.38	2.78	3.18	3.58
40	0.37	0.73	1.10	1.46	1.83	2.19	2.56	2.92	3.29
45	0.34	0.68	1.02	1.35	1.69	2.03	2.37	2.71	3.05
50	0.32	0.63	0.95	1.26	1.58	1.90	2.21	2.53	2.84
55	0.30	0.59	0.89	1.19	1.48	1.78	2.08	2.37	2.67
60	0.28	0.56	0.84	1.12	1.40	1.68	1.96	2.24	2.52
65	0.26	0.53	0.79	1.06	1.32	1.59	1.85	2.12	2.38
70	0.25	0.50	0.76	1.01	1.26	1.51	1.76	2.01	2.27
75	0.24	0.48	0.72	0.96	1.20	1.44	1.68	1.92	2.16
80	0.23	0.46	0.69	0.92	1.15	1.38	1.61	1.84	2.06
85	0.22	0.44	0.66	0.88	1.10	1.32	1.54	1.76	1.98
90	0.21	0.42	0.63	0.84	1.06	1.27	1.48	1.69	1.90
95	0.20	0.41	0.61	0.81	1.02	1.22	1.42	1.63	1.83
100	0.20	0.39	0.59	0.78	0.98	1.18	1.37	1.57	1.76
110	0.18	0.37	0.55	0.73	0.92	1.10	1.28	1.46	1.65
120	0.17	0.34	0.52	0.69	0.86	1.03	1.20	1.38	1.55
130	0.16	0.32	0.49	0.65	0.81	0.97	1.14	1.30	1.46
140	0.15	0.31	0.46	0.62	0.77	0.92	1.08	1.23	1.38
150	0.15	0.29	0.44	0.59	0.73	0.88	1.02	1.17	1.32
160	0.14	0.28	0.42	0.56	0.70	0.84	0.98	1.12	1.26
170	0.13	0.27	0.40	0.53	0.67	0.80	0.93	1.07	1.20
180	0.13	0.26	0.38	0.51	0.64	0.77	0.90	1.02	1.15
190	0.12	0.25	0.37	0.49	0.61	0.74	0.86	0.98	1.11
200	0.12	0.24	0.36	0.47	0.59	0.71	0.83	0.95	1.07
210	0.11	0.23	0.34	0.46	0.57	0.68	0.80	0.91	1.03
220	0.11	0.22	0.33	0.44	0.55	0.66	0.77	0.88	0.99
230	0.11	0.21	0.32	0.43	0.53	0.64	0.75	0.85	0.96
240	0.10	0.21	0.31	0.41	0.52	0.62	0.72	0.83	0.93

1. Values in table are equal to the total rate of runoff, in cubic feet per second, per acre of drainage area, for a 5 year frequency rainfall.

2. To obtain peak runoff rate, multiply value in table by drainage area in acres.

3. Table is not applicable for drainage areas greater than 250 acres.

TABLE 3-3

RATE OF RUNOFF PER ACRE FOR 25 YEAR DESIGN FREQUENCY
(Product of C x I, For Use With Equation 3-1)

Time of Concentration (Tc)	Runoff Coefficients (C)								
	0.10	0.20	0.30	0.40	0.50	0.60	0.70	0.80	0.90
5 min.	1.27	2.55	3.82	5.09	6.36	7.64	8.91	10.18	11.46
10	1.01	2.01	3.02	4.02	5.03	6.03	7.04	8.04	9.05
15	0.84	1.68	2.52	3.36	4.20	5.04	5.88	6.72	7.56
20	0.73	1.45	2.18	2.91	3.64	4.36	5.09	5.82	6.55
25	0.64	1.29	1.93	2.58	3.22	3.87	4.51	5.16	5.80
30	0.58	1.16	1.74	2.32	2.90	3.49	4.07	4.65	5.23
35	0.53	1.06	1.59	2.12	2.65	3.18	3.71	4.24	4.77
40	0.49	0.98	1.47	1.96	2.44	2.93	3.42	3.91	4.40
45	0.45	0.91	1.36	1.82	2.27	2.73	3.18	3.64	4.09
50	0.43	0.85	1.28	1.70	2.13	2.55	2.98	3.40	3.83
55	0.40	0.80	1.20	1.60	2.00	2.40	2.80	3.20	3.60
60	0.38	0.76	1.13	1.51	1.89	2.27	2.65	3.02	3.40
65	0.36	0.72	1.08	1.43	1.79	2.15	2.51	2.87	3.23
70	0.34	0.68	1.02	1.37	1.71	2.05	2.39	2.73	3.07
75	0.33	0.65	0.98	1.30	1.63	1.96	2.28	2.61	2.93
80	0.31	0.62	0.94	1.25	1.56	1.87	2.19	2.50	2.81
85	0.30	0.60	0.90	1.20	1.50	1.80	2.10	2.40	2.70
90	0.29	0.58	0.86	1.15	1.44	1.73	2.02	2.31	2.59
95	0.28	0.56	0.83	1.11	1.39	1.67	1.94	2.22	2.50
100	0.27	0.54	0.80	1.07	1.34	1.61	1.88	2.15	2.41
110	0.25	0.50	0.75	1.00	1.26	1.51	1.76	2.01	2.26
120	0.24	0.47	0.71	0.95	1.18	1.42	1.65	1.89	2.13
130	0.22	0.45	0.67	0.89	1.12	1.34	1.56	1.79	2.01
140	0.21	0.42	0.64	0.85	1.06	1.27	1.49	1.70	1.91
150	0.20	0.40	0.61	0.81	1.01	1.21	1.42	1.62	1.82
160	0.19	0.39	0.58	0.77	0.97	1.16	1.35	1.55	1.74
170	0.19	0.37	0.56	0.74	0.93	1.11	1.30	1.48	1.67
180	0.18	0.36	0.53	0.71	0.89	1.07	1.24	1.42	1.60
190	0.17	0.34	0.51	0.68	0.85	1.03	1.20	1.37	1.54
200	0.16	0.33	0.49	0.66	0.82	0.99	1.15	1.32	1.48
210	0.16	0.32	0.48	0.64	0.80	0.95	1.11	1.27	1.43
220	0.15	0.31	0.46	0.62	0.77	0.92	1.08	1.23	1.39
230	0.15	0.30	0.45	0.60	0.75	0.89	1.04	1.19	1.34
240	0.14	0.29	0.43	0.58	0.72	0.87	1.01	1.16	1.30

1. Values in table are equal to the total rate of runoff, in cubic feet per second, per acre of drainage area, for a 25 year frequency rainfall.

2. To obtain peak runoff rate, multiply value in table by drainage area in acres.

3. Table is not applicable for drainage areas greater than 250 acres.

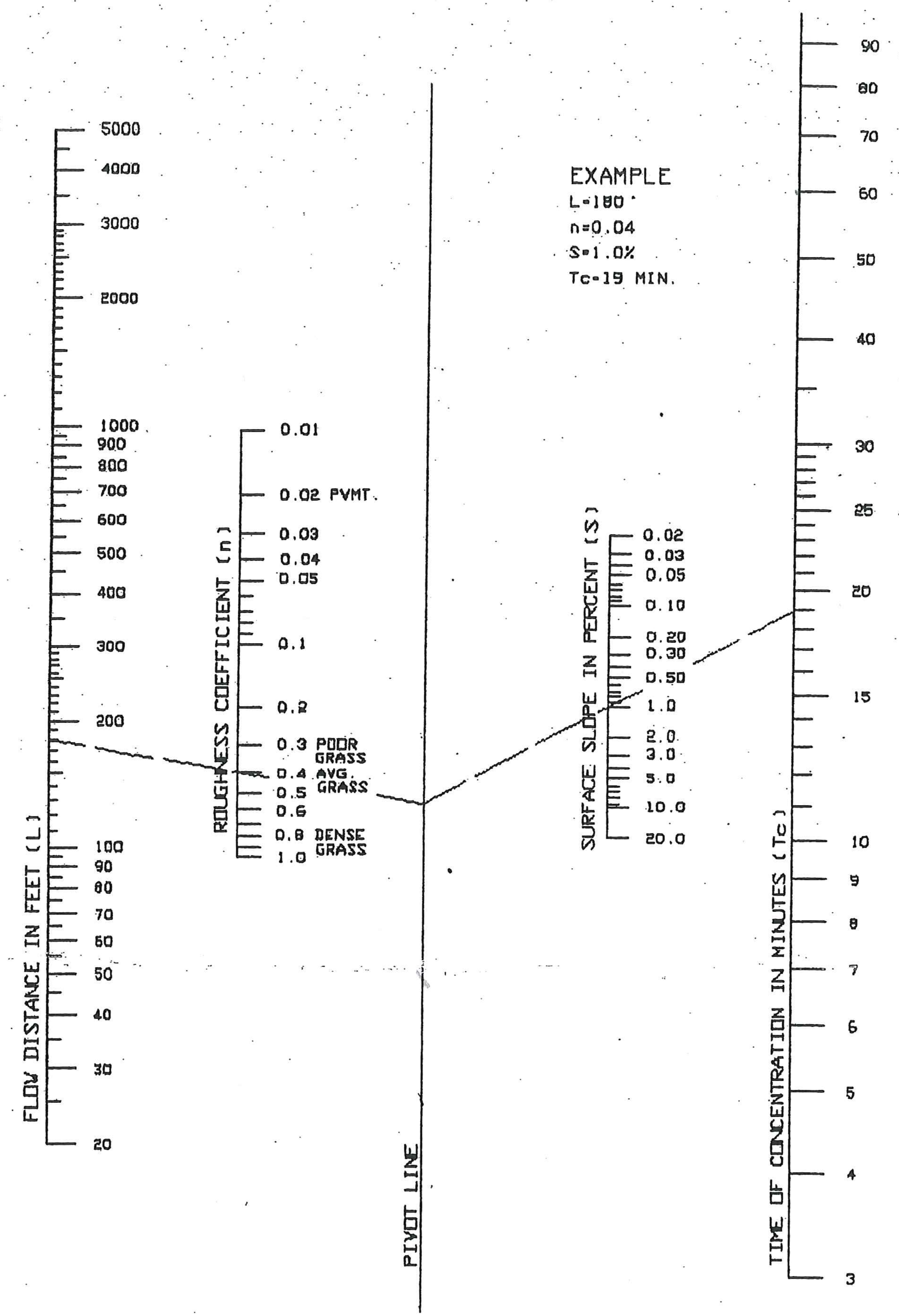


FIGURE 3-1
TIME OF CONCENTRATION FOR OVERLAND FLOW

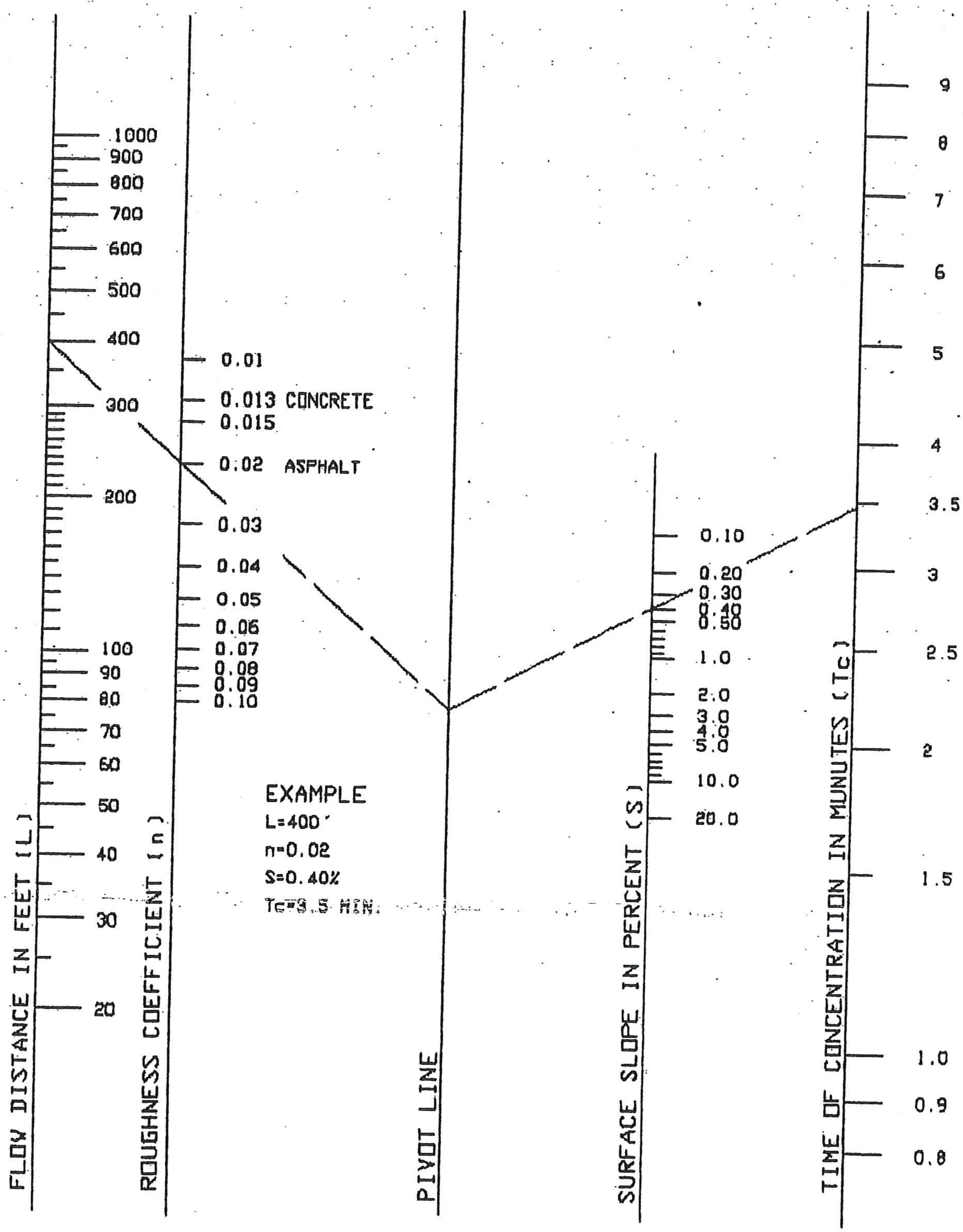


FIGURE 3-2
TIME OF CONCENTRATION FOR GUTTER FLOW

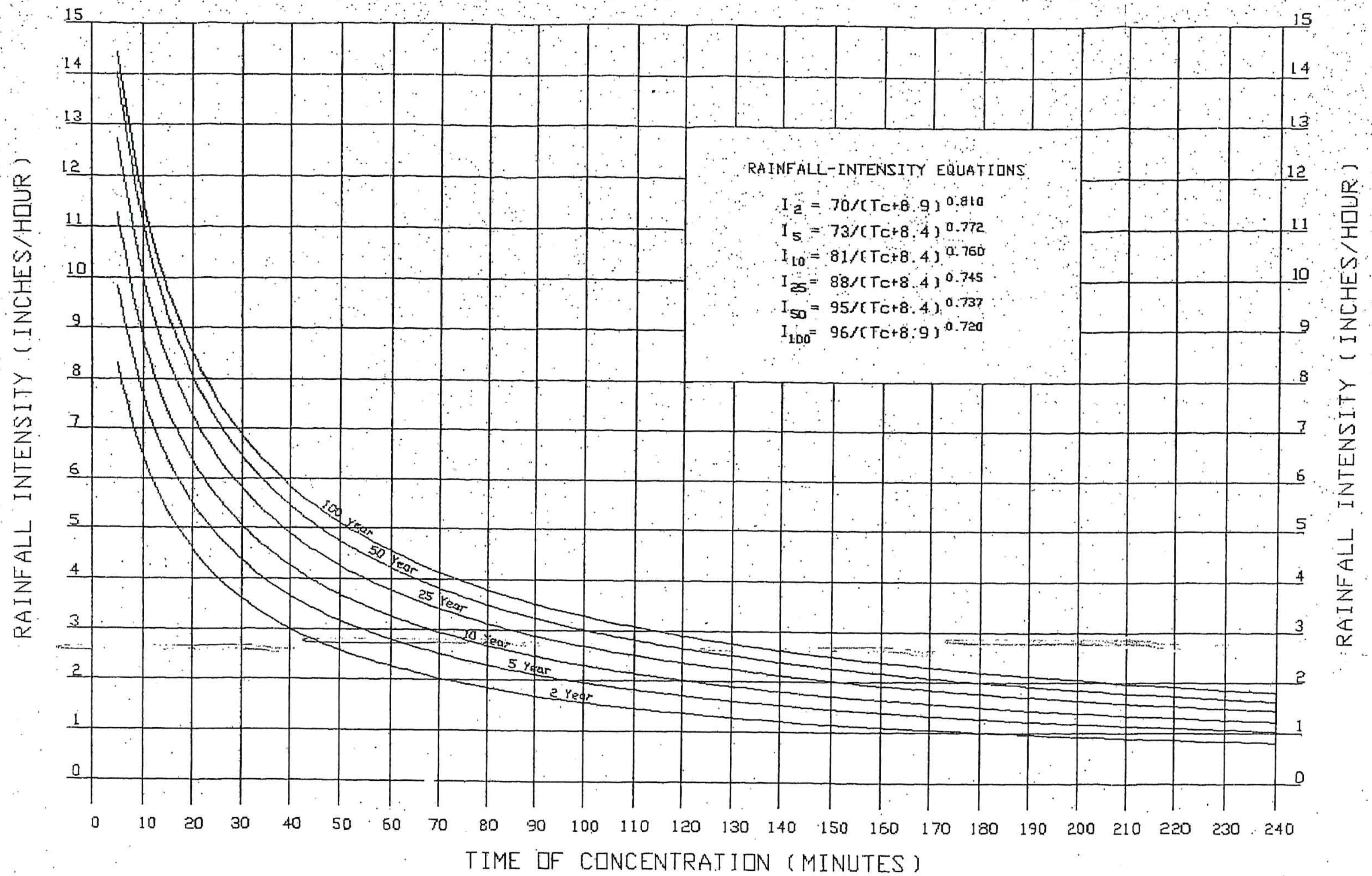


FIGURE 3-3
INTENSITY - DURATION - FREQUENCY CURVES

EXAMPLE 3-1

RATIONAL METHOD FOR SMALL WATERSHEDS

PROBLEM:

Determine the expected peak rate of runoff (Q_p) from Area "A-1" (see sketch on page 3-19) given the following design parameters.

Design Frequency	5 year	Total Area	1.60 acres
Roof Area	0.34 acres	Pavement Area	0.18 acres
Concrete Area	0.20 acres	Grass Area	0.88 acres
Soil Type	Clay	Average Lot Slope	1.0 %
Gutter Slope	0.40 %		

SOLUTION:

1. Determine a Composite Runoff Coefficient.

Roof Area = 0.34/1.60 =	21%	$C_{5\ year} = 0.84$
Pavement Area = .18/1.60 =	11%	$C_{5\ year} = 0.78$
Concrete Area = .20/1.60 =	13%	$C_{5\ year} = 0.84$
Grass Area = .88/1.60 =	55%	$C_{5\ year} = 0.18$

The composite runoff coefficient (C) =
(.21x.84)+(.11x.78)+(.13x.84)+(.55x.18) = <u>0.47</u>.

2. Determine Critical Time of Concentration.

The longest possible flow distance for Area "A-1" is 180 feet of average grass at 1.0% slope plus 400 feet of gutter at 0.40% slope.

From Figure 3-1, using a flow distance of 180 feet, a roughness coefficient of 0.40 for average grass, and an average slope of 1.0%, the time of concentration = 19 minutes.

From Figure 3-2, using a flow distance of 400 feet, a roughness coefficient of 0.20 for asphalt, and a slope of 0.40%, the time of concentration = 3.5 minutes.

Therefore, the critical time of concentration (T_c) to Inlet "A" equals 19 minutes plus 3.5 minutes, or <u>22.5 minutes</u>.

3. Determine Rainfall Intensity.

Using Equation 3-5 for a 5 year frequency storm and a time of concentration (T_c) of 22.5 minutes, the rainfall intensity,

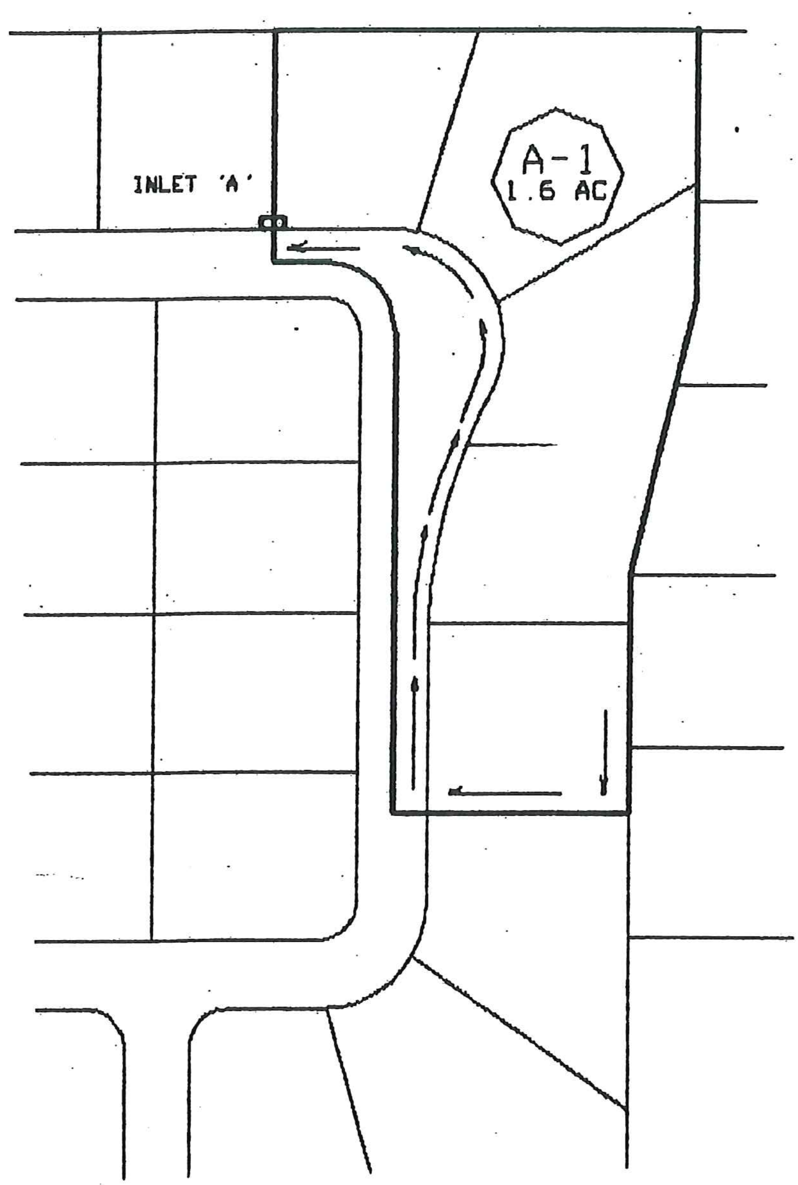
$(I_{5\ year}) = 73/(22.5+8.4)^{0.772}$ = <u>5.17 inches per hour</u>.

4. Determine Peak Rate of Runoff.

Using Equation 3-1, with C = 0.47, I = 5.17 in/hr, and A = 1.60 ac,
$Q_p = 0.47 \times 5.17 \times 1.60$ = <u>3.89 cfs</u>.

DESIGN DATA

TOTAL AREA	1.60 AC
ROOF AREA	0.34 AC
PAVEMENT AREA	0.18 AC
CONCRETE AREA	0.20 AC
GRASS AREA	0.90 AC
SOIL TYPE	CLAY



EXAMPLE 3-1
RATIONAL METHOD FOR SMALL WATERSHEDS

EXAMPLE 3-2

REGIONAL REGRESSION METHOD FOR RURAL WATERSHEDS

PROBLEM:

Determine the peak rate of runoff from an undeveloped tract of farm land given the following design parameters.

Design Frequency = 100 year
Tract Size = 300 acres

SOLUTION:

1. Determine Average Stream Bed Slope.

By the use of topographic maps or field survey, determine the average slope of the stream bed. The average slope is normally considered to be the stream bed slope along the middle 75% of the main stream channel. In areas drained by many small channels or where drainage is interrupted by levies, the average slope of the tract will usually prove sufficient.

For the purposes of this example assume an average stream bed slope of 0.30 percent, (0.30 feet per 100 feet).

2. Determine 100 Year Peak Runoff.

Using Equation 3-15, with an area of 300 acres and an average slope of 0.30 percent,

$Q_{100\ year} = 6.202 \times 300^{0.788} \times 0.30^{0.469} = \underline{316\ cfs}$.

EXAMPLE 3-3

REGIONAL REGRESSION METHOD FOR URBAN WATERSHEDS

PROBLEM:

Determine the peak rate of runoff for the same 300 acre tract used in Example 3-2, but assume 100 percent urbanization of the watershed.

Design Frequency = 100 year
Tract Size = 300 acres
Average Slope = 0.30%

SOLUTION:

1. Determine a Basin Development Factor (BDF).

If the tract of land being considered were currently developed, the designer could follow the procedure set out in Section 3.4.3 to determine a reasonable BDF. However, in this example, the regional regression method is being used to estimate the effects of future development of the watershed. Therefore, a basin development factor (BDF) of 10 will be assumed.

2. Determine 100 Year Peak Runoff.

Using Equation 3-21, with an area of 300 acres, an average slope of 0.30 percent, and a BDF of 10,

$$Q_{100\ year} = 13.045 \times 300^{0.796} \times 0.30^{0.385} \times (13-10)^{-0.32} = \underline{541\ cfs}$$

SECTION 4
STREET CAPACITY

CONTENTS

SECTION 4 - STREET CAPACITY

4.1.0 GENERAL

Curb and gutter streets can serve as an important and necessary part of a flood protection system even though their primary function is to move traffic. A street is generally depressed below natural ground and therefore ponds and conveys water during heavy rains. The possible extent of ponding must be analyzed to determine the effects on traffic as well as the potential for flooding of nearby homes and structures. Water which flows in a street, whether from rainfall directly onto the pavement surface or overland flow entering from adjacent land areas, will flow in the gutters of the street until it reaches an overflow point or some outlet, such as a storm sewer inlet. As the flow progresses downhill and additional areas contribute to the runoff, the width of flow will increase and progressively encroach into the traffic lane. When the extent of encroachment or the depth of ponding reaches the limits specified in Table 4-1, provisions must be made to intercept the water with an inlet or increase the gutter capacity by increasing the longitudinal slope of the street. Methods for determining gutter capacity for straight crowned and parabolically crowned streets are discussed below. The gutter capacities for various width pavements at varying slope have been calculated and are included in Tables 4-2 and 4-3.

The use of valley gutters and inverted crown pavement sections, (V-sections), shall not be allowed except in the case of concrete streets or alleys.

4.2.0 METHODS OF ANALYSIS

The rate at which water will flow in a channel is governed principally by the slope of the channel, the area and shape of the channel cross section, and the roughness of the surface in contact with the water. The formula generally used to determine the rate of flow is called Manning's equation. The expression is familiar to most hydraulic engineers as,

$$Q = 1.486/n \; A \; R^{2/3} \; S^{1/2} \qquad \text{(Eq. 4-1)}$$

where,

Q = Flow rate is cubic feet per second.
n = Manning's roughness coefficient.
A = Cross-sectional area of flow in square feet.
R = Hydraulic radius - defined as the area/wetted perimeter.
S = Longitudinal slope of the channel in feet per foot.
(Minimum allowable slope shall be 0.20 percent)

When the width of the channel is large in relation to the depth, as in the case of street flow, the value for the hydraulic radius (R) is approximately equal to the mean depth of flow.

4.2.1 Straight Crowned Streets

The flow in the gutter along a straight crowned street takes the shape of a triangular channel. For this special situation, Manning's equation was modified by C. F. Izzard in 1950 to include the transverse slope of the channel and may be expressed in the following form:

$$Q = 0.56\ Z/n\ D^{8/3}\ S^{1/2} \qquad \text{(Eq. 4-2)}$$

where,

Q = Gutter flow in cubic feet per second.
Z = 1/the cross slope of the pavement in feet per foot.
n = Manning's roughness coefficient (0.018 recommended).
D = Depth of flow at the gutter in feet.
S = Longitudinal gutter slope in feet per foot.
(Minimum allowable slope shall be 0.20 percent)

The gutter capacities for various width pavements at varying slope have been calculated and are included in Table 4-2. For streets with longitudinal slopes between those given, gutter capacity may be interpolated. The values for Table 4-2 were determined for a crown configuration which slopes at 3/8" per foot for 16 feet, then flattens to 1/4" per foot to the centerline of the pavement. The gutter capacities shown in Table 4-2 are not applicable for crown configurations other than that shown. The equation used to calculate crown height for a straight crowned pavement section is,

$$C = 6 + (W-32)/8 \qquad \text{(Eq. 4-3)}$$

where,

C = Crown height in inches, (minimum crown height is 6").
W = Street width in feet measured from back to back of curbs.

4.2.2 Parabolically Crowned Streets

Flow in the gutter of a parabolically crowned pavement section may be approximated using the following equation:

$$Q = 1.12/n\ A\ D^{2/3}\ S^{1/2} \qquad \text{(Eq. 4-4)}$$

where,

Q = Gutter flow in cubic feet per second.
n = Manning's roughness coefficient (0.018 recommended).
A = Cross-sectional area of flow in square feet.
D = Depth of flow at gutter in feet.
S = Longitudinal gutter slope in feet per foot.
(Minimum allowable slope shall be 0.20 percent)

The value for the cross-sectional area (A) is difficult to determine for each design case, therefore, gutter capacities for various width pavements and varying slopes have been calculated and are included in Table 4-3. For streets with longitudinal slopes between those given, gutter capacity may be interpolated. The values for Table 4-3 were determined only for crown heights shown and are not otherwise applicable. The equation used to calculate crown height for a parabolically crowned pavement section is,

$$C = 7 + (W-28)/4 \qquad \text{(Eq. 4-5)}$$

where,

C = Crown height in inches.
W = Street width in feet measured from back to back of curbs.

END OF SECTION

TABLE 4-1

MINIMUM CLEAR ROADWAY WIDTH

Street Width Back to Back	Street Width Face to Face	Maximum Ponding Depth	Maximum Ponding Width	Minimum Clear Roadway Width
28 ft	27 ft	0.5 ft	13.5 ft	0 ft
30 ft	29 ft	0.5 ft	14.5 ft	0 ft
36 ft	35 ft	0.5 ft	16 ft	3 ft
40 ft	39 ft	0.5 ft	16 ft	7 ft
44 ft	43 ft	0.5 ft	16 ft	11 ft
48 ft	47 ft	0.5 ft	16 ft	15 ft
60 ft	59 ft	0.5 ft	16 ft	27 ft
66 ft	65 ft	0.5 ft	16 ft	33 ft

1. When the depth or width of ponding reaches the maximum allowable dimensions shown above, provisions shall be made to intercept the flow and direct it into an underground drainage system.

