Town of
COPPER CANYON

ENGINEERING DESIGN MANUAL

OCTOBER 2022

TOWN OF COPPER CANYON

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PART I
GENERAL

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PART I - GENERAL

Section I – General

1.01 PURPOSE

The purpose of the Engineering Design Manual is to provide a set of guidelines for designing streets and arterials, drainage facilities, other public improvements and for preparing construction plans for such facilities which are to be owned, operated and/or maintained by the Town of Copper Canyon, Texas. These guidelines shall be used by the Town, Consulting Engineers employed by the Town for the above described improvement projects, and Engineers for private developments in the Town of Copper Canyon and its extra-territorial jurisdiction as well as for plat approval, issuance of building permits, issuance of earthwork permits, approval of construction plans by the Town, site plan approval, and for other construction within public rights-of-way and easements subject to Section 245 of the Texas Local Government Code. All projects shall meet state and federal requirements.

1.02 SCOPE

The scope of this section of the Design Manual includes the various design elements, criteria, standards and instructions required for the design of streets and arterials, drainage facilities, water lines, wastewater lines, and other public improvements.

1.03 STANDARD CONSTRUCTION DETAILS

In addition to the guidelines contained in this manual, the Town maintains drawings entitled "STANDARD CONSTRUCTION DETAILS", which are to be used in conjunction with this Design Manual in the preparation of engineering plans.

1.04 CORRELATION OF MANUAL AND STANDARD CONSTRUCTION DETAILS

The Engineering Design Manual and Standard Construction Details are complementary and what is called for by one shall be binding as if called for by both.

In case of conflict between the Engineering Design Manual and Standard Construction Details, the Town reserves the right to make the interpretation that is in the best interest of the Town.

1.05 UTILITY ASSIGNMENTS

Utilities are to be located in public rights-of-way in the location shown in Standard Details. The Town shall determine the location of utilities where special circumstances prevent the standard utility assignments from being used.

1.06 GENERAL NOTES

All construction plans for the projects described above shall contain the applicable general notes listed in Appendix "A".

1.07 CORRELATION OF MANUAL AND SUBDIVISION REGULATION ORDINANCE

The Engineering Design Manual (EDM) and Ordinance Chapter 10 – Subdivision Regulation (subdivision regulation) are complementary and what is called for by one shall be binding as if called for by both.

In case of conflict between the EDM and the subdivision regulation the more stringent criteria shall take precedence.

The Engineering Design Manual was adopted by the Copper Canyon Town Council October 8, 2012.

1.08 VARIANCE PROCEDURE

The Town of Copper Canyon Town Administrator and/or Town Engineer will consider variance requests on an individual basis when, due to geographic or topographic limitations of the site on which the facilities are to be constructed. In considering whether or not a variance should be granted, the Town Engineer shall consider the following factors:

a. The extent to which the proposed design meets other specific standards of the Ordinance; and
b. The extent to which the proposed design meets the spirit and intent of the Ordinance through the use of materials, design criteria and engineering which will protect the health, safety and general welfare of the public; and
c. The positive or negative impact of the proposed design on surrounding property uses and property values, in comparison to the expected impact of the facilities if same were built in strict conformity with the standards of this Ordinance; and
d. The extent to which the proposed design accomplishes the purposes of the Town's Engineering Design Manual and Standard Construction Details.

A variance shall not be granted to serve as a convenience to the applicant or for reasons related to economic hardship.

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PART II
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PART II - PAVING

Section I - STREET AND ARTERIAL CLASSIFICATIONS

1.01 GENERAL

Town streets and arterials are classified into types according to their use and locations as indicated in Table II-1. The basic types include the residential streets which provide direct access and frontage to adjacent properties, collectors/commercials which serve as the distributor-collector/commercial routes and provide direct access to adjacent properties, and minor and major arterials which carry high volumes of traffic. Each roadway is made up of elements which are related to the use of that particular facility. These elements include right-of-way, pavement width, median width if required, arrangement of traffic lanes, curb radii at intersections and other characteristics.

All new roadway in the Town of Copper Canyon shall be constructed in concrete.

The Town of Copper Canyon intends to maintain the rural character of certain areas of the Town as shown on the Land Use Development Concept Plan. Toward that end, rural roadway standards will be permitted in rural residential areas. Rural standards employ paved shoulders in lieu of curb and use bar ditches for drainage.

Section II - STREET AND ARTERIAL DIMENSIONS

2.01 GENERAL

Geometrics of streets and arterials may be defined as the geometry of the curbs or pavement areas which governs the movement of traffic within the confines of the right-of-way. Included in the geometrics are the pavement widths, degree of curvature, width of traffic lanes, shoulders, turning lanes, median width separating opposing traffic lanes, median nose radii, curb radii at street intersections, crown height, cross fall, geometric shapes of islands separating traffic movements and other features.

.

TABLE II-1

STREET AND ARTERIAL CLASSIFICATIONS
AND DIMENSIONS

STREET TYPE	DESCRIPTION	PVMT. WIDTH	MIN. ROW WIDTH	LANES	SHOULDER WIDTH	MIN. PARKWAY WIDTH	DESIGN SPEED (MPH)
Rural Standards							
Alley	Alley	12'	-	-	-	-	-
L2U-R	Local Residential Rural	24'	60'	2-12'	2-2'	18'	30
L2U-U	Local Residential Urban	28’	50’	2-14’	-	11’	30
L2U-C	Local Commercial Urban	37’	60’	2-18.5’	-	11’-6”	35
C2U-R	Local Collector Rural	36'	70'	2-12'	2-6'	17'	35
C2U-U	Local Collector Urban	37'	70'	2-12’	2-6.5’	11'-6"	35
M2U-R	Minor Arterial Rural (undivided)	40'	80-120'	2-12'	2-8'	20'	40
M2U-U	Minor Arterial Urban (undivided)	41'	80-120'	2-12'	2-8.5’	19'-6"	40
M4U-R	Minor Arterial Rural (4-lane undivided)	64’	100’-120’	4-12’	2-8’	18’	40
M4U-U	Minor Arterial Urban (4-lane undivided)	64’	100’-120’	4-12’	2-8’	18’	40
P4D-R	Primary Arterial Rural (divided)	2-32.5'	120'	4-12'	2-2'	17'-0"	40
P4D-U	Primary Arterial Urban (divided)	2-33'	100'	4-12'	2-7.5'	16'-6"	40

Note: 1. All pavement and median width dimensions are to back of curb or edge of pavement.
2. Refer to standard details.

2.02 DESIGN VEHICLES

The geometrics of Town streets and arterial intersections vary with the classification of intersecting streets. Criteria for the geometric design of intersections must be based on certain vehicle operating characteristics, and vehicle dimensions. The American Association of State Highway and Transportation Officials (AASHTO) has standardized vehicle criteria into three general designs which is published in the AASHTO Publication, "A Policy on Geometric Design of Highways and Streets", latest edition. In the design of street and thoroughfare intersections for the Town, these vehicle designs are adopted for use. Table II-2, Intersection Design Standards, shall be used for intersection design.

TABLE II-2

INTERSECTION DESIGN STANDARDS
(All dimensions are minimums)

	A_1*	A_1+	A_1#	A_2*	A_3	B	C	D	E	F	R_1	R_2	Corner Clip
P4D-R, -U	275’	150’	100’	150’	150’	150’	10’	330’	600’	60’	50’	50’	25 X 25
M4U-R, -U	200’	150’	100’	150’	150’	150’	10’	330’	600’	60’	50’	50’	25 X 25
M2U-R, -U	200’	150’	100’	150’	150’	150’	N/A	330’	N/A	N/A	40’	40’	25 X 25
C2U-R, -U	100’	150’	100’	100’	150’	150’	N/A	270’	N/A	N/A	30’	30’	25 X 25
L2U-C	100’	150’	100’	100’	150’	150’	N/A	270’	N/A	N/A	30’	30’	25 X 25
L2U-R, -U	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	30’	30’	15 X 15

* When intersecting street is a principal or minor arterial.
\+ When intersecting street is a collector, commercial or a rural road.
\# When intersecting street is a local street.
** For dual left-turn standards, consult the Town
A_1 and A_2 may be increased to allow for stacking truck traffic.
Corner clip based on 90 degree intersection, may be adjusted for angled intersection.
Radius and corner clip are based on highest classification street at intersection.

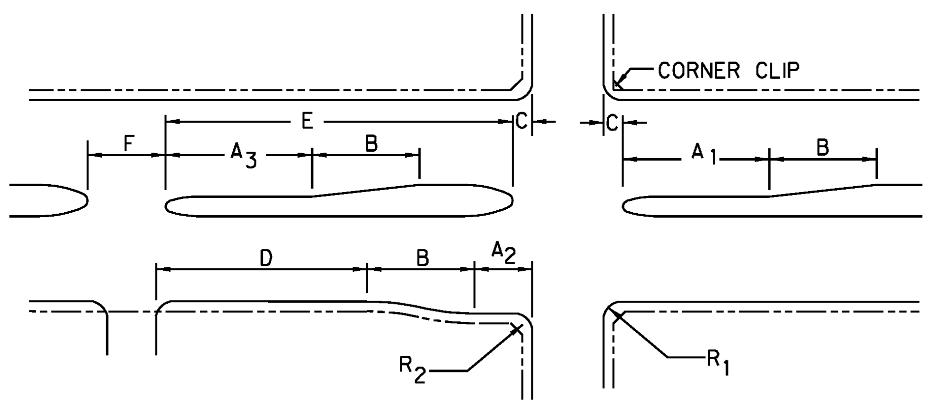

FIGURE II-1
INTERSECTION DESIGN DIAGRAM

2.03 DESIGN SPEED

The design speed is a primary factor in the horizontal and vertical alignment on Town streets and arterials. Design features such as curvature, superelevation, radii for turning movements and sight distance are directly related to the design speed. The design speed also affects features such as lane widths, pavement width, pavement cross-fall, pavement crown, and clearance.

The design speed is defined as the approximate maximum speed that can be maintained safely by a vehicle over a given section of road when conditions are so favorable that the design features of the roadway govern. The speed limit or posted speed is the maximum legal speed set by local authorities for a certain roadway or area. The design speed should always be greater than the likely legal speed limit for arterials.

The various street and arterial classifications, which make up the system within the Town, require different design speeds according to their use and location. The minimum design speeds for the various classifications within the Town of Copper Canyon are presented in Table II-1. Lower design speeds may be required for all classifications for unusual conditions of terrain or alignment.

2.04 HORIZONTAL GEOMETRICS

a. General

The horizontal geometrics of Town streets and arterials include the segment of geometric design associated with the alignment, intersections, pavement widths, and related geometric elements. The various classifications, utilizing the design speed as a control, must have certain horizontal and vertical geometrics to provide a safe economical facility for use by the public. All curves shall provide proper sight distances.

b. Horizontal Curves and Superelevation

The alignment of Town streets and arterials is usually determined by the alignment of the existing right-of-way or structures which cannot be relocated. Changes in the direction of a street or arterial are minimized by constructing a simple curve having a radius compatible with the speed of vehicular traffic. To increase the safety and reduce discomfort to drivers traversing a curved portion of a street or thoroughfare, the pavement may be superelevated.

Curvature in the alignment of arterials and collectors/commercials is allowed, but greater traffic volume and higher vehicle speeds which accompany these facilities tend to increase accidents on curving roadways. Curves in the alignment of residential streets usually provide aesthetic values to the residential neighborhoods without affecting the orderly flow of traffic or sacrificing safety.

A recommended minimum radius of curvature for vehicle design speed and pavement cross-slopes is shown in Table II-3. These are based on traffic consisting of typical present day automobiles operating under optimum weather conditions. There are other important considerations in the design of curves on Town streets and arterials including the location of intersecting streets, drives, bridges and topographic features. When superelevation is required on collectors/commercials and arterials, the following basic formula shall be used:

$$R = \frac{V^2}{15\,(e + f)}$$

where:

e = rate of roadway superelevation, foot per foot

f = Side friction factor (See Table II-3)

V = vehicle design speed, mph

R = radius of curve in feet

TABLE II-3

MINIMUM CENTERLINE RADIUS
FOR ROADWAYS

Rate of Superelevation (In./Ft.)	Residential	Collector/Commercial/ Minor Arterial		Principal Arterial
	DESIGN SPEED (MPH)			
	30 mph	35 mph	40 mph	45 mph
-1/2	500 ft	710 ft	930 ft	1290 ft
-3/8	465 ft	655 ft	855 ft	1175 ft
-1/4	430 ft	605 ft	790 ft	1080 ft
-1/8	400 ft	565 ft	740 ft	1000 ft
0	375 ft	530 ft	690 ft	935 ft
+1/8	355 ft	495 ft	650 ft	875 ft
+1/4	335 ft	470 ft	610 ft	820 ft
+3/8	320 ft	445 ft	580 ft	775 ft
+1/2	300 ft	420 ft	550 ft	730 ft

Street Classification	Side Friction Factor (f)
Residential Streets	0.160
Collector/Commercial Streets	0.155
Arterials	0.145

c. Turning Lanes

Turning lanes are provided at intersections to accommodate left-turning and right-turning vehicles. The primary purpose of these turning lanes is to provide storage for the turning vehicles. The secondary purpose is to provide space to decelerate from normal speed to a stopped position in advance of the intersection or to a safe speed for the turn in case a stop is unnecessary. Left turn lanes at intersections are 12 feet in width. When turning traffic is too heavy for a single lane and the cross street is wide enough to receive the traffic, two turning lanes may be provided.

The location of the median nose at the end of the left turn lane should be located so that left turning traffic will clear the median nose while making a left turn. Other considerations include adequate clearance between the median nose, thru traffic on the intersecting thoroughfare and locations of the median nose to properly clear the pedestrian crosswalks.

Minimum length of left turn lanes for major thoroughfares shall be as specified in Table II-2.

The actual length shall be approved by the Town based upon projected left turn volume.

d. Street Intersections

The intersection at grade of arterials, collector/commercial streets, and residential streets should be at ninety degree (90°) angles where possible. No street intersecting an arterial street shall vary from a ninety-degree (90°) angle of intersection by more than five degrees (5°). Intersection of collector/commercial or residential with streets shall not vary from ninety degrees (90°) by more than ten degrees (10°) Lanes shall be aligned for safe passage through the intersection.

e. Sidewalks

Sidewalk installation shall be at the discretion of the Town. All sidewalks shall conform to state laws for barrier free construction.

The standard concrete sidewalk is 4 feet in width for residential areas and 5 feet in width for commercial areas. Special sidewalk designs to include a 6-foot sidewalk located adjacent to the street will be considered for approval where warranted. For rural paving section sidewalks shall be located in sidewalk easements adjacent to right-of-way lines. Sidewalks shall not be located in ditches. One foot of width shall be added to all sidewalks abutting retaining walls. A 5-foot by 5-foot landing is required every 200 feet for sidewalks less than 5 feet wide.

Sidewalk alignments may be varied to avoid the removal of trees or the creation of excessive slopes when approved by the Town Engineer.

2.05 VERTICAL ALIGNMENT

a. Street Grades

The vertical alignment of Town streets and arterials should be designed to insure the safe operation of vehicles and should allow easy access to adjacent property. A safe travelway for vehicles is dependent on criteria which considers operating speeds, maximum grades, vertical curves and sight distance. In addition to these considerations, other factors related to vertical alignment include storm drainage, crown and cross slope and the grade and right-of-way elevation relationship.

1. Minimum Grades

   Minimum longitudinal grades for streets and arterials are required to insure proper flow of surface drainage toward inlets and to provide minimum ditch grades. Minimum grades are five tenths percent (0.5%) for all urban roadways. Valleys across intersections shall be a minimum of five tenths percent.

2. Maximum Grades

   Maximum longitudinal grades shall be compatible with the type of facility and the accompanying characteristics including the design speed, traffic conditions and sight distance.

   Arterials must move large volumes of traffic at faster speeds and flatter grades will better accommodate these characteristics. Truck and school bus traffic on these type facilities often controls traffic movement, particularly if steep grades prevent normal speeds. The normal maximum street grades allowed are shown in Table II-4. Steeper grades may be permitted for short lengths where topographical features or restricted alignment require.

TABLE II-4

MAXIMUM STREET GRADES

Street Types	Normal Maximum Grade In Percent
Residential	8%
Collector/Commercial	6%
Arterial	6%

b. Vertical Curves

When two longitudinal street grades intersect at a point of vertical intersection (PVI) and the algebraic difference in the grades is greater than one percent (1%) for design speed less than 45 mph or one-half (0.5%) for design speeds greater than 45 mph, a vertical curve is required. Vertical curves are utilized in roadway design to effect a gradual change between tangent grades and should result in a design which is safe, comfortable in operation, pleasing in appearance and adequate for drainage. The vertical curve shall be formed by a simple parabola and may be a crest vertical curve or a sag vertical curve.

c. Stopping Sight Distance

1. Crest Vertical Curve

When a vertical curve is required, it must not interfere with the ability of the driver to see length of street ahead. This length of street, called the stopping sight distance, should be of sufficient length to enable a person in a vehicle having a height of 3.50 feet above the pavement and traveling at design speed to stop before reaching an object in his path that is 0.5-foot in height.

The minimum stopping sight distance is the sum of two distances: first, the distance traversed by a vehicle from the instant the driver sights an object for which a stop is necessary, to the instant the brakes are applied; and second, the distance required to stop the vehicle after the brake application begins.

The minimum safe stopping sight distance and design speeds are shown in Table II-5. These sight distances are based on each design speed shown and based on a wet pavement. The length of crest vertical curve required for the safe stopping sight distance of each street type may be calculated using the formula $L = KA$ and the values of K for a crest vertical curve shown in Table II-5.

2. Sag Vertical Curve

When a sag vertical curve is required, the vertical curve shall be of sufficient length to provide a safe stopping sight distance based on headlight sight distance. The minimum length of sag vertical curve required to provide a safe stopping sight distance may be calculated using the formula $L = KA$ and values of K for a sag vertical curve are shown on Table II-5.

TABLE II-5

MINIMUM LENGTH OF VERTICAL CURVE

CREST VERTICAL CURVE	SAG VERTICAL CURVE
L = KA where	L = KA where
L = Minimum Length Vertical Curve required for safe stopping	L = Minimum Length Vertical Curve required for Headlight Control
K = Horizontal Distance in feet requires to affect a one percent change in gradient	K = Horizontal Distance in feet required to affect a one percent change in gradient
A = Algebraic Difference in grade	A = Algebraic Difference in grade

Street Type	Design Speed	Safe Stopping Sight Distance	Normal Crest Vertical Curve K	Normal Sag Vertical Curve K	Minimum Length of Curve
Local Residential	30	200	19	37	60
Local Collector/Commercial	35	250	29	49	100
Minor & Primary Arterial	40	305	44	64	100

d. Intersection Grades

The grade of an intersecting street with the principal street gutter should not generally be more than two percent (2%) either up or down within the first 20 feet beyond the curb line of the principal street. Grade changes greater than one percent (1%) will require vertical curves.

The grade of street or arterial, particularly at its intersections with another street, is of prime importance in providing a safe, comfortable riding surface. The intersection design of two arterials shall include grades which will result in a plane surface or at least a surface which approximates a plane surface. Grades in excess of 3% should be avoided. A maximum grade of 2% is desirable. A vehicle traveling on either thoroughfare should be able to traverse the intersection at the design speed without discomfort. For intersections involving streets of different classifications, the profile of street with the lesser classification shall be adjusted to meet the profile of the street with the higher classification. No valleys across major thoroughfares or collectors/commercials will be allowed. To accomplish a smooth transition, crossfall toward the median of one lane of each thoroughfare may be required. The use of storm drainage inlets in the median shall be avoided if possible.

In drawing the grades of intersecting thoroughfares in the profile view of plan/profile sheets, profiles of all four profiles shall be shown as a continuous line through the intersection. All intersections where any street is classified as a collector/commercial or arterial shall be contour graded with minimum contour intervals of 0.2 feet.

e. Street Cross Section

For curbed streets, the right-of-way shall be graded to drain to the street at a slope of 1/4" per foot. Street back slopes and embankment slopes shall not be steeper than 4:1.

2.06 SIGHT DISTANCE AT INTERSECTIONS

An important consideration in the design of Town streets and arterials is the vehicle attempting to cross the street or thoroughfare from the side street or drive. The operator of the vehicle attempting to cross should have an unobstructed view of the whole intersection and a length of the thoroughfare to be crossed sufficient to permit control of the vehicle to avoid collisions. The minimum sight distance considered safe under various assumptions of physical conditions and driver behavior is related directly to vehicle speeds and to the resultant distance traversed during perception and reaction time and during braking. This sight distance, which is termed intersection sight distance, can be calculated for different street or thoroughfare widths and for various grades upwards and downwards. Intersection sight by AASHTO publication "A Policy on Geometric Design of Highways and Streets," latest edition. Sight distance requirements are defined by Table II-6 and Figure II-2. As a minimum visibility triangles shall be provided as shown in Figure II-3.

TABLE II-6

SIGHT DISTANCE REQUIREMENTS

Design Speed (mph)	Stopping Sight Distance (feet)	Intersection Sight Distance for passenger Cars (feet)
30	200	335
35	250	390
40	305	445
45	360	500
55	495	610

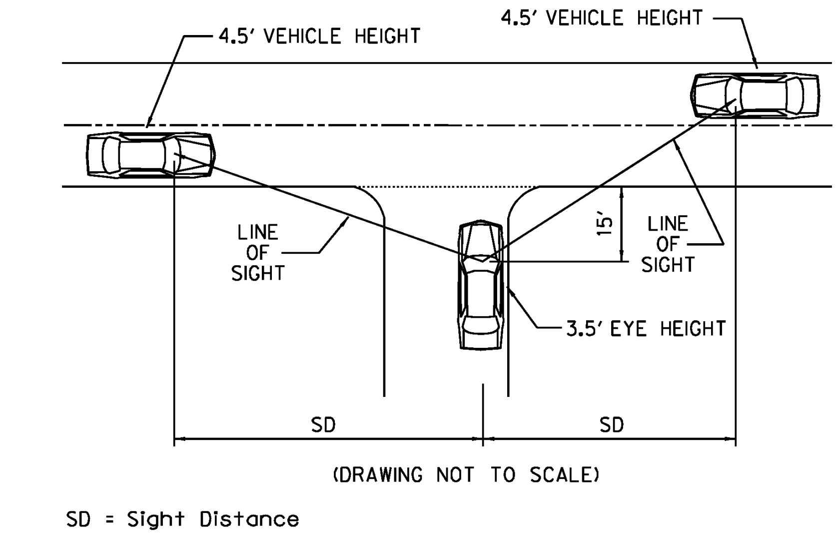

FIGURE II-2
SIGHT DISTANCE REQUIREMENTS

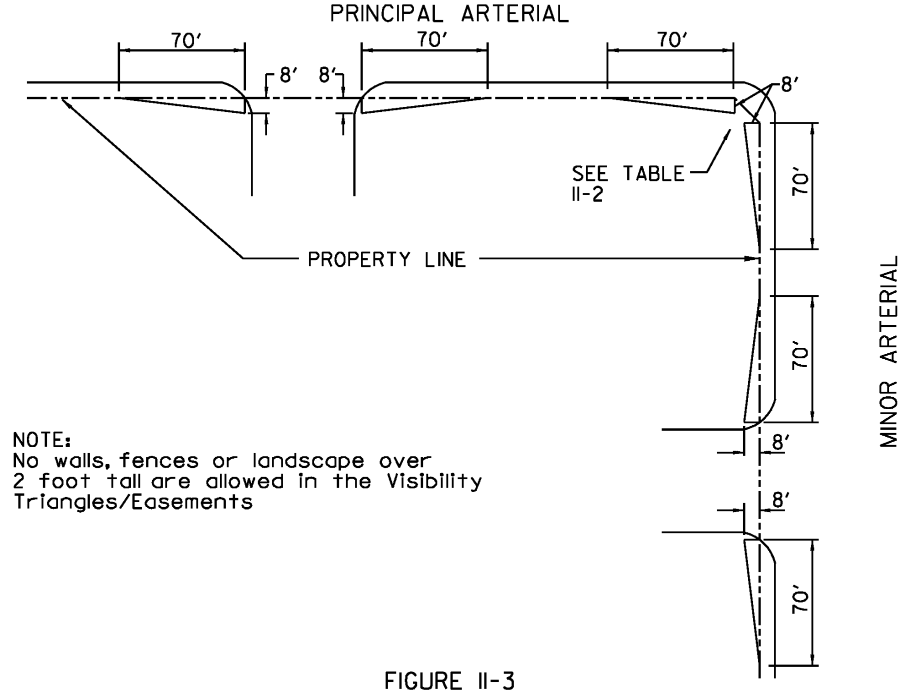

FIGURE II-3
VISIBILITY TRIANGLES

2.07 MEDIAN OPENINGS

The following standards for median openings are established to facilitate traffic movement and promote traffic safety:

Major Streets

Median openings will normally be permitted at all intersections with dedicated Town streets. Exceptions would be at certain minor streets where due to unusual conditions a hazardous situation would result.

Midblock median openings or other openings with turns permitted into adjacent property will not normally be permitted unless all the following conditions exist:

a. The property to be served is a significant traffic generator with demonstrated or projected trip generation of not less than two hundred and fifty (250) vehicles in a twelve-hour period.

b. The median opening is not less than 600 feet from another median opening.

2.08 CUL-DE-SACS

The maximum length of any cul-de-sac shall be 750 feet measured from edge of pavement/curb line of the intersecting street to the radius point of turn around. For cul-de-sac longer than 500 feet, the roadway shall have a minimum width of 26-feet. The right-of-way radius shall be 60 feet and the edge of pavement or face of curb radius 50 feet within the cul-de-sac turn around. All cul-de-sac turnarounds shall be visible from the intersecting street. Additional Right-of-Way may be required to accommodate required drainage.

2.09 PRIVATE STREET/ROAD

Private streets/roads may be permitted for single family residential developments only subject to the following:

A. Private streets/roads shall be defined as roadways serving a minimum of two lots and maximum of four lots, which are not dedicated to the public and are not maintained by the Town. Private streets/roads serving only one lot are considered driveways and not subject to this ordinance. Approval for a subdivision road must be granted by the P&Z and Town Council.

B. Private streets/roads do not require pavement but must be designed as an all-weather road and must be able to support an 85,000 lb fire truck.

C. Applicants must provide a letter from a licensed professional engineer in the State of Texas, indicating that the proposed roadway and any necessary culverts will support that load. Applicants must provide a letter from the ESD#l indicating that the proposed road will meet their requirements for access. Private roads must be maintained by the property owners that are served by the road.

   1. Deed restriction or a maintenance agreement or HOA must be provided to ensure this maintenance.
   2. The private road must be kept maintained sufficiently to provide emergency vehicle access.

3. The Town reserves the right, based on input from the ESD#l, to require the property owners to perform maintenance should the road deteriorate to the level that such emergency access is not afforded.

D. The plat for a subdivision with private roads must contain the following wording on the face of the plat:

"The streets shown on this plat have not been dedicated to the public for public access nor have they been accepted by the Town of Copper Canyon as public improvements for public maintenance. The streets shown on this plat shall be maintained by the property owners within the subdivision. The streets shall always be accessible to emergency vehicles, public and private utility maintenance and service personnel, solid waste collections services, the U.S. Postal Service, governmental employees in pursuit of their official duties, and courier services.

TABLE II-7

PRIVATE STREET/ROAD REQUIREMENTS

	Must meet Fire Code Requirements and have culvert if necessary?	**Number of Lots**	**Surface Required**	**Concrete or Asphalt Apron Approach Required?**
Driveway	Yes	1	None specified	Yes
Private Road	Yes	2-4	None specified	Yes
Subdivision	Yes	>4	As per Town Ordinances	Yes

2.10 STREET SIGNS

New streets shall be so named as to provide continuity of names with existing streets. Similar or identical street names to streets already existing in other parts of the Town and surrounding areas shall be avoided.

Each intersection shall have at least one street sign. Street signs and installation of such signs shall be in accordance with the type as used throughout the Town and the cost of purchasing and installing of street signs shall be borne by developer.

2.11 ACCESS

All platted lots shall have safe and reliable street access for daily use and emergency purposes. Except for lots that are provided access from an approved cul-de-sac, all subdivisions shall have two (2) means of access or approach. Where development phasing or constraints of the land prevent the provision of a second, separate means of access, the Town Council may accept a temporary street connection, a median divided entry or other appropriate means of access to satisfy the requirements of this Section.

2.12 CONNECTION OF STREETS FROM OTHER JURISDICTION

Areas outside the Town which are being subdivided or have been subdivided pursuant to the ordinances of another political subdivision or for which there exist no regulations shall not project streets into the corporate limits of the Town or tie into or have access onto existing Town of Copper Canyon roadways or streets without the approval of the Town Council.

A. Procedure in such cases.

1. An applicant for such access shall file an application with the Town Secretary, on forms to be supplied by the Town, which shall identify all access points to Town roads and streets for the land involved and contain all of the information required in the Town's Comprehensive Subdivision Ordinance, as amended.

2. After a pre-application conference with the Town Administrator and Town Engineer, the application shall be forwarded to the Planning and Zoning Commission, which shall hold public hearings after published notice required for zoning cases, as contained in the Town's Zoning Ordinance, as amended. After the public hearing and recommendation, the Planning and Zoning Commission shall forward the application to the Town Council for consideration.

3. The Town Council shall hold public hearings after published notice required for zoning cases, as contained in the Town's Zoning Ordinance, as amended. The Town Council may approve or reject the application for access or take such other action as is lawful and appropriate.

4. This procedure shall not apply to State or Federal designated roadways. This process shall not apply to existing roads being rebuilt, expanded or realigned by another governmental entity with Town of Copper Canyon participation as may be defined in a Town Council approved interlocal agreement with the other governmental entity.

B. Studies required.

1. The applicant may be required to conduct studies for review by the Planning and Zoning Commission and Town Council, giving a clear understanding of the effects of the traffic directly or indirectly caused by the proposed road or street extension and connection upon the Town's thoroughfare system and demonstrating the method by which the applicant shall accomplish the following objectives:

a. to coordinate public and private investment;

b. to minimize conflict between land uses;

c. to influence and manage the development of the Town;

d. to increase the benefits and cost-effectiveness of public investments;

e. to predict public infrastructure and service needs in advance of demand;

f. to ensure that Town facilities are located to best serve the citizens; and

g. to ensure public safety on affected Town roads and streets, both during and after construction.

2. A traffic impact analysis may be required by the Town which shall determine the effect and impact that the extension of roads and streets into the Town's transportation system will have on the Town's existing and planned roads and streets, including but not limited to the projected level of service that will occur on Town roads and streets and intersections, the mobility and access of traffic on Town roads and streets, any bottleneck considerations, any need for turn lanes, median openings, signs, signals, illumination, pedestrian facilities, the impact upon any equestrian and hike/bike trails or other transportation system components, and whether the projected traffic is consistent with adjacent land uses in the Town through which traffic will pass. These impacts must be quantified and reflect phases of proposed development in relation to any of the Town's capital improvement plans. The applicant shall pay to the Town a fee to cover the expense of the traffic impact analysis and the fee determination will be made by the Town after review of the type, size and location of the proposed road or street extension.

3. The applicant may be required to conduct studies which demonstrate that the proposed road or street extension will not adversely affect the public health, safety and general welfare of the citizens of the Town of Copper Canyon and will provide reasonable protection to properties within the Town that will be impacted by the additional traffic projected to come into the Town by virtue of the extension of the road or street into the Town's transportation system.

C. Engineering requirements.

1. The applicant shall prepare, or have prepared, and submit complete engineering plans in accordance with the requirements of all Town ordinances, for the design and construction of the road or street that is proposed to extend into the Town's transportation system.

2. All construction of roads or streets within the Town or that adjoin or otherwise touch a Town road or street shall be done in accordance with the Town's general design standards, from the point of such touching the existing road or street to the Town's corporate limits or the end of the radius of the curb return outside the Town, whichever is further.

3. The applicant must pay the required inspection fee in an amount required by the Town at the time of application.

4. The applicant shall provide as-built drawings and an appropriate maintenance bond, as otherwise provided in the Comprehensive Subdivision Ordinance, as amended.

5. The applicant shall be responsible for all construction costs of extensions of Town roads and streets and modifications to existing Town roads and streets. If such construction is determined by the Town to be impractical, then sufficient escrow funds shall be provided to the Town in lieu of construction of the required paving.

D. No vesting of rights by actions of other political subdivisions.

The approval of any zoning district classification, subdivision or development plat, or similar development applications or requests pursuant to the ordinances of some other political subdivision shall not vest any rights, as that term is used in Chapter 245 of the Texas Local Government Code, as amended, in the Town of Copper Canyon.

2.13 ALLEYS

A. Alleys are not permitted for single-family residential development. Alleys may be provided in commercial, industrial and retail districts and at the rear of multifamily residential building sites. In lieu of an alley, an emergency access easement shall be dedicated to provide circulation and access for emergency, health and all public safety vehicles.

B. [Reserved.]

C. Alleys, when required, shall be provided parallel, or approximately parallel to the frontage of the street.

D. The minimum right-of-way width of an alley shall be eighteen feet (18') and more may be required when proposed uses, utility services and peculiar needs of the property require wider alleys. Pavement width shall be a minimum of twelve feet (12').

E. Alleys should intersect streets at right angles or radially to curved streets where sharp changes in alignment cannot be avoided. Where two (2) alleys intersect, a cutoff of not less than fifteen feet (15') along each property line from the normal intersection of the property lines shall be provided to permit safe vehicular movement.

F. Dead-end alleys shall not be permitted.

G. All alleys shall be paved in accordance with this and other applicable ordinances of the Town where required.

H. Where driveways connect to alleys in commercial, industrial or retail areas, fences shall only be constructed along the rear lot line of any lot to a point where the driveway would intersect the alley pavement at ninety degrees (90º). Fences to be constructed along any driveway or perpendicular to alley shall not be constructed within five feet (5') of the rear lot line or alley easement line.

2.14 EASEMENTS

A. Easements for utility services shall be provided as may be required to provide access for all public and private utilities to each lot or tract within the development and such easement shall be planned for underground installations except where approval is given for reasons of public convenience or necessity. Easements for utility construction, service, and maintenance shall be provided in locations approved on the Final Plat and affected utilities according to the following standards:

1. All services for utilities shall be made available in each lot in such a manner so as to eliminate the necessity for disturbing the street and the alley pavement, curb, gutter, sidewalks, and drainage structures when connections are made, when approved by the Town.

2. Easements across lots on rear or side lot lines shall be at least fifteen feet (15') wide.

3. Emergency access easements shall have a clear unobstructed width of twenty-four feet (24') and shall connect at each end to a dedicated public street. An emergency access easement may be used as a driveway to gain access to parking or loading spaces but shall not be used for parking.

B. Where a subdivision is bounded by a watercourse, drainageway, channel, or stream there shall be provided a stormwater easement of a sufficient width such as a storm with a design frequency of one hundred (100) years shall not exceed the highest contours of that easement.

Section III - DRIVEWAY STANDARDS

3.01 DRIVEWAY REQUIREMENTS

A. Driveway entrances connecting to the Town's roadways or streets shall be provided culverts, sized by a Professional Engineer to carry the design flow in the roadside ditch, and reinforced concrete with concrete headwalls on each end. The driveway culvert shall have a minimum diameter of 18-inches. Property owner or his engineer shall provide enough information to the Town Engineer to justify the smaller diameter culvert.

B. For a lot with an existing house, the property owner may install a driveway culvert that is the same size as the largest culvert upstream or adjacent downstream from the driveway with a minimum diameter of 18-inches or obtain a variance from the Town Engineer. Concrete Headwalls shall be installed on each end. Property owner shall provide enough information to the Town Engineer to justify the smaller diameter culvert.

C. Residential driveways shall have a minimum 15-ft throat width and may taper to a minimum 12-ft driveway width.

Driveways shall be governed by Tables II-8. Refer to Figures II-1 and II-4.

TABLE II-8

DRIVEWAY REQUIREMENTS

	Residential (Min) (Max)	Commercial (Min) (Max)
A - Driveway Throat Width		
Local	15 – 28 ft	30 – 40 ft
Collector/Commercial	15 – 28 ft	30 – 40 ft
Minor Arterial	N/A	30 – 60 ft
Principal Arterial	N/A	30 – 60 ft
Driveway Curb Radius		
Local	5 ft (Min)	20 ft (Min)
Collector/Commercial	5 ft (Min)	25 ft (Min)
Minor Arterial	10 ft (Min)	30 ft (Min)
Principal Arterial	10 ft (Min)	35 ft (Min)
B - Minimum Centerline Driveway Spacing Along		
Local	25-38 ft	70 ft (Min)
Collector/Commercial	25-38 ft	120 ft (Min)
Minor Arterial	25-38 ft	170 ft (Min)
Principal Arterial	25-28 ft	230 ft (Min)
Driveway Angle	90°	90°
C - Minimum Distance from Driveway to Intersection		
Local	50 ft (Min)	100 ft (Min)
Collector/Commercial	50 ft (Min)	120 ft (Min)
Minor Arterial	150 ft (Min)	150 ft (Min)
Principal Arterial	150 ft (Min)	150 ft (Min)
Maximum Approach Grade		
Local / Collectors/Commercials	10% (Max)	6% (Max)
All Others	10% (Max)	6% (Max)
Right Turn Requirement	10% (Max)	6% (Max)

* Can be wider based on site requirements.
** Driveways should be used jointly at median openings.
Based on 40 mph.
Driveway width plus radius must be contained within the property frontage, between the extended property lines.
State Standards, if more restrictive, shall apply to State maintained roadways.

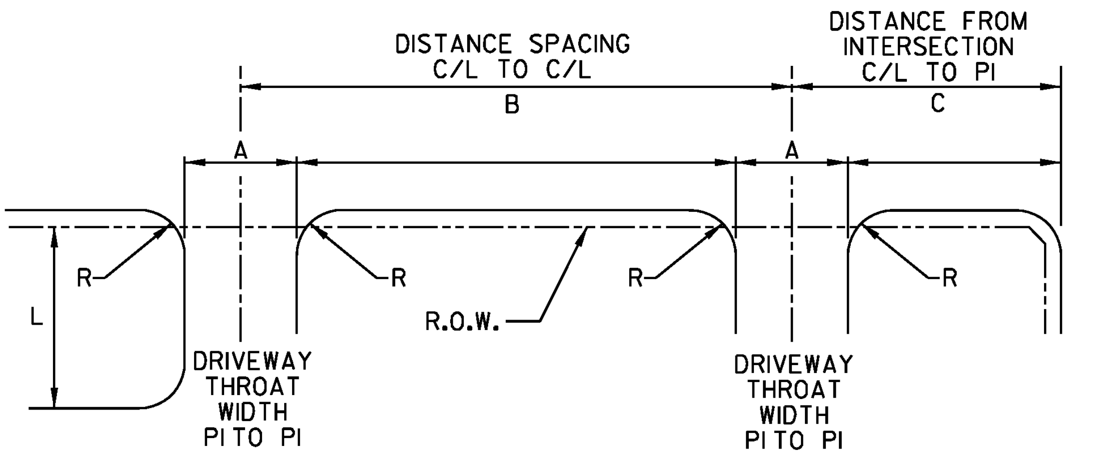

FIGURE II-4
DRIVEWAY REQUIREMENTS

Section IV - TRAFFIC IMPACT ANALYSIS GUIDELINES

4.01 DEFINITIONS

A. Projected traffic volumes – The number of vehicles that are expected/calculated to exist on streets after completion of the project.

B. Study area – The boundaries in which the study is conducted.

C. TIA (Traffic Impact Analysis) – An in-depth analysis of traffic.

D. Traffic queuing – A line of waiting vehicles.

E. Trip distribution – Estimates of percentage distribution of trips by turning movements from the proposed development.

F. Trip generation summary – A table summarizing the trip generation characteristics of the development for the entire day including AM and PM peak periods, rates and units used to calculate the number of trips.

G. Non-site traffic – Traffic not created or associated with the traffic generated by the project.

4.02 PURPOSE

The purpose of a Traffic Impact Analysis (TIA) is to assess the effects of specific development activity on the existing and planned roadway system. It is the intent of this ordinance to make traffic access planning an integral part of the development process.

4.03 APPLICABILITY

A. A Traffic Impact Analysis (TIA) will be required at the time of platting for land developments that are expected to meet a threshold level of change as described in Section 4.04 below, “When Traffic Impact Analysis (TIA) is Required”. The Town reserves the right to require a TIA for land developments that do not meet the threshold requirements but may impact a sensitive area with traffic issues or may be a known public concern.

B. A Traffic Impact Analysis (TIA) will be required when there is a request to amend the Thoroughfare Plan.

4.04 WHEN TRAFFIC IMPACT ANALYSIS (TIA) IS REQUIRED

A. A TIA will be required of the property owner (or designated agent) when an activity or change to the property occurs and any of the following occur:

1. More than 500 Peak Hour Trip (PHT) generation
2. More than 5,000 vehicle trips per day generation
3. More than 100 acres of property is involved
4. Any changes or alterations to the Town Thoroughfare Plan

B. The property owner (or designated agent) shall perform and submit to the Town of Copper Canyon a TIA performed at a minimum as established in Section 4.06, “Traffic Impact Analysis Requirements”. The TIA must be signed and sealed by a professional engineer, registered to practice in Texas, with experience in Transportation Engineering sufficient to assess traffic impacts.

C. The engineer conducting the study must be approved by the Town prior to performing the study. The Town of Copper Canyon Public Works Department must approve all TIA’s before final acceptance. After acceptance of the TIA, the review process will determine further actions.

4.05 <u>ROLES OF APPLICANT AND TOWN</u>

A TIA that is required of the applicant by the Town of Copper Canyon is part of the development review and approval process. The primary responsibility for assessing the traffic impacts associated with a proposed development rest with the applicant. The Town serves in a review capacity for this process.

4.06 <u>TRAFFIC IMPACT ANALYSIS (TIA) REQUIREMENTS</u>

A. The Traffic Impact Analysis (TIA) must be prepared and evaluated by a consultant who meets the qualifications described in Section 4.04 (b) to perform such studies.

B. The analysis is required to contain at a minimum, the following:

1. Traffic Analysis Map

(a) Land Use, Site and Study Area Boundaries, as defined (provide map).

(b) Existing and Proposed Site Uses.

(c) For TIA’s where land use is the basis for estimating projected traffic volumes and existing and Proposed Land Uses on both sides of boundary streets for all parcels within the study area (provide map).

(d) Existing and Proposed Roadways and Intersections of boundary streets within the study area of the subject property, including traffic conditions (provide map).

(e) All major driveways and intersecting streets adjacent to the property will be illustrated in sufficient detail to serve the purposes of illustrating traffic function. This may include showing lane widths, traffic islands, medians, sidewalks, curbs, traffic control devices (traffic signs, signals, and pavement markings), and a general description of the existing pavement condition.

(f) Photographs of adjacent streets of the development and an aerial photograph showing the study area.

2. Trip Generation and Design Hour Volumes (provide table).

(a) A trip generation summary table listing each type of land use, the building size assumed, average trip generation rates used (total daily traffic and a.m./p.m. peaks), and total trips generated shall be provided.

(b) Vehicular trip generation may be discounted in recognition of other reasonable and applicable modes, e.g., transit, pedestrian or bicycles. Trip generation estimates may also be discounted through the recognition of passby trips and internal site trip satisfaction. All such estimates shall be subject to the approval of the Town.

(c) Proposed trip generation calculations for single-story commercial properties shall be based on a Floor-to-Area (building size to parcel size) ratio of 0.25 or more.

3. Trip Distribution (provide figure by Site Exit/Entrance). The estimates for percentage distribution of trips by turning movements to/from the proposed development.

4. Trip Assignment (provide figure by site entrance and boundary street). The direction of approach of site-attracted traffic via the area's street system.

5. Existing and Projected Traffic Volumes (provide figure for each item). Existing traffic volumes are the numbers of vehicles on the streets of interest during the time periods listed below, immediately prior to the beginning of construction of the land development project. Projected traffic volumes are the number of vehicles, excluding the site-generated traffic, on the streets of interest during the time periods listed below, in the build-out year.

   (a) A.M. Peak Hour site traffic (including turning movements) if significant impact.

   (b) P.M. Peak Hour site traffic (including turning movements).

   (c) Weekend Peak Hour site traffic (including turning movements).

   (d) A.M. Peak Hour total traffic including site-generated traffic and Projected Traffic (including turning movements).

   (e) P.M. Peak Hour total traffic including site-generated traffic and Projected Traffic (including turning movements).

   (f) Weekend Peak Hour total traffic including site-generated traffic and Projected Traffic (including turning movements).

   (g) For special situations where peak traffic typically occurs at non-traditional times, e.g., major sporting venues, entertainment venues, large specialty Christmas stores, etc., any other Peak hour necessary for complete analysis (including turning movements).

   (h) Total daily existing traffic for street system in study area.

   (i) Total daily existing traffic for street system in study area and new site traffic.

   (j) Total daily existing traffic for street system in study area plus new site traffic and projected traffic from build-out of study area land uses.

6. Capacity Analysis (provide Analysis Sheets in Appendices).

(a) A capacity analysis shall be conducted for all public streets, intersections and junctions of major driveways with public streets, which are significantly impacted (as designated by the Town), by the proposed development within the previously defined study boundary.

(b) Capacity analysis will follow the principles established in the latest edition of the Transportation Research Board's *Highway Capacity Manual* (HCM), unless otherwise directed by the Transportation Services Director. Capacity will be reported in quantitative terms as expressed in the HCM and in terms of traffic Level of Service.

(c) Capacity analysis will include traffic queuing estimates for all critical applications where the length of queues is a design parameter, e.g., auxiliary turn lanes and at traffic gates.

7. Conclusions and Requirements.

(a) Roadways and intersections, within the Study Area, that are expected to operate at Level of Service D, E, or F, under traffic conditions including projected traffic plus site-generated traffic must be identified and viable recommendations made for raising the traffic conditions to Level of Service C or better (Level of Service A or B).

(b) Level of Service "C" is the design objective for all movements and under no circumstances will less than Level of Service "D" be deemed acceptable for site and non-site traffic including existing traffic at build-out of the study area. The Town must approve a Level of Service "D".

(c) For phased construction projects, implementation of traffic improvements must be accomplished prior to the completion of the project phase for which the capacity analyses show that they are required. Plats for project phases subsequent to a phase for which a traffic improvement is required may be approved only if the traffic improvements are completed or bonded.

(d) Voluntary efforts, beyond those herein required, to mitigate traffic impacts are encouraged as a means of providing enhanced traffic handling capabilities to users of the land development site as well as others.

(e) Traffic mitigation tools include, but are not limited to, pavement widening, turn lanes, median islands, access controls, curbs, sidewalks, traffic signalization, traffic signing, pavement markings, etc.

(f) The applicant will provide five (5) copies of the Draft Report for review and nine (9) copies of the Final Report for submittal.

8. Other Items

(a) The Town may require other items be included in the TIA above those listed above.

Section V - PAVEMENT DESIGN

5.01 STANDARD STREET AND ARTERIAL PAVEMENT DESIGN

All new roadways within the Town of Copper Canyon shall be constructed of reinforced concrete. Asphalt pavements may be used for temporary construction, if approved by the Town. Table II-9 shows the required pavement thickness for reinforce concrete pavement and the subgrade requirements for certain soil conditions for various street and thoroughfare types within the Town. The procedure for using this table requires that a soils investigation be made to include obtaining soil auger borings, classifying the soils encountered and determining the strength and physical properties of the underlying and supporting soils system in moisture content, and unit dry weight (see 5.02 – Geotechnical Investigation Required). For each soil classification encountered, the plasticity index shall be calculated and depending whether the P.I. is less or more than the critical percentage shown, the subgrade design shall consist of lime or cement treated subgrade as shown in Table II-9. Table II-9 also presents the minimum pavement thickness of reinforced portland cement concrete pavement for the various street and arterial types.

5.02 GEOTECHNICAL INVESTIGATION REQUIRED

A geotechnical investigation must be performed for all new developments within the Town of Copper Canyon containing public streets. As a minimum, the study must address the following:

- general soil and groundwater conditions
- earthwork recommendations
- recommendations for pavement subgrade type, depth, and concentration
- guidelines and recommendation for concrete pavement design

The investigation must be based on samples obtained from drilling or from excavations on the site. Samples must be tested in a laboratory. Tests must include as a minimum:

- moisture content and soil identification
- liquid and plastic limit determination
- unit weight determination
- Eades and Grim lime series tests
- soluble sulfate tests

The geotechnical investigation must be performed by a qualified geotechnical firm. A report with findings and recommendations must be prepared. The report shall bear the seal of a Licensed Engineer in the State of Texas.

5.03 GUIDELINES FOR STABILIZATION OF SUBGRADE SOILS CONTAINING SULFATES

Lime induced heaving has been a cause of pavement failures in the North Texas area. There are four components which are the culprits in sulfate induced stress in stabilized soils: calcium, aluminum, water, and sulfates. Together, and in the proper combination, these components will produce calcium-aluminate-sulfate-hydrate minerals with an expansion potential as large as 250%.

The best approach when dealing with lime stabilization of clay with significant soluble sulfate content is to force the formation of the deleterious minerals prior to compaction. If these minerals form during the mellowing period before placement and compaction, no damage will be done to the pavement. This can be done by providing adequate mellowing time (time delay between application of stabilizer and compaction of the stabilized soil) and with addition of adequate water.

Generally if the total level of soluble sulfates is below 2,000 ppm (parts per million), by weight of soil, then sulfate induced heaving is not of significant concern.

If sulfate content is greater than 2,000 ppm, specific recommendation shall be made by geotechnical engineer.

5.04 ALTERNATE PAVEMENT DESIGN

The Town Engineer will consider an alternate pavement design in lieu of selecting a design from Table II-9, particularly when there are circumstances which warrant an individual design.

TABLE II-9

STANDARD STREET AND THOROUGHFARE MINIMUM PAVEMENT DESIGN

Facility Type	Treated Subgrade P.I. less Than 15	Treated Subgrade P.I. = 15 or Greater	Concrete Pvmt. (1)
Fire Lane and Driveways	6” Cement	6” Lime	6”
Alleys	6” Cement	6” Lime	7”
Residential			
L2U- R	6” Cement	6” Lime	6”
L2U- R (Asphalt Alternative)	6” Cement	6” Lime	N/A
L2U-U	6” Cement	6” Lime	7”
Commercial			
L2U-C	6” Cement	6” Lime	7”
Collector			
C2U-U and R	6” Cement	6” Lime	7”
Minor Arterial			
M2U-U and R	6” Cement	6” Lime	8”
M4U-U and R	6” Cement	6” Lime	8”
Principal Arterial			
P4D-U and R	6” Cement	8” Lime	8”

NOTE: 1) Twenty-eight day concrete compressive strength shall not be less than 4,000 psi for machine-pour and 4,200 psi for hand-pour.

Section VI – BRIDGES

6.01 GENERAL

A. All vehicular bridges used for public roads, private roads and driveways shall meet HS-20 loading in accordance with the minimum design standards of the American Association of State Highway Transportation Officials (AASHTO) including the latest revisions thereof.

B. Bridge foundation construction shall be based on a foundation design prepared by a geotechnical engineering firm and based on a soil boring taken at the site of the bridge.

C. Bridges on private driveways serving a single residence shall have a minimum roadway surface (inside guardrails) of twelve (12) feet. All other bridges shall conform to the pavement width requirements of this Code.

D. All bridge embankments shall be constructed in accordance with North Central Texas Council of Governments Standard Specifications for Public Works Construction, Item 3.7 - embankments, including the latest revisions thereof.

E. All bridges over waterways in Federal Emergency Management Authority's (FEMA) designated floodplain areas shall require a Flood Study prepared by a registered professional engineer in accordance with guidelines published by FEMA's National Flood Insurance Program (NFIP). An applicant shall submit the Flood Study in conjunction with any Concept Plan, Development Plan, Preliminary Plat or Final Plat, and the Flood Study shall be in accordance with the Town's Flood Damage Prevention Ordinance, as amended.

Section VII - PERMANENT LANE MARKINGS

7.01 PAVEMENT MARKINGS PLAN AND SIGNAGE

Permanent lane markers shall be installed in accordance with the pavement markings plan and Town Standard Details.

Signage shall comply with the Manual Uniform Traffic Control (MUTC), latest edition, and Town's standard details.

Section VIII - LANDSCAPING IN PUBLIC RIGHT-OF-WAY

8.01 GENERAL

All unpaved public medians and parkways shall be landscaped with a minimum of four inches of topsoil, sodded or seeded in accordance with seeding requirements in the standard details and irrigated with a properly designed and installed system.

8.02 METERING

All water usage shall be metered and paid for by the developer until landscaping is accepted by the Town. Developers shall pay administrative fees, meter costs, and meter deposits, but shall be exempt from impact fees for meters installed on Town right-of-way. Within medians, no plantings or irrigation facilities shall be permitted within areas five feet or less in width from the edge of pavement or in median noses. Those areas shall be covered with brick pavers in accordance with the Standard Details.

8.03 OTHER REQUIREMENTS

A. For Town maintained landscapes, the requirements will be established in Section 8.01.

B. Trees or upright plantings must not be planted within 30 feet of intersections or utility poles. The Town may require greater setback for safety based on line of sight issues.

C. An 8-inch wide concrete mow strip shall be installed between all planting beds and grassed areas.

D. Seeded or sodded areas of medians shall be bermed a minimum of 6 inches.

E. Only trees with a mature height less than 30 feet may be planted closer than 20' either side of an overhead line. No trees shall be directly under utility lines.

F. Trees to be planted within the medians of divided roadways that are ultimately planned for widening by constructing additional lanes in the median shall not be planted within the path of future lanes. Trees shall not be planted within five (5) feet of existing or proposed curbs. Future lane widening shall be shown on the landscape plans.

F. Trees shall not be planted within five feet of existing or proposed water lines.

8.04 PLAN SUBMITTAL REQUIREMENTS

Landscape and irrigation construction plans shall be submitted as part of the overall construction plans associated with the related project. Plans shall bear license seal of a licensed landscape architect. The plans shall include the following:

A. A scale drawing (1" = 40' or 1" = 20'), prepared on 22" by 34" sheets clearly indicating the location, type, size and description of all proposed landscape materials and existing utilities.

B. The name of the project, name and address of the Developer, north arrow, scale, and legend.

C. The configuration, location, type and size of all irrigation, piping heads and controllers.

D. All details necessary to provide a constructible installation.

8.05 OWNERSHIP AND MAINTENANCE

A. Landscape areas shall be maintained by the Developer, owner, or Home Owner Association for a minimum of one year.

Section IX - STREET LIGHT REQUIREMENTS

9.01 GENERAL

Street lights shall be installed in commercial areas. The Developer shall pay the costs for installation, maintenance, and operations all street lighting. Street light luminaries shall be high pressure sodium (HPS). Street light location, materials, and design shall be approved by the Town.

9.02 STREET LIGHT GUIDELINES

A. Curtail and reverse any degradation of the nighttime visual environment and the night sky.

B. Minimize glare and obtrusive light by limiting outdoor lighting that is misdirected, excessive, or unnecessary.

C. Conserve energy and resources to the greatest extent possible.

D. All lighting installations shall be designed and installed to be fully shielded (full cutoff), except as in exceptions below, and shall have a maximum lamp wattage of 250 watts HID, or lumen equivalent, for commercial lighting, 100 watts incandescent, or equivalent lumens (approximately 1,600 lumen) for residential lighting. In residential areas, light should be shielded such that the lamp itself or the lamp image is not directly visible outside the ROW perimeter.

E. Luminaires shall be mounted on poles at least 11 feet high.

9.03 STREET LIGHT LOCATIONS

A. Street lights shall be installed at each intersection, at major curves, at ends of cul-de-sacs, and at intervals of between 200 and 600 feet.

B. Street lights shall be installed in the public right-of-way, in a location at least three (3) feet behind the face of curb. Where there is no curb, street lights shall be installed at least eight (8) feet from the edge of pavement. Street lights on major arterials shall be installed in the median, where a median exists. In conjunction with the development of any subdivision, street light location and installation shall be coordinated with Coserv Electric and the Town.

9.04 PLAN SUBMITTAL REQUIREMENTS

Street light plans shall be submitted as part of the overall construction plans associated with the related project. The plans shall include the following:

A. A layout of the entire subdivision showing the location of each street light.

B. A plan for the location of underground conduits. All street lights shall be served by underground electric unless approved in writing by the Town. All wiring shall be placed in minimum two (2) inch schedule 40 PVC conduit.

C. Standard street light details.

9.05 COSTS

The developer or HOA shall be responsible for all engineering, plan preparation, installation, maintenance, and operation costs required for installation of street lights.

TOWN OF COPPER CANYON
ENGINEERING DESIGN MANUAL

PART III

WATER AND WASTEWATER LINES

OCTOBER 2022

PART III – WATER AND WASTEWATER LINES

Section I – WATER

This section pertains to general design requirements for water system construction in the Town of Copper Canyon. Bartonville Water Supply Corporation provides the water within the Town of Copper Canyon. Approval from Bartonville Water Supply shall be obtained prior to connection to the existing water lines. Water lines shall be installed per the latest Bartonville Water Supply Corporation Standards.

1.01 GENERAL

Water mains shall be placed on the north and east sides of a street. Where applicable, line sizes shall comply with the Bartonville Water Supply Corp. Water Master Plan or subsequent revisions.

A. Mains shall be minimum 8-inch diameter pipe. For mains in commercial and manufacturing districts, a minimum of 12-inch diameter pipe will be required if the main is over 600 feet in length.

B. Dead end lines are not allowed.

C. Fire hydrant lead lines shall be 6-inch diameter pipe. Fire hydrant lead lines shall be no greater than 50 feet in length. Any fire hydrant lead line over 20 feet shall be 8-inch diameter pipe.

D. Water lines 12-inches and greater shall be profiled. P.I.s shall be stationed and elevations to 0.01 feet provided for all water lines.

1.02 WATER LINE MATERIAL

A. Water mains materials shall comply with Bartonville Water Supply Corp. latest design guidelines.

B. Ductile iron fittings with polywrap shall be used.

C. Water lines shall be minimum pressure Class 150.

D. All water mains outside utility easements which supply fire sprinkler systems shall be minimum 200 PSI working pressure and U.L. listed.

E. Water mains shall be standard sizes that are readily available, such as 8-inch, 12-inch, 16-inch, 18-inch, 20-inch, 24-inch, 30-inch, and 36-inch.

1.03 LOCATION

Water mains shall be constructed with extensions to the development boundary to allow for direct connection by future developments.

1.04 WATER VALVES

Valves 12-inches and smaller shall be placed on or near street property lines and shall be spaced at a maximum of 800 feet apart in residential and 500 feet in all other districts. They shall be placed in such a manner as to require preferably two, but not more than three valves to shut down each Town block, or as may be required to prevent shutting off more than one fire hydrant. On cross-feed mains without services, a maximum of four valves shall be used to shut down each block. Also, valves shall be placed at or near the ends of mains in such manner that a shut-down can be made for a future main extension without causing loss of service on the existing main. If valves cannot be located for a shut-down, restrained joints shall be used. The location of valves larger than 16-inches will be as approved by the Director of Engineering and Utilities. Valves 16-inches and under shall be Resilient Seat Gate Valves (RSGV). All valves will be gate valves.