TABLE 4-2

GUTTER CAPACITY FOR STRAIGHT CROWNED PAVEMENT SECTIONS

GUTTER CAPACITY (CFS)

Street Width (ft)	28	30	36	40	44	48	60	66
Crown Height (in)	6.00	6.00	6.50	7.00	7.50	8.00	9.50	10.25
Gutter Slope (%)								
0.20	6.1	6.6	7.0	7.0	7.0	7.0	7.0	7.0
0.25	6.9	7.3	7.8	7.8	7.8	7.8	7.8	7.8
0.30	7.5	8.1	8.6	8.6	8.6	8.6	8.6	8.6
0.35	8.1	8.7	9.3	9.3	9.3	9.3	9.3	9.3
0.40	8.7	9.3	9.9	9.9	9.9	9.9	9.9	9.9
0.45	9.2	9.9	10.5	10.5	10.5	10.5	10.5	10.5
0.50	9.7	10.4	11.1	11.1	11.1	11.1	11.1	11.1
0.55	10.2	10.9	11.6	11.6	11.6	11.6	11.6	11.6
0.60	10.6	11.4	12.1	12.1	12.1	12.1	12.1	12.1
0.65	11.1	11.9	12.6	12.6	12.6	12.6	12.6	12.6
0.70	11.5	12.3	13.1	13.1	13.1	13.1	13.1	13.1
0.75	11.9	12.7	13.6	13.6	13.6	13.6	13.6	13.6
0.80	12.3	13.1	14.0	14.0	14.0	14.0	14.0	14.0
0.85	12.6	13.6	14.5	14.5	14.5	14.5	14.5	14.5
0.90	13.0	13.9	14.9	14.9	14.9	14.9	14.9	14.9
0.95	13.4	14.3	15.3	15.3	15.3	15.3	15.3	15.3
1.00	13.7	14.7	15.7	15.7	15.7	15.7	15.7	15.7

1. Street width is measured from back of curb to back of curb.

2. Gutter capacity shown is for one side of the street.

3. Crown height is calculated using a cross slope of 3/8" per foot for the first 16 feet from the curb, then flattening to 1/4" per foot to the pavement centerline. The equation for calculating crown height for a straight crown pavement section is, $C = 6 + (W-32)/8$. (The minimum crown height is 6").

4. Gutter capacities do not apply for crown configurations other than those shown.

TABLE 4-3

GUTTER CAPACITY FOR PARABOLICALLY CROWNED PAVEMENT SECTIONS

GUTTER CAPACITY (CFS)

Street Width (ft)	28	30	36	40
Crown Height (in)	7.00	7.50	9.00	10.00
Gutter Slope (%)				
0.20	3.1	3.1	3.0	2.9
0.25	3.5	3.4	3.3	3.2
0.30	3.8	3.8	3.6	3.6
0.35	4.1	4.1	3.9	3.8
0.40	4.4	4.3	4.2	4.1
0.45	4.7	4.6	4.4	4.4
0.50	4.9	4.8	4.7	4.6
0.55	5.2	5.1	4.9	4.8
0.60	5.4	5.3	5.1	5.0
0.65	5.6	5.5	5.3	5.2
0.70	5.8	5.7	5.5	5.4
0.75	6.1	5.9	5.7	5.6
0.80	6.3	6.0	5.9	5.8
0.85	6.4	6.3	6.1	6.0
0.90	6.6	6.5	6.3	6.2
0.95	6.8	6.7	6.4	6.3
1.00	7.0	6.9	6.6	6.5

1. Street width is measured from back of curb to back of curb.

2. Gutter capacity shown is for one side of the street.

3. Crown height for parabolically crowned pavement sections are calculated using the equation, $C = 7.0 + (W-28)/4$.

4. Gutter capacities do not apply for crown configurations other than those shown.

4. Parabolic crowns are not recommended for streets greater than 40 feet in width.

SECTION 5
INLET CAPACITY

CONTENTS

SECTION 5 - INLEET CAPACITY

5.1.0 GENERAL

The purpose of any storm sewer inlet is to intercept excess surface runoff and direct it into an underground drainage system. If an inlet is undersized, it will cause excessive ponding and possible flooding of surrounding structures. Inlets should be designed so as not to interfere with pedestrian, bicycle, and vehicular traffic. Grate inlets should be used with extreme caution because of their increased tendency to clog and shall never be used within a public street. In order to resist clogging the height of the opening in a curb or throat type inlet shall be six (6) inches. Methods for calculating the capacity of various type inlets are provided in Section 5.2.0.

5.2.0 INLET HYDRAULICS

Flow through any inlet opening is governed by either of two flow equations, depending on the depth of ponding above or in front of the inlet opening.

5.2.1 Unsubmerged Inlets

For unsubmerged inlets, the flow through the opening may be considered weir flow and is governed by the following equation:

$$Q = CLH3/2 \qquad \text{(Eq. 5-1)}$$

where,

Q = Rate of flow through the inlet in cubic feet per second.
C = Weir coefficient. (3.0 recommended)
L = Length of opening, in feet, over which the water enters the inlet.
H = Depth of water above the weir in feet.

5.2.2 Submerged Inlets

In the case of a submerged inlet opening, where the water entering the inlet is ponding to a sufficient depth above the opening, the flow through the inlet is calculated as orifice flow. The equation governing orifice flow is,

$$Q = CA_n(2gH)^{1/2} \qquad \text{(Eq. 5-2)}$$

where,

Q = Rate of flow through the inlet in cubic feet per second.
C = Coefficient of discharge. (Section 5.2.1 and Table 5-1).
A_n = Net open area of the orifice in square feet.
g = acceleration of gravity. (32.2 ft/sec^2)
H = Depth of water in feet above the center of the orifice.

Inlets may be divided into several types: (1) throat or curb inlets, (2) grate inlets, (3) inlets at low points, and (4) inlets on grade. If the opening of the inlet is submerged, the flow through any of these types of inlets is determined using Equation 5-2. However, each inlet type will have a different discharge coefficient and efficiency factor. The maximum discharge coefficients and minimum efficiency factors are explained below for various inlet types. Table 5-1 contains a summary of all discharge coefficients and efficiency factors for easy reference.

A. Throat Inlets at Low Points

For calculating the capacity of throat inlets at low points use Equation 5-2 with a coefficient of discharge (C) equal to 0.60. No reduction for efficiency or clogging is required.

B. Throat Inlets on Grade

For throat inlets on grade, the capacity may be calculated using Equation 5-2. A coefficient of discharge (C) of 0.60 and an efficiency factor of 0.60 for bypass flow should be used. (Example C = 0.60 x 0.60 = 0.36). No correction for clogging is required.

C. Recessed Curb Inlets on Grade

For recessed curb inlets on grade, the capacity may be calculated using Equation 5-2. A coefficient of discharge (C) of 0.60 and an efficiency factor of 0.45 for bypass flow should be used. (Example C = 0.60 x 0.45 = 0.27). No correction for clogging is required.

D. Grate Inlets at Low Points

The capacity of grate inlets at low points may be calculated using either Equation 5-1, for low submergence, or Equation 5-2, for high submergence. For Equation 5-1 a weir coefficient of 3.0 is recommended. For Equation 5-2 the coefficient of discharge (C), equal to 0.60, should be reduced to allow for trash build up and clogging. The recommended clogging factors are 0.75 for grates in paved areas and 0.60 for grates in grassed areas. (Example C = 0.60 x 0.75 = 0.45 for paved areas ; C = 0.60 x 0.60 = 0.36 for grassed areas).

Because a grate inlet has an increased tendency to trap debris, such as leaves, grass clippings and paper, there use is discourage.

E. Grate Inlets on Grade

The bar spacing and geometry has a pronounced effect on the capacity of a grate inlet installed on grade. Long narrow bars placed parallel to the direction of flow are the most efficient arrangement. As cross-bars are added to reduce the opening size or to add strength, the capacity of the grate is reduced. It is recommended that for calculating the capacity of a grate inlet on grade the manufacturer's catalog be consulted to determine the appropriate coefficients. Because of the possibility of clogging, it is recommended that the coefficient of discharge (C) be reduced by the factors 0.75 for paved areas and 0.60 for grassed areas.

F. Combination Inlets

A combination inlet, consisting of one or more rectangular openings and a grated opening is an acceptable alternative to the problem of clogging associated with a grate inlet alone. The capacity of a combination inlet may be determined by summing the capacities calculated by the methods described above. The appropriate reduction factors shall be applied for each calculation.

END OF SECTION

TABLE 5-1

INLET DISCHARGE COEFFICIENTS
(For Use With Equations 5-1 and 5-2)

Inlet Type	Discharge Coefficient	Efficiency Factor	**Factored Coefficient**
All Inlets Under Weir Flow (Low Flow Conditions)	3.0	1.0	**3.0**
Throat Inlet at Low Point	0.60	1.0	**0.60**
Throat Inlet on Grade	0.60	0.60	**0.36**
Recessed Curb Inlet on Grade	0.60	0.45	**0.27**
Grate Inlet at Low Point (Paved Area)	0.60	0.75	**0.45**
Grate Inlet at Low Point (Grassed Area)	0.60	0.60	**0.36**
Grate Inlet on Grade (Paved Area)	Manufacturer's Recommendation	0.75	----
Grate Inlet on Grade (Grassed Area)	Manufacturer's Recommendation	0.60	----

EXAMPLE 5-1

THROAT INLET CAPACITY
CURB INLET AT A LOW POINT

PROBLEM:

Determine the capacity of a standard curb inlet given the following design parameters,

Throat Size = 0.5 ft x 5.0 feet = 2.5 sq.ft.
Inlet Location = Low Point.
Gutter Height = 9 in. (Includes 3" gutter depression at inlet).

SOLUTION:

1. **Determine Coefficient of Discharge.**

   From Table 5-1, the coefficient of discharge (C) for a throat inlet at a low point is <u>0.60</u>.

2. **Determine Head on Orifice.**

   The head on the orifice (H) is the depth of water above the <u>center</u> of the orifice. If the water is allowed to pond to a depth even with the top of curb, then the head on the orifice equals the gutter height minus one half the orifice height. H = 0.75 - (0.5/2) = <u>0.50 ft</u>.

3. **Determine Inlet Capacity.**

   Using Equation 5-2, with a discharge coefficient (C) of 0.60, a net open area (A) of 2.5 sq.ft., and a head (H) of 0.50 feet,

   $Q = 0.60 \times 2.5 \times (2 \times 32.2 \times 0.50)^{1/2} = \underline{8.5\ \text{cfs}}$.

EXAMPLE 5-2

THROAT INLET CAPACITY
CURB INLET ON GRADE

PROBLEM:

Determine the capacity of a standard curb inlet given the following design parameters,

Throat Size = 0.5 ft x 5.0 feet = 2.5 sq.ft.
Inlet Location = On Grade.
Gutter Height = 9 in. (Includes 3" gutter depression at inlet).

SOLUTION:

1. Determine Coefficient of Discharge.

From Table 5-1, the coefficient of discharge (C) for a throat inlet on grade, including a 40% reduction in efficiency for bypass flow, is 0.36.

2. Determine Head on Orifice.

The head on the orifice (H) is the depth of water above the center of the orifice. If the water is allowed to pond to a depth even with the top of curb, then the head on the orifice equals the gutter height minus one half the orifice height. $H = 0.75 - (0.5/2) = \underline{0.50\ ft}$.

3. Determine Inlet Capacity.

Using Equation 5-2, with a discharge coefficient (C) of 0.36, a net open area (A) of 2.5 sq.ft., and a head (H) of 0.50 feet,

$$Q = 0.36 \times 2.5 \times (2 \times 32.2 \times 0.50)^{1/2} = \underline{5.1\ cfs}.$$

EXAMPLE 5-3

GRATE INLET CAPACITY

PROBLEM:

Determine the required net open area for a grate inlet given the following design parameters,

Design Flow Rate (Q) = 4.5 cfs.
Inlet Location = Low point in a paved parking lot.
Maximum Allowable Ponding Depth Above Grate = 0.50 feet.

SOLUTION:

1. **Determine Coefficient of Discharge.**

   From Table 5-1, the coefficient of discharge (C) for a grate inlet at a low point, including a 25% reduction for clogging, is <u>0.45</u>.

2. **Determine the Required Open Area.**

   Using Equation 5-2, with a discharge coefficient (C) of 0.45, a flow rate (Q) of 4.5 cfs, and a head (H) of 0.50 feet,

   $4.5 = 0.45 \times A_n \times (2 \times 32.2 \times 0.50)^{1/2}$

   By simplifying and solving for A_n,

   $4.5 = 0.45 \times A_n \times 5.6745 = 2.55\ A_n$, therefore, A_n = <u>1.76 sq.ft</u>.

3. **Select Grate.**

   By reviewing manufacturer's data for various grate sizes, the engineer should be able to select a grate having a minimum of 1.76 sq.ft. of net opening.

EXAMPLE 5-4

COMBINATION INLET CAPACITY

PROBLEM:

Develop a rating curve, (flow rate verses head), for a combination inlet given the following design parameters,

Inlet Location = Low point in a grass lined swail.
Maximum Allowable Ponding Depth at Inlet = 3.00 feet.
Throat Size = 4 opening at 0.50 ft x 3.0 ft each, 6 sq.ft. total.
Net Open Area of Grate = 2.0 sq.ft.

Refer to the inlet sketch of page 5-9.

SOLUTION:

1. Determine Coefficient of Discharge.

From Table 5-1, the coefficient of discharge (C) for a throat inlet at a low point, with no reduction for clogging, is <u>0.60</u>.

From Table 5-1, the coefficient of discharge (C) for a grate inlet at a low point, including a 40% reduction for clogging, is <u>0.36</u>.

2. Determine the Head on Each Inlet Opening for Various Water Depths.

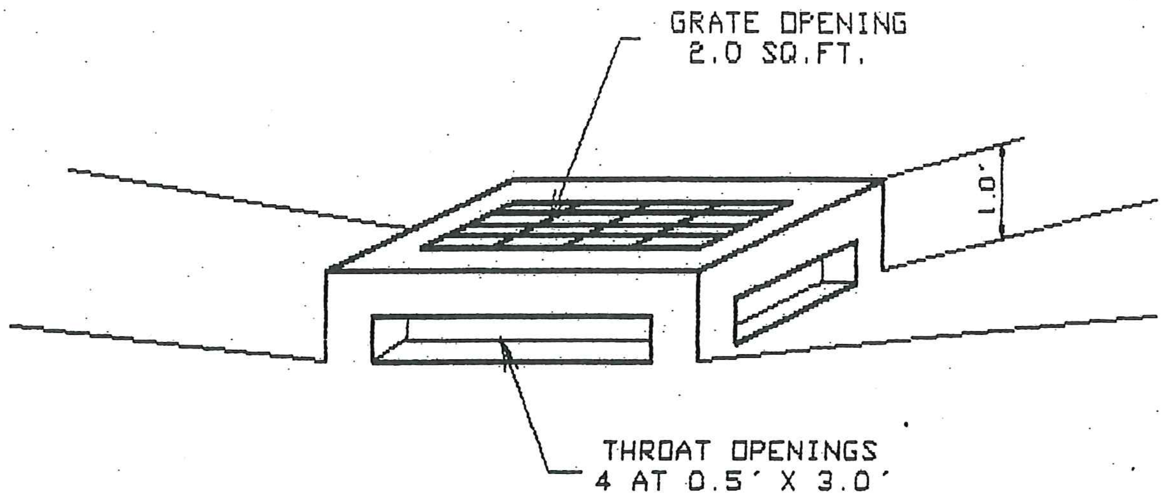
For the throat openings on the inlet, the head (H) is equal to the depth of the water above the center of the orifice, H = Water Depth (D) minus one half of the throat height, (H = D - 0.25).

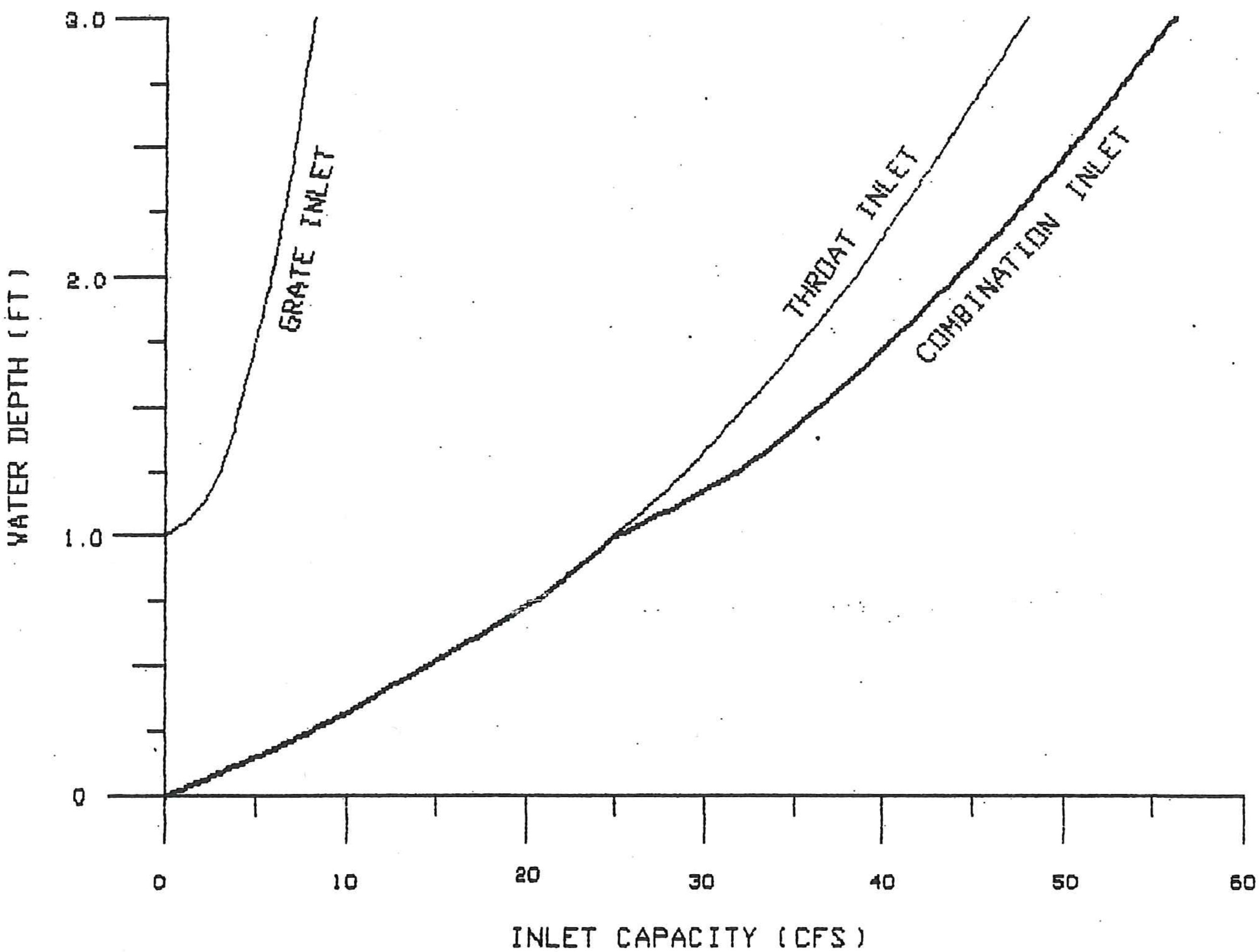
For the Grated opening on the inlet, the head (H) is equal to the depth of water above the top of the grate, (H = D - 1.0).

3. Determine the Total Inlet Capacity for a Range of Water Depth.

Using Equation 5-2, calculate the capacity of each opening for a range of water depths from 0.5 feet to 3.0 feet.

Water	Throat Inlet		Grate Inlet		Combination Inlet
Depth (ft)	Head (ft)	Capacity (cfs)	Head (ft)	Capacity (cfs)	Capacity (cfs)
0.50	0.25	14.4	0.0	0.0	14.4
0.75	0.50	20.4	0.0	0.0	20.4
1.00	0.75	25.0	0.0	0.0	25.0
1.25	1.00	28.9	0.25	2.9	31.8
1.50	1.25	32.3	0.50	4.1	36.4
1.75	1.50	35.4	0.75	5.0	40.4
2.00	1.75	38.2	1.00	5.8	44.0
2.25	2.00	40.9	1.25	6.5	47.4
2.50	2.25	43.3	1.50	7.1	50.4
2.75	2.50	45.7	1.75	7.6	53.3
3.00	2.75	47.9	2.00	8.2	56.1





EXAMPLE 5-4
RATING CURVE FOR CONBINATION INLET

SECTION 6
STORM SEWERS

CONTENTS

SECTION 6 - STORM SEWER

6.1.0 GENERAL

A properly designed storm sewer system should effectively convey the design runoff without producing upstream flooding, excessive velocities or silting. A thorough hydraulic analysis should be performed to assure that the system will operate effectively at the design discharge. Too often in the past a simplistic approach to storm sewer design was taken and conduits were sized based solely on friction loss nomographs. This approach frequently resulted in an undersized storm sewer system and upstream flooding.

This section presents the necessary equations and methods that will enable the engineer to quickly perform a comprehensive hydraulic analysis and reasonably predict the performance of the storm sewer system during the design storm event.

6.2.0 DESIGN REQUIREMENTS

There are several general requirements to observe when designing a storm sewer system. If followed, they will tend to alleviate the common mistakes made in storm sewer design.

(1) Do not discharge the contents of a larger pipe into a smaller pipe even though the capacity of the smaller pipe may be greater due to a steeper friction slope.

(2) At changes in pipe size, attempt to place the soffits of the two pipes an the same level rather than placing the flowlines at the same level. It may not be feasible to follow this rule in every instance, but an effort should be made to do so.

(3) Pipes shall not intersect at angles greater than 90 degrees, and whenever possible the intersection should be made at approximately 45 degrees or less.

(4) Select pipe sizes such that the velocity of flow will not appreciably decrease at inlets, junction boxes, manholes or changes in grade.

(5) Never use a beginning hydraulic grade line elevation lower than the soffit of the outfall pipe.

(6) Do not allow the pipe soffit to rise above the hydraulic gradient at any point in the system. If this occurs, the pipe will not run full under the design flow, the flow velocities will increase, and the actual headlosses will be greater than those calculated.

6.2.1 Allowable Pipe Materials

Pipe materials which are acceptable for use when constructing underground storm sewer systems within publicly owned rights-of-way or easements include:

(1) Reinforced Concrete Box
(2) Reinforced Concrete Pipe
(3) Corrugated Aluminum Pipe
(4) Asphalt Coated, Aluminized, Corrugated Steel Pipe
(Not allowed for installations below elevation 3.0)

6.2.2 Minimum Pipe Size

(1) Single Inlet Laterals Less Than 50 Ft Long - 15 inch diameter.
(2) All Other Storm Sewer Mains - 18 inch diameter.

6.2.3 Pipe Flow Velocities

(1) Minimum Flow Velocity - 2.0 feet per second.
(2) Maximum Flow Velocity - 10.0 feet per second.

6.2.4 Minimum Pipe Slope

In order to reduce silting under low flow conditions, the minimum slope for any storm sewer pipe shall be 0.10 percent, (i.e. 0.10 foot of fall per 100 feet).

6.2.5 Hydraulic Gradient

A. Beginning Hydraulic Gradients

In cases where a five (5) year design frequency swale or conduit enters an open channel, the elevation of the hydraulic gradient of the 25 year flow in the outfall channel shall be used as the beginning elevation for the five (5) year design. For conduits which outfall directly into the bay, the beginning hydraulic gradient elevation shall be set equal to elevation 5.0 (m.s.l.). However, in no instance shall the beginning hydraulic gradient elevation be set below the soffit of the outfall pipe.

In cases where a 25 year design frequency channel enters a 100 year design frequency channel, the hydraulic calculations for the side channel shall begin at the 25 year hydraulic gradient elevation in the outfall channel.

For 25 year design frequency channels which outfall directly into the bay, the beginning hydraulic gradient elevation shall be set equal to elevation 5.0 (m.s.l.).

For 100 year design frequency channels which outfall directly into the bay, the beginning hydraulic gradient elevation shall be set equal to elevation 12.0 (m.s.l.).

B. Maximum Hydraulic Grade Line Elevation

The elevation of the hydraulic grade line shall be calculated at every inlet, manhole, junction box, or other drainage appurtenance within the storm sewer system. The hydraulic grade line elevation within the pipe system shall not be allowed to rise above the following limits:

(1) Above a point six (6) inches below the top of curb for a curb and gutter street.
(2) Above a point six (6) inches below the centerline crown for a strip paved street.
(3) Above the center of orifice elevation for throat type inlets.
(4) Above the grate elevation for grate type inlets.
(5) Above finished natural ground.

6.2.6 Easement Requirements

Any storm sewer line, inlet, manhole, junction box, outfall, or other drainage appurtenance intended to serve more than one lot or tract of land, shall be contained within a properly dedicated drainage easement. The minimum width for such a drainage easement shall vary with the inside pipe diameter as follows:

Pipe or Box Width (D)	Easement Width
15" - 48"	15 feet
49" - 72"	20 feet
> 72"	10' + 2 x D

6.3.0 STORM SEWER HYDRAULICS

During heavy rains, water flowing in an underground storm sewer system will be under pressure. At inlets and manholes, the pressure is relieved and the water will rise to a level somewhat above the top of the pipe. The increase in water depth at these structures is known as surcharging, and the elevation to which the water rises is known as the hydraulic grade line elevation. By calculating the hydraulic grade line elevation at all points along a storm sewer system, the engineer is able to evaluate his selection of pipe sizes and limit surcharging to an allowable level.

The basic formulae used in the analysis of storm sewers are: (1) the continuity equation for calculation of the flow velocity, (2) Manning's equation for determining friction losses, and (3) the Bernoulli and Darcy-Weisbach equations for estimating minor losses. The form of the continuity equation used in storm sewer design is,

$$V = Q/A \tag{Eq. 6-1}$$

where,

- V = Average flow velocity in feet per second.
- Q = Flow rate in cubic feet per second.
- A = Cross-sectional area of flow in square feet.

The remaining equations are used in calculating energy loss and are discussed below.

6.4.0 ENERGY LOSSES

When designing storm sewer systems, good engineering practice requires that all the energy losses be taken into account. These losses are commonly referred to as head losses and may be classified as friction losses and minor losses. Friction losses are due to the forces between the water and the walls of the pipe, while minor losses are a result of turbulence created by changes in the flow direction and velocity. It should be pointed out that the term "minor loss" is a serious misnomer. In a typical storm sewer system the minor losses constitute a substantial percentage of the total energy loss and must be considered in the design. Minor losses increase as a function of the square of the mean velocity. Therefore, as the flow velocities increase, careful determination of these minor losses becomes increasingly important.

6.4.1 Friction Losses

The friction loss is a measure of the energy required to overcome the roughness of the pipe. In a straight length of conduit, flowing full, with constant cross section and roughness, the rate of loss from friction is constant and may be determined using a form of Manning's equation.

$$S_f = .453\ V^2 n^2 / R^{4/3} \tag{Eq. 6-2}$$

where,

- S_f = Friction slope in feet per foot.
- V = Average flow velocity in feet per second.
- n = Manning's roughness coefficient, (Table 6-1).
- R = Hydraulic radius in feet, (area/wetted perimeter).

The friction slope may be calculated directly by using Equation 6-2 or may be figured graphically using the Manning's equation nomograph, Figure 6-1.

The total friction loss in a length of pipe may be found by multiplying the friction slope (S_f) by the pipe length (L).

$$h_f = S_f\ L \tag{Eq. 6-3}$$

where,

h_f = Head loss due to friction in feet.
S_f = Friction slope in feet per foot, from Equation 6-2 or Figure 6-1.

6.4.2 Entrance Losses

Entrance losses result from the sudden contraction of the flow cross section when it enters a pipe from an inlet, manhole, or junction box. Entrance losses occur at every inlet, manhole, and junction box and shall be calculated using the following equation:

$$h_e = K_e\ V^2/(2g) \qquad \text{(Eq. 6-4)}$$

where,

h_e = Entrance loss in feet.
K_e = Entrance loss coefficient, (0.40).
V = Velocity of flow in downstream pipe in feet per second.
g = Acceleration due to gravity, (32.2 ft/sec^2).

6.4.3 Exit Losses

Exit losses result from the sudden enlargement as the flow exits the pipe section. Exit losses occur at every manhole, junction box, and outfall and shall be calculated using the equation,

$$h_x = K_x\ V^2/(2g) \qquad \text{(Eq. 6-5)}$$

where,

h_x = Exit loss in feet.
K_x = Exit loss coefficient, (Table 6-2).
V = Velocity of flow in upstream pipe in feet per second.
g = Acceleration due to gravity, (32.2 ft/sec^2).

The value for the exit loss coefficient (K_x) is a function of the change in diameter of the upstream and downstream pipes. Values for K_x are given in Table 6-2 for various diameter ratios.

6.4.4 Losses in Bends

The minor losses in a bend result from a disruption of the normal velocity distribution in the pipe section. The loss due to bends shall be calculated using the equation,

$$h_b = K_b\ V^2/(2g) \qquad \text{(Eq. 6-6)}$$

where,

h_b = Bend loss in feet.
K_b = Bend loss coefficient, (Table 6-3).
V = Velocity of flow in pipe in feet per second, (for manhole and junction boxes use the velocity in the upstream pipe).
g = Acceleration due to gravity, (32.2 ft/sec^2).

The value for the bend loss coefficient (K_b) varies with the radius and deflection angle of the bend. For manholes, junction boxes and wye connections, assume that the radius of curvature is equal to the diameter of the pipe, (i.e. R = D). Values for K_b are given in Table 6-3.

6.5.0 EROSION PROTECTION

One of the common areas for erosion in channels is around pipe outfalls. Erosion can occur in the channel bottom and on the opposite bank due to the turbulence caused by the outfall pipe. In addition, seepage may occur from around an improperly installed drainage pipe, causing erosion along the outside wall of the pipe. The best way to prevent pipe seepage is to: (1) insure that the pipe joints are tight and in good condition, (2) use a cement stabilized backfill near the pipe outfall, and (3) insure that the backfill is compacted properly around the pipe. Whenever possible, a pipe outfall should point down stream at approximately a 45 degree angle and in no case should the angle of intersection be greater than 90 degrees. The outfall pipe shall be mitered to match the side slope of the channel and not allowed to protrude into the ditch.

Channel protection or riprap shall be placed at pipe outfalls having a diameter of 36 inches or larger. The protection may be rock, concrete rubble, or concrete plating. However, rock or concrete rubble is preferable to smooth concrete because of its flexibility and higher roughness coefficient. The width of the riprap shall be equal to the inside pipe diameter plus three (3) feet. The riprap protection shall extend out from the end of the pipe a distance specified by the following equation:

$$L = 0.75\ DV \qquad \text{(Eq. 6-7)}$$

where,

L = Minimum length of erosion protection in feet.
D = Pipe diameter in feet.
V = Outlet velocity in feet per second.

For pipes discharging into a channel with a bottom width less than the minimum length calculated using Equation 6-7, the erosion protection shall extend up to the top of pipe elevation on the opposite bank.

6.6.0 DESIGN PROCEDURE

Figure A-2, Storm Sewer Design Work Sheet, included in the appendix, provides step by step format for calculating the energy losses and hydraulic grade line elevation for an underground storm sewer system. Calculations must begin at the upper end of the drainage system and proceed downstream to a point with a known hydraulic grade elevation. After completing Columns 1 through 23 in the Storm Sewer Design Work Sheet, the hydraulic grade elevation is calculated, from the most downstream point working upstream, by adding the total head losses, Column 23, to the hydraulic grade elevation, Column 24, of the point immediately downstream.

END OF SECTION

TABLE 6-1

MANNING'S ROUGHNESS COEFFICIENTS (n)
(For Use With Equation 6-2)

Pipe Material	Minimum Roughness Coefficient (n)
1. Concrete	
a. Pipe	0.012
b. Box	0.012
2. Corrugated Metal	
a. Plain and Asphalt Coated	
Annular 2.67 x .5 (all sizes)	0.024
Helical 2.67 x .5 (18" dia.)	0.014
Helical 2.67 x .5 (24" dia.)	0.016
Helical 2.67 x .5 (36" dia.)	0.019
Helical 2.67 x .5 (48" dia.)	0.020
Helical 2.67 x .5 (60" dia.)	0.021
Helical 2.67 x .5 (over 60")	0.021
Annular 3 x 1 (all sizes)	0.027
Helical 3 x 1 (48" dia.)	0.023
Helical 3 x 1 (54" dia.)	0.023
Helical 3 x 1 (60" dia.)	0.024
Helical 3 x 1 (66" dia.)	0.025
Helical 3 x 1 (72" dia.)	0.026
Helical 3 x 1 (over 72")	0.027
Corrugations 6 x 2 (60" dia.)	0.033
Corrugations 6 x 2 (72" dia.)	0.032
Corrugations 6 x 2 (120" dia.)	0.030
Corrugations 6 x 2 (180" dia.)	0.028
b. Fully Asphalt Lined	0.012
c. Fully Concrete Lined	0.012
3. Plastic	
a. Smooth	0.011
b. Corrugated (18" & 24" dia.)	0.020

1. Plastic pipe is not allowed within public owned right-of-way or easements.

TABLE 6-2

EXIT LOSS COEFFICIENTS (K_x)
(For Use With Equation 6-5)

D_2/D_1	K_x
1.0	0.0
1.2	0.10
1.4	0.25
1.6	0.38
1.8	0.48
2.0	0.56
2.5	0.70
3.0	0.78
4.0	0.87
5.0	0.91
10.0	0.96
> 10	1.00

1. D_2/D_1 is the diameter of the downstream pipe divided by the diameter of upstream pipe.
2. D_2 may not be smaller then D_1.
3. When using square, arch or elliptical pipe, replace the diameter ratio with $(A_2^{1/2})/(A_1^{1/2})$, where A_1 and A_2 are the cross-sectional area of the upstream and downstream pipes respectively.

TABLE 6-3

BEND LOSS COEFFICIENTS (K_b)
(For Use With Equation 6-6)

	Bend Loss Coefficient (K_b)		
Deflection Angle	R=D	R = 2D to 8D	R = 8D to 20D
0 deg	0.0	0.0	0.0
22.5	0.15	0.13	0.10
45	0.25	0.20	0.15
60	0.30	0.25	0.18
90	0.35	0.30	0.20

1. R is the radius of the bend in feet.
2. D is the diameter of the pipe in feet.
3. For bends in manholes and junction boxes, use R=D.
4. For bends with a radius large than 20 pipe diameters, $K_b = 0$.

TABLE 6-4

PROPERTIES OF CIRCULAR AND ARCH CONDUITS

Circular Conduits			Arch Conduits		
Diameter (in)	Area (sq.ft.)	Hyd. Radius (ft)	Span x Rise (in x in)	Area (sq.ft.)	Hyd. Radius (ft)
15	1.23	0.31	17 x 13	1.1	0.28
18	1.77	0.38	21 x 15	1.6	0.34
21	2.41	0.44	24 x 18	2.2	0.40
24	3.14	0.50	28 x 20	2.9	0.46
27	3.98	0.56	-------	---	----
30	4.91	0.63	35 x 24	4.5	0.57
33	5.94	0.69	-------	---	----
36	7.07	0.75	42 x 29	6.5	0.69
42	9.62	0.88	49 x 33	8.9	0.81
48	12.57	1.00	57 x 38	11.6	0.92
54	15.90	1.13	64 x 43	14.7	1.04
60	19.64	1.25	71 x 47	18.1	1.15
66	23.76	1.38	77 x 52	21.9	1.27
72	28.27	1.50	83 x 57	26.0	1.38
78	33.18	1.63	86 x 67	32.1	1.57
84	38.49	1.75	93 x 72	37.0	1.68
90	44.18	1.88	101 x 76	42.4	1.80
96	50.27	2.00	108 x 80	48.0	1.91
102	56.75	2.13	116 x 85	54.2	2.03
108	63.62	2.25	123 x 89	60.5	2.14
114	70.88	2.38	131 x 94	67.4	2.26
120	78.54	2.50	138 x 98	74.5	2.37

Note: Dimensions shown for arch conduits may vary with corrugation size and manufacturer.

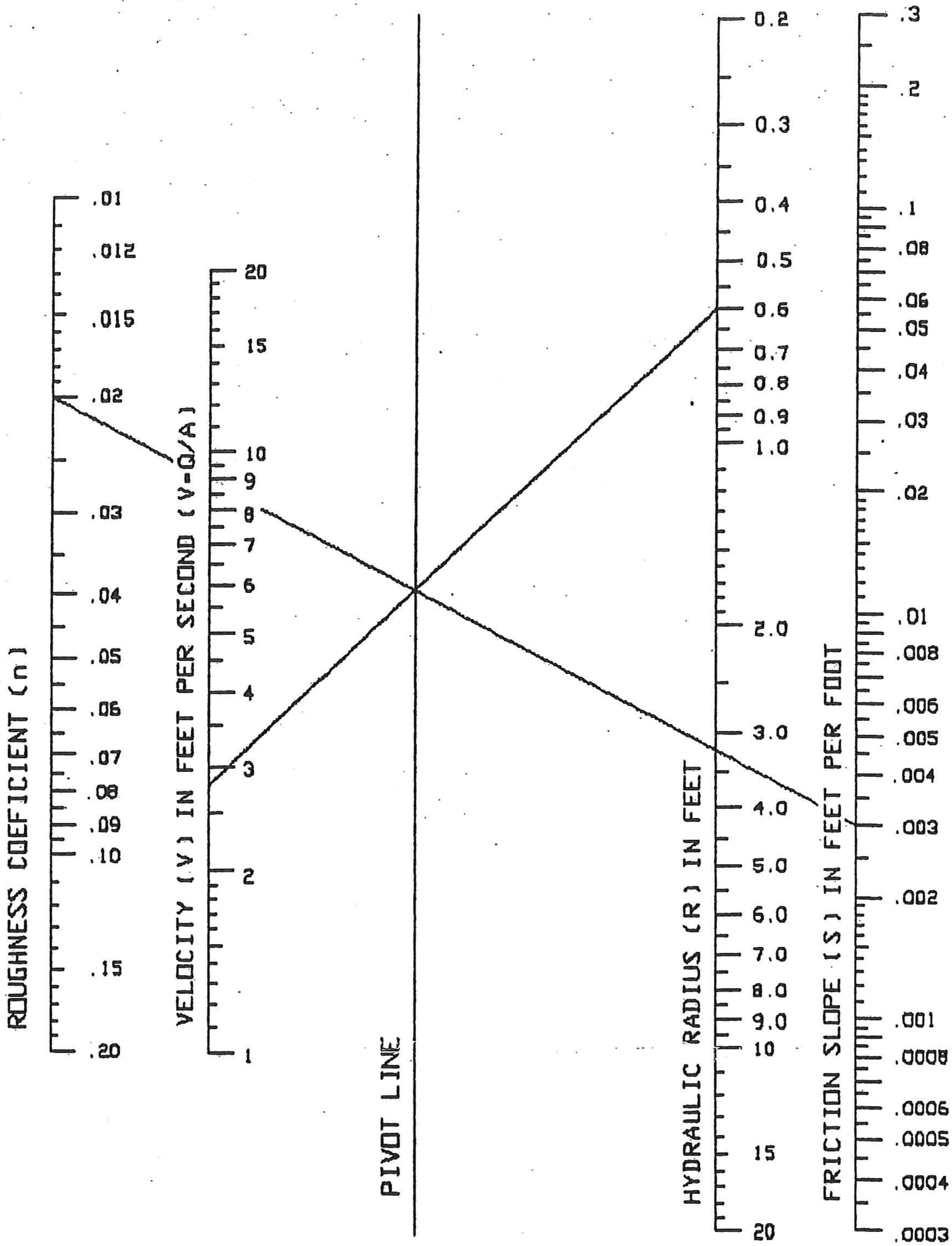
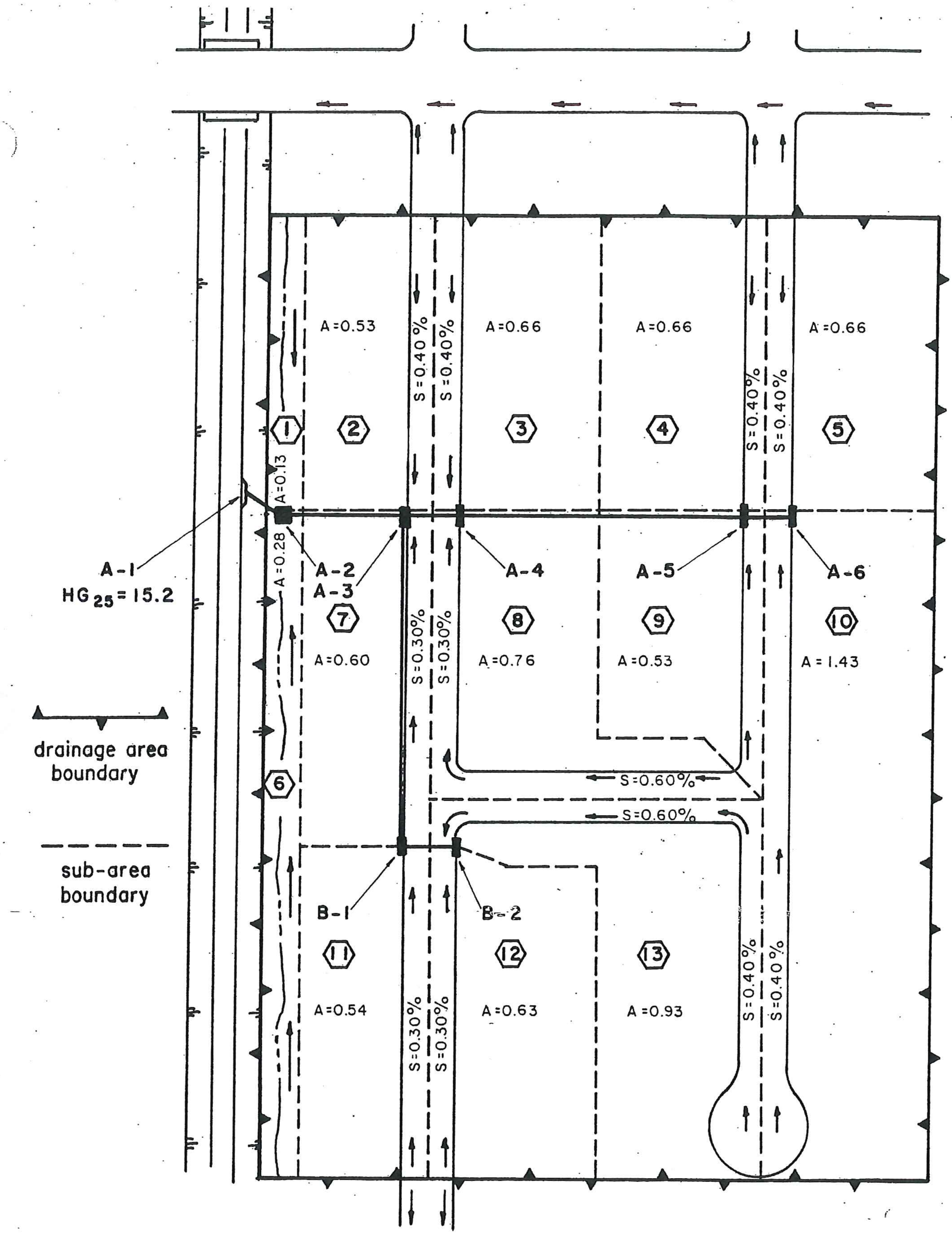


FIGURE 6-1
SOLUTION TO MANNING'S EQUATION
FOR ANY CROSS SECTION



EXAMPLE 6-1
STORM SEWER DESIGN

PROJECT NAME EXAMPLE 6-1

STORM SEWER LINE "A"

DESIGN FREQUENCY 5 year

SHEET 1 OF 2

DESIGN POINT	DETERMINATION OF RUNOFF: DRAINAGE AREA: SUB-AREAS	DRAINAGE AREA: TOTAL AREA	TIME OF CONCENTRATION: TOTAL TIME	TIME OF CONCENTRATION: CONDUIT TIME	RUNOFF COEF. (C)	RAINFALL INTENSITY (I)	RUNOFF/ ACRE (CI)	TOTAL RUNOFF (Q)	CONDUIT DESIGN: DIST. TO NEXT PNT. (L)	PIPE SIZE	X-SECT AREA (Ap)	CONDUIT SLOPE (S)	CONDUIT ENTER: CROWN	CONDUIT ENTER: INVERT	CONDUIT EXIT: CROWN	CONDUIT EXIT: INVERT	HYDRAULIC DESIGN: FLOW VELOCITY (V)	ENERGY LOSSES: FRICTION	ENERGY LOSSES: ENTRANCE	ENERGY LOSSES: EXIT	ENERGY LOSSES: BEND	ENERGY LOSSES: TOTAL	HYDRAULIC GRADE LINE ELEVATION	COMMENTS
---	------	(acres)	(minutes)	(minutes)	----	(in/hr)	(cfs/ac)	(cfs)	(ft)	(in)	(ft²)	(%)	(ft)	(ft)	(ft)	(ft)	(ft/sec)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25
A-6	5,10	2.09	25		0.48	4.86	2.33	4.87					--	--	14.68	13.18							15.97	
				0.2					36	18	1.77	0.28					2.75	0.06	0.05	0.03	0	0.14		
A-5	4,5,9,10	3.28	25.2		0.48	4.84	2.32	7.61					14.58	13.08	14.58	12.58							15.83	
				1.8					260	24	3.14	0.10					2.42	0.25	0.04	0.01	0	0.30		
A-4	3-5 8-10	4.70	27.0		0.48	4.65	2.23	10.48					14.32	12.32	14.32	11.82							15.53	
				0.3					36	30	4.91	0.28					2.13	0.02	0.03	0.01	0	0.06		
A-3	2-5 7-13	7.93	27.3		0.48	4.62	2.22	17.60					14.22	11.72	14.22	11.22							15.47	
				0.8					120	36	7.07	0.10					2.49	0.07	0.04	0	0	0.11		
A-2	1-13	8.34	28.1		0.48	4.54	2.18	18.18					14.10	11.10	14.10	11.10							15.36	
									30	36	7.07	0.33					2.57	0.02	0.04	0.10	0	0.16		
A-1													14.0	11.0	--	--							15.2	Known 25 Year HG

1. Design point designation.
   (Note: Begin calculations at the point farthest from the draingage outfall.)
2. Drainage sub-area designation for all areas contributing flow to the design point.
3. Total drainage area contributing flow to the design point.
4. Time of concentration to the design point, column 4 plus column 5. (Section 3.4.2.A).
5. Time along conduit, column 10 divided by column 10. (Section 3.4.2.A(2)).
6. Runoff coefficient. (Section 3.4.4 and Table 3-1).
7. Rainfall intensity. (Section 3.4.2.B).
8. Runoff per acre, column 6 times column 7 or Table 3-2 and 3-3.
9. Total runoff, column 3 times column 8.
10. Distance to next downstream design point.
11. Selected pipe size.
12. Cross-sectional area of pipe. (Table 6-4).
13. Conduit slope. (Note: Minimum slope is 0.10 %.)
14. Crown elevation of conduit entering the design point.
15. Invert elevation of conduit entering the design point.
16. Crown elevation of conduit exiting the design point.
17. Invert elevation of conduit exiting the design point.
18. Flow velocity in conduit, column 9 divided by column 12.
    (Note: Flow velocity must be maintained between 2 and 10 ft/s.)
19. Friction loss. (Section 6.4.1).
20. Entrance loss. (Section 6.4.2).
21. Exit loss. (Section 6.4.3).
22. Bend loss. (Section 6.4.4).
23. Total head loss, sum of columns 19, 20, 21, and 22.
24. Hydraulic grade line elevation, column 24 plus column 23.
    (Note: The hydraulic grade line elevations are calculated beginning at the down stream point having a known, or previously calculated water surface elevation.)

EXAMPLE 6-1
STORM SEWER DESIGN

PROJECT NAME EXAMPLE 6-1

STORM SEWER LINE "B"

DESIGN FREQUENCY 5 year

SHEET 2 OF 2

DESIGN POINT	DETERMINATION OF RUNOFF: DRAINAGE AREA: SUB-AREAS	DRAINAGE AREA: TOTAL AREA	TIME OF CONCENTRATION: TOTAL TIME	TIME OF CONCENTRATION: CONDUIT TIME	RUNOFF COEF. (C)	RAINFALL INTENSITY (I)	RUNOFF/ ACRE (CI)	TOTAL RUNOFF (Q)	CONDUIT DESIGN: DIST. TO NEXT PNT. (L)	PIPE SIZE	X-SECT AREA (Ap)	CONDUIT SLOPE (S)	CONDUIT ENTER: CROWN	CONDUIT ENTER: INVERT	CONDUIT EXIT: CROWN	CONDUIT EXIT: INVERT	HYDRAULIC DESIGN: FLOW VELOCITY (V)	ENERGY LOSSES: FRICTION	ENERGY LOSSES: ENTRANCE	ENERGY LOSSES: EXIT	ENERGY LOSSES: BEND	ENERGY LOSSES: TOTAL	HYDRAULIC GRADE LINE ELEVATION	COMMENTS
---	------	(acres)	(minutes)	(minutes)	----	(in/hr)	(cfs/ac)	(cfs)	(ft)	(in)	(ft²)	(%)	(ft)	(ft)	(ft)	(ft)	(ft/sec)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25
B-2	12,13	1.56	25		0.48	4.86	2.33	3.63					--	--	15.22	13.72							16.28	
				0.3					36	18	1.77	0.28					2.05	0.04	0.03	0.02	0.02	0.11		
B-1	11-13	2.10	25.3		0.48	4.83	2.32	4.87					15.12	13.62	15.12	13.62							16.17	
									300	18	1.77	0.30					2.75	0.54	0.05	0.07	0.04	0.70		
A-3													14.22	12.72	14.22	11.22							15.47	5 year HG from Sheet 1 of 1

1. Design point designation.
   (Note: Begin calculations at the point farthest from the draingage outfall.)
2. Drainage sub-area designation for all areas contributing flow to the design point.
3. Total drainage area contributing flow to the design point.
4. Time of concentration to the design point, column 4 plus column 5. (Section 3.4.2.A).
5. Time along conduit, column 10 divided by column 10. (Section 3.4.2.A(2)).
6. Runoff coefficient. (Section 3.4.1 and Table 3-1).
7. Rainfall intensity. (Section 3.4.2.B).
8. Runoff per acre, column 6 times column 7 or Table 3-2 and 3-3.
9. Total runoff, column 3 times column 8.
10. Distance to next downstream design point.
11. Selected pipe size.
12. Cross-sectional area of pipe. (Table 6-4).
13. Conduit slope. (Note: Minimum slope is 0.10 %.)
14. Crown elevation of conduit entering the design point.
15. Invert elevation of conduit entering the design point.
16. Crown elevation of conduit exiting the design point.
17. Invert elevation of conduit exiting the design point.
18. Flow velocity in conduit, column 9 divided by column 12.
    (Note: Flow velocity must be maintained between 2 and 10 ft/s.)
15. Friction loss. (Section 6.4.1).
20. Entrance loss. (Section 6.4.2).
21. Exit loss. (Section 6.4.3).
22. Bend loss. (Section 6.4.4).
23. Total head loss, sum of columns 19, 20, 21, and 22.
24. Hydraulic grade line elevation, column 24 plus column 23.
    (Note: The hydraulic grade line elevations are calculated beginning at the down stream point having a known, or previously calculated water surface elevation.)

EXAMPLE 6-1
STORM SEWER DESIGN

SECTION 7
OPEN CHANNELS

CONTENTS

SECTION 7 - OPEN CHANNELS

7.1.0 GENERAL

Open channels have some distinct advantages to underground storm sewer. Channels usually provide more carrying capacity for less initial cost, are less expensive to expand, and provide a larger degree of detention storage than an underground storm sewer system. Some disadvantages to open channels are a higher maintenance cost, and larger right-of-way requirements. The engineer should carefully weigh the advantages and disadvantages for each installation.

7.2.0 OPEN CHANNEL HYDRAULICS

Open channel flow is classified according to the change in flow depth with respect to time and location. Flow in an open channel is said to be "uniform" if the depth of flow remains the same along the entire length being considered. If the depth of flow changes along the length of the channel the flow is said to be "varied". Varied flow may be further classified as "gradually varied" and "rapidly varied". The flow is rapidly varied if the changes in depth occur abruptly over a comparatively short distance, and gradually varied if the changes in depth occur over a comparatively long distance.

7.3.0 UNIFORM FLOW

When a section of channel is sufficiently long and unchanging such that the forces of gravity are balanced with the channel's resistance, the flow is said to be uniform. Under such circumstances, the depth of flow and flow velocity remain constant throughout the channel reach.

The most widely used open channel formula is Manning's equation:

$$V = (1.486/n)\ R^{2/3}\ S^{1/2} \qquad \text{(Eq. 7-1)}$$

where,

V = Average flow velocity in feet per second.
n = Manning's roughness coefficient.
R = Hydraulic radius in feet, (area/wetted perimeter).
S = Slope of the friction gradient in feet per foot.
(same as the channel bottom slope for true uniform flow).

Equation 7-1 is applicable when the channel slope (S) is less than about 10 percent, (i.e. 10 feet of fall per 100 feet). Under such conditions the sloped length of the channel does not differ significantly from the horizontal length.

Figure 7-1 at the end of this Section may be used to solve Manning's equation for slope, hydraulic radius, velocity or roughness coefficient when any three (3) parameters are known.

7.3.1 Normal Depth

For any given channel condition, roughness coefficient, discharge, and slope, there is only one (1) possible depth for maintaining a uniform flow. This depth is referred to as the normal depth and is determined using Manning's equation for discharge:

$$Q = (1.486/n)\ A\ R^{2/3}\ S^{1/2} \qquad \text{(Eq. 7-2)}$$

where,

- Q = Total discharge in cubic feet per second.
- n = Manning's roughness coefficient.
- A = Cross-sectional area of flow in square feet.
- R = Hydraulic radius in feet, (area/wetted perimeter).
- S = Slope of the friction gradient in feet per foot. (same as the channel bottom slope for true uniform flow)

The normal depth for any type cross section may be calculated by substituting expressions involving depth, for A and R in Equation 7-2 and solving by trial and error.

Because of small variations in cross section, slope, and roughness, true uniform flow is more often a theoretical abstraction than an actuality. The engineer must realize that uniform depth computations provide only an approximation of what will occur but that such computations are often adequate and useful.

7.3.2 Critical Depth

The critical depth for flow in an open channel is defined as the depth for which the specific energy (sum of the depth and velocity head) is a minimum. It may be shown mathematically that critical depth occurs when:

$$Q^2/g = A^3/T \qquad \text{(Eq. 7-3)}$$

where,

- Q = Discharge in cubic feet per second.
- A = Cross-sectional area of flow in square feet.
- T = Top width of flow in feet.
- g = Acceleration due to gravity (32.2 ft/sec^2).

Critical depth for any shaped section may be computed by substituting expressions involving depth, for A and T in Equation 7-3 and solving by trial and error.

The equations used to solve for critical depth in some commonly encountered channel sections are:

Rectangular channel,

$$Q = 5.675\ B\ D_c^{3/2} \qquad \text{(Eq. 7-4)}$$

Triangular channel,

$$Q = 4.01\ Z\ D_c^{5/2} \qquad \text{(Eq. 7-5)}$$

Trapezoidal channel,

$$Q = 5.765\ D_c^{3/2}\ ((B+Z\ D_c)^3/(B+2\ Z\ D_c))^{1/2} \qquad \text{(Eq. 7-6)}$$

where,

Q = Discharge in cubic feet per second.
B = Bottom width of the channel in feet.
Dc = Critical depth in feet.
Z = Side slope, (horizontal/vertical).

7.3.3 Critical Slope

Critical slope is that slope which will sustain a given discharge in a given channel section at uniform, critical depth. A channel having a constant cross section, roughness coefficient, and discharge, will carry that discharge at uniform, critical depth if the channel slope is equal to the critical slope. If the channel slope is less than critical, the depth will be greater than critical depth and the flow is said to be subcritical. If the channel slope is greater than critical, the depth will be less than critical depth, and the flow is said to be supercritical. The general equation for critical slope of an open channel is given by:

$$Sc = 14.56\ n^2\ Dm/R^{4/3} \qquad \text{(Eq. 7-7)}$$

where,

Sc = Critical slope in feet per foot.
n = Manning's roughness coefficient.
Dm = Mean depth of flow in feet, (A/T).
R = Hydraulic radius, (A/P).

7.3.4 Significance of Critical Flow

Uniform flow at or near critical depth is unstable. This results from the fact that the relationship between energy head and depth of flow is easily disturbed by minor changes in energy. Variations in the channel roughness, cross section, slope, or minor deposits of sediment may cause large fluctuations in the flow depth that are significant to channel operation. For any design utilizing a slope near the critical slope, it is recommended that the design computations be based on a various flow rates and 'n' values in order to establish the probable range of operating conditions. Because of the unstable flow, channels carrying uniform flow at or near critical depth should not be used.

7.4.0 GRADUALLY VARIED FLOW

Uniform flow rarely occurs in natural streams because of changes in depth, width, and slope of the channel. Although the simplest design for constructing a channel is a uniform cross-section, constant slope and no obstructions, this is not always feasible because of topographic conditions. Therefore, the engineer is often concerned with nonuniform flow in open channels. A frequently utilized method of solving problems of gradually varied flow is the standard step backwater analysis. The standard step method may be performed using a hand calculator, or for more complex situations, the U. S. Army Corps of Engineers' program, HEC-2.

7.4.1 Standard Step Backwater Analysis

The standard step method for backwater analysis is an iterative process in which the Bernoulli energy equation is solved to find the water profile of a channel. The channel is divided into segments, and the water surface profile is determined by trial and error for each successive segment. The procedure outlined below is applicable for analyzing new or existing channels with no overbank flow.

STEP 1 Determine Channel Cross-sections

Cross-sections should be determined by field survey or topographic maps and spaced such that the channel configuration between sections is largely uniform. In areas where the channel properties change rapidly, the distance between sections should decrease. Cross-sections should be perpendicular to the flow in the channel during flooding conditions.

STEP 2 Determine Manning's Roughness Coefficients

Determine the Manning's roughness coefficient for each cross-section using the methods described in Sections 7.6.1 and 7.6.2. If a single Manning's 'n' value is not appropriate for describing the entire cross-section, the section may be divided into several subsections. However, the mathematical calculations involving subsections are more complex, and it is recommended that they be performed with the aid of a computer program.

NOTE: For the remainder of the instructions it will be necessary to distinguish between the many sections used in the calculations. A standard step backwater analysis is performed on only one channel segment at a time, making use of only two consecutive cross-sections per iteration. The cross-section on the downstream end of each segment shall be referred to with the subscript (i), and the upstream cross-section shall designated as (i+1). After the calculations for a segment are complete, cross-section (i+1) would become cross-section (i) for the next iteration.

STEP 3 Determine Water Surface Elevation for Cross-section (i)

The accuracy of a water surface profile, in the vicinity of the first several cross-sections, is largely dependent on an accurate determination of the starting water surface. If a starting water surface is not available from a previous hydraulic study, the beginning water surface may be estimated assuming normal flow and Manning's equation. A more accurate profile may be determined by calculating backwater profiles based on several arbitrary starting elevations. After plotting each of the profiles, it will be found that they will tend to converge to a common curve at some point upstream. This is because each successive calculation brings the profile nearer to the actual uniform depth. The backwater analysis should begin far enough downstream so that the point of convergence is downstream of the area of interest.

STEP 4 Assume Water Surface Elevation for Cross-section (i+1)

The next step is to assume a trial water surface elevation for the upstream cross-section. A reasonable first assumption may be made by maintaining the same cross-sectional area or depth of flow as the downstream cross section.

STEP 5 Calculate Area, Wetted Perimeter, Hydraulic Radius, and Velocity

A. Calculate the velocity of flow (V_i) for cross-section (i). If this is the first section in a backwater analysis, the velocity may be found by dividing the flow rate (Q) by the total area of flow (A) for the section. Otherwise, the flow velocity in section (i), is the same as the flow velocity previously computed for that section.

B. Calculate the area of flow (A), wetted perimeter (P), hydraulic radius (R), and flow velocity (V), for cross-section (i+1). The hydraulic radius is equal to the area divided by the wetted perimeter. The flow velocity is equal to the flow rate divided by the total cross-sectional area.

STEP 6 Calculate Friction Losses

The losses due to friction are calculated as the length of channel between sections multiplied by the average friction slope of the upstream and downstream cross-section. The friction slope for each cross-section is calculated using Manning's equation as follows:

$$h_f = S_{f(avg)}\ L \qquad \text{(Eq. 7-8)}$$

where,

h_f = head losses due to friction between section (i) and section (i+1) in feet.

L = The flow distance between sections (i) and (i+1) in feet.

$S_{f(avg)} = (S_{f(i)}+S_{f(i+1)})/2$

$S_{f(i+1)} = 0.453\ Q^2\ n^2/(A^2\ R^{4/3})$

Q = Total flow in cubic feet per second.
n = Manning's roughness coefficient.
A = Area of flow for cross-section (i+1) in square feet.
R = Hydraulic radius of cross-section (i+1) in feet.

STEP 7 Calculate Velocity Head Losses

The velocity head loss is estimated as the difference in velocity head between the downstream and the upstream section. The change in velocity head may be positive or negative.

$$h_v = V_1^2/(2g)-V_2^2/(2g) \qquad \text{(Eq. 7-9)}$$

where,

h_v = Change in velocity head between sections in feet.
V_1 = Average velocity at cross-section (i).
V_2 = Average velocity at cross-section (i+1)
g = Acceleration due to gravity. (32.2 ft/sec^2)

STEP 8 Calculate Eddy Losses

The eddy losses between sections are losses due to turbulence and are estimated based on the velocity head (h_v) determined previously. If the velocity head is positive, the eddy losses (h_e) are equal to ten (10) percent of the velocity head. If the velocity head is negative, then the eddy losses are estimated as fifty (50) percent of the velocity head.

$$h_e = 0.10\ h_v \quad (\text{for } h_v > 0) \qquad \text{(Eq. 7-10)}$$

$$h_e = 0.50\ h_v \quad (\text{for } h_v < 0) \qquad \text{(Eq. 7-11)}$$

where,

h_e = Eddy loss in feet between cross-sections (i) and (i+1).
h_v = Velocity head loss from Equation 7-9.