1.05 FIRE HYDRANTS

A. Number and Locations

A sufficient number of fire hydrants shall be installed to provide hose stream protection for every point on the exterior wall of the building. There shall be sufficient hydrants to concentrate the required fire flow, as recommended by the publication "GUIDE FOR DETERMINATION OF REQUIRED FIRE FLOW" published by the Insurance Service Office, around any building with an adequate flow available from the water system to meet this required flow. Fire hydrant markers shall be provided at each hydrant. In addition, the following guidelines shall be met or exceeded:

1. SINGLE FAMILY AND DUPLEX RESIDENTIAL - As the property is developed, fire hydrants shall be located at all intersecting streets and at intermediate locations between intersections at a maximum spacing of 500 feet between fire hydrants as measured along the route that fire hose is laid by a fire vehicle. All buildings shall be within a 500 foot radius of a fire hydrant.

2. MULTIFAMILY RESIDENTIAL - As the property is developed, fire hydrants shall be located at all intersecting streets and at intermediate locations between intersections at a maximum spacing of 400 feet as measured along the length of the center line of the roadway, and the front of any structure at grade and shall be no further than 400 feet from a minimum of two fire hydrants as measured along the route that a fire hose is laid by a fire vehicle. All buildings shall be within a 400 foot radius of a fire hydrant.

3. OTHER DISTRICTS - As the property is developed, fire hydrants shall be located at all intersecting streets and at intermediate locations between intersections at a maximum spacing of 300 feet as measured along the length of the center line of the roadway, and the front of any structure at grade and shall be no further than 400 feet from a minimum of two fire hydrants as measured along the route that a fire hose is laid by a fire vehicle. All buildings shall be within a 300 foot radius of a fire hydrant.

4. PROTECTED PROPERTIES - Fire hydrants required providing a supplemental water supply for automatic fire protection systems shall be within 100 feet of the fire department connection for such system.

5. Fire hydrants shall be installed along all fire lane areas as follows:

   a) <u>Non-Residential Property or Use</u>

      (I). within 150 feet of the main entrance.

      (II). within 100 feet of any fire department connection.

      (III). at a maximum intermediate spacing of 300 feet as measured along the length of the fire lane.

   b) <u>Apartment, Townhouse, or Cluster Residential Property or Use</u>

      (IV). within 100 feet of any fire department connection.

      (V). at maximum intermediate spacing of 400 feet as measured along the length of the fire lane.

6. Generally, no fire hydrant shall be located closer than fifty (50') feet to a non-residential building or structure unless approved by the Town.

7. In instances where access between the fire hydrant and the building which it is intended to serve may be blocked, extra fire hydrants shall be provided to improve the fire protection. Railroads, expressways, major thoroughfares and other man-made or natural obstacles are considered as barriers.

8. Along divided arteries fire hydrants shall be installed on both sides of the roadway so as to preclude the need for laying hose across the roadway.

B. Restrictions

   a. All required fire hydrants shall be as required by the North Central Texas Council of Governments Specifications, Fourth Edition and Addenda and shall be placed on water mains of no less than six (6") inches in size.

   b. Valves shall be placed on all fire hydrant leads.

   c. Required fire hydrants shall be installed so the break away point will be no less than three (3") inches, and no greater than five (5") inches above the grade surface.

   d. Fire hydrants shall be located a minimum of two (2') feet and a maximum of six (6') feet behind the curb line, depending on the location of the sidewalk. The fire hydrant shall not be in the sidewalk.

   e. All required fire hydrants placed on private property shall be adequately protected by either curb stops or concrete filled steel posts or other methods as approved by the Town and shall be in easements. Installation and maintenance of stops or posts to be the responsibility of the landowner on whose property said fire hydrant is placed.

f. All required fire hydrants shall be installed so that the steamer connection will face the fire lane or street, or as directed by the Town.

g. Fire hydrants, when placed at intersections or access drives to parking lots, when practical, shall be placed so that no part of the fire truck will block the intersection or parking lot access when connections to the fire hydrant are made.

h. Fire hydrants, required by this article, and located on private property, shall be accessible to the Fire Department at all times.

i. Fire hydrants shall be located at street or fire lane intersections, when feasible.

j. Fire hydrant bonnet shall be painted according to Standard Details.

1.06 FIRE LINE METERING

Bartonville Water Supply Corporation will own, operate and maintain all fire lines serving fire hydrants. Such fire lines shall be designed and constructed in accordance with the Bartonville Water Supply Corporation standards. Sprinkler service lines, fire line connections and other fire lines which are not maintained by the Town shall be equipped with either a water meter or a detector check assembly having a capacity equal to the required fire flow. Water meters and detector check assemblies shall be constructed in accordance with Town standards.

1.07 MINIMUM COVER

The minimum cover to the top of the pipe must vary with the valve stem. In general, the minimum cover below the street grade should be as follows: 12-inch and smaller, 4.0 feet. Lines larger than 12-inches shall have 5.0 to 6.0 feet of cover. Water lines with more than 6.0 feet of cover shall be approved by the Town and Bartonville Water Supply Corporation.

1.08 CLEARANCES BETWEEN WATER AND WASTEWATER LINES:

Clearances between water and wastewater lines shall meet TCEQ requirements. The minimum clearances for water and wastewater lines crossing storm drains shall be two (2) feet or one-half (0.5) feet when the water or wastewater line is encased.

1.09 METER BOX AND SERVICE

A service with a meter box shall be constructed from the main to a point just behind the curb line, usually in advance of paving. The location of the meter box is as shown on the Utility Assignments detail sheets and as shown on the Town of Copper Canyon Details. On multiple residential and business properties, the desired size and location is usually specified by the owners. Minimum requirements for water service sizes are:

1. One-inch single water services are required to serve all single-family residential lots. Combination meters with service connections to two residences are not allowed.

2. The size of apartment, condominium, or multi-family services will depend on the number of units served with a minimum of one meter per building.

1.10 SERVICE CONNECTIONS

1. Service connections shall not be allowed to fire hydrant leads.

2. Service connections shall not be allowed to transmission mains.

Section II – WASTEWATER

2.01 GENERAL

This section pertains to general design requirements for wastewater collection system construction in the Town of Copper Canyon. The Town of Copper Canyon currently does not have a sanitary wastewater system. If a developer desires to connect to the wastewater collection from Flower Mound, Highland Village, or Upper Trinity, they shall obtain approval from the Town of Copper Canyon and the agency that owns the wastewater collection system. Developers shall comply with the latest wastewater requirements from the agency they propose to connect. In the absence of specific standards, all collection, treatment, and disposal systems shall be designed in accordance with the most current criteria adopted by the Texas Commission on Environmental Quality (TCEQ), Chapter 317, "Design Criteria for Sewerage Systems". Drawings must be submitted to TCEQ for review and approval. Approval letter from TCEQ must be submitted to the Town.

2.02 SEWER LINES

A. Standard sewer line sizes are 8", 12", 15", 18", 21", 24", 27", and 30" diameter. Other sizes must be approved by the Town Engineer.

B. Sewer lines shall be a minimum of 8 inches in diameter.

C. Gravity sewer lines shall be constructed at a minimum depth of 5'. Pressure lines shall be constructed at a minimum depth of four (4) feet. Sewer lines shall be located in the parkway on the south and west side of the roadway and are required to be constructed on both sides of a State Highway. No service lines will be allowed to cross a State Highway. Deviations of these requirements may be approved by the Town Engineer or his designee in circumstances where compliance is not physically feasible. All sewer lines, whether main lines or service lines, crossing existing streets shall be placed by dry boring within an encasement. Open cut excavation will not be allowed to cross existing streets.

D. Easements for sewer line construction shall meet the following requirements:
   1. The easement width shall be a minimum of 15 feet.
   2. If the sewer line is less than 12 feet deep, the outside diameter of the sewer line shall be located a minimum distance of 6 feet from the edge of the easement, and if other utilities are located in the same easement, the outside diameter of the sewer line shall be located a minimum distance of 3 feet from the outside diameter of the other utilities.
   3. If the sewer line is greater than 12 feet deep, the outside diameter of the sewer line shall be located a minimum distance of 9 feet from the edge of the easement, and if other utilities are located in the same easement, the outside diameter of the sewer line shall be located a minimum distance of 6feet from the outside diameter of the other utilities.
   4. Parallel lines will require an additional 5' easement width, a minimum of 6'from deeper line.

E. Sewer line shall be located 3 feet from back of curb or 3’ feet from edge of pavement, opposite side from water line, if there is no curb and gutter.

F. All sewers shall be designed with consideration for serving the full drainage area subject to collection by the sewer in question.

G. Sewers should be designed with straight alignment whenever possible. When horizontal curvatures must be used, the maximum joint deflection should be in accordance with the pipe manufacturer's recommendations and comply with TCEQ requirements.

H. All sewer line installations must extend to the borders of the subdivision or property as required for future extensions of the collection system, regardless of whether such extensions are required for service within the subdivision or property. Must end with a manhole. The amount of trench excavation shall not exceed 200 (two hundred) feet from the end of the pipe laying operations, and no more than 300 (three hundred) feet of total open trench will be allowed. At the end of each work day, all trench excavation shall be backfilled to the end of the pipe laying operation. Barricades, safety fencing, and lights will be required around any open trench left overnight.

I. All sewers shall be designed with hydraulic slopes sufficient to give mean velocities, when flowing full or half full, of no less than two feet (2') per second and no more than ten feet (10’) per second on Kutter's or Manning's formulas using an "n" value of 0.013, at design flow. Design flow, slopes, and velocities shall also conform to (TAC 30, Chapter 317, Design Criteria for Sewage System).

J. Materials
    1. Polyvinyl Chloride (PVC) Pipe
        a) All sanitary sewer pipes shall be PVC pipe type SDR-35 for gravity sewer lines constructed less than twelve feet (12') deep. Type SDR-26 shall be provided where gravity sewer lines exceeds twelve feet (12'). C900 DR18 PVC pipe is required for depths greater than 24'. If service connections are needed on sewer pipe constructed below fifteen feet (15') in depth for a minimum of 500', a parallel line shall be constructed at a shallower depth, specifically for service connections. Ribbed pipe will be allowed on deeper pipe if no service lines are connected to that line. Pressure sewer pipe shall be C900 DR18 PVC.
        b) All gravity PVC sanitary sewer pipe shall be green in color. Pressure sewer pipe shall be white in color.
        c) PVC sewer pipe and fittings shall conform to the current ASTM Designation D 3034 for 8"-15" and ASTM Designation F 679 for greater than 15".

K. Installation
    1. General
        a) Spacing of pipes shall comply with latest TCEQ standards.
        b) All installations shall conform to ASTM Designation D2321, and the latest NCTCOG Specifications as amended by these standards.
        c) Construction shall begin at downstream end of project and continue upstream with bell facing upstream. No upstream piping shall be installed before downstream piping unless approved by the Town Engineer.
        d) When PVC pipe is used for, green marker tape with the wording "Buried Sanitary Sewer" shall be installed in the backfill material no more than twelve inches (12") above the top of the pipe.
        e) The amount of trench excavation shall not exceed 200 (two hundred)

feet from the end of the pipe laying operations, and no more than 300(three hundred) feet of total open trench will be allowed. At the end of each workday, all trench excavation shall be backfilled to the end of the pipe laying operation. Barricades and lights will be required around any open trench left overnight, for any trench within right-of-way or public access easement.

f) Approved plugs shall be installed at the open ends of the line at the end of each working day. All joints shall be assembled free of dirt and any foreign matter.

g) Water jetting, jacking or missile type bores shall not be permitted under any condition.

h) Reference Part B Section V for trench testing.

i) When a 150 psi rated sewer line is required due to its proximity to a water line, the 150 psi rated pipe shall terminate at a manhole on each end. The pipe shall be extended to the interior wall of the manhole. No external boot connection will be allowed.

2.03 MANHOLES

A. Manholes shall be located at all intersections of sewer lines and at intermediate spacing along the line. Generally the maximum spacing should not exceed 500feet. Manholes should be located at all changes in grade and at the ends of all sewer lines that will be extended.

B. A manhole is required at the junction of sewer lines with different inside pipe diameters.

C. A tenth foot (.1') of fall is required through the manhole when a change in flow direction occurs.

D. The flow line into a manhole must not be greater than 2’ above the flow line out of the manhole. Where the flow line in is greater than two feet (2') above the flow line out, a drop inlet is required.

E. Minimum manhole inside diameter is four feet (4').

a. If depth is greater than 12' the minimum diameter shall be 5 feet.

F. Installation

a. Use the following table to determine sanitary sewer manhole sizes:

Table III-1
Minimum Manhole Size

Pipe Sizes	**Diameter of Manhole**
8" through 18"	4'
21" through 30"	5'
33" through 48"	6'
>48	Special Design

Note:

1. If the proposed design requires the sewer line to be placed at depths greater than shown above, the design will require approval by the Town Manager or his designee.
2. The clear distance between the outside of adjacent pipes shall not be less than two feet.

G. Drop-connection manholes shall have a minimum inside diameter of five feet (5'), with a single interior drop connection. Drop MHS for lines over 12' in-depth or drop MHS with more than 1 interior drop. Must be a minimum of 6' in diameter.

H. Materials
All manholes shall be constructed of cast-in-place or precast concrete.

1. Cast-in-place Manhole
   a) Minimum cast in place manhole wall thickness is eight inches (8"). For depth's greater than 20 feet a special design will be required.
   b) The manhole foundation shall be poured on undisturbed soil and shall have a minimum thickness of eight inches (8").
   c) The inlet and outlet pipes shall be poured into the foundation of the manhole. The pipe shall extend one-and-one-half inches (1 1/2") into the manhole.
   d) The invert shall be shaped and smoothed so that no projections will exist and the invert shall be self cleaning. The invert floor shall have a slope of one-inch (1") per foot.
   e) Concrete work shall conform to all requirements of ACI 301, Standard Specification for Structural Concrete, published by the American Concrete Institute, except as modified herein.
   f) Detailing of concrete reinforcement and accessories shall be in accordance with ACI Publication 315.
   g) Portland Cement shall be Type II, low-alkali and conform to ASTM Designation C-150.
   h) The manhole shall not be backfilled within 12 hours after the concrete placement.
   i) The face of curb shall be sawed with an "MH" to mark the location of all manholes. The location of the stamp shall be a line that intersects the center of the manhole cover and the curb perpendicular to the centerline of the street. For manholes located in intersections, the curb shall be stamped at the closest location to the manhole.

2. Precast Manhole
   a) Minimum pre-cast wall thickness is 5".
   b) Precast manholes shall be constructed in accordance to ASTM Designation C-478.
   c) Manhole base shall have a spread footing and be placed on a minimum of twelve-inches (12") of ¾" crushed rock.

2.04 <u>Manhole Frame and Cover</u>

A. Cover

1. Materials
   All manhole covers shall include Town approved brass ID marker.
   All manhole covers shall conform to the Standard Specifications for Ductile Iron Castings, ASTM A536.

2. Installation
   i. All manhole covers shall be 32-inches in diameter.
   ii. Manhole covers shall indicate "Sanitary Sewer".

3. Manufacturers
   i. Certain Teed PAMREX
   ii. Power Seal

B. Frames
   1. Materials
      All manhole frames shall conform to the Standard Specifications for Ductile Iron Castings, ASTM A536.

C. Extension Ring
   1. Materials
      All precast reinforced concrete extension rings shall conform to ASTM C-478.
   2. Installation
      i. The number of extension ring sections shall be kept to a minimum (i.e. Use 1-12" extension ring instead of 2-6" extension rings).
      ii. A 1" x 3 ½" bitumastic gasket shall be used to seal the extension ring at both joints.
      iii. Reference latest NCTCOG specs for max height of neck and minimum opening diameter.

2.05 <u>Sewer Service</u>

A. No sewer service line (lateral) shall be less than 4" in nominal diameter. Commercial sewer laterals shall be 6" minimum diameter.

B. Sewer laterals shall be located at the center of the lot and extended to the property line and be a minimum of 10 feet downstream of the water service. The end of the lateral shall have a green detector pad with tape extending up to final grade.

C. Sewer service laterals shall have no more than 6' of cover at the property line.

D. Materials
   1. Polyvinyl Chloride (PVC) Pipe
      i. All lateral sewer service lines shall be PVC pipe type SDR-35.
      ii. All PVC sanitary sewer pipe used for lateral services shall be green in color.

E. Installation
   2. Polyvinyl Chloride (PVC) Pipe
      i. All service laterals shall be installed in accordance with the sanitary sewer embedment and backfill standards.
      ii. All service laterals below proposed area to be paved shall be installed and properly backfilled prior to the subgrade preparation and pavement construction.
      iii. All lateral locations shall be saw-cut into the curb with an "II" at the point the lateral crosses the curb with 4” high lettering painted green. The lateral indicator mark shall be placed at the edge of pavement when there is no curb and gutter.

2.06 <u>Cleanouts</u>

A. Residential Service Line cleanouts shall be placed at property line, and to final grade prior to acceptance of subdivisions. Cleanouts shall not be placed in future sidewalk location.

B. Materials

1. Polyvinyl Chloride (PVC) Pipe
    i. All cleanouts are to be constructed of PVC pipe type SDR-35.
    ii. All PVC sanitary sewer pipe shall be green in color.
    iii. PVC sewer pipe and fittings shall conform to the current ASTM Designation D 3034 for 4"-15" and ASTM Designation F 679 for greater than 15".

2.07 Main Line Cleanouts

Main line cleanouts are not allowed.

2.08 Aerial Sewer

A. The piers for the aerial crossing shall be designed in accordance with the guidelines of the Ductile Iron Pipe Research Association.

B. Aerial sewer crossing shall be located in areas where the sewer line can not be constructed with the appropriate minimum cover. The design engineer shall design the aerial crossing in accordance with these standards and as approved by the Town Manager or his designee.

C. Pier placement and spacing shall be determined according to soils analysis performed by a geotechnical engineer. Piers shall be placed at a maximum span distance as indicated by the design engineer's calculations.

D. Pier placement and spacing along with a soils report shall be submitted to the Town Engineer.

E. Materials

1. Pipe
    i. All above ground sewer installations shall be ductile iron, minimum Class150, utilizing restrained joints and shall have a wall thickness required forth size and span as designed or approved alternate. The pipe shall haven internal polyurethane coating.
    ii. The aerial pipe shall be connected to the sanitary sewer pipe by means of a manhole on each side of the aerial crossing.
2. Piers
    Piers to be constructed with a minimum of Class A 3,000 psi reinforced concrete.

F. Installation

1. Pipe
    The design engineer shall submit a pipe design for approval by the Town Engineer.

2. Piers
    The design engineer shall submit a pier design for approval by the Town Engineer.

2.09 Lift Stations

A. Instrumentation and Control

1. The voltage supplied for pump operation shall be 3 phase, 480 volts. Converting single phase power to three phase power using additional mechanical equipment shall not be allowed.
2. Wet well level indication shall be accomplished through use of an ultra sonic level

sensing device. The developer shall comply with the standards of the agency they are connecting the wastewater. Currently the Town of Flower Mound standard for this item is the Milltronics HydroRanger.

3. The current standard for the Town of Flower Mound is Motorola MOSCAD Remote Terminal Unit (RTU) shall be installed at all lift stations. Programming of the RTU is the responsibility of the contractor, and shall be coordinated with the Town. The Motorola RTU shall be a “smart” RTU, utilizing a micro-compressor for communications, calculations, local control, and data storage. The RTU shall be modular capable of receiving and transmitting control messages to and from the MASTER STATION via the existing repeater. An integral radio transceiver, shall be provided for direct communication with the MASTER STATION via the existing repeater. The RTU shall be the MOTOROLA ACE 3600. Substitutions will not be considered, unless the town has changed their RTV system.
4. Submersible pumps shall be provided with moisture and motor over temperature sensors.
5. Discharge flow from the lift station shall be measured by using a magnetic flow meter. The meter manufacturer installation requirements shall be followed to ensure accuracy of flow measurements. The meter shall be placed in a concrete vault with an aluminum access door rated for the anticipated traffic load. Combining the meter vault with the valve vault is acceptable. The vault will be designed to drain or pump water accumulation in the vault to the wet well. The Magneto flow® Mag Meter manufactured by Badger Meter Inc., is an approved model.

B. Pumps, Piping, Valves and Wells

1. Pumps shall be sized to operate at optimum efficiency. Minimum acceptable efficiency at the operating point will be sixty percent (60%).
2. Each pump discharge must have a cushioned swing check valve and isolation valve.
3. Inlet piping shall be designed to minimize turbulence.
4. Valves shall be located in a separate vault from the wet-well.
5. The valve vault shall have a concrete floor with a drain line connecting the vault to the wet-well. The drain line shall have a casketed flap valve at the discharge into the wet-well to prevent wet-well contents from entering the valve vault.
6. Wet-well working volume shall be sized to allow for the recommended pump cycle of 6 minutes for each pump, with no more than 10 starts per hour.
7. Lift station piping shall be designed with an additional emergency pump connection, allowing the station to be operated with the primary pump(s) out of service for an extended period of time.

C. Site Requirements

1. All lift station sites are required to have a minimum 6 foot chain link fence. Fencing specifications are as follows:
   a) Fence fabric shall be hot dip galvanized 9 gauge steel.
   b) Three strands of hot dip galvanized barbed wire are required above the top rail and must terminate at the corner posts with brace bands.
   c) A 12' double gate is required for vehicle traffic.
   d) Posts shall be schedule 40 hot dip galvanized steel. Post shall be placed in concrete.
2. A concrete pad will be required at the front of the control cabinet. The pad shall provide a 3' working area away from the face of the cabinet and extend the width of the enclosure mounting structure. Pad depth shall be a typical 4".
3. Crushed stone will be required inside the fenced area of the station. This requirement includes a water penetrating weed barrier covered with a minimum

crushed stone in accordance with NCTCOG Item 2.1.8.(d).

4. 12" x 6" concrete perimeter curb is required to contain the crushed stone. The curb shall be 6" in width and extend approximately 6" below finished grade.
5. A potable water service shall be provided at the station site. A one inch service with a one inch angle stop and a RPZ back-flow preventer shall be installed in an appropriately sized meter box.
6. The site shall be graded to drain away from the station to prevent stormwater inflow or infiltration into the wet-well.
7. The site shall be located outside of the 100-year flood plain.
8. The site shall not be located within 100 feet of an existing or proposed residence.
9. The lift station site shall include a driveway area for maintenance vehicles to park off public roadway while performing maintenance. The minimum driveway length shall be 15 feet.
10. A concrete driveway turning area is required where access drives extend more than 20 feet from main roads. The driveway area shall be "T" shaped with the applicable turning radius. The minimum driveway width shall be 15feet.
11. Lift stations are prohibited from placement within public right-of-way or easements.

D. Materials

1. Instrumentation and Control
    a) All enclosures, with the exception of the metering base, shall be NEMA 4X stainless steel.
    b) All electric conduit will be epoxy coated rigid steel or aluminum rigid.
2. Pump Accessories, Piping, Valves and Wells
    a) Wet-well interior piping shall be fabricated of flanged Class 150 Ductile Iron.
    b) Fasteners used for pipe connection shall be 318 stainless steel.
    c) Pump guide bars, guide bar brackets, cable/chain hooks and pump lifting chains shall all be fabricated of stainless steel.
    d) Pump access door, and door frames must be fabricated from aluminum.
    e) Isolation valves shall be resilient seated gate valves in accordance with the water standards.
    f) Swing check valves shall be in accordance with AWWA C-508. Eight inch (8") and larger check valves shall be equipped with a bottom mounted oil dash pot.
    g) Wet wells shall be constructed of concrete. Fiberglass or steel wet wells are not acceptable.
    h) The wet well interior concrete surface shall be coated with TNEMEC series 218 MotarClad surfacing material, followed by TNEMEC series 436Perma-Sheield FR at 125 mills dry film thickness, or Town approved equivalent. Application shall be per manufacturer recommendation.
    i) All submersible wet well pumps shall be equipped with an automatic mixing/flushing valve attached to the pump volute. This accessory item will direct a water jet across the floor of the wet well to temporarily suspend settled materials. This valve will operate by using the hydraulic energy created by the operation of a pump. An additional mixer needed to suspend settled material will not be accepted.

E. Installation

1. Instrumentation and Control
    a) All stations shall be equipped with a 12 volt/DC flashing strobe placed at an elevation visible by passing traffic.

b) Stations shall be equipped with radio equipment compatible with the Town of Flower Mound existing SCADA equipment. Antenna mountings, masts, and cables shall be provided for continuous and accurate telemetry and control.

c) All stations will be equipped with a magnetic flow meter located on the discharge pipe. The flow meter shall be contained in a concrete vault with an aluminum access door. Installation of the meter shall be in accordance with manufacturer's installation recommendations to ensure accuracy of flow valves.

d) Modifications to the existing SCADA system will be required with the addition of any new station. The installing contractor shall provide the following information to the Town prior to beginning this modification process:

(a) The general contractor shall provide evidence that the instrumentation subcontractor has maintained a continuous business operation for at least five (5) years.

(b) The general contractor shall provide evidence that the instrumentation subcontractor maintains a staff of competent technicians and Licensed installers who are Motorola factory trained.

(c) The general contractor shall provide evidence that the instrumentation subcontractor maintains a fully staffed service shop for supplying demand service calls on systems and maintains an office of operation within a reasonable response distance (generally in the Texas Counties of Denton, Tarrant, or Dallas) to the Town of Flower Mound Wastewater Treatment Plant.

(d) The creation of new Human-Machine Interface (HMI) SCADA display screens, and modifications of existing HMF displays, must be proposed and approved prior to installation.

e) Enclosures shall be mounted on an appropriately sized mounting structure. Mounting structures shall be constructed of 6" x 2" x 0.25" hot dip galvanized steel channel stock. Intersections shall be bolted, not welded, with stainless steel fasteners. Aluminum or epoxy coated steel unistrut may be attached to the mounting structure to facilitate placement of enclosures. The legs of the mounting structure shall be set at least 24"below grade and be encased in concrete.

2. Pumps, Piping Valves and Wells

a) Pump access doors and door frames must be fabricated from aluminum with a recessed lifting handle, locking lever to hold the door in the open position, and a method of placing a No. 5 Master padlock on the door for safety and security.

b) All submersible wet well pumps shall be equipped with an automatic mixing/flushing valve attached to the pump volute. This accessory item will direct a water jet across the floor of the wet well to temporarily suspend settled materials. This valve will operate by using the hydraulic energy created by the operation of a pump. An additional mixer needed to suspend settled material will not be accepted.

2.10 Testing Procedures

A. Sewer Lines

1. A Town inspector shall be in attendance for each testing procedure. Testing shall be performed by a company certified by the pipe manufacturer.
2. Deflection Testing. Upon completion of sanitary sewer pipe installation, the contractor shall pull a mandrel through the pipe to test for a maximum 5% deflection, unless otherwise specified.
3. Video Inspection. Before acceptance of a subdivision or project by the Town, the contractor will be required to retain a qualified company to perform a video inspection of the sewer mains in the subdivision at the contractor's expense. Prior to video inspection, sewer mains shall be flushed. The video inspection shall be done no sooner than ten days prior to final acceptance of the project.
4. Criteria for Repair and Reinspection:
   - i. Pulled or slipped joints
   - ii. Water infiltration
   - iii. No standing water will be permitted in sewer lines with a slope greater than or equal to 1%. Standing water shall be permitted in sewer lines with a slope less than 1% to a maximum depth of 20% of the nominal inside diameter of the pipe.
   - iv. Structural damage to the pipe
   - v. The Town will make the final determination if repairs are required. A final set of tapes and logs shall be given to the designated Town Inspector of the project. Furnish two copies of audio/video inspection in DVD format. By audio on tape, the operator must note the following:
     - (a) Date and time of recording
     - (b) Developer's or contractor's name
     - (c) Project name and contract number
     - (d) Name of company performing the inspection
     - (e) The location of line, designation, main size, direction of run, identify every 50-foot station, and identify the station of each manhole.
5. Air Testing

   Air Testing shall be in accordance with NCTCOG requirements.

B. Manholes

A Town inspector shall be in attendance for each testing procedure.
Vacuum Testing shall be in accordance with latest NCTCOG requirements.

TOWN OF COPPER CANYON
ENGINEERING DESIGN MANUAL

PART IV
STORM WATER MANAGEMENT

OCTOBER 2022

This document references the regional 2006 *i*SWM Manual prepared by the North Central Texas Council of Governments. The original iSWM document has been revised to present Town of Copper Canyon Storm Water Management Criteria. Some of the *i*SWM sections which are not currently adopted are available in the Appendices for technical reference, utilization by developers for enhancement of land development projects, and potential future adoption by the Town, as needed.

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INTRODUCTION

This design manual is needed to update the policies and criteria for storm water facilities within the Town of Copper Canyon and its extraterritorial jurisdiction. New policies and criteria are needed to reflect the changes that have occurred in community standards, technology and environmental regulations that impact storm water management. The primary motivation for this new manual is to guide the community in drainage policy and criteria so that new development does not increase flooding, erosion, and water quality problems.

This manual is intended to provide a guideline for the most commonly encountered storm water or flood control designs in the Town of Copper Canyon. It can also be used as a guide for watershed master plans and for design of remedial measures for existing facilities. This manual was developed for users with knowledge and experience in the applications of standard engineering principles and practices of storm water design and management. There will be situations not completely addressed or covered by this manual. Any variations from the practices established in this manual must have the approval of the Town Engineer. Close coordination with the staff of the Town is recommended and encouraged during the planning, design and construction of all storm water facilities. This Storm Water Management Design Manual is adopted and becomes effective on October 8, 2012.

Relationship of Town of Copper Canyon Manual to Regional *integrated* Storm Water Management (*i*SWM) Manual

The Town of Copper Canyon design manual references the regional 2006 *i*SWM manual, developed by the North Central Texas Council of Governments (NCTCOG). The 2006 *i*SWM manual was prepared for the 16-county north central Texas region (and includes sections that are not being adopted or are being modified by the Town of Copper Canyon. The digital version of this manual is on the Town of Copper Canyon (www.coppercanyon-tx.org). Copies of these documents can be downloaded from the Town's website or ordered from the Town for the cost of reproduction.

Notes and Abbreviations

Notes and abbreviations used in the Local Criteria Section:

1. Town - Town of Copper Canyon

Contact Information

Contacts for the Town of Copper Canyon Storm Water Management Design Manual can be reached at the Engineering Department at: 940-241-2677 or at the website: www.coppercanyon-tx.org.

References

integrated Storm Water Management Design Manual for Site Development, 2006 Edition. NCTCOG, Arlington, TX.

Note: Additional references are included in individual chapters or appendices.

GOALS AND OBJECTIVES OF THE TOWN OF COPPER CANYON STORM WATER MANAGEMENT PROGRAM

1. Establish and implement drainage policy and criteria so that new development does not create or increase flooding problems, cause erosion or pollute downstream water bodies.

2. Facilitate the continuation of comprehensive watershed planning that promotes orderly growth and results in an integrated system of public and private storm water infrastructure.

3. Minimize flood risks to citizens and properties, and stabilize or decrease streambank and channel erosion on creeks, channels, and streams.

4. Improve storm water quality in creeks, rivers, and other water bodies, remove pollutants, enhance the environment and mimic the natural drainage system, to the extent practicable, in conformance with the Texas Pollutant Discharge Elimination System (TPDES) permit requirements.

5. Support multi-use functions of storm water facilities for trails, green space, parks, greenways or corridors, storm water quality treatment, and other recreational and natural features, provided they are compatible with the primary functions of the storm water facility.

6. Encourage a more standardized, integrated land development process by bringing storm water planning into the conceptual stages of land development.

TOWN OF COPPER CANYON STORM WATER POLICY STATEMENTS

1. All development within the Town of Copper Canyon Town Limits or Extra-territorial Jurisdiction (ETJ) shall include planning, design, and construction of storm drainage systems in accordance with this Storm Water Management Design Manual, and Subdivision Ordinance.

2. Conceptual, Preliminary and Final Drainage Studies and Plans may be required for proposed developments within the Copper Canyon Town Limits or its ETJ, in conformance with this Storm Water Management Design Manual. Specific submittal requirements depend on the complexity of the project and requirements of the Subdivision Ordinance and Zoning Ordinance. The checklists for each stage of this three-tier process are included in the Appendix C.

3. All drainage related plans and studies shall be prepared and sealed by a Licensed Professional Engineer with a valid license from the State of Texas. The Engineer shall attest that the design was conducted in accordance with this Storm Water Management Design Manual.

4. For currently developed areas within the Town of Copper Canyon with planned re-development, storm water discharges and velocities from the project should not exceed discharges established by procedures presented in this manual but also shall not exceed discharges and velocities from current (existing) developed conditions, unless the downstream storm drainage system is designed (or adequate) to convey the future (increased) discharges and velocities.

5. All drainage studies and design plans shall be formulated and based upon ultimate, fully developed watershed or drainage area runoff conditions. In certain circumstances where regional detention is in place or a master plan has been adopted, a development may plan to receive less than ultimate developed flow from upstream areas with the approval of the Town Engineer. The rainfall frequency criteria for storm water facilities, as enumerated within this Storm Water Management Design Manual, shall be utilized for all drainage studies and design plans.

6. Proposed storm water discharge rates and velocities from a development shall not exceed the runoff from existing, pre-development conditions, unless a detailed study is prepared that demonstrates that no unacceptable adverse impacts will be created. Adverse impacts include: new or increased flooding of existing structures, significant increases in flood elevations over existing roadways, unacceptable rises in base flood elevations or velocities, and new or increased stream bank erosion or increased occurrence of nuisance flows.

7. If a proposed development drains into an improved channel or storm water drainage system designed under a previous Town of Copper Canyon drainage policy, then the hydraulic capacities of downstream facilities must be checked to verify that increased flows, caused by the new development, will not exceed the capacity of the existing system or cause increased downstream structure flooding. If there is not sufficient capacity to prevent increased downstream flooding, then detention or other acceptable measures must be adopted to accommodate the increase in runoff due to the proposed development.

8. Storm water runoff may be stored in detention and retention basins to mitigate potential downstream problems caused by a proposed development. Proposed detention or retention basins shall be analyzed both individually and as a part of the watershed system, to assure compatibility with one another and with the Town's storm water management master plans for that watershed (if available). Storage of storm water runoff, near points of rainfall occurrence, such as the use of parking lots, ball fields, property line swales, parks, road embankments, borrow pits and on-site ponds is desirable and encouraged.

9. Alternatives to detention or retention for mitigation of potential downstream problems caused by proposed development include: acquisition of expanded drainage easements, ROW, or property owner agreements; downstream channel and/or roadway bridge/culvert improvements or stream

bank erosion protection. These alternatives will be considered by the Town Engineer on a case-by-case basis.

10. All proposed developments within the Copper Canyon Town Limits or its ETJ shall comply with all local, county, state and federal regulations and all required permits or approvals shall be obtained by the developer.

11. The policy of the Town is to avoid substantial or significant transfer of storm water runoff from one basin to another and to maintain historical drainage paths whenever possible. However, the transfer of storm water from basin to basin may be necessary in certain instances and will be reviewed and a variance can be made by the Town Engineer in accordance with established variance procedures.

CHAPTER 1 – STORM WATER MANAGEMENT SYSTEM PLANNING AND DESIGN

Chapter 1 of this Manual provides a foundation for Storm Water Management in terms of basic philosophy, principles, definitions, and land development site planning and design practices, and should therefore be utilized for general guidance throughout the development process. Any reference to Water Quality and Streambank Protection Volume controls are not adopted by Town of Copper Canyon at this time.

Section 1.1 – Storm Water Management Planning

Depending on the complexity of the project or submittal requirements as dictated in the Subdivision Ordinance and the Zoning Ordinance, storm water management plans may be prepared and submitted to the Town of Copper Canyon in the progressive planning stages of a land development project with the Conceptual Storm Water Management Plan and Preliminary and Final Plans. The Conceptual Plan is an important consideration in that it allows the developer and their design engineer to propose a potential site layout and gives Town staff the opportunity to comment on a storm water management plan concept prior to significant planning and design effort on the part of the design engineer.

1.1.1 Conceptual Storm Water Management Plan

Based upon the review of existing conditions and site analysis, the design engineer should develop a Conceptual Storm Water Management Plan for the project. During the concept plan stage the site designer will develop a conceptual layout of the site and its storm water management system design and layout. The Conceptual Plan allows the design engineer to propose a potential site layout and gives the developer and the Town of Copper Canyon a “first look” at the storm water management system for the proposed development. The Conceptual Plan should be submitted to the Town before detailed preliminary site plans are developed.

It is extremely important at this stage that storm water design is integrated into the overall site design concept in order to best reduce the impacts of the development as well as provide for the most cost-effective and environmentally sensitive approach. Using hydrologic calculations, the goal of mimicking pre-development conditions can serve a useful purpose in planning the storm water management system.

The following steps should be followed in developing the Conceptual Plan with the help of the Checklist for Conceptual Plan found in Appendix C of this manual:

1. Use *storm water management* design practices as applicable to develop the site layout, including:
   - Preserving the natural feature conservation areas defined in the site analysis
   - Fitting the development to the terrain and minimizing land disturbance
   - Reducing impervious surface area through various techniques
   - Preserving and utilizing the natural drainage system wherever possible
2. Calculate conceptual estimates of the design requirements for flood control based on the conceptual plan site layout
3. Perform screening and conceptual selection of appropriate structural storm water controls and identification of potential siting locations

4. Existing conditions:
   - Copy of applicable digital orthophotos showing proposed project boundaries
   - A topographic map of existing site conditions (no greater than a 2-foot contour interval recommended) with drainage basin boundaries indicated and project boundaries shown
   - Total size area (acres)
   - Benchmarks used for site control
   - Perennial and intermittent streams
   - Mapping of predominant soils from USDA soil surveys
   - Boundaries of existing predominant vegetation
   - Location and boundaries of other natural feature protection and conservation areas such as wetlands, lakes, ponds, floodplains, stream buffers and other setbacks (e.g., drinking water well setbacks, septic setbacks, etc.)
   - Location of existing roads, buildings, parking areas and other impervious surfaces
   - Existing utilities (e.g., water, sewer, gas, electric) and easements
   - Location of existing conveyance systems such as grass channels, swales, and storm drains
   - Flow paths
   - Location of floodplain/floodway limits and relationship of site to upstream and downstream properties and drainages
   - Location and dimensions of existing channels, bridges or culvert crossings
5. Conceptual Site Layout
   - Complete the Conceptual Plan Checklist
   - Hydrologic analysis to determine conceptual runoff rates, volumes, and velocities to support selection of Storm Water Controls
   - Conceptual estimates of three (3) storm design approach requirements
   - Conceptual selection, location, and size of proposed structural storm water controls
   - Conceptual limits of proposed clearing and grading
6. Submit to Town of Copper Canyon for review and comment

1.1.2 Preliminary Storm Water Management Plan

The Preliminary Storm Water Management Plan ensures that requirements and criteria are being complied with and opportunities are being taken to minimize adverse impacts from the development. This step builds upon the data developed in the Conceptual Plan by refining and providing more detail to the concepts identified.

The Preliminary Plan should consist of maps, narrative, and supporting design calculations (hydrologic and hydraulic). The preliminary drainage study associated with this plan will include a downstream assessment of properties that could be impacted by the development. These studies will include adequate hydrologic analysis to determine the existing, proposed, and fully-developed runoff for the drainage area that is affected by the proposed development and will include hydraulic studies that define the “adequate outfall”. The development storm water management plan shall address existing downstream, off-site drainage conveyance system(s); and shall define the discharge path from the outlet of the on-site storm water facilities to the off-site drainage system(s) and/or appropriate receiving waters. See Section 2.1.8 ("Downstream Hydrologic Assessment") for guidance on the details of this downstream assessment. As a minimum, the Town of Copper Canyon requires assessment of the 2-, 10-, and 100- year 24- hour events. This preliminary drainage study and storm water management plan will include:

1. A topographical map of the entire watershed (not just the area of the proposed development) generally not smaller than 1"=200' (or other such scale approved by the Town Engineer), delineating the watershed boundary(s) and runoff design point(s), existing and proposed land use and zoning, and the size and description of the outfall drainage facilities and receiving streams.
2. Computation tables showing drainage areas, runoff coefficients, time of concentration, rainfall intensities and peak discharge for the required design storms, for both existing and proposed (ultimate development) conditions, at all design points for each component of the storm water system (streets, pipes, channels, detention ponds, etc.).
3. Any proposed changes to watershed boundaries (i.e. by re-grading, where permissible by Texas Water Code). If significant changes to watershed boundary are made, more extensive analyses of downstream impact and mitigating detention will be required and a variance obtained from the Town of Engineer.
4. FEMA Flood Hazard Areas, if applicable.
5. In addition any required Corps of Engineer's Section 404 permits, Conditional Letters of Map Revision (CLOMR), Letters of Map Revision (LOMR) or other permits relating to lakes and streams required by any federal, state or local authorities. These must be documented in the Drainage Study.
6. Detailed off-site outfall information. This shall include the presence of existing or proposed drainage structures, bridges or systems; documentation of existing versus proposed developed site as well as ultimate runoff, identification of downstream properties which might be impacted by increased runoff, and proposed detention or other means of mitigation. Downstream impacts shall generally be delineated to a point where the drainage from the proposed development has no impact on the receiving stream or on any downstream drainage systems within the "zone of influence".
7. Report with technical documentation.

The completed Preliminary Plan (including Checklist in Appendix C) should be submitted to the Town for review and comment.

1.1.3 Final Storm Water Management Plan

The Final Storm Water Management Site Plan adds further detail to the Preliminary Plan and reflects changes that are requested or required by the local review authority. The Final Plan should include all of the revised elements of the Preliminary Plan as well as the following items:

A Final Drainage Study and Storm Water Management Plan for development of all or a portion (i.e. phase one or phase two, etc.) of the overall development shall be prepared and submitted to the Town of Copper Canyon. This submittal shall generally include the information listed below.

1. Conformance with the Preliminary Storm Water Management Plan and Study.
2. Submission of detailed drainage calculations and detailed design plans.
3. The submission of a cover sheet signed by the Town Engineer indicating the approval of the detailed construction drawings for the proposed development is sufficient to clear a plat drainage study comment.

4. Final drainage studies shall be approved based on the submission of a signed cover sheet and drainage map with calculations from the approved engineering construction drawings. Where Town approval of construction plans is not required, the above information required for preliminary drainage studies, as well as construction plans for any drainage improvements, prepared according to criteria in the current Town of Copper Canyon plan review checklists, shall be submitted.

5. Note that unless specifically approved in a Floodplain Development Permit issued through the Town Engineer, no work may be performed in the FEMA regulatory floodway without a FEMA-approved Conditional Letter of Map Revision (CLOMR). No development activities may occur in the FEMA regulatory floodplain without an approved Floodplain Development Permit.

6. Description and copies of any applicable federal, state, and/or local environmental permits such as USACE Regulatory Program permits, 401 water quality certification, or construction TPDES permits. Permits must be obtained prior to or in conjunction with final plan submittal, including:

7. Notice of Intent (NOI) or Construction Site Notice, as appropriate, for TPDES permits

8. Permits obtained for any other storm water related development requirements (i.e. USACE Regulatory Program permits, erosion control, grading, water rights permits, TCEQ dam safety, etc.)

9. Description of waiver requests

The completed Final Storm Water Management Plan (including Checklist in Appendix C) should be submitted to the Town for final approval prior to any construction activities on the development site.

1.1.4 Local Government Responsibilities during Construction and Operation

The Town of Copper Canyon Process includes:

Construction Phase

1. Pre-construction Meeting - Where possible, a pre-construction meeting shall occur before any clearing or grading is initiated on the site. This step ensures that the owner-developer, contractor, engineer, inspector, and plan reviewer can be sure that each party understands how the plan will be implemented on the site.
2. Periodic Inspections - Periodic inspections during construction by Town of Copper Canyon representatives. Inspection frequency may vary with regard to site size and location; however, monthly inspections are a minimum target.
3. Final Inspection - A final inspection is needed to ensure that the construction conforms to the intent of the approved design. Prior to accepting the infrastructure components, issuing an occupancy permit, and releasing any applicable bonds, the Town of Copper Canyon will ensure that: (a) temporary erosion control measures have been removed; (b) storm water controls are unobstructed and in good working order; (c) permanent vegetative cover has been established in exposed areas; (d) any damage to natural feature protection and conservation areas have been mitigated; (e) conservation areas and buffers have been adequately marked or signed; and (f) any other applicable conditions have been met.
4. Record Drawings - Record drawings of the structural storm water controls, drainage facilities, and other infrastructure components will be provided to the Town of Copper Canyon by the developer.

Maintenance

1. Maintenance Plan - If private maintenance is planned, a maintenance plan, prepared by the developer, will outline the scope of activities, schedule, costs, funding source, and responsible parties. Vegetation, sediment management, access, and safety issues will be addressed.
2. Notification of Property Owners - If necessary, the Town of Copper Canyon will notify property owners of any maintenance responsibilities, through a legal disclosure, upon sale or transfer of property. Ideally, preparation of maintenance plans should be a requirement of the Plan preparation and review process.
3. Ongoing Maintenance – it will be clearly detailed in the Final Storm Water Management Plan which entity has responsibility for operation and maintenance of all structural storm water controls and drainage facilities.
4. Annual Inspections - Annual inspections of private storm water management facilities will be conducted by the owner.

Section 1.2 –Planning and Design Approach

In general, the Town of Copper Canyon currently follows the flood control and streambank protection components of the planning and design approach. Streambank protection is a requirement in Copper Canyon, but there is not a standard requirement to provide extended release detention for the streambank protection volume. Post construction water quality protection is not currently a standard requirement in Copper Canyon. However, the Town encourages land developers to consider the use of post construction water quality measures as regulations may be developed to comply with the Town's MS4 permit.

1.2.1 Introduction

This section presents an integrated approach for meeting the storm water runoff quality and quantity management goals by addressing the key adverse impacts of development on storm water runoff. The purpose is to provide guidance for designing a comprehensive storm water management system as part of the Storm Water Management Plan to:

- Remove pollutants in storm water runoff to protect water quality;
- Regulate discharge from the site to minimize downstream bank and channel erosion; and
- Control conveyance of runoff within and from the site to minimize flood risk to people and property.

The Design Approach is a coordinated set of design standards that allow the site engineer to design and size storm water controls to address these goals. Each of the Design Steps should be used in conjunction with the others to address the overall storm water impacts from a development site. When used as a set, the Design Approach controls the entire range of hydrologic events, from the smallest runoff-producing rainfalls up to the 100-year, 24-hour storm. The design approach for each of the goals above is summarized in Table 1.2-1.

Table 1.2-1 Steps for *integrated* Design Approach for Storm Water Control and Impact Mitigation

Steps	Approach
Step 1: Downstream Assessment	Conduct a downstream assessment to the point at which the discharge from the proposed development no longer has a significant impact upon the receiving stream or storm drainage system.

Table 1.2-1 Steps for *integrated* Design Approach for Storm Water Control and Impact Mitigation	
Step 2: Water Quality Protection	Achieved using one or a combination of the following options: (1) Reduce imperviousness by using Site Design Practices; (2) Provide approved community storm water pollution prevention programs/activities as designated in an approved TPDES Storm Water permit.
Step 3: Streambank Protection	Provide streambank protection from erosion due to increased storm water volumes and velocities caused by development using one of the following options: (1) Document acceptable downstream conditions; (2) Improve downstream conditions; (3) Maintain existing downstream conditions.
Step 4: Flood Control	Onsite: Minimize localized site flooding of streets, sidewalks, and properties by a combination of on-site storm water controls and conveyance systems. These systems will be designed for the "streambank protection" and "conveyance" storm event frequencies. Depending upon their location, function, and the requirements of the local jurisdiction, the full build-out 100-year storm event is to be conveyed on-site such that no resulting habitable structural flooding occurs. Downstream: Based on the downstream assessment, manage downstream flood impacts caused by the increase of storm water discharges from the development using one of the following options: (1) Document acceptable downstream conditions; (2) Improve downstream conditions; (3) Maintain existing downstream conditions; (4) Maintain existing on-site conditions. Flood impact reduction may be achieved by a combination of on-site control, downstream protection, floodplain management, and/or other mitigation measures.

1.2.2 Downstream Assessment

The downstream assessment described in Section 2.1.8 of the Manual will include the necessary hydrologic and hydraulic analyses to clearly demonstrate that the limits of the Zone of Influence have been identified, and that along the drainage route to that location, these parameters are met:

- No new or increased flooding of existing structures for 2-,10- and 100-year floods.
- No significant increases (0.1'or less) in flood elevations over existing roadways for the 2-, 10- and 100-year floods.
- No significant rise (0.1' or less) in 100-year flood elevations, unless contained in existing channel, roadway, drainage easement and/or R.O.W.
- No significant increases(maximum 5%) in channel velocities for the 2-, 10- and 100-year floods. Post-development channel velocities cannot be increased more than 5% above pre-development velocities or exceed the applicable maximum permissible velocities shown in tables 4.4-2 and 4.4-3. Exceptions to these criteria will require certified geotechnical/geomorphologic studies that provide documentation those higher velocities will not create additional erosion.
- No increases in downstream discharges caused by the proposed development that, in combination with existing discharges, exceeds the existing capacity of the downstream storm drainage system.

1.2.3 Flood Control

The intent of the flood control criteria is to provide for public safety; minimize on-site and downstream flood impacts from the "streambank protection", "conveyance", and 100-year storm events; maintain the boundaries of the mapped 100-year floodplain; and protect the physical integrity of the on-site storm water controls as well as the downstream storm water and flood control facilities.

Flood control analyses are based on the following three (3) storm events. The storm frequency for each event is listed below.

- *"Streambank Protection":* 2-year, 24-hour storm event
- *"Conveyance"*: 25-year, 24-hour storm event
- 100-year, 24-hour storm event

Flood control must be provided for on-site conveyance, as well as downstream outfalls as described in the following sections.

1.2.3.1 On-Site Conveyance

The "streambank protection" and "conveyance" storm events are used to design standard levels of flood protection for streets, sidewalks, structures, and properties within the development. This is typically handled by a combination of conveyance systems including street and roadway gutters, inlets and drains, storm drain pipe systems, culverts, and open channels. Other storm water controls may affect the design of these systems.

The design storms used to size the various on-site conveyance systems will vary depending upon their location and function. For example, open channels, culverts, and street rights-of way are generally designed for larger events (100-year storm), whereas inlets and storm drain pipes are designed for smaller events (25- year storm).

It is recommended that once the initial set of controls are selected in the Storm Water Management Plan design, the full build-out 100-year, 24-hour storm be routed through the on-site conveyance system and storm water controls to determine the effects on the systems, adjacent property, and downstream areas. Even though the conveyance systems may be designed for smaller storm events, overall, the site should be designed appropriately to safely pass the resulting flows from the "full build-out" 100-year storm event with no flood waters entering habitable structures.

On-site flood control has many considerations for the safeguarding of people and property. On residential streets, for the "streambank protection" and "conveyance" storm events, the safe passage of vehicular traffic is an important concern. For the 100-year storm events, traffic may be limited in order to utilize all or portions of the right-of-way for storm water conveyance in order to protect properties. As such, the effective management of storm water throughout the development for the full range of storm events is needed.

1.2.3.2 Downstream Flood Control

The downstream assessment is the first step in the process to determine if a specific development will have a flooding impact on downstream properties, structures, bridges, roadways, or other facilities. This assessment should be conducted downstream of a development to the point where the discharge from the proposed development no longer has a significant impact upon the receiving stream or storm drainage system. Hydrologic and hydraulic evaluations must be conducted to determine if there are areas of concerns, i.e. an increase of the Base Flood Elevations. The local jurisdiction should be consulted to obtain records and maps related to the National Flood Insurance Program and the availability of Flood Insurance Studies and Flood Insurance Rate Maps (FIRMS) which will be helpful in this assessment.

The downstream flood control criterion is based on an analysis of the "streambank protection" and "conveyance" storm events, as well as the 100-year, 24-hour storm, (denoted Q_{p100}).

Initially, the assessment will determine if the downstream receiving system has adequate capacity in its "full build-out" floodplain. To make this determination, Q_f, the runoff which the stream can handle without having an impact on downstream properties, structures, bridges, roadways, or other facilities, must be determined. There are four options by which a community can address downstream flood control. These steps closely follow the four steps for streambank protection.

<u>Option 1: Acceptable Downstream Conditions</u>

The developer should provide all supporting calculations and/or documentation to show that the existing downstream conveyance system has capacity (Q_f) to safely pass the Q_{p100} discharge from the new development. Systems shown to be adequate are reflective of areas where attempts have been made to keep flood-susceptible development out of the full build out floodplain through a combination of regulatory controls, storm water master planning, and incentives. This includes communities that have regulated floodplains for fully-developed conditions. This approach recognizes the impacts of new development might not be completely mitigated at the extreme flood level and provides a much greater assurance that local flooding will not be a problem because people and structures are kept out of harm's way.

<u>Option 2: Downstream Improvements</u>

If the downstream receiving system does not have adequate capacity, then the developer may choose to provide improvements to the off-site, downstream conveyance system. If this option is chosen the proposed improvements must be designed to adequately convey post-developed storm water peak discharges for the three (3) storm events. The improvements must also extend to the point at which the discharge from the proposed development no longer has a significant impact upon the receiving stream or storm drainage system. The developer must provide supporting calculations and/or documentation that the downstream peak discharges and water surface elevations are safely conveyed by the proposed system, without endangering downstream properties, structures, bridges, roadways, or other facilities.

<u>Option 3: Maintain Existing Downstream Conditions</u>

If the downstream receiving system does not have adequate capacity, then the developer may also choose to provide storm water controls to reduce downstream flood impacts. These controls include on-site controls such as detention, retention, and regional controls. Storm water master plans are a necessity to attempt to ensure public safety for the extreme storm event. The developer must provide supporting calculations and/or documentation that the controls will be designed and constructed so that there is no increase in downstream peak discharges or water surface elevations due to development. One-site controls shall be designed to release the peak discharge for the pre-development 2-year, 10-year, and 100-year storm events.

<u>Option 4: Maintain Existing On-Site Conditions</u>

Lastly, on-site controls may be used to maintain existing peak discharges from the development site. The developer must provide supporting calculations and/or documentation that the on-site controls will be designed and constructed to maintain on-site existing conditions. It is important to note that Option 4 does not require a downstream assessment. It is meant as a solely detention-based approach to addressing downstream flood control. For many developments however, the results of a downstream assessment may show that significantly less flood control is required than "detaining to predeveloped conditions". This method may also exacerbate downstream flooding problems due to timing of flows, as discussed in Section 2.1.8. Therefore it is strongly recommended that a downstream assessment be performed for all developments, and that Option 4 only be used when Options 1, 2, and 3 are not feasible.

The following items should be considered when providing downstream flood control.

- *Peak-Discharge and Hydrograph Generation:* Hydrograph methods provided in Section 2.1 can be used to compute the peak discharge rate and runoff for the three (3) storm events ("Streambank Protection", "Conveyance", and 100-year).
- *Rainfall Depths:* The rainfall depth of the three storm events can be determined from rainfall tables included in Chapter 2.
- *Off-site Drainage Areas:* Off-site drainage areas should be modeled as "full build-out" for the three storm events storm events to ensure safe passage of future flows.
- *Downstream Assessment:* If flow is being detained on-site, downstream areas should be checked to ensure there is no peak flow or water surface increase above pre-development conditions to the point where the undetained discharge from the proposed development no longer has a significant impact upon the receiving stream or storm drainage system. More detail on Downstream Assessments is given in Section 2.1.8.

Section 1.3 – *integrated* Storm Water Controls

Appendix G contains summaries, discussions and examples of storm water controls that can be implemented in land development to meet the goals of protecting water quality, minimizing streambank erosion, and reducing flood volumes. Although primarily oriented toward water quality issues, these storm water controls bring additional and valuable benefits for flood control and streambank protection. Many of the listed storm water control features and techniques enhance the aesthetics and value of land developments, as well as providing a drainage function, and are recommended for use in Copper Canyon, when applicable.

Special storm water controls are not required for water quality treatment by the Town of Copper Canyon at this time unless downstream conditions dictate. Although not mandated, the use of these storm water controls are recognized as inherently valuable for application in overall storm water management. The Town of Copper Canyon encourages developers to use water quality storm water controls and will evaluate any proposed controls based on this section.

CHAPTER 2 – HYDROLOGIC ANALYSIS

Section 2.1 – Estimating Runoff

2.1.1 - Introduction to Hydrologic Methods

Hydrology deals with estimating flow peaks, volumes, and time distributions of storm water runoff. The analysis of these parameters is fundamental to the design of storm water management facilities, such as storm drainage systems and structural storm water controls. In the hydrologic analysis of a development/redevelopment site, there are a number of variable factors that affect the nature of storm water runoff from the site. Some of the factors that need to be considered include:

- Rainfall amount and storm distribution
- Drainage area size, shape, and orientation
- Ground cover and soil type
- Slopes of terrain and stream channel(s)
- Antecedent moisture condition
- Rainfall abstraction rates (Initial and continued)
- Storage potential (floodplains, ponds, wetlands, reservoirs, channels, etc.)
- Watershed development potential
- Characteristics of the local drainage system

There are a number of empirical hydrologic methods available to estimate runoff characteristics for a site or drainage subbasin; however, the following methods have been selected to support hydrologic site analysis for the design methods and procedures included in this Manual:

- Rational Method
- SCS Unit Hydrograph Method
- Snyder's Unit Hydrograph Method
- USGS & TXDOT Regression Equations

These methods were selected based upon a verification of their accuracy in duplicating local hydrologic estimates for a range of design storms throughout the state and the availability of equations, nomographs, and computer programs to support the methods.

Table 2.1.1-1 lists the hydrologic methods and the circumstances for their use in various analysis and design applications. Table 2.1.1-2 provides some limitations on the use of several methods.

In general:
The Rational Method is recommended for small highly impervious drainage areas such as parking lots and roadways draining into inlets and gutters.

The SCS Method is the recommended hydrograph method in Copper Canyon. Use of Snyder's Unit Hydrograph Method requires approval of the TOWN ENGINEER.

The USGS (U.S. Geological Survey) and TxDOT (Texas Department of Transportation) regression equations are recommended for drainage areas with characteristics within the ranges given for the equations. These equations should be used with caution when there are significant storage areas within the drainage basin or where other drainage characteristics indicate general regression equations might not be appropriate. USGS and TxDOT equations are only allowed with the approval of TOWN ENGINEER.

Table 2.1.1-1 Applications of Recommended Hydrologic Methods

Method	Manual Section	Rational Method	SCS Method	Snyder's Unit Hydrograph	USGS / TXDOT Equations
Storage Facilities	4.5		✓	✓	
Outlet Structures	4.6		✓	✓	
Gutter Flow and Inlets	3.2	✓			
Storm Drain Pipes	3.2	✓	✓	✓	
Culverts	4.2	✓	✓	✓	✓
Bridges	4.3		✓	✓	
Roadside Ditches	4.4	✓	✓	✓	
Open Channels	4.4		✓	✓	✓
Energy Dissipation	4.7		✓	✓	

Table 2.1.1-2 Constraints on Using Recommended Hydrologic Methods		
Method	**Size Limitations[1]**	**Comments**
Rational[2]	0 – 100 acres	Method can be used for estimating peak flows and the design of small site or subdivision storm sewer systems.
Unit Hydrograph **(SCS)**[3]	**Any Size**	Method can be used for estimating peak flows and hydrographs for all design applications.
Unit Hydrograph **(Snyder's)**[4]	> 100 acres	Method can be used for estimating peak flows and hydrographs for all design applications. **This method can only be used with approval of the TOWN ENGINEER.**
TXDOT Regression Equations	10 to 100 mi^2	Method can be used for estimating peak flows for rural design applications **for comparison purposes only**. **This method can only be used with approval of the TOWN ENGINEER.**
USGS Regression Equations	3 – 40 mi^2	Method can be used for estimating peak flows for urban design applications **for comparison purposes only**. **This method can only be used with approval of the TOWN ENGINEER.**

[1]Size limitations refer to the drainage basin for the storm water management facility (e.g., culvert, inlet). **These do not necessarily apply to master drainage plans**.
[2]**The version of the Rational Method described in Section 4.5.4.2 may be used to calculate detention storage volumes for drainage areas of 10 acres or less and preliminary estimates for drainage areas of less than 100 acres. The engineer is cautioned that the method could underestimate the storage volume**.
[3]**This refers to SCS routing methodology included in many readily available programs (such as HEC-HMS or HEC-1) that utilize this methodology**. The Simplified SCS Method described in Section 2.1.5.7 may be used only with the approval of the TOWN ENGINEER.
[4]This refers to the Snyder's routing methodology included in many readily available programs (such as HEC-HMS or HEC-1) that utilize this methodology.