STEP 9 Calculate Water Surface Elevation for Section (i+1)

The calculated water surface elevation for section (i+1) is equal to the water surface elevation of section (i) plus the total head loss between sections.

$$WS_{calc} = WS_{section\ (i)} + h_f + h_v + h_e \qquad \text{(Eq. 7-12)}$$

where,

WS_{calc} = The calculated water surface elevation for section (i+1).
$WS_{section\ (i)}$ = The water surface elevation for section (i).
h_f = Friction head loss from Equation 7-8.
h_v = Velocity head loss from Equation 7-9.
h_e = Eddy head loss from Equation 7-10 or 7-11.

STEP 10 Compare Assumed and Calculated Water Surface Elevations

A. Compare the water surface elevation calculated in Step 9 with the water surface assumed in Step 3. If the accuracy is not acceptable, (difference greater than 0.1'), then return to Step 3 and assume a new water surface elevation.

B. If the difference between the calculated and the assumed water surface elevations is less than 0.1' then go to Step 4 and begin calculations for the next channel segment.

7.4.2 HEC-2 Water Surface Profiles

The HEC-2 computer program was developed by the Hydrologic Engineering Center of the U. S. Army Corps of Engineers in the early 1970's under the direction of Bill S. Eichert. HEC-2 is the most widely used program for computing water surface elevations through a channel reach during a particular storm event. The program is based on the standard step method and Manning's equation for open channel flow, with special routines capable of analyzing bridges, culverts, and other structures.

The HEC-2 program should be used instead of hand calculation when: (1) the water surface exceeds the channel banks, (2) there are several changes in the channel section due to bridges, culverts, transitions, or drop structures, and (3) analyzing non-uniform channels under existing and proposed conditions.

The Corps of Engineers will no longer sell or support computer programs for the private sector. The HEC is **not** discontinuing the HEC-2 program, but has turned over the responsibility of sales and support to private companies.

7.5.0 RAPIDLY VARIED FLOW

Rapidly varied flow is characterized by an abrupt change in the water surface elevation. The change in elevation may become so abrupt that the flow profile is virtually broken, resulting in high turbulence. The most commonly encountered examples of rapidly varied flow would be the hydraulic jump, flow over a weir and flow over a free outfall.

7.5.1 Hydraulic Jump

When water flowing at greater than critical velocity enters water flowing at less than critical velocity and sufficient depth, a hydraulic jump develops. In the jump, the depth increases from an original supercritical depth to a subcritical depth. The engineer must be able to anticipate when there is a potential for a hydraulic jump and provide the necessary erosion protection.

7.5.2 Weir Flow

The equation for discharge over a weir cannot be derived precisely because, not only does the flow pattern of one weir differ from that of another weir, but the flow pattern for a given weir varies with the discharge. However, empirical equations have been derived, through experimentation, to calculate the flow over a weir. A commonly used formula for free discharge over a rectangular weir is,

$$Q = CL(H+V^2/(2g))^{3/2} \qquad \text{(Eq. 7-13)}$$

where,

- Q = Discharge over the weir in cubic feet per second.
- C = Empirical coefficient of discharge. (The recommended value of C for a broad crested weir is 3.0).
- L = Length of the weir in feet.
- H = Head on the weir in feet.
- V = Approach velocity, in feet per second, measured a distance 3H upstream of weir.
- g = Acceleration due to gravity, (32.2 ft/sec^2).

By neglecting the approach velocity, which is normally small, the equation for weir flow may be simplified to,

$$Q = CLH^{3/2} \qquad \text{(Eq. 7-14)}$$

where,

- Q = Discharge over the weir in cubic feet per second.
- C = Empirical coefficient of discharge. (The recommended value of C for a broad crested weir is 3.0).
- L = Length of the weir in feet.
- H = Head on the weir in feet.

Submerged discharge is the term used to describe flow over a weir when the downstream tailwater elevation is above the crest elevation of the weir. The experimental data available for submerged flow indicates a wide range in the effect of submergence. Therefore, precise results should not be expected from submergence computations. The flow over a submerged weir may be estimated by the equation,

$$Q = C_s CLH_1^{3/2} \qquad \text{(Eq. 7-15)}$$

where,

- Q = Discharge over the weir in cubic feet per second.
- Cs = Coefficient of submergence, (Figure 7-2).
- C = Empirical coefficient of discharge. (The recommended value of C for a broad crested weir is 3.0)
- L = Length of the weir in feet.
- H_1 = Head on the weir in feet.

The value of C_s may be obtained from the graph, Figure 7-2 base on the ratio of H_2/H_1, where H_1 equals the upstream head on the weir, neglecting the approach velocity, and H_2 equals the difference in elevation between the tailwater and the weir crest.

7.5.3 Free Outfall

Free outfall is the term used to describe unrestricted flow over an abrupt drop in a channel, (i.e. a waterfall). If the flow in the channel is subcritical, (depth is greater than critical depth), the flow velocity will increase as the water approaches the outfall. Theoretically the flow would pass through critical depth at the brink of the free outfall. However, curvature of the streamlines in the vicinity of the outfall alters the flow conditions and results in a depth at the brink considerably less than critical depth. The depth at the brink of a free outfall has been observed experimentally to be about 71 percent of the critical depth, ($D = 0.71\ d_c$). Critical depth actually occurs at a distance equal to approximately four (4) times critical depth upstream from the brink. Such occurrences of critical depth are very useful for backwater profiles because they provide a starting point for calculations. Often times this calculation may be used to determine if a check dam is necessary to reduce the high flow velocities caused by the draw down effects which occur upstream of the drop.

7.6.0 MANNING'S ROUGHNESS COEFFICIENT

Manning's equation is an empirical formula. The roughness coefficient 'n' is used to quantitatively express the degree of retardation of flow. The value of 'n' indicates not only the roughness of the sides and bottom of the channel, but also all other types of irregularities on the channel section and profile.

7.6.1 Roughness Coefficients for Existing Channels

Over the years there has been a tendency to regard the selection of a Manning's roughness coefficient as an arbitrary or an intuitive process. However, a more consistent approach is to evaluate the primary factors which affect 'n'. These factors are: (1) irregularity of the channel sides and bottom, (2) variations in shape and size of the cross sections, (3) obstructions, (4) vegetation, and (5) meandering of the channel.

The general procedure for estimating a roughness coefficient involves the selection of a basic value of 'n' for a straight, uniform, smooth channel, then, through careful consideration of the factors listed above, the selection of modifying values associated with each factor. The modifying factors are added to the basic value to obtain the roughness coefficient for the channel under consideration. The basic formula for estimating Manning's roughness coefficient for existing or natural channels is,

$$n = (n_0 + n_1 + n_2 + n_3 + n_4)m \qquad \text{(Eq. 7-16)}$$

where,

n = Manning's roughness coefficient.
n_0 = Basic 'n' for a straight, uniform, smooth channel.
n_1 = Modifying factor for channel irregularity.
n_2 = Modifying factor for cross section variation.
n_3 = Modifying factor for obstructions.
n_4 = Modifying factor for vegetation.
m = Modifying factor for meandering.

Proper values for n_0 to n_4 and m are listed in Table 7-1 at the end of this section and a discussion on the selection of each modifying factor follows.

A. Basic Roughness Coefficient (n_0)

The basic roughness coefficient is selected assuming a hypothetical channel which is straight, uniform, smooth and without vegetation. The selection is made based on the material forming the channel, either earth, rock, or gravel.

B. Modifying Factor for Channel Irregularity (n_1)

The selection is based on the degree of roughness or irregularity of the surfaces of channel sides and bottom. The actual surface irregularity is compared to the best surface to be expected for the natural material involved, and is classified as smooth, minor, moderate, or severe.

C. Modifying Factor for Cross Section Variation (n_2)

When selecting n_2, consider the changes in size of the cross section and determine the rate at which the changes take place. Changes of considerable magnitude, if they are gradual and uniform, do not cause significant turbulence. The greatest turbulence is associated with abrupt changes in cross section. Consider the degree to which the changes cause the greatest depth of flow to move from side to side of the channel.

D. Modifying Factor for Obstructions (n_3)

The modifying factor for obstructions are selected based on the presence and characteristics of obstructions such as debris deposits, stumps, exposed roots, and fallen logs. Care should be taken that conditions considered in other steps are not reevaluated or double-counted by this step. In judging the relative effect of obstructions, consider: (1) the degree to which the obstructions occupy or reduce the average cross-section area, (2) the shape of the obstruction (sharp-edged objects induce greater turbulence than curved objects), and (3) the position and spacing of obstructions both transversely and longitudinally.

E. Modifying Factor for Vegetation (n_4)

The retarding effect of vegetation is due primarily to the turbulence induced as the water flows around the limbs, stems and foliage, and secondarily to the reduction in cross-sectional area. In judging the retardation effect of vegetation the engineer should consider the following: (1) the height of the vegetation in relation to the depth of flow, (2) the vegetations resistance to bending, (3) the degree to which the cross-sectional area is reduced, (4) and the longitudinal and transverse distribution of vegetation. When selecting the value for n_4, the effect of vegetation is classified as follows:

1. Low for conditions comparable to the following: (a) dense growths of flexible grasses or weeds, where the average depth of flow is two to three times the height of the vegetation, and (b) supple seedling trees where the average depth of flow is three to four times the height of the vegetation.

2. Medium for conditions comparable to the following: (a) grass where the average depth of flow is one to two times the height of the grass, (b) stemmy grasses, weeds of seedlings with moderate cover where the depth of flow is two to three times the height of vegetation, and (c) moderately dense brush along the side slopes of a channel with no significant vegetation along the channel bottom.

3. High for conditions comparable to the following: (a) grass where the average depth of flow is about equal to the height of the grass, (b) bushy willows inter-grown with some weeds along the side slopes of a channel with little growth along the channel bottom.

4. Very High for conditions comparable to the following: (a) grass where the average depth of flow is less than one half the height of the vegetation, (b) trees inter-grown with weeds, and (c) dense growth of cattails or "rose-hedge" along the channel bottom.

F. Modifying Factor for Meandering (m)

The modifying factor for meandering of the channel depends on the ratio of the meander length to the straight length of the channel reach. The meandering is considered minor for rations of 1.0 to 1.2, appreciable for ratios 1.2 to 1.5, and severe for ratios of 1.5 or greater.

In applying the above method for determining the 'n' value, it should be noted that: (1) The values given in Table 7-1 were developed from a study of 40 to 50 cases of small to moderate channels, and the method is questionable when applied to large channels with a hydraulic radius in excess of 15 feet, and (2) the method applies only to unlined natural streams, floodways and drainage channels.

7.6.2 Roughness Coefficients for New Channels

The recommended Manning's roughness coefficients for use in designing new channels are listed in Table 7-2. Values have been selected which will account for the anticipated degree of vegetative growth and erosion.

7.7.0 ENERGY LOSSES

Manning's equation is used to estimate the energy losses due to frictional forces in a channel. Other sources of losses in open channels occur at confluences, transitions, bends, culverts, and bridges. When computing water surface profiles, either by hand or with the help of a computer program, the engineer must include the significant sources of energy loss. When using a backwater computer program, the engineer must determine whether or not the program accounts for nonfrictional losses, and if not, must include such losses in the calculations by hand.

7.7.1 Confluences

The alignment of confluences is critical in reducing erosion and energy losses caused by turbulence and eddies. The variables to be considered when analyzing channel junctions are: (1) the angle of intersection, (2) the shape and dimensions of the channels, (3) flow rates, and (4) flow velocities. The intersecting angle between a side channel and a main channel should be as small as possible. For an angle of intersection of less than 90 degrees, energy losses are generally negligible. Intersection angles greater than 90 degrees cause severe turbulence and should not be used.

7.7.2 Bends

Channel bends or curves should be as gradual as possible to reduce erosion and turbulence. Channel bends with a radius of curvature measured from the channel centerline of more than three (3) times the top width of the channel, do not produce significant energy losses. Bends with a radius of curvature less than three (3) times the channel top width will cause increased head losses and should be avoided.

7.7.3 Expansion and Contraction Losses

Transitions in channels should be designed to create a minimum of flow disturbance and thus a minimum energy loss. Transitions generally occur at culverts, and where cross sections change due to increased flow or a change in right-of-way. The transition may consist of a change in cross-sectional area or cross section geometry. The angle of transition for newly designed channels shall be less than 12 degrees, (i.e. 20 feet per 100 feet).

Expansion and contraction losses must be accounted for in backwater calculations. Losses at transitions are generally expressed in terms of the absolute change in velocity head between sections downstream and upstream of the transition, and may be estimated using the following equation:

$$h_t = C(V_2^2 - V_1^2)/(2g) \qquad \text{(Eq. 7-17)}$$

where,

h_t = Head loss, in feet, due to expansion or contraction.
C = Empirical expansion or contraction coefficient.
V_1 = Average channel velocity of the downstream section in feet per second.
V_2 = Average channel velocity of the upstream section in feet per second.
g = Acceleration due to gravity, (32.2 ft/sec^2).

Typical transition loss coefficients for subcritical flow are as follows:

Transition Type	Contraction	Expansion
Gradual or Warped	0.1	0.3
Bridge sections, wedge, or straight-lined	0.3	0.5
Abrupt or square edged	0.6	0.8

When computing the backwater profile through a transition, engineering judgment must be used in selecting the reach length. As a general guide line, the velocity between any two cross sections should not differ by more than 50 percent.

7.7.4 Losses Through Culverts and Bridges

Methods for estimating the energy losses through culverts and bridges is covered in Section 8 of this manual.

7.8.0 EROSION PROTECTION AND VELOCITY CONTROL STRUCTURES

Channel erosion is generally caused by: (1) excessive velocities in the channel, (2) flow over the banks of the channel, (3) turbulent flow at confluences or bends, and (4) high flow velocities entering the channel from a pipe outfall or culvert. Each of these sources of erosion can be minimized if proper measures are included in the design and construction of the channel and its appurtenances. Adequate grass cover or structural lining in the channel can often alleviate problems due to excessive velocity and secondary flow. The minimum requirements for lined and unlined channels are shown in Figures 7-4, 7-5, and 7-6.

7.8.1 Channel Lining

Channel linings are often used to: (1) increase the flow capacity, (2) to reduce easement requirements, (3) to prevent erosion, and (4) to reduce maintenance problems. The primary problems associated with channel linings are high initial cost and often low aesthetic value. Various materials may be used for lining channels including, reinforced concrete, rock or concrete rubble, gabions, or fabriform. If a short section of lining is to be used for erosion control, the rock or concrete rubble is preferable to smooth concrete due to its higher roughness coefficient. However, when reinforced concrete plating is used, a transition material such as rock or concrete rubble shall be place for a minimum distance of 10 feet downstream of the toewall. This will help to slow the water and protect the toewall from scour.

All channels which are to be maintained by the City shall have a poured in place reinforced concrete bottom, extending a minimum width of eight (8) feet. The purpose for this is to allow easy access for maintenance equipment, reduce erosion and silting, and provide a permanent flow line reference for cleaning.

7.8.2 Maximum Allowable Side Slopes

A. Earth Side Slopes

In order to allow for mowing and maintenance, the maximum side slope for an unlined channel shall be 3 to 1, (i.e. three (3) feet horizontally for every one (1) foot vertically).

B. Concrete Side Slopes

Because concrete slopes require no mowing, the maximum side slope for a lined channel shall be 1 to 1 (i.e. one (1) foot horizontally for every one (1) foot vertically), up to a maximum height of five (5) feet. Above five (5) feet, the maximum side slope for concrete lined channels shall be 2 to 1, (i.e. two (2) feet horizontally for every one (1) foot vertically).

7.8.3 Maximum and Minimum Velocities

The ideal situation in open channel flow would be velocities that will cause neither silting nor erosion. To prevent the build-up of sediment in the channel, the minimum design velocity should be 2.0 feet per second in grass channels or grass channels with a concrete bottom and 2.5 feet per second in fully concrete lined channels. Furthermore, the longitudinal grade should be kept constant or increasing, if possible, to avoid silting. The maximum permissible velocity in a channel will depend on the soil type and vegetative cover. The values below may be used as a guide for selecting safe velocities in various type channels.

Channel Description	Minimum Velocity	Maximum Velocity
Grass lined, sandy soil	2.0 ft/sec	4.0 ft/sec
Grass lined, clay soil	2.0 ft/sec	5.0 ft/sec
Grass slopes, concrete bottom, sandy soil	2.0 ft/sec	4.0 ft/sec
Grass slopes, concrete bottom, clay soil	2.0 ft/sec	5.0 ft/sec
Riprap lined	2.0 ft/sec	8.0 ft/sec
Fully concrete lined	2.5 ft/sec	10.0 ft/sec

7.8.4 Backslope Drainage Systems

The purpose of a backdrain system is to collect overland flow from the channel overbanks and prevent it from flowing down the side slopes of the channel. The use of a backslope drainage swale is required for all unlined or partially lined channels where the possibility of overbank flow exists. When sizing the backslope drainage system, the engineer should carefully consider the drainage area to be intercepted. If the main channel passes through a large undeveloped area, the spacing of collection inlets will need to be reduced. Sufficient right-of-way must be provided to contain the flow in the backslope drain. The required design parameters for constructing a backdrain system are as follows:

A. Design Frequency

The backslope drainage system shall be designed for a five (5) year frequency storm.

B. Minimum Depth

The minimum depth of the backslope swale shall be 0.5 feet.

C. **Maximum Depth**

The maximum depth of the backslope swale shall be 2.0 feet.

D. **Minimum Grade**

The minimum longitudinal grade for the swale flow line shall be 0.20 percent.

E. **Maximum Side Slope**

Because the backslope drainage swale may be used as a means of access for maintenance equipment, the maximum side slope shall be 4 to 1, (i.e. four (4) feet horizontally for every one (1) foot vertically).

7.8.5 Drop Structures

The function of a drop structure is to reduce the channel velocities by flattening the upstream and downstream flow lines. The drop structure may be a straight drop or a sloped drop type, depending on the size and flow conditions. Both types of drop structures generally produce supercritical flow and a hydraulic jump downstream of the drop. The designer must provide sufficient erosion protection downstream to contain the hydraulic jump and return the flow to subcritical. Additionally, a phenomenon known as draw down will occur on the upstream side of the drop. Draw down is a rapid reduction in the depth of flow caused when water is allowed to free fall over a drop or weir structure. It is usually necessary to reduce the cross-sectional area of the channel at the drop structure in order to maintain a constant water surface and non-erosive velocities upstream. This is normally done by incorporating a check dam or weir into the design of the drop. All drop structures should be analyzed over a full range of flows and tailwater condition. Refer to Section 7.5.3 for additional information on analyzing flow over a free outfall.

7.8.6 Confluences

At the intersection of a side channel and a main channel, the extent of erosion protection varies with the velocity in the side channel and the angle of intersection. The required minimum dimensions for erosion protection at the intersection of two ditches is shown in Figure 7-3.

7.8.7 Bends

Channel bends or curves shall be made as gradual as possible to reduce erosion. For channel bends between 45 and 90 degrees, with a radius of curvature, measured from the centerline of the channel, less than two and one half (2 1/2) times the top width of the channel, erosion protection shall be used. Protection shall extend to a height equal to the design water surface, and extend a minimum of 20 feet downstream of the bend on the outside bank. Additional protection may be necessary on the channel bottom, inside bank, or further than 20 feet downstream if velocities and channel geometry indicate a potential erosion problem. Bends greater than 90 degrees should not be used. Super elevation of the water surface should be anticipated for bends greater than 45 degrees and additional slope protection provided. The height of super elevation may be approximated using the formula,

$$h = V^2\, T/(R\, g) \qquad \text{(Eq. 7-18)}$$

where,

- h = super elevation of outside water surface in feet.
- V = Average velocity in the channel in feet per second.
- T = Top width of the channel in feet.
- R = Radius of curvature measured from the centerline of the channel in feet.
- g = Acceleration due to gravity, (32.2 ft/sec^2).

7.8.8 Pipe Outfalls

Erosion protection is normally required in areas of high turbulence or velocity typically found at the outfall of backslope drains and storm sewers into the main channel. Erosion protection for pipe outfalls should be designed in accordance with Section 6.5.0 of this manual.

7.8.9 Culverts

In areas where culvert outlet velocities exceed the allowable maximum velocity for the channel, erosion protection should be installed in accordance with Section 8.4.2 of this manual.

7.9.0 REQUIRED EASEMENTS

All open channels, which serve more than one lot or tract of land, shall be placed within properly dedicated drainage easements. These easements shall be of sufficient width to contain the channel under the design flow, backslope drainage system, and provide unobstructed maintenance access to the channel. Where the backslope drainage system is of sufficient width, it may serve as a means of maintenance access. However, where a backslope drainage system is not required or is of insufficient width, a minimum of ten (10) feet shall be dedicated continuously along both sides of the channel for use as maintenance access.

END OF SECTION

TABLE 7-1

MANNING'S ROUGHNESS COEFFICIENTS FOR EXISTING CHANNELS
(For Use With Equation 7-16)

	Modifying Factor	Channel Condition	Value
n_0	Basic Coefficient	Earth	0.020
		Cut Rock	0.025
		Fine Gravel	0.024
		Course Gravel	0.028
n_1	Channel Irregularity	Smooth	0.000
		Minor	0.005
		Moderate	0.010
		Severe	0.020
n_2	Cross Section Variation	Gradual	0.000
		Alternates Occasionally	0.005
		Alternates Frequently	0.012
n_3	Obstructions	Negligible	0.000
		Minor	0.012
		Appreciable	0.025
		Severe	0.050
n_4	Vegetation	Low	0.008
		Medium	0.018
		High	0.038
		Very High	0.075
m	Meandering	Minor	1.00
		Appreciable	1.15
		Severe	1.30

Equation 7-16, $n = (n_0 + n_1 + n_2 + n_3 + n_4)m$

TABLE 7-2

MANNING'S ROUGHNESS COEFFICIENTS FOR NEW CHANNELS
(For Use With Manning's Equation)

Channel Type and Description	Manning's Coefficient (n)
1. Fully Concrete Lined Channels	0.012
2. Fabric Formed Concrete	0.025
3. Interlocking Precast Concrete Blocks	0.028
4. Rock or Concrete Rubble Lined	0.030
5. Gabions (Rock filled Wire Baskets)	0.032
6. Grass Slopes with Concrete Bottom	0.025
7. Grass Lined Channels	0.035

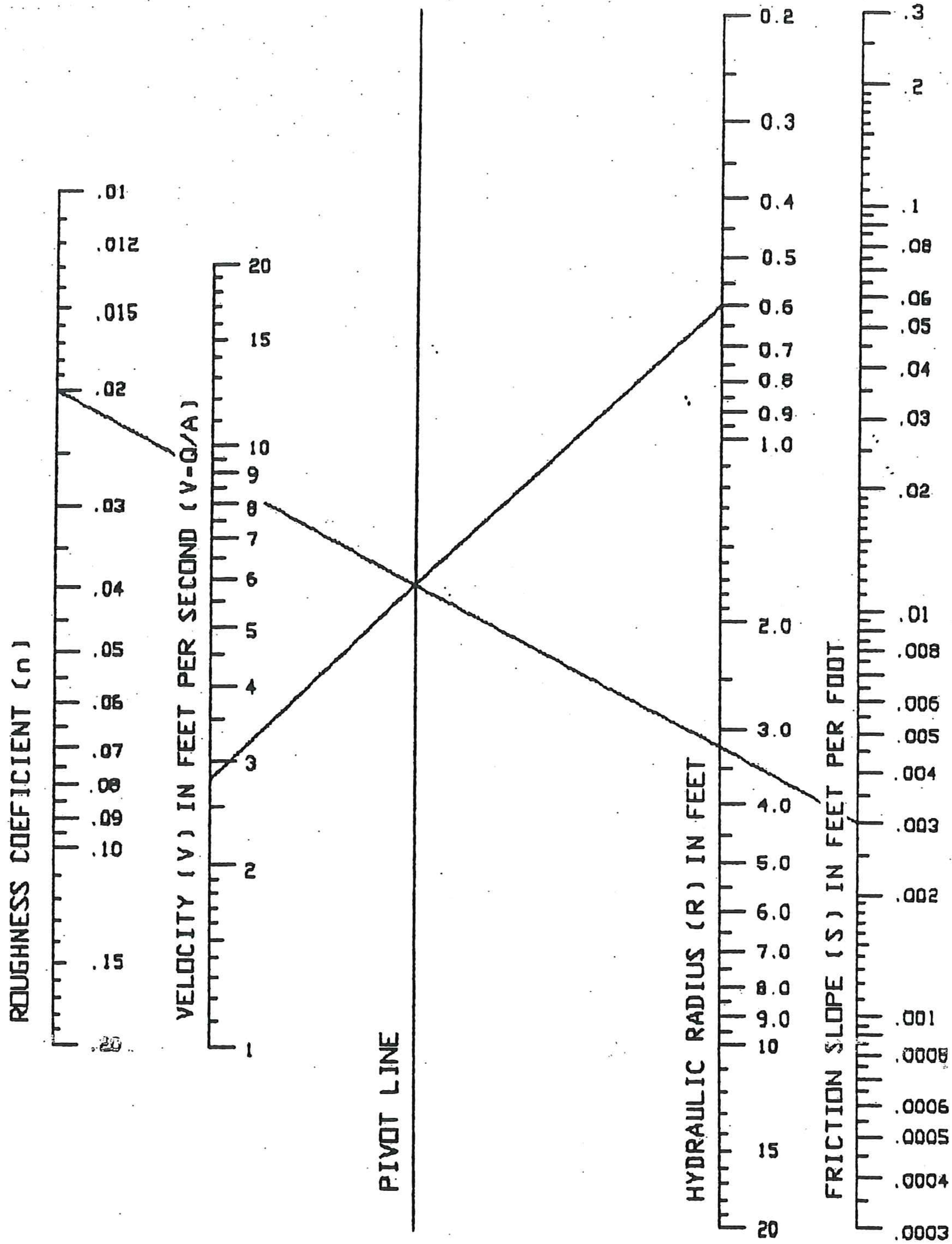
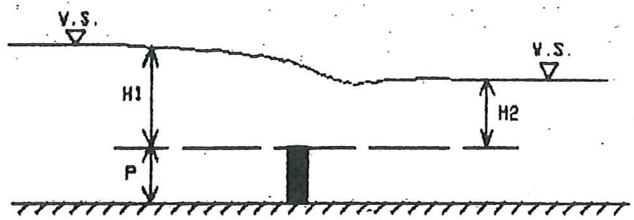


FIGURE 7-1
SOLUTION TO MANNING'S EQUATION
FOR ANY CROSS SECTION



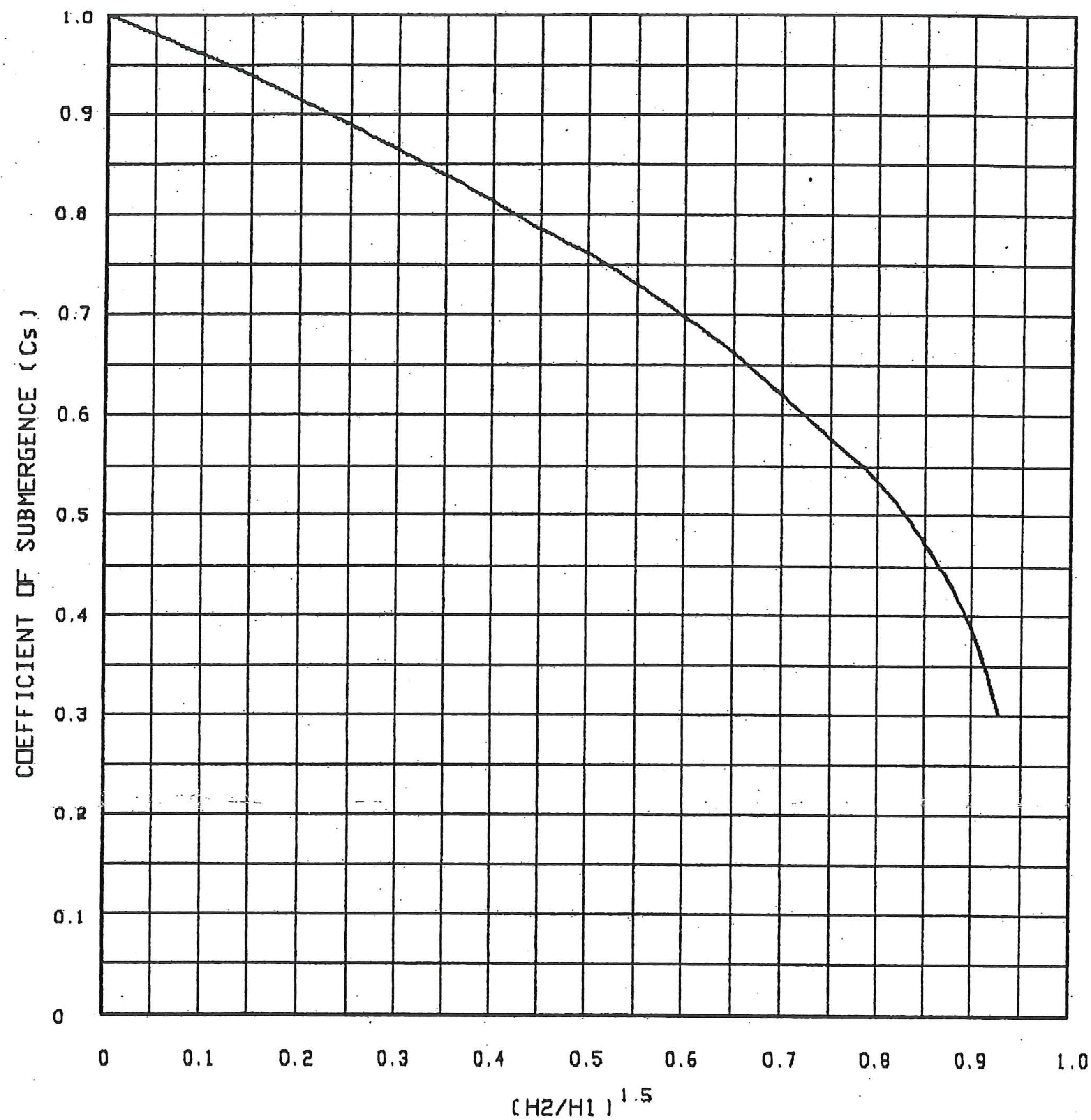
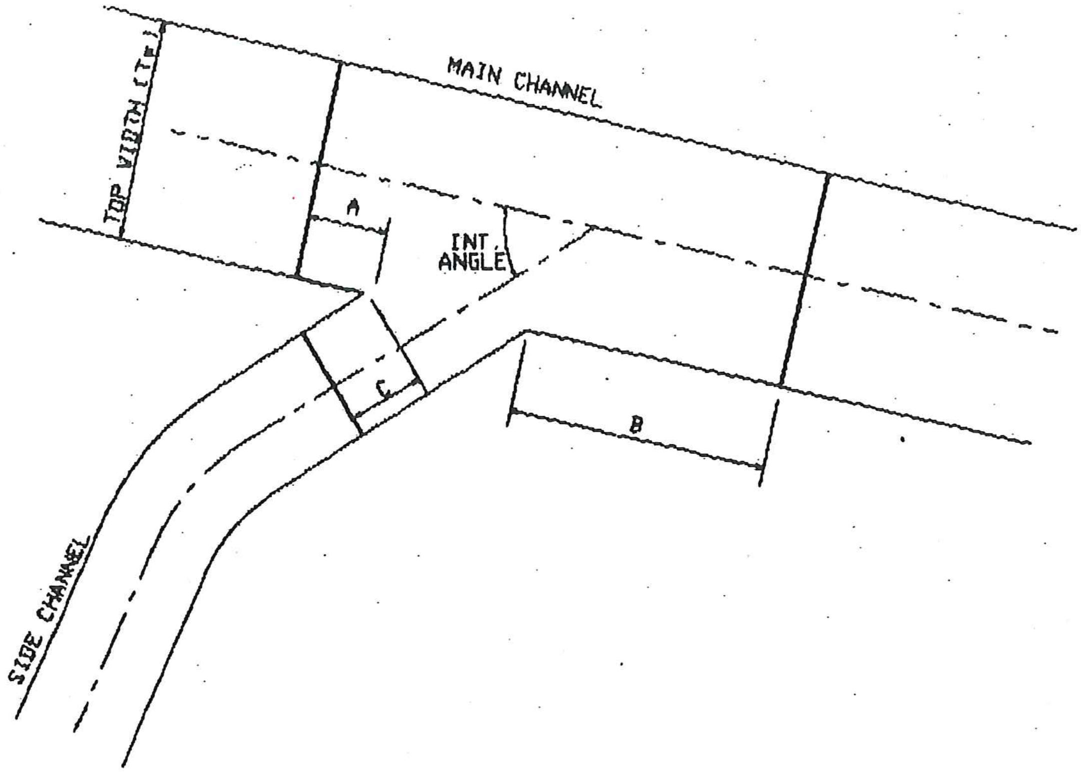


FIGURE 7-2
COEFFICIENT OF SUBMERGENCE FOR WEIR FLOW



MINIMUM RECOMMENDED EROSION PROTECTION

INTERSECTION ANGLE	SIDE CHANNEL VELOCITY	LOCATION	DISTANCE
0 to 45	< 3 ft/sec	A	0 ft
		B	0 ft
		C	0 ft
0 to 45	> 3 ft/sec	A	20 ft
		B	1.3 x Tw (50' min)
		C	20 ft
45 to 90	< 3 ft/sec	A	0 ft
		B	0 ft
		C	0 ft
45 to 90	> 3 ft/sec	A	20 ft
		B	0.5 x Tw (50' min)
		C	20 ft

FIGURE 7-3
EROSION PROTECTION AT CONFLUENCES

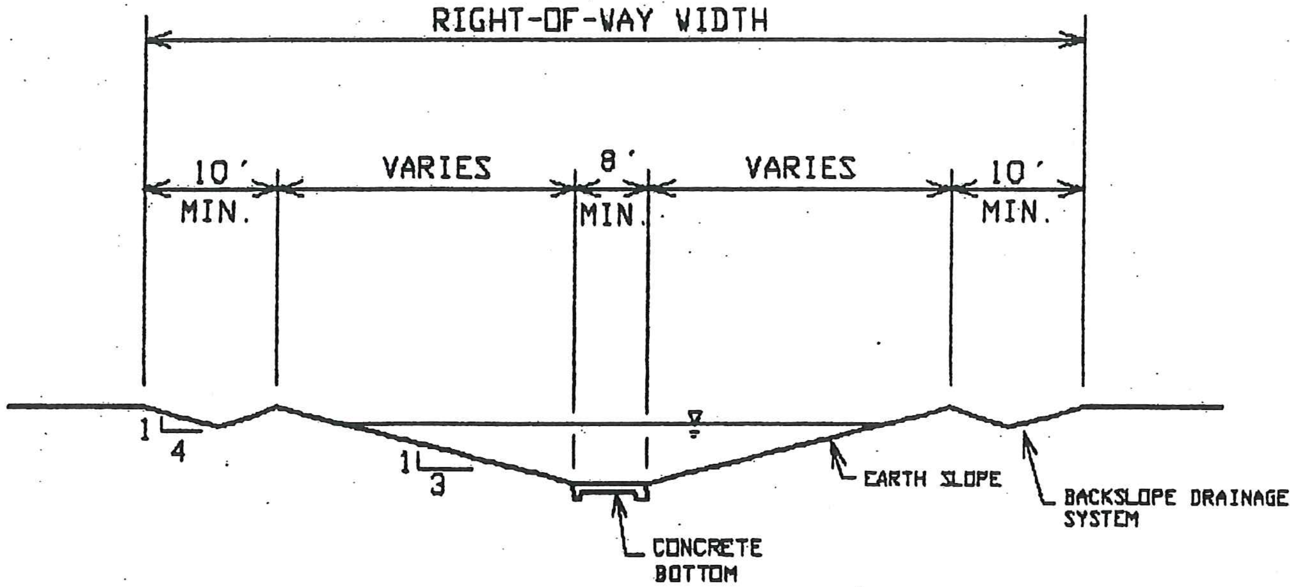


FIGURE 7-4
MINIMUM REQUIREMENTS FOR UNLINED CHANNELS

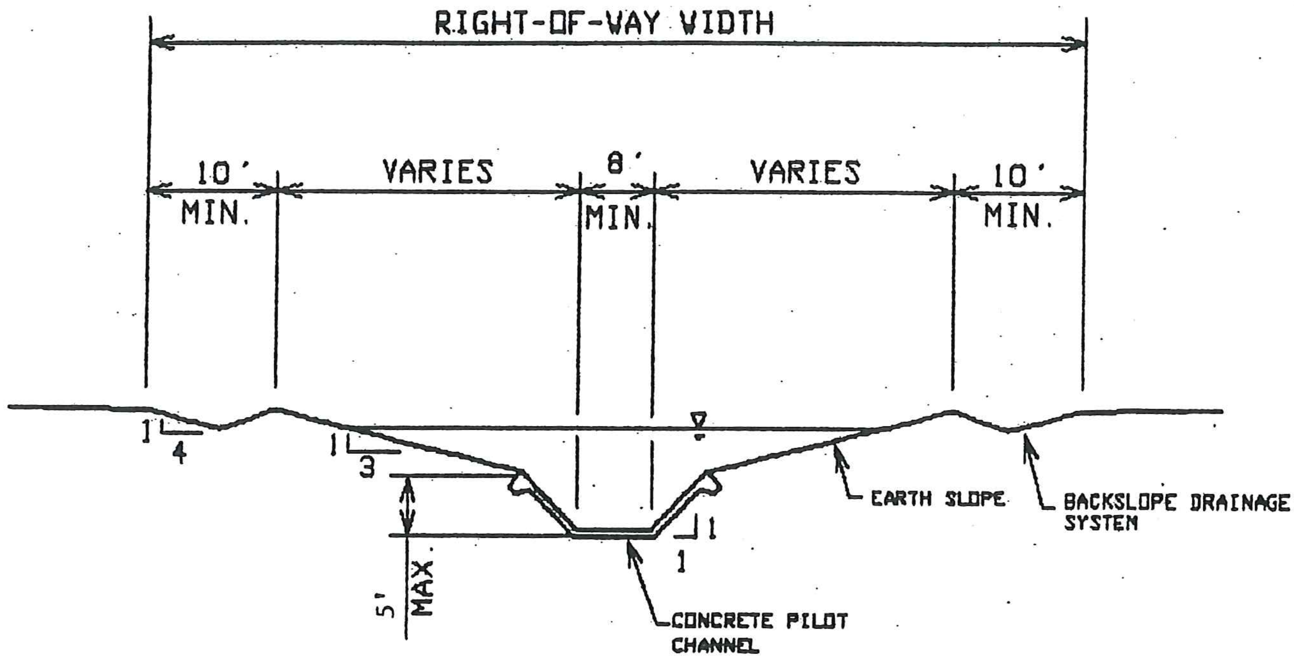
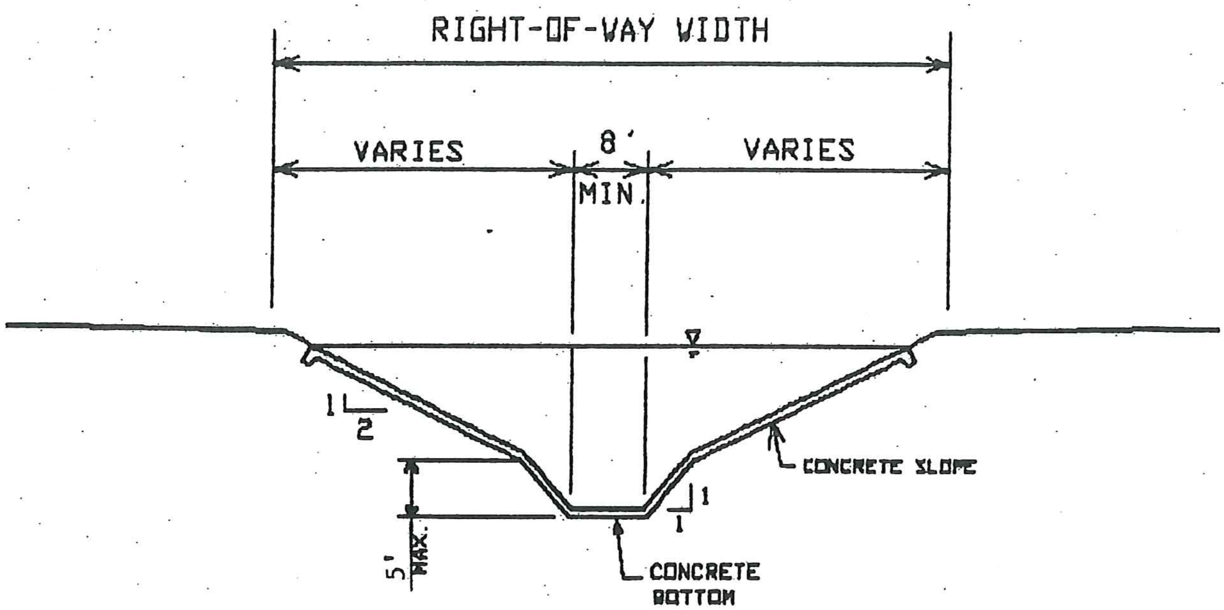
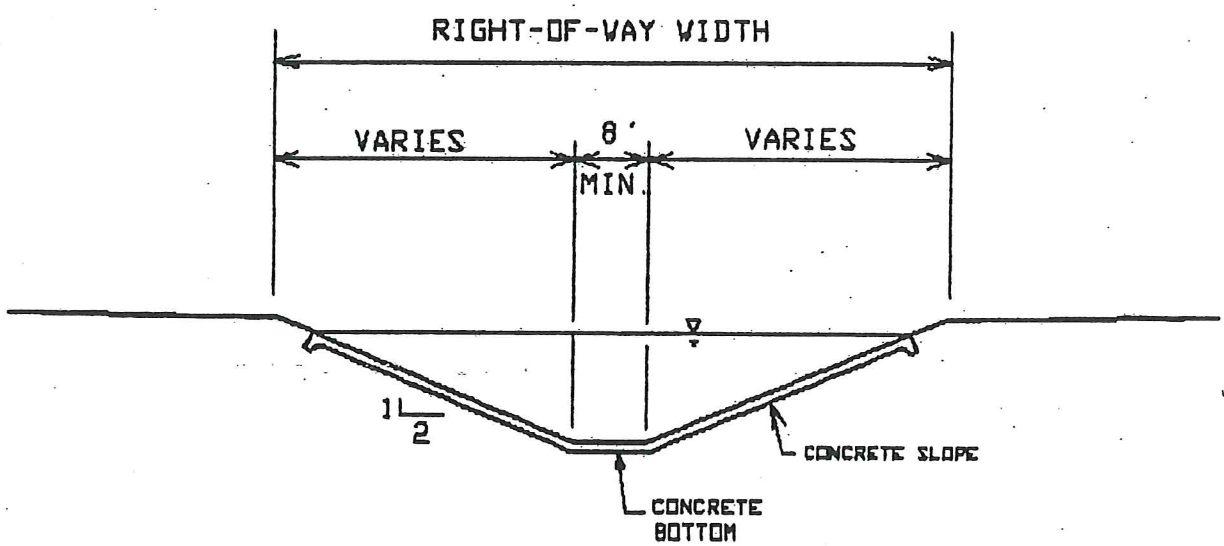


FIGURE 7-5
MINIMUM REQUIREMENTS FOR CONCRETE PILOT CHANNELS



FULLY LINED WITH PILOT CHANNEL



FULLY LINED WITHOUT PILOT CHANNEL

FIGURE 7-6
MINIMUM REQUIREMENTS FOR FULLY LINED CHANNELS

SECTION 8
CULVERTS

CONTENTS

SECTION 8 - CULVERTS

8.1.0 GENERAL

The function of a drainage culvert is to convey the design storm flow without causing excessive backwater, overtopping of the roadway, or creating erosive downstream velocities. When selecting a culvert, the engineer must keep the upstream water surface elevation and downstream velocities within allowable limits.

8.2.0 DESIGN REQUIREMENTS

8.2.1 Culvert Materials

A. Residential Driveway Culverts

The following materials shall be acceptable for use as residential driveway culverts:

(1) Reinforced Concrete Box
(2) Reinforced Concrete Pipe
(3) Corrugated Aluminum Pipe
(4) Asphalt Coated, Aluminized, Corrugated Steel Pipe
(5) Asphalt Coated, Corrugated Steel Pipe
(6) Corrugated Steel Pipe

B. Commercial Driveway Culverts

The following materials shall be acceptable for use as commercial driveway culverts:

(1) Reinforced Concrete Box
(2) Reinforced Concrete Pipe
(3) Corrugated Aluminum Pipe
(4) Asphalt Coated, Aluminized, Corrugated Steel Pipe
(5) Asphalt Coated, Corrugated Steel Pipe

C. Roadway Culverts

The following materials shall be acceptable for use as roadway culverts:

(1) Reinforced Concrete Box
(2) Reinforced Concrete Pipe
(3) Corrugated Aluminum Pipe
(4) Asphalt Coated, Aluminized, Corrugated Steel Pipe

8.2.2 Minimum Culvert Size

The minimum allowable culvert diameter shall be as follows:

(1) Residential Driveway Culverts - 12 inches in diameter.
(2) Commercial Driveway Culverts - 18 inches in diameter.
(3) Roadway Culverts - 18 inches in diameter.

8.2.3 Minimum Culvert Slope

The minimum longitudinal slope for any type culvert shall be 0.10 percent, (i.e. 0.10 feet of fall per 100 feet).

8.2.4 Minimum Culvert Velocity

The minimum design flow velocity for any culvert shall be 2.0 feet per second.

8.2.5 Maximum Culvert Velocity

The maximum allowable culvert velocity shall be 8.0 feet per second.

In most all cases of culvert flow, the total head losses through the culvert are a function of the square of the mean flow velocity. Therefore, limiting the maximum allowable culvert velocity, provides a simple means for limiting the magnitude of losses through the culvert.

The maximum allowable culvert velocity may be exceeded in those cases where a smaller culvert size would have no effect on the flood plain elevation beyond the limits of the proposed development. An example of where higher culvert velocities would be allowable is where a control section, such as a drop structure or weir, is located immediately upstream of the culvert.

8.3.0 CULVERT HYDRAULICS

The hydraulic capacity of a culvert depends not only on the culvert's physical characteristics, but also on the flow conditions. Laboratory tests and field observations show that there are two (2) major types of culvert flow: (1) inlet controlled flow, and (2) outlet controlled flow. Inlet controlled flow occurs when the culvert barrel is capable of carrying the water more efficiently than it can enter the culvert. Under outlet control conditions, the inlet will allow water to enter the culvert faster than the barrel is capable of discharging it. Further discussion of both flow types follows.

8.3.1 Inlet Controlled Flow

Under inlet control, the cross-sectional area of the barrel, the inlet geometry, and the depth of the headwater are the only factors affecting capacity. The slope of the culvert must be steep enough so that the culvert does not flow full. If the headwater depth is insufficient to submerge the top of the culvert opening, flow though the culvert entrance is characterized as weir flow. If the headwater depth submerges the top of the culvert, but the culvert does not flow full, then flow through the culvert entrance is classified as orifice flow. The nomographs, Figures 8-1 ,8-2, and 8-3 may be used to determine the headwater depth for culverts flowing under inlet control conditions.

8.3.2 Outlet Controlled Flow

Under outlet control conditions, the flow though the culvert is influenced by the headwater depth, the tailwater depth, the inlet geometry, and the area, slope, roughness, and length of the culvert barrel. The capacity of a culvert flowing under outlet control is calculated using the Bernoulli equation. In the application of this equation, an energy balance is determined between the headwater at the culvert entrance and the tailwater at the culvert outlet. This energy balance is a function of inlet losses, frictional losses, velocity head, tailwater depth, slope and culvert length. The general equation used in determining headwater depth for a culvert flowing under outlet control is as follows:

$$Hw = H + h_0 - SL \qquad \text{(Eq. 8-1)}$$

where,

Hw = Vertical distance, in feet, from the upstream flow line of the culvert to the upstream water surface.
h_0 = Vertical distance, in feet, from the downstream flow line of the culvert to the downstream hydraulic grade line. (See Section 8.3.2 (B)).
L = Culvert length in feet.
S = Culvert slope in feet per foot.
H = Total head losses through the culvert in feet, $H = h_e + h_f + h_v$, (See Section 8.3.2 (A)).

where,

h_e = entrance head loss in feet.
h_f = friction head loss in feet.
h_v = velocity head in feet.

A. Minor Head Losses (H)

Minor head losses are a measure of the total energy required to force water through a culvert. The losses through a culvert may be characterized as: (1) entrance head losses, (2) friction head losses, and (3) velocity head losses. The formulas used for calculating each of these losses are as follows:

$$h_e = K_e(V_2^2 - V_1^2)/(2g) \quad \text{(Eq. 8-2)}$$

$$h_f = L(Qn/(1.486AR^{2/3}))^2 \quad \text{(Eq. 8-3)}$$

$$h_v = V_2^2/(2g) \quad \text{(Eq. 8-4)}$$

$$H = h_e + h_f + h_v \quad \text{(Eq. 8-5)}$$

where,

H = Total head losses through the culvert in feet.
h_e = Entrance head losses in feet.
h_f = Frictional head losses in feet.
h_v = Velocity head in feet.
K_e = Entrance loss coefficient, (See Table 8-1).
V_2 = Velocity of flow in culvert assuming full pipe flow, in feet per second.
V_1 = Velocity of flow approaching culvert in feet per second. (Normally assumed to be negligible)
A = Area of culvert in square feet.
n = Manning's roughness coefficient.
L = Length of the culvert in feet.
R = Hydraulic radius in feet, (Area/Wetted Perimeter).
g = Acceleration due to gravity (32.2 ft/sec^2).

The total head losses (H) for a culvert flowing under outlet control may be calculated using Equation 8-5 or by using the outlet control nomographs, Figure 8-4, 8-5 or 8-6, included in this section.

B. Downstream Hydraulic Grade Line (h_0)

The downstream hydraulic grade line used in Equation 8-1 is defined as the depth from the culvert flow line to the water surface at the culvert outlet. The value for h_0 may be approximated by the following equation:

$$h_0 = (dc+D)/2 \text{ or } Tw \quad \text{(whichever is greater)} \quad \text{(Eq. 8-6)}$$

where,

h_0 = Downstream hydraulic grade line used in Equation 8-1.
dc = Critical depth in culvert in feet. (Section 8.3.2 (C)).
D = Height of culvert in feet.
Tw = Tailwater depth of the receiving channel.

C. Critical Depth (dc)

Critical depth is defined as the depth for which the specific energy (sum of the depth and velocity head) is a minimum. For any given discharge and channel shape there is only one critical depth. The general formula for critical flow in any shaped section is,

$$Q^2/g = A^3/T \qquad \text{(Eq. 8-7)}$$

where,

Q = Critical discharge in cubic feet per second.
A = Cross-sectional area of flow in square feet.
T = Water surface width, in feet, at critical discharge.
g = Acceleration due to gravity (32.2 ft/sec^2).

For the special case of a rectangular channel or box culvert, Equation 8-7 may be simplified to the following form:

$$dc = (q^2/g)^{1/3} \qquad \text{(Eq. 8-8)}$$

where,

dc = Critical depth in feet.
q = Discharge per unit width of box.
g = Acceleration due to gravity (32.2 ft/sec^2).

Applying Equation 8-7 to a circular conduit is rather tedious and therefore has not been included. The nomograph in Figure 8-7 will provide a reasonable means of determining critical depth for the most commonly encountered pipe sizes.

D. Tailwater Depth (Tw)

In a culvert flowing with outlet control, the tailwater depth is an important factor in determining headwater depth. If the water surface elevation in the receiving channel has not been determined by a previous hydraulic study, then the engineer must make a realistic estimate of the possible tailwater conditions. A field inspection should be made to check for downstream constrictions that may produce a backwater affect at the culvert outlet. Tailwater elevations are often controlled by downstream obstructions or by flood water elevations in another channel. An approximation of the depth of flow in an improved channel with no downstream constrictions may be made using Manning's equation for uniform flow. However, if the water surface in the outlet channel is influenced by downstream obstructions, a backwater analysis is required. Details for determining the water surface elevation by Manning's equation or by backwater analysis are included in Section 7 of this manual.

8.4.0 OUTLET VELOCITY

The outlet velocity from a culvert is normally higher than the velocity in the receiving channel. Therefore, the engineer must make an estimate of the culvert discharge velocity and, in many cases, provide downstream erosion control to protect the channel banks. It is almost always more economical to provide erosion protection than to try and increase the culvert size to reduce the velocity.

The outlet velocity for a culvert may be determined using the continuity equation as follows:

$$V = Q/A \qquad \text{(Eq. 8-9)}$$

where,

V = Mean velocity in feet per second.
Q = Total culvert discharge in cubic feet per second.
A = Cross-sectional area of flow in square feet corresponding to the depth of flow at the outlet.

Control Type	Flow Conditions	Depth of Flow at Outlet
Inlet Control	Tw > D	Full Depth
Inlet Control	Tw < D	Normal Depth
Outlet Control	Tw > D	Full Depth
Outlet Control	D > Tw > dc	Tailwater Depth
Outlet Control	Tw < dc	Critical Depth

8.5.0 EROSION CONTROL

8.5.1 Culvert Headwalls and Endwalls

Properly sized headwalls and endwalls shall be provided on all commercial driveways and roadway culverts placed within city right-of-way. In addition, headwalls and endwalls shall be required for all residential driveway culverts in subdivisions platted after the effective date of this manual. Their function is to anchor the culvert, to prevent erosion and scour, and to insure bank stability. For culverts less than 42 inches in height, the headwall shall be sloped at 4 to 1 or flatter, (i.e. four (4) feet horizontal for every one (1) foot vertical), to reduce the safety hazard to motor vehicles. For larger culverts, the headwalls may be sloped or vertical. Wingwalls and aprons may be required depending on the particular site conditions. When flared wingwalls are used, they should be aligned with the direction of the approaching and receiving channel and not with the culvert axis.

8.5.2 Downstream Erosion Protection

High discharge velocities from the culvert may cause severe turbulence, damaging the downstream channel or roadway embankment. For culverts with an outlet velocity in excess of the allowable velocity of the receiving channel, downstream erosion protection is required. The downstream protection may be constructed of reinforced concrete, rock, or concrete rubble. Because of the higher roughness coefficient, rock and concrete rubble is preferable to smooth finished concrete for larger installations. The minimum length for downstream erosion protection may be estimated using the following equation:

$$L = 0.75\ DV \qquad \text{(Eq. 8-10)}$$

where,

L = Minimum length of downstream protection in feet.
D = Culvert height in feet.
V = Discharge velocity is feet per second.

When using a sloped headwall, the length of the slope may be considered as part of the required protection. For example, a 2 ft. diameter culvert with a discharge velocity of 6 ft/sec, would require 0.75 x 2 x 6 = 9 ft. of erosion protection. However, if the headwall is sloped at 4.5 to 1, (4.5 x 2 ft. = 9 ft.), no additional protection would be required.

8.6.0 DESIGN PROCEDURE

It is possible, by involved mathematical computations, to determine the probable type of flow under which a culvert will operate for a given set of conditions. However, such computations can be avoided by determining the headwater depth necessary for a given discharge under both inlet and outlet control, and the larger of the two will define the type of flow control and the corresponding headwater depth. Below is the recommended procedure for selecting a culvert size. The instructions listed below are intended to be used with the Culvert Design Worksheet, Figure A-3, included in the appendix.

STEP 0 List Design Data

Make a list of the required design data including, design discharge (Q), length of culvert (L), slope of culvert (S), culvert type, Manning's roughness coefficient (n), entrance type, and entrance loss coefficient (k_e).

STEP 1 Select Trial Culvert Size

A. Assume a first trial culvert size by using the approximating equation, A = Q/6, where, A = cross-sectional area of trial culvert, Q = design discharge, and 6 is the estimated barrel velocity.

B. If the trial size is too large because of height restrictions, multiple culverts may be used by dividing the total discharge (Q), by the number of barrels.

STEP 2 Calculate Cross-sectional Area of Culvert

Calculate the cross-sectional area of the culvert pipe. Table 6-4 may be used for circular and arch pipe sections.

STEP 3 Calculate Hydraulic Radius of Culvert

Calculate the hydraulic radius of the culvert pipe using the equation R = A/P, where R = hydraulic radius in feet, A = cross-sectional area of culvert pipe in square feet, and P = the total perimeter of the culvert in feet. Table 6-4 list the hydraulic radius of most circular and arch pipe sections.

STEP 4 Determine Headwater Depth / Culvert Height Ratio

Assuming inlet controlled flow, determine the required headwater depth / culvert height ratio, (Hw/D), using the appropriate inlet control nomograph, Figures 8-1, 8-2, or 8-3.

STEP 5 Determine Headwater Depth Assuming Inlet Control

Calculate the headwater depth assuming inlet controlled flow conditions by multiplying the results of Step 4 by the culvert height in Step 1. If the headwater depth is greater than the desired, try another size culvert and return to Step 1.

STEP 6 Determine Tailwater Depth

Determine the tailwater depth (Tw) at the outlet for the design flood conditions. (Refer to Section 8.4.2 (D).

STEP 7 Determine Critical Depth

Determine the critical depth of flow in the culvert under the design discharge. Use Equation 8-8 for rectangular culverts and Figure 8-7 for circular sections.

STEP 8 Determine Downstream Hydraulic Grade Line, ($Tw < d_c$)

A. If the tailwater depth from Step 6 is greater than the critical depth from Step 7, then skip to Step 9, else,

B. Set the downstream hydraulic grade line (h_0) equal to the critical depth (d_c), skip to Step 11.

STEP 9 Determine Downstream Hydraulic Grade Line, ($D > Tw > d_c$)

A. If the tailwater depth from Step 6 is greater than the culvert height, Step 1, then skip to Step 10, else,

B. Set the downstream hydraulic grade line (h_0) equal the tailwater depth (Tw), skip to Step 11.

STEP 10 Determine Downstream Hydraulic Grade Line, ($Tw > D$)

Set the downstream hydraulic grade line (h_0) equal the tailwater depth (Tw).

STEP 11 Determine Total Head Loss Assuming Outlet Control

Assuming outlet controlled flow conditions, find the total head loss (H) using Equation 8-5 or by using the appropriate nomograph, Figure 8-4, 8-5, or 8-6.

STEP 12 Determine Headwater Depth Assuming Outlet Control

Assuming outlet controlled flow, find the headwater depth (Hw) using Equation 8-1.

STEP 13 Recalculate Headwater Depth If $Hw > 1.5D$ and $Tw < d_c$

A. If the headwater depth from Step 12 is less than 1.5 times the culvert height, then skip to Step 14.

B. If the tailwater depth from Step 6 is greater than the critical depth, Step 7, then skip to step 14.

C. If $Hw > 1.5D$ and $Tw < d_c$, then increase the headwater depth calculated in Step 12 by the amount, $(D-d_c)/2$.

STEP 14 Control Type

The culvert flow type will be either inlet or outlet controlled. If the headwater calculated in Step 5 is greater than that calculated in Steps 12 or 13, then the culvert is operating under inlet control. Otherwise, the culvert is flowing under outlet control.

STEP 15 Select Actual Headwater Depth

The actual headwater depth for the design culvert will be the greater of the three depths calculated in Steps 5, 12, and 13.

STEP 16 Calculate Actual Culvert Velocity

Calculate the actual culvert outlet velocity using Equation 8-9 and the cross-sectional area corresponding to the depth of flow at the outlet.

STEP 17 Comments

If the headwater depth or outlet velocity shown in Steps 15 or 16 are above or below the desired, then select another culvert size and return to Step 1.