2.1.2 - Symbols and Definitions

To provide consistency within this section as well as throughout this Manual, the symbols listed in Table 2.1.2-1 will be used. These symbols were selected because of their wide use in technical publications. In some cases, the same symbol is used in existing publications for more than one definition. Where this occurs in this section, the symbol will be defined where it occurs in the text or equations.

Table 2.1.2-1 Symbols and Definitions

Symbol	**Definition**	**Units**
A	Drainage area or area	acres or square feet
B_f	Baseflow	acre-feet
C	Runoff coefficient	-
C_f	Frequency factor	-
CN	SCS-runoff curve number	-
D	Time interval	hours
E	Evaporation	ft
E_t	Evapotranspiration	ft
F	Pond and swamp adjustment factor	-
G_h	Hydraulic gradient	-
I or i	Rainfall intensity	in/hr

Table 2.1.2-1 Symbols and Definitions

Symbol	Definition	Units
I	Percent of impervious cover	%
I	Infiltration	acre-feet
I_a	Initial abstraction from total rainfall	in
k_h	Infiltration rate	ft/day
L	Flow length	ft
n	Manning roughness coefficient	-
NRCS	Natural Resources Conservation Service (formerly SCS)	-
O_f	Overflow	acre-feet
P	Accumulated rainfall	in
P_2	2-year, 24-hour rainfall	in
P_w	Wetted perimeter	ft
PF	Peaking factor	-
Q	Rate of runoff	cfs (or inches)
Q_i	Peak inflow discharge	cfs
Q_o	Peak outflow discharge	cfs
Q_p	Peak rate of discharge	cfs
Q_{wq}	Water Quality peak rate of discharge	cfs
q	Storm runoff during a time interval	in
q_u	Unit peak discharge	cfs (or cfs/mi^2/inch)
R	Hydraulic radius	ft
R_o	Runoff	acre-feet
R_v	Runoff Coefficient	-
S	Ground slope	ft/ft or %
S	Potential maximum retention	in
S	Slope of hydraulic grade line	ft/ft
SCS	Soil Conservation Service (Now NRCS)	-
SP_v	Streambank Protection Volume	acre-feet
T	Channel top width	ft
T_L	Lag time	hours
T_p	Time to peak	hours
T_t	Travel time	hours
t	Time	min
t_c	Time of concentration	min

Table 2.1.2-1 Symbols and Definitions		
Symbol	**Definition**	**Units**
TIA	Total impervious area	%
V	Velocity	ft/s
V	Pond volume	acre-feet
V_d	Developed runoff volume	in
V_f	Flood control volume	acre-feet
V_r	Runoff volume	acre-feet
V_s	Storage volume	acre-feet
WQ_v	Enhanced water quality protection volume	acre-feet

2.1.3 - Rainfall Estimation

The first step in any hydrologic analysis is an estimation of the rainfall that will fall on the site for a given time period. The amount of rainfall can be quantified with the following characteristics:

Duration (hours) – Length of time over which rainfall (storm event) occurs

Depth (inches) – Total amount of rainfall occurring during the storm duration

Intensity (inches per hour) – Depth divided by the duration

The Frequency of a rainfall event is the recurrence interval of storms having the same duration and volume (depth). This can be expressed either in terms of *exceedance probability* or *return period*.

Exceedance Probability – Probability that a storm event having the specified duration and volume will be exceeded in one given time period, typically in years.

Return Period – Average length of time between events, which have the same duration and volume.

Thus, if a storm event with a specified duration and volume has a 1% chance of occurring in any given year, then it has an exceedance probability of 0.01 and a return period of 100 years.

The standard 24-hr duration storm event, for watersheds larger than 500 acres (0.78 square miles), was utilized to establish rainfall parameters. Point rainfall depths were obtained from the *Atlas of Depth-Duration Frequency of Precipitation Annual Maxima for Texas, USGS Scientific Investigation Report 2004-5041, Asquith 2004* based on a central location in the Town of Copper Canyon for 2 yr through 500 yr return periods. One year return period rainfall data was calculated by extrapolation of the 2 yr through 500 yr rainfall data. The rainfall depths are listed in Table 2.1.3-1. The rainfall intensities are listed in Table 2.1.3-2. The rainfall depths and intensities listed in tables 2.1.3-1 and 2.1.3-2 will be used throughout Copper Canyon and its ETJ.

Rainfall intensities for the 16 counties which participate in the NCTCOG area (see Figure 1.1) are provided in Section 5.0 and should be used for all hydrologic analysis within the given county. The values in these tables were derived in the following way: New IDF values for the 1-year through 500-year storm return periods were determined for the NCTCOG area on a county by county basis. All values were plotted and smoothed to ensure continuity. The values were smoothed by fitting an equation of the form:

i = b/(t + d)e (1.1)

where i is inches per hour and t is the rainfall duration in minutes. The parameters b, d and e are found at the top of each of the tables in Section 5.0.

Refer to Table 2.1.3-1 for Tabular Values for Denton County Rainfall tables determined from the IDF curve.

Table 2.1.3-1 Rainfall Intensity-Duration for Denton County

Hours	Minutes	1	2	5	10	25	50	100
		Return Period (Years)						
	Coefficients	1	2	5	10	25	50	100
	e	0.82089	0.80553	0.79891	0.78388	0.76912	0.76817	0.7566
	b	43.381	50.455	65.467	70.683	78.538	89.853	95.776
	d	T8	9	11	11	11	12	12
Hours	Minutes	Rainfall Intensity (inches per hour)						
	5	5.28	6.02	7.15	8.04	9.31	10.19	11.23
0.083	6	4.97	5.7	6.81	7.67	8.89	9.76	10.75
	7	4.7	5.41	6.5	7.33	8.5	9.36	10.32
	8	4.46	5.15	6.23	7.03	8.16	9	9.93
	9	4.24	4.92	5.98	6.75	7.84	8.67	9.57
	10	4.04	4.71	5.75	6.5	7.55	8.36	9.24
	11	3.87	4.52	5.54	6.27	7.29	8.08	8.93
	12	3.71	4.34	5.35	6.05	7.04	7.82	8.65
	13	3.56	4.18	5.17	5.85	6.82	7.58	8.39
	14	3.43	4.04	5	5.67	6.61	7.36	8.14
0.250	15	3.31	3.9	4.85	5.5	6.41	7.14	7.91
	16	3.19	3.77	4.7	5.34	6.23	6.95	7.7
	17	3.09	3.66	4.57	5.19	6.05	6.76	7.5
	18	2.99	3.55	4.44	5.05	5.89	6.59	7.31
	19	2.9	3.44	4.32	4.91	5.74	6.43	7.13
	20	2.81	3.35	4.21	4.79	5.6	6.27	6.96
	21	2.73	3.26	4.11	4.67	5.46	6.12	6.8
	22	2.66	3.17	4.01	4.56	5.34	5.99	6.65
	23	2.59	3.09	3.91	4.45	5.21	5.85	6.5
	24	2.52	3.02	3.82	4.35	5.1	5.73	6.36
	25	2.46	2.95	3.74	4.26	4.99	5.61	6.23
	26	2.4	2.88	3.66	4.17	4.89	5.5	6.11
	27	2.34	2.81	3.58	4.08	4.79	5.39	5.99
	28	2.29	2.75	3.51	4	4.69	5.28	5.88
	29	2.24	2.69	3.44	3.92	4.6	5.18	5.77
0.500	30	2.19	2.64	3.37	3.85	4.51	5.09	5.66
	31	2.14	2.58	3.31	3.77	4.43	5	5.56
	32	2.1	2.53	3.24	3.71	4.35	4.91	5.47
	33	2.06	2.49	3.18	3.64	4.28	4.83	5.38
	34	2.02	2.44	3.13	3.58	4.2	4.75	5.29
	35	1.98	2.39	3.07	3.51	4.13	4.67	5.2
	36	1.94	2.35	3.02	3.46	4.06	4.59	5.12
	37	1.91	2.31	2.97	3.4	4	4.52	5.04
	38	1.87	2.27	2.92	3.35	3.94	4.45	4.96
	39	1.84	2.23	2.88	3.29	3.88	4.38	4.89
	40	1.81	2.19	2.83	3.24	3.82	4.32	4.82
	41	1.78	2.16	2.79	3.19	3.76	4.26	4.75
	42	1.75	2.13	2.74	3.15	3.71	4.2	4.68
	43	1.72	2.09	2.7	3.1	3.65	4.14	4.62
	44	1.69	2.06	2.66	3.06	3.6	4.08	4.56
0.750	45	1.67	2.03	2.63	3.01	3.55	4.02	4.5
	46	1.64	2	2.59	2.97	3.5	3.97	4.44
	47	1.62	1.97	2.55	2.93	3.46	3.92	4.38
	48	1.59	1.94	2.52	2.89	3.41	3.87	4.32
	49	1.57	1.92	2.49	2.85	3.37	3.82	4.27
	50	1.55	1.89	2.45	2.82	3.33	3.77	4.22
	51	1.53	1.86	2.42	2.78	3.28	3.73	4.17
	52	1.51	1.84	2.39	2.75	3.24	3.68	4.12
	53	1.49	1.82	2.36	2.71	3.21	3.64	4.07
	54	1.47	1.79	2.33	2.68	3.17	3.6	4.02
	55	1.45	1.77	2.3	2.65	3.13	3.55	3.98
	56	1.43	1.75	2.28	2.62	3.09	3.51	3.93
	57	1.41	1.73	2.25	2.59	3.06	3.48	3.89
	58	1.39	1.71	2.22	2.56	3.03	3.44	3.85
	59	1.37	1.69	2.2	2.53	2.99	3.4	3.81
1	60	1.36	1.67	2.17	2.5	2.96	3.36	3.77
2	120	0.81	1.01	1.33	1.55	1.85	2.11	2.38
3	180	0.59	0.74	0.99	1.15	1.38	1.58	1.79
6	360	0.34	0.43	0.58	0.68	0.83	0.95	1.09
12	720	0.19	0.25	0.34	0.4	0.49	0.57	0.65
24	1440	0.11	0.14	0.2	0.23	0.29	0.33	0.39

2.1.4 – Rational Method

2.1.4.1 Introduction

An important formula for determining the peak runoff rate is the Rational Formula. It is characterized by:

- Consideration of the entire drainage area as a single unit
- Estimation of flow at the most downstream point only
- The assumption that rainfall is uniformly distributed over the drainage area and is constant over time

The Rational Formula adheres to the following assumptions:

- The predicted peak discharge has the same probability of occurrence (return period) as the rainfall intensity (I)
- The runoff coefficient (C) is constant during the storm event

When using the Rational Method some precautions should be considered:

- In determining the C value (runoff coefficient based on land use) for the drainage area, hydrologic analysis should take into account any future changes in land use that might occur during the service life of the proposed facility.
- Since the Rational Method uses a composite C and a single t_c value for the entire drainage area, if the distribution of land uses within the drainage basin will affect the results of hydrologic analysis (e.g., if the impervious areas are segregated from the pervious areas), then the basin should be divided into sub-drainage basins.
- The charts, graphs, and tables included in this section are given to assist the engineer in applying the Rational Method. The engineer should use sound engineering judgment in applying these design aids and should make appropriate adjustments when specific site characteristics dictate adjustments are appropriate.

2.1.4.2 Application

The Rational Method can be used to estimate storm water runoff peak flows for the design of gutter flows, drainage inlets, storm drainpipe, culverts, and small ditches. It is most applicable to small, highly impervious areas. The recommended maximum drainage area that should be used with the Rational Method is 100 acres.

The Rational Method should not be used for storage design or any other application where a more detailed routing procedure is required. However, the method described in Section 4.5.4.2 can be used for design of small (10 acres or less) detention facilities, or for preliminary estimates of larger detention facilities.

The Rational Method should not be used for calculating peak flows downstream of bridges, culverts, or storm sewers that may act as restrictions causing storage, which impacts the peak rate of discharge.

2.1.4.3 Equations

The Rational Formula estimates the peak rate of runoff at any location in a watershed as a function of the drainage area, runoff coefficient, and the mean rainfall intensity for a duration equal to the time of concentration, t_c (the time required for water to flow from the most remote point of the basin to the location being analyzed).

The Rational Formula is expressed as follows:

$$Q = CIA \qquad (2.1.2)$$

where:

- Q = maximum rate of runoff (cfs)
- C = runoff coefficient representing a ratio of runoff to rainfall
- I = average rainfall intensity for a duration equal to the t_c (in/hr)
- A = drainage area contributing to the design location (acres)

The coefficients given in Table 2.1.4-2 are applicable for storms with return periods less than or equal to 10 years. Less frequent, higher intensity storms may require modification of the coefficient because infiltration and other losses have a proportionally smaller effect on runoff (Wright-McLaughlin Engineers, 1969). The adjustment of the Rational Method for use with major storms can be made by multiplying the right side of the Rational Formula by a frequency factor C_f. The Rational Formula now becomes:

$$Q = C_f CIA \qquad (2.1.3)$$

The C_f values that can be used are listed in Table 2.1.4-1. The product of C_f times C shall not exceed 1.0.

Table 2.1.4-1 Frequency Factors for Rational Formula

Recurrence Interval (years)	**C_f**
10 or less	1.0
25	1.1
50	1.2
100	1.25

2.1.4.4 Time of Concentration

Use of the Rational Formula requires the time of concentration (t_c) for each design point within the drainage basin. The duration of rainfall is then set equal to the time of concentration and is used to estimate the design average rainfall intensity (I). The time of concentration consists of an overland flow time to the point where the runoff is concentrated or enters a defined drainage feature (e.g., open channel) plus the time of flow in a closed conduit or open channel to the design point.

Figure 2.1.4-1 can be used to estimate overland flow time. For each drainage area, the distance is determined from the inlet to the most remote point in the tributary area. From a topographic map, the average slope is determined for the same distance. The runoff coefficient (C) is determined by the procedure described in a subsequent section of this chapter. In urban areas, the length of overland flow distance should realistically be no more than 50 – 100 feet.

Although there is no formula for the graph shown in Figure 2.1.4-1, the formula often used, which seems to match the nomograph very closely, is as follows:

$$T_c = 1.8(1.1 - C)(D)^{0.5}/(S)^{(1/3)} \qquad (2.1.4)$$

where:

T_c = time of concentration (min)
C = average or composite runoff coefficient
D = distance from upper end of watershed to outlet (ft)
S = average slope along distance “D”, in percent (ft/100 ft)

Example: Given the following values, determine the time of concentration using (1) Equation 2.1.4, and (2) Figure 2.1.4-1: D = 250 ft, C = 0.7, S = 0.50% slope.

Figure 2.1.4-1 gives approximately 15 minutes.

$\mathbf{T_c = 1.8(1.1 - 0.7)(250)^{0.5}/(0.50)^{(1/3)} = 14.34\ min}$

Other methods and charts may be used to calculate overland flow time if approved by the local review authority.

Generally, the time of concentration for overland flow is only a part of the overall design problem. Often one encounters swale flow, confined channel flow, and closed conduit flow travel times that must be added as part of the overall time of concentration. After first determining the average flow velocity in the pipe or channel, the travel time is obtained by dividing velocity into the pipe or channel length. Velocity can be estimated by using the nomograph shown in Figure 2.1.4-2. More guidance on travel time estimation is given in Section 2.1.5.6.

To obtain the total time of concentration, the pipe or open channel flow time must be calculated and added to the inlet time. For example, if the flow time in a channel is 15 minutes and the overland flow time from the ridge line to the channel is 10 minutes, then the total time of concentration is 25 minutes. Note that the time of concentration cannot be less than 10 minutes.

The following table shows recommended minimum and maximum times of concentration based on land use categories. These represent times to the most upstream inlet (minimum inlet time). Computed downstream travel times will be added to determine times of concentration through the system.

Land Use	Minimum (minutes)	Maximum (minutes)
Residential Development	10	30
Commercial and Industrial	10	25
Central Business District	10	15

Two common errors should be avoided when calculating time of concentration. First, in some cases runoff from a portion of the drainage area which is highly impervious may result in a greater peak discharge than would occur if the entire area were considered. Second, when designing a drainage system, the overland flow path is not necessarily the same before and after development and grading operations have been completed. Selecting overland flow paths in excess of 50 feet for impervious areas should be done only after careful consideration.

2.1.4.4 Rainfall Intensity (I)

The rainfall intensity (I) is the average rainfall rate in in/hr for a duration equal to the time of concentration for a selected return period. Once a particular return period has been selected for design and a time of concentration calculated for the drainage area, the rainfall intensity can be determined from Rainfall-Intensity-Duration data given in the rainfall tables in Section 2.1.3.

2.1.4.6 Runoff Coefficient (C)

The runoff coefficient (C) is the variable of the Rational Method least susceptible to precise determination and requires judgment and understanding on the part of the design engineer. While engineering judgment will always be required in the selection of runoff coefficients, typical coefficients represent the integrated effects of many drainage basin parameters. Table 2.1.4-2 gives the recommended runoff coefficients for the Rational Method.

It is often desirable to develop a composite runoff coefficient based on the percentage of different types of surfaces in the drainage areas. Composites can be made with the values from Table 2.1.4-2 by using percentages of different land uses. In addition, more detailed composites can be made with coefficients for different surface types such as rooftops, asphalt, and concrete streets and sidewalks. The composite procedure can be applied to an entire drainage area or to typical "sample" blocks as a guide to the selection of reasonable values of the coefficient for an entire area.

It should be remembered that the Rational Method assumes that all land uses within a drainage area are uniformly distributed throughout the area. If it is important to locate a specific land use within the drainage area, then another hydrologic method should be used where hydrographs can be generated and routed through the drainage system.

It may be that using only the impervious area from a highly impervious site (and the corresponding high C factor and shorter time of concentration) will yield a higher peak runoff value than by using the whole site. This should be checked particularly in areas where the overland portion is grassy (yielding a long t_c) to avoid underestimating peak runoff.

Table 2.1.4-2 Recommended Runoff Coefficient Values

Description of Area	Runoff Coefficients (C)
Lawns:	
Sandy soil, flat, 2%	0.10
Sandy soil, average, 2 - 7%	0.15
Sandy soil, steep, > 7%	0.20
Clay soil, flat, 2%	0.17
Clay soil, average, 2 - 7%	0.22
Clay soil, steep, > 7%	0.35
Agricultural	0.30
Forest	0.15
Streams, Lakes, Water Surfaces	1.00
Business:	
Downtown areas	0.95
Neighborhood areas	0.70
Residential:	
Single Family (1/8 acre lots)	0.65
Single Family (1/4 acre lots)	0.60
Single Family (1/2 acre lots)	0.55
Single Family (1+ acre lots)	0.45
Multi-Family Units, (Light)	0.65
Multi-Family, (Heavy)	0.85
Commercial/Industrial:	
Light areas	0.70
Heavy areas	0.80
Parks, cemeteries	0.25
Playgrounds	0.35
Railroad yard areas	0.40
Streets:	
Asphalt and Concrete	0.95
Brick	0.85
Drives, walks, and roofs	0.95
Gravel areas	0.50
Graded or no plant cover:	
Sandy soil, flat, 0 - 5%	0.30
Sandy soil, flat, 5 - 10%	0.40
Clayey soil, flat, 0 - 5%	0.50
Clayey soil, average, 5 - 10%	0.60

2.1.4.7 Example Problem

Following is an example problem that illustrates the application of the Rational Method to estimate peak discharges.

Estimates of the maximum rate of runoff are needed at the inlet to a proposed culvert for a 100-year return period.

SITE DATA

From a topographic map of the Town and a field survey, the area of the drainage basin upstream from the point in question is found to be 23 acres. In addition, the following data were measured:

Average overland slope = 2.0%

Length of overland flow = 50 ft

Length of main basin channel = 2,250 ft

Slope of channel = .018 ft/ft = 1.8%

Roughness coefficient (n) of channel was estimated to be 0.090

From existing land use maps, land use for the drainage basin was estimated to be:
Residential (single family – ¼ acre lots) - 80%
Graded - sandy soil, 3% slope - 20%

From existing land use maps, the land use for the overland flow area at the head of the basin was estimated to be: Lawn - sandy soil, 2% slope

OVERLAND FLOW

A runoff coefficient (C) for the overland flow area is determined from Table 2.1.4-2 to be 0.10.

Time of Concentration

From Figure 2.1.4-1 with an overland flow length of 50 ft, slope of 2% and a C of 0.10, the overland flow time is 10 min. Channel flow velocity is determined from Figure 2.1.4-2 to be 3.1 ft/s (n = 0.090, R = 1.62 (from channel dimensions) and S = .018). Therefore,

$$\text{Flow Time} = \frac{2{,}250 \text{ feet}}{(3.1 \text{ ft/s}) / (60 \text{ s/min})} = 12.1 \text{ minutes}$$

and $t_c = 10 + 12.1 = 22.1$ min (use 22 min)

Rainfall Intensity

From Table 2.1.3-2 using a duration equal to 22 minutes,

I_{100} (100-yr return period) = 6.65 in/hr

Runoff Coefficient

A weighted runoff coefficient (C) for the total drainage area is determined below by utilizing the values from Table 2.1.4-2.

Land Use	**Percent of Total Land Area**	**Runoff Coefficient**	**Weighted Runoff Coefficient**
Residential			
(Single Family – ¼ acre lots)	0.80	0.60	0.48
Graded area	0.20	0.30	0.06

Total Weighted Runoff Coefficient = 0.54

*Column 3 equals column 1 multiplied by column 2.

Peak Runoff

The estimate of peak runoff for a 25-yr design storm for the given basin is:
$Q_{25} = C_fCIA = (1.10)(.54)(5.65)(23) = 77.2$ cfs

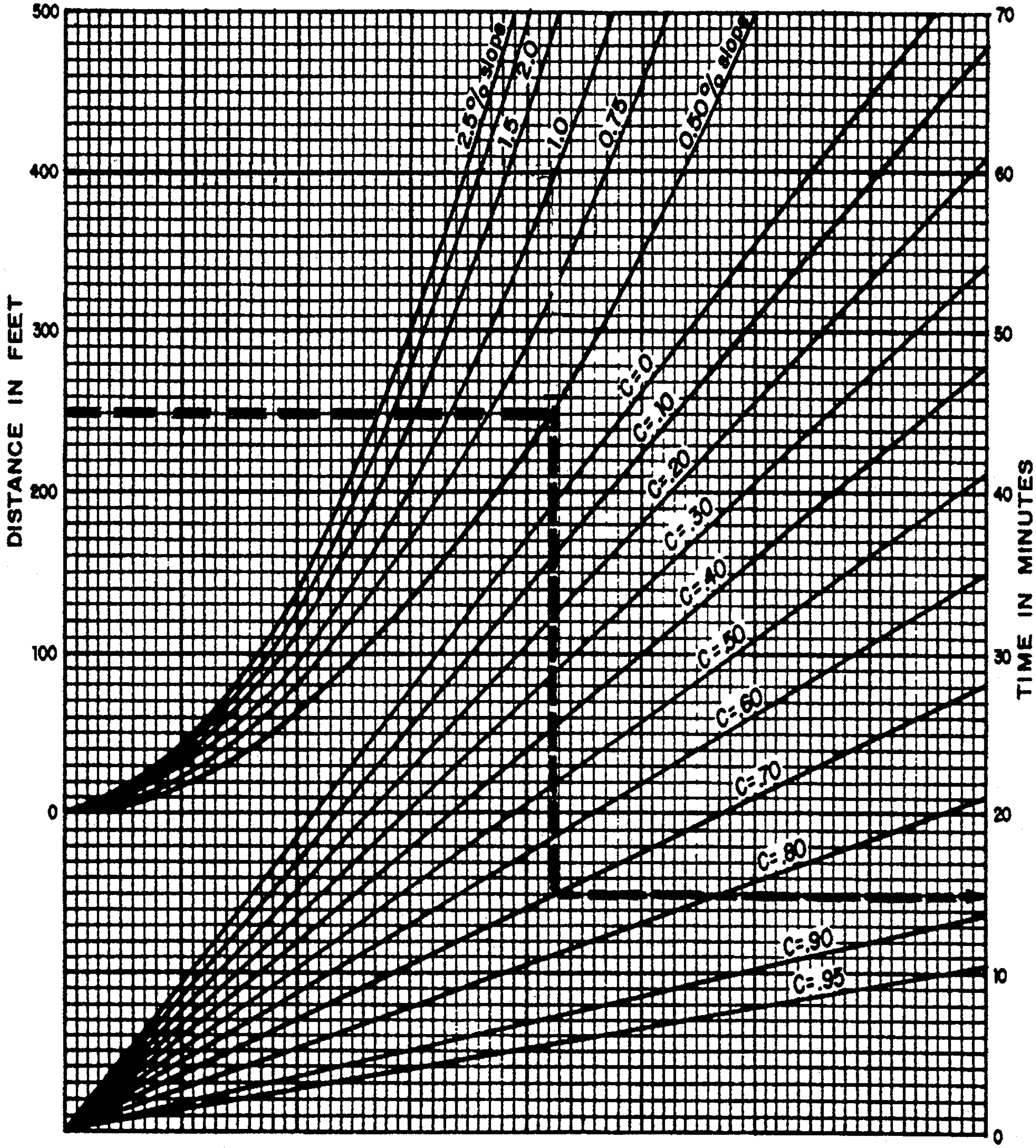
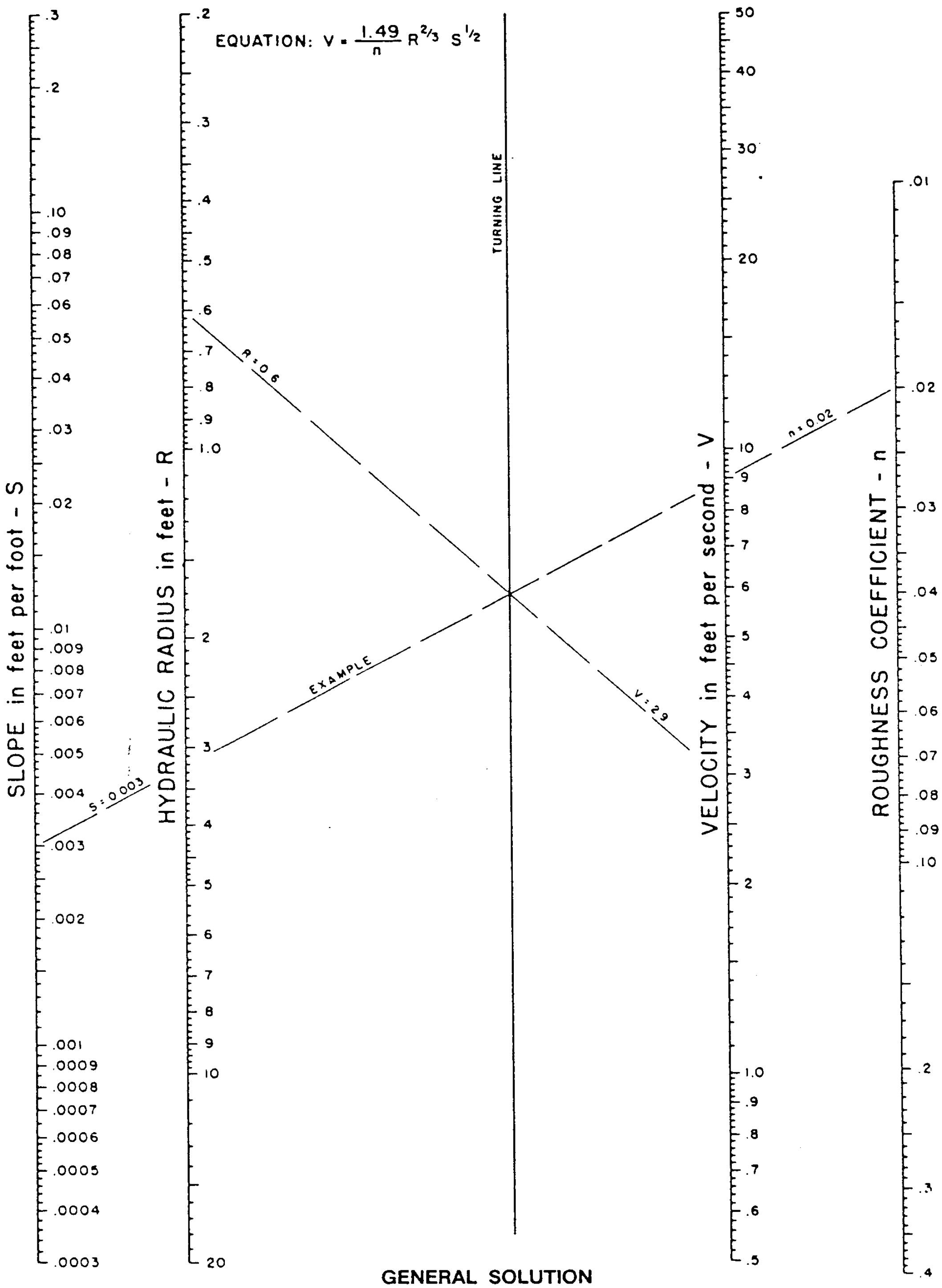


Figure 2.1.4-1 Rational Formula - Overland Time of Flow Nomograph
(Source: Airport Drainage, Federal Aviation Administration, 1965)



Reference: USDOT, FHWA, HDS-3 (1961).

Figure 2.1.4-2 Manning's Equation Nomograph

(Source: USDOT, FHWA, HDS-3 (1961))

2.1.5 – SCS Hydrologic Method

2.1.5.1 Introduction

The Soil Conservation Service[1] (SCS) hydrologic method requires basic data similar to the Rational Method: drainage area, a runoff factor, time of concentration, and rainfall. The SCS approach, however, is more sophisticated in that it also considers the time distribution of the rainfall, the initial rainfall losses to interception and depression storage, and an infiltration rate that decreases during the course of a storm. Details of the methodology can be found in the SCS National Engineering Handbook, Section 4, Hydrology.

A typical application of the SCS method includes the following basic steps:

- Determination of curve numbers that represent different land uses within the drainage area.
- Calculation of time of concentration to the study point.
- The recommended rainfall data for the SCS Method is that presented in Section 2.1.3 The Type III SCS rainfall distribution, CAN ONLY BE USED WITH APPROVAL OF THE TOWN ENGINEER Note: See Figure 2.1.5-1 for the geographic boundaries for the different SCS rainfall distributions.
- Using the unit hydrograph approach, the hydrograph of direct runoff from the drainage basin can be developed.

2.1.5.2 Application

The SCS method can be used for both the estimation of storm water runoff peak rates and the generation of hydrographs for the routing of storm water flows. Thus, the SCS method can be used for most design applications, including storage facilities and outlet structures, storm drain systems, culverts, small drainage ditches, open channels, and energy dissipators.

Town of Copper Canyon allows the hydrograph routing method for subdrainage areas of any size but will not allow the Simplified Method, except as approved by TOWN ENGINEER. Figure 2.1.6-1 presents a sample computation sheet for presentation of unit hydrograph method results. This form should be completed even if the computations are performed on acceptable computer programs HEC-1 or HEC-HMS. **Rainfall for application of the SCS Hydrologic (Routing) Method shall be based on rainfall data in Table 2.1.3-1.**

2.1.5.3 Equations and Concepts

The hydrograph of outflow from a drainage basin is the sum of the elemental hydrographs from all the sub-areas of the basin, modified by the effects of transit time through the basin and storage in the stream channels. Since the physical characteristics of the basin including shape, size, and slope are constant, the unit hydrograph approach assumes there is considerable similarity in the shape of hydrographs from storms of similar rainfall characteristics. Thus, the unit hydrograph is a typical hydrograph for the basin with a runoff volume under the hydrograph equal to one (1.0) inch from a storm of specified duration. For a storm of the same duration but with a different amount of runoff, the hydrograph of direct runoff can be expected to have the same time base as the unit hydrograph and ordinates of flow proportional to the runoff volume. Therefore, a storm that produces 2 inches of runoff would have a hydrograph with a flow equal to twice the flow of the unit hydrograph. With 0.5 inches of runoff, the flow of the hydrograph would be one-half of the flow of the unit hydrograph.

[1]The Soil Conservation Service is now known as the Natural Resources Conservation Service (NRCS)

The following discussion outlines the equations and basic concepts used in the SCS method.

Drainage Area - The drainage area of a watershed is determined from topographic maps and field surveys. For large drainage areas it might be necessary to divide the area into sub-drainage areas to account for major land use changes, obtain analysis results at different points within the drainage area, combine hydrographs from different sub-basins as applicable, and/or route flows to points of interest.

Rainfall - The SCS method applicable to northeast Texas is based on a storm event that has a Type III time distribution. This distribution is used to distribute the 24-hour volume of rainfall for the different storm frequencies (Figure 2.1.5-1).

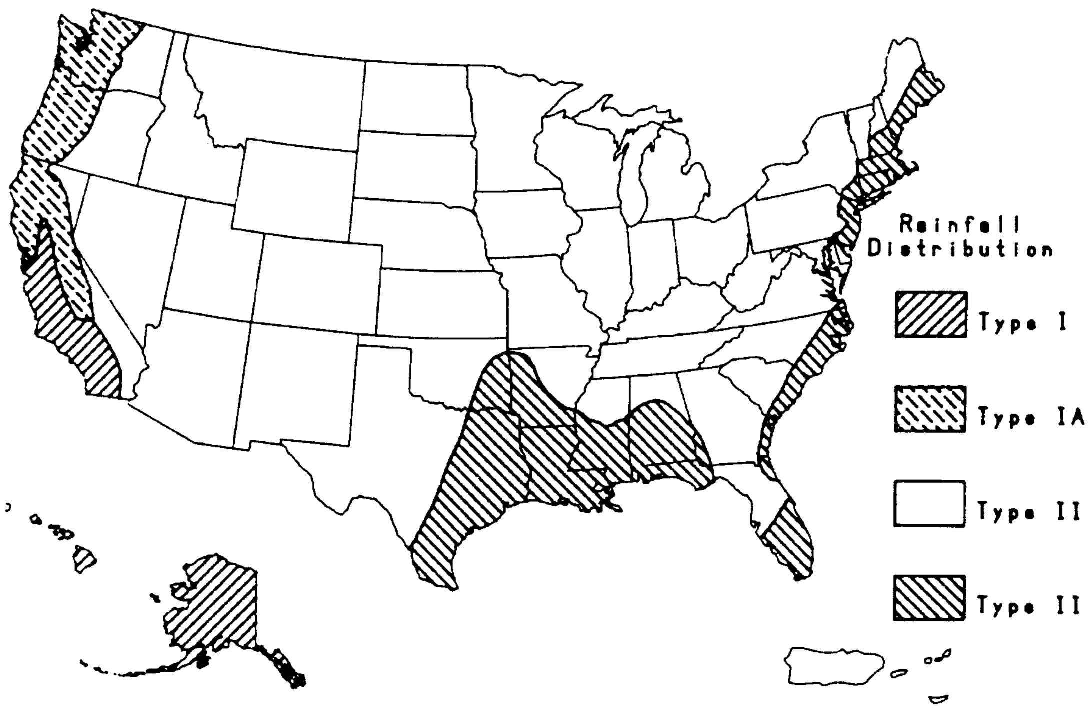


Figure 2.1.5-1 Approximate Geographic Boundaries for SCS Rainfall Distributions

Rainfall-Runoff Equation - A relationship between accumulated rainfall and accumulated runoff was derived by SCS from experimental plots for numerous soils and vegetative cover conditions. The following SCS runoff equation is used to estimate direct runoff from 24-hour or 1-day storm rainfall. The equation is:

$$\mathbf{Q = (P - I_a)^2 / [(P - I_a) + S]} \qquad \textbf{(2.1.5)}$$

where:

- Q = accumulated direct runoff (in)
- P = accumulated rainfall (potential maximum runoff) (in)
- I_a = initial abstraction including surface storage, interception, evaporation, and infiltration prior to runoff (in)
- S = 1000/CN - 10 where CN = SCS curve number

An empirical relationship used in the SCS method for estimating I_a is:

$$I_a = 0.2S \qquad (2.1.6)$$

This is an average value that could be adjusted for flatter areas with more depressions if there are calibration data to substantiate the adjustment. Table 2.1.5-3 provides values of I_a for a wide range of curve numbers (CN).

Substituting 0.2S for I_a in equation 2.1.5, the equation becomes:

$$Q = (P - 0.2S)^2 / (P + 0.8S) \qquad (2.1.7)$$

Figure 2.1.5-2 shows a graphical solution of this equation. For example, 4.1 inches of direct runoff would result if 5.8 inches of rainfall occurred on a watershed with a curve number of 85.

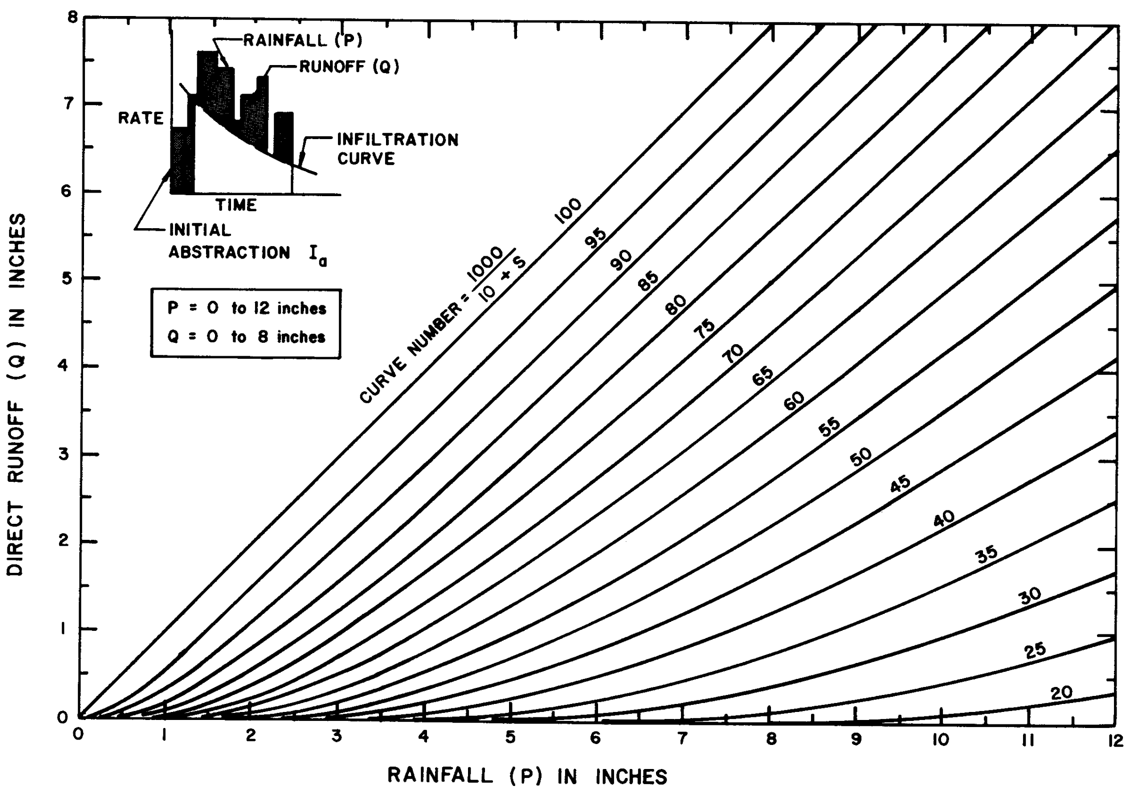


Figure 2.1.5-2 SCS Solution of the Runoff Equation

(Source: SCS, TR-55, Second Edition, June 1986)

Equation 2.1.6 can be rearranged so the curve number can be estimated if rainfall and runoff volume are known. The equation then becomes (Pitt, 1994):

$$CN = 1000/[10 + 5P + 10Q - 10(Q^2 + 1.25QP)^{1/2}] \qquad (2.1.8)$$

2.1.5.4 Runoff Factor (CN)

The principal physical watershed characteristics affecting the relationship between rainfall and runoff are land use, land treatment, soil types, and land slope. The SCS method uses a combination of soil conditions and land uses (ground cover) to assign a runoff factor to an area. These runoff factors, called runoff curve numbers (CN), indicate the runoff potential of an area. The higher the CN, the higher the runoff potential. Soil properties influence the relationship between runoff and rainfall since soils have differing rates of infiltration. Based on infiltration rates, the SCS has divided soils into four hydrologic soil groups.

Group A Soils having a low runoff potential due to high infiltration rates. These soils consist primarily of deep, well-drained sands and gravels.

Group B Soils having a moderately low runoff potential due to moderate infiltration rates. These soils consist primarily of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures.

Group C Soils having a moderately high runoff potential due to slow infiltration rates. These soils consist primarily of soils in which a layer exists near the surface that impedes the downward movement of water or soils with moderately fine to fine texture.

Group D Soils having a high runoff potential due to very slow infiltration rates. These soils consist primarily of clays with high swelling potential, soils with permanently high water tables, soils with a clay pan or clay layer at or near the surface, and shallow soils over nearly impervious parent material.

A list of soils throughout the State of Texas and their hydrologic classification can be found in the publication *Urban Hydrology for Small Watersheds, 2nd Edition, Technical Release Number 55, 1986*. Soil Survey maps can be obtained from local USDA Natural Resources Conservation Service offices for use in estimating soil type. Appendix D contains hydrologic soils classification data for northcentral Texas. Specific data can be found on-line through NRCS at http://soils.usda.gov/.

Consideration should be given to the effects of urbanization on the natural hydrologic soil group. If heavy equipment can be expected to compact the soil during construction or if grading will mix the surface and subsurface soils, appropriate changes should be made in the soil group selected. Also, runoff curve numbers vary with the antecedent soil moisture conditions. Average antecedent soil moisture conditions (AMC II) are recommended for most hydrologic analysis. Areas with high water table conditions may want to consider using AMC III antecedent soil moisture conditions. This should be considered a calibration parameter for modeling against real calibration data. Table 2.1.5-1 gives recommended curve number values for a range of different land uses.

When a drainage area has more than one land use, a composite curve number can be calculated and used in the analysis. It should be noted that when composite curve numbers are used, the analysis does not take into account the location of the specific land uses but sees the drainage area as a uniform land use represented by the composite curve number.

Composite curve numbers for a drainage area can be calculated by using the weighted method as presented below.

<u>Composite Curve Number Calculation Example</u>

Land Use	Percent of Total Land Area	Curve Number	Weighted Curve Number (% area x CN)
Residential 1/8 acre Soil Group B	0.80	0.85	0.68
Meadow Good condition Soil Group C	0.20	0.71	0.14

Total Weighted Curve Number = 0.68 + 0.14 = 0.82

The different land uses within the basin should reflect a uniform hydrologic group represented by a single curve number. Any number of land uses can be included, but if their spatial distribution is important to the hydrologic analysis, then sub-basins should be developed and separate hydrographs developed and routed to the study point.

2.1.5.5 Urban Modifications of the SCS Method

Several factors, such as the percentage of impervious area and the means of conveying runoff from impervious areas to the drainage system, should be considered in computing CN for developed areas. For example, do the impervious areas connect directly to the drainage system, or do they outlet onto lawns or other pervious areas where infiltration can occur?

The curve number values given in Table 2.1.5-1 are based on directly connected impervious area. An impervious area is considered directly connected if runoff from it flows directly into the drainage system. It is also considered directly connected if runoff from it occurs as concentrated shallow flow that runs over pervious areas and then into a drainage system. It is possible for curve number values from urban areas to be reduced by not directly connecting impervious surfaces in the drainage system, but allowing runoff to flow as sheet flow over significant pervious areas.

The following discussion will give some guidance for adjusting curve numbers for different types of impervious areas.

<u>Connected Impervious Areas</u>

The CNs provided in Table 2.1.5-1 for various land cover types were developed for typical land use relationships based on specific assumed percentages of impervious area. These CN values were developed on the assumptions that:

- Pervious urban areas are equivalent to pasture in good hydrologic condition, and
- Impervious areas have a CN of 98 and are directly connected to the drainage system.

If all of the impervious area is directly connected to the drainage system, but the impervious area percentages or the pervious land use assumptions in Table 2.1.5-1 are not applicable, use Figure 2.1.5-3 to compute a composite CN. For example, Table 2.1.5-1 gives a CN of 70 for a 1/2-acre lot in hydrologic soil group B, with an assumed impervious area of 25%. However, if the lot has 20% impervious area and a pervious area CN of 61, the composite CN obtained from Figure 2.1.5-3 is 68. The CN difference between 70 and 68 reflects the difference in percent impervious area.

Table 2.1.5-1 Runoff Curve Numbers[1]

Cover Description		**Curve numbers for hydrologic soil groups**			
Cover type and hydrologic condition	*Average percent impervious area[2]*	A	B	C	D
Cultivated Land:					
Without conservation treatment		72	81	88	91
With conservation treatment		62	71	78	81
Pasture or range land:					
Poor condition		68	79	86	89
Good condition		39	61	74	80
Meadow:					
Good condition		30	58	71	78
Wood or forest land:					
Thin stand, poor cover		45	66	77	83
Good cover		25	55	70	77
Open space (lawns, parks, golf courses, cemeteries, etc.)[3]					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved; curbs and storm drains (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Urban districts:					
Commercial and business	85%	89	92	94	95
Industrial	72%	81	88	91	93

Table 2.1.5-1 Runoff Curve Numbers[1]

Cover Description		**Curve numbers for hydrologic soil groups**			
Cover type and hydrologic condition	*Average percent impervious area[2]*	A	B	C	D
Residential districts by average lot size:					
1/8 acre or less (town house)	65%	77	85	90	92
1/4 acre	38%	61	75	83	87
1/3 acre	30%	57	72	81	86
1/2 acre	25%	54	70	80	85
1 acre	20%	51	68	79	84
2 acres	12%	46	65	77	82
Developing urban areas and newly graded areas (previous areas only, no vegetation)		77	86	91	94

[1] Average runoff condition, and $I_a = 0.2S$

[2] The average percent impervious area shown was used to develop the composite CNs. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. If the impervious area is not connected, the SCS method has an adjustment to reduce the effect.

[3] CNs shown are equivalent to those of pasture. Composite CNs may be computed for other combinations of open space cover type.

Table 2.1.5-1A
Imperviousness for Land Uses

Land Use Classification	Characteristic Imperviousness
Brush	1%
Open	1%
Commercial	85%
Industrial	72%
Residential – Low-Density	25%
Residential – Moderate Density	35%
Residential – High Density	45%
Residential – Multi-Family	72%
Pavement	100%
Water	100%

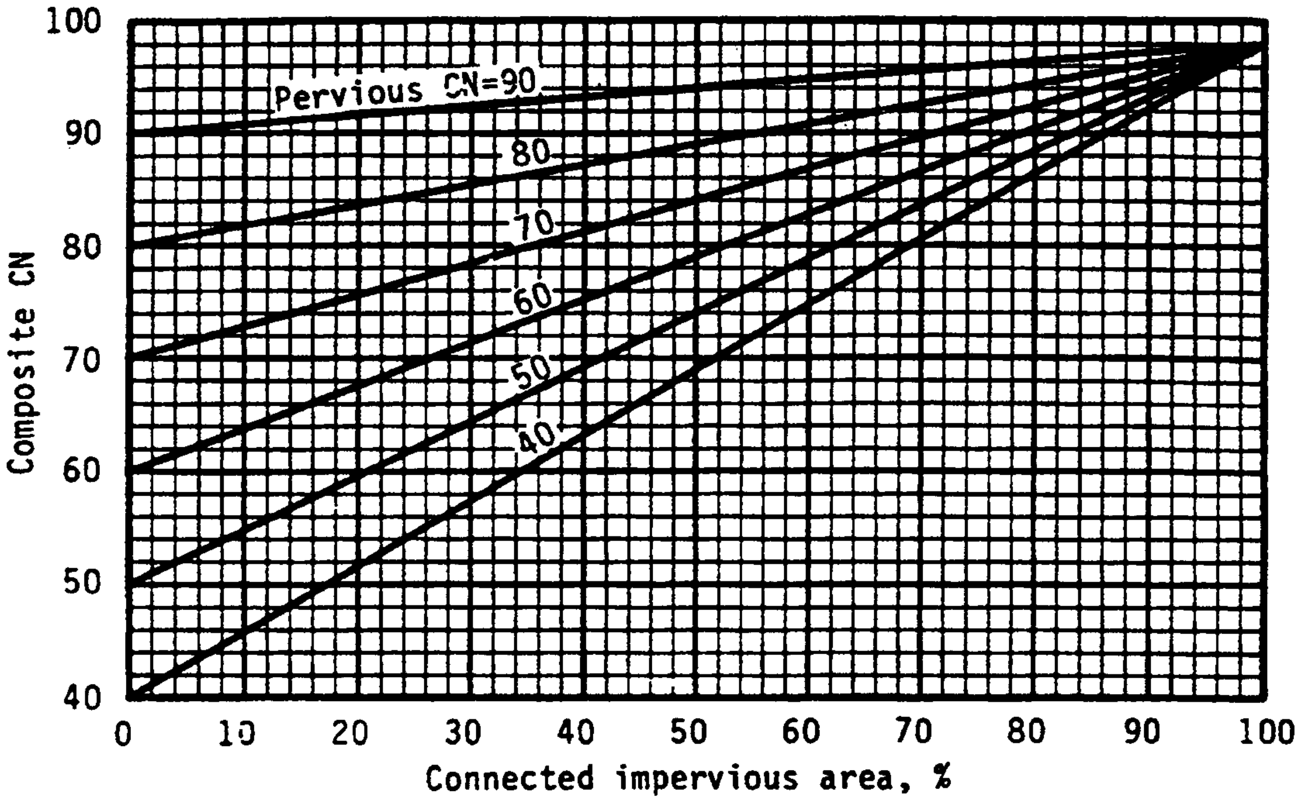


Figure 2.1.5-3 Composite CN with Connected Impervious Areas
(Source: SCS, TR-55, Second Edition, June 1986)

Unconnected Impervious Areas

Runoff from these areas is spread over a pervious area as sheet flow. To determine CN when all or part of the impervious area is not directly connected to the drainage system, (1) use Figure 2.1.5-4 if total impervious area is less than 30% or (2) use Figure 2.1.5-3 if the total impervious area is equal to or greater than 30%, because the absorptive capacity of the remaining pervious areas will not significantly affect runoff.

When the impervious area is less than 30%, obtain the composite CN by entering the right half of Figure 2.1.5-4 with the percentage of total impervious area and the ratio of total unconnected impervious area to total impervious area. Then move left to the appropriate pervious CN and read down to find the composite CN. For example, for a 1/2-acre lot with 20% total impervious area (75% of which is unconnected) and pervious CN of 61, the composite CN from Figure 2.1.5-4 is 66. If all of the impervious area is connected, the resulting CN (from Figure 2.1.5-3) would be 68.

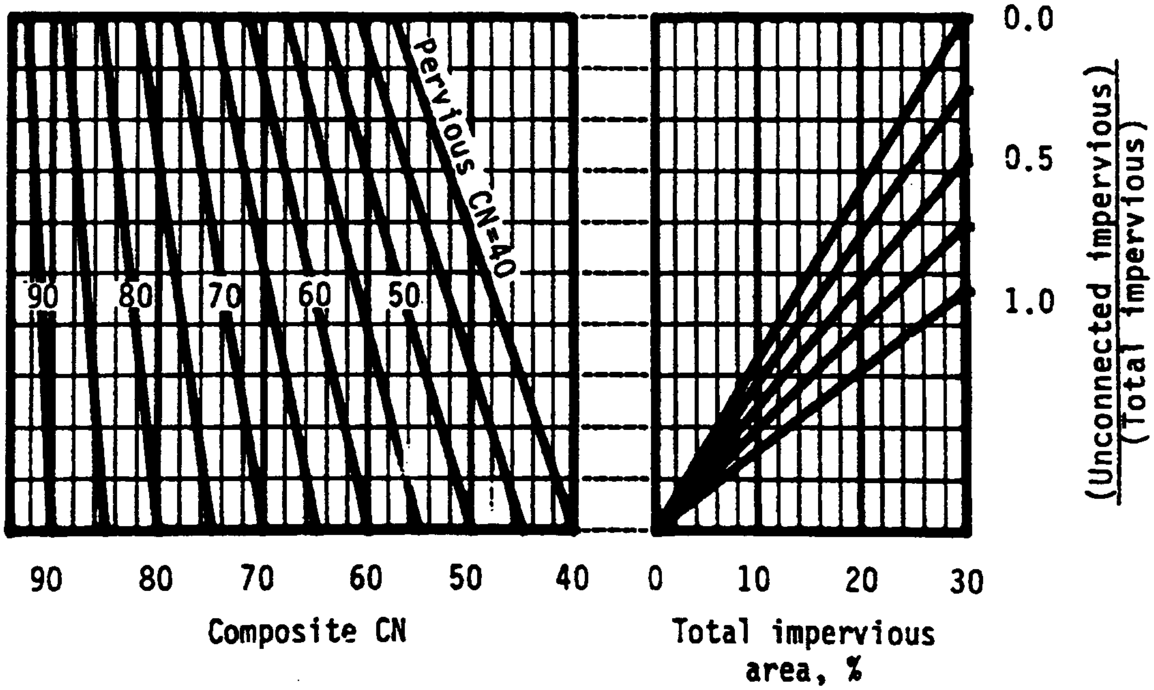


Figure 2.1.5-4 Composite CN with Unconnected Impervious Areas

(Total Impervious Area Less Than 30%)
(Source: SCS, TR-55, Second Edition, June 1986)

2.1.5.6 Travel Time Estimation

Travel time (T_t) is the time it takes water to travel from one location to another within a watershed, through the various components of the drainage system. Time of concentration (t_c) is computed by summing all the travel times for consecutive components of the drainage conveyance system from the hydraulically most distant point of the watershed to the point of interest within the watershed. Following is a discussion of related procedures and equations (USDA, 1986).

Travel Time

Water moves through a watershed as sheet flow, shallow concentrated flow, open channel flow, or some combination of these. The type of flow that occurs is a function of the conveyance system and is best determined by field inspection.

Travel time is the ratio of flow length to flow velocity:

$$T_t = L/3600V \tag{2.1.9}$$

where:

T_t	=	travel time (hr)
L	=	flow length (ft)
V	=	average velocity (ft/s)
3600	=	conversion factor from seconds to hours

Sheet Flow

Sheet flow can be calculated using the following formula:

$$T_t = \frac{0.42\,(nL)^{0.8}}{60\,(P_2)^{0.5}(S)^{0.4}} = \frac{0.007(nL)^{0.8}}{(P_2)^{0.5}(S)^{0.4}} \tag{2.1.10}$$

where:

T_t	=	travel time (hr)
n	=	Manning roughness coefficient (see Table 2.1.5-2)
L	=	flow length (ft),
P_2	=	2-year, 24-hour rainfall
S	=	land slope (ft/ft)

Table 2.1.5-2 Roughness Coefficients (Manning's n) for Sheet Flow[1]	
Surface Description	**n**
Smooth surfaces	
(concrete, asphalt, gravel or bare soil)	0.011
Fallow	
(no residue)	0.05
Cultivated soils:	
Residue cover < 20%	0.06
Residue cover > 20%	0.17
Grass:	
Short grass prairie	0.15
Dense grasses[2]	0.24
Bermuda grass	0.41
Range	
(natural)	0.13
Woods[3]	
Light underbrush	0.40
Dense underbrush	0.80

[1] The n values are a composite of information by Engman (1986).
[2] Includes species such as bluestem grass, buffalo grass, grama grass, and native grass mixtures.
[3] When selecting n, consider cover to a height of about 0.1 ft. This is the only part of the plant cover that will obstruct sheet flow.
Source: SCS, TR-55, Second Edition, June 1986.

Shallow Concentrated Flow

After 50 to 100 feet, sheet flow usually becomes shallow concentrated flow. The average velocity for this flow can be determined from Figure 2.1.5-5, in which average velocity is a function of watercourse slope and type of channel.

Average velocities for estimating travel time for shallow concentrated flow can be computed from using Figure 2.1.5-5, or the following equations. These equations can also be used for slopes less than 0.005 ft/ft.

Unpaved $\mathbf{V = 16.13(S)^{0.5}}$ **(2.1.11)**

Paved $\mathbf{V = 20.33(S)^{0.5}}$ **(2.1.12)**

where:

V = average velocity (ft/s)

S = slope of hydraulic grade line (watercourse slope, ft/ft)

After determining average velocity using Figure 2.1.5-5 or equations 2.1.11 or 2.1.12, use equation 2.1.9 to estimate travel time for the shallow concentrated flow segment.

<u>Open Channels</u>

Velocity in channels should be calculated from the Manning equation. Open channels are assumed to begin where surveyed cross section information has been obtained, where channels are visible on aerial photographs, where channels have been identified by the local municipality, or where stream designations appear on United States Geological Survey (USGS) quadrangle sheets. Manning's equation or water surface profile information can be used to estimate average flow velocity. Average flow velocity for travel time calculations is usually determined for bank-full elevation assuming low vegetation winter conditions.

Manning's equation is

$$\mathbf{V = (1.49/n)\ (R)^{2/3}\ (S)^{1/2}} \qquad \textbf{(2.1.13)}$$

where:

- V = average velocity (ft/s)
- R = hydraulic radius (ft) and is equal to A/P_w
- A = cross sectional flow area (ft^2)
- P_w = wetted perimeter (ft)
- S = slope of the hydraulic grade line (ft/ft)
- n = Manning's roughness coefficient for open channel flow

After average velocity is computed using equation 2.1.13, T_t for the channel segment can be estimated using equation 2.1.9.

LIMITATIONS

- Equations in this section should not be used for sheet flow longer than 50 feet for <u>impervious surfaces.</u>
- In watersheds with storm sewers, carefully identify the appropriate hydraulic flow path to estimate t_c.
- A culvert or bridge can act as detention structure if there is significant storage behind it. Detailed storage routing procedures should be used to determine the outflow through the culvert or bridge.