END OF SECTION

TABLE 8-1

ENTRANCE LOSS COEFFICIENTS FOR CULVERTS
(For Use With Equation 8-2 and 8-5)

Entrance Type	Entrance Coefficient (K_e)
REINFORCED CONCRETE PIPE	
Headwall With or Without Wingwalls	
Socket End	0.20
Square Edge	0.50
Projecting Entrance (no headwall or wingwalls)	
Socket End	0.20
Square Edge (RCP)	0.50
Sloping Entrance	
Mitered Pipe	0.50
Sloping Headwall	0.50
CORRUGATED METAL PIPE	
Headwall With or Without Wingwalls	0.50
Projecting Entrance (no headwall or wingwalls)	0.90
Sloping Entrance	
Mitered Pipe	0.70
Sloping Headwall	0.50
BOX CULVERTS	
Headwall (no wingwalls)	0.50
Wingwalls at 30 to 75 deg. to Barrel Axis	0.40
Wingwalls at 15 and 90 deg. to Barrel Axis	0.50
Wingwalls Parallel to Culvert Walls	0.70

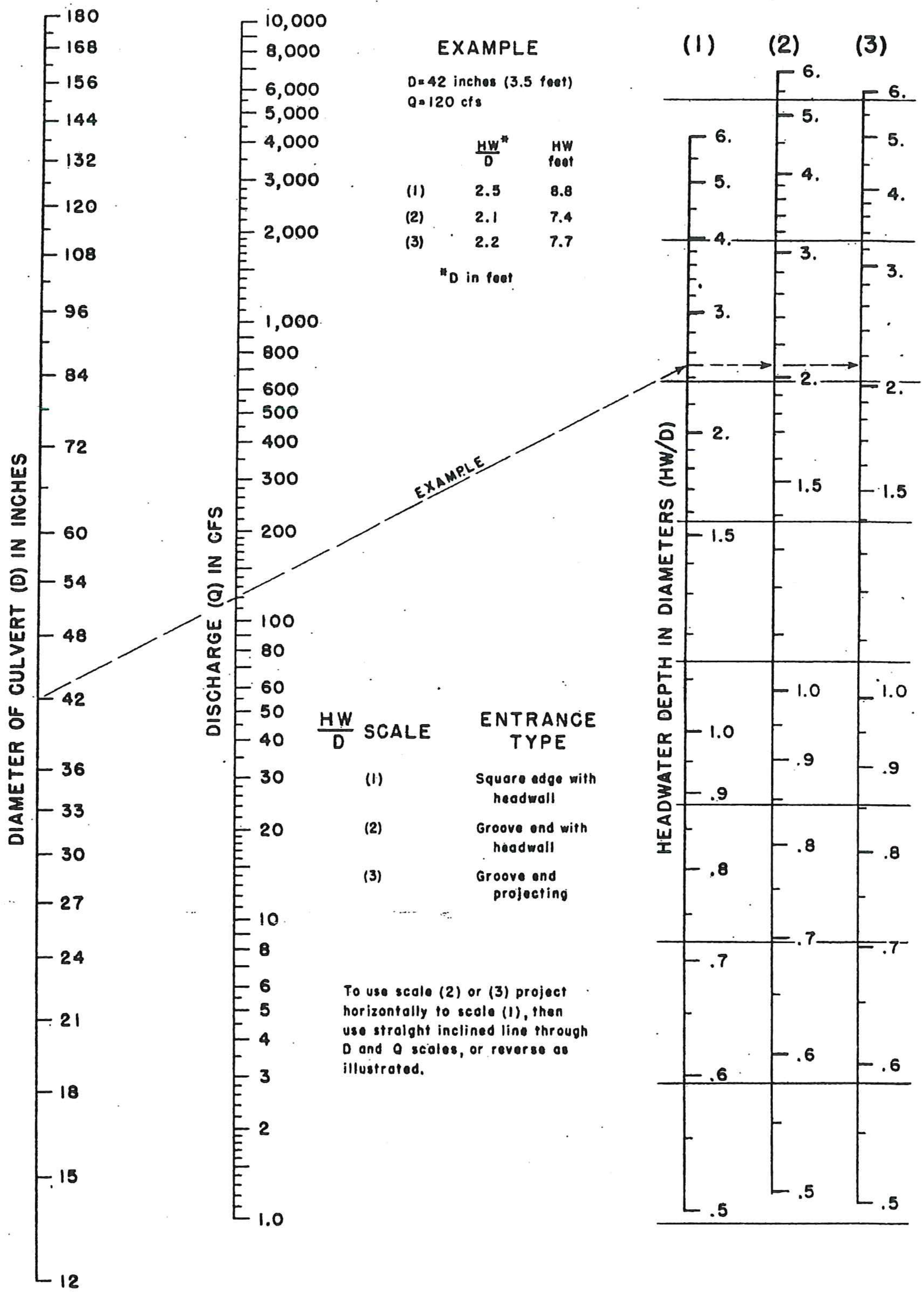


FIGURE 8-1
HEADWATER DEPTH FOR CONCRETE PIPE CULVERTS
WITH INLET CONTROL

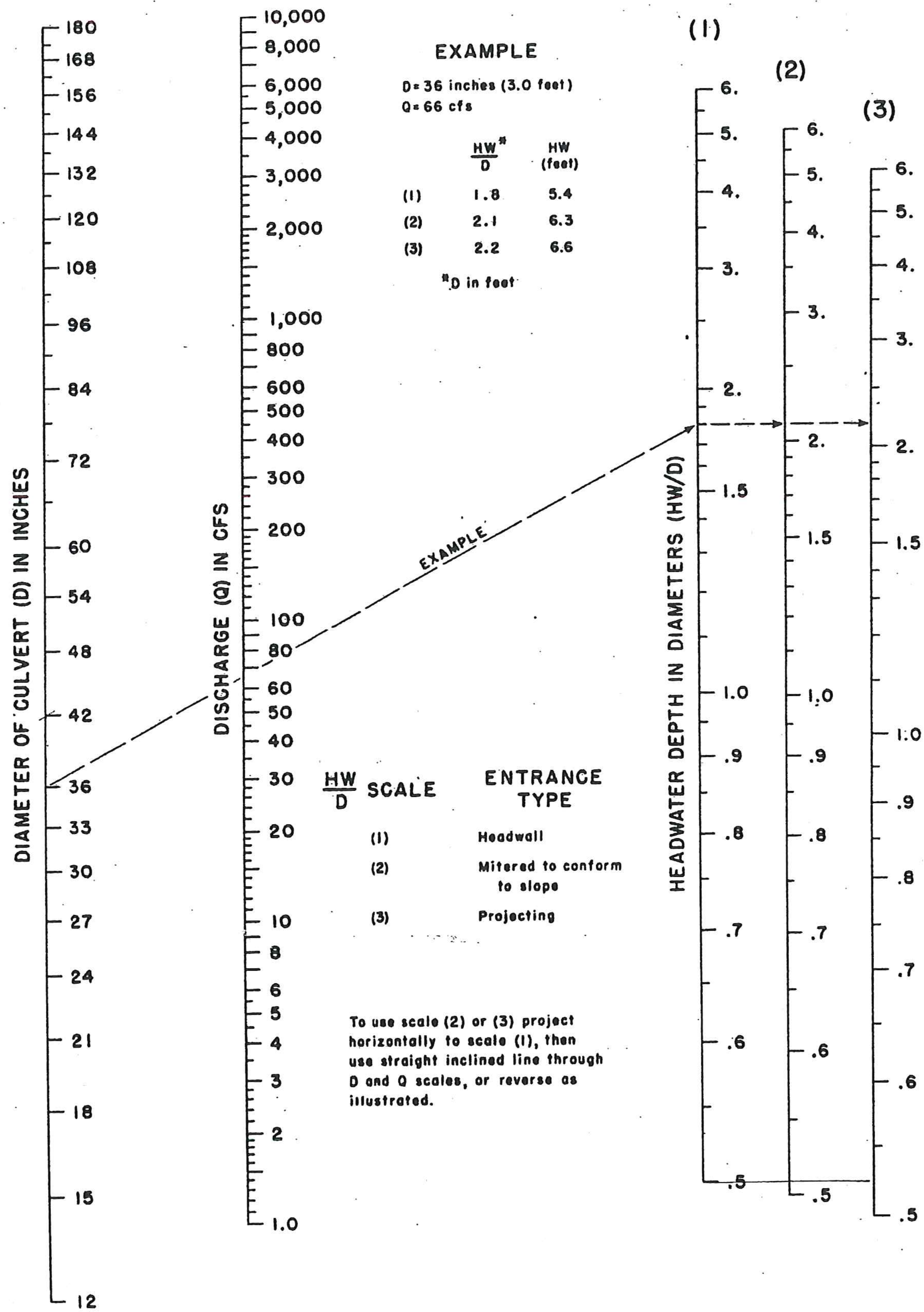


FIGURE 8-2
HEADWATER DEPTH FOR CORRUGATED METAL PIPE CULVERTS
WITH INLET CONTROL

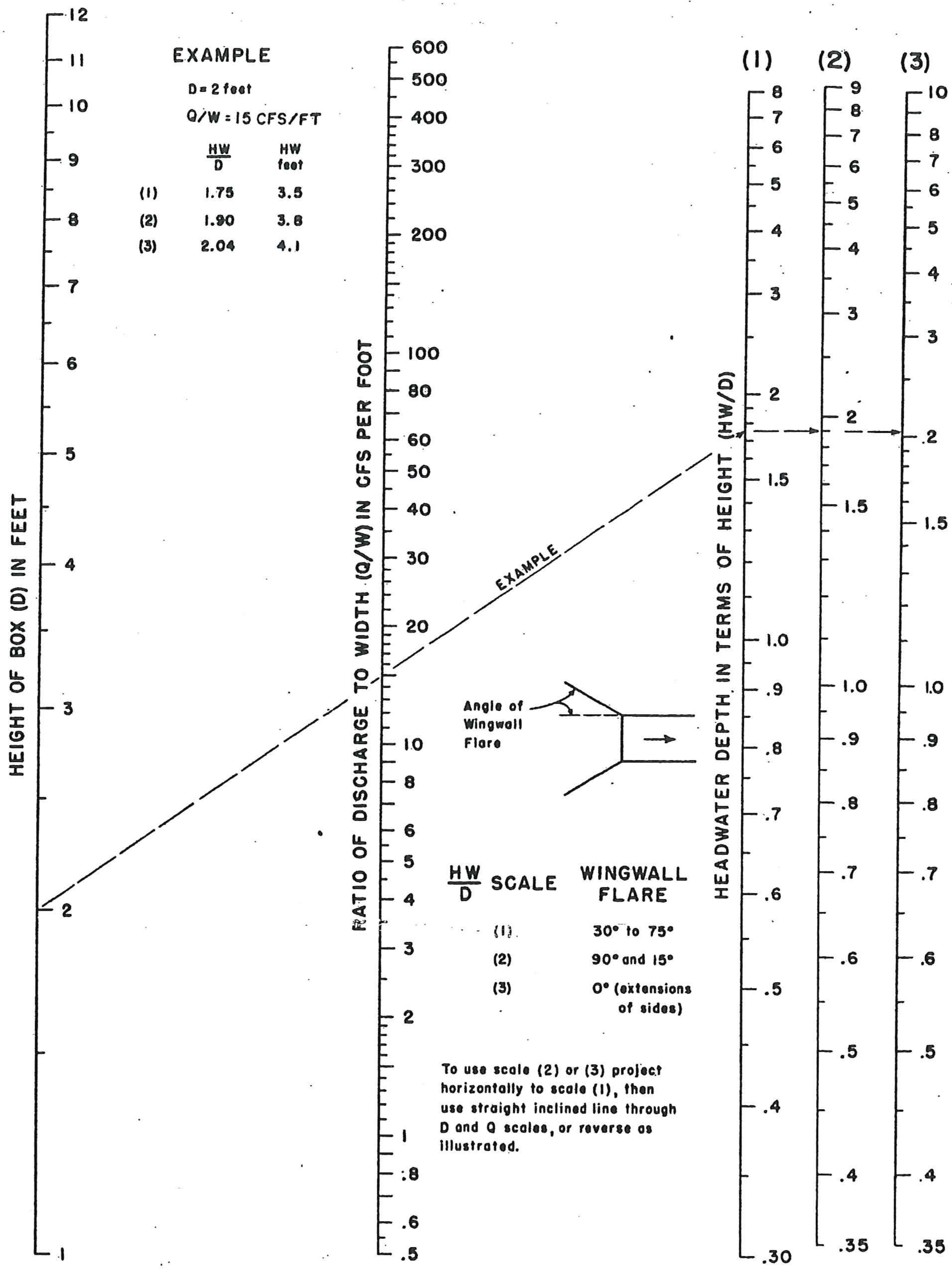
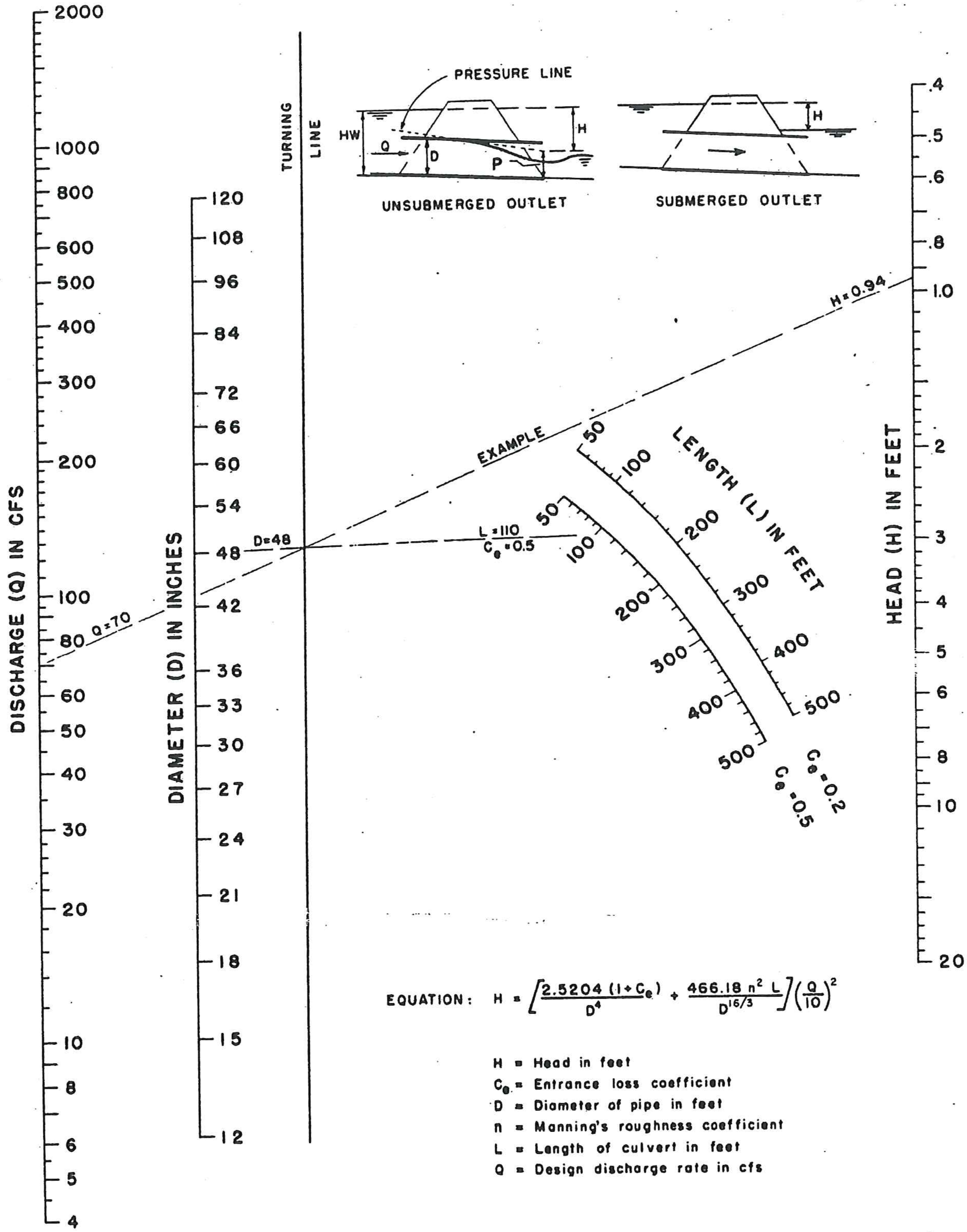


FIGURE 8-3
HEADWATER DEPTH FOR CONCRETE BOX CULVERTS
WITH INLET CONTROL



EQUATION: $H = \left[\frac{2.5204\,(1+C_e)}{D^4} + \frac{466.18\,n^2\,L}{D^{16/3}}\right]\left(\frac{Q}{10}\right)^2$

H = Head in feet
C_e = Entrance loss coefficient
D = Diameter of pipe in feet
n = Manning's roughness coefficient
L = Length of culvert in feet
Q = Design discharge rate in cfs

FIGURE 8-4
TOTAL HEAD LOSS FOR CONCRETE PIPE CULVERTS
WITH OUTLET CONTROL

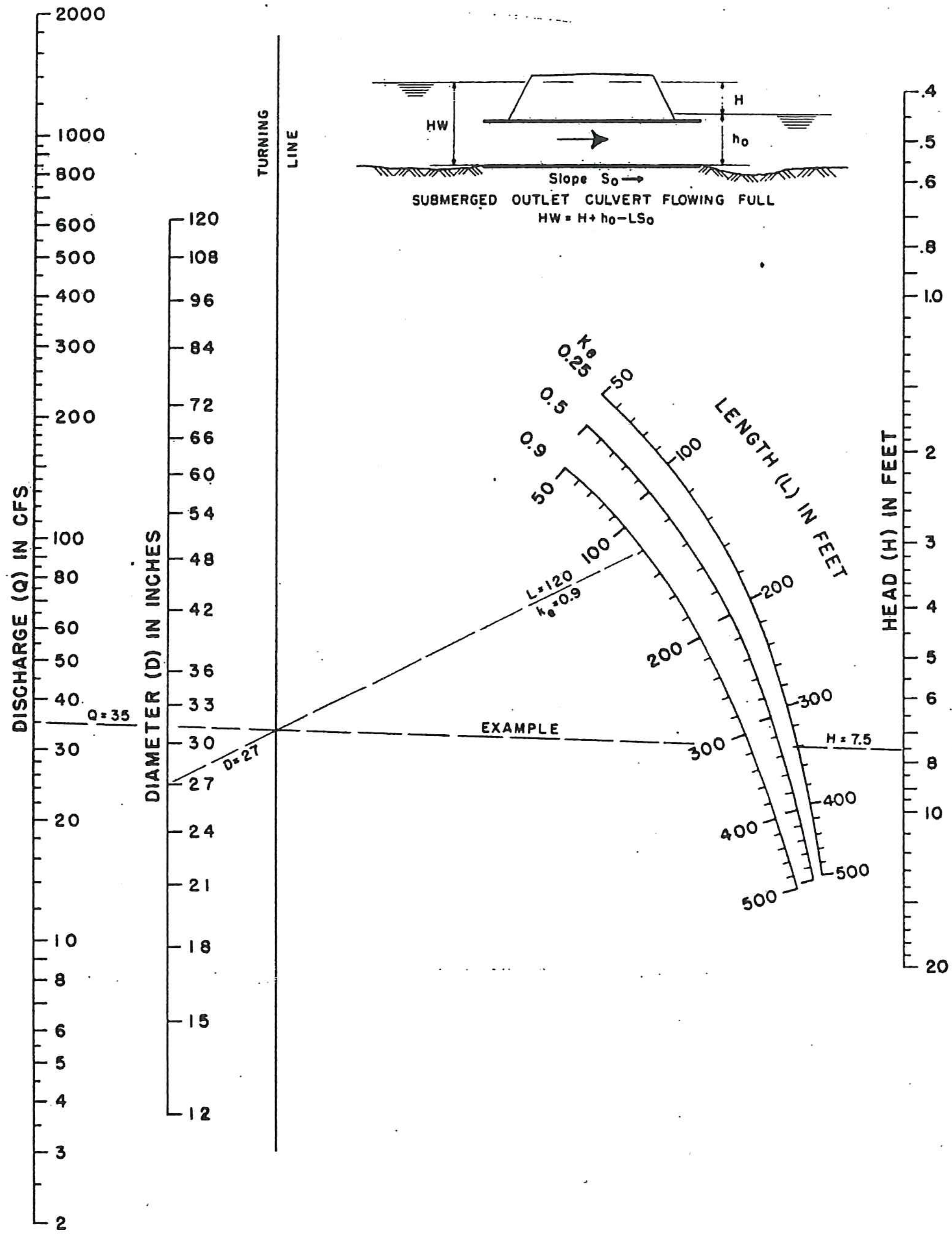
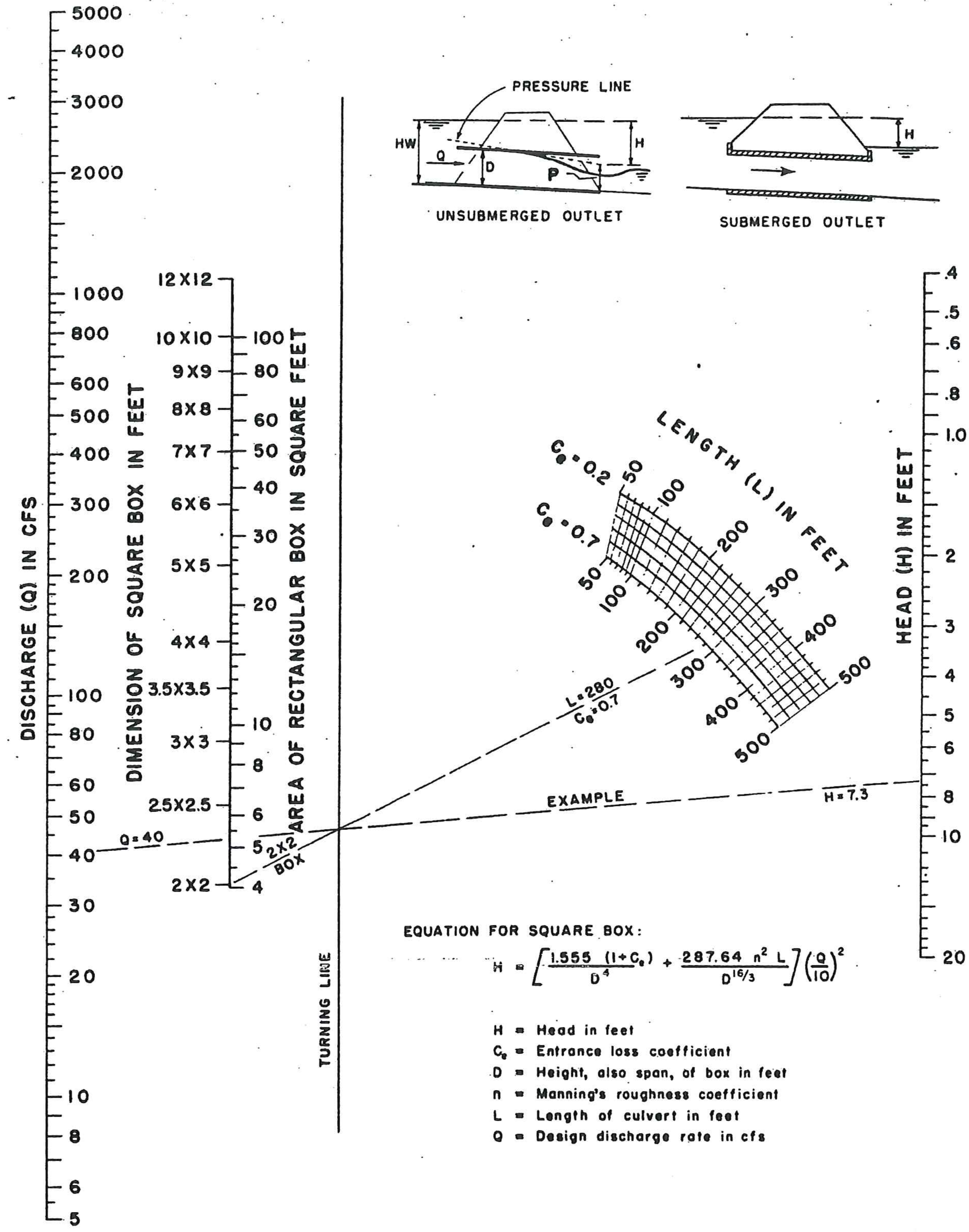


FIGURE 8-5
TOTAL HEAD LOSS FOR CORRUGATED METAL PIPE CULVERTS
WITH OUTLET CONTROL



EQUATION FOR SQUARE BOX:

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

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

FIGURE 8-6
TOTAL HEAD LOSS FOR CONCRETE BOX CULVERTS
WITH OUTLET CONTROL

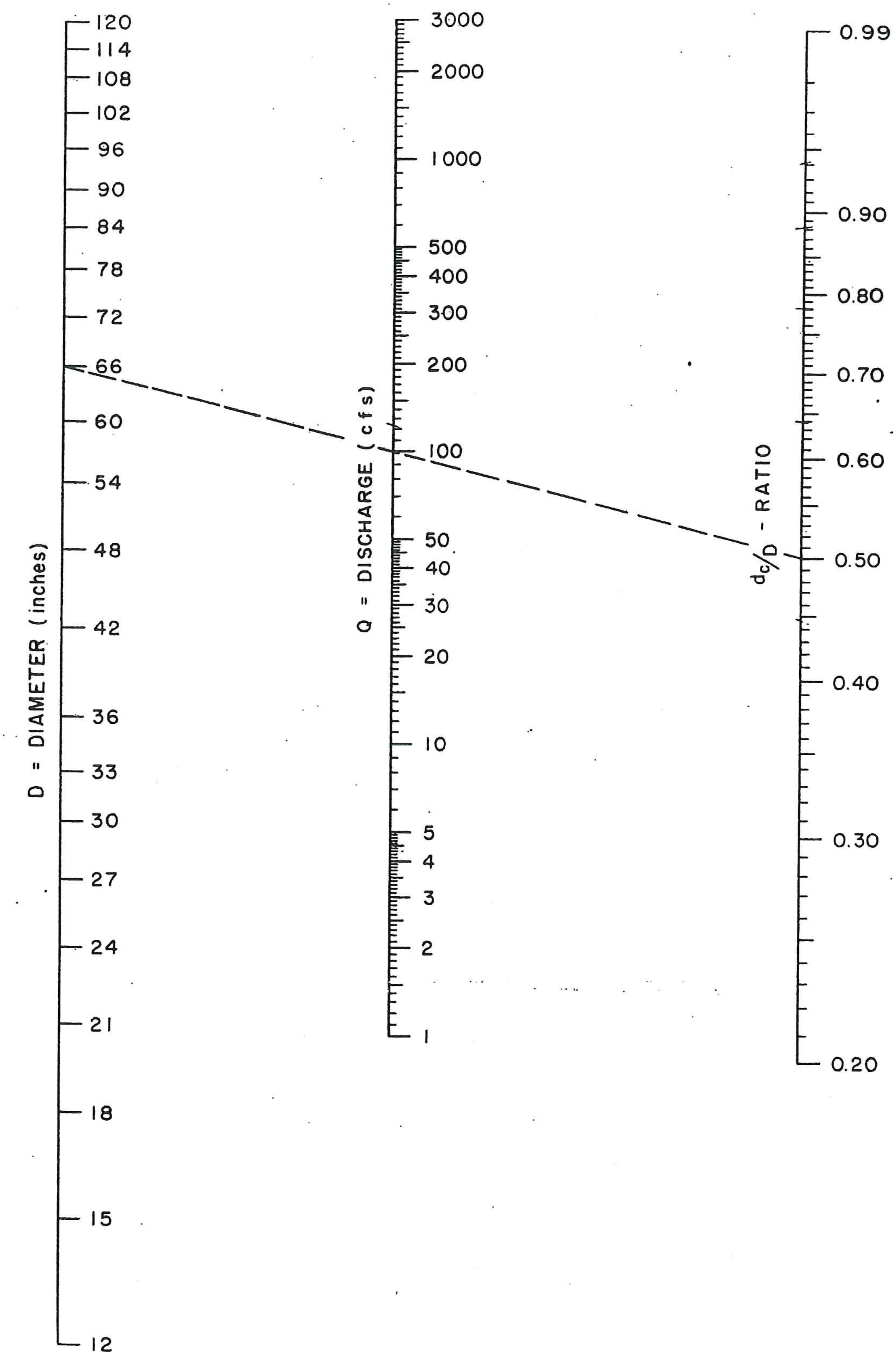


FIGURE 8-7
CRITICAL DEPTH FOR CIRCULAR CONDUITS

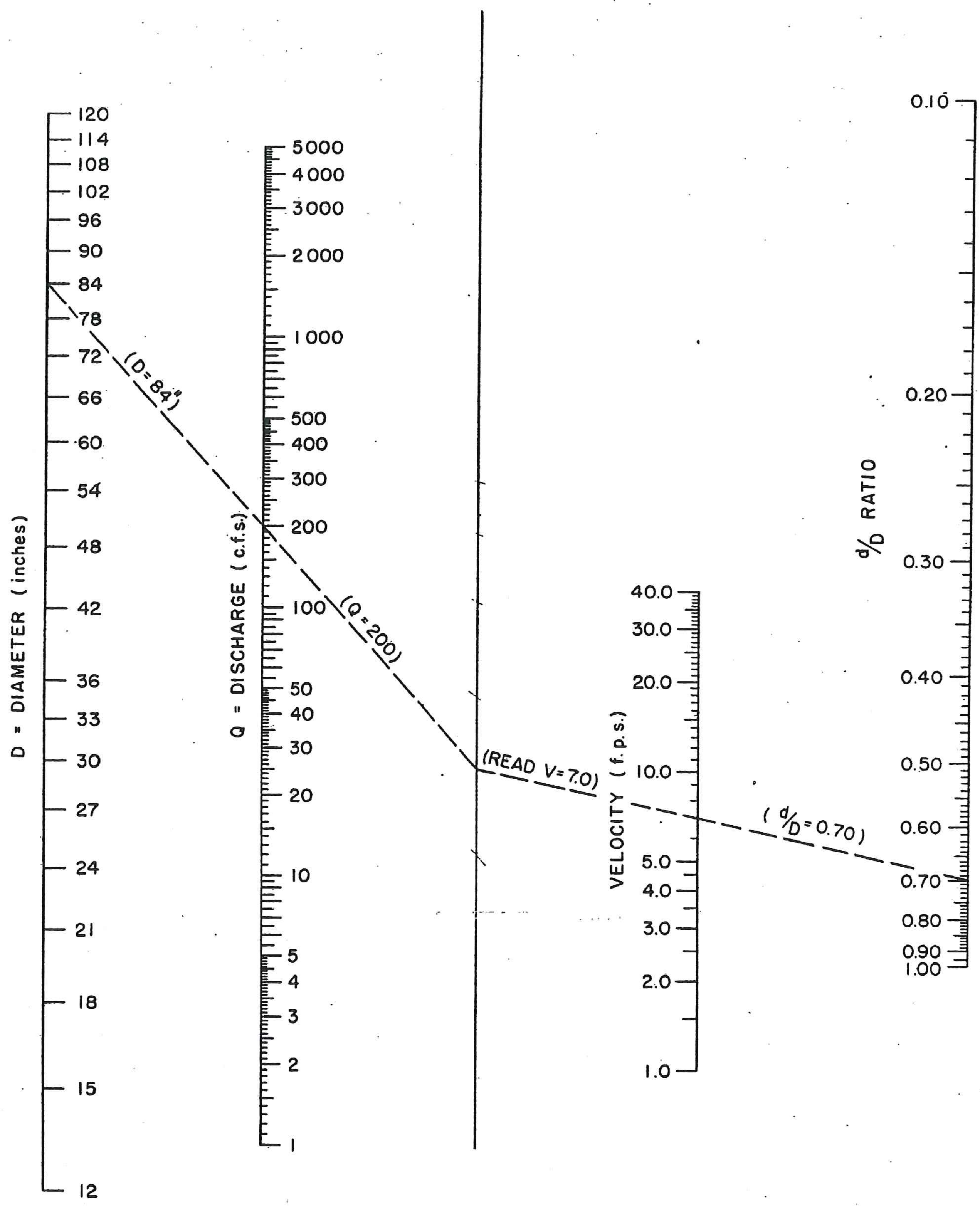
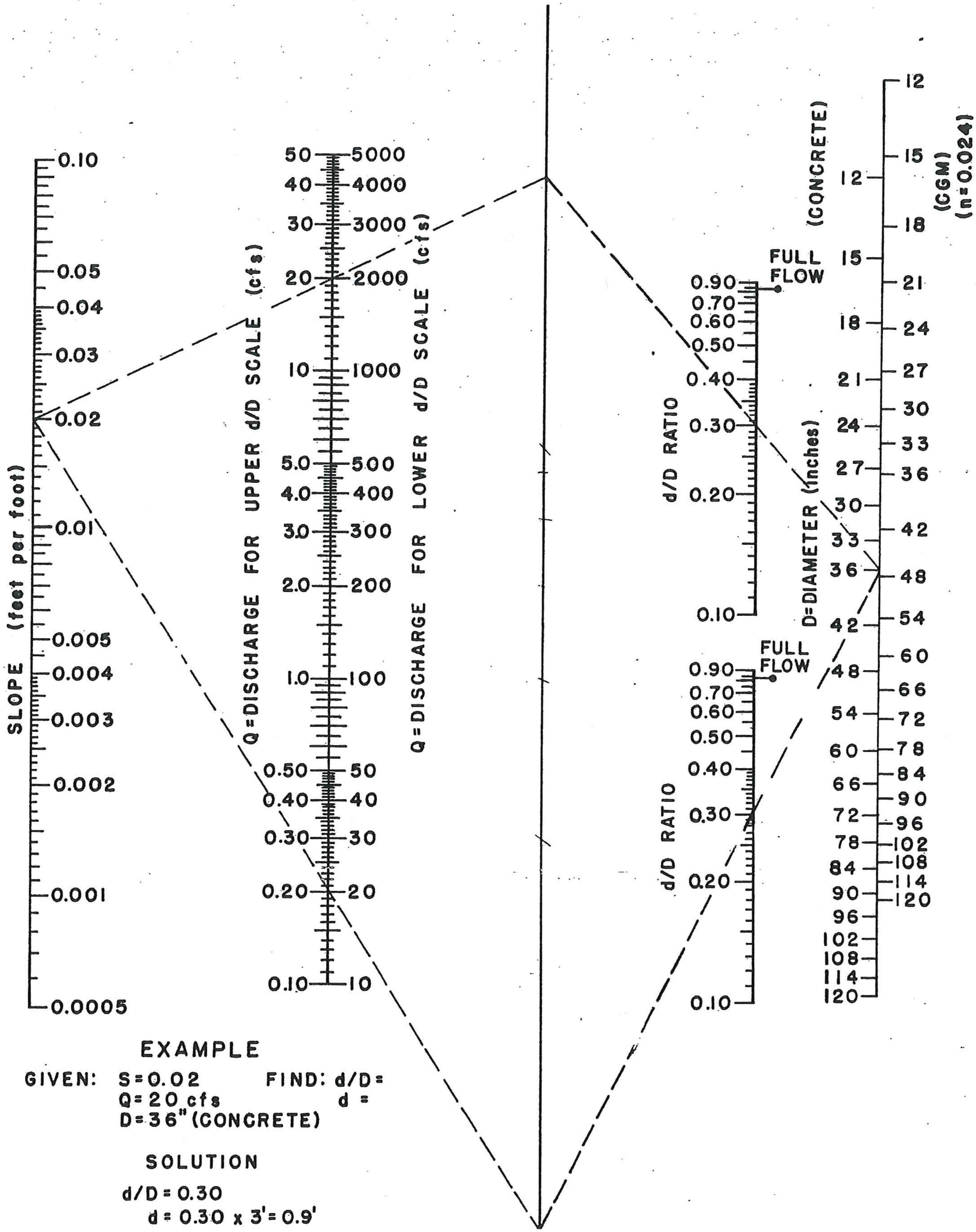


FIGURE 8-8
VELOCITY IN CIRCULAR CONDUITS



EXAMPLE

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

SOLUTION

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

FIGURE 8-9
UNIFORM FLOW IN CIRCULAR CONDUITS

PROJECT ___AMPLE 8-1___

DESIGN DISCHARGE (Q) ___200___ cfs

CULVERT LENGTH (L) ___100___ ft

CULVERT SLOPE (S) ___0.002___ ft/ft

CULVERT TY ___Concrete Box___

MANNING'S ROUGHNESS (n) ___0.012___

ENTRANCE TYPE ___90° wings___

ENTRANCE LOSS COEFFICIENT (Ke) ___0.5___

CULVERT SIZE	X-SECTIONAL AREA	HYDRAULIC RADIUS	INLET CONTROL		OUTLET CONTROL								SOLUTION			
							Tw < dc	D > Tw > dc	Tw > D			IF Hw>1.5D & Tw<dc THEN Hw = Hw + (D-dc)/2				
			HW/D	Hw	Tw	dc	ho=dc	ho=Tw	ho=Tw	H	Hw		CONTROL TYPE	Hw	V_{out}	COMMENTS
---	(ft²)	(ft)	---	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(in/out)	(ft)	(ft/s)	------
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
8 x 4	32	1.33	1.15	4.6	0	2.7	2.7	--	--	1.1	3.6	--	Inlet	4.6	8.0	Assume normal depth at outlet DN=3.11
			Figure 8-3		Free Outfall Conditions-No Backwater	Equation 8-8				Equation 8-5 or Figure 8-6	Equation 8-1					

EXAMPLE 8-1
CULVERT FLOWING WITH INLET CONTROL

PROJECT ___MPLE 8-2

DESIGN DISCHARGE (Q) 200 cfs

CULVERT LENGTH (L) 100 ft

CULVERT SLOPE (S) 0.002 ft/ft

CULVERT T Concrete Box

MANNING'S ROUGHNESS (n) 0.012

ENTRANCE TYPE 90° wings

ENTRANCE LOSS COEFFICIENT (Ke) 0.5

			INLET CONTROL		OUTLET CONTROL								SOLUTION			
							Tw < dc	D > Tw > dc	Tw > D							
CULVERT SIZE	X-SECTIONAL AREA	HYDRAULIC RADIUS	HW/D	Hw	Tw	dc	ho=dc	ho=Tw	ho=Tw	H	Hw	IF Hw>1.5D & Tw<dc THEN Hw = Hw + (D-dc)/2	CONTROL TYPE	Hw	V_{out}	COMMENTS
---	(ft^2)	(ft)	---	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(in/out)	(ft)	(ft/s)	------
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
8 x 4	32	1.33	1.15	4.6	5.0	2.7	--	--	5.0	1.1	5.9	--	Outlet	5.9	6.25	
			Figure 8-3			Equation 8-8				Equation 8-5 or Figure 8-6	Equation 8-1					

EXAMPLE 8-2
CULVERT FLOWING WITH OUTLET CONTROL

SECTION 9
BRIDGES

CONTENTS

9.1.0 GENERAL

Because a bridge is difficult and expensive to enlarge, all newly designed bridges should be sized based on the ultimate design flow. The low chord should be at least one (1) foot above the expected flood elevation, and the bridge should be designed to completely span the ultimate size channel. Bridges should be designed to maintain the same cross-sectional area and wetted perimeter, (with the exception of the support piers), as the ultimate upstream channel. Bents should be aligned parallel to the longitudinal axis of the channel so as to minimize their obstruction to the flow and should be placed as far away from the channel centerline as possible.

9.2.0 BRIDGE HYDRAULICS

The hydraulic analysis of a bridge is dependent on the headwater and tailwater depth at the bridge. If the tailwater depth is below the low chord the bridge may be analyzed assuming "low flow" conditions. If the tailwater depth is above the low chord and the headwater is below the bridge deck, the bridge opening operates as an orifice and should be analyzed as "pressure flow". If the headwater depth is above the bridge deck, there is both "pressure flow" and "weir flow".

9.2.1 Low Flow Conditions

Under low flow conditions, the total head loss through the bridge may be calculated based on Yarnell's equation as follows:

$$h_t = 2K(K+10w-0.6)(a+15a^4)V^2/(2g) \qquad \text{(Eq. 9-1)}$$

where,

h_t = Total head loss through the bridge in feet.
K = Coefficient for pier shape.
w = Ratio of velocity head to tailwater depth, $(V^2/(2gT_w))$.
a = Ratio of obstructed area to unobstructed area, (A_o/A_u).
V = Velocity of flow through bridge in feet per second.
g = Acceleration due to gravity, (32.2 ft/sec^2).
Tw = Tailwater depth in feet.
A_o = Obstructed area in square feet.
A_u = Unobstructed area in square feet.

Pier shape coefficients (K) for use in Equation 9-1 may be taken from the following table:

Pier Shape and Description	Coefficient (K)
Semicircular nose and tail	0.90
Twin-cylinder piers with connecting diaphragm	0.95
Twin-cylinder piers without diaphragm	1.05
90 degree triangular nose and tail	1.05
Square nose and tail	1.25

9.2.2 Pressure Flow Conditions

Under pressure flow conditions, the total head loss through the bridge may be calculated based on the orifice flow equation.

$$h_t = (Q/A_u)^2 k/(2g) \qquad \text{(Eq. 9-2)}$$

where,

h_t = Total head loss through the bridge in feet.
Q = Design flow rate in cubic feet per second.
A_u = Unobstructed area of bridge opening in square feet.
k = Total Loss Coefficient, (use 1.56).
g = Acceleration due to gravity, (32.2 ft/sec^2).

9.2.3 Combination Pressure Flow and Weir Flow

If the headwater depth calculated using Equation 9-2, is above the lowest point on the bridge deck or nearby roadway embankment, then there exists a combination of pressure flow below the bridge and weir flow over the roadway. To analyze a bridge under these conditions, the engineer must rewrite Equation 9-2 in the form,

$$Q_p = A_u(h_t k/(2g))^{1/2} \qquad \text{(Eq. 9-3)}$$

This equation, combined with the weir flow equation from Section 7 of this manual,

$$Q_w = CLH^{3/2} \qquad \text{(Eq. 9-4)}$$

where,

Q_w = Weir flow over roadway in cubic feet per second.
L = Length of submerged roadway in feet.
H = Vertical distance between headwater and roadway in feet.
C = Weir Coefficient of discharge, (use C=2.6 for bridge deck and C=3.0 for roadway approaches).

By assuming a headwater depth, calculating the pressure flow (Q_p) using Equation 9-3 and calculating the weir flow (Q_w) using Equation 9-4, the designer will be able to determine the proper headwater depth by trial and error.

9.3.0 EROSION CONTROL

For a properly sized bridge installation, erosion protection will usually not be necessary. In the case of an extremely wide upstream floodway, it may be necessary to plate the upstream entrance to the bridge or construct spur dikes to prevent flow from occurring parallel to the roadway embankment.

END OF SECTION

SECTION 10
SITE GRADING

CONTENTS

10.1.0 GENERAL

Due to the generally flat terrain in Port Lavaca, it is not economically feasible to convey the runoff from extreme rainfall events entirely through an underground sewer system. Local flooding and street ponding will occur in some areas because storm sewer pipes are not large enough to handle the infrequent, high intensity storms. For this reason, the engineer must analyze the street layout and site grading so as to assure that the runoff from an extremely high intensity storm will be conveyed to a primary drainage channel safely.

10.2.0 STREET OVERFLOW

When the capacity of an underground storm sewer system is exceeded and street ponding begins to occur, careful planning can reduce the flooding hazard to adjacent properties. Awareness of overflow problems in the early stages of planning is essential in developing a successful overflow system. Special attention must be given to existing topography, street layout, and pavement grades.

When evaluating the final design, the engineer should visualize the street system as not having any underground storm sewer. With this assumption, consider: (1) where the storm flow would go as ponding levels increase, (2) how deep the water would rise at one inlet before it begins to spill downhill to the next, and (3) how the overflow will ultimately reach the major drainage outfall. By analyzing the street system is this manner, the engineer will be able to identify potential problem areas and correct them. Three examples of potentially dangerous flow patterns are depicted in Figure 10-1 and include:

A. Cul-de-sac streets sloping downhill designed so that any overflow may only escape through building lots.

B. Placing a sharp curve in a roadway at a low area so that street overflow might be directed through building lots.

C. Several streets tying into one street on the low side of a hill so that any overflow is directed through adjacent lots.

Through foresight and careful planning of street layout and grades, many of these potential problems may be avoided. However, where site restrictions dictate such flow patterns, the engineer should include the necessary emergency spillways in the design. Addition storm sewer capacity can usually be provided by a surface swale or increased pipe and inlet capacity in the problem area. If surface swales are used, they should be concrete lined to prevent adjacent land owners from filling them in or fencing across them.

Recommended solutions to undesirable flow patters are depicted in Figure 10-2.

10.3.0 OFFSITE DRAINAGE

Sheet flow from an adjacent, undeveloped area into a proposed drainage system can create a localized flood hazard by overloading the inlets and flooding nearby lots. In addition, the relatively long time of concentration for an undeveloped tract may cause street ponding to continue for several hours after the storm. Any drainage plan submitted to the city for review must address the drainage of all adjacent property, both under undeveloped and fully developed conditions. Provisions must be made to divert all overland flow from adjacent property which currently drains across the proposed development.

Redirecting the overland flow can usually be achieved by the use of drainage swales located along the periphery of the development. These drainage swales should be placed in temporary drainage easements and later filled in as the adjacent area develops. Drainage swales should be relatively shallow, and the excavated soil should be placed continuously along the development side of the swale to prevent flow from over-running the ditch. The swale should have sufficient grade to prevent standing water, but not great enough to cause an erosion problem.

If the undeveloped area is to drain into the underground storm sewer system, additional inlets should be provided to intercept the flow prior to it reaching the street system. The additional flow, assuming ultimate development, shall be included in the design of an underground storm sewer system.

10.4.0 RESIDENTIAL BLOCK GRADING

Proper lot and block grading is an important aspect to any subdivision design. The objective is to establish the street grades, floor elevations, and lot grades in the proper relationship to each other and to the existing topography. A well graded site combines ease of maintenance and flood protection without sacrificing aesthetics. Two acceptable block grading techniques are described below.

10.4.1 Type 1 Block Grading

Type 1 block grading is the most commonly used grading scheme for residential subdivisions. A ridge is built up along the rear lot lines and each lot is sloped to drain surface water directly to the street. Type 1 grading is the most simple and desirable type of block grading because each lot is drained independent of any adjoining lots. The minimum slope from the rear lot line ridge to the top of curb in the street is 1.0 percent. Figure 10-3 shows the drainage pattern for Type 1 block grading.

10.4.2 Type 2 Block Grading

Although not as desirable as Type 1 block grading, Type 2 block grading is sometimes necessary for areas with a steep cross slope. With Type 2 block grading, the lots on the higher side of the block are allowed to drain to the rear where the runoff is collected in a drainage swale or alley. Drainage swales along the rear lot lines must be placed within permanent easements thereby assuring continuous access for maintenance. The construction of walls, buildings, and any other drainage obstructions, such as dense planting or tight fencing, must be legally prohibited in the easement area. The minimum longitudinal grade for such swales shall be 1.0 percent. Figure 10-4 illustrates the typical drainage pattern for Type 2 block grading.

10.5.0 RESIDENTIAL LOT GRADING

Providing protective slopes away from all sides of a building is an essential element in all lot grading techniques. The purpose of the slopes is to quickly drain roof water and other runoff away from the building. Where a protective slope intersects a slope draining towards the building, a drainage swale of adequate depth and width must be provided to carry away the surface water and prevent flooding of the building. The single most important grade relationship for proper lot drainage is the house slab elevation in relation to the street curb. If the slab elevation is too low, adequate protective slopes and drainage swales cannot be constructed. A slab elevation which is too high will result in unnecessary terracing, expensive outside stairs, and an awkward appearance. The minimum grade for protective slopes and drainage swales on a residential lot is 1.0 percent. The two acceptable lot grading types are discussed below.

10.5.1 Type A Lot Grading

In Type A lot grading, rear swales behind the building convey surface water from the rear yard to side yard swales which then carry the water to the street. Type A lot grading is used for all lots in an area with Type 1 block grading and is used on the low side lots for an area with Type 2 block grading. Figure 10-5 illustrates typical Type A lot grading.

10.5.2 Type B Lot Grading

In Type B lot grading, the yard is sloped both to the street and to the rear. This type of grading eliminates the need for a rear drainage swail behind the building and requires only side swales and protective slopes to drain the lot. The majority of the roof water should be directed to the abutting street thereby reducing the amount of water flowing to the rear lot line. Type B lot grading is used for the lots on the high side of an area with Type 2 block grading and for lots backing up to an open drainage ditch. Figure 10-6 shows typical Type B lot grading.

10.5.3 Minimum Residential Slab Elevation

As stated earlier, the single most important factor controlling proper lot drainage is the height of the finished slab above the adjoining street.

The minimum slab elevation for a residential structure varies depending on the lot and block grading type, lot size, setback, and building depth. Because of the many factors affecting slab height, the actual building elevation should be determined for each lot, or group of typical lots, to ensure adequate drainage. Formulae for calculating minimum slab elevations for typical Type A and Type B graded lots are given in Figures 10-5 and 10-6 respectively. However, regardless of the lot size or configuration, the finished floor of the habitable portion of the residence shall be elevated a minimum of 24 inches above the top of curb for Type A lot grading and for Type B graded lots abutting a drainage channel. For Type B graded lots, in an area with Type 2 block grading, the minimum house slab may be reduced to 15 inches above the top of curb. For residences abutting a strip paved street, the minimum slab height shall be measured from the pavement crown. In the case of corner lots or lots with double frontage, the higher of the abutting streets shall govern.

10.6.0 COMMERCIAL AND INDUSTRIAL LOT GRADING

10.6.1 Minimum Grades

All commercial and industrial lots shall be graded to provide positive drainage away from buildings and toward streets and/or storm drainage facilities. The following minimum slopes shall be utilized:

Surface Type	Minimum Slope
Unpaved Areas	1.0 percent
Asphalt Surfaces	1.0 percent
Asphalt Valleys	0.5 percent
Concrete Surfaces	0.4 percent
Concrete Valleys	0.2 percent

10.6.2 Minimum Slab Elevation

In any case the finished floor of any structure shall be a minimum of 12 inches above the top of curb, (or pavement crown for strip paved streets), along any perimeter street.

10.6.3 Internal Drainage

A. Small Parking Lots

Parking lots having ten (10) parking spaces, or less, may drain surface runoff directly onto the abutting street without installation of an underground storm sewer system.

B. Medium and Large Parking Lots

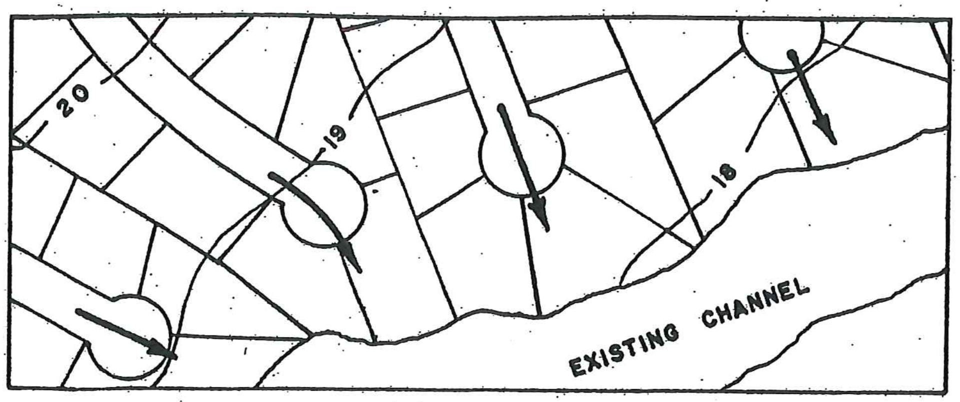
Parking lots having more than ten (10) parking spaces shall be allowed to drain a maximum of 3000 square feet of property onto the abutting street. Runoff from any additional area shall be collected on site, in an underground storm sewer system.

C. Exception to Internal Drainage Requirements

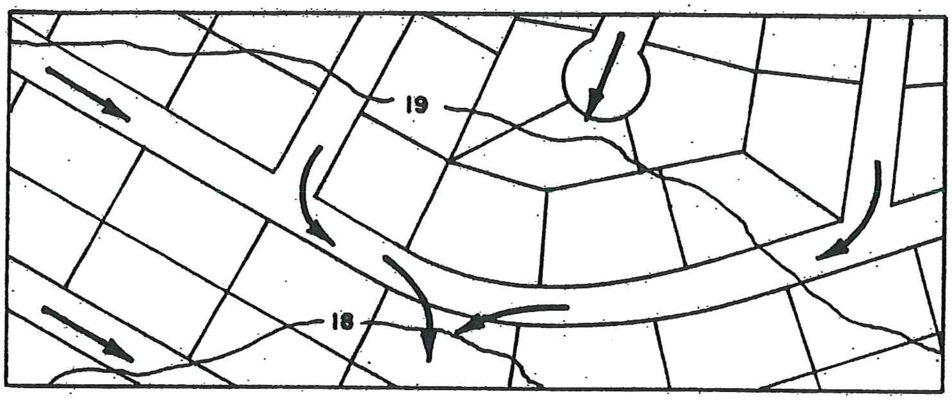
(1) In cases where there exists sufficient gutter capacity, medium and large parking lots shall be allowed to surface drain additional flow, above that allowed by Section 10.6.3 (B), to the street. The quantity of this flow shall be limited to the existing street capacity as determined by Section 4 of this manual.

(2) In cases where the City Engineer or Planning Board, finds it to be in the best interest of the City to allow surface type drainage, the requirement for internal drainage may be postponed or waived.

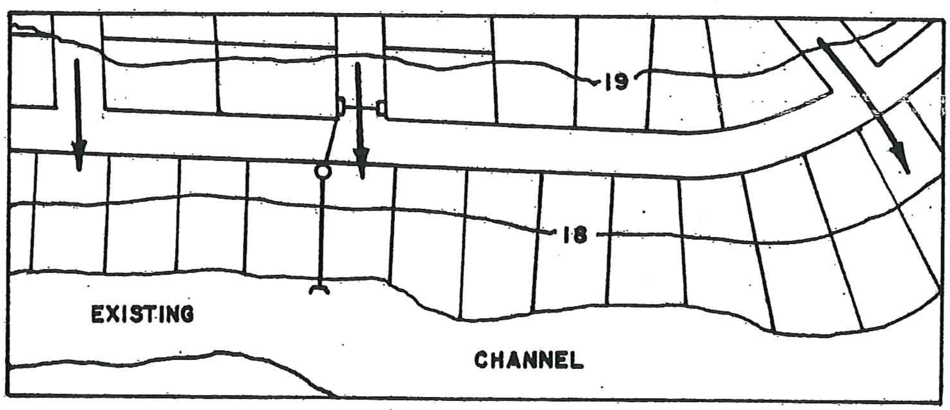
END OF SECTION



CUL-DE-SAC STREETS SLOPING DOWN HILL

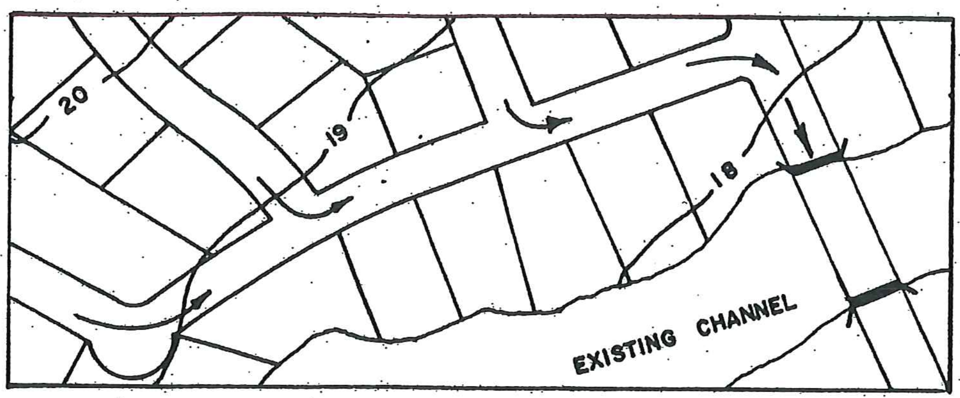


A CURVE OR TURN IN A ROADWAY LOCATED IN A LOW AREA

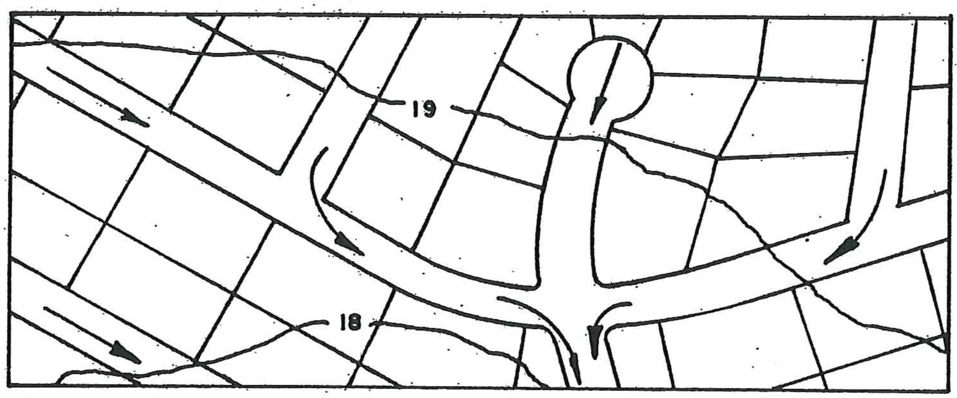


MANY STREETS "T"ING INTO A LOWER STREET
WITH NO EMERGENCY OVERFLOW PROVISIONS

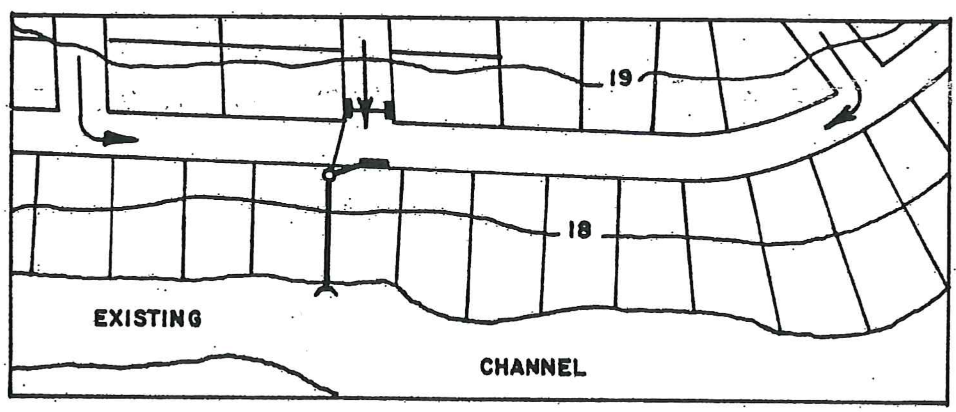
FIGURE 10-1
UNDESIRABLE FLOW PATTERNS



STREET USED TO CONVEY OVERFLOW TO EXISTING CHANNEL



STREET USED AS EMERGENCY OVERFLOW FOR LOW AREA



OVERSIZED INLET AND STORM SEWER LINE
USED AS EMERGENCY OVERFLOW

FIGURE 10-2
ACCEPTABLE FLOW PATTERNS

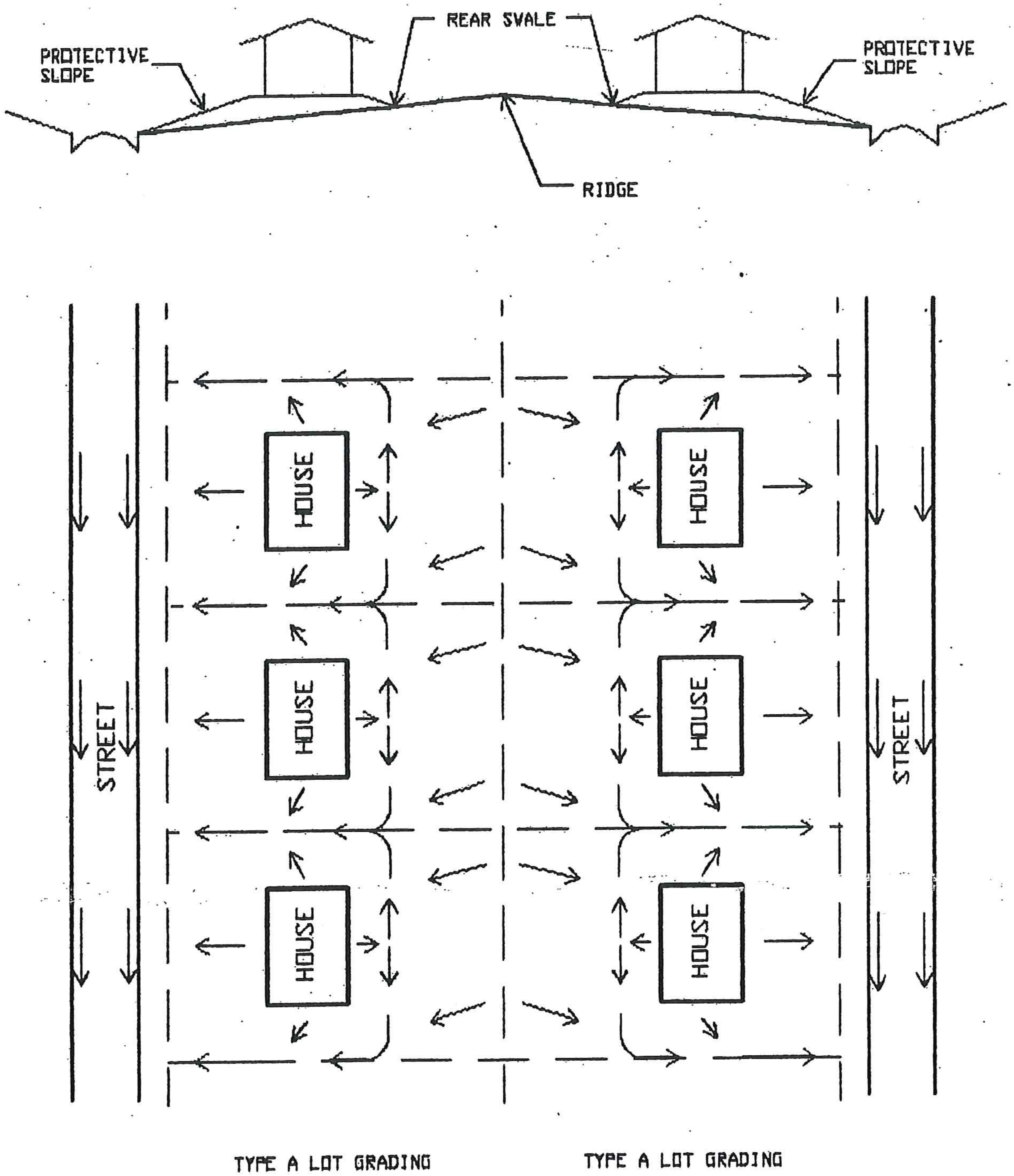


FIGURE 10-3
TYPE 1 BLOCK GRADING

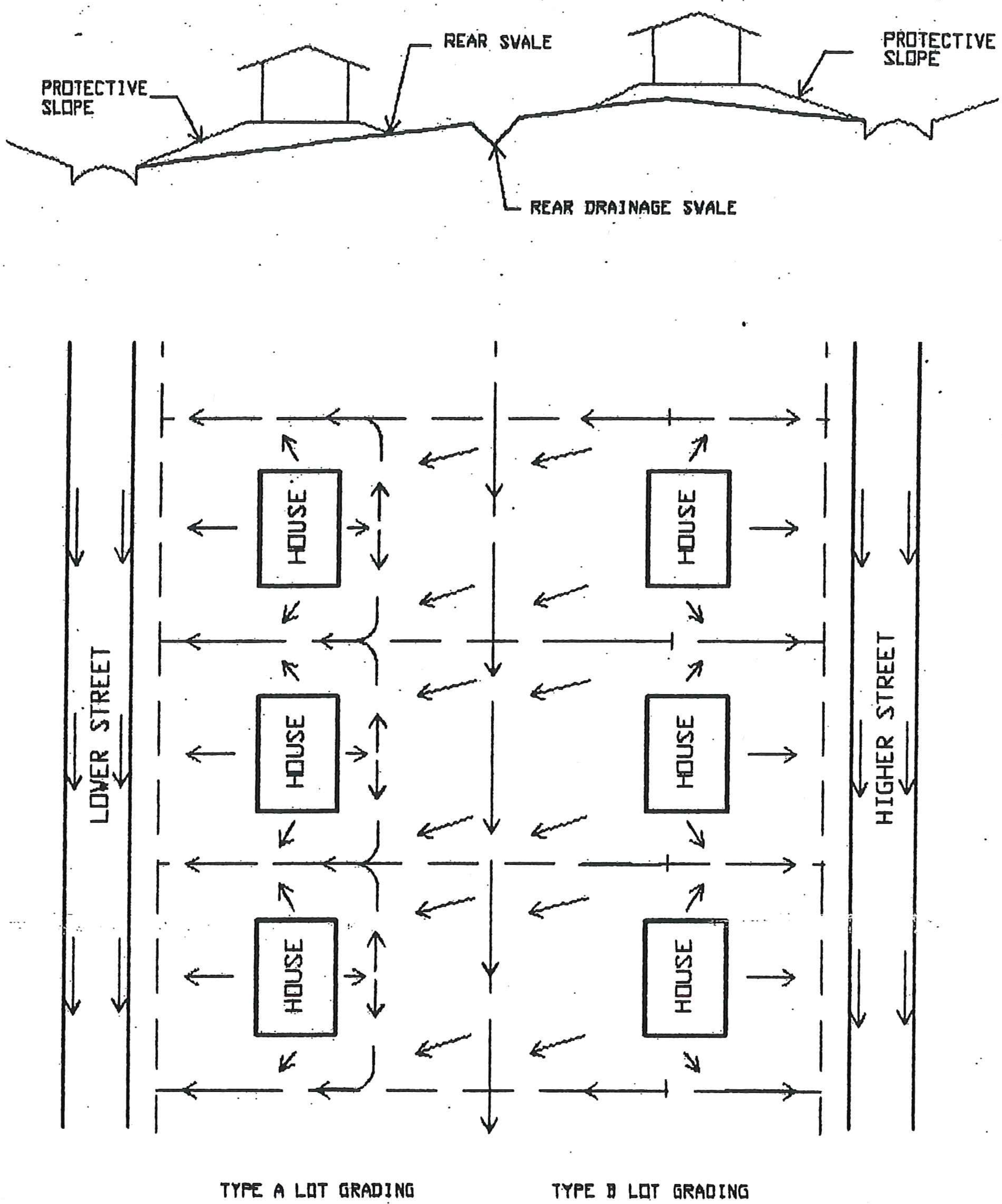
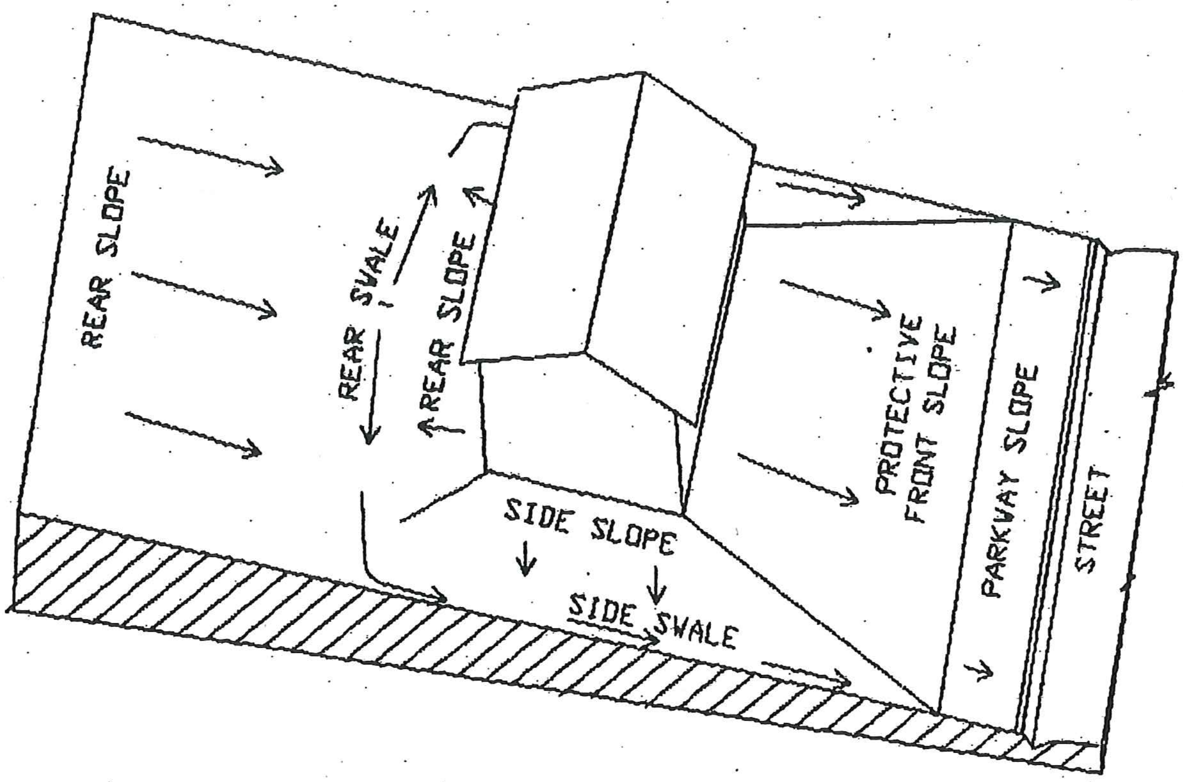