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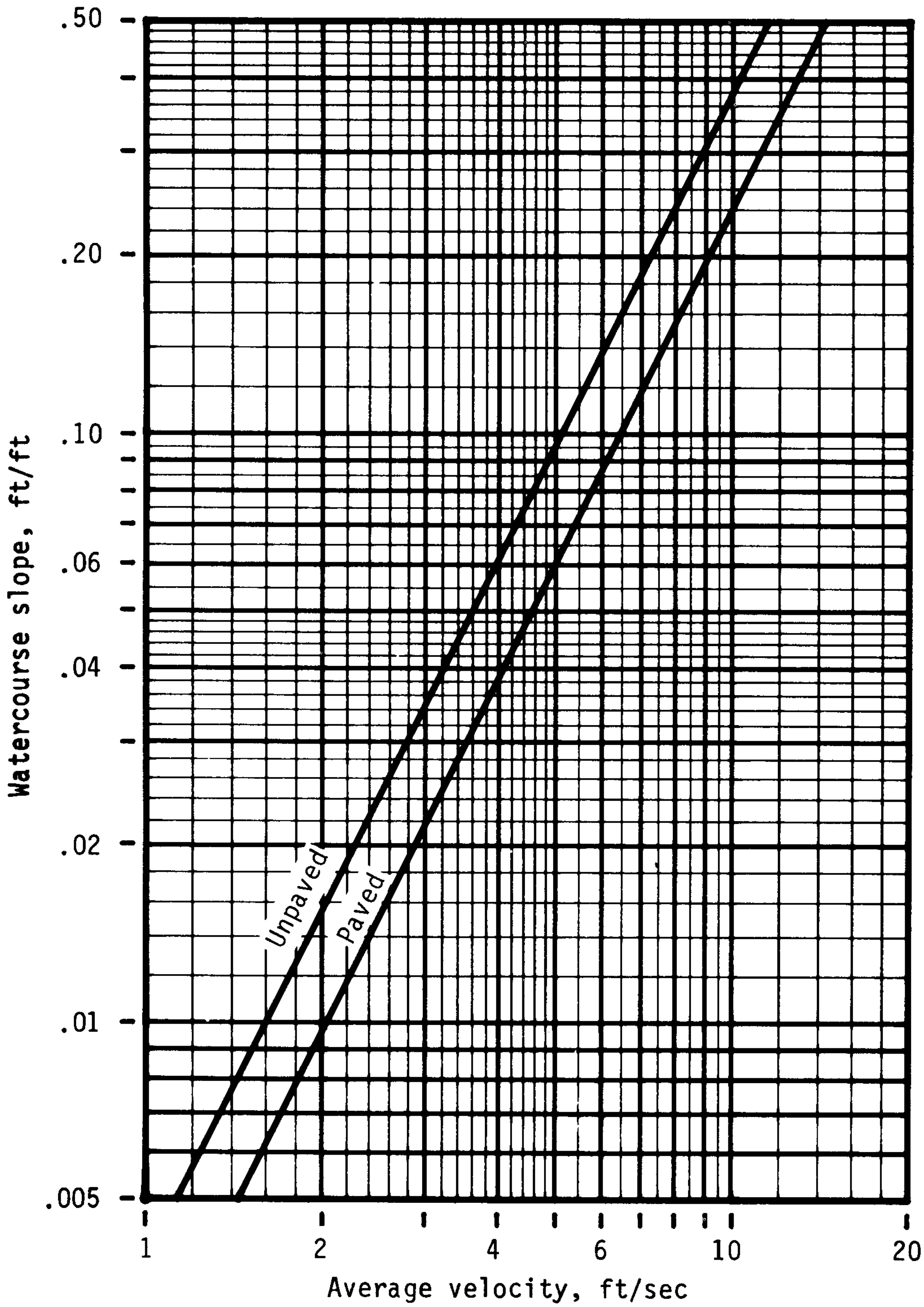


Figure 2.1.5-5 Average Velocities – Shallow Concentrated Flow

(Source: SCS, TR-55, Second Edition, June 1986)

2.1.5.7 Simplified SCS Peak Runoff Rate Estimation

THIS METHOD SHOULD BE ONLY USED WITH SPECIFIC APPROVAL OF THE CITY ENGINEER.

The following SCS procedures were taken from the SCS Technical Release 55 (USDA, 1986) which presents simplified procedures to calculate storm runoff volume and peak rate of discharges. These procedures are applicable to small drainage areas (typically less than 2,000 acres) with homogeneous land uses, which can be described by a single CN value. The peak discharge equation is:

$$Q_p = q_u A Q F_p \tag{2.1.14}$$

where:

Q_p = peak discharge (cfs)
q_u = unit peak discharge (cfs/mi^2/in)
A = drainage area (mi^2)
Q = runoff (in)
F_p = pond and swamp adjustment factor

Computations for the peak discharge method proceed as follows:

- The 24-hour rainfall depth (P) is determined from the rainfall in table 2.1.3-1 for the selected return frequency.
- The runoff curve number, CN, is estimated from Table 2.1.5-1 and direct runoff, Q, is calculated using equation 2.1.7.
- The CN value is used to determine the initial abstraction, I_a, from Table 2.1.5-3, and the ratio I_a/P is then computed (P = accumulated 24-hour rainfall).
- The watershed time of concentration is computed using the procedures in subsection 2.1.5.6 and is used with the ratio I_a/P to obtain the unit peak discharge (q_u) from Figure 2.1.5-6 for the Type II rainfall distribution. If the ratio I_a/P lies outside the range shown in the figures, either the limiting values or another peak discharge method should be used. Note: Figure 2.1.5-6 is based on a peaking factor of 484. If a peaking factor of 300 is needed, these figures are not applicable and the simplified SCS method should not be used. Peaking factors are discussed further in Section 2.1.5.9.
- The pond and swamp adjustment factor, F_p, is estimated from below:

Pond and Swamp Areas (%*)	F_p
0	1.00
0.2	0.97
1.0	0.87
3.0	0.75
5.0	0.72

*Percent of entire drainage basin

- The peak runoff rate is computed using equation 2.1.14.

Table 2.1.5-3 I_a Values for Runoff Curve Numbers

Curve Number	I_a (in)	Curve Number	I_a (in)
40	3.000	70	0.857
41	2.878	71	0.817
42	2.762	72	0.778
43	2.651	73	0.740
44	2.545	74	0.703
45	2.444	75	0.667
46	2.348	76	0.632
47	2.255	77	0.597
48	2.167	78	0.564
49	2.082	79	0.532
50	2.000	80	0.500
51	1.922	81	0.469
52	1.846	82	0.439
53	1.74	83	0.410
54	1.704	84	0.381
55	1.636	85	0.353
56	1.571	86	0.326
57	1.509	87	0.299
58	1.448	88	0.273
59	1.390	89	0.247
60	1.333	90	0.222
61	1.279	91	0.198
62	1.226	92	0.174
63	1.175	93	0.151
64	1.125	94	0.128
65	1.077	95	0.105
66	1.030	96	0.083
67	0.985	97	0.062
68	0.941	98	0.041
69	0.899		

Source: SCS, TR-55, Second Edition, June 1986

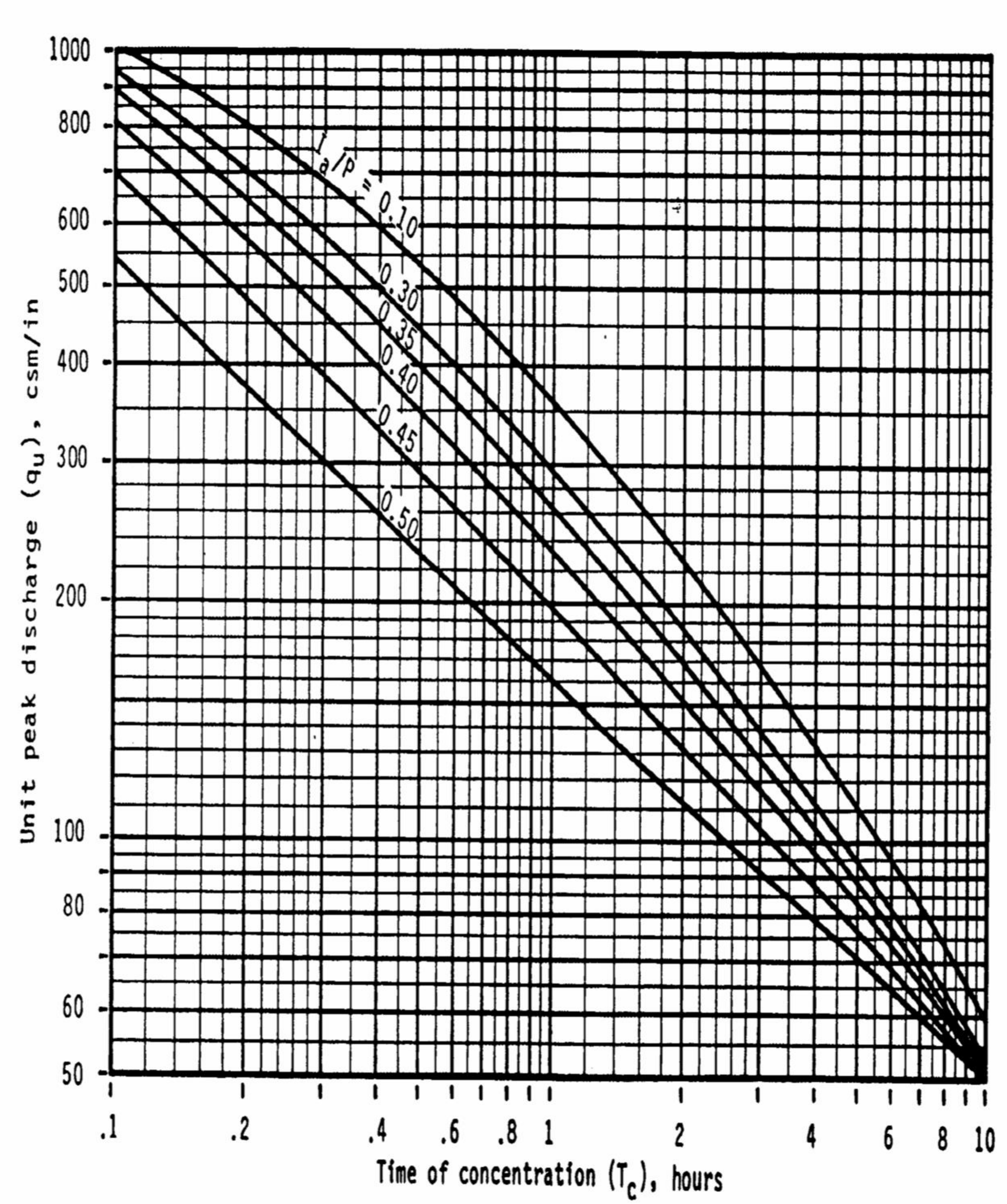


Figure 2.1.5-6 SCS Type II Unit Peak Discharge Graph

(Source: SCS, TR-55, Second Edition, June 1986)

2.1.5.8 Example Problem 1

Compute the 100-year peak discharge for a 50-acre watershed, which will be developed as follows:

- Pasture / range land - good condition (hydrologic soil group D) = 10 ac
- Pasture / range land - good condition (hydrologic soil group C) = 10 ac
- 1/3 acre residential (hydrologic soil group D) = 20 ac
- Industrial development (hydrological soil group C) = 10 ac

Other data include the following: Total impervious area = 18 acres, % of pond / swamp area = 0

Computations

Calculate rainfall excess:

- The 100-year, 24-hour rainfall is 9.36 inches (.39 in/hr x 24 hours – From Table).
- Composite weighted runoff coefficient is:

Dev. #	Area	% Total	CN	Composite CN
1	10 ac.	0.20	80	18.2
2	10 ac.	0.20	74	14.8
3	20 ac.	0.40	86	34.4
4	10 ac.	0.20	91	18.2
Total	50 ac.	1.00		83

* from Equation 2.1.7 Q (100-year) = 7.28 inches

Calculate time of concentration

The hydrologic flow path for this watershed = 1,890 ft

Segment	Type of Flow	Length (ft)	Slope (%)
1	Overland n = 0.24	40	2.0
2	Shallow channel (unpaved)	750	1.7
3	Main channel*	1100	0.50

* For the main channel, n = .06 (estimated), width = 10 feet, depth = 2 feet, rectangular channel

Segment 1 - Travel time from equation 2.1.10 with P_2 = 3.36 inches
(0.14 x 24 – Table 2.1.3-1)

$T_t = [0.42(0.24 \text{ X } 40)^{0.8}] / [(3.36)^{0.5} (.020)^{0.4}] = 6.69$ minutes

Segment 2 - Travel time from Figure 2.1.5-5 or equation 2.1.11
V = 2.1 ft/sec (from equation 2.1.11)
T_t = 750 / 60 (2.1) = 5.95 minutes

Segment 3 - Using equation 2.1.13
$V = (1.49/.06)\ (1.43)^{0.67}\ (.005)^{0.5} = 2.23$ ft/sec
T_t = 1100 / 60 (2.23) = 8.22 minutes

t_c = 6.69 + 5.95 + 8.22 = 20.86 minutes (.35 hours)

Calculate I_a/P for CN = 83 (Table 2.1.5-1), I_a = .410 (Table 2.1.5-3)

$I_a/P = (.410 / 9.36) = .05$
(Note: Use $I_a/P = .10$ to facilitate use of Figure 2.1.5-6.)

Unit discharge q_u (100-year) from Figure 2.1.5-6 = 650 csm/in

Calculate peak discharge with $F_p = 1$ using equation 2.1.14

$Q_{100} = 650\ (50/640)(7.28)(1) = 370$ cfs

2.1.5.9 Hydrograph Generation

In addition to estimating the peak discharge, the SCS method can be used to estimate the entire hydrograph from a drainage area. The SCS has developed a Tabular Hydrograph procedure that can be used to generate the hydrograph for small drainage areas (less than 2,000 acres). The Tabular Hydrograph procedure uses unit discharge hydrographs that have been generated for a series of time of concentrations. In addition, SCS has developed hydrograph procedures to be used to generate composite flood hydrographs. For the development of a hydrograph from a homogeneous developed drainage area and drainage areas that are not homogeneous, where hydrographs need to be generated from sub-areas and then routed and combined at a point downstream, the engineer is referred to the procedures outlined by the SCS in the 1986 version of TR-55 available from the National Technical Information Service in Springfield, Virginia 22161. The catalog number for TR-55, "Urban Hydrology for Small Watersheds," is PB87-101580.

The unit hydrograph equations used in the SCS method for generating hydrographs includes a constant to account for the general land slope in the drainage area. This constant, called a peaking factor, can be adjusted when using the method. A default value of 484 for the peaking factor represents rolling hills – a medium level of relief. SCS indicates that for mountainous terrain the peaking factor can go as high as 600, and as low as 300 for flat (coastal) areas.

A value of 484 should be used for most areas of North Texas; however, there are flat areas where a lesser value may be appropriate.

The development of a runoff hydrograph from a watershed is a laborious process not normally done by hand calculation. For that reason, only an overview of the process is given here to assist the designer in reviewing and understanding the input and output from a typical computer program. There are choices of computational interval, storm length (if the 24-hour storm is not going to be used), and other “administrative” parameters, which are applicable to each computer program.

The development of a runoff hydrograph for a watershed or one of many sub-basins within a more complex model involves the following steps:

- Development or selection of a design storm hyetograph. Often the SCS 24-hour storm described in subsection 2.1.5.3 is used. This storm is recommended for use in North Texas.
- Development of curve numbers and lag times for the watershed using the methods described in subsections 2.1.5.4, 2.1.5.5, and 2.1.5.6.
- Development of a unit hydrograph using the standard (peaking factor of 484) dimensionless unit hydrograph. See discussion below.
- Step-wise computation of the initial and infiltration rainfall losses and, thus, the excess rainfall hyetograph using a derivative form of the SCS rainfall-runoff equation (Equation 2.1.7).
- Application of each increment of excess rainfall to the unit hydrograph to develop a series of runoff hydrographs, one for each increment of rainfall (this is called “convolution”).
- Summation of the flows from each of the small incremental hydrographs (keeping proper track of time steps) to form a runoff hydrograph for that watershed or sub-basin.

To assist the designer in using the SCS unit hydrograph approach with a peaking factor of 484, Figure 2.1.5-7 and Table 2.1.5-4 have been developed. The unit hydrograph with a peaking factor of 300 is shown in the figure for comparison purposes, but, typically, should not be used for areas in North Texas.

The procedure to develop a unit hydrograph from the dimensionless unit hydrograph in the table below is to multiply each time ratio value by the time-to-peak (T_p) and each value of q/q_u by q_u calculated as:

$$q_u = (PF\ A) / (T_p) \qquad \textbf{(2.1.15)}$$

where:

q_u = unit hydrograph peak rate of discharge (cfs)

PF = peaking factor (484)

A = area (mi^2)

d = rainfall time increment (hr)

T_p = time to peak = $d/2 + 0.6\ t_c$ (hr)

For ease of spreadsheet calculations, the dimensionless unit hydrograph for 484 can be approximated by the equation:

$$\frac{q}{qu} = \left(\frac{t}{T_p}\ e^{[1-(t/Tp)]} \right)^{X} \qquad \textbf{(2.1.16)}$$

where X is 3.79 for the PF=484 unit hydrograph.

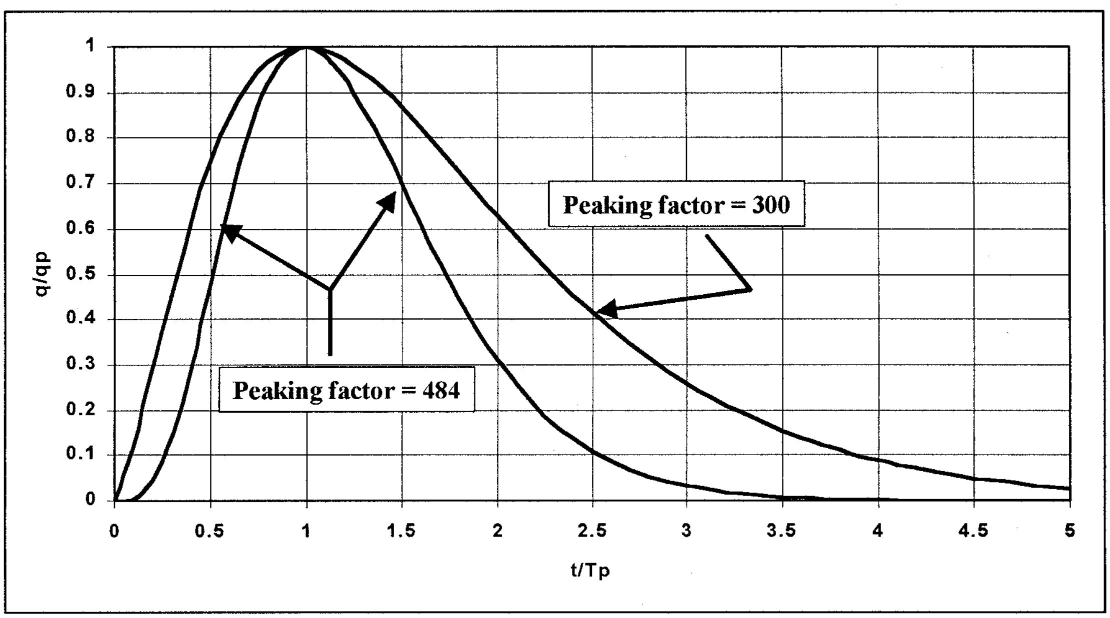


Figure 2.1.5-7 Dimensionless Unit Hydrographs for Peaking Factors of 484 and 300

Table 2.1.5-4 Dimensionless Unit Hydrograph With Peaking Factor of 484

	484	
t/T_t	q/q_u	Q/Q_p
0.0	0.0	0.0
0.1	0.005	0.000
0.2	0.046	0.004
0.3	0.148	0.015
0.4	0.301	0.038
0.5	0.481	0.075
0.6	0.657	0.125
0.7	0.807	0.186
0.8	0.916	0.255
0.9	0.980	0.330
1.0	1.000	0.406
1.1	0.982	0.481
1.2	0.935	0.552
1.3	0.867	0.618
1.4	0.786	0.677
1.5	0.699	0.730
1.6	0.611	0.777
1.7	0.526	0.817
1.8	0.447	0.851
1.9	0.376	0.879
2.0	0.312	0.903
2.1	0.257	0.923
2.2	0.210	0.939
2.3	0.170	0.951
2.4	0.137	0.962
2.5	0.109	0.970
2.6	0.087	0.977
2.7	0.069	0.982
2.8	0.054	0.986
2.9	0.042	0.989
3.0	0.033	0.992
3.1	0.025	0.994
3.2	0.020	0.995
3.3	0.015	0.996
3.4	0.012	0.997
3.5	0.009	0.998
3.6	0.007	0.998
3.7	0.005	0.999
3.8	0.004	0.999
3.9	0.003	0.999
4.0	0.002	1.000

2.1.5.10 Example Problem 2

Compute the unit hydrograph for the 50-acre watershed in example 2.1.5.8.

Computations

Calculate T_p and time increment

The time of concentration (t_c) is calculated to be 20.86 minutes for this watershed. If we assume a computer calculation time increment (d) of 3 minutes then:

T_p = d/2 + 0.6t_c = 3/2 + 0.6 * 20.86 = 14.02 minutes (0.234 hrs)

Calculate q_{pu}

q_u = PF A/T_p = (484 * 50/640) / (0.234) = 162 cfs

Calculate unit hydrograph.

Based on spreadsheet calculations using equations 2.1.15 and 2.1.16, the table below has been derived.

Time		484	
t/T_p	time (min)	q/q_u	Q
0	0	0	0.00
0.21	3	0.06	9.23
0.43	6.0	0.35	56.77
0.64	9.0	0.72	117.29
0.86	12.0	0.96	155.09
1.00	14.02	1.00	162.00
1.07	15.0	0.99	160.57
1.28	18.0	0.88	142.42
1.50	21.0	0.70	113.52
1.71	24.0	0.52	83.69
1.93	27.0	0.36	58.12
2.14	30.0	0.24	38.51
2.35	33.0	0.15	24.56
2.57	36.0	0.09	15.18
2.78	39.0	0.06	9.14
3.00	42.0	0.03	5.38
3.21	45.0	0.02	3.10
3.42	48.0	0.01	1.76
3.64	51.0	0.01	0.99
3.85	54.0	0.00	0.54
4.07	57.0	0.00	0.30
4.28	60.0	0.00	0.16
4.49	63.0	0.00	0.09
4.71	66.0	0.00	0.05
4.92	69.0	0.00	0.02
5.14	72.0	0.00	0.01
5.35	75.0	0.00	0.01
5.56	78.00	0.00	0.00

2.1.5.11 Hydrologic Stream Routing

Water requires a certain amount of time to travel down a stream or channel reach. A flood wave is attenuated by friction and channel storage as it passes through the reach. The process of computing the travel time and attenuation of water flowing in the reach is often called routing.

Hydrologic routing involves the balancing of inflow, outflow, and volume of storage through the use of the continuity equation. The relation between the outflow rate and storage in the system is also required.

Travel time and attenuation characteristics vary widely between different streams. The travel time is dependent on characteristics such as length, slope, friction, and flow depth. Attenuation is also dependent on friction, in addition to other characteristics such as channel storage. Many routing methods have been developed under different assumptions and for different stream types. Some of the routing methods include: kinematic wave, lag, modified Puls, Muskingum, Muskingum-Cunge 8-point section, and Muskingum-Cunge standard section.

The routing methods selected for use in Copper Canyon are the Modified Puls and the Muskingum-Cunge methods (USACE, HEC-HMS, 2000 and Bedient and Huber, 1988).

2.1.6 – Snyder's Unit Hydrograph Method

2.1.6.1 Introduction

Snyder's unit hydrograph method is a method utilized by the Corps of Engineers Fort Worth District for hydrologic studies in the Dallas –Fort Worth region, and is also commonly used by consultants and other entities within the NCTCOG region. It is similar in nature to the SCS method, in that it also considers the time distribution of the rainfall, the initial rainfall losses to interception and depression storage, and an infiltration rate that decreases during the course of a storm. **THIS METHOD IS ONLY ALLOWED WITH THE SPECIFIC APPROVAL OF THE TOWN ENGINEER**.

2.1.6.2 Application

Snyder's unit hydrograph method may be used for drainage areas 100 acres or larger. This method, detailed in the U.S. Army Corps of Engineers Engineering Manual (EM 1110-2-1405), *Flood-Hydrograph Analysis and Computations* and The Bureau of Reclamation's "Flood Hydrology Manual, A Water Resources Technical Publication," utilizes the following equations:

$$t_p = C_t (L\ L_{ca})^{0.3} \quad (2.1.17)$$

$$t_r = t_p \div 5.5 \quad (2.1.18)$$

$$q_p = C_p 640 \div t_p \quad (2.1.19)$$

$$t_{pR} = t_p + 0.25(t_R - t_r) \quad (2.1.20)$$

$$q_{pR} = C_p 640 \div t_{pR} \quad (2.1.21)$$

$$q_{pR} = q_p\, t_p \div t_{pR} \quad (2.1.22)$$

$$Q_p = q_p\, A \quad (2.1.23)$$

The terms in the above equations are defined as:

t_r = The standard unit rainfall duration, in hours.

t_R = The unit rainfall duration in hours other than standard unit, t_r, adopted in specific study.

t_p = The lag time from midpoint of unit rainfall duration, t_r, to peak of unit hydrograph in hours.

t_{pR} = The lag time from midpoint of unit rainfall duration, t_R, to peak of unit hydrograph in hours.

q_p = The peak rate of discharge of unit hydrograph for unit rainfall duration, t_r, in cfs/sq. mi.
q_{pR} = The peak rate of discharge in cfs/sq mi. of unit in hydrograph for unit rainfall duration, t_R.
Q_p = The peak rate of discharge of unit hydrograph in cfs.
A = The drainage area in square miles.
L_{ca} = The river mileage from the design point to the centroid of gravity of the drainage area.
L = The river mileage from the given station to the upstream limits of the drainage area.
C_t = Coefficient depending upon units and drainage basin characteristics.
C_p = Coefficient depending upon units and drainage basin characteristics.

The coefficient C_t is a regional coefficient for variations in slopes within the watershed. Typical values of C_t for eastern Texas range from 1.0 to 2.9 The value of C_t for the East Fork Trinity River is 2.0. C_t for a watershed can be estimated if the lag time, t_p, stream length, L, and distance to the basin centroid, L_{ca}, are known. Lag times can also be calculated based on the Travel Time (Tt) as defined in Section 2.1.5.6 based on the equation Tt x 0.6. The coefficient C_p is the peaking coefficient, which typically ranges from 0.3 to 1.2 with an average value of 0.75, and is related to the flood wave and storage conditions of the watershed. The C_p value for the East Fork Trinity River is 0.69. Larger values of C_p are generally associated with smaller values of C_t. Typical values of C_p are listed in Table 2.1.6-1.

Table 2.1.6-1 Typical Values of C_p

Typical Drainage Area Characteristics	Value of C_p
Undeveloped Areas w/ Storm Drains	
Flat Basin Slope (less than 0.50%)	0.55
Moderate Basin Slope (0.50% to 0.80%)	0.58
Steep Basin Slope (greater than 0.80%)	0.61
Moderately Developed Area	
Flat Basin Slope (less than 0.50%)	0.63
Moderate Basin Slope (0.50% to 0.80%)	0.66
Steep Basin Slope (greater than 0.80%)	0.69
Highly Developed/Commercial Area	
Flat Basin Slope (less than 0.50%)	0.70
Moderate Basin Slope (0.50% to 0.80%)	0.73
Steep Basin Slope (greater than 0.80%)	0.77

2.1.6.3 Determination of Percent Urbanization and Percent Sand

The lag time, t_p, is the critical parameter in establishing the timing of the response of a watershed to rainfall. The degree of urbanization is an important variable that determines the value of the lag time. Thomas L. Nelson, Fort Worth District, USACE, defined the general relationship between the lag time, t_p, and the percent of Urbanization, %Urb, and presented a set of Urbanization Curves for the Dallas-Fort Worth area in 1970.

The soil type of a watershed also plays an important role in its response to rainfall. It was found that predominantly sandy soils responded differently to rainfall than predominantly clayey soils. Therefore, two sets of Urbanization Curves were developed to better define the lag time, one set for sandy soils and one set for clayey soils. A paper by Paul K. Rodman, Fort Worth District, USACE presented urbanization curves in 1977 for both "clay loam" and "clay" in the Fort Worth-Dallas area and other Texas locations.

Stream Routing

The Modified Puls and Muskingum-Cunge are acceptable routing methods. See Section 2.1.5.11 for an explanation of routing methods and references for further information.

2.1.7 – USGS and TxDOT Regression Methods

2.1.7.1 Introduction

Regional regression equations are the most commonly accepted method for establishing peak flows at larger ungauged sites (or sites with insufficient data for a statistical derivation of the flood versus frequency relation). Regression equations have been developed to relate peak flow at a specified return period to the physiography, hydrology, and meteorology of the watershed.

Regression analyses use stream gauge data to define hydrologic regions. These are geographic regions having very similar flood frequency relationships and, as such, commonly display similar watershed, channel, and meteorological characteristics; they are often termed hydrologically homogeneous geographic areas. For this manual, the USGS regression equations are used to determine peak flows in urban drainage areas, and the TXDOT regression equations are used to determine peak flows in rural drainage areas. It may be difficult to choose the proper set of regression equations when the design site lies on or near the hydrologic boundaries of relevant studies. Another problem occurs when the watershed is partly or totally within an area subject to mixed population floods.

The following suggestions should be considered when using regression equations:

- Conduct a field visit to compare and assess the watershed characteristics for comparison with other watersheds.
- Collect all available historical flood data.
- Use the gathered data to interpret any discharge values.

2.1.7.2 TxDOT Equations for Rural (or Undeveloped) Basins

The Texas Department of Transportation (TxDOT) has a regression method for estimating peak discharges for rural basins. For a complete discussion of the development of these equations consult Chapter 5, Section 11 of the TxDOT Hydraulic Design Manual, available online at http://manuals.dot.state.tx.us/docs/colbridg/forms/hyd.pdf or the reference USGS, 1997.

2.1.7.3 Rural (or Undeveloped) Basin Application

Equation 2.1.24 applies to rural, uncontrolled watersheds. Figure 2.1.7-1 presents the geographic extents of each region. Note that Copper Canyon lies within Region 7. Table 2.1.7-2 presents the coefficients and limits of applicability for Region 7. Generally, use this equation to compare with the results of other methods, check existing structures, or where it is not practicable to use any other method, keeping in mind the importance of the facility being designed.

$$Q_T = aA^bSH^cSL^d \qquad (2.1.24)$$

where:

- Q_T = T-year discharge (cfs)
- A = contributing drainage area (sq. mi.)
- SH = basin-shape factor defined as the ratio of main channel length squared to contributing drainage area (sq. mi./sq. mi.)
- SL = mean channel slope defined as the ratio of headwater elevation of longest channel minus main channel elevation at site to main channel length (ft./mi.). Note: This differs from previous rural regression equations in which slope was defined between points 10 and 85 percent of the distance along the main channel from the outfall to the basin divide.
- a, b, c, d = multiple linear regression coefficients dependent on region number and frequency.

The equation to be used for Region 7 is found in Table 2.1.7-2.

Regions 3, 4, and 7 have two sets of coefficients. For these regions, if the drainage area is between 10 and 100 sq. mi., determine a weighted discharge (Q_w) as shown in Equation 2.1.30.

$$Q_w = (2 - \log(A/z))Q_1 + (\log(A/z)-1)Q_2 \qquad (2.1.25)$$

where:

- Q_w = weighted discharge (cfs)
- A = contributing drainage area (sq. mi.)
- z = 1.0 for English measurements units
- Q_1 = discharge based on regression coefficients for A < 32 sq. mi. (cfs)
- Q_2 = discharge based on regression coefficients for A ≥ 32 sq. mi. (cfs)

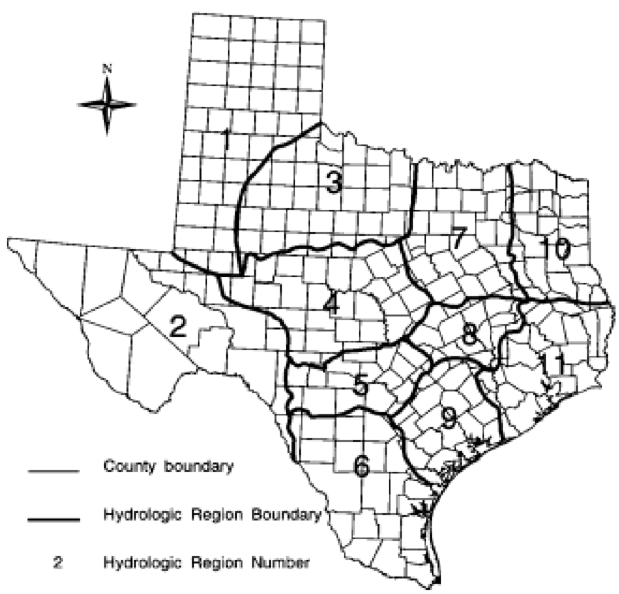


Figure 2.1.7-1 Hydrologic Regions for Statewide Rural Regression Equations

Source: TXDOT, 2002

Table 2.1.7-2 Regression Equations for Estimation of Peak-Streamflow Frequency for Hydrologic Regions of Texas[1]		
[yr, year; A, contributing drainage area in square miles; SH, basin shape factor – ration of length of longest mapped channel (stream length) squared to contributing drainage area (dimensionless); SL, stream slope in feet per mile – ration of change in elevation of (1) longest mapped channel from site (or station) to headwaters to (2) length of longest mapped channel]		
Hydrologic region and recurrence interval	**Weighted least-squares regression equation for corresponding recurrence interval**	**Range of indicated independent variables in corresponding region (units as noted)**
Region 7 (sites with contributing drainage area less than 32 square miles)[2]		
2 yr	$Q_2 = 832\ A^{0.568} SL^{-0.285}$	A: 0.20 to 78.7
5 yr	$Q_5 = 584\ A^{0.610}$	
10 yr	$Q_{10} = 831.2\ A^{0.592}$	SH: 0.037 to 36.6
25 yr	$Q_{25} = 1196\ A^{0.576}$	
50 yr	$Q_{50} = 1505\ A^{0.566}$	SL: 7.25 to 116
100 yr	$Q_{100} = 1842\ A^{.558}$	
Region 7 (sites with contributing drainage area greater than 32 square miles)[2]		
2 yr	$Q_2 = 129\ A^{0.578}\ SL^{0.364}$	A: 13 to 2,615
5 yr	$Q_5 = 133\ A^{0.605}\ SL^{0.578}$	
10 yr	$Q_{10} = 178\ A^{0.644}\ SL^{0.699}\ SH^{-0.239}$	SH: 1.66 to 36.6
25 yr	$Q_{25} = 219\ A^{0.651}\ SL^{0.776}\ SH^{-0.267}$	
50 yr	$Q_{50} = 261\ A^{0.653}\ SL^{0.817}\ SH^{-0.291}$	SL: 3.85 to 31.9
100 yr	$Q_{100} = 313\ A^{0.654}\ SL^{0.849}\ SH^{-0.316}$	
1. Source: U.S.G.S., 1997, pp. 62-65.		
2. Use Equation 2.1.29 to calculate a weighted discharge for streams with contributing drainage area falling within the arrange of 10 to 100 square miles.		

2.1.7.4 Example Problem

For the 100-year flood, calculate the peak discharge for a rural drainage area on Bayou Lanana at Nacogdoches, Texas.

- DRAINAGE AREA = 38.8 MI^2
- MAIN CHANNEL SLOPE = 13.13 FT/MI
- Main Channel Length= 14.24 mi.
- Shape Factor = $(\text{channel miles})^2$ divided by Area = 5.23

<u>Peak Discharge Calculations</u>

The 100-year Rural Peak Discharge determination will necessitate the use of Equations 2.1.24 and 2.1.25 because the drainage area is in the range of 10-100.

$Q_1 \quad = 1842\ A^{0.558} = 1842(38.8)^{0.558} = 14{,}186$ cfs

$Q_2 \quad = 313\ A^{0.654}\ SL^{0.849}\ SH^{-0.316}$

$= 313(38.8)^{0.654}\ (13.13)^{0.849}\ (5.23)^{-0.316}$

$= 18{,}072$ cfs

$Q_{100} = (2-\log(A))Q_1 + (\log(A)-1)Q_2$

$= (2 - \log(38.8))14{,}186 + (\log(38.8)-1)\ 18{,}072$ cfs

$= 16{,}474$ cfs

Section 2.1.8 – Downstream Hydrologic Assessment

A major objective of the storm water management criteria presented in this Manual is to protect downstream properties from flood and erosion impacts due to upstream development. Some entity's criteria require the designer to control peak flow at the outlet of a site such that post-development peak discharge equals pre-development peak discharge. It has been shown that in certain cases this does not always provide effective water quantity control downstream from the site and may actually exacerbate flooding problems downstream. The reasons for this have to do with (1) the timing of the flow peaks, and (2) the total increase in volume of runoff. Further, due to a site's location within a watershed, there may be very little reason for requiring flood control from a particular site. This section outlines a suggested procedure for determining the impacts of post-development storm water peak flows and volumes that a community may require as part of a developer's storm water management site plan.

2.1.8.1 Reasons for Downstream Problems

<u>Flow Timing</u>

If water quantity control (detention) structures are indiscriminately placed in a watershed and changes to the flow timing are not considered, the structural control may actually increase the peak discharge downstream. The reason for this may be seen in Figure 2.1.8-1. The peak flow from the site is reduced appropriately, but the timing of the flow is such that the combined detained peak flow (the larger dashed triangle) is actually higher than if no detention were required.

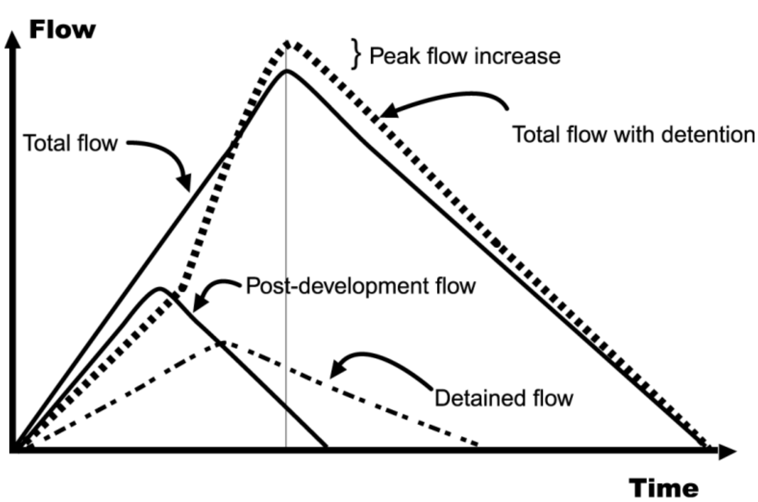


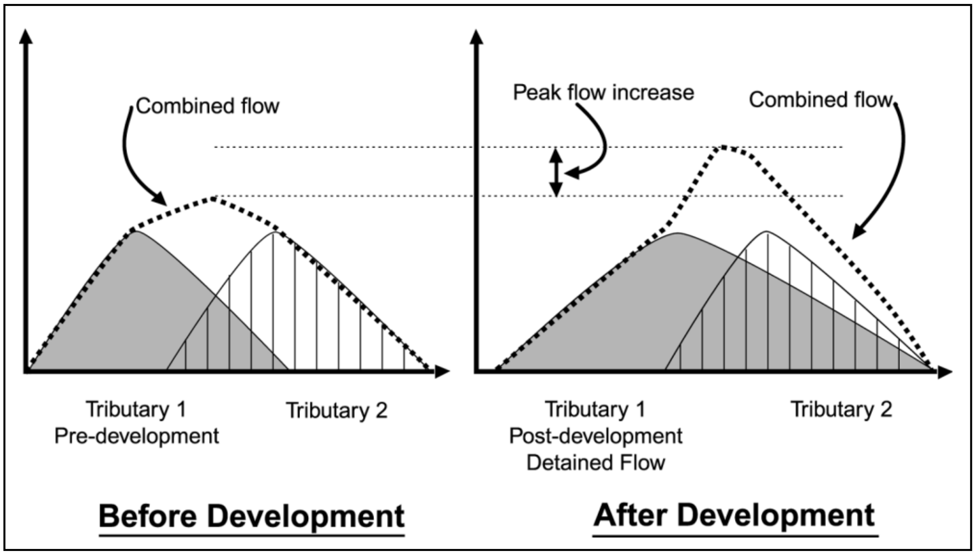
Figure 2.1.8-1 Detention Timing Example

In this case, the shifting of flows to a later time brought about by the detention pond actually makes the downstream flooding worse than if the post-development flows were not detained. This is most likely to happen if detention is placed on tributaries towards the bottom of the watershed, holding back peak flows and adding them as the peak from the upper reaches of the watershed arrives.

Increased Volume

An important impact of new development is an increase in the total runoff volume of flow. Thus, even if the peak flow is effectively attenuated, the longer duration of higher flows due to the increased volume may combine with downstream tributaries to increase the downstream peak flows. Figure 2.1.9-2 illustrates this concept. The figure shows the pre- and post-development hydrographs from a development site (Tributary 1). The post-development runoff hydrograph meets the flood protection criteria (i.e., the post-development peak flow is equal to the pre-development peak flow at the outlet from the site). However, the post-development combined flow at the first downstream tributary (Tributary 2) is higher than pre-development combined flow. This is because the increased volume and timing of runoff from the developed site increases the combined flow and flooding downstream. In this case, the detention volume would have to have been increased to account for the downstream timing of the combined hydrographs to mitigate the impact of the increased runoff volume.

Figure 2.1.8-2 Effect of Increased Post-Development Runoff Volume with Detention on a Downstream Hydrograph



2.1.8.2 Methods for Downstream Evaluation

The downstream assessment is a tool by which the impacts of development on storm water peak flows and velocities are evaluated downstream. The assessment extends from an outfall of a development to a point downstream, determined by one of two methods:

- *Zone of Influence* – Point downstream where the discharge from a proposed development no longer has a significant impact upon the receiving stream or storm drainage system
- *Adequate Outfall* – Location of acceptable outfall that does not create adverse flooding or erosion conditions downstream

These methods recognize the fact that a structural control providing detention has a "zone of influence" downstream where its effectiveness can be felt. Beyond this zone of influence the storm water effects of a structural control become relatively small and insignificant compared to the runoff from the total drainage area at that point. Based on studies and master planning results for a large number of sites, a general rule of thumb is that zone of influence can be considered to be the point where the drainage area controlled by the detention or storage facility comprises 10% of the total drainage area. This is known as the *10% Rule*. As an example, if a structural control drains 10 acres, the zone of influence ends at the point where the total drainage area is 100 acres or greater.

Typical steps in a downstream assessment include:

1. Determine the outfall location of the site and the pre- and post-development site conditions.
2. Using a topographic map determine a preliminary lower limit of the zone of influence (approximately 10% point).
3. Using a hydrologic model determine the pre-development peak flows and velocities at each junction beginning at the development outfall and ending at the next junction beyond the 10% point. Undeveloped off-site areas are modeled as "full build-out" for both the pre- and post-development analyses. The discharges and velocities are evaluated for three storms:

"Streambank Protection" storm, 2- year, 24-hour event

"Conveyance" storm, 25-year, 24-hour event

100-year, 24-hour storm event

4. Change the land use on the site to post-development conditions and rerun the model.
5. Compare the pre- and post-development peak discharges and velocities at the downstream end of the model. If the post-developed flows are higher than the pre-developed flows for the same frequency event, or the post-developed velocities are higher than the allowable velocity of the downstream receiving system, extend the model downstream. Repeat steps 3 and 4 until the post-development flows are less than the pre-developed flows, and the post-developed velocities are below the allowable velocity. Allowable velocities are given in Tables 4.4-2 and 4.4-3 in Chapter 4. (See guidelines in Section 1.2.2)
6. If shown that no peak flow increases occur downstream, and post-developed velocities are allowable, then the control of the flood protection volume (Q_f) can be waived by the local authority. The developer saves the cost of sizing a detention basin for flood control. In this case the downstream assessment saved the construction of an unnecessary structural control facility that would have been detrimental to the watershed flooding problems. In some communities this situation may result in a fee being paid to the local government in lieu of detention. That fee would go toward alleviating downstream flooding or making channel or other conveyance improvements.
7. If peak discharges are increased due to development, or if downstream velocities are erosive, one of the following options are required.
   - Document that existing downstream conveyance is adequate to convey post-developed storm water discharges (Option 1 for Streambank Protection and Flood Control). (See guidelines in section 1.2.2.)
   - Work with the Town to reduce the flow elevation and/or velocity through channel or flow conveyance structure improvements downstream. (Option 2 for Streambank Protection and Flood Control)
   - Design an on-site structural control facility such that the post-development flows do not increase the peak flows, and the velocities are not erosive, at the outlet and the determined junction locations.

2.1.8.3 Example Problem

Figure 2.1.8-3 illustrates the concept of the ten-percent rule for two sites in a watershed.

Discussion
Site A is a development of 10 acres, all draining to a wet Extended Detention (ED) storm water pond.*The flood portions of the design are going to incorporate the ten-percent rule. Looking downstream at each tributary in turn, it is determined that the analysis should end at the tributary marked "80 acres." The 100-acre (10%) point is in between the 80-acre and 120-acre tributary junction points.

The assumption is that if there is no peak flow increase or erosive velocities at the 80-acre point then the same will be true through the next stream reach downstream through the 10% point (100 acres) to the 120-acre point. The designer constructs a simple HEC-1 model of the 80-acre areas using single, "full build-out" condition sub-watersheds for each tributary. Key detention structures existing in other tributaries must be modeled. An approximate curve number is used since the *actual* peak flow is not key for initial analysis; only the increase or decrease is important. The accuracy in curve number determination is not as significant as an accurate estimate of the time of concentration. Since flooding is an issue downstream, the pond is designed (through several iterations) until the peak flow does not increase, and velocities are not erosive, at junction points downstream to the 80-acre point. Site B is located downstream at the point where the total drainage area is 190 acres. The site itself is only 6 acres. The first tributary junction downstream from the 10% point is the junction of the site outlet with the stream. The total 190 acres is modeled as one basin with care taken to estimate the time of concentration for input into the TR-20 model of the watershed. The model shows a detention facility, in this case, will actually <u>increase</u> the peak flow in the stream. Please see section 1.2.2 for additional guidelines to be considered in downstream hydrologic and hydraulic assessments.

***Extended Detention is not currently required by the Town of Copper Canyon**

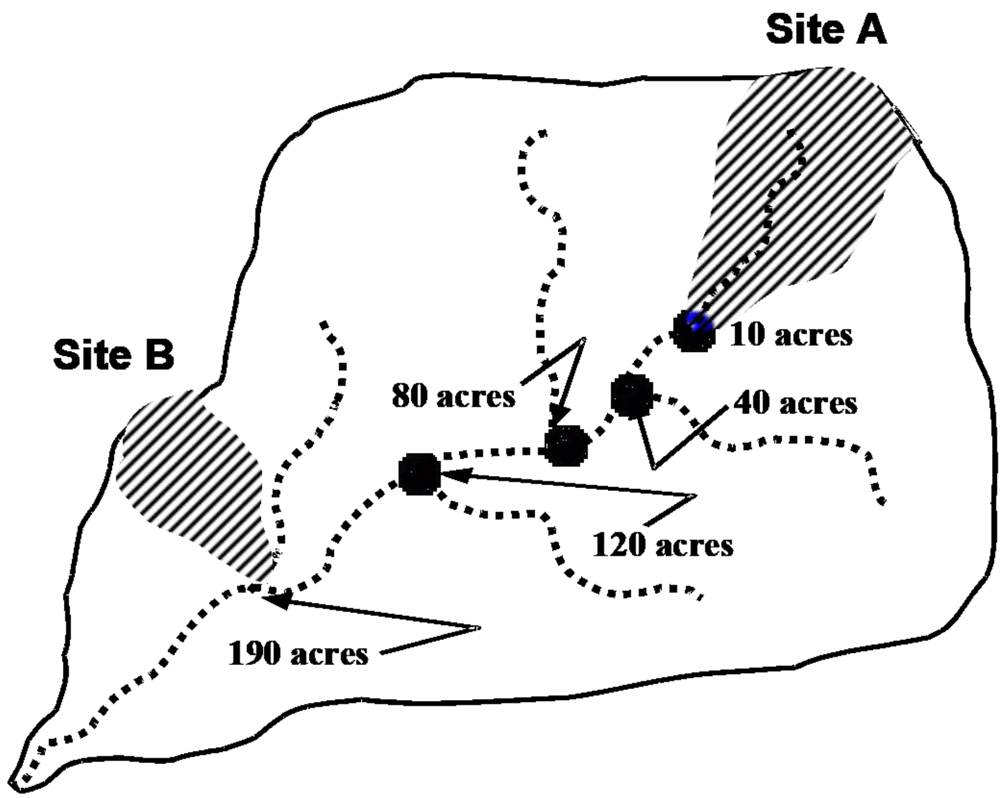


Figure 2.1.8-3 Example of the Ten-Percent Rule

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CHAPTER 3 – HYDRAULIC DESIGN OF STREETS AND CLOSED CONDUITS

Section 3.1 – Storm Water Street and Closed Conduit Design Overview

3.1.1 – Storm Water System Design

3.1.1.1 Introduction

Storm water system design is an integral component of both site and overall storm water management design. Good drainage design must strive to maintain compatibility and minimize interference with existing drainage patterns; control flooding of property, structures, and roadways for design flood events; and minimize potential environmental impacts on storm water runoff.

3.1.1.2 System Components

The on-site flood control systems are designed to remove storm water from areas such as streets and sidewalks for public safety reasons. The drainage system can consists of inlets, street and roadway gutters, roadside ditches, small channels and swales, storm water ponds and wetlands, and small underground pipe systems which collect storm water runoff from mid-frequency storms and transport it to structural control facilities, pervious areas, and/or the larger storm water systems (i.e., natural waterways, large man-made conduits, and large water impoundments).

The storm water (major) system consists of natural waterways, large man-made conduits, and large water impoundments. In addition, the major system includes some less obvious drainageways such as overload relief swales and infrequent temporary ponding areas. The storm water system includes not only the trunk line system that receives the water, but also the natural overland relief which functions in case of overflow from or failure of the on-site flood control system. Overland relief must not flood or damage houses, buildings or other property.

This chapter is intended to provide design criteria and guidance on several on-site flood control system components, including street and roadway gutters, inlets, and storm drain pipe systems (Section 3.2). Chapter 4 covers the design of culverts (Section 4.2); vegetated and lined open channels (Section 4.4); storage and design (Section 4.5); outlet structures (Section 4.6); and energy dissipation devices for outlet protection (Section 4.7). The rest of this section covers important considerations to keep in mind in the planning and design of storm water drainage facilities

3.1.1.3 Checklist for Planning and Design

The following is a general procedure for drainage system design on a development site.

A. Analyze topography, including:
   1. Check off-site drainage pattern. Where is water coming onto the site? Where is water leaving the site?
   2. Check on-site topography for surface runoff and storage, and infiltration
      a. Determine runoff pattern: high points, ridges, valleys, streams, and swales. Where is the water going?
      b. Overlay the grading plan and indicate watershed areas: calculate square footage (acreage), points of concentration, low points, etc.

B. Analyze other site conditions, including:
   1. Land use and physical obstructions such as walks, drives, parking, patios, landscape edging, fencing, grassed area, landscaped area, tree roots, etc.
   2. Soil type (infiltration rates).
   3. Vegetative cover (slope protection).
C. Check potential drainage outlets and methods, including:
   1. On-site (structural control, receiving water)
   2. Off-site (highway, storm drain, receiving water, regional control)
   3. Natural drainage system (swales)
   4. Existing drainage system (drain pipe)
D. Analyze areas for probable location of drainage structures and facilities.
E. Identify the type and size of drainage system components required. Design the drainage system and integrate with the overall storm water management system and plan.

3.1.2 – Key Issues in Storm Water System Design

3.1.2.1 Introduction

The traditional design of storm water systems has been to collect and convey storm water runoff as rapidly as possible to a suitable location where it can be discharged. This manual takes a different approach wherein the design methodologies and concepts of drainage design are to be integrated with the objectives for water quantity and quality control. This means:

- Storm water systems are to remove water efficiently enough to meet flood protection criteria and level of service requirements, and
- These systems are to complement the ability of the site design and structural storm water controls to mitigate the major storm water impacts of urban development.

The following are some of the key issues in integrating water quantity and quality control consideration in storm water system design.

3.1.2.2 General Design Considerations

- Storm water systems should be planned and designed so as to generally conform to natural drainage patterns and discharge to natural drainage pathways within a drainage basin. These natural drainage pathways should only be modified as a last resort to contain and safely convey the peak flows generated by the development.
- Runoff must be discharged in a manner that will not cause adverse impacts on downstream properties or storm water systems. In general, runoff from development sites within a drainage basin should be discharged at the existing natural drainage outlet or outlets. If the developer wishes to change discharge points he or she must demonstrate that the change will not have any adverse impacts on downstream properties or storm water (minor) systems.
- It is important to ensure that the combined on-site flood control system and major storm water system can handle blockages and flows in excess of the design capacity to minimize the likelihood of nuisance flooding or damage to private properties. If failure of minor storm water systems and/or major storm water structures occurs during these periods, the risk to life and property could be significantly increased.

- In establishing the layout of storm water systems, it is essential to ensure that flows are not diverted onto private property for flows equal or less than the major storm water system design capacity.

3.1.2.3 Street and Roadway Gutters

- Gutters are efficient flow conveyance structures. This is not always an advantage if removal of pollutants and reduction of runoff is an objective. Therefore, impervious surfaces should be disconnected hydrologically where possible, and runoff should be allowed to flow across pervious surfaces or through vegetated channels. Gutters should be used only after other options have been investigated and only after runoff has had as much chance as possible to infiltrate and filter through vegetated areas.
- It may be possible not to use gutters at all, or to modify them to channel runoff to off-road pervious areas or open channels. For example, curb opening type designs take roadway runoff to smaller feeder grass channels. Care should be taken not to create erosion problems in off-road areas. Protection during construction, establishment of strong stands of vegetation, and active maintenance may be necessary in some areas.
- Use typical road sections that use grass channels or swales instead of gutters to provide for pollution reduction and reduce the impervious area required. Figure 3.1-1 illustrates a roadway cross section that eliminates gutters for residential neighborhoods. Flow can also be directed to center median strips in divided roadway designs. To protect the edge of pavement, ribbons of concrete can be used along the outer edges of asphalt roads.

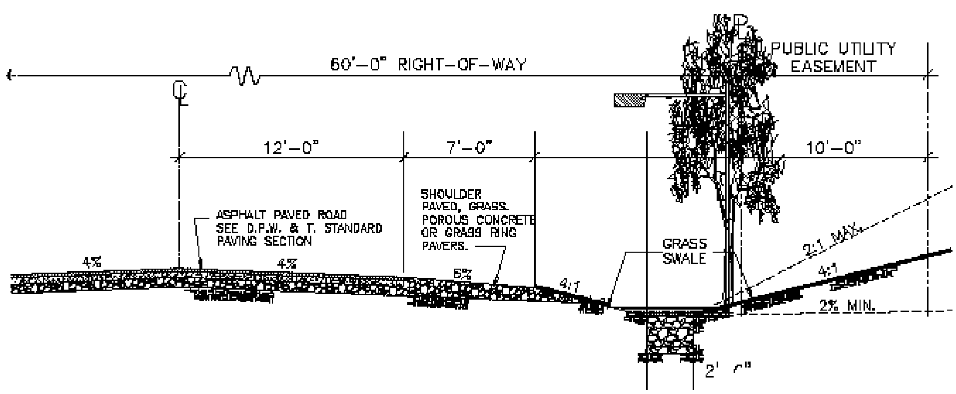


Figure 3.1-1 Alternate Roadway Section without Gutters

3.1.2.4 Inlets and Drains

- Inlets should be located to maximize the overland flow path, take advantage of pervious areas, and seek to maximize vegetative filtering and infiltration. For example, it might be possible to design a parking lot so water flows into vegetated areas prior to entering the nearest inlet.
- Inlet location should not compromise safety or aesthetics. It should not allow for standing water in areas of vehicular or pedestrian traffic, but should take advantage of natural depression storage where possible.
- Inlets should be located to serve as overflows for structural storm water controls.
- The choice of inlet type should match its intended use. A sumped inlet may be more effective supporting water quality objectives.

Use several smaller inlets instead of one large inlet in order to:

- Prevent erosion on steep landscapes by intercepting water before it accumulates too much volume and velocity.
- Provide a safety factor. If a drain inlet clogs, the other surface drains may pick up the water.
- Improve aesthetics. Several smaller drains will be less obvious than one large drain.

Spacing smaller drain inlets will give surface runoff a better chance of reaching the drain. Water will have to travel farther to reach one large drain inlet.

3.1.2.5 Closed Conduit Systems (Storm Drains/Sewers)

- The use of storm water design practices should be considered to reduce the overall length of a closed conduit storm water system.
- Shorter and smaller conveyances can be designed to carry runoff to nearby holding areas or natural conservation areas.
- Ensure that storms in excess of closed conduit design flows can be safely conveyed through a development without damaging structures or flooding major roadways. This is often done through design of both a major and minor drainage system. The on-site flood control system carries the mid-frequency design flows while larger runoff events may flow across lots and along streets as long as it will not cause property damage or impact public safety.

3.1.3 – Design Storm Recommendations

Storm Sewer System

- The design storm is a minimum 25-year for the closed conduit systems in residential and commercial areas and for thoroughfares. The 100-year storm is the design storm for the combination of the closed conduit and surface drainage system.
- Runoff from the design closed conduit storm must be contained within the permissible spread of water in the gutter. The 100-year storm flow must be contained within the ROW. Adequate inlet capacity shall be provided to intercept surface flows before the street ROW capacity is exceeded. Note: The capacity of the underground system may be required to exceed the 25-year design closed conduit storm in order to satisfy the 100-year storm criteria.
- Enclosed drainage systems for all street types shall be designed to contain the 25-year storm. The 25-year flow must not exceed curb depth. 100-year flows shall be contained within drainage easement and/or ROW. Safe overflow routing with supporting calculations shall be provided and indicated on plans. Grading plans must accommodate the necessary capacities to contain the 100-year flow within the street right-of-way or drainage easements.
- The closed conduit HGL must be equal to or below the gutter line for pipe systems and one (1) foot or more below top of curb at inlets. For situations where no ROW exists, the 100 year HGL must be below finished ground. The 100-year HGL will be tracked carefully throughout the system and described in the hydraulic calculation tables (Figures 3.2-26 and 3.2-27) in the construction drawings.

Sump Inlets

In sag or sump conditions, the storm drain and sump inlets should be sized to intercept and convey the 25-year storm, provided that a positive overflow is provided for the remainder of the 100-year storm. When the overflow route is between residential lots or otherwise constricted, the positive overflow structure must be concrete or other acceptable non-earthen structure with a minimum bottom width of 6 feet extending from the sump inlet to the storm sewer outfall. If the upstream pipe already conveys more than 25-year peak discharge, then the downstream pipe must have at least the same capacity from sump to outfall, and an inlet must still be installed at sump to allow for emergency overflow. In the event that a structural overflow is not practical, then the underground system must be sized to convey the 100-year storm.

Section 3.2 – On-Site Flood Control System Design

3.2.1 – Overview

On-Site Flood Control Systems, also known as minor drainage systems, quickly remove runoff from areas such as streets and sidewalks for public safety purposes. The on-site flood control system consists of inlets, street and roadway gutters, roadside ditches, small channels and swales, and small underground pipe systems which collect storm water runoff and transport it to structural control facilities, pervious areas, and/or the larger storm water system (i.e., natural waterways, large man-made conduits, and large water impoundments).

This section is intended to provide criteria and guidance for the design of on-site flood control system components including:

- Street and roadway gutters
- Storm water inlets
- Storm drain pipe systems

Ditch, channel and swale design criteria and guidance are covered in Section 4.4, *Open Channel Design*.

Procedures for performing gutter flow calculations are based on a modification of Manning's Equation. Inlet capacity calculations for grate, curb, and combination inlets are based on information contained in HEC-12 (USDOT, FHWA, 1984). Storm drain system design is based on the use of the Rational Method Formula.