FIGURE 10-4
TYPE 2 BLOCK GRADING



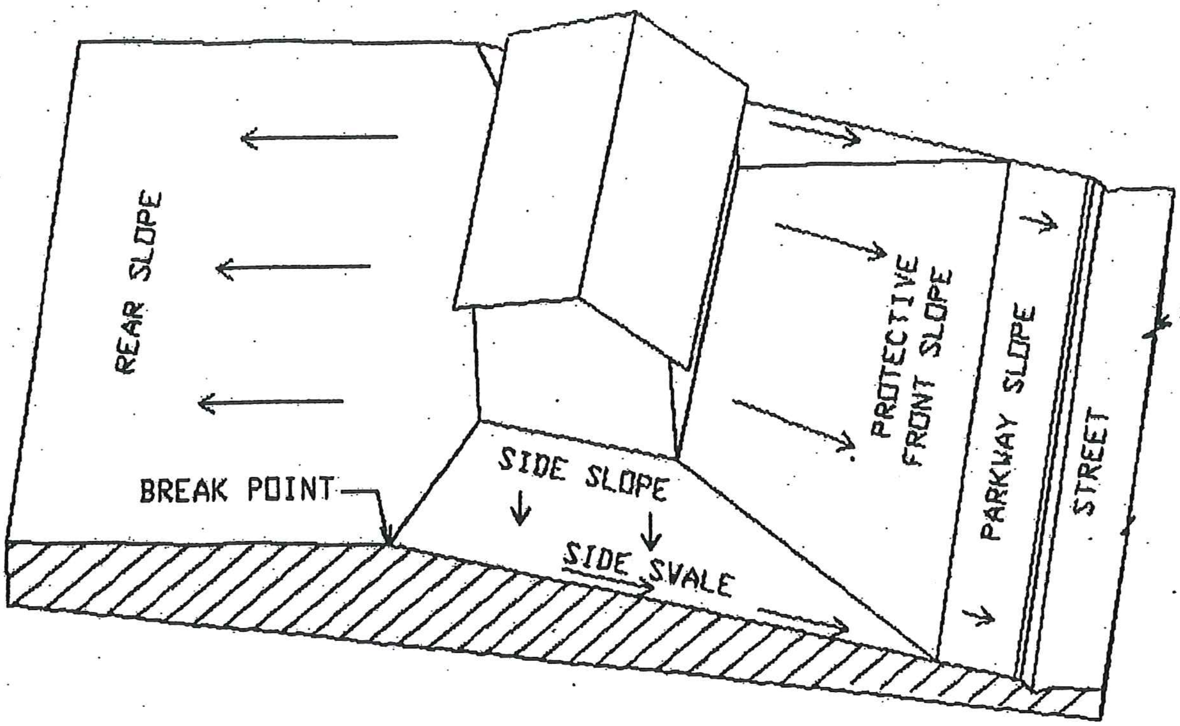
MINIMUM LOT GRADES

PARKWAY SLOPE	2.0%
SIDE SWALE	1.0%
REAR SWALE	1.0%
REAR SLOPE	3-6 in.
RISE TO SLAB AT REAR OF HOUSE	6 in.

MINIMUM SLAB ELEVATION = TOP OF CURB + PARKWAY x 0.02 + (BLDG. SET BACK + HOUSE DEPTH + 25 FT.) x 0.01 + 0.75 FT.

(SHALL NOT BE LESS THAN 24 INCHES ABOVE TOP OF CURB)

FIGURE 10-5
TYPE A LOT GRADING



NOTE:
BREAK POINT MAY BE LOCATED NO CLOSER TO THE STREET THAN THE FRONT OF THE HOUSE.

MINIMUM LOT GRADES

PARKWAY SLOPE	2.0%
SIDE SWALE	1.0%
REAR SLOPE	1.0%
SIDE SLOPE	3-6 In.
RISE TO SLAB	6 In.

MINIMUM SLAB ELEVATION = TOP OF CURB + PARKWAY x 0.02 + DISTANCE TO BREAK POINT X 0.01 + 0.75 FT.

(SHALL NOT BE LESS THAN 24 INCHES ABOVE TOP OF CURB)

(REFER TO SECTION 10.5.3 FOR EXCEPTIONS)

FIGURE 10-6
TYPE B LOT GRADING

SECTION 11
STORM WATER DETENTION

CONTENTS

11.1.0 GENERAL

In an area such as Port Lavaca, which is characterized by flat terrain, the introduction of impervious cover and improved runoff characteristics serves to increase the flood peaks quite dramatically. When physical, topographic, and economic conditions allow it, channel improvements downstream of the development are usually used to prevent increased flooding. However, in situations where downstream channel improvements are not currently feasible, on-site storage of storm water runoff is the most effective way to allow development of properties without increasing the flood potential downstream. This section of the manual presents background information of storm water storage techniques and detailed guidelines and criteria for the design of storm water storage facilities.

11.2.0 STORAGE CLASSIFICATION

Storage systems are specifically designed for either retention or detention of storm water. Retention storage applies to the concept of collecting and storing runoff for an extended period of time. The runoff is only released after the storm has ended and the downstream water surface has subsided. When detention storage is used, the storm runoff is released continuously throughout the storm, but at a very slow rate, thereby reducing the peak flow downstream of the facility. This is accomplished by controlling the maximum discharge with a flow-limiting outfall structure.

The physical features of a particular site, as well as the type of development proposed, will often dictate which type of storage facility may be utilized. Since detention facilities are most often designed to remain dry when not in use, they can provide dual purpose functions such as parking lots and recreational areas.

11.3.0 STORAGE DETERMINATION

Much of the basic methodology currently used for sizing a storm water detention facility centers around the Rational Formula. This is the same equation used to calculate the peak rate of runoff in Section 3.4.0 of this manual. However, since this method yields only the peak flow rate, various modifications have been made to the rational method in order to generate a complete runoff hydrograph and thus approximate actual conditions. One of the easiest to use procedures for generating a runoff hydrograph is the Modified Rational Method.

11.3.1 Modified Rational Method

The modified rational method is derived from the rational method previously discussed in Section 3.4.0. This procedure determines the critical storm duration which will produce the largest pond storage requirement. To apply the modified rational method, the engineer develops a straight-line inflow hydrograph that is a crude approximation of the runoff hydrograph for a storm of a given duration. The release rate, or outflow hydrograph for the pond is then superimposed onto the graph and the area between the outflow and inflow hydrographs is equal to the required storage. This process is repeated for a range of storm durations to determine the maximum required volume.

A. Inflow Hydrograph

For each storm duration, the peak flow rate is calculated by the basic Rational Method, (Section 3.4.0), except that the intensity used is that for the time of storm duration rather than the time of concentration of the watershed. The rising limb of the hydrograph is a straight line from zero (0) to the peak flow rate, extending over a period equal to the proposed time of concentration. A straight line then continues across the graph at the peak flow rate until the storm duration time has been reached. The falling limb of the hydrograph is a mirror image of the rising limb.

B. Outflow Hydrograph

The shape of the outflow hydrograph is dependent upon the size, type, and capacity of the pond outfall. Three variations of the outflow hydrograph are discussed below.

(1) Retention Ponds

As stated earlier, the water from a retention type facility is not release until after the storm has subsided, therefore, the outflow hydrograph is equal to zero. For a retention type facility, the entire volume of runoff defined by the inflow hydrograph must be stored.

(2) Variable Rate Orifice and Weir Outfalls

The outflow hydrograph from a pond with a variable rate orifice or weir outfall may be approximated by a straight line beginning at zero and extending to a point on the falling limb of the inflow hydrograph which is equal to the maximum allowable release rate.

(3) Pumped Outfalls

The outflow hydrograph for a detention pond with a pumped outfall may be approximated by a straight line beginning at zero and increasing to the maximum release rate over a period of time equal to the proposed time of concentration. The outflow hydrograph then continues horizontally at the maximum allowable release rate across the rest of the graph.

C. Maximum Allowable Release Rate

The maximum allowable release rate from a storm water storage facility shall be controlled by the capacity of the downstream drainage system. If no downstream improvements are to be made, then the maximum release rate shall be equal to the current, undeveloped, runoff rate.

D. Limitations Of The Modified Rational Method

1. The maximum contributing watershed shall be less than 100 acres.

2. All off-site flows shall be diverted around the pond.

3. Outflow hydrographs produced by the modified rational method shall not be used as inflow hydrographs for another pond. (i.e. cascading ponds cannot be analyzed using the modified rational method.)

4. On-site flows which do not enter the pond are referred to as bypass flow and shall be subtracted from the allowable peak flow release rate for the development.

11.3.2 HEC-1 Flood Hydrograph Method

In cases where the detention facility does not fall within the limitations stated above, (i.e. drainage area greater than 100 acres, offsite flow entering the pond, or cascading ponds), the modified rational method is not applicable and a more detailed analysis is required. The recommended method of analysis for these situations is the HEC-1 Flood Hydrograph Program developed by the U.S. Army Corps of Engineers. The recommended technique for reservoir routing in the Modified Puls Method. In order to route the inflow hydrograph through the detention facility using the HEC-1 model, a relationship must be established between the volume of storage in the pond and the corresponding amount of discharge through the outflow structure. In most cases in Port Lavaca this relationship is directly dependent on the elevation of the tailwater at the outlet of the outflow structure. For the purposes of establishing an outflow rating curve, the tailwater in the receiving channel shall be assumed to be at all times at the level of the 25 year storm.

11.4.0 PUMPED DETENTION SYSTEMS

Pumped detention systems shall be allowed as a temporary solution only. The facility must be designed such that the pumps will not be necessary after the ultimate downstream channel improvements have been completed. In addition, pump detention facilities shall only be approved for use under the following conditions:

1. A gravity system is not feasible from an engineering or economic

standpoint.

2. At least two (2) pumps are provided, each of which is sized to pump the design flow rate. If more than two (2) pumps are used, any two (2) must be capable of pumping the design flow rate.

3. The selected design outflow rate must not aggravate downstream flooding. (Example: A pump system designed to discharge at the undeveloped 25 year flow rate each time the system comes on-line could increase downstream flooding for short duration storms.

4. Fencing of the control panel is provided to prevent unauthorized operation and vandalism.

5. A source of funding is provided to operate and maintain the pumps on a continuous basis.

6. An emergency power supply is provided. (An alternative to the emergency power supply would be to size the system as a retention pond with sufficient storage to operate safely without power.)

It is recommended that if a pump system is desired, preliminary review by the City Engineer be obtained prior to any detailed engineering analysis.

11.5.0 EROSION CONTROL

Storm water detention and retention facilities shall be constructed according to the same minimum requirements set out in Section 7 for open channels.

The maximum side slope for earth embankments shall be 3 to 1, (i.e. three (3) feet horizontally for every one (1) foot vertically). For concrete plated sides the maximum side slope shall be 2 to 1 with an optional four (4) foot deep pilot channel sloped at 1 to 1.

In cases where overland flow would otherwise flow over unprotected side slopes, a backslope drainage system shall be required and shall be designed in accordance with Section 7.8.4 of this manual.

11.6.0 REQUIRED EASEMENTS

All storm drainage detention and retention facilities shall be place within a dedicated drainage easement. The easement shall be of sufficient size to contain the pond under design flow, backslope drainage system, and provide unobstructed maintenance access to the pond.

END OF SECTION

LIST OF VERTICAL BENCHMARKS

LOCATION	DESCRIPTION	ELEVATION
Barton Ditch		
Virginia Street	"X" NW corner of headwall	16.49
Sand Crab Blvd.	"X" NW corner of headwall	19.11
Half League Road	"X" NE corner of headwall	19.21
HWY 35 East Bound	Disc NE corner of headwall	18.78
HWY 35 West Bound	"X" NW corner of headwall	19.31
Corporation Ditch		
Alcoa Drive	"X" top of NW wingwall	10.10
Austin Street	"[]" N corner of headwall	19.47
Harbor of Refuge Ditch		
FM 1090	"X" NW corner of headwall	8.84
Little Chocolate Bayou		
HWY 35	"X" corner of N headwall	19.85
HWY 87	"X" corner of N headwall	19.48
Austin Street	"X" N corner of headwall	13.82
Lynn's Bayou		
Oakglen Drive	"X" SW corner of headwall	14.31
Commerce Street	"X" NW corner of bridge	12.84
Broadway Street	Disc NW corner of bridge	12.85
Village Road Ditch		
Broadway	"X" center of N headwall	17.46
HWY 35	"X" center of S headwall	18.70
West Branch Lynn's Bayou		
Six Mile Road	"[]" SW corner of headwall	20.72
Wilson Ditch		
HWY 87	"X" NW corner of headwall	19.37
HWY 238	"X" NW corner of headwall	15.45
HWY 35	Disc N corner of headwall	18.05

PROJECT NAME ______________________

SHEET ______ OF ______

STORM SEWER LINE ______________________

DESIGN FREQUENCY ______________________

DETERMINATION OF RUNOFF						ESIGN				COMMENTS
DESIGN POINT	DRAINAGE AREA		TIME OF CONCENTRATION		RUN COE (C	SSES			HYDRAULIC GRADE LINE ELEVATION	
	SUB-AREAS	TOTAL AREA	TOTAL TIME	CONDUIT TIME		XIT	BEND	TOTAL		
---	------	(acres)	(minutes)	(minutes)	---	ft)	(ft)	(ft)	(ft)	
1	2	3	4	5	6	21	22	23	24	25

1. Design point designation.
   (Note: Begin calculations at the point farthest from t
2. Drainage sub-area designation for all areas contributin
3. Total drainage area contributing flow to the design poi
4. Time of concentration to the design point, column 4 plu
5. Time along conduit, column 18 divided by column 10. (Se
6. Runoff coefficient. (Section 3.4.1 and Table 3-1).
7. Rainfall intensity. (Section 3.4.2.B).
8. Runoff per acre, column 6 times column 7 or Table 3-2 a

t, column 9 divided by column 12.
st be maintained between 2 and 10 ft/s.)
6.4.1).
6.4.2).
3).
4).
columns 19, 20, 21, and 22.
evation, column 24 plus column 23.
ade line elevations are calculated beginning at the down stream
or previously calculated water surface elevation.)

PROJECT ______________________

DESIGN DISCHARGE (Q) ______________ cfs

CULVERT LENGTH (L) ______________ ft

CULVERT SLOPE (S) ______________ ft/ft

CULVERT TYPE ______________

MANNING'S ROUGHNESS (n) ______________

ENTRANCE TYPE ______________

ENTRANCE LOSS COEFFICIENT (Ke) ______________

CULVERT SIZE	X-SECTIONAL AREA	HYDRAULIC RADIUS	INLET CONTROL		OUTLET CONTROL								SOLUTION			
							Tw < dc	D > Tw > dc	Tw > D							
			HW/D	Hw	Tw	dc	ho=dc	ho=Tw	ho=Tw	H	Hw	IF Hw>1.5D & Tw<dc THEN Hw = Hw + (D-dc)/2	CONTROL TYPE	Hw	V_{out}	COMMENTS
---	(ft²)	(ft)	---	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(in/out)	(ft)	(ft/s)	------
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17

FIGURE A-3
CULVERT DESIGN WORK SHEET

PROJECT ______________________

DESIGN DISCHARGE (Q) ______________ cfs

CULVERT LENGTH (L) ______________ ft

CULVERT SLOPE (S) ______________ ft/ft

CULVERT TYPE ______________

MANNING'S ROUGHNESS (n) ______________

ENTRANCE TYPE ______________

ENTRANCE LOSS COEFFICIENT (Ke) ______________

			INLET CONTROL		OUTLET CONTROL								SOLUTION			
CULVERT SIZE	X-SECTIONAL AREA	HYDRAULIC RADIUS	HW/D	Hw	Tw	dc	Tw < dc ho=dc	D > Tw > dc ho=Tw	Tw > D ho=Tw	H	Hw	IF Hw>1.5D & Tw<dc THEN Hw = Hw + (D-dc)/2	CONTROL TYPE	Hw	V_{out}	COMMENTS
---	(ft^2)	(ft)	---	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(in/out)	(ft)	(ft/s)	------
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17

FIGURE A-3
CULVERT DESIGN WORK SHEET

GLOSSARY

ABUTMENT - A wall supporting the end of a bridge or span, and sustaining the pressure of the abutting earth.

APRON - A floor or lining of concrete, timber, or other suitable material located at the inlet or discharge side of hydraulic structures box culverts, spillways, etc., designed to protect the waterway from erosion from falling water or turbulent flow.

BACKWATER - The rise of the water level upstream due to an obstruction or constriction in the channel.

BACKWATER CURVE - The term applied to the longitudinal profile of the water surface in an open channel when flow is steady but non-uniform.

CHANNEL ROUGHNESS - The estimated measure of texture at the perimeters of channels and conduits. Usually represented by the Manning coefficient "n" used in the Manning Equation.

CONDUIT - Any open or closed device for conveying flowing water.

CRITERIA - A standard or rule on which a judgment or decision is based.

CRITICAL FLOW - The state of flow for a given discharge at which the specific energy is a minimum with respect to the bottom of the conduit.

CRITICAL SCOPE - The minimum slope of a conduit which will produce critical flow.

CROWN - (1) The highest point on a transverse section of a conduit, (2) the highest point of a roadway cross section.

CULVERT - Pipe of other conduit through which flow passes under a road or driveway.

CURB - A vertical or sloping structure located along the edge of a roadway, normally constructed integrally with the gutter, which strengthens and protects the pavement edge and clearly defines the pavement edge to vehicle operators.

DAM - A barrier constructed across a watercourse for the purpose of either temporarily or permanently impounding water.

DESIGN STORM or FLOOD - The storm or flood which is used as the basis for design, i.e., against which the structure is designed to provide a stated degree of protection or other specified result.

DETENTION - The storage of storm runoff for a controlled release during and immediately following the design storm. The term "detention" is often used to describe both detention and retention facilities.

DISCHARGE (Q) - Rate of flow usually expressed in cubic feet per second.

DRAINAGE AREA - The area contributing storm runoff to a stream or drainage system at a particular point.

DROP STRUCTURES - The function of a drop structure is to reduce channel velocities by allowing for flatter upstream and downstream channel slopes.

EMPIRICAL EQUATION - A mathematical relation which is based on measured prototype or laboratory data.

ENERGY GRADIENT LINE - A line representing the energy in flowing water. The elevation of the energy line is equal to the summation of elevation of the flow line plus the depth plus the velocity head plus the pressure head.

ENTRANCE LOSS - Head lost in eddies or friction at the inlet to a conduit, headwall or structure.

EROSION - Wear or scouring in a channel, opening, or outlet structure caused by hydraulic action.

EXIT LOSS - Head lost due to eddies, friction, a loss in velocity head at the exit of a conduit.

FLOOD CONTROL - The elimination or reduction of flood losses by the construction of flood storage reservoirs, channel improvements, dikes and levees, by-pass channels, or other engineering works.

FLOODPLAIN - Geographically the entire area subject to flooding.

FREEBOARD - The distance between the calculated water surface elevation and the maximum physical elevation of the channel or pond, which is provided as an additional factor of safety.

FREQUENCY (of storms, floods) - Average recurrence interval of events, over long periods of time. Mathematically, frequency is the reciprocal of the exceedance probability.

FRICTION SLOPE - The friction head or loss per unit length of channel or conduit. For uniform flow the friction slope coincides with the energy gradient, but where a distinction is made between energy losses due to bends, expansions, impacts, etc., a distinction must also be made between the friction slope and the energy gradient. The friction slope is equal to the bed or surface slope only for uniform flow in uniform open channels.

GABION - A wire basket containing rocks which is placed uniformly with others to provide protection against erosion.

GRADE - The inclination or slope of a channel, conduit, or natural ground surface, usually expressed in terms of the ratio of vertical rise to horizontal distance.

GUTTER - A shallow concrete waterway adjacent to a curb for conveying street flow.

HEAD - The height of water above any datum.

HEADWALL - The normal functions of properly designed headwalls and endwalls are to anchor the culvert in order to prevent movement due to hydraulic and soil pressures, to control erosion and scour resulting from excessive velocities and turbulence and to prevent adjacent soil from sloughing into the waterway opening.

HEADWATER - (1) The upper reaches of a stream near its sources; (2) the region where ground waters emerge to form a surface stream; (3) the headwater depth on the upstream side of a structure. (See Entrance Head)

HYDRAULIC GRADIENT - A hydraulic profile of the piezometric level of the water, representing the sum of the depth of flow and the pressure head. In open channel flow it is the water surface.

HYDRAULIC JUMP - The hydraulic jump is an abrupt rise in the water surface which occurs in an open channel when water flowing at supercritical velocity transitions to subcritical velocity. The transition through the jump results in a marked loss of energy, evidenced by turbulence of the flow within the area of the jump. The hydraulic jump is sometimes used as a means of energy dissipation.

HYDRAULIC RADIUS - The cross-sectional area divided by the wetted perimeter. Often, the hydraulic radius approximates the average depth of flow in a channel section.

HYDRAULICS - A branch of science that deals with practical applications of the mechanics of water movement.

HYDROGRAPH - A graph or table showing discharge versus time at a given point on a stream or conduit.

a. Synthetic Hydrograph - Runoff or unit hydrographs which are devised by empirical means (as opposed to derivation based upon natural, measured data).

b. Unit Hydrograph - The direct runoff hydrograph resulting from one inch of precipitation excess distributed uniformly over a watershed for a specified duration.

HYDROLOGY - The science that deals with the processes governing the depletion and replenishment of the water resources of the earth.

HYETOGRAPH - A histogram or graph of rainfall intensity versus time for a storm.

IMPERVIOUS - A term applied to a material through which water cannot pass, or through which water passes with great difficulty.

INFILTRATION - The absorption of water by the soil, either as it falls as precipitation, or from a stream flowing over the surface.

INLET - The inflow point for a storm sewer system which is usually associated with streets (e.g., curb opening inlets, grate inlets, etc.).

INTENSITY - The rate of accumulation of rainfall, usually in inches per hour.

INVERT - The floor, bottom, or lowest portion of the internal cross section of a conduit. Used particularly with reference to sewers, tunnels, and drains.

MANNING COEFFICIENT - The coefficient of roughness used in the Manning Equation.

MAY - A permissive condition. Not required.

ORIFICE - An opening with a closed perimeter and regular form, through which water may flow.

OVERLAND FLOW - Runoff which is not considered concentrated.

PEAK FLOW - The maximum rate of flow past a particular point for a given storm.

RAINFALL DURATION - The length of time over which a discrete rainfall event lasts.

GLOSSARY
(continued)

RAINFALL FREQUENCY - The average recurrence interval of rainfall events, averaged over long periods of time.

RAINFALL INTENSITY - The rate of accumulation of rainfall, usually in inches per hour.

RATIONAL FORMULA - A Traditional means of relating runoff from an area and the intensity of the storm rainfall, (Q=CIA).

RECOMMENDED - A condition which should be met if it is physically and economically reasonable to do so.

REGIONAL ANALYSIS - A study which has produce regional regression equations relating various watershed and climatological parameters to discharge.

REQUIRED - This is a mandatory condition. Where certain requirements in the design or application of the guidelines are described with the "required" stipulation, it is mandatory that they be met.

RETENTION - The storage of storm runoff for a control release immediately following the design storm.

RIPRAP - Forms of bank protection. usually using concrete or rock. Riprap is a term applied to material which is dumped rather than placed more carefully.

RURAL WATERSHED - A watershed unaffected by urban development.

RUNOFF - That part of the precipitation which reaches a channel, drain or sewer.

RUNOFF COEFFICIENT (C) - A decimal number used in the Rational Formula which defines the runoff characteristics of the drainage area.

SHALL - This is a mandatory condition. Where certain requirements in the design or application of the guidelines are described with the "shall" stipulation, it is mandatory that they be met.

SHOULD - Recommended but not mandatory.

SOCKET END - Sometimes referred to as the grooved end or female end of a section of concrete pipe. The recessed end of the pipe.

SOFFIT - The bottom of the top of a pipe. In a sewer pipe, the uppermost point on the inside of the structure.

TAILWATER - The depth of flow in the channel immediately downstream of a hydraulic structure.

TIME OF CONCENTRATION - The time associated with the travel of runoff from an outer point in a drainage area to the point being considered.

UNIFORM FLOW - Open channel flow in which the depth of flow is the same at every section of the channel, for a constant flow rate.

URBAN WATERSHED - A watershed which is influenced by urban development. Typically, the runoff hydrograph for such a watershed is affected by an increased flood peak and a shortened time to peak.

WATERSHED - The total area contributing storm runoff to a channel or stream.

WEIR - An obstruction in an open channel over which water flows.

BIBLIOGRAPHY

American Iron and Steel Institute, Handbook of Steel Drainage & Highway Construction Products, Third Edition, 1983.

American Iron and Steel Institute, Modern Sewer Design, First Edition, Washington, D.C., 1980.

Brater, E. F. and H. W. King, Handbook of Hydraulics, Sixth Edition, McGraw-Hill Book Company, N.Y., 1976.

Chow, V. T., Handbook of Applied Hydrology, McGraw Hill Book Company, N.Y., 1964.

City of Austin, Public Works Department, Drainage Criteria Manual, Austin Texas, June, 1988.

City of Victoria, Storm Drainage Design Manual, Victoria, Texas, September, 1980.

Federal Emergency Management Agency, Flood Insurance Study, Calhoun County Texas, Unincorporated Areas, December, 1984.

Federal Emergency Management Agency, Floodplain Management, Ways of Estimating Wave Heights in Coastal Hazard Areas, April, 1981.

Federal Housing Administration, Neighborhood Standards for Southwestern Texas, Land Planning Bulletin No. 3, March, 1961.

Fort Bend County Drainage District, Drainage Criteria Manual, Fort Bend County, Texas, November, 1987.

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

Linsley, R. K. and J. B. Franzini, Water Resources Engineering, Third Edition, McGraw-Hill Book Company, N.Y., 1979

NEENAH Foundry Company, NEENAH Inlet Grate Capacities, 1980.

State Department of Highways and Public Transportation, Hydraulic Manual, Third Edition, December 1985.

U. S. Army Corps of Engineers, Application of the HEC-2 Bridge Routines, Training Document No. 6, Davis, California, June, 1974.

U. S. Army Corps of Engineers, Computer Determination of Flow Through Bridges, Technical Paper No. 20, Davis, California, July, 1970.

U. S. Army Corps of Engineers, HEC-2 Water Surface Profiles, Users Manual, Davis, California, September, 1982.

BIBLIOGRAPHY
(continued)

U. S. Geological Survey, Water Resources Division, Technique for Estimating the Magnitude and Frequency of Floods in Texas, Open File Report 77-110, 1977.

U. S. Soil Conservation Service, "Hydraulics, Section 5", National Engineering Handbook. (NEH-5), August, 1972.

U. S. Soil Conservation Service, "Hydrology, Section 4", National Engineering Handbook. (NEH-4), August, 1972.