3.2.1.2 General Criteria

The requirement for the Town of Copper Canyon’s typical street sections are presented in Table 3.1-1

Table 3.1-1 Typical Street Sections and Storm Sewer Criteria

Street Type	Min. Back to Back Width (ft)	Section Type	Closed Conduit Design Storm	Inlet Type		Flow Spread Limits (ft)
				Recessed or Non-Recessed	Depressed or Non-Depressed	
Residential UrbanStreet	31	Rooftop	25 yr	Either	Either	Top of Curb or Roadway Centerline
Urban Collector	37	Rooftop	25 yr	Either	Either	One 12’ Lane Clear
Arterial	48	Rooftop	25 yr	Either	Either	One 12’ Lane Clear (each side)
Residential Boulevard	20/20*	Rooftop	25 yr	Either	Either	One Lane Clear (each side)
Collector Boulevard	25/25*	Rooftop	25 yr	Either	Either	One Lane Clear (each side)

* Each Side

Must use roadway sections as approved by Town of Copper Canyon. See "Standard Construction Details" (Chapter 24, Section 24-80 of Municipal Ordinances) for drawings of these sections.

3.2.2 – Symbols and Definitions

To provide consistency within this section as well as throughout this Manual, the symbols listed in Table 3.2-1 will be used. These symbols were selected because of their wide use. In some cases, the same symbol is used in existing publications for more than one definition. Where this occurs in this section, the symbol will be defined where it occurs in the text or equations.

Table 3.2-1 Symbols and Definitions

Symbol	Definition	Units
a	Gutter depression	in
A	Area of cross section	ft^2
d or D	Depth of gutter flow at the curb line	ft
D	Diameter of pipe	ft
E_o	Ratio of frontal flow to total gutter flow Q_w/Q	-
g	Acceleration due to gravity (32.2 ft/s^2)	ft/s^2
h	Height of curb opening inlet	ft
H	Head loss	ft
K	Loss coefficient	-
L or L_T	Length of curb opening inlet	ft
L	Pipe length	ft
n	Roughness coefficient in the modified Manning's formula for triangular gutter flow	-
P	Perimeter of grate opening, neglecting bars and side against curb	ft
Q	Rate of discharge in gutter	cfs
Q_i	Intercepted flow	cfs
Q_S	Gutter capacity above the depressed section	cfs
S or S_x	Cross Slope - Traverse slope	ft/ft
S or S_L	Longitudinal slope	ft/ft
S_f	Friction slope	ft/ft
S'_w	Depression section slope	ft/ft
T	Top width of water surface (spread on pavement)	ft
T_s	Spread above depressed section	ft
V	Velocity of flow	ft/s
W	Width of depression for curb opening inlets	ft
Z	T/d, reciprocal of the cross slope	-

3.2.3 – Street and Roadway Gutters

Effective drainage of street and roadway pavements is essential to the maintenance of the roadway service level and to traffic safety. Water on the pavement can interrupt traffic flow, reduce skid resistance, increase potential for hydroplaning, limit visibility due to splash and spray, and cause difficulty in steering a vehicle when the front wheels encounter puddles. Surface drainage is a function of transverse and longitudinal pavement slope, pavement roughness, inlet spacing, and inlet capacity. The design of these elements is dependent on storm frequency and the allowable spread of storm water on the pavement surface.

This section presents design guidance for gutter flow hydraulics originally published in HEC-12, Drainage of Highway Pavements and AASHTO's Model Drainage Manual.

3.2.3.1 Formula

The following form of Manning's Equation should be used to evaluate gutter flow hydraulics:

$$Q = [0.56/n]\, S_x^{5/3}\, S^{1/2}\, T^{8/3} \qquad \textbf{(3.2.1)}$$

where:

- Q = gutter flow rate, cfs
- S_x = pavement cross slope, ft/ft
- n = Manning's roughness coefficient
- S = longitudinal slope, ft/ft
- T = width of flow or spread, ft

Equation 3.2.1 was first presented by C.F. Izzard in 1946.

3.2.3.2 Nomograph

Figure 3.2-1 is a nomograph for solving Equation 3.2.1. Manning's n values for various pavement surfaces are presented in Table 3.2-2 below. Note: the nomograph will not work on slopes steeper than 3% - 4% for 2 and 3 lane thoroughfares. Also, the “Q” in the nomograph is only for n = 0.016.

3.2.3.3 Manning's n Table

Table 3.2-2 Manning's n Values for Street and Pavement Gutters

Type of Gutter or Pavement	Manning's n
Concrete gutter, troweled finish	0.014
Asphalt pavement:	
Smooth texture	0.015
Rough texture	0.019
Concrete gutter with asphalt pavement:	
Smooth	0.015
Rough	0.018
Concrete pavement:	
Float finish	0.017
Broom finish	0.019
For gutters with small slopes, where sediment may accumulate, increase above values of n by	0.002

Note: Based on the statement of Izzard (1946) and confirmation by model studies (Ickert and Crosby, 2003), the n-values given in Table 4-3 of HEC No. 22, 2001, were increased by 18% to derive the n-values in this table.

3.2.3.4 Uniform Cross Slope

The nomograph in Figure 3.2-1 is used with the following procedures to find gutter capacity for uniform cross slopes:

Condition 1: Find spread, given gutter flow.

Step 1 Determine input parameters, including longitudinal slope (S), cross slope (S_x), gutter flow (Q), and Manning's n.

Step 2 Draw a line between the S and S_x scales and note where it intersects the turning line.

Step 3 Draw a line between the intersection point from Step 2 and the appropriate gutter flow value on the capacity scale. If Manning's n is 0.016, use Q from Step 1; if not; use the product of Q and n (Qn).

Step 4 Read the value of the spread (T) at the intersection of the line from Step 3 and the spread scale.

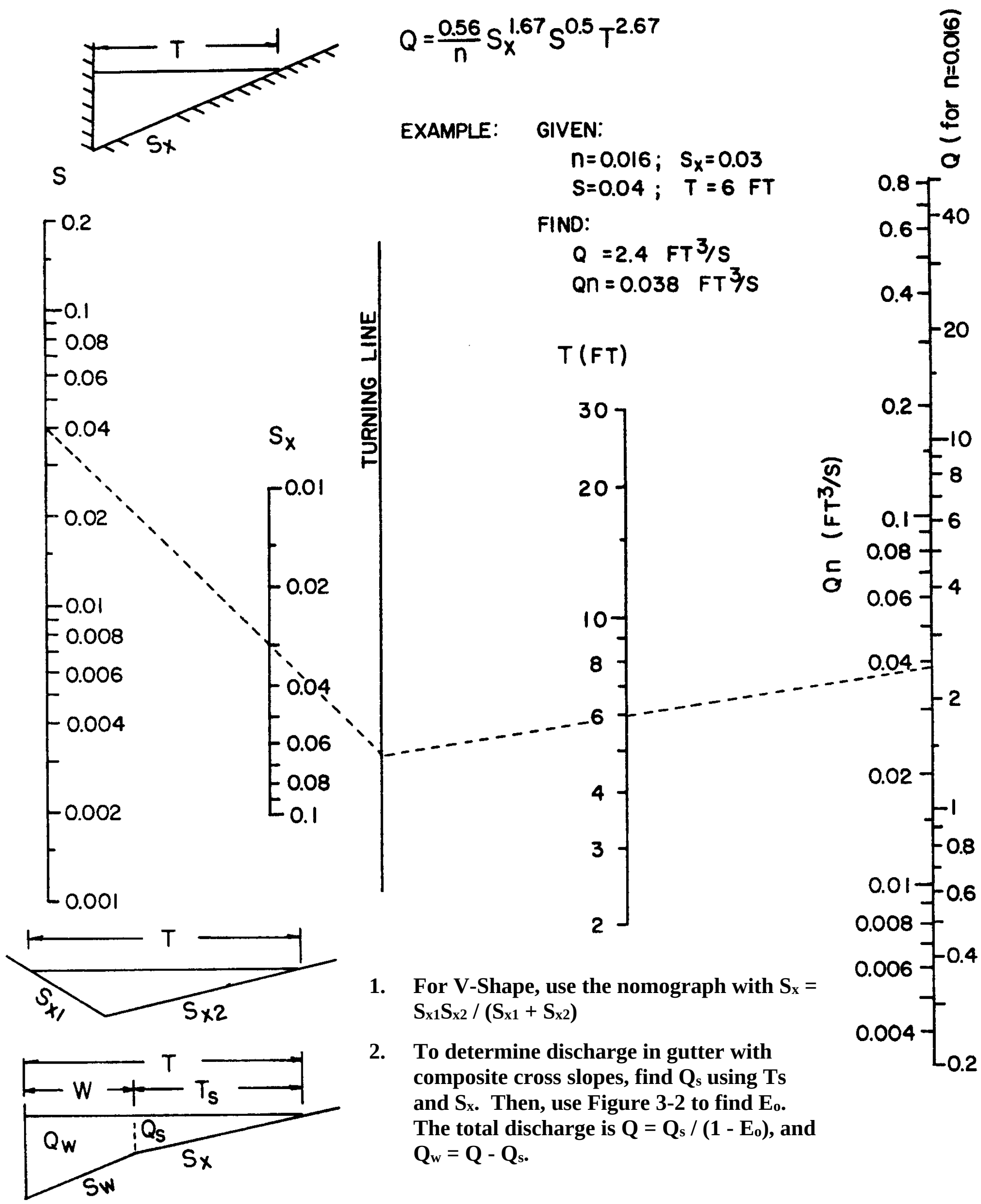
Condition 2: Find gutter flow, given spread.

Step 1 Determine input parameters, including longitudinal slope (S), cross slope (S_x), spread (T), and Manning's n.

Step 2 Draw a line between the S and S_x scales and note where it intersects the turning line.

Step 3 Draw a line between the intersection point from Step 2 and the appropriate value on the T scale. Read the value of Q or Qn from the intersection of that line on the capacity scale.

Step 4 For Manning's n values of 0.016, the gutter capacity (Q) from Step 3 is selected. For other Manning's n values, the gutter capacity times n (Qn) is selected from Step 3 and divided by the appropriate n value to give the gutter capacity.



1. **For V-Shape, use the nomograph with $S_x = S_{x1}S_{x2} / (S_{x1} + S_{x2})$**
2. **To determine discharge in gutter with composite cross slopes, find Q_s using Ts and S_x. Then, use Figure 3-2 to find E_o. The total discharge is $Q = Q_s / (1 - E_o)$, and $Q_w = Q - Q_s$.**

Figure 3.2-1 Flow in Triangular Gutter Sections
(Source: AASHTO Model Drainage Manual, 1991)

***SEE SECTION 3.2.3.2 FOR APPLICABILITY**

3.2.3.5 Composite Gutter Sections

Figure 3.2-2 in combination with Figure 3.2-1 can be used to find the flow in a gutter with width (W) less than the total spread (T). Such calculations are generally used for evaluating composite gutter sections or frontal flow for grate inlets. Please note that the nomograph in Figure 3.2-1 does not completely address cases where the crown elevation is lower that the top of curb elevation. For those cases a combination of Figure 3.2-1 and 3.2-2 can be used or a standard hydraulics program such as HEC-RAS or FlowMaster can be applied.

Figure 3.2-3 provides a direct solution of gutter flow in a composite gutter section. The flow rate at a given spread or the spread at a known flow rate can be found from this figure. Figure 3.2-3 involves a complex graphical solution of the equation for flow in a composite gutter section. Typical of graphical solutions, extreme care in using the figure is necessary to obtain accurate results.

Condition 1: Find spread, given gutter flow.

Step 1 Determine input parameters, including longitudinal slope (S), cross slope (S_x), depressed section slope (S_w), depressed section width (W), Manning's n, gutter flow (Q), and a trial value of gutter capacity above the depressed section (Q_s).

Step 2 Calculate the gutter flow in W (Q_w), using the equation:

$$\mathbf{Q_w = Q - Q_s} \qquad \mathbf{(3.2.2)}$$

Step 3 Calculate the ratios Q_w/Q or E_o and S_w/S_x and use Figure 3.2-2 to find an appropriate value of W/T.

Step 4 Calculate the spread (T) by dividing the depressed section width (W) by the value of W/T from Step 3.

Step 5 Find the spread above the depressed section (T_s) by subtracting W from the value of T obtained in Step 4.

Step 6 Use the value of T_s from Step 5 along with Manning's n, S, and S_x to find the actual value of Q_s from Figure 3.2-1.

Step 7 Compare the value of Q_s from Step 6 to the trial value from Step 1. If values are not comparable, select a new value of Q_s and return to Step 1.

Condition 2: Find gutter flow, given spread.

Step 1 Determine input parameters, including spread (T), spread above the depressed section (T_s), cross slope (S_x), longitudinal slope (S), depressed section slope (S_w), depressed section width (W), Manning's n, and depth of gutter flow (d).

Step 2 Use Figure 3.2-1 to determine the capacity of the gutter section above the depressed section (Q_s). Use the procedure for uniform cross slopes, substituting T_s for T.

Step 3 Calculate the ratios W/T and S_w/S_x, and, from Figure 3.2-2, find the appropriate value of E_o (the ratio of Q_w/Q).

Step 4 Calculate the total gutter flow using the equation:

$$\mathbf{Q = Q_s / (1 - E_o)} \qquad \mathbf{(3.2.3)}$$

where:

Q = gutter flow rate, cfs

Q_s = flow capacity of the gutter section above the depressed section, cfs

E_o = ratio of frontal flow to total gutter flow (Q_w/Q)

Step 5 Calculate the gutter flow in width (W), using Equation 3.2.2.

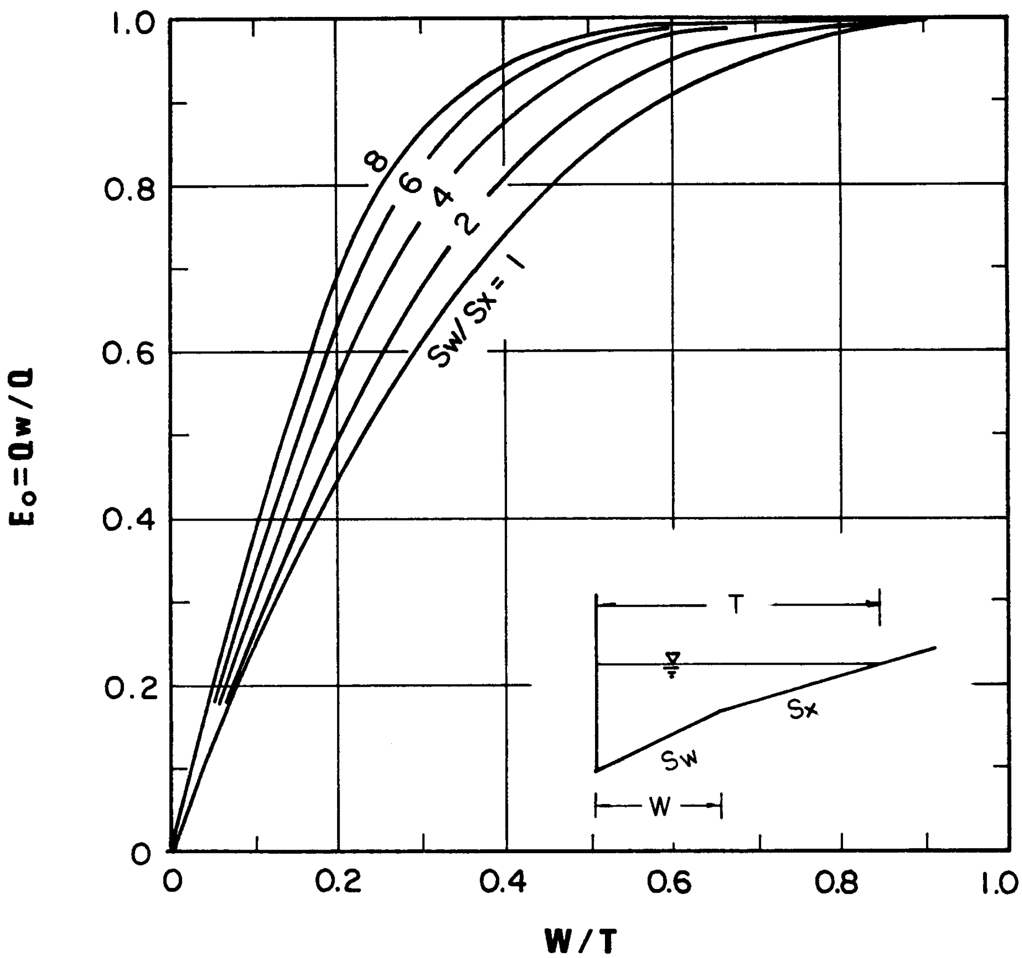


Figure 3.2-2 Ratio of Frontal Flow to Total Gutter Flow
(Source: AASHTO Model Drainage Manual, 1991)

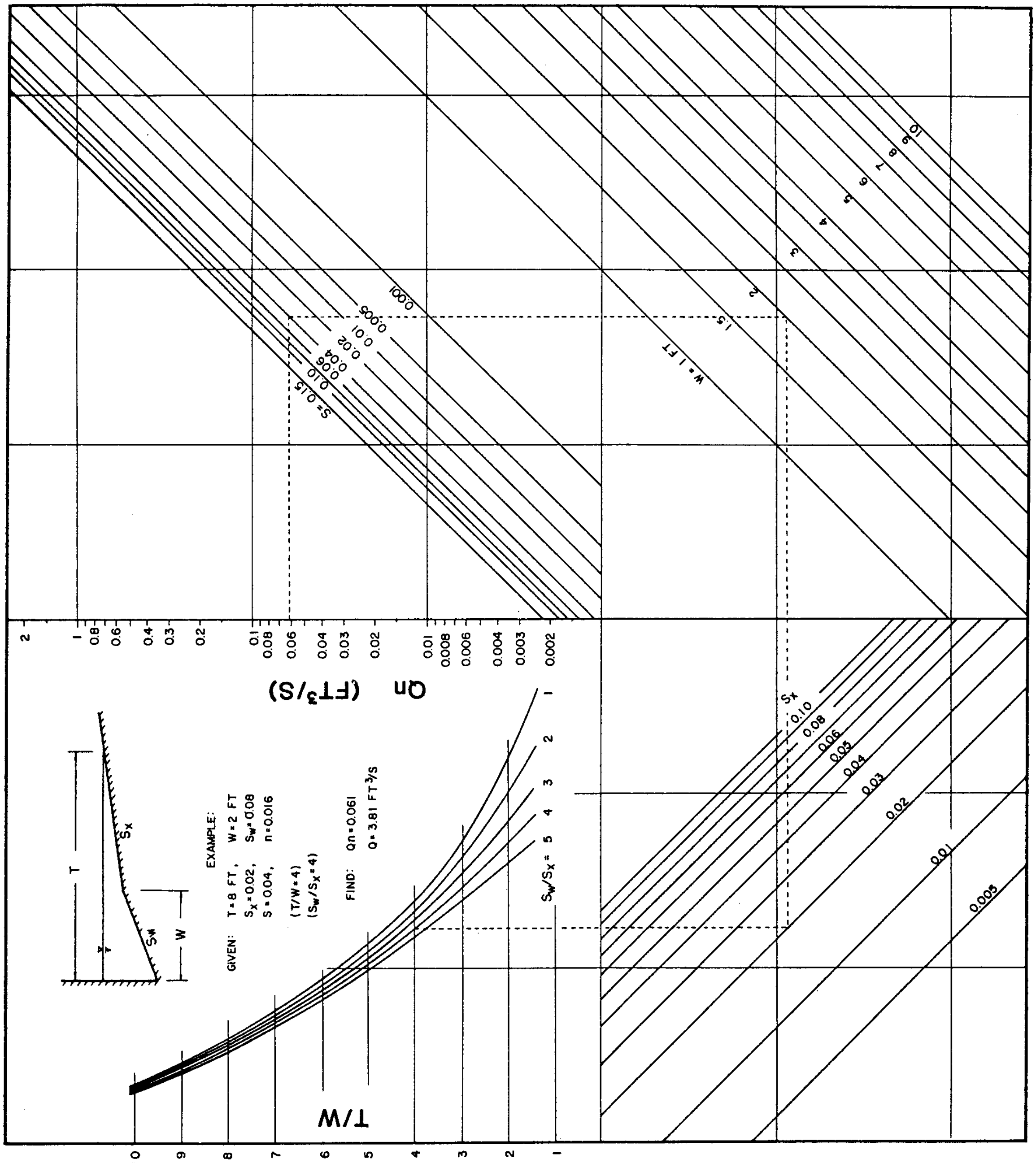


Figure 3.2-3 Flow in Composite Gutter Sections
(Source: AASHTO Model Drainage Manual, 1991)

3.2.3.6 Examples

<u>Example 1</u>

Given:

T = 8 ft
S_x = 0.025 ft/ft
n = 0.015
S = 0.01 ft/ft

Find:

1. Flow in gutter at design spread
2. Flow in width (W = 2 ft) adjacent to the curb

Solution:

a. From Figure 3.2-1, $Qn = 0.03$

$Q = Qn/n = 0.03/0.015 = 2.0$ cfs

b. $T = 8 - 2 = 6$ ft

$(Qn)_2 = 0.014$ (Figure 3.2-1) (flow in 6-foot width outside of width (W))

$Q = 0.014/0.015 = 0.9$ cfs

$Q_w = 2.0 - 0.9 = 1.1$ cfs

Flow in the first 2 ft adjacent to the curb is 1.1 cfs and 0.9 cfs in the remainder of the gutter.

<u>Example 2</u>

Given:

T = 6 ft
S_w = 0.0833 ft/ft
T_s = 6 - 1.5 = 4.5 ft
W = 1.5 ft
S_x = 0.03 ft/ft
n = 0.014
S = 0.04 ft/ft

Find:

Flow in the composite gutter

Solution:

1. Use Figure 3.2-1 to find the gutter section capacity above the depressed section.

$Q_s n = 0.038$

$Q_s = 0.038/0.014 = 2.7$ cfs

2. Calculate W/T = 1.5/6 = 0.25 and

   S_w/S_x = 0.0833/0.03 = 2.78

   Use Figure 3.2-2 to find E_o = 0.64

3. Calculate the gutter flow using Equation 3.2.3

   Q = 2.7/(1 - 0.64) = 7.5 cfs

4. Calculate the gutter flow in width, W, using Equation 3.2.2

   Q_W = 7.5 - 2.7 = 4.8 cfs

3.2.4 – Storm Water Inlets

Inlets are drainage structures used to collect surface water through grate or curb openings and convey it to storm drains or direct outlet to culverts. Grate inlets subject to traffic should be bicycle safe and be load-bearing adequate. Appropriate frames should be provided.

Inlets used for the drainage of highway surfaces can be divided into three major classes:

- Grate Inlets – These inlets include grate inlets consisting of an opening in the gutter covered by one or more grates, and slotted inlets consisting of a pipe cut along the longitudinal axis with a grate or spacer bars to form slot openings.
- Curb-Opening Inlets – These inlets are vertical openings in the curb covered by a top slab.
- Combination Inlets – These inlets usually consist of both a curb-opening inlet and a grate inlet placed in a side-by-side configuration, but the curb opening may be located in part upstream of the grate.

Inlets may be classified as being on a continuous grade or in a sump. The term "continuous grade" refers to an inlet located on the street with a continuous slope past the inlet with water entering from one direction. The "sump" condition exists when the inlet is located at a low point and water enters from both directions. Sump areas should have an overflow route or channel.

Where significant ponding can occur, in locations such as underpasses and in sag vertical curves in depressed sections, it is good engineering practice to place flanking inlets on each side of the inlet at the low point in the sag. The flanking inlets should be placed so they will limit spread on low gradient approaches to the level point and act in relief of the inlet at the low point if it should become clogged or if the design spread is exceeded.

The design of grate inlets will be discussed in subsection 3.2.5, curb inlet design in Section 3.2.6, and combination inlets in Section 3.2.7.

3.2.5 – Grate Inlet Design

3.2.5.1 Grate Inlets on Grade

The capacity of an inlet depends upon its geometry and the cross slope, longitudinal slope, total gutter flow, depth of flow, and pavement roughness. The depth of water next to the curb is the major factor in the interception capacity of both gutter inlets and curb opening inlets. At low velocities, all of the water flowing in the section of gutter occupied by the grate, called frontal flow, is intercepted by grate inlets, and a small portion of the flow along the length of the grate, termed side flow, is intercepted. On steep slopes, only a portion of the frontal flow will be intercepted if the velocity is high or the grate is short and splash-over occurs. For grates less than 2 feet long, intercepted flow is small.

A parallel bar grate is the most efficient type of gutter inlet; however, when crossbars are added for bicycle safety, the efficiency is greatly reduced. Where bicycle traffic is a design consideration, the curved vane grate and the tilt bar grate are recommended for both their hydraulic capacity and bicycle safety features. They also handle debris better than other grate inlets but the vanes of the grate must be turned in the proper direction. Where debris is a problem, consideration should be given to debris handling efficiency rankings of grate inlets from laboratory tests in which an attempt was made to qualitatively simulate field conditions. Table 3.2-6 presents the results of debris handling efficiencies of several grates. Debris handling efficiencies were based on the total number of simulated leaves arriving at the grate and the number passed.

The ratio of frontal flow to total gutter flow, E_o, for straight cross slope is expressed by the following equation:

$$E_o = Q_w/Q = 1 - (1 - W/T)^{2.67} \qquad (3.2.6)$$

where:

Q = total gutter flow, cfs

Q_W = flow in width W, cfs

W = width of depressed gutter or grate, ft

T = total spread of water in the gutter, ft

Table 3.2-6 Grate Debris Handling Efficiencies

Rank	Grate	Longitudinal Slope	
		(0.005)	(0.04)
1	CV - 3-1/4 - 4-1/4	46	61
2	30 - 3-1/4 - 4	44	55
3	45 - 3-1/4 - 4	43	48
4	P - 1-7/8	32	32
5	P - 1-7/8 - 4	18	28
6	45 - 2-1/4 - 4	16	23
7	Reticuline	12	16
8	P - 1-1/8	9	20

Source: "Drainage of Highway Pavements" (HEC-12), Federal Highway Administration, 1984.

Figure 3.2-2 provides a graphical solution of E_o for either depressed gutter sections or straight cross slopes. The ratio of side flow, Q_s, to total gutter flow is:

$$Q_s/Q = 1 - Q_w/Q = 1 - E_o \qquad (3.2.7)$$

The ratio of frontal flow intercepted to total frontal flow, R_f, is expressed by the following equation:

$$R_f = 1 - 0.09\,(V - V_0) \qquad (3.2.8)$$

where:

V = velocity of flow in the gutter, ft/s (using Q from Figure 3.2-1)

V_o = gutter velocity where splash-over first occurs, ft/s (from Figure 3.2-4)

This ratio is equivalent to frontal flow interception efficiency. Figure 3.2-4 provides a solution of equation 3.2.8, which takes into account grate length, bar configuration and gutter velocity at which splash-over occurs. The gutter velocity needed to use Figure 3.2-4 is total gutter flow divided by the area of flow. The ratio of side flow intercepted to total side flow, R_s, or side flow interception efficiency, is expressed by:

$$\mathbf{R_s = 1 / [1 + (0.15V^{1.8}/S_xL^{2.3})]} \qquad \textbf{(3.2.9)}$$

where:

L = length of the grate, ft

Figure 3.2-5 provides a solution to equation 3.2.9.

The efficiency, E, of a grate is expressed as:

$$\mathbf{E = R_fE_o + R_s(1 - E_o)} \qquad \textbf{(3.2.10)}$$

The interception capacity of a grate inlet on grade is equal to the efficiency of the grate multiplied by the total gutter flow:

$$\mathbf{Q_i = EQ = Q[R_fE_o + R_s(1 - E_o)]} \qquad \textbf{(3.2.11)}$$

The following example illustrates the use of this procedure.

Given:

W = 2 ft

T = 8 ft

S_x = 0.025 ft/ft

S = 0.01 ft/ft

E_o = 0.69

Q = 3.0 cfs

V = 3.1 ft/s

Gutter depression = 2 in

Find:

Interception capacity of:

1. a curved vane grate, and
2. a reticuline grate 2-ft long and 2-ft wide

Solution:

From Figure 3.2-4 for Curved Vane Grate, R_f = 1.0

From Figure 3.2-4 for Reticuline Grate, R_f = 1.0

From Figure 3.2-5 R_s = 0.1 for both grates

From Equation 3.2.11:

Q_i = 3.0[1.0 X 0.69 + 0.1(1 - 0.69)] = 2.2 cfs

For this example, the interception capacity of a curved vane grate is the same as that for a reticuline grate for the sited conditions.

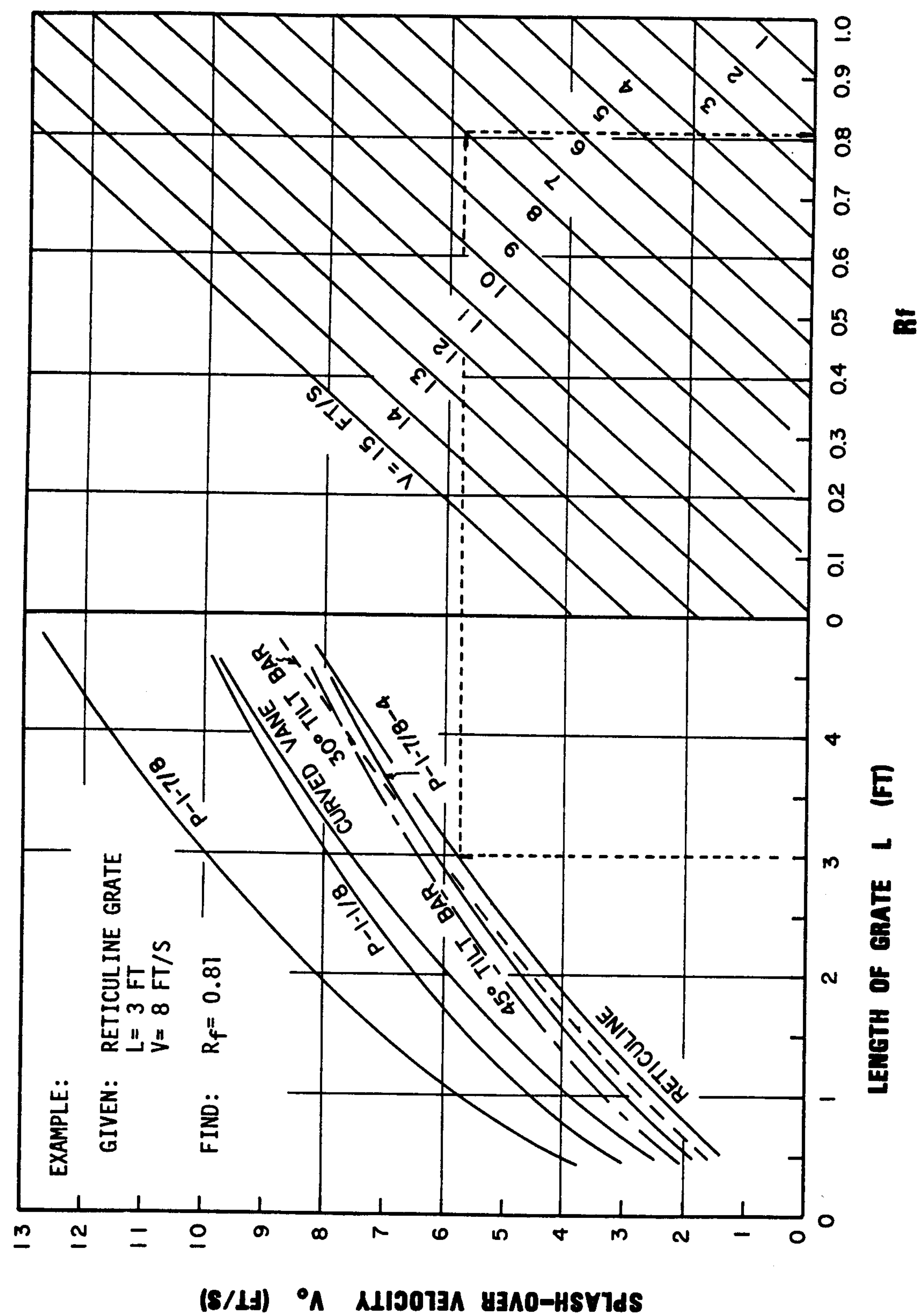


Figure 3.2-4 Grate Inlet Frontal Flow Interception Efficiency
(Source: HEC-12, 1984)

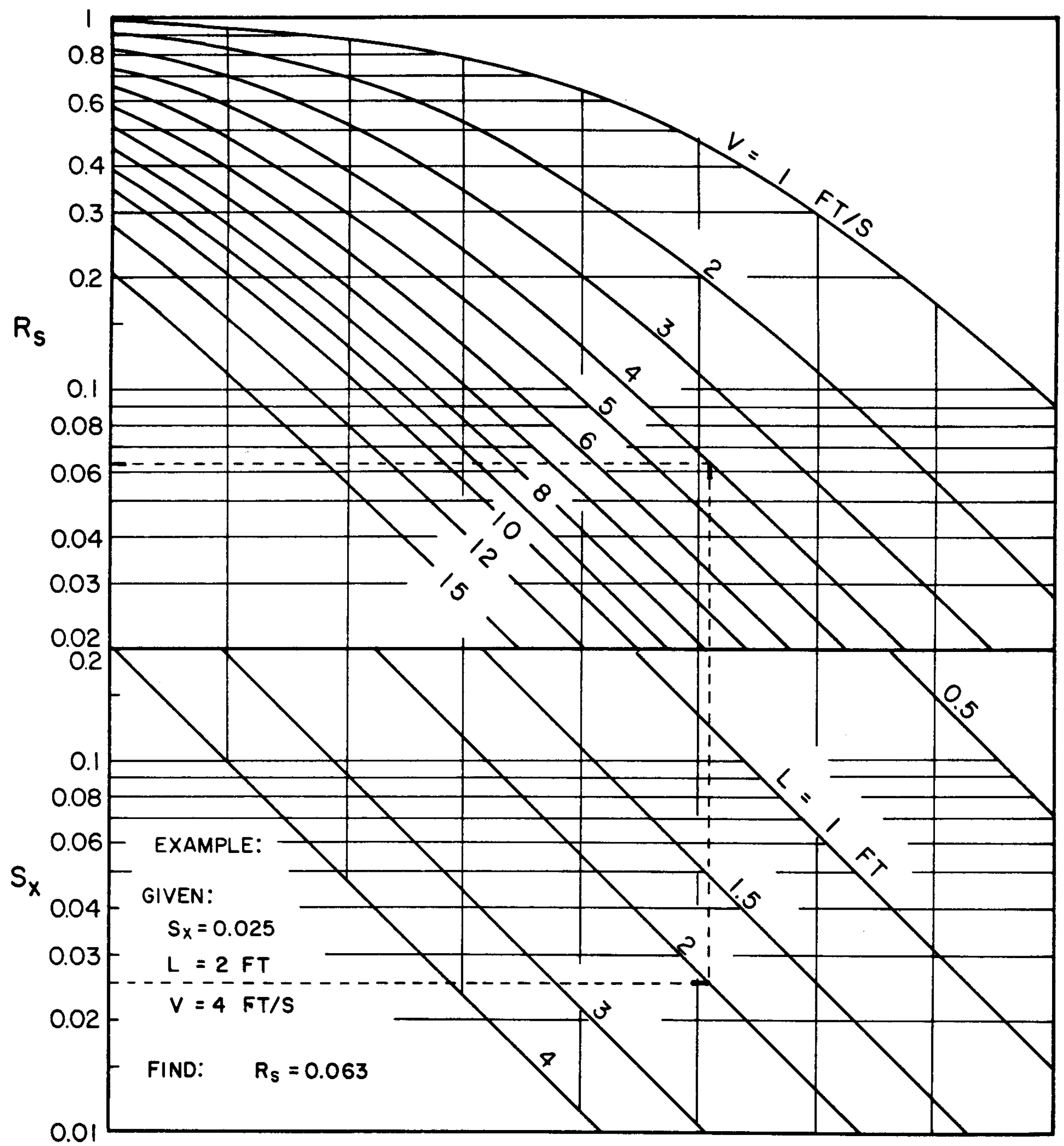


Figure 3.2-5 Grate Inlet Side Flow Interception Efficiency
(Source: HEC-12, 1984)

3.2.5.2 Grate Inlets in Sag

A grate inlet in a sag operates as a weir up to a certain depth, depending on the bar configuration and size of the grate, and as an orifice at greater depths. For a standard gutter inlet grate, weir operation continues to a depth of about 0.4 feet above the top of grate and when depth of water exceeds about 1.4 feet, the grate begins to operate as an orifice. Between depths of about 0.4 feet and about 1.4 feet, a transition from weir to orifice flow occurs.

The capacity of grate inlets operating as a weir is:

$$\mathbf{Q_i = CPd^{1.5}} \qquad \textbf{(3.2.12)}$$

where:

P = perimeter of grate excluding bar widths and the side against the curb, ft

C = 3.0

d = depth of water above grate, ft

and as an orifice is:

$$\mathbf{Q_i = CA(2gd)^{0.5}} \qquad \textbf{(3.2.13)}$$

where:

C = 0.67 orifice coefficient

A = clear opening area of the grate, ft^2

g = 32.2 ft/s^2

Figure 3.2-6 is a plot of equations 3.2.12 and 3.2.13 for various grate sizes. The effect of grate size on the depth at which a grate operates as an orifice is apparent from the chart. Transition from weir to orifice flow results in interception capacity less than that computed by either weir or the orifice equation. This capacity can be approximated by drawing in a curve between the lines representing the perimeter and net area of the grate to be used. The following example illustrates the use of this figure.

Given:

A symmetrical sag vertical curve with equal bypass from inlets upgrade of the low point; allow for 50% clogging of the grate.

Q_b = 3.6 cfs

Q = 8 cfs, 25-year storm

T = 10 ft, design

S_x = 0.05 ft/ft

d = TS_x = 0.5 ft

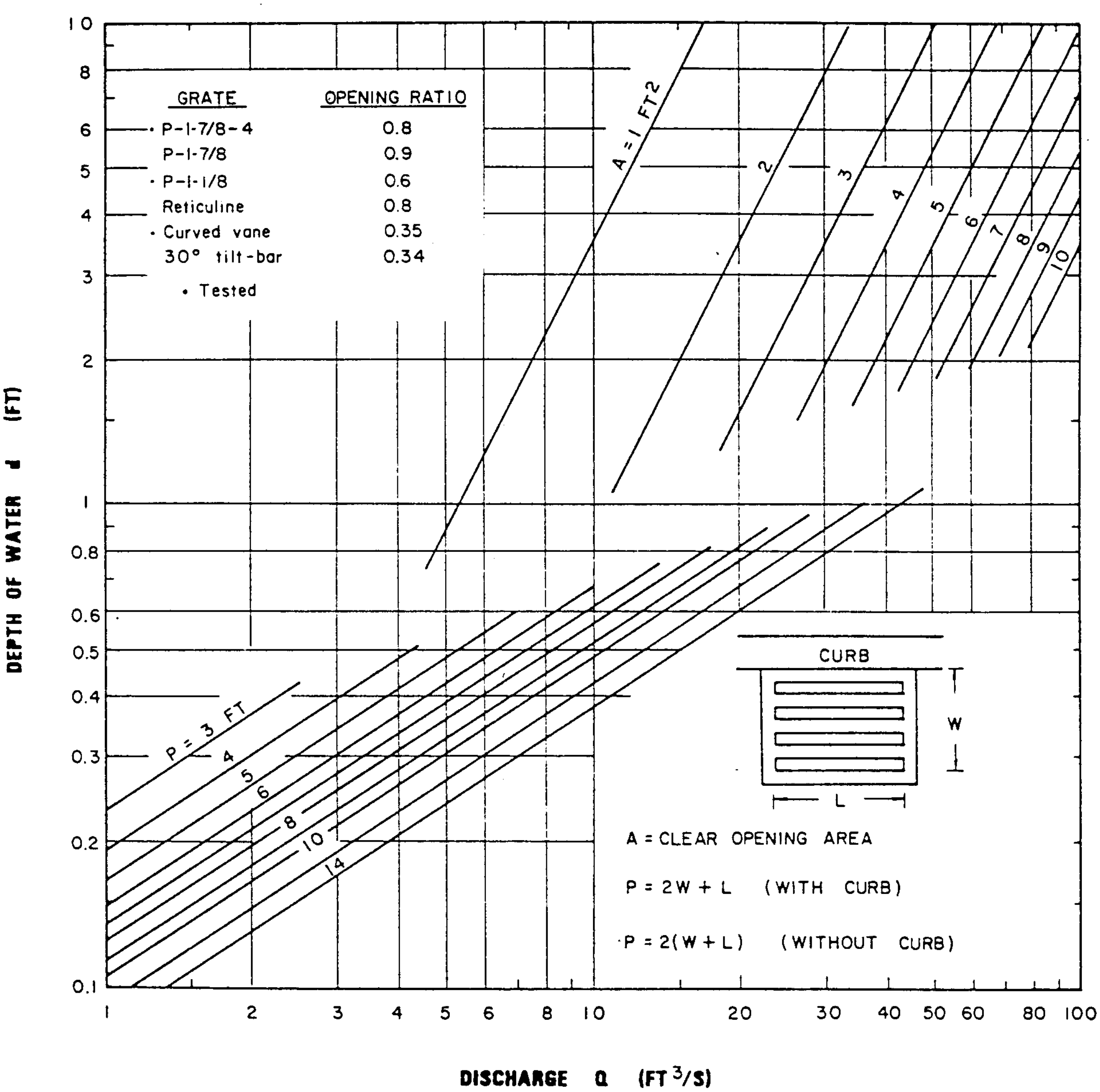
Find:

Grate size for design Q. Check spread at S = 0.003 on approaches to the low point.

Solution:

From Figure 3.2-6, a grate must have a perimeter of 8 ft to intercept 8 cfs at a depth of 0.5 ft.

Some assumptions must be made regarding the nature of the clogging in order to compute the capacity of a partially clogged grate. If the area of a grate is 50% covered by debris so that the debris-covered portion does not contribute to interception, the effective perimeter will be reduced by a lesser amount than 50%. For example if a 2-ft x 4-ft grate is clogged so that the effective width is 1 ft, then the perimeter, P = 1 + 4 + 1 = 6 ft, rather than 8 ft, the total perimeter, or 4 ft, half of the total perimeter. The area of the opening would be reduced by 50% and the perimeter by 25%.



Reference: USDOT, FHWA, HEC-12 (1984).

Figure 3.2-6 Grate Inlet Capacity in Sag Conditions
(Source: HEC-12, 1984)

Therefore, assuming 50% clogging along the length of the grate, a 4 x 4, a 2 x 6, or a 3 x 5 grate would meet requirements of an 8-ft perimeter 50% clogged.

Assuming that the installation chosen to meet design conditions is a double 2 x 3 ft grate, for 50% clogged conditions: P = 1 + 6 + 1 = 8 ft

For 25-year flow: d = 0.5 ft (from Figure 3.2-6)

The American Society of State Highway and Transportation Officials (AASHTO) geometric policy recommends a gradient of 0.3% within 50 ft of the level point in a sag vertical curve.

Check T at S = 0.003 for the design and check flow:

Q = 3.6 cfs, T = 8.2 ft (25-year storm) (from Figure 3.2-1)

Thus a double 2 x 3-ft grate 50% clogged is adequate to intercept the design flow at a spread that does not exceed design spread, and spread on the approaches to the low point will not exceed design spread. However, the tendency of grate inlets to clog completely warrants consideration of a combination inlet, or curb-opening inlet in a sag where ponding can occur, and flanking inlets on the low gradient approaches.

3.2.6 – Curb Inlet Design

3.2.6.1 Curb Inlets on Grade

Following is a discussion of the procedures for the design of curb inlets on grade. Curb-opening inlets are effective in the drainage of pavements where flow depth at the curb is sufficient for the inlet to perform efficiently. Curb openings are relatively free of clogging tendencies and offer little interference to traffic operation. They are a viable alternative to grates in many locations where grates would be in traffic lanes or would be hazardous for pedestrians or bicyclists.

The length of curb-opening inlet required for total interception of gutter flow on a pavement section with a straight cross slope is determined using Figure 3.2-7. The efficiency of curb-opening inlets shorter than the length required for total interception is determined using Figure 3.2-8.

The length of inlet required for total interception by depressed curb-opening inlets or curb-openings in depressed gutter sections can be found by the use of an equivalent cross slope, S_e, in the following equation:

$$\mathbf{S_e = S_x + S'_w E_o} \qquad \textbf{(3.2.14)}$$

where:

E_o = ratio of flow in the depressed section to total gutter flow

S'_w = cross slope of gutter measured from the cross slope of the pavement, S_x

S'_w = (a/12W)

where:

a = gutter depression, in

W = width of depressed gutter, ft

It is apparent from examination of Figure 3.2-7 that the length of curb opening required for total interception can be significantly reduced by increasing the cross slope or the equivalent cross slope. The equivalent cross slope can be increased by use of a continuously depressed gutter section or a locally depressed gutter section.

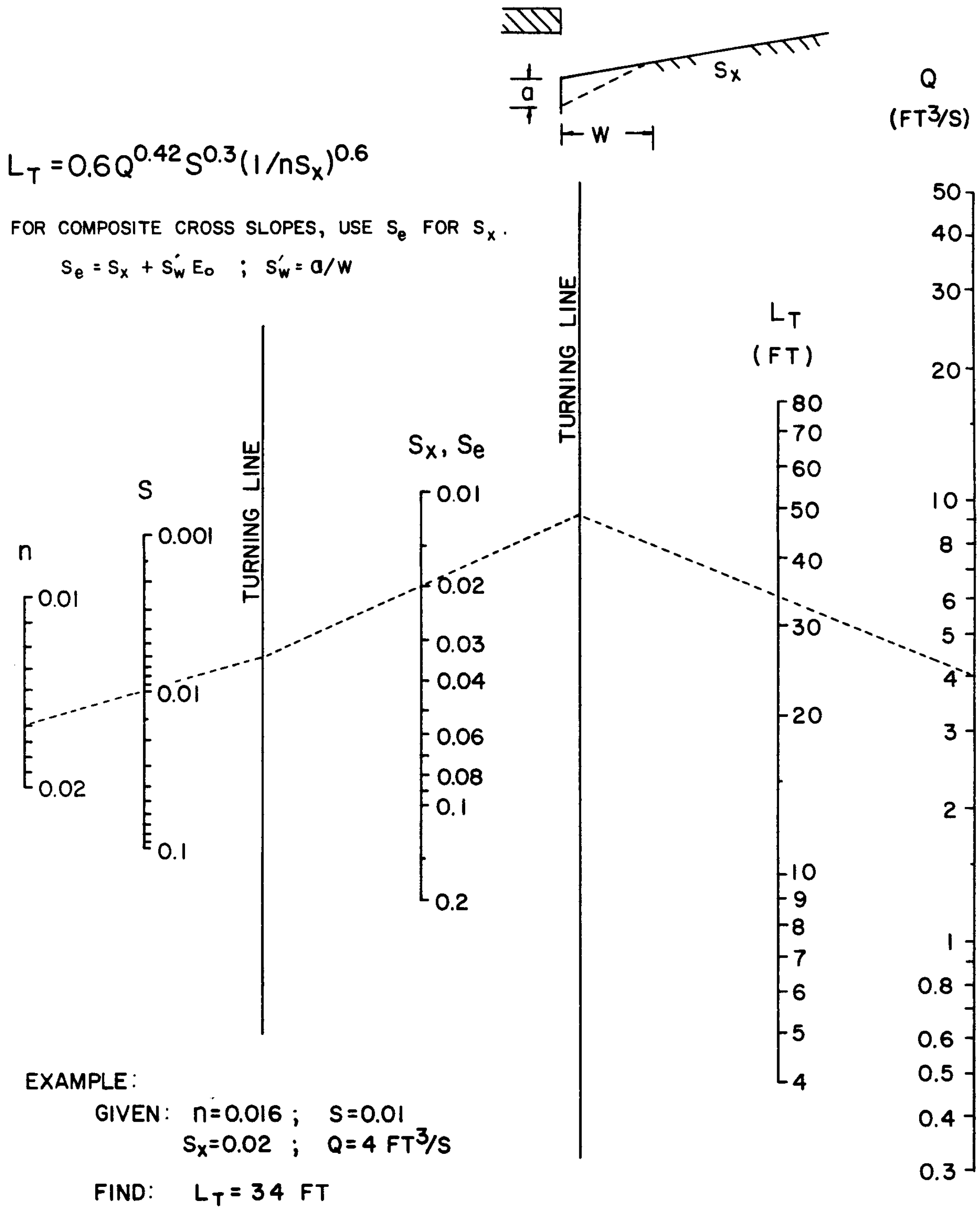


Figure 3.2-7 Curb-Opening and Slotted Drain Inlet Length for Total Interception
(Source: HEC-12, 1984)

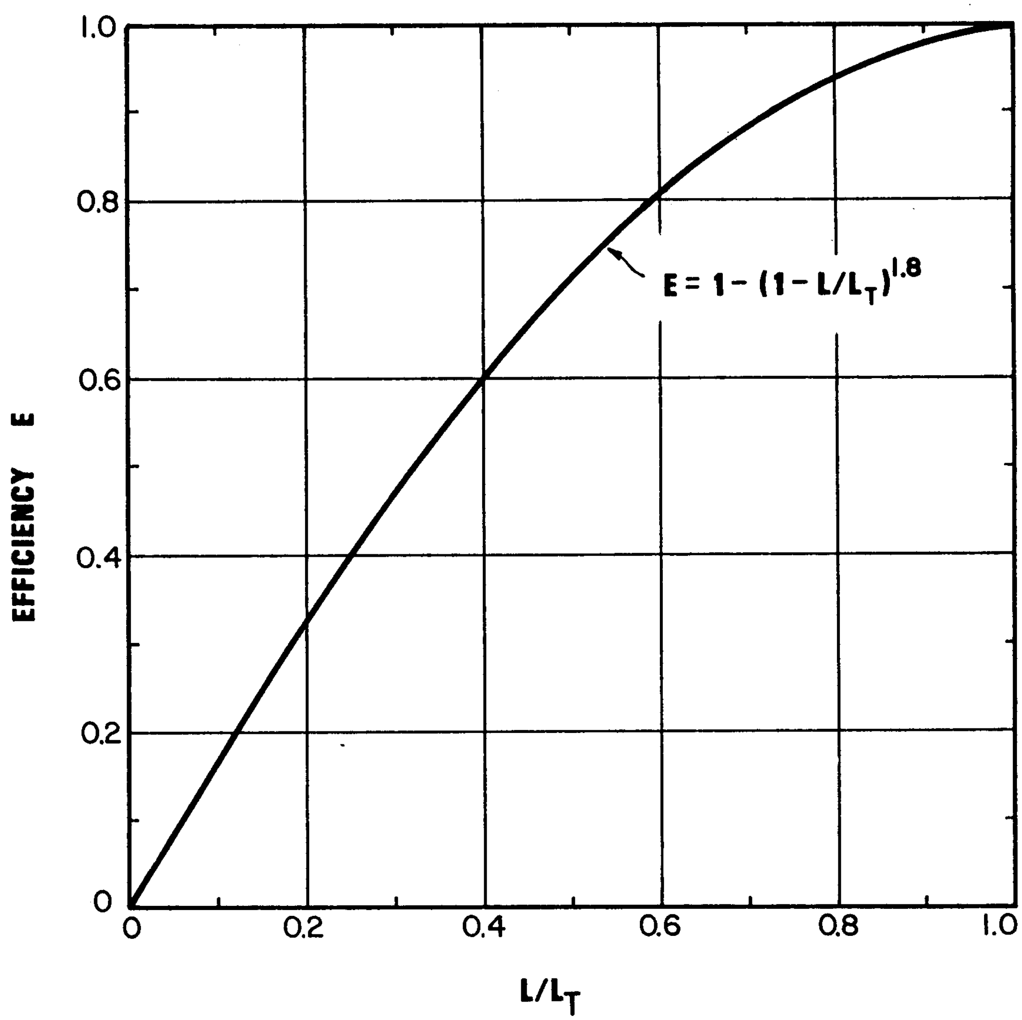


Figure 3.2-8 Curb-Opening and Slotted Drain Inlet Interception Efficiency
(Source: HEC-12, 1984)

Design Steps

Steps for using Figures 3.2-7 and 3.2-8 in the design of curb inlets on grade are given below.

Step 1 Determine the following input parameters:

Cross slope = S_x (ft/ft)

Longitudinal slope = S (ft/ft)

Gutter flow rate = Q (cfs)

Manning's n = n

Spread of water on pavement = T (ft) from Figure 3.2-1

Step 2 Enter Figure 3.2-7 using the two vertical lines on the left side labeled n and S. Locate the value for Manning's n and longitudinal slope and draw a line connecting these points and extend this line to the first turning line.

Step 3 Locate the value for the cross slope (or equivalent cross slope) and draw a line from the point on the first turning line through the cross slope value and extend this line to the second turning line.

Step 4 Using the far right vertical line labeled Q locate the gutter flow rate. Draw a line from this value to the point on the second turning line. Read the length required from the vertical line labeled L_T.

Step 5 If the curb-opening inlet is shorter than the value obtained in Step 4, Figure 3.2-8 can be used to calculate the efficiency. Enter the x-axis with the L/L_T ratio and draw a vertical line upward to the E curve. From the point of intersection, draw a line horizontally to the intersection with the y-axis and read the efficiency value.

Example

Given:

S_x = 0.03 ft/ft

n = 0.016

S = 0.035 ft/ft

Q = 5 cfs

S'_w = 0.083 (a = 2 in, W = 2 ft)

Find:

1. Q_i for a 10-ft curb-opening inlet
2. Q_i for a depressed 10-ft curb-opening inlet with a = 2 in, W = 2 ft, T = 8 ft (Figure 3.2-1)

Solution:

1. From Figure 3.2-7, L_T = 41 ft, L/L_T = 10/41 = 0.24

   From Figure 3.2-8, E = 0.39, Q_i = EQ = 0.39 x 5 = 2 cfs
2. Qn = 5.0 x 0.016 = 0.08 cfs

   S_w/S_x = (0.03 + 0.083)/0.03 = 3.77

   T/W = 3.5 (from Figure 3.2-3)

   T = 3.5 x 2 = 7 ft

   W/T = 2/7 = 0.29 ft

E_o = 0.72 (from Figure 3.2-2)

Therefore, $S_e = S_x + S'_wE_o$ = 0.03 + 0.083(0.72) = 0.09

From Figure 3.2-7, L_T = 23 ft, L/L_T = 10/23 = 0.43

From Figure 3.2-8, E = 0.64, Q_i = 0.64 x 5 = 3.2 cfs

The depressed curb-opening inlet will intercept 1.6 times the flow intercepted by the undepressed curb opening and over 60% of the total flow.

3.2.6.2 Curb Inlets in Sumps

For the design of a curb-opening inlet in a sump location, the inlet operates as a weir to depths equal to the curb opening height and as an orifice at depths greater than 1.4 times the opening height. At depths between 1.0 and 1.4 times the opening height, flow is in a transition stage.

The capacity of curb-opening inlets in a sump location can be determined from Figure 3.2-9, which accounts for the operation of the inlet as a weir and as an orifice at depths greater than 1.4h. This figure is applicable to depressed curb-opening inlets and the depth at the inlet includes any gutter depression. The height (h) in the figure assumes a vertical orifice opening (see sketch on Figure 3.2-9). The weir portion of Figure 3.2-9 is valid for a depressed curb-opening inlet when $d \leq (h + a/12)$.

The capacity of curb-opening inlets in a sump location with a vertical orifice opening but without any depression can be determined from Figure 3.2-10. The capacity of curb-opening inlets in a sump location with other than vertical orifice openings can be determined by using Figure 3.2-11.

Design Steps

Steps for using Figures 3.2-9, 3.2-10, and 3.2-11 in the design of curb-opening inlets in sump locations are given below.

Step 1 Determine the following input parameters:
Cross slope = S_x (ft/ft)

Spread of water on pavement = T (ft) from Figure 3.2-1

Gutter flow rate = Q (cfs) or dimensions of curb-opening inlet [L (ft) and H (in)]

Dimensions of depression if any [a (in) and W (ft)]

Step 2 To determine discharge given the other input parameters, select the appropriate figure (3.2-9, 3.2-10, or 3.2-11 depending on whether the inlet is in a depression and if the orifice opening is vertical).

Step 3 To determine the discharge (Q), given the water depth (d), locate the water depth value on the y-axis and draw a horizontal line to the appropriate perimeter (p), height (h), length (L), or width x length (hL) line. At this intersection draw a vertical line down to the x-axis and read the discharge value.

Step 4 To determine the water depth given the discharge, use the procedure described in Step 3 except enter the figure at the value for the discharge on the x-axis.

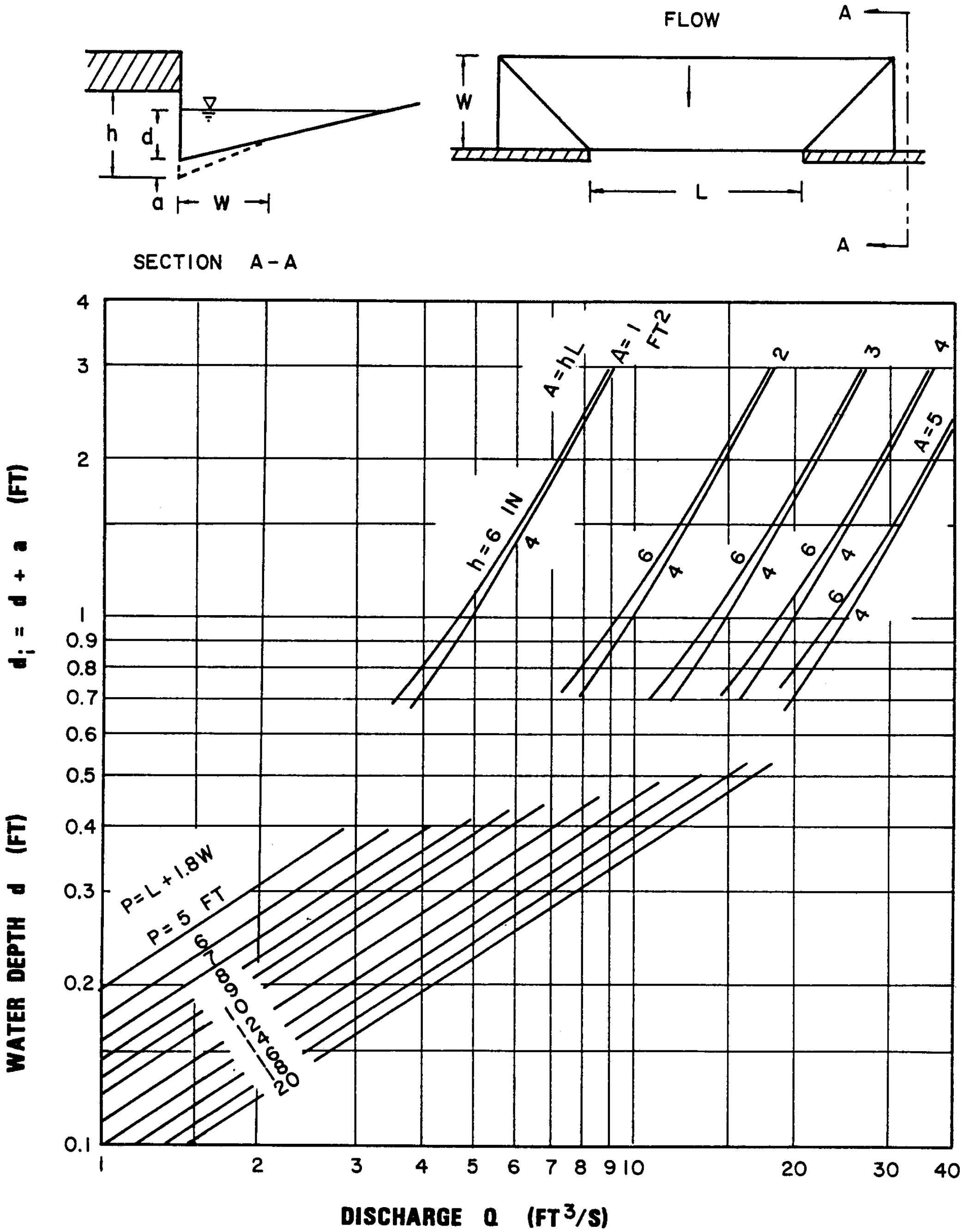


Figure 3.2-9 Depressed Curb-Opening Inlet Capacity in Sump Locations
(Source: AASHTO Model Drainage Manual, 1991)

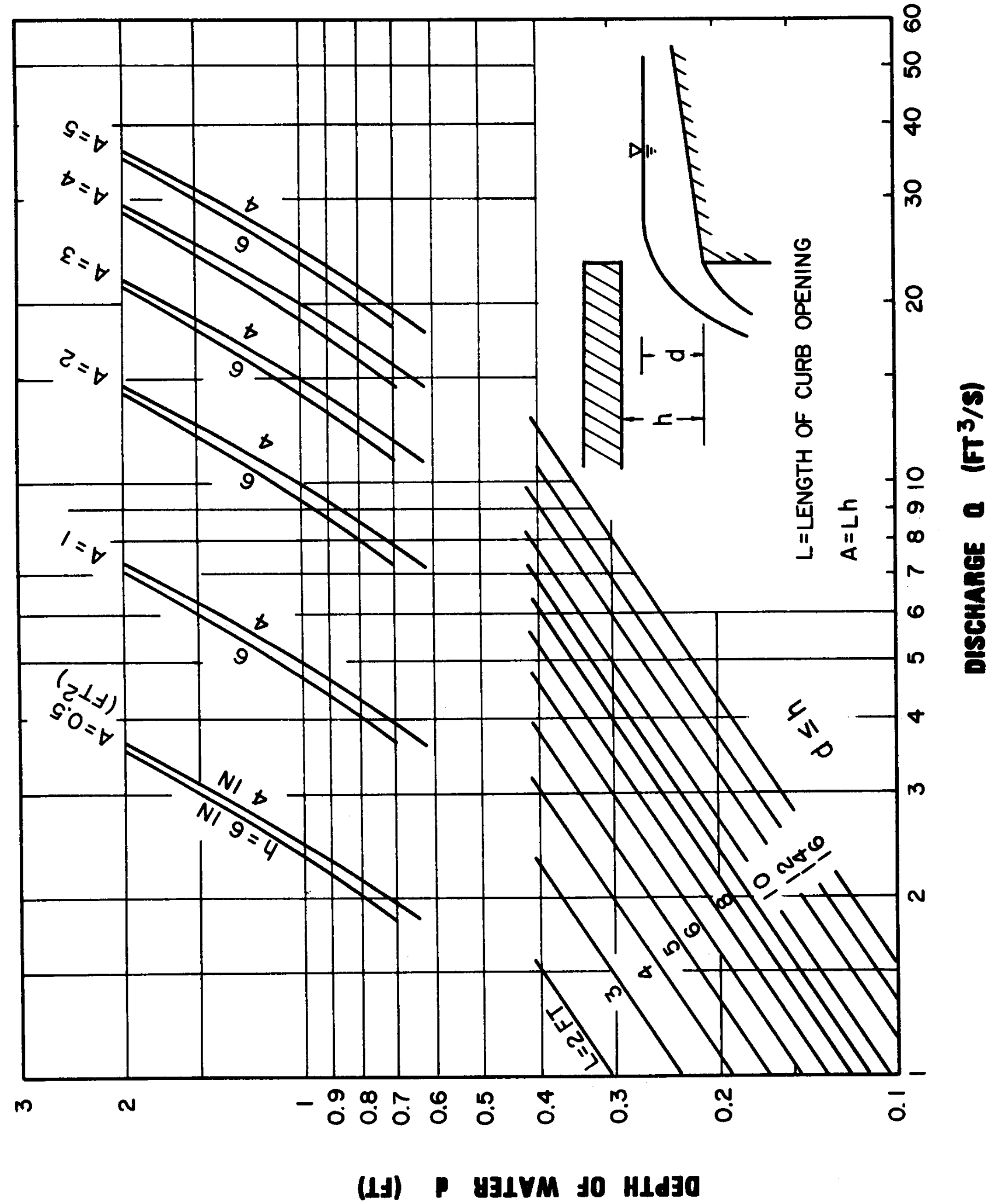


Figure 3.2-10 Curb-Opening Inlet Capacity in Sump Locations
(Source: AASHTO Model Drainage Manual, 1991)

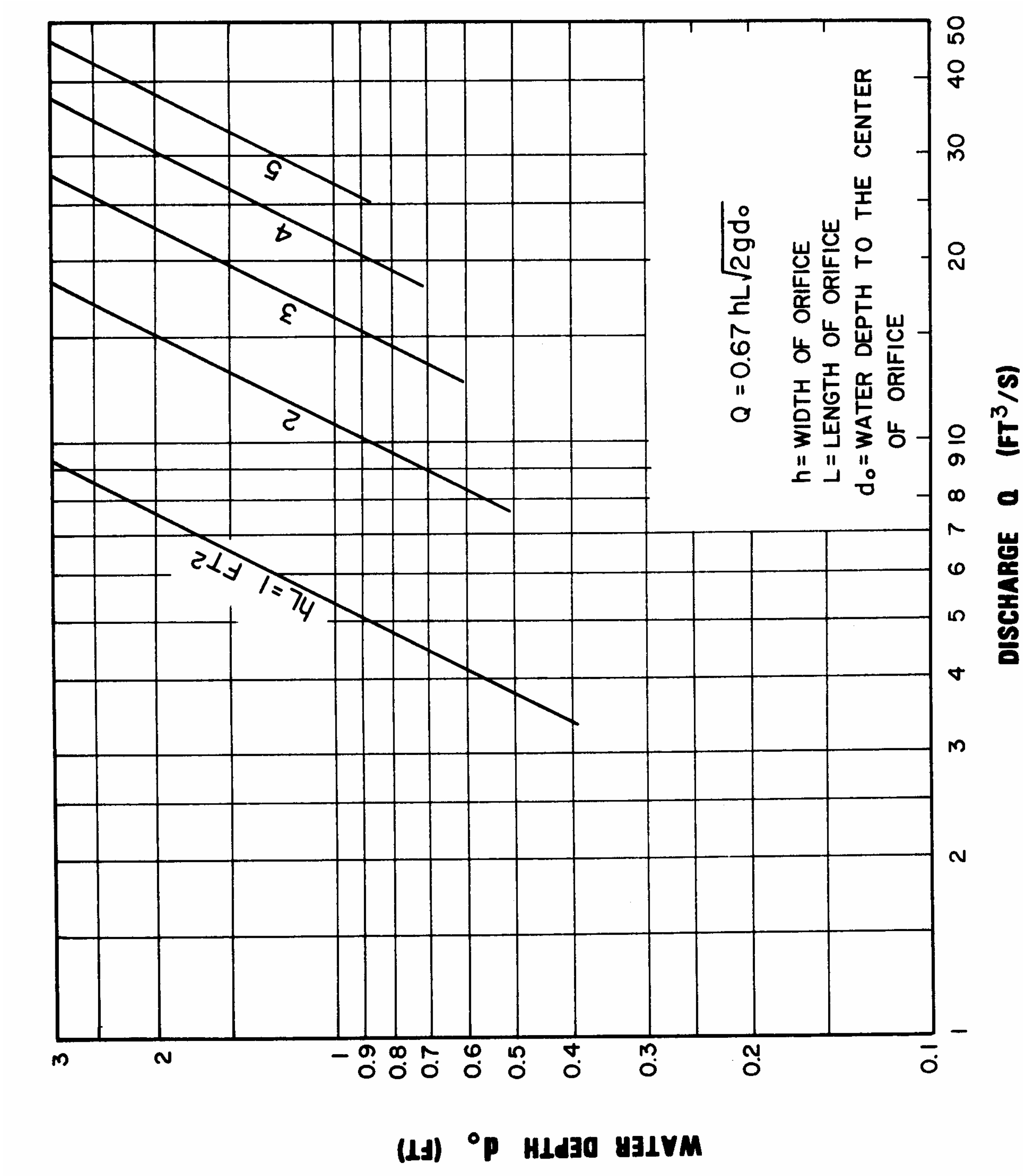


Figure 3.2-11 Curb-Opening Inlet Orifice Capacity for Inclined and Vertical Orifice Throats
(Source: AASHTO Model Drainage Manual, 1991)

Example:"

Given:

Curb-opening inlet in a sump location

L = 5 ft

h = 5 in

1. Undepressed curb opening

   S_x = 0.05 ft/ft

   T = 8 ft

2. Depressed curb opening

   S_x = 0.05 ft/ft

   a = 2 in

   W = 2 ft

   T = 8 ft

Find:

Discharge Q_i

Solution:

1. d = TS_x = 8 x 0.05 = 0.4 ft

   d < h

   From Figure 3.2-10, Q_i = 3.8 cfs

2. d = 0.4 ft

   h + a/12 = (5 + 2/12)/12 = 0.43 ft

   since d < 0.43 the weir portion of Figure 3.2-9 is applicable (lower portion of the figure).

   P = L + 1.8W = 5 + 3.6 = 8.6 ft

   From Figure 3.2-9, Q_i = 5 cfs

At d = 0.4 ft, the depressed curb-opening inlet has about 30% more capacity than an inlet without depression.

3.2.7 – Combination Inlets

3.2.7.1 Combination Inlets on Grade

On a continuous grade, the capacity of an unclogged combination inlet with the curb opening located adjacent to the grate is approximately equal to the capacity of the grate inlet alone. Thus capacity is computed by neglecting the curb opening inlet and the design procedures should be followed based on the use of Figures 3.2-4, 3.2-5, and 3.2-6.

3.2.7.2 Combination Inlets in Sump

All debris carried by storm water runoff that is not intercepted by upstream inlets will be concentrated at the inlet located at the low point, or sump. Because this will increase the probability of clogging for grated inlets, it is generally appropriate to estimate the capacity of a combination inlet at a sump by neglecting the grate inlet capacity. Assuming complete clogging of the grate, Figures 3.2-9, 3.2-10, and 3.2-11 for curb-opening inlets should be used for design.

3.2.8 – Closed Conduit Systems (Storm Drains/Sewers)

Storm drain pipe systems, also known as *storm sewers*, are pipe conveyances used for transporting runoff from roadway and other inlets to outfalls at other structural storm water controls and receiving waters. Pipe drain systems are suitable mainly for medium to high-density residential and commercial/industrial development where the use of natural drainageways and/or vegetated open channels is not feasible.

Closed conduit system are composed of different lengths and sizes of conduits (system segments) connected by appointment structures (system nodes). Segments are most often circular pipe, but can be a box or other enclosed conduit. Materials used are usually corrugated metal, plastic, and concrete but may be of other materials.

Appurtenant structures serve many functions. Inlets, access holes, and junction chambers are presented in sections 3.2.8.1 through 3.2.8.3.

3.2.8.1 Inlets

The primary function is to allow surface water to enter the closed conduit system. Inlet structures may also serve as access points for cleaning and inspection. Typical inlets structures are a standard drop inlet, catch basin, curb inlet, combination inlet, and Y inlet. (See Figures 3.2-12 and 3.2-13).

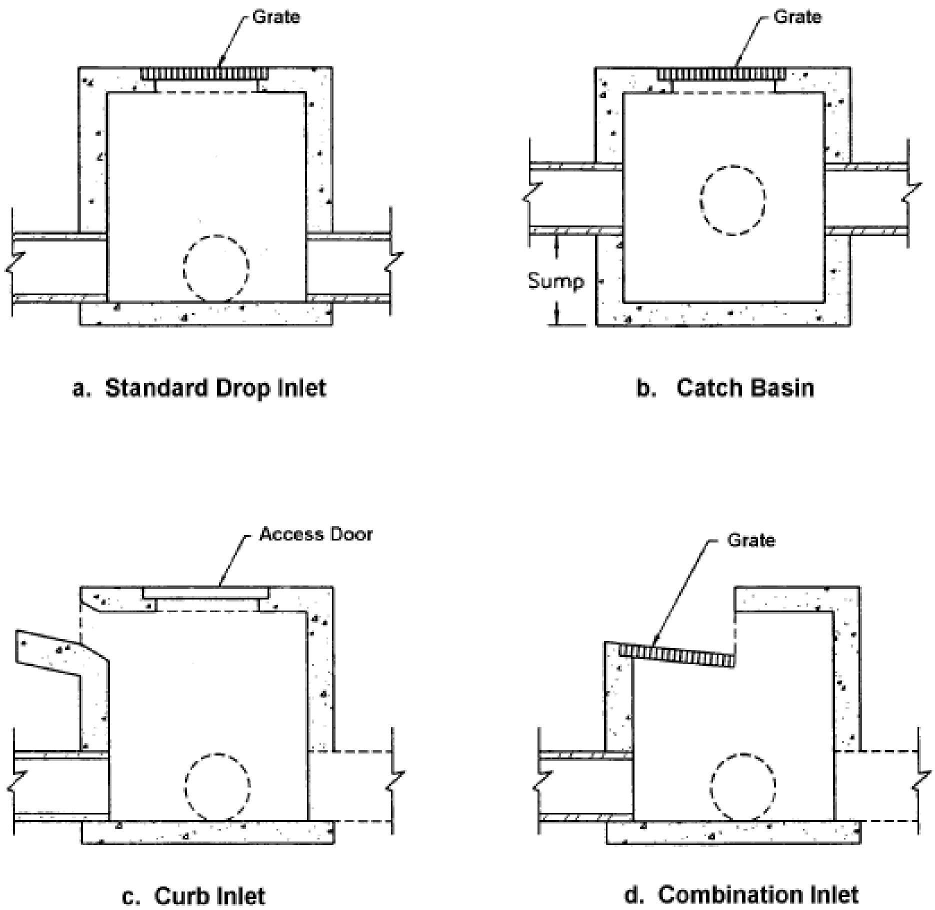


Figure 3.2-12 Inlet Structures
(HEC 22, 2001)

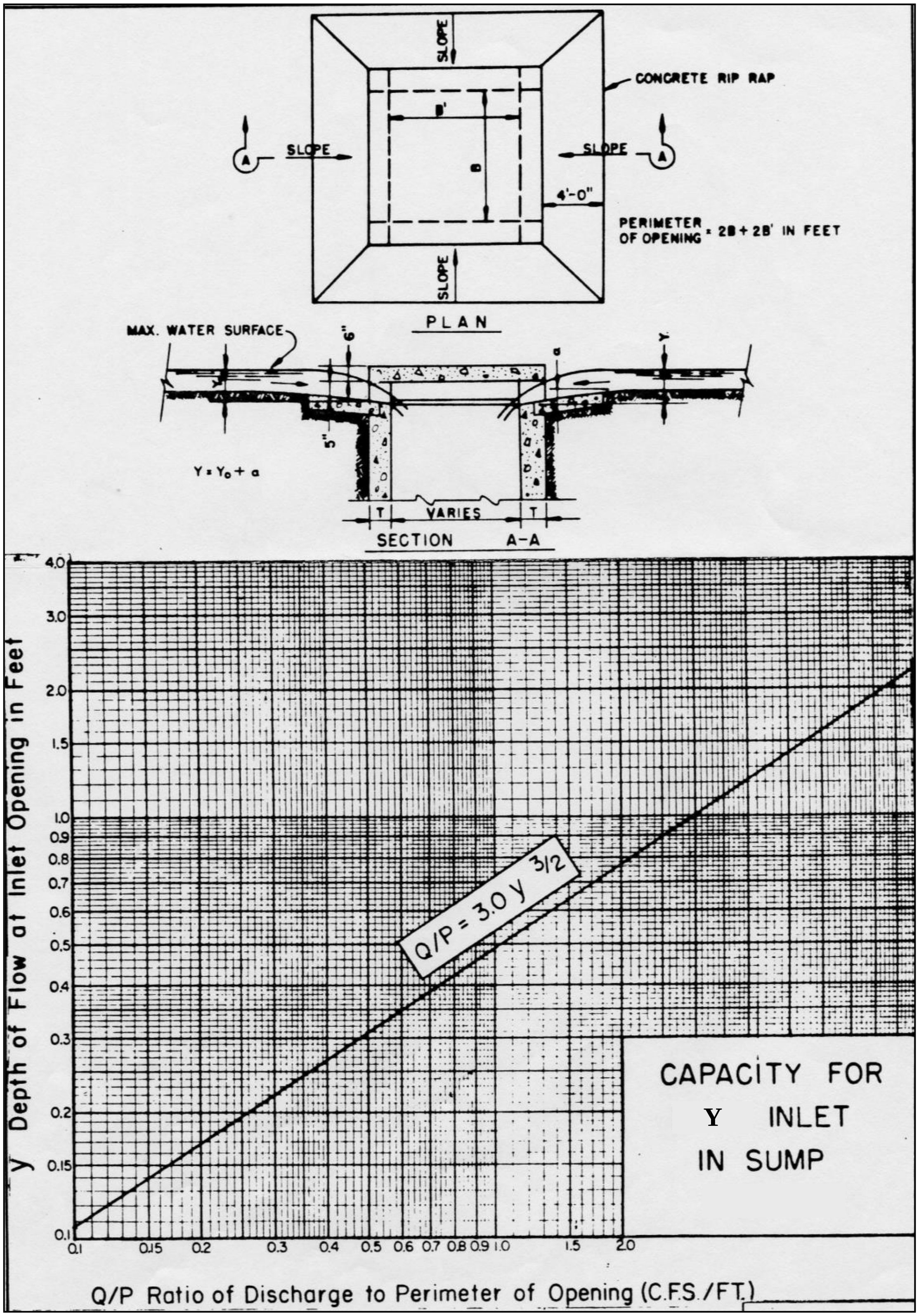


Figure 3.2-13 Capacity for Y Inlet in Sump

(Fort Worth, 1967)

Inlet structures are located at the upstream end and at intermediate points within the closed conduit system. Inlet placement is generally a trial and error procedure that attempts to produce the most economical and hydraulically effective system (HEC 22, 2001).

3.2.8.2 Access Holes (Manholes)

The primary function of an access hole is to provide access to the closed conduit system. An access hole can also serve as a flow junction and can provide ventilation and pressure relief. Typical access holes are shown in Figures 3.2-14 and 3.2-15 (HEC 22, 2001). The materials commonly used for access hole construction are precast concrete and cast-in-place concrete.

Spacing criteria have been established by the City. At a minimum, access holes should be located at the following points:

- Where two or more storm drains converge
- Where pipe sizes change
- Where a change in alignment occurs
- Where a change in grade occurs

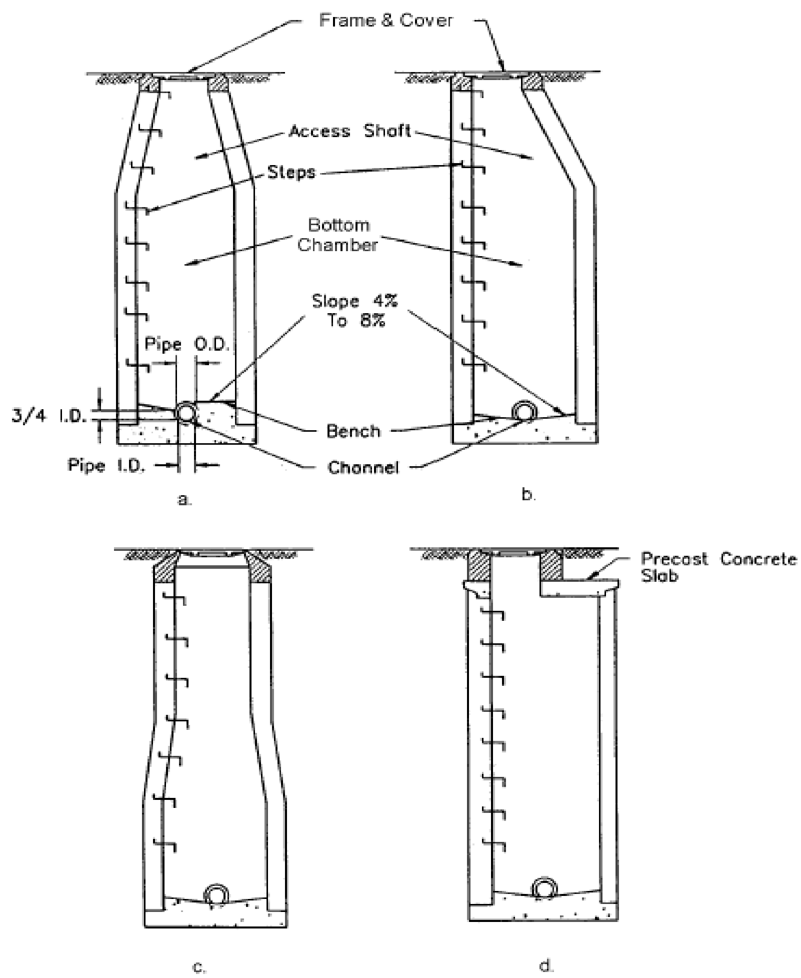


Figure 3.2-14 Typical Access Hole Configurations.
(HEC22, 2001)

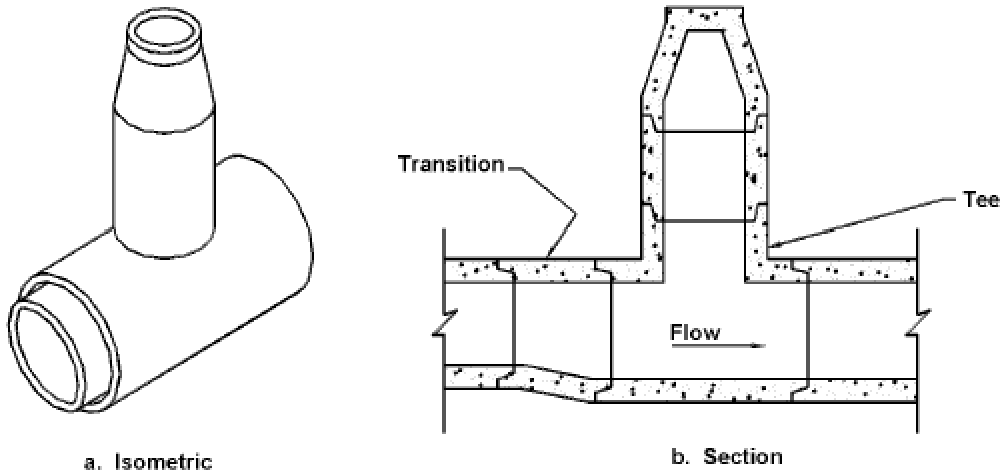


Figure 3.2-15 "Tee" Access Hole for Large Storm Drains
(HEC 22, 2001)

Access holes may be needed at intermediate points along straight runs of closed conduits. Table 3.2-7 gives recommended maximum spacing criteria.

Table 3.2-7 Access Hole Spacing Criteria

(HEC 22, 2001)

Pipe Size (inches)	Suggested Maximum Spacing (feet)
12-24	300
27-36	400
42-54	500
60 and up	1000

3.2.8.3 Junction Chambers

A junction chamber, or junction box, is a special design underground chamber used to join two or more large storm drain conduits. This type of structure is usually required where storm drains are larger than the size that can be accommodated by standard access holes. For smaller diameter storm drains, access holes are typically used instead of junction chambers. Junction chambers by definition do not need to extend to the ground surface and can be completely buried. However, it is recommended that riser structures be used to provide surface access and/or to intercept surface runoff.

Materials commonly used for junction chamber construction include pre-cast concrete and cast-in-place concrete. On storm drains constructed of corrugated steel, the junction chambers are sometimes made of the same material.

To minimize flow turbulence in junction boxes, flow channels and benches are typically built into the bottom of the chambers. Where junction chambers are used as access points for the storm drain system, their location should adhere to the spacing criteria outlined in Table 3.2-7.

3.2.8.4 Design Criteria

Specific design criteria will likely vary from community to community. In the design of closed conduit systems, the following are offered for consideration in setting local criteria:

- For ordinary conditions, storm drain pipes should be sized on the assumption that they will flow full or practically full under the design discharge but will not be placed under pressure head. The Manning Formula is recommended for capacity calculations.

- The maximum hydraulic gradient should not produce a velocity that exceeds 15 ft/s.

- The minimum desirable physical slope is the slope that will produce a velocity of 2.5 feet per second when the storm sewer is flowing full.

- If the potential water surface elevation exceeds 1 foot below ground elevation for the design flow, the top of the pipe, or the gutter flow line, whichever is lowest, adjustments are needed in the system to reduce the elevation of the hydraulic grade line.

3.2.8.5 General Design Procedure

The design of storm drain systems generally follows these steps:

Step 1 Determine inlet location and spacing as outlined earlier in this section.

Step 2 Prepare a tentative plan layout of the storm sewer drainage system including:

a. Location of storm drains
b. Direction of flow
c. Location of manholes
d. Location of existing facilities such as water, gas, or underground cables

Step 3 Determine drainage areas and compute runoff using the Rational Method

Step 4 After the tentative locations of inlets, drain pipes, and outfalls (including tailwaters) have been determined and the inlets sized, compute the rate of discharge to be carried by each storm drain pipe and determine the size and gradient of pipe required to care for this discharge. This is done by proceeding in steps from the upstream end of a line downstream to the point at which the line connects with other lines or the outfall, whichever is applicable. The discharge for a run is calculated, the pipe serving that discharge is sized, and the process is repeated for the next run downstream. The storm drain system design computation form (Figure 3.2-25) can be used to summarize hydrologic, hydraulic and design computations.

Step 5 Examine assumptions to determine if any adjustments are needed to the final design.

The rate of discharge at any point in the storm drainage system is not the sum of the inlet flow rates of all inlets above the section of interest. It is generally less that this total. The Rational Method is the most common means of determining design discharges for storm drain design. The time of concentration is very influential in the determination of the design discharge using the Rational Method. The time of concentration is defined as the period required for water to travel from the most hydraulically distant point of the watershed to the point of interest. The designer is usually concerned with two different times of concentration: one for inlet spacing and the other for pipe sizing. The time of concentration for inlet spacing is the time required for water to flow from the hydraulically most distant point of the unique drainage area contributing only to that inlet. Typically, this is the sum of the times required for water to travel overland to the pavement gutter and along the length of the gutter between inlets. If the total time of concentration to the upstream inlet is less than five minutes, a minimum time of concentration of five minutes is used as the duration of rainfall. The time of concentration for each successive inlet should be determined independently in the same manner as was used for the first inlet.

The time of concentration for pipe sizing is defined as the time required for water to travel for the most hydraulically distant point in the total contributing watershed to the design point. Typically, this time consists of two components: (1) the time for overland and gutter flow to reach the first inlet, and (2) the time to flow through the storm drainage system to the point of interest.

The flow path having the longest time of concentration to the point of interest in the storm drainage system will usually define the duration used in selecting the intensity value in the Rational Method. Exceptions to the general application of the Rational Equation exist. For example, a small relatively impervious area within a larger drainage area may have an independent discharge higher than that of the total area. This anomaly may occur because of the higher runoff coefficient (C value) and higher intensity resulting from a short time of concentration. If an exception does exist, it can generally be classified as one of two exception scenarios.

The first exception occurs when a highly impervious section exists at the most downstream area of a watershed and the total upstream area flows through the lower impervious area. When this situation occurs, two separate calculations should be made.

- First, calculate the runoff from the total drainage area with its weighed C value and the intensity associated with the longest time of concentration.
- Second, calculate the runoff using only the smaller less pervious area. The typical procedure would be followed using the C value for the small less pervious area and the intensity associated with the shorter time of concentration.

The results of these two calculations should be compared and the largest value of discharge should be used for design.

The second exception exists when a smaller less pervious area is tributary to the larger primary watershed. When this scenario occurs, two sets of calculations should also be made.

- First, calculate the runoff form the total drainage area with its weighted C value and the intensity associated with the longest time of concentration.
- Second, calculate the runoff to consider how much discharge from the larger primary area is contributing at the same time the peak from the smaller less pervious tributary area is occurring. When the small area is discharging, some discharge from the larger primary area is also contributing to the total discharge. In this calculation, the intensity associated with the time of concentration from the small less pervious area is used. The portion of the larger primary area to be considered is determined by the following: $A_c=A\ (t_{c1}/t_{c2})$.

A_c is the most downstream part of the larger primary area that will contribute to the discharge during the time of concentration associated with the smaller, less pervious area. A is the area of the larger primary area, t_{c1} is the time of concentration of the smaller, less pervious, tributary area, and t_{c2} is the time of concentration associated with the larger primary area as is used in the first calculation. The C value to be used in this computation should be the weighted C value of the smaller less pervious tributary area and the area A_c. The area to be used in the Rational Method would be the area of the less pervious area plus A_c. The second calculation should only be considered when the less pervious area is tributary to the area with the longer time of concentration and is at or near the downstream end of the total drainage area.

Finally, compare the results of these calculations with the largest value of discharge used for design.

3.2.8.6 Capacity Calculations

The design procedures presented here assume flow within each storm drain segment is steady and uniform. This means the discharge and flow depth in each segment are assumed to be constant with respect to time and distance. Also, since storm drain conduits are typically prismatic, the average velocity throughout a segment is considered to be constant.

In actual storm drainage systems, the flow at each inlet is variable, and flow conditions are not truly steady or uniform. However, since the usual hydrologic methods employed in storm drain design are based on computed peak discharges at the beginning of each run, it is a conservative practice to design using the steady uniform flow assumption.

Although at times flow in a closed conduit may be under pressure or at other times the conduit may flow partially full, the usual design assumption is that the conduit is flowing full but not under pressure. Under this assumption the rate of head loss is the same as the slope of the pipe (S_f=S, ft/ft). Designing for full flow is a conservative assumption since the peak flow actually occurs at 93 percent of full flow.

The most widely used formula for determining the hydraulic capacity of storm drain pipes for gravity and pressure flows is the Manning's Formula, expressed by the following equation:

$$\mathbf{V = (1.486/n)\, R^{2/3}\, S^{1/2}} \qquad \textbf{(3.2.15)}$$

where:

- V = mean velocity of flow, ft/s
- R = the hydraulic radius, ft - defined as the area of flow divided by the wetted flow surface or wetted perimeter (A/WP)
- S = the slope of hydraulic grade line, ft/ft
- n = Manning's roughness coefficient

In terms of discharge, the above formula becomes:

$$\mathbf{Q = (1.486/n)\, A\, R^{2/3}\, S^{1/2}} \qquad \textbf{(3.2.16)}$$

where:

- Q = rate of flow, cfs
- A = cross sectional area of flow, ft^2

For pipes flowing full, the area is $(\pi/4)D^2$ and the hydraulic radius is D/4, so, the above equations become:

$V = [0.590\ D^{2/3}S^{1/2}]/n$ **(3.2.17)**

$Q = [0.463\ D^{8/3}S^{1/2}]/n$ **(3.2.18)**

where:

D = diameter of pipe, ft

S = slope of the pipe = S_f hydraulic grade line, ft/ft

The Manning's equation can be written to determine friction losses for storm drain pipes as:

$H_f = [0.453\ n^2V^2L]/[R^{4/3}]$ **(3.2.19)**

$H_f = [(2.87\ n^2V^2L]/[D^{4/3}]$ **(3.2.20)**

$H_f = [(185n^2(V^2/2g)L]/[D^{4/3}]$ **(3.2.21)**

where:

H_f = total head loss due to friction, ft (S_f x L)

n = Manning's roughness coefficient

D = diameter of pipe, ft

L = length of pipe, ft

V = mean velocity, ft/s

R = hydraulic radius, ft

g = acceleration of gravity = 32.2 ft/sec^2

A nomograph solution of Manning's Equation for full flow in circular conduits is presented in Figure 3.2-16. Representative values of the Manning's coefficient for various storm drain materials are provided in Table 3.2-9. It should be remembered that the values in the table are for new pipe tested in a laboratory. Actual field values for conduits may vary depending on the effect of abrasion, corrosion, deflection, and joint conditions.

Figure 3.2-17 illustrates storm drain capacity sensitivity to the parameters in the Manning's equation. This figure can be used to study the effect changes in individual parameters will have on storm drain capacity. For example, if the diameter of a storm drain is doubled, its capacity will be increased by a factor of 6.0; if the slope is doubled, the capacity is increased by a factor of 1.4; however, if the roughness is doubled, the pipe capacity will be reduced by 50 percent.

The hydraulic elements graph in Figures 3.2-18a and 3.2-18b is provided to assist in the solution of the Manning's equation for part full flow in storm drains. The hydraulic elements chart shows the relative flow conditions at different depths in a circular pipe and makes the following important points:

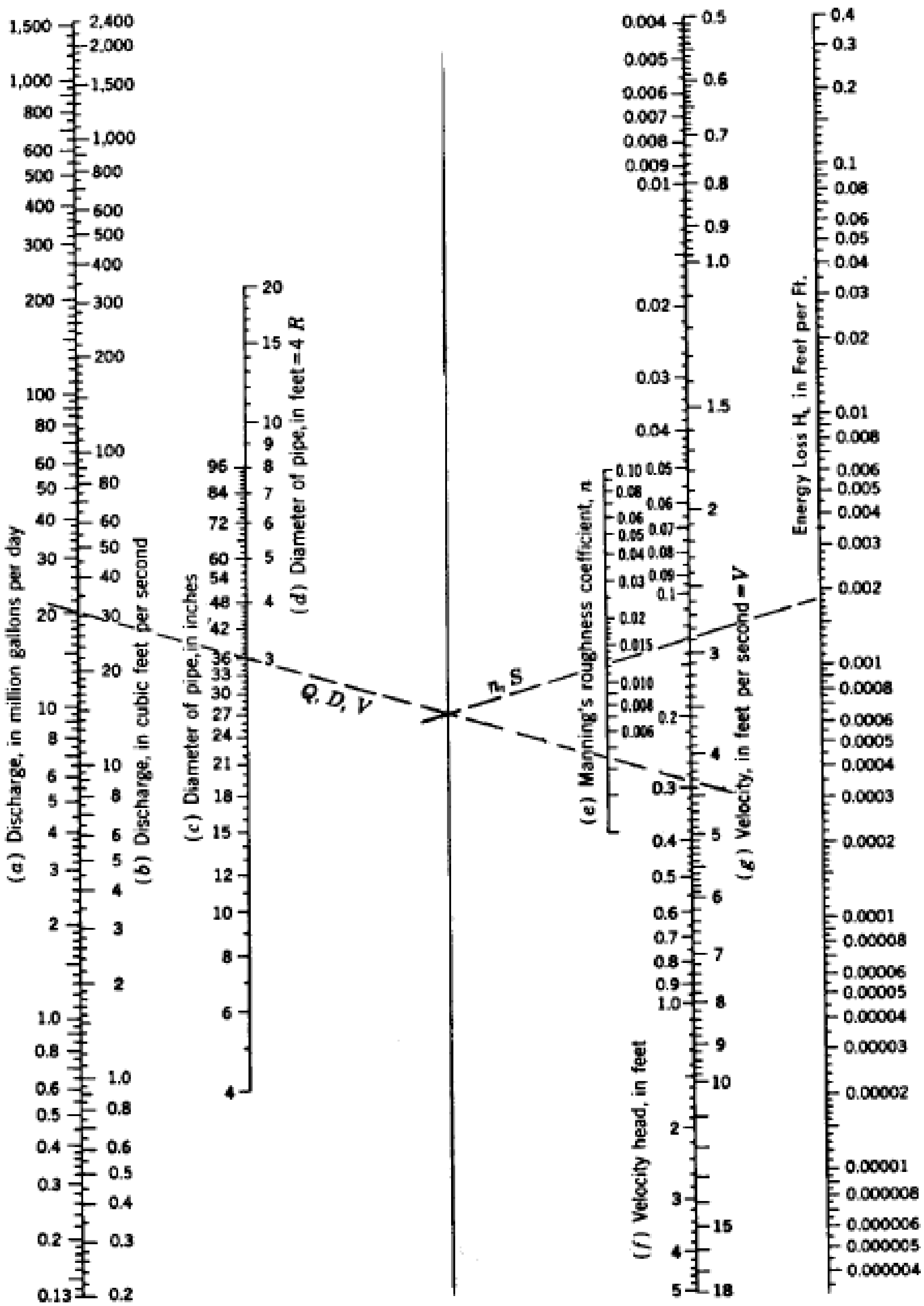
1. Peak flow occurs at 93 percent of the height of the pipe. This means that if the pipe is designed for full flow, the design will be slightly conservative.
2. The velocity in a pipe flowing half-full is the same as the velocity for full flow.
3. Flow velocities for flow depths greater than half-full are greater than velocities at full flow.

4. As the depth of flow drops below half-full, the flow velocity drops off rapidly. The shape of a storm drain conduit also influences its capacity. Although most storm drain conduits are circular, a significant increase in capacity can be realized by using an alternate shape. Table 3.2-8 provides a tabular listing of the increase in capacity which can be achieved using alternate conduit shapes that have the same height as the original circular shape, but have a different cross sectional area. Although these alternate shapes are generally more expensive than circular shapes, their use can be justified in some instances based on their increased capacity.

Table 3.2-8 Increase in Capacity of Alternate Conduit Shapes Based on a Circular Pipe with the Same Height

(HEC-22, 2001)

	Area (Percent Increase)	Conveyance (Percent Increase)
Circular	--	--
Oval	63	87
Arch	57	78
Box (B = D)	27	27



Alignment chart for energy loss in pipes, for Manning's formula.
Note: Use chart for flow computations, $H_L = S$

Figure 3.2-16 Solution of Manning's Equation for Flow in Storm Drains-English Units
(Taken from "Modern Sewer Design" by American Iron and Steel Institute)

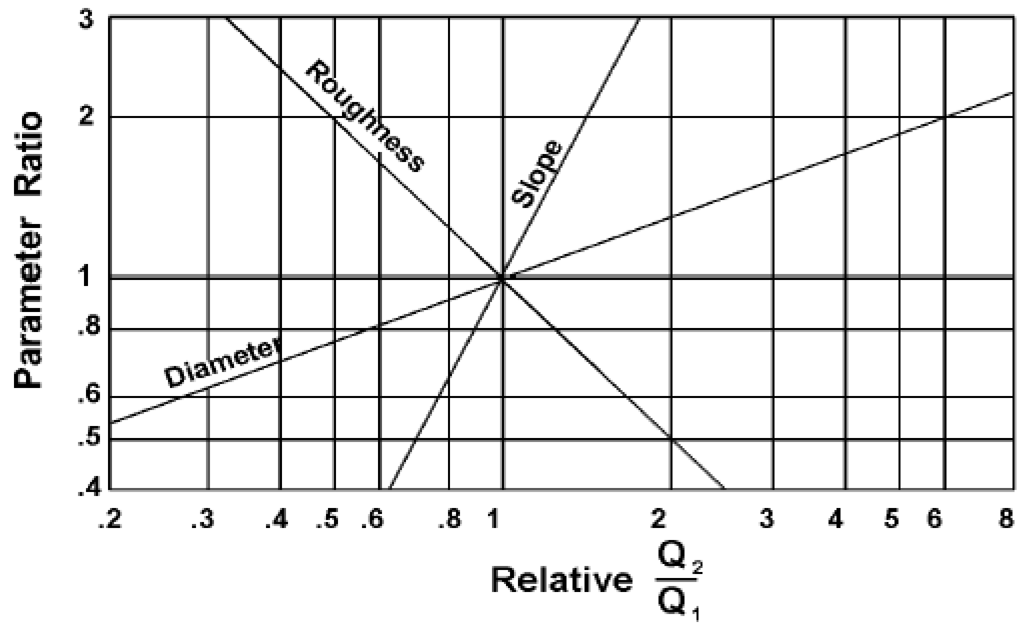


Figure 3.2-17 Storm Drain Capacity Sensitivity
(HEC 22, 2001)

Table 3.2-9 Manning's Coefficients for Storm Drain Conduits
(HEC 22, 2001)

Type of Culvert	Roughness or Corrugation	Manning's n
Concrete Pipe	**Smooth**	**0.010-0.011**
Concrete Boxes	**Smooth**	**0.012-0.015**
Spiral Rib Metal Pipe	**Smooth**	**0.012-0.013**
Corrugated Metal Pipe,	**68 by 13 mm**	**0.022-0.027**
Pipe-Arch and Box	**2-2/3 by 1/2 in**	
(Annular or Helical	**Annular**	
Corrugations -- see Figure B-3 in Reference 2, Manning's n varies with barrel size)	**68 by 13 mm**	**0.011-0.023**
	2-2/3 by 1/2 in	
	Helical	
	150 by 25 mm	**0.022-0.025**
	6 by 1 in	
	Helical	
	125 by 25 mm	**0.025-0.026**
	5 by 1 in	
	75 by 25 mm	**0.027-0.028**
	3 by 1 in	
	150 by 50 mm	
	6 by 2 in	**0.033-0.035**
	Structural Plate	
	230 by 64 mm	
	9 by 2-1/2 in	**0.033-0.037**
	Structural Plate	
Corrugated Polyethylene	**Smooth**	**0.009-0.015**
Corrugated Polyethylene	**Corrugated**	**0.018-0.025**
Polyvinyl chloride (PVC)	**Smooth**	**0.009-0.011**

***NOTE: The Manning's n values indicated in this table were obtained in the laboratory and are supported by the provided reference. Actual field values for culverts may vary depending on the effect of abrasion, corrosion, deflection, and joint conditions.**

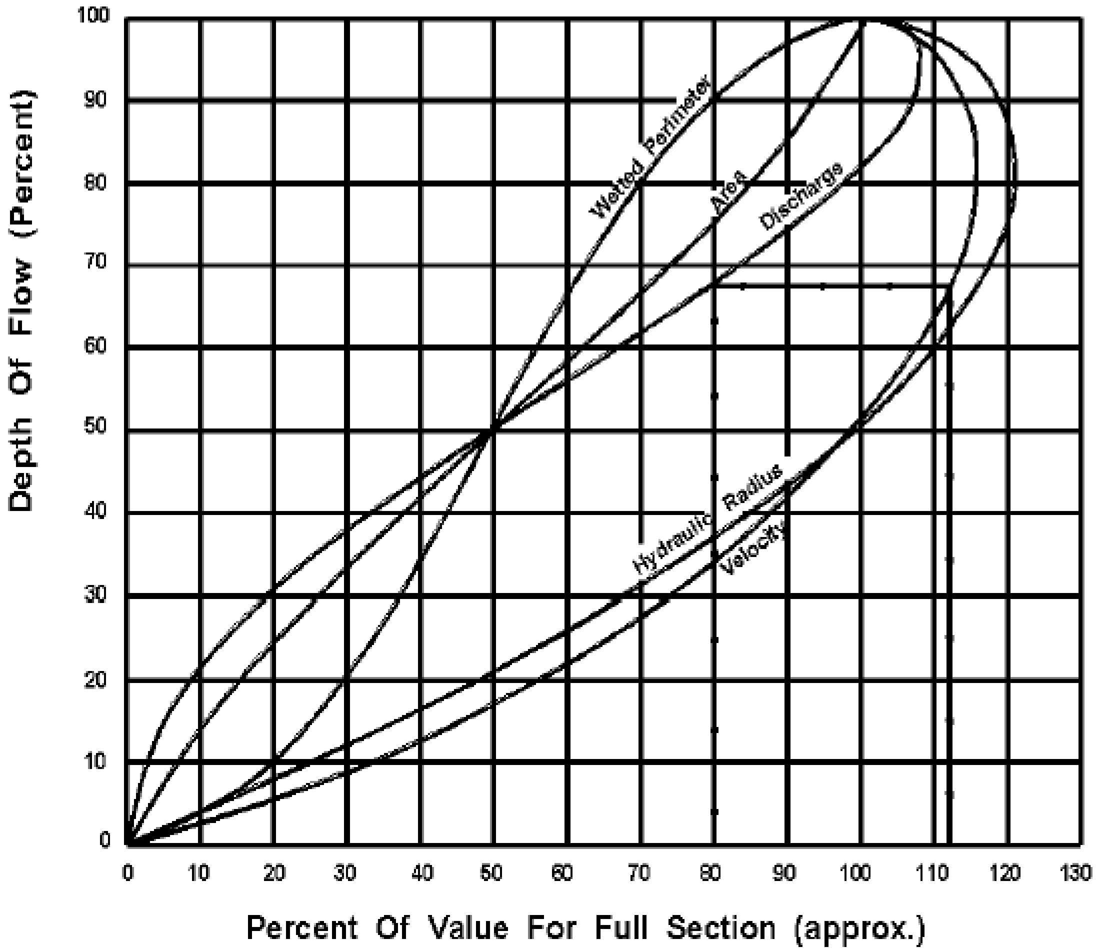


Figure 3.2-18a Hydraulic Elements of Circular Section
(HEC 22, 2001)

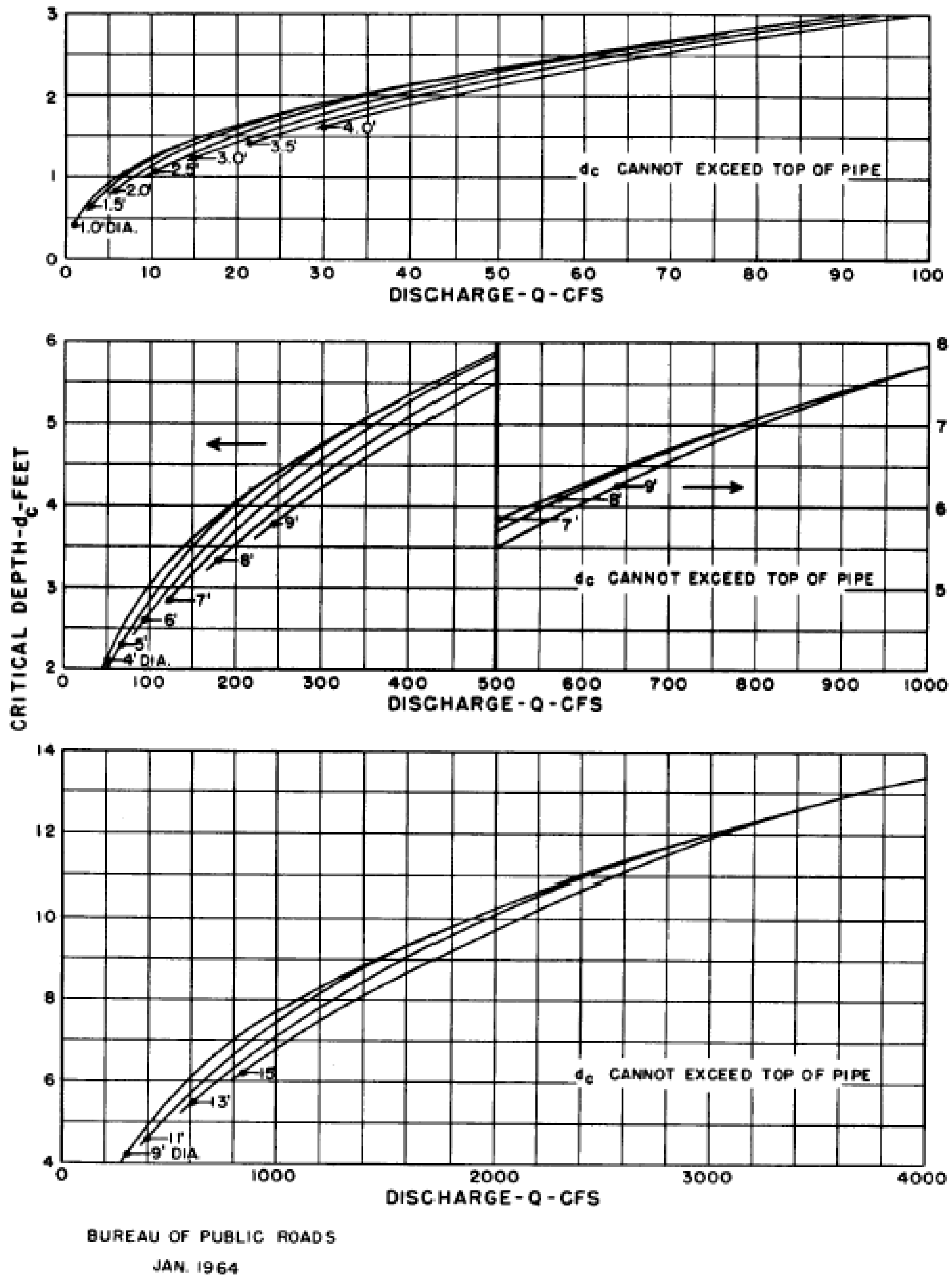


Figure 3.2-18b Critical Depth in Circular Pipe-English Units
(HEC 22, 2001)

3.2.8.7 Minimum Grades and Desirable Velocities

The minimum slopes are calculated by the modified Manning's formula:

$$\mathbf{S = [(nV)^2]/[2.208R^{4/3}]} \qquad \textbf{(3.2.22)}$$

where:

S = the slope of the hydraulic grade line, ft/ft
n = Manning's roughness coefficient
V = mean velocity of flow, ft/s
R = hydraulic radius, ft (area dived by wetted perimeter)

For circular conduits flowing full but not under pressure, R=D/4, and the hydraulic grade line is equal to the slope of the pipe. For these conditions equation 3.2.22 may be expressed as:

$$\mathbf{S = 2.87(nV)^2/D^{4/3}} \qquad \textbf{(3.2.23)}$$

For a minimum velocity of 2.5 fps, the minimum slope equation becomes:

$$\mathbf{S = 17.938(n^2/D^{4/3})} \qquad \textbf{(3.2.24)}$$

where:

D = diameter, ft

Table 3.2-10 gives minimum slopes for concrete pipe with an n-value of 0.013.

Minimum Grades

Storm drains should operate with velocities of flow sufficient to prevent excessive deposits of solid materials; otherwise objectionable clogging may result. The controlling velocity is near the bottom of the conduit and considerably less than the mean velocity of the sewer. Storm drains shall be designed to have a minimum mean velocity flowing full of 2.5 fps. Table 3.2-10 gives minimum slopes for concrete pipe (n = 0.013) flowing at 2.5 fps.

Desirable Velocities

Velocities in sewers are important mainly because of the possibilities of excessive erosion on the storm drain inverts. Table 3.2-11 shows the desirable velocities for most storm drainage design.

Table 3.2-10 Minimum Grades for Storm Drains for 2.5 fps	
Pipe Size (inches)	**Concrete Pipe (n = 0.013) Slope ft/ft**
15	0.0023
18	0.0018
21	0.0014
24	0.0012
27	0.0010
30	0.0009
33	0.0008
36	0.0007
39	0.0006
42	0.0006
45	0.0005
48	0.0005
54	0.0004
60	0.0004
66	0.0003
72	0.0003
78	0.0003
84	0.0002
96	0.0002

Table 3.2-11 Desirable Velocity In Storm Drains	
Description	**Maximum Desirable Velocity**
Culverts (All types)	15 fps.
Storm Drains (Inlet laterals)	No Limit
Storm Drains (Collectors)	15 fps.
Storm Drains (Mains)	12 fps.

3.2.8.8 Storm Drain Storage

If downstream drainage facilities are undersized for the design flow, a structural storm water control may be needed to reduce the possibility of flooding. The required storage volume can also be provided by using larger than needed storm drain pipe sizes and restrictors to control the release rates at manholes and/or junction boxes in the storm drain system. The same design criteria for sizing structural control storage facilities are used to determine the storage volume required in the system (see Section 4.5 for more information).

3.2.8.9 Energy Grade Line/Hydraulic Grade Line

The energy grade line (EGL) is an imaginary line that represents the total energy along a channel or conduit carrying water. Total energy includes elevation head, velocity head and pressure head.

$$\mathbf{E = V^2/2g + p/v + z} \qquad \textbf{(3.2.25)}$$

where:

E	=	Total energy, ft
$V^2/2g$	=	Velocity head, ft (kinetic energy)
p	=	Pressure, lbs/ft^2
v	=	Unit weight of water, 62.4 lbs/ft^3
p/v	=	Pressure head, ft (potential energy)
z	=	Elevation head, ft (potential energy)

Bernoulli's Law expressed between points one (1) and two (2) in a closed conduit accounts for all energy forms and energy losses. The general form of the law may be written as:

$$\mathbf{V_1^2/2g + p_1/v + z_1 = V_2^2/2g + p_2/v + z_2 - H_f - \Sigma H_m} \qquad \textbf{(3.2.26)}$$

where:

H_f	=	Pipe friction loss, ft
ΣH_m	=	Sum of minor or form losses, ft

The calculation of the EGL for the full length of the system is critical to the evaluation of a storm drain. In order to develop the EGL it is necessary to calculate all of the losses through the system. The energy equation states that the energy head at any cross section must equal that in any other downstream section plus the intervening losses. The intervening losses are typically classified as either friction losses or form losses. The friction losses can be calculated using the Manning's Equation. Form losses are typically calculated by multiplying the velocity head by a loss coefficient, K. Various tables and calculations exist for developing the value of K depending on the structure being evaluated for loss. Knowledge of the location of the EGL is critical to the understanding and estimating the location of the hydraulic grade line (HGL).

The hydraulic grade line (HGL) is a line coinciding with the level of flowing water at any point along an open channel. In closed conduits flowing under pressure, the hydraulic grade line is the level to which water would rise in a vertical tube at any point along the pipe. The hydraulic grade line is used to aid the designer in determining the acceptability of a proposed storm drainage system by establishing the elevation to which water will rise when the system is operating under design conditions.

HGL, a measure of flow energy, is determined by subtracting the velocity head ($V^2/2g$) from the EGL. Energy concepts can be applied to pipe flow as well as open channel flow. Figure 3.2-19 illustrates the energy and hydraulic grade lines for open channel and pressure flow in pipes.

When water is flowing through the pipe and there is a space of air between the top of the water and the inside of the pipe, the flow is considered as open channel flow and the HGL is at the water surface. When the pipe is flowing full under pressure flow, the HGL will be above the crown of the pipe. When the flow in the pipe just reaches the point where the pipe is flowing full, this condition lies in between open channel flow and pressure flow. At this condition the pipe is under gravity full flow and the flow is influenced by the resistance of the total circumference of the pipe. Under gravity full flow, the HGL coincides with the crown of the pipe.

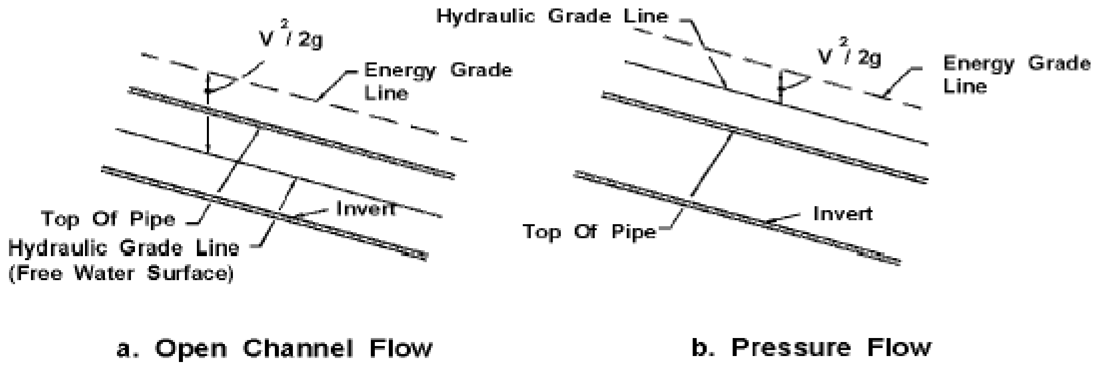


Figure 3.2-19 Hydraulic and Energy Grade Lines in Pipe Flow
(HEC 22, 2001)

Inlet surcharging and possible access hole lid displacement can occur if the hydraulic grade line rises above the ground surface. A design based on open channel conditions must be carefully planned as well, including evaluation of the potential for excessive and inadvertent flooding created when a storm event larger than the design storm pressurizes the system. As hydraulic calculations are performed, frequent verification of the existence of the desired flow condition should be made. Storm drainage systems can often alternate between pressure and open channel flow conditions from one section to another.

A detailed procedure for evaluating the energy grade line and the hydraulic grade line for storm drainage systems is presented in section 3.2.8.12.

3.2.8.10 Storm Drain Outfalls

All storm drains have an outlet where flow from the storm drainage system is discharged. The discharge point can be a natural river or stream, an existing storm drainage system, or a channel which is either existing or proposed for the purpose of conveying the storm water. The procedure for calculating the energy grade line through a storm drainage system begins at the outfall. Therefore, consideration of outfall conditions is an important part of storm drain design.

Several aspects of outfall design must be given serious consideration. These include the flowline or invert (inside bottom) elevation of the proposed storm drain outlet, tailwater elevations, the need for energy dissipation, and the orientation of the outlet structure.

The flowline or invert elevation of the proposed outlet should be equal to or higher than the flowline of the outfall. If this is not the case, there may be a need to pump or otherwise lift the water to the elevation of the outfall.

The tailwater depth or elevation in the storm drain outfall must be considered carefully. Evaluation of the hydraulic grade line for a storm drainage system begins at the system outfall with the tailwater elevation. For most design applications, the tailwater will either be above the crown of the outlet or can be considered to be between the crown and critical depth of the outlet. The tailwater may also occur between the critical depth and the invert of the outlet. However, the starting point for the hydraulic grade line determination should be either the design tailwater elevation or the average of critical depth and the height of the storm drain conduit, $(d_c + D)/2$, whichever is greater.

An exception to the above rule would be for a very large outfall with low tailwater where a water surface profile calculation would be appropriate to determine the location where the water surface will intersect the top of the barrel and full flow calculations can begin. In this case, the downstream water surface elevation would be based on critical depth or the design tailwater elevation, whichever was highest.

If the outfall channel is a river or stream, it may be necessary to consider the joint or coincidental probability of two hydrologic events occurring at the same time to adequately determine the elevation of the tailwater in the receiving stream. The relative independence of the discharge from the storm drainage system can be qualitatively evaluated by a comparison of the drainage area of the receiving stream to the area of the storm drainage system. For example, if the storm drainage system has a drainage area much smaller than that of the receiving stream, the peak discharge from the storm drainage system may be out of phase with the peak discharge from the receiving watershed. Table 3.2-12 provides a comparison of discharge frequencies for coincidental occurrence for the 2-, 5-, 10-, 25-, 50-, and 100-year design storms. This table can be used to establish an appropriate design tailwater elevation for a storm drainage system based on the expected coincident storm frequency on the outfall channel. For example, if the receiving stream has a drainage area of 200 acres and the storm drainage system has a drainage area of 2 acres, the ratio of receiving area to storm drainage area is 200 to 2 which equals 100 to 1. From Table 3.2-8 and considering a 25-year design storm occurring over both areas, the flow rate in the main stream will be equal to that of a five year storm when the drainage system flow rate reaches its 10-year peak flow at the outfall. Conversely, when the flow rate in the main channel reaches its 10-year peak flow rate, the flow rate from the storm drainage system will have fallen to the 5- year peak flow rate discharge. This is because the drainage areas are different sizes, and the time to peak for each drainage area is different.

Table 3.2-12 Frequencies for Coincidental Occurrences
(TxDOT, 2002)

Area ratio	2-year design		5-year design	
	Main Stream	**Tributary**	**Main Stream**	**Tributary**
10,000:1	1	2	1	5
	2	1	5	1
1,000:1	1	2	2	5
	2	1	5	2
100:1	2	2	2	5
	2	2	5	5
10:1	2	2	5	5
	2	2	5	5
1:1	2	2	5	5
	2	2	5	5
Area ratio	**10-year design**		**25-year design**	
	main stream	**tributary**	**main stream**	**tributary**
10,000:1	1	10	2	25
	10	1	25	2
1,000:1	2	10	5	25
	10	2	25	5
100:1	5	10	10	25
	10	5	25	10
10:1	10	10	10	25
	10	10	25	10
1:1	10	10	25	25
	10	10	25	25
Area ratio	**50-year design**		**100-year design**	
	main stream	**tributary**	**main stream**	**tributary**
10,000:1	2	50	2	100
	50	2	100	2
1,000:1	5	50	10	100
	50	5	100	10
100:1	10	50	25	100
	50	10	100	25
10:1	25	50	50	100
	50	25	100	50
1:1	50	50	100	100
	50	50	100	100

There may be instances in which an excessive tailwater causes flow to back up the storm drainage system and out of inlets and access holes, creating unexpected and perhaps hazardous flooding conditions. The potential for this should be considered. Flap gates placed at the outlet can sometimes alleviate this condition; otherwise, it may be necessary to isolate the storm drain from the outfall by use of a pump station.

Energy dissipation may be required to protect the storm drain outlet. Protection is usually required at the outlet to prevent erosion of the outfall bed and banks. Riprap aprons or energy dissipators should be provided if high velocities are expected. See Section 4.7 for guidance on design of Energy Dissipation Structures.

The **orientation of the outfall** is another important design consideration. Where practical, the outlet of the storm drain should be positioned in the outfall channel so that it is pointed in a downstream direction. This will reduce turbulence and the potential for excessive erosion. If the outfall structure can not be oriented in a downstream direction, the potential for outlet scour must be considered. For example, where a storm drain outfall discharges perpendicular to the direction of flow of the receiving channel, care must be taken to avoid erosion on the opposite channel bank. If erosion potential exists, a channel bank lining of riprap or other suitable material should be installed on the bank. Alternatively, an energy dissipator structure could be used at the storm drain outlet.

3.2.8.11 Energy Losses

Prior to computing the hydraulic grade line, all energy losses in pipe runs and junctions must be estimated. In addition to the principal energy involved in overcoming the friction in each conduit run, energy (or head) is required to overcome changes in momentum or turbulence at outlets, inlets, bends, transitions, junctions, and access holes. The following sections present relationships for estimating typical energy losses in storm drainage systems. The application of some of these relationships is included in the design example in section 8.2.8.13.

3.2.8.11.1 Pipe Friction Losses

The major loss in a storm drainage system is the friction or boundary shear loss. The head loss due to friction in a pipe is computed as follows:

$$H_f = S_f L \qquad \textbf{(3.2.27)}$$

where:

H_f = friction loss, ft

S_f = friction slope, ft/ft

L = length of pipe, ft

Section 3.2.8.6 gives the equation for computing the friction loss in pipes flowing full.

The friction slope in equation 3.2.27 is also the slope of the hydraulic gradient for a particular pipe run. As indicated by equation 3.2.27, the friction loss is simply the hydraulic gradient multiplied by the length of the run. Since this design procedure assumes steady uniform flow in open channel flow, the friction slope will match the pipe slope for part full flow. Pipe friction losses for full flow can be determined by the use of Equation 3.2.20.

3.2.8.11.2 Exit Losses

The exit loss from a storm drain outlet is a function of the change in velocity at the outlet of the pipe. For a sudden expansion such as at an endwall, the exit loss is:

$$\mathbf{H_o = 1.0\ [(V_o^2/2g) - (V_d^2/2g)]} \qquad \textbf{(3.2.28)}$$

where:

V_o = average outlet velocity

V_d = channel velocity downstream of outlet

Note that when $V_d = 0$, as in a reservoir, the exit loss is one velocity head. For part full flow where the pipe outlets in a channel with water moving in the same direction as the outlet water, the exit loss may be reduced to virtually zero.

3.2.8.11.3 Bend Losses

The bend loss coefficient for storm drain design is minor but can be estimated using the following formula (AASHTO, 1991):

$$\mathbf{h_b = 0.0033\ (\Delta)\ (V^2/2g)} \qquad \textbf{(3.2.29)}$$

where:

Δ = angle of curvature in degrees

3.2.8.11.4 Transition Losses

A transition is a location where a conduit or channel changes size. Typically, transitions should be avoided and access holes should be used when pipe size increases. However, sometimes transitions are unavoidable. Transitions include expansions, contractions, or both. In small storm drains, transitions may be confined within access holes. However, in larger storm drains or when a specific need arises, transitions may occur within pipe runs as illustrated in Figures 3.2-20 and 3.2-21.

Energy losses in expansions or contractions in non-pressure flow can be expressed in terms of the kinetic energy at the two ends. Contraction and expansion losses can be evaluated with equations 3.2-30 and 3.2-31 respectively.

$$\mathbf{H_c=K_c\,[V_1^2/(2g)- V_2^2/(2g)]} \qquad \textbf{(3.2.30)}$$

$$\mathbf{H_e=K_e\,[V_1^2/(2g)- V_2^2/(2g)]} \qquad \textbf{(3.2.31)}$$

where:

K_e = expansion coefficient

K_c = contraction coefficient (0.5 K_e)

V_1 = velocity upstream of transition

V_2 = velocity downstream of transition

g = acceleration due to gravity (32.2 ft/s_2)

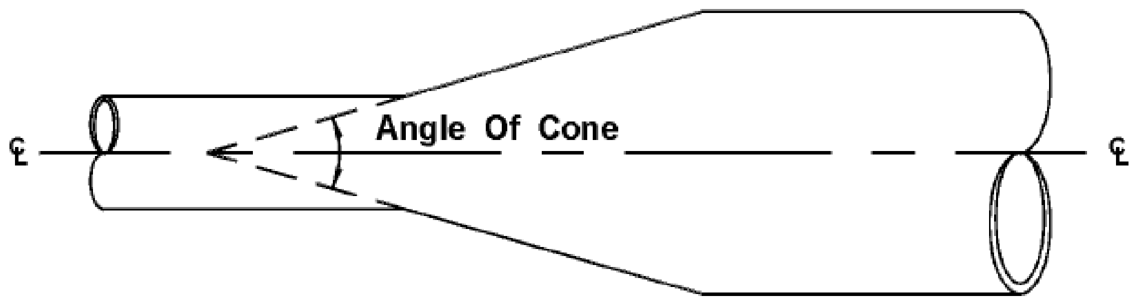


Figure 3.2-20 Angle of Cone for Pipe Diameter Changes

For gradual contractions, it has been observed that $K_c = 0.5\ K_e$. Typical values of K_e for gradual enlargements are tabulated in Table 3.2-13a. Typical values of K_c for sudden contractions are tabulated in Table 3.2-13b. The angle of the cone that forms the transition is defined in Figure 3.2-20.

Table 3.2-13a Typical Values for K_e for Gradual Enlargement of Pipes in Non-Pressure Flow

D_2/D_1	Angle of Cone						
	10°	20°	45°	60°	90°	120°	180°
1.5	0.17	0.40	1.06	1.21	1.14	1.07	1.00
3	0.17	0.40	0.86	1.02	1.06	1.04	1.00

D_2/D_1= Ratio of Diameter of larger pipe to smaller pipe (ASCE, 1992)

Table 3.2-13b Typical Values of K_c for Sudden Pipe Contractions

D_2/D_1	K_c
0.2	0.5
0.4	0.4
0.6	0.3
0.8	0.1
1	0

D_2/D_1= Ratio of Diameter of smaller pipe to larger pipe (ASCE, 1992)

Table 3.2-15 Values of K_e for Determining Loss of Head due to Gradual Enlargement in Pipes

For storm drain pipes functioning under pressure flow, the loss coefficients listed in Tables 3.2-14 and 3.2-15 can be used with Equation 3.2.32 for sudden and gradual expansions respectively. For sudden contractions in pipes with pressure flow, the loss coefficients listed in Table 3.2-16 can be used in conjunction with Equation 3.2.33 (ASCE, 1992).

$$H_e = K_e \left(V_1^2/2g\right) \qquad \textbf{(3.2.32)}$$

$$H_c = K_c \left(V_2^2/2g\right) \qquad \textbf{(3.2.33)}$$

where:

- Ke = expansion coefficient (Tables 3.2-14 and 3.2-15)
- Kc = contraction coefficient (Table 3.2.16)
- V1 = velocity upstream of transition
- V2 = velocity downstream of transition
- g = acceleration due to gravity 32.2 ft/s^2

Table 3.2-14 Values of K_e for Determining Loss of Head due to Sudden Enlargement in Pipes

D_2/D_1	Velocity, V_1, in feet Per Second												
	2.0	**3.0**	**4.0**	**5.0**	**6.0**	**7.0**	**8.0**	**10.0**	**12.0**	**15.0**	**20.0**	**30.0**	**40.0**
1.2	0.11	0.10	0.10	0.10	0.10	0.10	0.10	0.09	0.09	0.09	0.09	0.09	0.08
1.4	0.26	0.26	0.25	0.24	0.24	0.24	0.24	0.23	0.23	0.22	0.22	0.21	0.20
1.6	0.40	0.39	0.38	0.37	0.37	0.36	0.36	0.35	0.35	0.34	0.33	0.32	0.32
1.8	0.51	0.49	0.48	0.47	0.47	0.46	0.46	0.45	0.44	0.43	0.42	0.41	0.40
2.0	0.60	0.58	0.56	0.55	0.55	0.54	0.53	0.52	0.52	0.51	0.50	0.48	0.47
2.5	0.74	0.72	0.70	0.69	0.68	0.67	0.66	0.65	0.64	0.63	0.62	0.60	0.58
3.0	0.83	0.80	0.78	0.77	0.76	0.75	0.74	0.73	0.72	0.70	0.69	0.67	0.65
4.0	0.92	0.89	0.87	0.85	0.84	0.83	0.82	0.80	0.79	0.78	0.76	0.74	0.72
5.0	0.96	0.93	0.91	0.89	0.88	0.87	0.86	0.84	0.83	0.82	0.80	0.77	0.75
10.0	1.00	0.99	0.96	0.95	0.93	0.92	0.91	0.89	0.88	0.86	0.84	0.82	0.80
∞	1.00	1.00	0.98	0.96	0.95	0.94	0.93	0.91	0.90	0.88	0.86	0.83	0.81

D_2/D_1 = ratio of diameter of larger pipe to smaller pipe
V_1 = velocity in smaller pipe (upstream of transition)
(ASCE, 1992)

D_2/D_1	Angle of Cone										
	2º	6º	10º	15º	20º	25º	30º	35º	40º	50º	60º
1.1	0.01	0.01	0.03	0.05	0.10	0.13	0.16	0.18	0.19	0.21	0.23
1.2	0.02	0.02	0.04	0.09	0.16	0.21	0.25	0.29	0.31	0.35	0.37
1.4	0.02	0.03	0.06	0.12	0.23	0.30	0.36	0.41	0.44	0.50	0.53
1.6	0.03	0.04	0.07	0.14	0.26	0.35	0.42	0.47	0.51	0.57	0.61
1.8	0.03	0.04	0.07	0.15	0.28	0.37	0.44	0.50	0.54	0.61	0.65
2.0	0.03	0.04	0.07	0.16	0.29	0.38	0.46	0.52	0.56	0.63	0.68
2.5	0.03	0.04	0.08	0.16	0.30	0.39	0.48	0.54	0.58	0.65	0.70
3.0	0.03	0.04	0.08	0.16	0.31	0.40	0.48	0.55	0.59	0.66	0.71
∞	0.03	0.05	0.08	0.16	0.31	0.40	0.49	0.56	0.60	0.67	0.72

D_2/D_1 = ratio of diameter of larger pipe to smaller pipe
Angle of cone is the angle in degrees between the sides of the tapering section
(ASCE, 1992)

Table 3.2-16 Values of K_e for Determining Loss of Head due to Sudden Contraction

D_2/D_1	Velocity, V_1, in feet Per Second												
	2.0	3.0	4.0	5.0	6.0	7.0	8.0	10.0	12.0	15.0	20.0	30.0	40.0
1.1	0.03	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.05	0.05	0.06
1.2	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.08	0.08	0.08	0.09	0.11	0.11
1.4	0.17	0.17	0.17	0.17	0.17	0.17	0.17	0.18	0.18	0.18	0.18	0.19	0.20
1.6	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.25	0.25	0.24
1.8	0.34	0.34	0.34	0.34	0.34	0.34	0.33	0.33	0.32	0.32	0.32	0.29	0.27
2.0	0.38	0.38	0.37	0.37	0.37	0.37	0.36	0.36	0.35	0.34	0.33	0.31	0.29
2.2	0.40	0.40	0.40	0.39	0.39	0.39	0.39	0.38	0.37	0.37	0.35	0.33	0.30
2.5	0.42	0.42	0.42	0.41	0.41	0.41	0.40	0.40	0.39	0.38	0.37	0.34	0.31
3.0	0.44	0.44	0.44	0.43	0.43	0.43	0.42	0.42	0.41	0.40	0.39	0.36	0.33
4.0	0.47	0.46	0.46	0.46	0.45	0.45	0.45	0.44	0.43	0.42	0.41	0.37	0.34
5.0	0.48	0.48	0.47	0.47	0.47	0.46	0.46	0.45	0.45	0.44	0.42	0.38	0.35
10.0	0.49	0.48	0.48	0.48	0.48	0.47	0.47	0.46	0.46	0.45	0.43	0.40	0.36
∞	0.49	0.49	0.48	0.48	0.48	0.47	0.47	0.47	0.46	0.45	0.44	0.41	0.38

D_2/D_1 = ratio of diameter of larger pipe to smaller pipe
V_1 = velocity in smaller pipe (upstream of transition)
(ASCE, 1992)

3.2.8.11.5 Junction Losses

A pipe junction is the connection of a lateral pipe to a larger trunk pipe without the use of an access hole structure. The minor loss equation for a pipe junction is a form of the momentum equation as follows:

$$H_j=[((Q_o V_o) - (Q_i V_i) - (Q_l V_l \cos\theta)) / (0.5g(A_o-A_i))] + h_i - h_o \quad (3.2.34)$$

where:

H_j = junction loss (ft)

Q_o, Q_i, Q_l = outlet, inlet, and lateral flows respectively (ft^3/s)

V_o, V_i, V_l = outlet, inlet, and lateral velocities, respectively (ft/s)

h_o, h_i = outlet and inlet velocity heads (ft)

A_o, A_i = outlet and inlet cross-sectional areas (ft^2)

θ = angle between the inflow and outflow pipes (Figure 3.2-21)

3.2.8.11.6 Inlet and Access Hole Losses - Preliminary Estimate

The initial layout of a storm drain system begins at the upstream end of the system. The designer must estimate sizes and establish preliminary elevations as the design progresses downstream. An approximate method for estimating losses across an access hole is provided in this section. This is a preliminary estimate only and will not be used when the energy grade line calculations are made. Methods defined in Section 3.2.8.11.7 will be used to calculate the losses across an access hole when the energy grade line is being established.

The approximate method for computing losses at access holes or inlet structures involves multiplying the velocity head of the outflow pipe by a coefficient as represented in Equation 3.2.35. Applicable coefficients (K_{ah}) are tabulated in Table 3.2-17. This method can be used to estimate the initial pipe crown drop across an access hole or inlet structure to offset energy losses at the structure. The crown drop is then used to establish the appropriate pipe invert elevations. However, this method is used only in the preliminary design process and should not be used in the EGL calculations.

$$H_{ah}=K_{ah}\,(V_o^2/2g) \qquad (3.2.35)$$

Table 3.2-17 Head Loss Coefficients
(FHA, Revised 1993)

Structure Configuration	**K_{ah}**
Inlet-straight run	0.5
Inlet-angled through	
90º	1.5
60º	1.25
45º	1.1
22.5º	0.7
Manhole-straight run	0.15
Manhole-angled through	
90º	1
60º	0.85
45º	0.75
22.5º	0.45

3.2.8.11.7 Inlet and Access Hole Losses for EGL Calculations - Energy-Loss Methodology

Various methodologies have been advanced for evaluating losses at access holes and other flow junctions. The energy loss method presented in this section is based on laboratory research and does not apply when the inflow pipe invert is above the water level in the access hole.

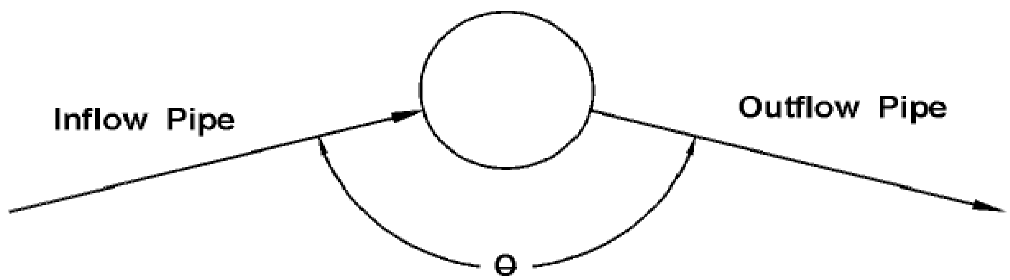


Figure 3.2-21. Head Loss Coefficients

The energy loss encountered going from one pipe to another through an access hole is commonly represented as being proportional to the velocity head of the outlet pipe. Using K to represent the constant of proportionality, the energy loss, H_{ah}, is approximated by Equation 3.2.36. Experimental studies have determined that the K value can be approximated by the relationship in Equation 3.2.37 when the inflow pipe invert is below the water level in the access hole.

$$H_{ah} = K (V_o^2/2g) \qquad (3.2.36)$$

$$K = K_O C_D C_d C_Q C_p C_B \qquad (3.2.37)$$

where:

- K = adjusted loss coefficient
- K_o = initial head loss coefficient based on relative access hole size
- C_D = correction factor for pipe diameter (pressure flow only)
- C_d = correction factor for flow depth
- C_Q = correction factor for relative flow
- C_p = correction factor for plunging flow
- C_B = correction factor for benching
- V_o = velocity of outlet pipe

For cases where the inflow pipe invert is above the access hole water level, the outflow pipe will function as a culvert, and the access hole loss and the access hole HGL can be computed using procedures found in *Hydraulic Design of Highway Culverts* (HDS-5, 1985). If the outflow pipe is flowing full or partially full under outlet control, the access hole loss (due to flow contraction into the outflow pipe) can be computed by setting K in Equation 3.2.36 to Ke as reported in Table 3.2-

18. If the outflow pipe is flowing under inlet control, the water depth in the access hole should be computed using the inlet control nomographs in HDS- 5 (for example see Figure 4.3-2a in Section 4.2).

The initial head loss coefficient, K_o in Equation 3.2.37, is estimated as a function of the **relative access hole** size and the angle of deflection between the inflow and outflow pipes as represented in Equation 3.2.38. This deflection angle is represented in Figure 3.2-21.

$$\mathbf{K_o = 0.1\,(b/D_o)(1-\sin\theta) + 1.4(b/D_o)^{0.15}\sin\theta} \qquad \textbf{(3.2.38)}$$

where:

θ = angle between the inflow and outflow pipes (Figure 3.2-21)

b = access hole or junction diameter

D_o = outlet pipe diameter

A change in head loss due to differences in **pipe diameter** is only significant in pressure flow situations when the depth in the access hole to outlet pipe diameter ratio, d_{aho}/D_o, is greater than 3.2. In these cases a correction factor for pipe diameter, C_D, is computed using Equation 3.2.39. Otherwise C_D is set equal to 1.

$$\mathbf{C_D=(D_o/D_i)^3} \qquad \textbf{(3.2.39)}$$

where:

D_o = outgoing pipe diameter

D_i = inflowing pipe diameter

Table 3.2-18 Coefficients for Culverts; Outlet Control, Full, or Partly Full

Type of Structure and Design of Entrance	Coefficient K_e
Pipe, Concrete	
Projecting from fill, socket end (groove-end)	0.2
Projecting from fill, sq. cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove-end)	0.2
Square-edge	0.5
Rounded (radius = 1/12 D)	0.2
Mitered to conform to fill slope	0.7
*End-section conforming to fill slope	0.5
Beveled edges, 33.7º or 45º levels	0.2
Side-or slope-tapered inlet	0.2
Pipe, or Pipe-Arch, Corrugated Metal	
Project from fill (no headwall)	0.9
Headwall or headwall and wingwalls square-edge . . .	0.5
Mitered to conform to fill slope, paved or unpaved slope	0.7
*End-section conforming to fill slope	0.5
Beveled edges, 33.7º or 45º bevels	0.2
Side-or slope-tapered inlet	0.2
Box, Reinforced Concrete	
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of 1/12 barrel dimension, or beveled edges on 3 sides	0.2
Wingwalls at 30º to 75º to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of 1/2 barrel dimension, or beveled top edge	0.2
Wingwall at 10º to 25º to barrel	
Square-edged at crown	0.05
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7
Side-or slope-tapered inlet	0.2

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

(Source: Reference HDS No.5, 1985)

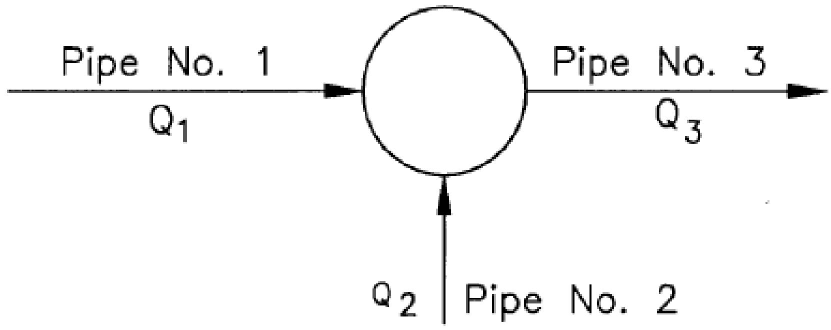


Figure 3.2-22 Relative flow effect

The correction factor for **flow depth**, C_d, is significant only in cases of free surface flow or low pressures, when the d_{aho}/D_o ratio is less than 3.2. In cases where this ratio is greater than 3.2, C_d is set equal to 1. To determine the applicability of this factor, the water depth in the access hole is approximated as the level of the hydraulic grade line at the upstream end of the outlet pipe. The correction factor is calculated using Equation 3.2.38.

$$\mathbf{C_D = 0.5(d_{aho}/D_o)^{0.6}} \qquad \textbf{(3.2.40)}$$

where:

d_{aho} = water depth in access hole above the outlet pipe invert

D_o = outlet pipe diameter

The correction factor for **relative flow**, C_Q, is a function of the angle of the incoming flow as well as the percentage of flow coming in through the pipe of interest versus other incoming pipes. It is computed using Equation 3.2.39. The correction factor is only applied to situations where there are 3 or more pipes entering the structure at approximately the same elevation. Otherwise, the value of C_Q is equal to 1.0.

$$\mathbf{C_Q = (1- 2\sin\theta)\,[1-(Q_i / Q_o)]^{0.75} + 1} \qquad \textbf{(3.2.41)}$$

where:

C_Q = correction factor for relative flow

θ = the angle between the inflow and outflow pipes (Figure 3.2-22)

Q_i = flow in the inflow pipe

Q_o = flow in the outflow pipe

As can be seen from Equation 3.2.41, C_Q is a function of the angle of the incoming flow as well as the ratio of inflow coming through the pipe of interest and the total flow out of the structure. To illustrate this effect, consider the access hole shown in Figure 3.2-23 and assume the following two cases to determine the correction factor of pipe number 2 entering the access hole. For each of the two cases, the angle between the inflow pipe number 1 and the outflow pipe, θ, is 180^o.

Case 1:

$Q_1 = 3\ ft^3/s$

$Q_2 = 1\ ft^3/s$

$Q_3 = 4\ ft^3/s$

Using Equation 3.2.39,

$C_Q = (1 - 2 \sin \theta)(1 - Q_i/Q_o)^{0.75} + 1$

$C_Q = (1 - 2 \sin 180^o)(1 - 3/4)^{0.75} + 1$

$C_Q = 1.35$

Case 2:

$Q_1 = 1.0\ ft^3/s$

$Q_2 = 3.0\ ft^3/s$

$Q_3 = 4.0\ ft^3/s$

Using Equation 3.2.39,

$C_Q = (1 - 2 \sin \theta)(1 - Q_i/Q_o)^{0.75} + 1$

$C_Q = (1 - 2 \sin 180^o)(1 - 1/4)^{0.75} + 1$

$C_Q = 1.81$

The correction factor for **plunging flow**, C_p, is calculated using Equation 3.2.42. This correction factor corresponds to the effect another inflow pipe, plunging into the access hole, has on the inflow pipe for which the head loss is being calculated. Using the notations in Figure 3.2-23, C_p is calculated for pipe #1 when pipe #2 discharges plunging flow. The correction factor is only applied when $h > d_{aho}$. Additionally, the correction factor is only applied when a higher elevation flow plunges into an access hole that has both an inflow line and an outflow in the bottom of the access hole. Otherwise, the value of C_p is equal to 1.0. Flows from a grate inlet or a curb opening inlet are considered to be plunging flow and the losses would be computed using Equation 3.2.40.

$$C_p = 1 + 0.2(h/D_o)\,[(h - d_{aho})/D_o] \qquad \textbf{(3.2.42)}$$

where:

- C_p = correction for plunging flow
- h = vertical distance of plunging flow from the flow line of the higher elevation inlet pipe to the center of the outflow pipe
- D_o = outlet pipe diameter
- d_{aho} = water depth in access hole relative to the outlet pipe invert

The correction for **benching** in the access hole, C_B, is obtained from Table 3.2-19. Figure 3.2-23 illustrates benching methods listed in Table 3.2-19. Benching tends to direct flow through the access hole, resulting in a reduction in head loss. For flow depths between the submerged and unsubmerged conditions, a linear interpolation is performed.

Table 3.2-19 Correction for Benching
(HEC 22, 2001)

Bench Type	Correction Factors, C_B	
	Submerged*	Unsubmerged**
Flat or Depressed Floor	1.00	1.00
Half Bench	0.95	0.15
Full Bench	0.75	0.07

*pressure flow, $d_{aho}/D_o \geq 3.2$

**free surface flow, $d_{aho}/D_o \leq 3.2$

In summary, to estimate the head loss through an access hole from the outflow pipe to a particular inflow pipe using the energy-loss method, multiply the above correction factors together to get the head loss coefficient, K. This coefficient is then multiplied by the velocity head in the outflow pipe to estimate the minor loss for the connection.

3.2.8.11.8 Composite Energy Loss Method

The Energy Loss Method described in Section 3.2.8.11.7 resulted from preliminary experimental and analytical techniques that focused on relatively simple access hole layout and a small number of inflow pipes. A more suitable method is available to analyze complex access holes that have, for example, many inflow pipes. This complex method, referred to as the Composite Energy Loss Method, is implemented in the FHWA storm drain analysis and design package HYDRA (GKY, 1994). Details on the method are described in the HYDRA program technical documentation and the associated research report (Chang, et. al., 1994).

This complex minor loss computation approach focuses on the calculation of the energy loss from the inflow pipes to the outflow pipe (Chang, et. al., 1994). The methodology can be applied by determining the estimated energy loss through an access hole given a set of physical and hydraulic parameters. Computation of the energy loss allows determination and analysis of the energy gradeline and hydraulic gradeline in pipes upstream of the access hole. This methodology only applies to subcritical flow in pipes.

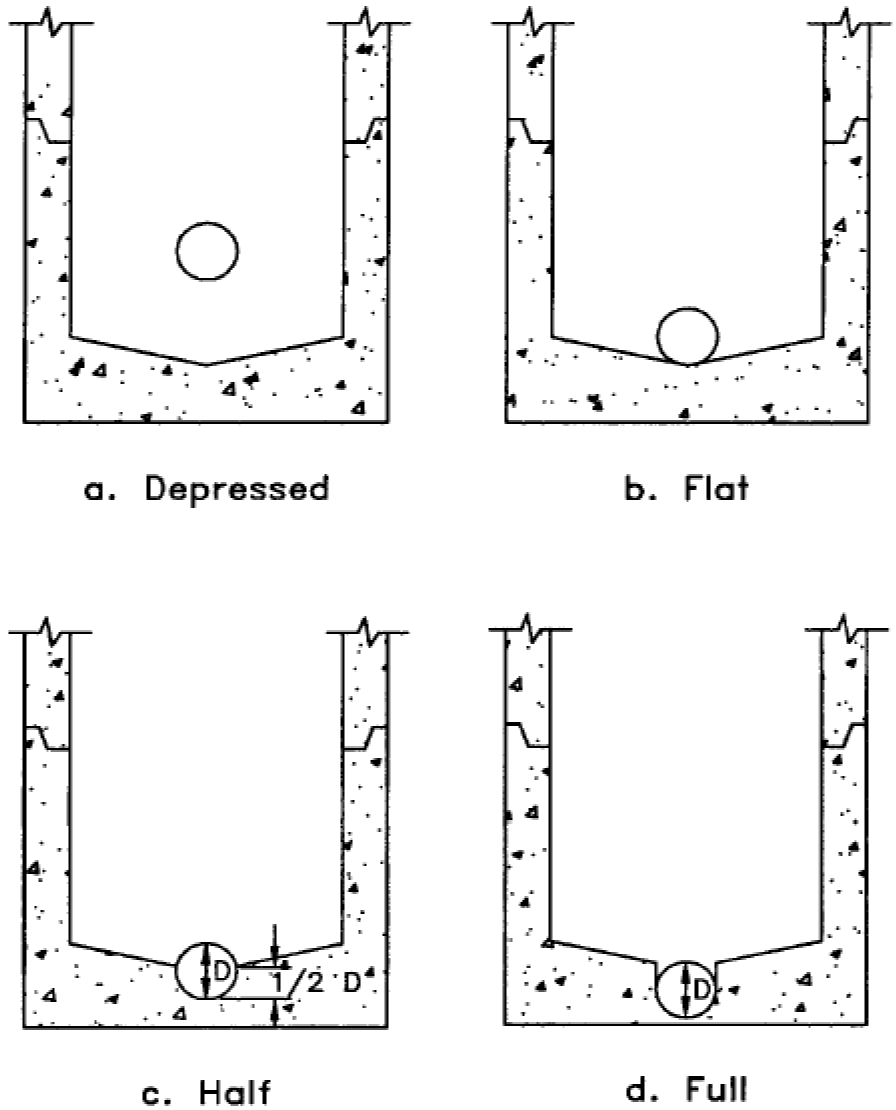


Figure 3.2-23 Access to Benching Methods

3.2.8.12 Preliminary Design Procedure

The preliminary design of storm drains can be accomplished by using the following steps and the storm drain computation sheet provided in Figure 3.2-25. This procedure assumes that each storm drain will be initially designed to flow full under gravity conditions. The designer must recognize that when the steps in this section are complete, the design is only preliminary. Final design is accomplished after the energy grade line and hydraulic grade line computations have been completed (See Section 3.2.8.9).

Step 1 Prepare a working plan layout and profile of the storm drainage system establishing the following design information:

a. Location of storm drains.
b. Direction of flow.
c. Location of access holes and other structures.
d. Number or label assigned to each structure.
e. Location of all existing utilities (water, sewer, gas, underground cables, etc.).

Step 2 Determine the following hydrologic parameters for the drainage areas tributary to each inlet to the storm drainage system:

a. Drainage areas.
b. Runoff coefficients.
c. Travel time

Step 3 Using the information generated in Steps 1 and 2, complete the following information on the design form for each run of pipe starting with the upstream most storm drain run:

a. "From" and "To" stations, Columns 1 and 2b, "Length" of run, Column 3
b. "Length" of run, Column 3
c. "Inc." drainage area, Column 4

   The incremental drainage area tributary to the inlet at the upstream end of the storm drain run under consideration.

d. "C," Column 6

   The runoff coefficient for the drainage area tributary to the inlet at the upstream end of the storm drain run under consideration. In some cases a composite runoff coefficient will need to be computed.

e. "Inlet" time of concentration, Column 9

   The time required for water to travel from the hydraulically most distant point of the drainage area to the inlet at the upstream end of the storm drain run under consideration.

f. "System" time of concentration, Column 10

   The time for water to travel from the most remote point in the storm drainage system to the upstream end of the storm drain run under consideration. For the upstream most storm drain run this value will be the same as the value in Column 9. For all other pipe runs this value is computed by adding the "System" time of concentration (Column 10) and the "Section" time of concentration (Column 17) from the previous run together to get the system time of concentration at the upstream end of the section under consideration (See Section 3.2.8.3 for a general discussion of times of concentration).

Step 4 Using the information from Step 3, compute the following:

a. "TOTAL" area, Column 5

   Add the incremental area in Column 4 to the previous sections total area and place this value in Column 5.

b. "INC." area x "C," Column 7

Multiply the drainage area in Column 4 by the runoff coefficient in Column 6. Put the product, CA, in Column 7.

c. "TOTAL" area x "C," Column 8

Add the value in Column 7 to the value in Column 8 for the previous storm drain run and put this value in Column 8.

d. "I," Column 11

Using the larger of the two times of concentration in Columns 9 and 10, and an Intensity-Duration-Frequency (IDF) curve, determine the rainfall intensity, I, and place this value in Column 11.

e. "TOTAL Q," Column 12

Calculate the discharge as the product of Columns 8 and 11. Place this value in Column 12.

f. "SLOPE," Column 21

Place the pipe slope value in Column 21. The pipe slope will be approximately the slope of the finished roadway. The slope can be modified as needed.

g. "PIPE DIA.," Column 13

Size the pipe using relationships and charts presented in Section 3.2.8.6 to convey the discharge by varying the slope and pipe size as necessary. The storm drain should be sized as close as possible to a full gravity flow. Since most calculated sizes will not be available, a nominal size will be used. The designer will decide whether to go to the next larger size and have part full flow or whether to go to the next smaller size and have pressure flow.

h. "CAPACITY FULL," Column 14

Compute the full flow capacity of the selected pipe using Equation 3.2.18 and put this information in Column 14.

i. "VELOCITIES," Columns 15 and 16

Compute the full flow and design flow velocities (if different) in the conduit and place these values in Columns 15 and 16. If the pipe is flowing full, the velocities can be determined from V = Q/A, Equations 3.2.17 and 3.2.18. If the pipe is not flowing full, the velocity can be determined from Figure 3.2-18a.

j. "SECTION TIME," Column 17

Calculate the travel time in the pipe section by dividing the pipe length (Column 3) by the design flow velocity (Column 16). Place this value in Column 17.

k. "CROWN DROP," Column 20

Calculate an approximate crown drop at the structure to off-set potential structure energy losses using Equation 3.2.33 introduced in Section 3.2.8.11.6. Place this value in Column 20.

l. "INVERT ELEV.," Columns 18 and 19

Compute the pipe inverts at the upper (U/S) and lower (D/S) ends of this section of pipe, including any pipe size changes that occurred along the section.

Step 5 Repeat steps 3 and 4 for all pipe runs to the storm drain outlet. Use equations and nomographs to accomplish the design effort.

Step 6 Check the design by calculating the energy grade line and hydraulic grade line as described in Section 3.2.8.9.

PRELIMINARY STORM DRAIN COMPUTATION SHEET

COMPUTED BY ______ DATE ______ ROUTE ______________
CHECKED BY ______ DATE ______ SECTION ______________
PAGE ______ OF ______ COUNTY ______________

STR. ID		LENGTH	DRAINAGE AREA		RUNOFF COEFF. "C"	"AREA" X "C"		TIME OF CONCENTRATION		RAIN "I"	RUNOFF "Q"	PIPE DIA.	Q FULL	VELOCITY		SEC TIME	INVERT ELEV.		CROWN DROP	SLOPE
FROM	TO	( )	INC ( )	TOTAL ( )		INC. ( )	TOTAL ( )	INLET (min)	SYSTEM (min)	(/hr)	(³ /s)	( )	(³/s)	FULL (/s)	DESIGN (/s)	(min)	U/S ( )	D/S ( )	( )	(/)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)

Figure 3.2-24 Preliminary Storm Drain Computation Sheet

3.2.8.13 Energy Grade Line Evaluation Procedure

This section presents a step-by-step procedure for manual calculation of the energy grade line (EGL) and the hydraulic grade line (HGL) using the energy loss method. For many storm drainage systems, computer methods such as HYDRAIN (FHWA, 1994) are an efficient means of evaluating the EGL and the HGL. However, it is important that the designer understand the analysis process so that he can better interpret the output from computer generated storm drain designs.

Figure 3.2-25 provides a sketch illustrating use of the two grade lines in developing a storm drainage system. The following step-by-step procedure can be used to manually compute the EGL and HGL. The computation tables in Figure 3.2-26 and Figure 3.2-27 can be used to document the procedure outlined below.

Before outlining the computational steps in the procedure, a comment relative to the organization of data on the form is appropriate. In general, a line will contain the information on a specific structure and the line downstream from the structure. As the table is started, the first two lines may be unique. The first line will contain information about the outlet conditions. This may be a pool elevation or information on a known downstream system. The second line will be used to define the conditions right at the end of the last conduit. Following these first two lines the procedure becomes more general. A single line on the computation sheet is used for each junction or structure and its associated outlet pipe. For example, data for the first structure immediately upstream of the outflow pipe and the outflow pipe would be tabulated in the third full line of the computation sheet (lines may be skipped on the form for clarity). Table A (Figure 3.2-26) is used to calculate the HGL and EGL elevations while table B (Figure 3.2-27) is used to calculate the pipe losses and structure losses. Values obtained in table B are transferred to table A for use during the design procedure. In the description of the computation procedures, a column number will be followed by a letter A or B to indicate the appropriate table to be used.

EGL computations begin at the outfall and are worked upstream taking each junction into consideration. Many storm drain systems are designed to function in a subcritical flow regime. In subcritical flow, pipe and access hole losses are summed to determine the upstream EGL levels. If supercritical flow occurs, pipe and access losses are not carried upstream. When a storm drain section is identified as being supercritical, the designer should advance to the next upstream pipe section to determine its flow regime. This process continues until the storm drain system returns to a subcritical flow regime.

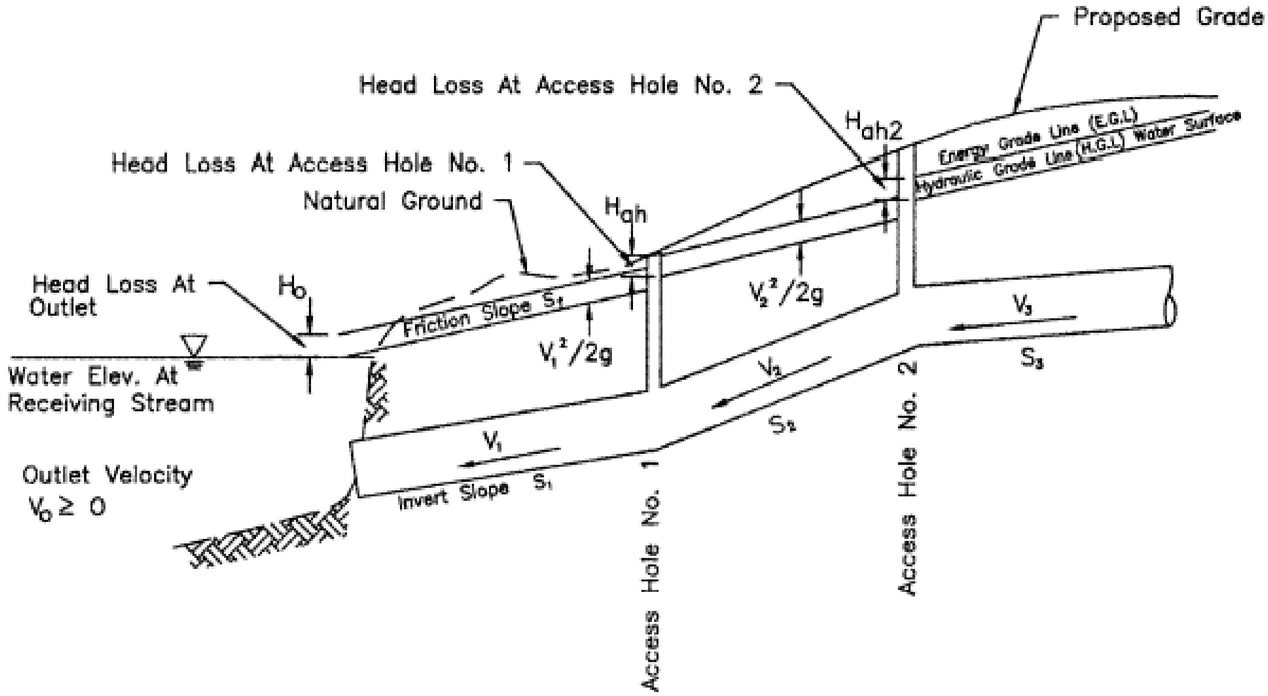


Figure 3.2-25 Energy and Hydraulic Grade Line Illustration

The EGL computational procedure follows:

Step 1 The first line of Table A includes information on the system beyond the end of the conduit system. Define this as the stream, pool, existing system, etc. in column 1A. Determine the EGL and HGL for the downstream receiving system. If this is a natural body of water, the HGL will be at the water surface. The EGL will also be at the water surface if no velocity is assumed or will be a velocity head above the HGL if there is a velocity in the water body. If the new system is being connected to an existing storm drain system, the EGL and the HGL will be that of the receiving system. Enter the HGL in Column 14A and the EGL in Column 10A of the first line on the computation sheet.

Step 2 Identify the structure number at the outfall (this may be just the end of the conduit, but it needs a structure number), the top of conduit (TOC) elevation at the outfall end, and the surface elevation at the outfall end of the conduit. Place these values in Columns 1A, 15A, and 16A respectively. Also add the structure number in Col.1B.

Step 3 Determine the EGL just upstream of the structure identified in Step 2. Several different cases exist as defined below when the conduit is flowing full:

Case 1: If the TW at the conduit outlet is greater than $(d_c + D)/2$, the EGL will be the TW elevation plus the velocity head for the conduit flow conditions.

Case 2: If the TW at the conduit outlet is less than $(d_c + D)/2$, the EGL will be the HGL plus the velocity head for the conduit flow conditions. The equivalent hydraulic grade line, EHGL, will be the invert plus $(d_c + D)/2$.

ENERGY GRADE LINE COMPUTATION SHEET - TABLE A

COMPUTED BY ________ DATE ________ ROUTE ________________
CHECKED BY ________ DATE ________ SECTION ________________
PAGE ________ OF ________ COUNTY ________________
INITIAL TAILWATER ELEV. ________________

Str. ID (1)	D ( ) (2)	Q (³/s) (3)	L ( ) (4)	V (/s) (5)	d ( ) (6a)	d_c ( ) (6b)	$V^2/2g$ ( ) (7)	S_f (/) (8)	Total Pipe Loss (table B) ( ) (9)	EGL_o ( ) (10)	K (table B) (11)	$K(V^2/2g)$ ( ) (12)	EGL_i ( ) (13)	HGL ( ) (14)	U/S TOC ( ) (15)	Surf. Elev. ( ) (16)

Figure 3.2-26 Energy Grade Line Computation Sheet - Table A

ENERGY GRADE LINE COMPUTATION SHEET - TABLE B

COMPUTED BY ______ DATE ______ ROUTE ______
CHECKED BY ______ DATE ______ SECTION ______
PAGE ______ OF ______ COUNTY ______

Str. ID	Pipe Losses ( )						Structure Losses ( )							
	H_f	h_b	H_c	H_e	H_j	Total	d_{aho}	K_o	C_D	C_d	C_Q	C_p	C_B	K
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)

Figure 3.2-27 Energy Grade Line Computation Sheet - Table B

The velocity head needed in either Case 1 or 2 will be calculated in the next steps, so it may be helpful to complete Step 4 and work Step 5 to the point where velocity head (Col. 7A) is determined and then come back and finish this step. Put the EGL in Column 13A.

Note: The values for d_c for circular pipes can be determined from Figure 3.2-18b. Charts for other conduits or other geometric shapes can be found in *Hydraulic Design of Highway Culverts*, HDS-5, and cannot be greater than the height of the conduit.

Step 4 Identify the structure ID for the junction immediately upstream of the outflow conduit (for the first conduit) or immediately upstream of the last structure (if working with subsequent lines) and enter this value in Columns 1A and 1B of the next line on the computation sheets. Enter the conduit diameter (D) in column 2A, the design discharge (Q) in Column 3A, and the conduit length (L) in Column 4A.

Step 5 If the barrel flows full, enter the full flow velocity from continuity in Column 5A and the velocity head ($V^2/2g$) in column 7A. Put "full" in Column 6a and not applicable (n/a) in Column 6b of Table A. Continue with Step 6. If the barrel flows only partially full, continue with Step 5A.

Note: If the pipe is flowing full because of high tailwater or because the pipe has reached its capacity for the existing conditions, the velocity will be computed based on continuity using the design flow and the full cross sectional area. Do not use the full flow velocity determined in Column 15 of the Preliminary Storm Drain Computation Form for part-full flow conditions. For part-full conditions discussed in Step 5, the calculations in the preliminary form may be helpful. Actual flow velocities need to be used in the EGL/HGL calculations.

Step 5A Part full flow: Using the hydraulic elements graph in Figure 3.2-18a with the ratio of part full to full flow (values from the preliminary storm drain computation form), compute the depth and velocity of flow in the conduit. Enter these values in Column 6a and 5 respectively of Table A. Compute the velocity head (V2/2g) and place in Column 7A.

Step 5B Compute critical depth for the conduit using Figure 3.2-18b. If the conduit is not circular, see HDS-5 for additional charts. Enter this value in Column 6b of Table A.

Step 5C Compare the flow depth in Column 6a (Table A) with the critical depth in Column 6b (Table A) to determine the flow state in the conduit. If the flow depth in Column 6a is greater than the critical depth in Column 6b, the flow is subcritical, continue with Step 6. If the flow depth in Column 6a is less than or equal to the critical depth in Column 6b, the flow is supercritical, continue with Step 5D. In either case, remember that the EGL must be higher upstream for flow to occur. If after checking for super critical flow in the upstream section of pipe, assure that the EGL is higher in the pipe than in the structure.

Step 5D Pipe losses in a supercritical pipe section are not carried upstream. Therefore, enter a zero (0) in Column 7B for this structure.

Step 5E Enter the structure ID for the next upstream structure on the next line in Columns 1A and 1B. Enter the pipe diameter (D), discharge (Q), and conduit length (L) in Columns 2A, 3A, and 4A respectively of the same line.

Note: After a downstream pipe has been determined to flow in supercritical flow, it is necessary to check each succeeding upstream pipe for the type of flow that exists. This is done by calculating normal depth and critical depth for each pipe. If normal depth is less than the diameter of the pipe, the flow will be open channel flow and the critical depth calculation can be used to determine whether the flow is sub or supercritical. If the flow line elevation through an access hole drops enough that the invert of the upstream pipe is not inundated by the flow in the downstream pipe, the designer goes back to Step1A and begins a new design as if the downstream section did not exist.

Step 5F Compute normal depth for the conduit using Figure 3.2-18a and critical depth using Figure 3.2-18b. If the conduit is not circular see HDS-5 for additional charts. Enter these values in Columns 6A and 6b of Table A.

Step 5G If the pipe barrel flows full, enter the full flow velocity from continuity in Column 5A and the velocity head ($V^2/2g$) in Column 7A. Go to Step 3, Case 2 to determine the EGL at the outlet end of the pipe. Put this value in Column 10A and go to Step 6. For part full flow, continue with Step 5H.

Step 5H Part full flow: Compute the velocity of flow in the conduit and enter this value in Column 5A. Compute the velocity head ($V^2/2g$) and place in Column 7A.

Step 5I Compare the flow depth in Column 6a with the critical depth in Column 6b to determine the flow state in the conduit. If the flow depth in Column 6a is greater than the critical depth in Column 6b, the flow is subcritical, continue with Step 5J. If the flow depth in Column 6a is less than or equal to the critical depth in Column 6b, the flow is supercritical, continue with Step 5K.

Step 5J Subcritical flow upstream: Compute EGL_o at the outlet of the previous structure as the outlet invert plus the sum of the outlet pipe flow depth and the velocity head. Place this value in Column 10A of the appropriate structure and go to Step 9.

Step 5K Supercritical flow upstream: Access hole losses do not apply when the flow in two (2) successive pipes is supercritical. Place zeros (0) in Columns 11A, 12A, and 15B of the intermediate structure (previous line). The HGL at the structure is equal to the pipe invert elevation plus the flow depth. Check the invert elevations and the flow depths both upstream and downstream of the structure to determine where the highest HGL exists. The highest value should be placed in Column 14A of the previous structure line. Perform Steps 20 and 21 and then repeat Steps 5E through 5K until the flow regime returns to subcritical. If the next upstream structure is end-of-line, skip to step 10b then perform Steps 20, 21, and 24.

Step 6 Compute the friction slope (S_f) for the pipe using Equation 3.2.19 divided by L[$S_f = H_f/L = [185\ n^2\ (V^2/2g)]/D^{4/3}$] for a pipe flowing full. Enter this value in Column 8A of the current line. If full flow does not exist, set the friction slope equal to the pipe slope.

Step 7 Compute the friction loss (H_f) by multiplying the length (L) in Column 4A by the friction slope (S_f) in Column 8A and enter this value in Column 2B. Compute other losses along the pipe run such as bend losses (h_b), transition contraction (H_c) and expansion (H_e) losses, and junction losses (H_j) using Equations 3.2.27 through 3.2.32 and place the values in Columns 3B, 4B, 5B, and 6B, respectively. Add the values in 2B, 3B, 4B, 5B, and 6B and place the total in Column 7B and 9A.

Step 8 Compute the energy grade line value at the outlet of the structure (EGL_o) as the EGL_i elevation from the previous structure (Column 13A) plus the total pipe losses (Column 9A). Enter the EGL_o in Column 10A.

Step 9 Estimate the depth of water in the access hole (estimated as the depth from the outlet pipe invert to the hydraulic grade line in the pipe at the outlet). Computed as EGL_o (Column 10A) minus the pipe velocity head in Column 7A minus the pipe invert elevation (from the preliminary storm drain computation form). Enter this value in Column 8B. If supercritical flow exists in this structure, leave this value blank and skip to Step 5E.

Step 10 If the inflow storm drain invert is submerged by the water level in the access hole, compute access hole losses using Equations 3.2.36 and 3.2.37. Start by computing the initial structure head loss coefficient, K_o, based on relative access hole size. Enter this value in Column 9B. Continue with Step 11. If the inflow storm drain invert is not submerged by the water level in the access hole, compute the head in the access hole using culvert techniques from HDS-5 as follows:

a. If the structure outflow pipe is flowing full or partially full under outlet control, compute the access hole loss by setting K in Equation 3.2.35 to K_e as reported in Table 3.2-16. Enter this value in Column 15B and 11A, continue with Step 17. Add a note on Table A indicating that this is a drop structure.

b. If the outflow pipe functions under inlet control, compute the depth in the access hole (HGL) using Figure 4.2-2(a). If the storm conduit shape and material is other than circular concrete, select the appropriate inlet control nomograph from HDS-5. Add these values to the access hole invert to determine the HGL. Since the velocity in the access hole is negligible, the EGL and HGL are the same. Enter HGL in Col.14A and EGL in Col.13A. Add a note on Table A indicating that this is a drop structure. Go to Step 20.

Step 11 Using Equation 3.2.39 compute the correction factor for pipe diameter, C_D, and enter this value in Column 10B. Note, this factor is only significant in cases where the d_{aho}/D_o ratio is greater than 3.2.

Step 12 Using Equation 3.2.40 compute the correction factor for flow depth, C_D, and enter this value in Column 11B. Note, this factor is only significant in cases where the d_{aho}/D_o ratio is less than 3.2.

Step 13 Using Equation 3.2.41, compute the correction factor for relative flow, CQ, and enter this value in Column 12B. This factor = 1.0 if there are less than 3 pipes at the structure.

Step 14 Using Equation 3.2.42, compute the correction factor for plunging flow, C_p, and enter this value in Column 13B. This factor = 1.0 if there is no plunging flow. This correction factor is only applied when $h>d_{aho}$.

Step 15 Enter in Column 14B the correction factor for benching, C_B, as determined from Table 3.2-18. Linear interpolation between the two columns of values will most likely be necessary.

Step 16 Using Equation 3.2.37, compute the value of K and enter this value in Column 15B and 11A.

Step 17 Compute the total access hole loss, H_{ah}, by multiplying the K value in Column 11A by the velocity head in Column 7A. Enter this value in Column 12A.

Step 18 Compute EGL_i at the structure by adding the structure losses in Column 12A to the EGL_o value in Column 10A. Enter this value in Column 13A.

Step 19 Compute the hydraulic grade line (HGL) at the structure by subtracting the velocity head in Column 7A from the EGL_i value in Column 13A. Enter this value in Column 14A.

Step 20 Determine the top of conduit (TOC) value for the inflow pipe (using information from the storm drain computation sheet) and enter this value in Column 15A.

Step 21 Enter the ground surface, top of grate elevation or other high water limits at the structure in Column 16A. If the HGL value in Column 14A exceeds the limiting elevation, design modifications will be required.

Step 22 Enter the structure ID for the next upstream structure in Column 1A and 1B of the next line. When starting a new branch line, skip to Step 24.

Step 23 Continue to determine the EGL through the system by repeating Steps 4 through 23. (Begin with Step 2 if working with a drop structure. This begins the design process again as if there were no system down stream from the drop structure).

Step 24 When starting a new branch line, enter the structure ID for the branch structure in Column 1A and 1B of a new line. Transfer the values from Columns 2A through 10A and 2B to 7B associated with this structure on the main branch run to the corresponding columns for the branch line. If flow in the main storm drain at the branch point is subcritical, continue with Step 9; if supercritical, continue with Step 5E.

3.2.8.14 Storm Drain Design Example

The following storm drain design example illustrates the application of the design procedures outlined in Sections 3.2.8.11, 3.2.8.12 and 3.2.8.13.

Example of Preliminary Storm Drain Design

Given: The roadway plan and section illustrated in Figure 3.2-28, inlet drainage area information in Table 3.2-20, and duration intensity information in Table 3.2-21. All grates are type P 50 x 100, all piping is reinforced concrete pipe (RCP) with a Manning's n value of 0.013, and the minimum design pipe diameter = 18 in for maintenance purposes.

Find:

(1) Using the procedures outlined in Section 3.2.8.11 determine appropriate pipe sizes and inverts for the system illustrated in Figure 3.2-29.

(2) Evaluate the HGL for the system configuration determined in part (1) using the procedure outlined in Section 3.2.8.12.

Solution:

(1) Preliminary Storm Drain Design

Step 1. Figure 3.2-29 illustrates the proposed system layout including location of storm drains, access holes, and other structures. All structures have been numbered for reference. Figure 3.2-30 (a) and (b) illustrate the corresponding storm drain profiles.

Step 2. Drainage areas, runoff coefficients, and times of concentration are tabulated in Figure 3.2-31. Example problems documenting the computation of these values are included in this chapter.

Starting at the upstream end of a conduit run, Steps 3 and 4 from Section 3.2.8.11 are completed for each storm drain pipe. A summary tabulation of the computational process is provided in Figure 3.2-31. The column by column computations for each section of conduit follow:

Table 3.2-20 Drainage Area Information for Design Example			
Inlet No.	**Drainage Area (ac)**	**"C"**	**Time of Concentration (min)**
40	0.64	0.73	3
41	0.35	0.73	2
42	0.32	0.73	2
43	--	--	--
44			

Table 3.2-21 Intensity/Duration Data Design Example									
Time (min)	5	10	15	20	30	40	50	60	120
Intensity (in/hr)	7.1	5.9	5.1	4.5	3.5	3	2.6	2.4	1.4

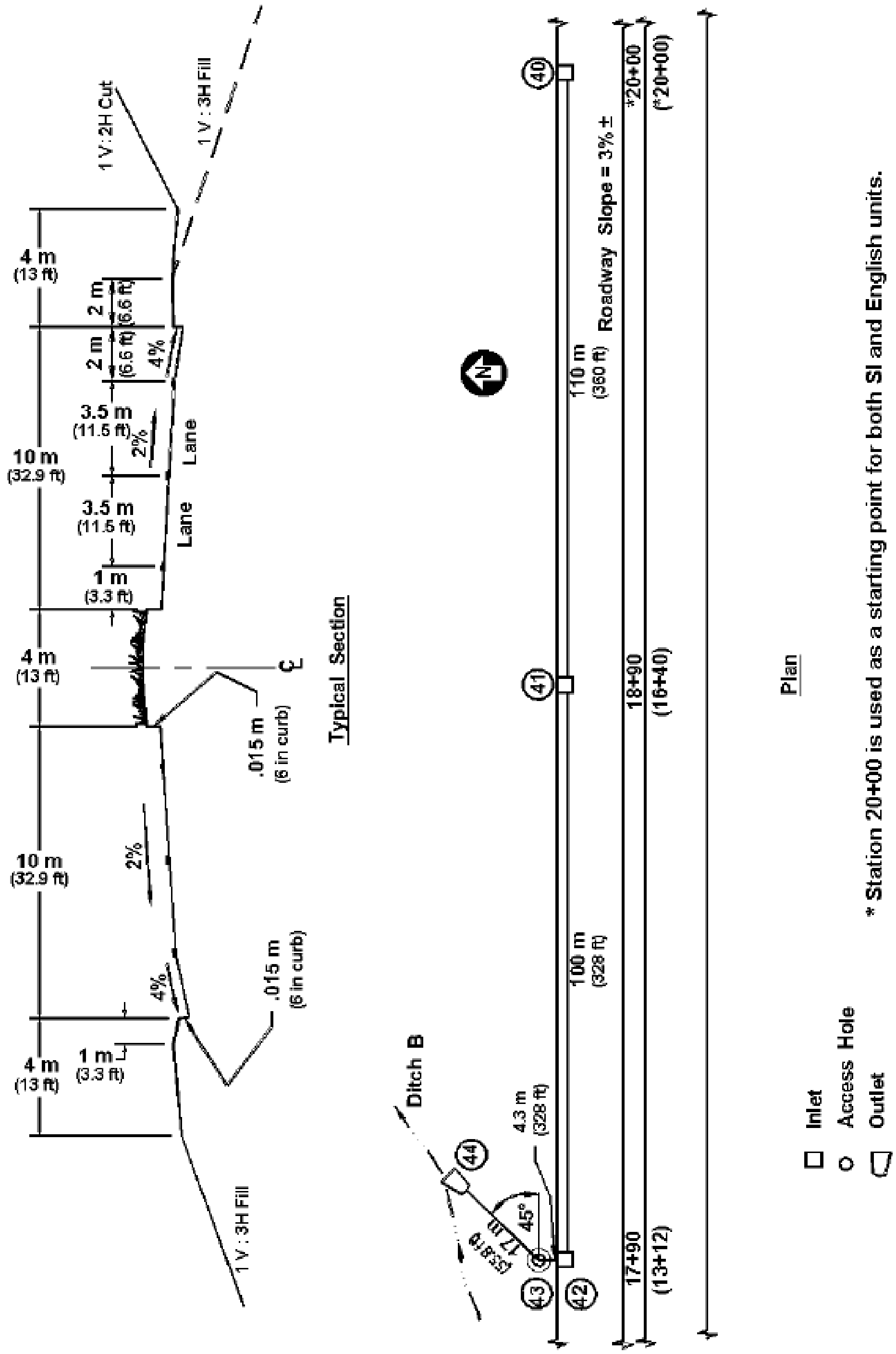


Figure 3.2-28 Roadway Plan and Sections for Example

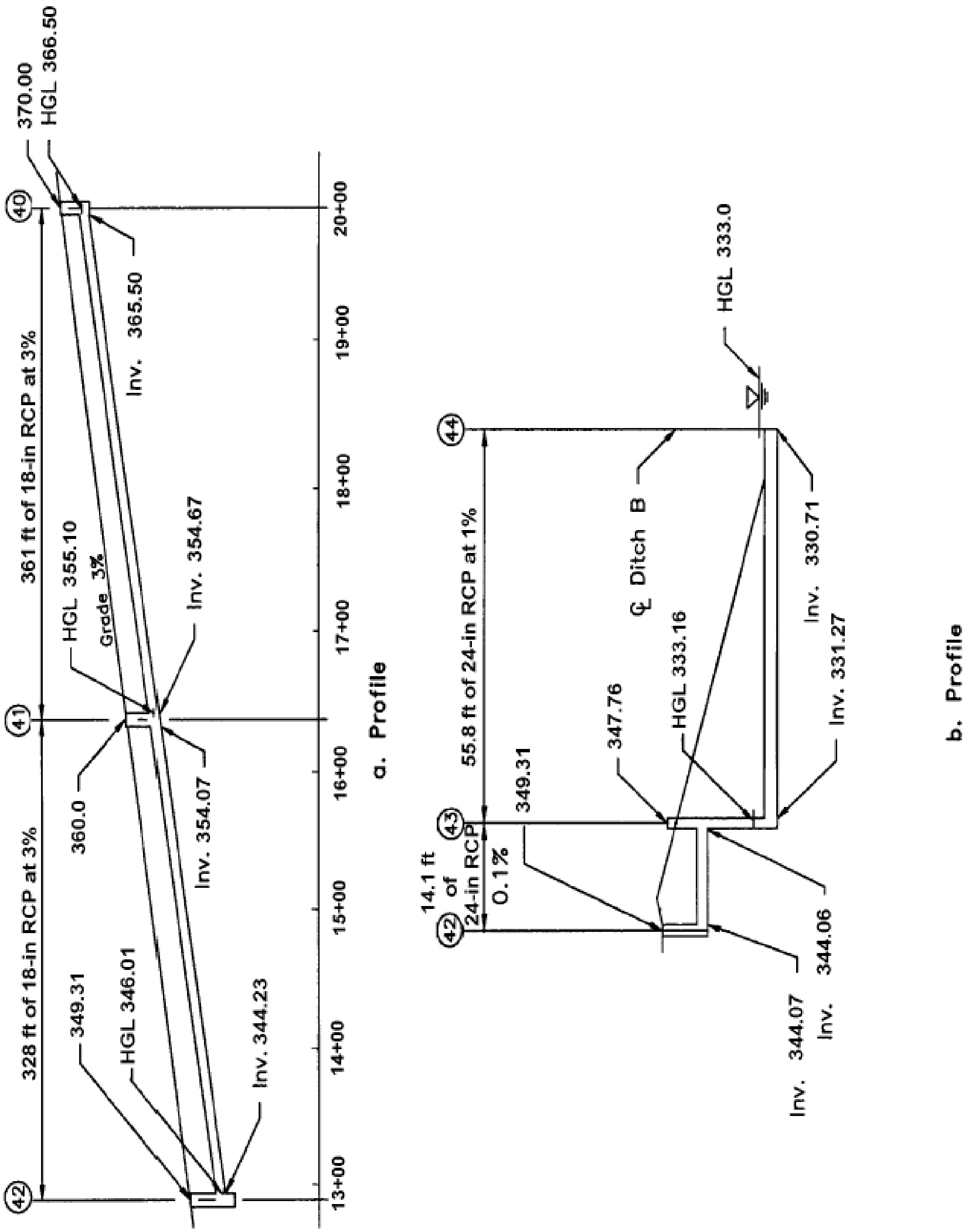


Figure 3.2-29 Storm Drain Profiles for Example

PRELIMINARY STORM DRAIN COMPUTATION SHEET
(English Solution)

COMPUTED BY ______ DATE ______
CHECKED BY ______ DATE ______
PAGE ______ OF ______

ROUTE ______
SECTION ______
COUNTY ______

STR. ID		LENGTH	DRAINAGE AREA		RUNOFF COEFF. "C"	"AREA" X "C"		TIME OF CONCENTRATION		RAIN "I"	RUNOFF "Q"	PIPE DIA.	Q FULL	VELOCITY		SEC TIME	INVERT ELEV.		CROWN DROP	SLOPE
FROM	TO	(Ft)	INC (ac)	TOTAL (ac)		INC. (ac)	TOTAL (ac)	INLET (min)	SYSTEM (min)	(in/hr)	(ft^3 /s)	(ft)	(ft^3/s)	FULL (ft /s)	DESIGN (ft /s)	(min)	U/S (ft)	D/S (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)
40	41	361.0	0.64	0.64	0.73	0.47	0.47	3	5	7.1	3.3	1.50	18.1	10.3	7.52	1	365.50	354.67	----	0.03
41	42	328.0	0.35	0.99	0.73	0.25	0.72	2	5	7.1	5.1	1.5	18.1	10.3	8.7	1	354.07	344.23	0.6	0.03
42	43	14.1	0.32	1.31	0.73	0.23	0.95	2	5	7.1	6.75	2.0	7.12	2.28	2.6	0	344.07	344.06	0.16	0.001
43	44	55.8	0.00	1.31	-	-	0.95	-	5	7.1	6.75	2.0	22.52	7.22	6.10	0	331.27	330.71	12.79	0.01

Figure 3.2-30 Storm Drain Computation Sheet for Example

Structure 40 to 41

Col. 1 From structure 40		
Col. 2 To structure 41		
Col. 3 Run Length	L = 2000 ft - 1639 ft L = 361 ft	Figure 3.2-30
Col. 4 Inlet Area	A_i = 0.64 ac	Figure 3.2-31
Col. 5 Total Area	A_t = 0.64 ac	Total area up to inlet 40
Col. 6 "C"	C = 0.73	Figure 3.2-31
Col. 7 Inlet CA	CA = (0.64)(0.73) CA = 0.47 ac	Col. 4 times Col. 6
Col. 8 Sum CA	ΣCA = 0.47 + 0 ΣCA= 0.47 ac	Col. 7 plus previous Col. 8
Col. 9 Inlet Time	t_i = 3 min	Figure 3.2-31
Col. 10 Sys. Time	t_c = 3 min (use 5 min)	same as Col. 9 for upstream most section
Col. 11 Intensity	I = 7.1 in/hr	Table 3.2-21; System time less than 5 minutes therefore, use 5 minutes
Col. 12 Runoff	Q = C_f (CA) (I) Q = (0.47) (7.1) / 1.0 Q = 3.3 ft^3/sec	Equation 2.1.3; C_f = 1.0 Col. 8 times Col. 11 multiplied by 1.0
Col. 21 Slope	S = 0.03	select desired pipe slope

Col. 13 Pipe Dia.	$D = [(Qn)/(K_Q\ S_o^{0.5})]^{0.375}$	Equation 3.2.18 or Figure 3.2-16
	$D = [(3.3)(0.013)/(0.46)(0.03)^{0.5})]^{0.375}$	
	$D = 0.8$ ft	
	$D_{min} = 1.5$ ft	use D_{min}
Col. 14 Full Cap	$Qf = (K_Q/n)\ D^{2.67}\ S_o^{0.5}$	Equation 3.2.18 or Figure 3.2-16
	$Qf = (0.46/0.013)\ (1.5)^{2.67}\ (0.03)^{0.5}$	
	$Qf = 18.1 ft^3/s$	
Col. 15 Vel. Full	$V_f = (K_V/n)\ D^{0.67}\ S_o^{0.5}$	Equation 3.2.17 or Figure 3.2-16
	$V_f = (0.59/0.013)\ (1.5)^{0.67}\ (0.03)^{0.5}$	
	$V_f = 10.3$ ft/s	
Col. 16 Vel. Design	$Q/Q_f = 3.3/18.1 = 0.18$	
	$V/V_f = 0.73$	Figure 3.2-18a
	$V = (0.73)\ (10.3)$	
	$V = 7.52$ ft/s	
Col. 17 Sect. Time	$t_s = L/V = 361 / 7.52 / 60$	Col. 3 divided by Col. 16
	$t_s = 0.8$ min; use 1 min	
Col. 20 Crown Drop	= 0	Upstream most invert
Col. 18 U/S Invert	= Grnd - 3.0 ft - dia	3 ft = min cover
	= 370.0 - 3.0 - 1.5	Ground elevation from Figure 3.2-30
	= 365.5 ft	
Col. 19 D/S Invert	= (365.5) - (361.0)(0.03)	Col. 18 - (Col. 3)(Col. 21)
	= 354.67 ft	

At this point, the pipe should be checked to determine if it still has adequate cover.

354.67 + 1.5 + 3.0 = 359.17	Invert elev. + Diam + min cover

Ground elevation of 360.0 ft is greater than 359.17 ft so OK

Structure 41 to 42

Col. 1 From	= 41	
Col. 2 To	= 42	
Col. 3 Run Length	L = 1639 - 1311 \ L = 328 ft	Figure 3.2-30
Col. 4 Inlet Area	A_i = 0.35 ac	Figure 3.2-31
Col. 5 Total Area	A_t = 0.35 + 0.64 A_t = 0.99 ac	
Col. 6 "C"	C = 0.73	Figure 3.2-31
Col. 7 Inlet CA	CA = (0.35)(0.73) CA = 0.25 ac	Col. 4 times Col. 6
Col. 8 Sum CA	ΣCA = 0.25 + 0.47 ΣCA = 0.72 ac	Col. 7 plus previous Col. 8
Col. 9 Inlet Time	t_i = 2 min	Table 3.2-20
Col. 10 Sys. Time	t_c = 4 min (use 5 min)	Col. 9 + Col. 17 for line 40-41
Col. 11 Intensity	I = 7.1 in/hr	Table 3.2-21; system time equals 5 min
Col. 12 Runoff	Q = (CA)(I)/(K_u) Q = (0.72) (7.1) / 1.0 Q = 5.1 ft^3/sec by 1.0	Equation 2.1.3 Col. 8 times Col. 11 divided
Col. 21 Slope	S = 0.03	select desired pipe slope

Col. 13 Pipe Dia.	$D = [(Qn)/(K_Q\ S_o^{0.5})]^{0.375}$	Equation 3.2.18 or Figure 3.2-16
	$D = [(5.1)\ (0.013)/(0.46)(0.03)^{0.5}]^{0.375}$	
	$D = 0.93$ ft	
	$D_{min} = 1.5$ ft use D_{min}	use D_{min}
Col. 14 Full Cap.	$Q_f = (K_Q/n)\ D^{2.67}\ S_o^{0.5}$	Equation 3.2.18 or Figure 3.2-16
	$Q_f = (0.46/0.013)(1.5)^{2.67}(0.03)^{0.5}$	
	$Q_f = 18.1$ ft3/s	
Col. 15 Vel. Full	$V_f = (K_V/n)\ D^{0.67}\ S_o^{0.5}$	Equation 3.2.18 or Figure 3.2-16
	$V_f = (0.59/0.013)(1.5)^{0.67}\ (0.03)^{0.5}$	
	$V_f = 10.3$ ft/s	
Col. 16 Vel. Design	$Q/Q_f = 5.1/18.1 = 0.28$	
	$V/V_f = 0.84$	Figure 3.2-18a
	$V = (0.84)\ (10.3)$	
	$V = 8.7$ ft/s	
Col. 17 Sect. Time	$T_s = L/V = 328\ /\ 8.75\ /\ 60$	Col. 3 divided by Col. 16
	$T_s = 0.6$ min; use 1 min	
Col. 20 Crown Drop	$= H_{ah} = K_{ah}\ (V^2\ /\ 2g)$	Equation 3.2.36 with Table 3.2-16
		$K_{ah} = 0.5$ for inlet - straight run
	$= (0.5)(8.7)^2\ /\ [(2)(32.2)]$	
	$= 0.6$ ft	
Col. 18 U/S Invert	$= 354.67 - 0.6$	Downstream invert of upstream
	$= 354.07$ ft	conduit minus estimated structure loss (drop)
Col. 19 D/S Invert	$= (354.07) - (328)(0.03)$	Col. 18 - (Col. 3)(Col. 21)
	$= 344.23$ ft	

Structure 42 to 43

Col. 1 From structure	= 42	
Col. 2 To structure	= 43	
Col. 3 Run Length	L = 14.1 ft	Figure 3.2-30
Col. 4 Inlet Area	A_i = 0.32 ac	Figure 3.2-31
Col. 5 Total Area	A_t = 0.32 + 0.99 A_t = 1.31 ac	Col. 4 plus structure 41 total area
Col. 6 "C"	C = 0.73	Figure 3.2-31
Col. 7 Inlet CA	CA = (0.32)(0.73) CA = 0.23 ac	Col. 4 times Col. 6
Col. 8 Sum CA	ΣCA = 0.23 + 0.72 ΣCA = 0.95 ac	Col. 7 plus structure 41 total CA values
Col. 9 Inlet Time	t_i = 2 min	Table 3.2-20
Col. 10 Sys. Time	t_c = 5 min	Col. 9 + Col. 17 for line 40-41 plus Col.17 for line 41-42
Col. 11 Intensity	I = 7.1 in/hr	Table 3.2-21
Col. 12 Runoff	Q = (CA) (I) Q = (0.95) (7.1) Q = 6.75 ft^3/sec	Col. 8 times Col. 11
Col. 21 Slope	S = 0.001	Select desired pipe slope

Col. 13 Pipe Dia.	$D = [(Qn)/(K_Q\ S_o^{0.5})]^{0.375}$	Equation 3.2.18 or Figure 3.2-16
	$D = [(6.75)(0.013)/(0.46)(0.001)^{0.5}]^{0.375}$	
	$D = 1.96$ ft	Use nominal size
	$D = 2.0$ ft	
Col. 14 Full Cap .	$Q_f = (K_Q/n)(D^{2.67})(S_o^{0.5})$	Equation 3.2.18 or Figure 3.2-16
	$Q_f = (0.46/(0.013)(2.0)^{2.67}\ (0.001)^{0.5}$	
	$Q_f = 7.12\ ft^3/s$	
Col. 15 Vel. Full	$V_f = (K_v/n)\ D^{0.67}\ S_o^{0.5}$	Equation 3.2.18 or Figure 3.2-16
	$V_f = (0.59)/(0.013)(2.0)^{0.67}\ (0.001)^{0.5}$	
	$V_f = 2.28$ ft/s	
Col. 16 Vel. Design	$Q/Q_f = 6.75/7.12 = 0.95$	
	$V/V_f = 1.15$	Figure 3.2-18a
	$V = (1.15)\ (2.28)$	
	$V = 2.6$ ft/s	
Col. 17 Sect. Time	$t_s = L/V = 14.1\ /\ 2.6\ /\ 60$	Col. 3 divided by Col. 16
	$t_s = 0.09$ min, use 0.0 min	
Col. 20 Crown Drop	$= H_{ah} = K_{ah}\ (V^2\ /\ 2g)$	Equation 3.2.36 and Table 3.2-13; $K_{ah} = 1.5$
	$= (1.5)(2.6)^2\ /\ [(2)(32.2)]$	for inlet - angled through 90 degrees
	$= 0.16$ ft	
Col. 18 U/S Invert	= 344.23 - 0.16	Downstream invert of upstream
	= 344.07 ft	conduit minus estimated structure loss (drop)
Col. 19 D/S Invert	= 344.07 - (14.1)(0.001)	Col. 18 - (Col. 3)(Col. 21)
	= 344.06 ft	

Structure 43 to 44

Col. 1 From	= 43	
Col. 2 To	= 44	
Col. 3 Run Length	L = 55.8 ft	Figure 3.2-30
Col. 4 Inlet Area	Ai = 0.0 ac	Figure 3.2-31
Col. 5 Total Area	At = 1.31 ac	Col. 4 plus structure 42 total area
Col. 6 "C"	C = n/a	Figure 3.2-31
Col. 7 Inlet CA	CA = 0.0	Col. 4 times Col. 6
Col. 8 Sum CA	ΣCA = 0.00 + 0.95 ΣCA = 0.95 ac	Col. 7 plus structure 42 total CA value
Col. 9 Inlet Time	n/a	No inlet
Col. 10 Sys. Time	t_c = 5 min	Col. 10 + Col. 17 for line 42-43
Col. 11 Intensity	I = 7.1 in/hr	Table 3.2-21
Col. 12 Runoff	Q = (CA) I Q = (0.95) (7.1) Q = 6.75 ft^3/sec	Col. 8 times Col. 11
Col. 21 Slope	S = 0.01	Select desired pipe slope
Col. 13 Pipe Dia.	$D = [(Qn)/(K_Q\ S_o^{0.5})]^{0.375}$ $D = [(6.75)(0.013)/(0.46)(0.01)^{0.5}]^{0.375}$ D = 1.27 ft D = 2.0 ft	Equation 3.2.18 or Figure 3.2-16 U/S conduit was 2.0 ft. - Do not reduce size inside the system

Col. 14 Full Cap.	$Qf = (KQ/n)(D^{2.67})(So^{0.5})$	Equation 3.2.18 or Figure 3.2-16
	$Qf = (0.46)/(0.013)(2.0)^{2.67} (0.01)^{0.5}$	
	$Qf = 22.52$ ft^3/s	
Col. 15 Vel. Full	$V_f = (K_v/n)\ D^{0.67}\ S_o^{0.5}$	Equation 3.2.17 or Figure 3.2-16
	$V_f = (0.59)/(0.013)(2.0)^{0.67} (0.01)^{0.5}$	
	$V_f = 7.22$ ft/s	
Col. 16 Vel. Design	$Q/Q_f = 6.75/22.52 = 0.30$	
	$V/V_f = 0.84$	Figure 3.2-18a
	$V = (0.84)\ (7.22)$	
	$V = 6.1$ ft/s	
Col. 17 Sect. Time	$t_s = 55.8 /6.1 / 60$	Col. 3 divided by Col. 16
	$t_s = 0.15$ min, use 0.0 min	
Col. 19 D/S Invert	= 330.71 ft	Invert at discharge point in ditch
Col. 18 U/S Invert	= 330.71 + (55.8)(0.01)	Col. 19 + (Col. 3)(Col. 21)
	= 331.27 ft	
Col. 20 Crown Drop	= 344.06 - 331.27	Col. 19 previous run - Col. 18
	= 12.79 ft straight run	

(2) Energy Grade Line Evaluation Computations - English Units

The following computational procedure follows the steps outlined in Section 3.2.8.12 above. Starting at structure 44, computations proceed in the upstream direction. A summary tabulation of the computational process is provided in Figure 3.2-32 English and Figure 3.2-33 English. The column by column computations for each section of storm drain follow:

RUN FROM STRUCTURE 44 TO 43

Outlet

Step 1	Col. 1A	Outlet	
	Col. 14A	HGL = 333.0	Downstream pool elevation
	Col. 10A	EGL = 333.0	Assume no velocity in pool

Structure 44

Step 2	Col. 1A, 1B	Str. ID = 44	Outlet
	Col. 15A	Invert = 330.71 ft	Outfall invert
		TOC = 330.71 + 2.0	Top of storm drain at outfall
		TOC = 332.71	
		Surface Elev = 332.71	Match TOC
Step 3		HGL = TW = 333.0	From Step 1
		$EGL_i = HGL + V^2/2g$	Use Case 1 since TW is above the top of conduit
		$EGL_i = 333.0 + 0.07$	
	Col. 13A	$EGL_i = 333.07$	EGL_i for str. 44

Structure 43

Step 4	Col. 1A, 1B	Str. ID = 43	Next Structure
	Col. 2A	D = 2.0 ft	Pipe Diameter
	Col. 3A	Q = 6.75 cfs	Conduit discharge (design value)
	Col. 4A	L = 55.8 ft	Conduit length
Step 5	Col. 5A	V = Q/A	Velocity; use full barrel velocity since outlet is submerged.
		$V = 6.75/[(\pi/4)(2.0)^2]$	
		V = 2.15 ft/s	
	Col. 7A	$V^2/2g = (2.15)^2/(2)(32.2)$	Velocity head in conduit
		= 0.07 ft	
Step 6	Col. 8A	$S_f = [(Qn)/(K_Q D^{2.67})]^2$	Equation 3.2.18

		$S_f = [(6.75)(0.013)/(0.46)(2.0)^{2.67}]^{2}$	
		$S_f = 0.00090$ ft/ft	
Step 7	Col. 2B	$H_f = S_f L$	Equation 3.2.27
		$H_f = (0.0009)(55.8)$	Col. 8A x Col. 4A
		$H_f = 0.05$	
	Col. 7B &	$h_b, H_c, H_e, H_j = 0$	
	Col. 9A	Total = 0.05 ft	

ENERGY GRADE LINE COMPUTATION SHEET - TABLE A
(English Solution)

COMPUTED BY ________ DATE ________ ROUTE ____________
CHECKED BY ________ DATE ________ SECTION ____________
PAGE ________ OF ________ COUNTY ____________
INITIAL TAILWATER ELEV. ____________

Str. ID	D	Q	L	V	d	d_c	$V^2/2g$	S_f	Total Pipe Loss (table B)	EGL_o	K table B	$K(V^2/2g)$	EGL_i	HGL	U/S TOC	Surf. Elev.
	(ft)	(ft³/s)	(ft)	(fps)	(ft)		(ft)	(ft / ft)	(ft)	(ft)		(ft)	(ft)	(ft)	(ft)	(ft)
(1)	(2)	(3)	(4)	(5)	(6a)	(6b)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16
OUTLET										333.00				333.00		
44													333.07		332.71	332.71
43	2.0	6.75	55.8	2.15	FULL	n/a	0.07	0.0009	0.05	333.12	0.5	0.04	333.16	*333.16	346.06	347.76
											*(Drop Structure)					
43	(New Outlet)			2.6		0.8	0.10						345.56	345.46	346.06	347.76
		(Above calculations are for inlet end of STR 43 for conduit 42-43)														
42	2.0	6.75	14.1	2.6	1.56	0.80	0.10	0.001	0.014	345.57	0.62	0.06	346.11	346.01	345.73	349.31
41	1.5	5.10	328.0	8.65	0.56	0.85	1.16	-	0	355.79	-	-	355.98	355.10	356.17	360.0
40	1.5	3.35	361.0	7.52	0.43	0.70	0.88		0	-	0	0	366.50	366.50	367.0	370.0

Figure 3.2-31 Energy Grade Line Computation Sheet, Table A, for English Example

ENERGY GRADE LINE COMPUTATION SHEET - TABLE B
(English Solution)

COMPUTED BY ______ DATE ______ ROUTE ______
CHECKED BY ______ DATE ______ SECTION ______
PAGE ______ OF ______ COUNTY ______

	Pipe Losses (ft)						Structure Losses (ft)							
Str. ID (1)	H_f (2)	h_b (3)	H_c (4)	H_e (5)	H_j (6)	Total (7)	d_{aho} (8)	K_o (9)	C_D (10)	C_d (11)	C_Q (12)	C_p (13)	C_B (14)	K (15)
44														
	0.05					0.05								
43							1.89							0.5
	0.014					0.014								
42							1.40	1.55	1.0	0.40	1.0	1.0	1.0	0.62
						0.0								
41														
														0.0
40							1.0							

Figure 3.2-32 Energy Grade Line Computation Sheet, Table B, for English Example

Step 8	Col. 10A	EGL_o = EGLi + pipe loss EGL_o = 333.07 + 0.05 EGL_o = 333.12 ft	
		HGL = 333.12 - 0.07 = 333.05 TOC = 331.27 + 2.0 = 333.27	Check for full flow - close Assumption OK
Step 9	Col. 8B	Not applicable due to drop structure	
Step 10	Col. 9B and 11A	Ke = 0.5	Inflow pipe invert much higher than d_{aho}. Assume square edge entrance
Step 17	Col. 12A	$K(V^2/2g) = (0.50)(0.07)$ $K(V^2/2g) = 0.04$ ft	Col. 11A times Col. 7A
Step 18	Col. 13A	$EGL_i = EGL_o$ EGL_i = 333.12 + 0.04 EGL_i = 333.16 ft	Col 10A plus 12A
Step 19	Col. 14A	HGL = EGL_i = 333.16 ft	For drop structures, the HGL is the same as the EGL
		d_{aho} = HGL- invert = 333.16 - 331.27 = 1.89 ft	Col. 8B
Step 20	Col. 15A	U/S TOC = Inv. + Dia. U/S TOC = 344.06 + 2.0 U/S TOC = 346.06 ft	From storm drain comp. sheet (Figure 3.2-31)
Step 21	Col. 16A	Surf. Elev. = 347.76 ft 347.76 > 333.09	From Figure 3.2-30. Surface elev. exceeds HGL, OK

Step 2	Col. 1A, 1B	Str. ID = 43	Drop Structure - new start
	Col. 15A	U/S TOC = 344.06 + 2.0	
		= 346.06	
	Col. 16A	Surface Elev = 347.76	
Step 3		HGL' = inv. + (d_c+D)/2	Calculate new HGL - Use Case 2
		HGL' = 344.06 + (0.80 + 2.0)/2	dc from Figure 3.2-18b
	Col. 14A	HGL = 345.46 ft	
		EGL = HGL + $V^2/2g$	V = 2.6 fps from Prelim. Comp. Sht.
		EGL = 345.46 + 0.10	
	Col. 13A	EGL = 345.56 ft	

Structure 42

Step 4	Col.1A	Str. ID = 42	
	Col. 2A	D = 2.0 ft	Pipe Diameter
	Col. 3A	Q = 6.75 cfs	Conduit discharge (design value)
	Col. 4A	L = 14.1 ft	Conduit length
Step 5A	Col. 5A	V = 2.6 ft/s	For flow: Actual velocity from storm drain computation sheet.
		$Q/Qf = 6.75 / 7.12 = 0.95$	
	Col. 6A	d_n = 1.56 ft Chart 26	Figure 3.2-31
	Col. 7A	$V^2/2g = (2.6)^2/(2)(32.2)$	Velocity head in conduit
		$V^2/2g = 0.10$ ft	
Step 5B	Col.6bA	d_c = 0.80 ft	From HDS-5
Step 5C		$d_n < d_c$	Flow is subcritical
Step 6	Col. 8A	$S_f = 0.001$	Conduit not full so S_f = pipe slope
			d_n = 1.56 (Figure 3.2.18a)
			d_c = 0.80 (HDS-5)
			Flow is subcritical
Step 7		$H_f = S_f L$	Equation 3.2.27
		$H_f = (0.001)(14.1)$	Col. 8A x Col. 5A
	Col. 2B	H_f = 0.014 ft	
		$h_b, H_c, H_e, H_j = 0$	
	Col. 7B and 9A	Total = 0.014 ft	
Step 8		$EGL_o = EGL_i$ + total pipe loss	Col. 14A plus Col. 9A
		$EGL_o = 345.56 + 0.014$	
	Col. 10A	EGL_o = 345.57 ft	

Step 9	Col. 8B	d_{aho} = EGL_o - velocity head - pipe invert da_{ho} = 345.57 - 0.10 - 344.07 d_{aho} = 1.40 ft	Col. 10A - Column 7A - pipe invert
Step 10	Col. 9B	K_o =0.1(b/Do)(1-sin θ)+1.4(b/Do)0.15 sin (θ) b = 4.0 ft D_o = 2.0 ft $\theta = 90^o$ $K_o = 0.1(4.0/2.0)(1 - \sin 90)+ 1.4(4.0/2.0)^{0.15} \sin 90$ K_o = 1.55	Equation 3.2.38 Access hole diameter. Col. 2A - outlet pipe diam Flow deflection angle
Step 11	Col. 10B	$C_D = (D_o/D_i)^3$ d_{aho} = 1.40 d_{aho}/D_o = (1.40/2.0) d_{aho}/D_o = 0.70 < 3.2 C_D = 1.0	Equation 3.2.39; pipe diameter Column 8B therefore
Step 12	Col. 11B	$Cd = 0.5 (d_{aho}/ D_o)^{0.6}$ d_{aho}/D_o = 0.70 < 3.2 $C_d = 0.5 (1.4/2.0)^{0.6}$ C_d = 0.40	Equation 3.2.40; Flow depth correction.
Step 13	Col. 12B	$CQ = (1-2 \sin \theta)(1-Qi/Qo)^{0.75}+1$ CQ = 1.0	Equation 3.2.41; relative flow No additional pipes entering
Step 14	Col. 13B	$C_p = 1 + 0.2(h/D_o)[(h-d)/D_o]$ C_p = 1.0	Equation 3.2.42; plunging flow No plunging flow
Step 15	Col. 14B	C_B = 1.0	Benching Correction, flat floor (Table 3.2-15)
Step 16	Col. 15B and 11A	$K = K_o C_D C_d C_Q C_p C_B$ K = (1.55)(1.0)(0.40)(1.0)(1.0)(1.0) K = 0.62	Equation 3.2.37

Step 17	Col. 12A	$K(V^2/2g) = (0.62)(0.10)$ $K(V^2/2g) = 0.06$ ft	Col. 11A times Col. 7A
Step 18	Col. 13A	$EGL_i = EGL_o + K(V2/2g)$ $EGL_i = 346.05 + 0.06$ $EGL_i = 346.11$	Col. 10A plus 12A
Step 19	Col. 14A	$HGL = EGL_i - V^2/2g$ $HGL = 346.11 - 0.10$ $HGL = 346.01$ ft	Col. 13A minus Col. 7A
Step 20	Col 15A	U/S TOC = Inv. + Dia. U/S TOC = 344.23 + 1.5 U/S TOC = 345.73 ft	Information from storm drain comp. sheet (Figure 3.2-31)
Step 21	Col 16A	Surf. Elev. = 349.31 ft 349.31 > 345.96	From Figure 3.2-30 Surface elev. exceeds HGL, OK

Structure 41

Step 4	Col. 1A, 1B	Str. ID = 41	Next Structure
	Col. 2A	D = 1.50 ft	Pipe Diameter
	Col. 3A	Q = 5.10 cfs	Conduit discharge (design value)
	Col. 4A	L = 328 ft	Conduit length
Step 5	Part full flow from column's 12 and 15 of storm drain computation sheet.		Continue with Step 5A
Step 5A		$Q/Q_f = 5.1/18.1 = 0.28$	
		$d/d_f = 0.37$	Figure 3.2-18a
		$d = (0.37)(1.5)$	
	Col. 6aA	$d = 0.56$ ft	

		$V/V_f = 0.84$	Figure 3.2-18a
		$V = (0.84)(10.3)$	
	Col. 5A	$V = 8.65$ fps	
		$V^2/2g = (8.65)2/(2)(32.2)$	Velocity head
	Col. 7A	$V^2/2g = 1.16$ ft	
Step 5B	Col. 6bA	$d_c = 0.85$ ft	Figure 3.2-18b
Step 5C		$0.56 < 0.85$	Supercritical flow since $d_n < d_c$
Step 5D	Col. 7B	Total pipe loss = 0	

Structure 40

Step 5E	Col. 1A,1B	Str. Id. = 40	Next structure
	Col. 2A	$D = 1.5$ ft	Pipe diameter
	Col. 3A	$Q = 3.35$ cfs	Conduit discharge (design)
	Col. 4A	$L = 361.0$ ft	Conduit length
Step 5F		$Q/Q_f = 3.3/18.1 = 0.18$	
		$d/d_c = 0.29$	Figure 3.2-18a
		$d = (0.29)(1.5)$	
	Col. 6aA	$d = 0.43$ ft	
	Col. 6bA	$d_c = 0.7$ ft	Figure 3.2-18b
Step 5H		$V/V_f = 0.73$	Figure 3.2-18a
		$V = (0.73)(10.3)$	
	Col. 5A	$V = 7.52$ fps	
	Col. 7A	$V^2/2g = (7.52)^2/(2)(32.2)$	Velocity head
		$V^2/2g = 0.88$ ft	
Step 5I		$d_n = 0.43$ ft < 0.70 ft$= d_c$	Supercritical flow since $d_n < d_c$

Step 5K	Col. 11A, and 15B	$K = 0.0$	Str. 41 line; supercritical flow;
	Col. 12A	$K(V^2/2g) = 0$	no structure losses

Since both conduits 42-41 and 41-40 are supercritical - establish HGL and EGL at each side of access hole 41.

		HGL = Inv. + d	
		HGL = 354.07 + 0.56	D/S Invert + Flow depth
		HGL = 354.63 ft	
		EGL = 354.63 + 1.16	
		HGL + velocity head	
	Col. 10A	EGL = 355.79 ft	EGL_o of Str.41
		HGL = 354.67 + 0.43	U/S invert + Flow depth
	Col. 14A	HGL = 355.10 ft	Highest HGL
		EGL = 355.10 + 0.88	HGL + velocity head
	Col. 13A	EGL = 355.98 ft	EGL_i of Str. 41
Step 20	Col. 15A	U/S TOC = Inv. + Dia.	Information from storm drain comp
		U/S TOC = 354.67 + 1.5	Sheet (Figure 3.2-31) for Str. 41
		U/S TOC = 356.17 ft	
Step 21	Col. 16A	Surf. Elev. = 360.0 ft	From Figure 3.2-30.
		360.0 > 355.10	Surface elev. > HGL, OK
Step 10b	Col. 8B	d_{aho} = 0.67 (1.5) = 1.0 ft	Figure 3.3-2a, HW/D = 0.67
		HGL = Str. 40 Inv. + d_{aho}	Structure Inv. from storm
		HGL = 365.50 + 1.0.	drain comp. sheet
	Col. 14A	HGL = 366.50 ft	
	Col.13A	EGL = 366.50 ft	Assume no velocity in str.
Step 20	Col. 15A	U/S TOC = Inv. + Dia.	Information from storm drain comp. sheet (Figure 3.2-31) for Str. 40
		U/S TOC = 365.5 + 1.5	
		U/S TOC = 367.0 ft	
Step 21	Col. 16A	Surf. Elev. = 370.0 ft	From Figure 3.2-30
		370.0 ft > 366.50 ft	Surface Elev. > HGL, OK

See Figures 3.2-31 and 3.2-32 for the tabulation of results. The final HGL values are indicated in Figure 3.2-30.

Section 3.3 – General Design and Construction Standards

Materials

Only reinforced concrete pipe is allowed under pavement for public and private storm drains in the Town of Copper Canyon. Corrugated plastic pipe (profile wall with smooth interior), including High-Density Polyethylene (HDPE) pipe and Corrugated PVC (CPVC), may be used in private property when not under pavement.

In selecting roughness coefficients for concrete pipe, consideration will be given to the average conditions at the site during the useful life of the structure. The ‘n’ value of 0.015 for concrete pipe shall be used primarily in analyzing old sewers where alignment is poor and joints have become rough. If, for example, concrete pipe is being designed at a location where it is considered suitable, and there is reason to believe that the roughness would increase through erosion or corrosion of the interior surface, slight displacement of joints or entrance of foreign materials. A roughness coefficient will be selected which in the judgment of the designer, will represent the average condition. Any selection of ‘n’ values below the minimum or above the maximum, either for monolithic concrete structures, or concrete pipe, will have to have written approval of the Town Engineer.

The hydraulic grade line shall in no case be above the surface of the ground or street gutter for the design storm. Allowance of head must also be provided for future extensions of the storm drainage system. In all cases the maximum HGL must be 12” below top of curb at any inlet.

Utilities

In the design of a storm drainage system, the engineer is frequently confronted with the problem of crossings between the proposed storm drain and existing or proposed utilities such as water, gas and sanitary sewer lines. The Town of Copper Canyon prefers a minimum of 2 feet of clearance with all conflicting utilities. All utilities in the vicinity of a proposed storm drain shall be clearly indicated on both plan and profile sheets.

Headwalls, Culverts, and Other Structures

For headwalls, culverts and other structures, Standard Construction Details adopted by the Town of Copper Canyon shall be used. The appropriate detail sheets for non-standard structures should be included in any construction plans.

Minimum Pipe Sizes and Depths

Minimum pipe sizes are 24” diameter for mains and 18” diameter for inlet leads. Minimum sizes of conduits of other shapes should have equivalent cross-sectional areas. Minimum depth of storm sewer from outside top of conduit to top of curb is 30 inches.

Pipe Connections and Curved Alignment

Prefabricated wye and tee connections supplied by the pipe manufacturer are required. Radial pipe can also be fabricated by the pipe manufacturer and shall be used through all curved alignments. However, designers should use bends or large radius curves where practical. When field connections or field radii must be used, all joints and gaps must be fully grouted with a collar to prevent voids and cave-ins caused by material washout into the storm drain.

Inlets

Curb inlets shall be 10, 15 or 20 feet in length. Proposed inlet lengths greater than 20 feet must be approved by the TOWN ENGINEER. Care should be taken in laying out inlets to allow for adequate driveway access between the inlet and the far property line.

Streets

To minimize standing water, the minimum concrete street grade shall be 0.50% and 1.0% for asphalt streets. Along a curve, this grade shall be measured along the outer gutter line. The minimum grade along a cul-de-sac or elbow gutter shall be 0.50%. Alternatively, elbows may be designed with a valley gutter along the normal outer gutter line, with two percent cross slope from curb to the valley gutter. The minimum grade for any valley gutter shall be 0.50%. Where a crest or sag is designed on a residential street, a PVI shall be used instead of a vertical curve where the total gradient change is no more than two percent ($\Delta \leq 2.0\%$).

Flow in Driveways and Intersections

At any intersection, only one street shall be crossed with surface drainage and this street shall be the lower classified street. Where an alley or street intersects a street, inlets shall be placed in the intersecting alley or street whenever the combination of flow down the alley or intersecting street would cause the capacity of the downstream street to be exceeded. Inlets shall be placed upstream from an intersection whenever possible.

The cumulative flows from existing driveways shall be considered and inlets provided as necessary where the flow exceeds the specified design capacity of the street.

Section 3.4 – Easements for Closed Conduit Systems

Minimum easement requirements for storm sewer pipe shall be as follows:

Table 3.4-1 Closed Conduit Easements

Pipe Size	Minimum Easement Width Required
39” and under	15 Feet
42” through 54”	20 Feet
60” through 66”	25 Feet
72” through 102”	30 Feet

The outside face of the proposed storm drain line shall be placed at least five (5) feet off either edge of the storm drain easement. The proposed centerline of overflow swales shall normally coincide with the centerline of the easement.

Box culverts shall have an easement width equal to the width of the box plus twenty (20) additional feet. The edge of the box should be located at least ten (10) feet from either edge of the easement.

Drainage easements will generally extend beyond an outfall headwall to provide for velocity dissipation devices and an area for maintenance operations. Drainage easements along a required outfall channel or ditch shall be provided until the flowline reaches an acceptable outfall.

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CHAPTER 4 – HYDRAULIC DESIGN OF CULVERTS, BRIDGES, OPEN CHANNELS, AND DETENTION STRUCTURES

Section 4.1 – Storm Water Open Channels, Culverts, Bridges, and Detention Structure Design Overview

4.1.1 – Storm Water System Design

4.1.1.1 Introduction

Storm water system design is an integral component of both site and overall storm water management design. Good drainage design must strive to maintain compatibility and minimize interference with existing drainage patterns; control flooding of property, structures, and roadways for design flood events; and minimize potential environmental impacts on storm water runoff.

4.1.1.2 System Components

The storm water system components consist of all the *integrated* site design practices and storm water controls utilized on the site. Three considerations largely shape the design of the storm water systems: water quality, streambank protection, and flood control.

The on-site flood control systems are designed to remove storm water from areas such as streets and sidewalks for public safety reasons. The drainage system can consists of inlets, street and roadway gutters, roadside ditches, small channels and swales, storm water ponds and wetlands, and small underground pipe systems which collect storm water runoff from mid-frequency storms and transport it to structural control facilities, pervious areas, and/or the larger storm water systems (i.e., natural waterways, large man-made conduits, and large water impoundments).

The storm water (major) system consists of natural waterways, open channels, large man-made conduits, and large water impoundments. In addition, the major system includes some less obvious drainage ways such as overload relief swales and infrequent temporary ponding areas. The storm water system includes not only the trunk line system that receives the water, but also the natural overland relief which functions in case of overflow from or failure of the on-site flood control system. Overland relief must not flood or damage houses, buildings or other property.

This chapter is intended to provide design criteria and guidance on several on-site flood control system components, including culverts (Section 4.2), bridges (Section 4.3), vegetated and lined open channels (Section 4.4), storage design (Section 4.5), outlet structures (Section 4.6), and energy dissipation devices for outlet protection (Section 4.7). The rest of this section covers important considerations to keep in mind in the planning and design of storm water drainage facilities.

4.1.1.3 Checklist for Planning and Design

The following is a general procedure for drainage system design on a development site.

A. Analyze topography, including:
   1. Check off-site drainage pattern. Where is water coming onto the site? Where is water leaving the site?
   2. Check on-site topography for surface runoff and storage, and infiltration
      a. Determine runoff pattern: high points, ridges, valleys, streams, and swales. Where is the water going?
      b. Overlay the grading plan and indicate watershed areas: calculate square footage (acreage), points of concentration, low points, etc.

B. Analyze other site conditions, including:
   1. Land use and physical obstructions such as walks, drives, parking, patios, landscape edging, fencing, grassed area, landscaped area, tree roots, etc.
   2. Soil type (infiltration rates).
   3. Vegetative cover (slope protection).

C. Check potential drainage outlets and methods, including:
   1. On-site (structural control, receiving water)
   2. Off-site (highway, storm drain, receiving water, regional control)
   3. Natural drainage system (swales)
   4. Existing drainage system (drain pipe)
   4. Analyze areas for probable location of drainage structures and facilities.
   5. Identify the type and size of drainage system components required. Design the drainage system and integrate with the overall storm water management system and plan.

4.1.2 – Key Issues in Storm Water System Design

4.1.2.1 Introduction

The traditional design of storm water systems has been to collect and convey storm water runoff as rapidly as possible to a suitable location where it can be discharged. This manual takes a different approach wherein the design methodologies and concepts of drainage design are to be integrated with the objectives for water quantity and quality control. This means:

- Storm water systems are to remove water efficiently enough to meet flood protection criteria and level of service requirements, and
- These systems are to complement the ability of the site design and structural storm water controls to mitigate the major storm water impacts of urban development.

The following are some of the key issues in integrating water quantity and quality control consideration in storm water system design.

4.1.2.2 General Design Considerations

- Storm water systems should be planned and designed so as to generally conform to natural drainage patterns and discharge to natural drainage pathways within a drainage basin. These natural drainage pathways should only be modified as a last resort to contain and safely convey the peak flows generated by the development.

- Runoff must be discharged in a manner that will not cause adverse impacts on downstream properties or storm water systems. In general, runoff from development sites within a drainage basin should be discharged at the existing natural drainage outlet or outlets. If the developer wishes to change discharge points he or she must demonstrate that the change will not have any adverse impacts on downstream properties or storm water (minor) systems.
- It is important to ensure that the combined on-site flood control system and major storm water system can handle blockages and flows in excess of the design capacity to minimize the likelihood of nuisance flooding or damage to private properties. If failure of minor storm water systems and/or major storm water structures occurs during these periods, the risk to life and property could be significantly increased.
- In establishing the layout of storm water systems, it is essential to ensure that flows are not diverted onto private property during flows up to the major storm water system design capacity.

4.1.2.3 Culverts

- Culverts can serve double duty as flow retarding structures in grass channel design. Care should be taken to design them as storage control structures if depths exceed several feet, and to ensure safety during flows.
- Improved entrance designs can absorb considerable slope and energy for steeper sloped designs, thus helping to protect channels.

4.1.2.4 Bridges

- Bridges enable streams to maintain flow conveyance.
- Bridges are usually designed so that they are not submerged.
- Bridges may be vulnerable to failure from flood-related causes.
- Flow velocities through bridge openings should not cause scour within the bridge opening or in the stream reaches adjacent to the bridge.

4.1.2.5 Open Channels

- Open channels provide opportunities for reduction of flow peaks and pollution loads. They may be designed as wet or dry enhanced swales or grass channels.
- Channels can be designed with natural meanders improving both aesthetics and pollution removal through increased contact time.
- Grass channels generally provide better habitat than hardened channel sections, though studies have shown that riprap interstices provide significant habitat as well. Velocities should be carefully checked at design flows and the outer banks at bends should be specifically designed for increased shear stress and super elevation.
- Compound sections can be developed to carry the annual flow in the lower section and higher flows above them. Figure 4.1-1 illustrates a compound section that carries 2% of the 100-year storm and 100-year flood flows within banks. This reduces channel erosion at lower flows, and meandering, self-forming low flow channels that attack banks. The shelf in the compound section should have a minimum 1:12 slope to ensure drainage.

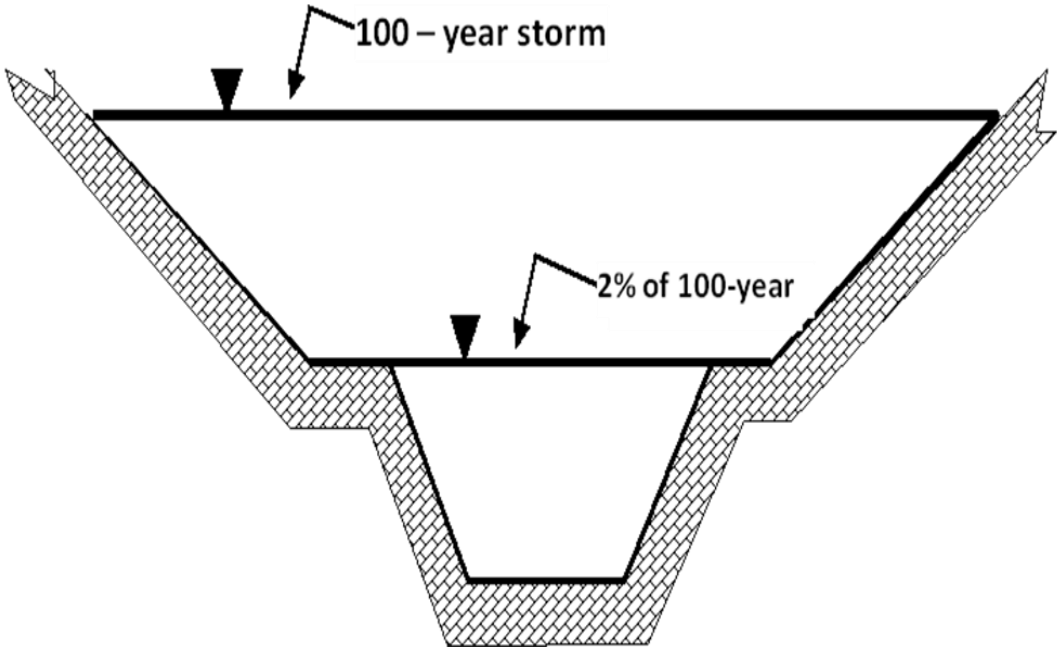


Figure 4.1-1 Compound Channel

- Flow control structures can be placed in the channels to increase residence time. Higher flows should be calculated using a channel slope from the top of the cross piece to the next one if it is significantly different from the channel bottom for normal depth calculations. Channel slope stability can also be ensured through the use of grade control structures that can serve as pollution reduction enhancements if they are set above the channel bottom. Regular maintenance is necessary to remove sediment and keep the channels from aggrading and losing capacity for larger flows.

4.1.2.6 Storage Design

- Storm water storage within a storm water system is essential to providing the extended detention of flows for water quality treatment and downstream streambank protection, as well as for peak flow attenuation of larger flows for flood protection.
- Runoff storage can be provided within an on-site flood control system through the use of structural storm water controls and/or nonstructural features.
- Storm water storage can be provided by detention, extended detention, or retention.
- Storage facilities may be provided on-site, or as regional facilities designed to manage storm water runoff from multiple projects.

4.1.2.7 Outlet Structures

- Outlet structures provide the critical function of the regulation of flow for structural storm water controls.
- Outlet structures may consist of a single stage outlet structure, or several outlet structures combined to provide multi-stage outlet control.

- Smaller, more protected outlet structures should be used for water quality and streambank protection flows.
- Large flows, such as flood flows, are typically handled through a broad crested weir, a riser with different sized openings, a drop inlet structure, or a spillway through an embankment.

4.1.2.8 Energy Dissipators

- Energy dissipators should be designed to return flows to non-eroding velocities to protect downstream channels.
- Care must be taken during construction that design criteria are followed exactly. The designs presented in this Manual have been carefully developed through model and full-scale tests. Each part of the criteria is important to their proper function.

4.1.3 - Design Storm Recommendations

Listed below are the design storm recommendations for various storm water drainage system components to be designed and constructed in accordance with the minimum storm water management standards.

Roadway Culvert Design

100-year storm for fully developed watershed conditions **unless Open Channel Design (Section 4.4), FEMA or TxDOT criteria control.**

Bridge Design

100-year storm for fully developed watershed conditions.

Open Channel Design

100-year storm for fully developed watershed conditions. For roadside ditches 10-year storm in ditch and 100-year storm in roadway right-of-way.

Energy Dissipation Design

100-year design for fully developed watershed conditions.

Storage (Detention Basin Design)

2-year, 10-year and 100-year storm for the critical storm duration (i.e. 3 hour, 6 hour or 24 hour duration) that results in the maximum (or near maximum) peak flow. Analysis should consider both existing watershed plus developed site conditions and fully developed watershed conditions.

Section 4.2 – Culvert Design

4.2.1 Overview

A *culvert* is a short, closed (covered) conduit that conveys storm water runoff under an embankment or away from the street right-of-way. The primary purpose of a culvert is to convey surface water, but properly designed it may also be used to restrict flow and reduce downstream peak flows.

The hydraulic and structural designs of a culvert must be such that minimal risks to traffic, property damage, and failure from floods prove the results of good engineering practice and economics. For economy and hydraulic efficiency, engineers should design culverts to operate with the inlet submerged during flood flows, if conditions permit. Design considerations include site and roadway data, design parameters (including shape, material, and orientation), hydrology (flood magnitude versus frequency relation), and channel analysis (stage versus discharge relation).

Section 4.2.2 – Symbols and Definitions

To provide consistency within this section as well as throughout this Manual the symbols listed in Table 4.2-1 will be used. These symbols were selected because of their wide use.

Table 4.2-1 Symbols and Definitions

Symbol	Definition	Units
A	Area of cross section of flow	ft^2
B	Barrel width	ft
C_d	Overtopping discharge coefficient	-
D	Culvert diameter or barrel depth	in or ft
d	Depth of flow	ft
d_c	Critical depth of flow	ft
d_u	Uniform depth of flow	ft
g	Acceleration of gravity	ft/s
H_f	Depth of pool or head, above the face section of invert	ft
h_o	Height of hydraulic grade line above outlet invert	ft
HW	Headwater depth above invert of culvert (depth from inlet invert to upstream total energy grade line)	ft
K_e	Inlet loss coefficient	-
L	Length of culvert	ft
N	Number of barrels	-
Q	Rate of discharge	cfs
S	Slope of culvert	ft/f
TW	Tailwater depth above invert of culvert	ft
V	Mean velocity of flow	ft/s
V_c	Critical velocity	ft/s

Section 4.2.3 – Design Criteria

The design of a culvert should take into account many different engineering and technical aspects at the culvert site and adjacent areas. The following design criteria should be considered for all culvert designs as applicable.

4.2.3.1 Frequency Flood

Town of Copper Canyon requires a 100-year design storm for fully developed watershed with headwater (HW – upstream WSEL) 1' below the adjacent curb. Only reinforced concrete culvert structures are acceptable.

The 100-year frequency storm shall be routed through all culverts to be sure building structures (e.g., houses, commercial buildings) are not flooded or increased damage does not occur to the highway or adjacent property for this design event.

4.2.3.2 Velocity Limitations

Both minimum and maximum velocities should be considered when designing a culvert. The maximum velocity should be consistent with channel stability requirements at the culvert outlet. The maximum allowable velocity 15 feet per second and outlet protection shall be provided where discharge velocities will cause erosion problems. To ensure self-cleaning during partial depth flow, a minimum velocity of 2.5 feet per second, for the 1-year flow, when the culvert is flowing partially full is required.

4.2.3.3 Buoyancy Protection

Headwalls, endwalls, slope paving, or other means of anchoring to provide buoyancy protection should be considered for all flexible culverts.

4.2.3.4 Length and Slope

The culvert length and slope should be chosen to approximate existing topography and, to the degree practicable, the culvert invert should be aligned with the channel bottom and the skew angle of the stream, and the culvert entrance should match the geometry of the roadway embankment. The maximum slope using concrete pipe is 10% and for CMP is 14% before pipe-restraining methods must be taken. Maximum vertical distance from throat of intake to flowline in a drainage structure is 10 feet. Drops greater than 4 feet will require additional structural design.

4.2.3.5 Debris Control

In designing debris control structures, it is recommended that the Hydraulic Engineering Circular No. 9 entitled *Debris Control Structures* be consulted.

4.2.3.6 Headwater Limitations

Headwater is water above the culvert invert at the entrance end of the culvert. The allowable headwater elevation is that elevation above which damage may be caused to adjacent property and/or the roadway and is determined from an evaluation of land use upstream of the culvert and the proposed or existing roadway elevation. It is this allowable headwater depth that is the primary basis for sizing a culvert.

The following criteria related to headwater should be considered:

- The *allowable headwater* is the depth of water that can be ponded at the upstream end of the culvert during the design flood, which will be limited by one or more of the following constraints or conditions:
    1. Headwater be non-damaging to upstream property.
    2. Ponding depth be no greater than the low point in the road grade unless overflow has been allowed by the roadway design or at the applicable design criteria, such as the 10-year or 25-year flood level.
    3. Ponding depth be no greater than the elevation where flow diverts around the culvert.
    4. Elevations established to delineate floodplain zoning.
    5. 12-inch (or applicable) freeboard requirements.
- The headwater should be checked for the 100-year flood to ensure compliance with flood plain management criteria and for most facilities the culvert should be sized to maintain flood-free conditions on major thoroughfares with 18-inch freeboard at the low-point of the road.
- The maximum acceptable outlet velocity should be identified (see subsection 4.4.3).
- Either the headwater should be set to produce acceptable velocities, or stabilization or energy dissipation should be provided where these velocities are exceeded.
- In general, the constraint that gives the lowest allowable headwater elevation establishes the criteria for the hydraulic calculations.
- Other site-specific design considerations should be addressed as required.

4.2.3.7 Tailwater Considerations

The hydraulic conditions downstream of the culvert site must be evaluated to determine a tailwater depth for a range of discharge. At times there may be a need for calculating backwater curves to establish the tailwater conditions. The following conditions must be considered:

- If the culvert outlet is operating with a free outfall, the critical depth and equivalent hydraulic grade line should be determined.
- For culverts that discharge to an open channel, the stage-discharge curve for the channel must be determined. See Section 4.4, *Open Channel Design*.
- If an upstream culvert outlet is located near a downstream culvert inlet, the headwater elevation of the downstream culvert may establish the design tailwater depth for the upstream culvert.
- If the culvert discharges to a lake, pond, or other major water body, the expected high water elevation of the particular water body may establish the culvert tailwater.

4.2.3.8 Storage

If storage is being assumed or will occur upstream of the culvert, refer to subsection 4.2.4.6 regarding storage routing as part of the culvert design.

4.2.3.9 Culvert Inlets

Hydraulic efficiency and cost can be significantly affected by inlet conditions. The inlet coefficient K_e, is a measure of the hydraulic efficiency of the inlet, with lower values indicating greater efficiency. Recommended inlet coefficients are given in Table 4.2-2.

4.2.3.10 Inlets with Headwalls

Headwalls may be used for a variety of reasons, including increasing the efficiency of the inlet, providing embankment stability, providing embankment protection against erosion, providing protection from buoyancy, and shortening the length of the required structure. Headwalls are required for all culverts and where buoyancy protection is necessary. If high headwater depths are to be encountered, or the approach velocity in the channel will cause scour, a short channel apron should be provided at the toe of the headwall.

This apron should extend at least one pipe diameter upstream from the entrance, and the top of the apron should not protrude above the normal streambed elevation.

4.2.3.11 Wingwalls and Aprons

Wingwalls are used where the side slopes of the channel adjacent to the entrance are unstable or where the culvert is skewed to the normal channel flow.

4.2.3.12 Improved Inlets

Where inlet conditions control the amount of flow that can pass through the culvert, improved inlets can greatly increase the hydraulic performance of the culvert.

4.2.3.13 Material Selection

Reinforced concrete pipe (RCP), pre-cast and cast in place concrete boxes are recommended for use (1) under a roadway, (2) when pipe slopes are less than 1%, or (3) for all flowing streams. RCP. High-density polyethylene (HDPE) pipe may also be used as specified in the municipal regulations. Table 4.2-3 gives recommended Manning's n values for different materials.

4.2.3.14 Culvert Skews

Culvert skews shall not exceed 45 degrees as measured from a line perpendicular to the roadway centerline without approval.

4.2.3.15 Culvert Sizes

The minimum allowable pipe diameter shall be 18 inches.

4.2.3.16 Weep Holes

Weep holes are sometimes used to relieve uplift pressure on headwalls and concrete rip-rap. Filter materials should be used in conjunction with the weep holes in order to intercept the flow and prevent the formation of piping channels through the fill embankment. The filter materials should be designed as an underdrain filter so as not to become clogged and so that piping cannot occur through the pervious material and the weep hole.

Table 4.2-2 Inlet Coefficients

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

[1] Although laboratory tests have not been completed on K_e values for High-Density Polyethylene (HDPE) pipes, the K_e values for corrugated metal pipes are recommended for HDPE pipes.

* Note: "End Section conforming to fill slope", made of either metal or concrete, are the sections commonly available from manufacturers. From limited hydraulic tests they are equivalent in operation to a headwall in both inlet and outlet control. Some end sections incorporating a closed taper in their design have a superior hydraulic performance. These latter sections can be designed using the information given for the beveled inlet.

Source: HDS No. 5, 2001

Table 4.2-3 Manning's n Values		
Type of Conduit	**Wall & Joint Description**	**Manning's n**
Concrete Pipe	Good joints, smooth walls	0.012
	Good joints, rough walls	0.016
	Poor joints, rough walls	0.017
Concrete Box	Good joints, smooth finished walls	0.012
	Poor joints, rough, unfinished walls	0.018
High Density Polyethylene (HDPE)	Corrugated Smooth Liner	0.015
	Corrugated	0.020
Polyvinyl Chloride (PVC)		0.011

Source: HDS No. 5, 2001

Note: For further information concerning Manning n values for selected conduits consult Hydraulic Design of Highway Culverts, Federal Highway Administration, 2001, HDS No. 5, pages 201 - 208.

4.2.3.17 Outlet Protection

See Section 4.5 for information on the design of outlet protection.

4.2.3.18 Erosion and Sediment Control

Erosion and sediment control shall be in accordance with the latest approved Erosion and Control Ordinance for Town of Copper Canyon.

4.2.3.19 Environmental Considerations

Where compatible with good hydraulic engineering, a site should be selected that will permit the culvert to be constructed to cause the least impact on the stream or wetlands. This selection must consider the entire site, including any necessary lead channels.

4.2.3.20 Safety Considerations

Roadside safety should be considered for culverts crossing under roadways. Guardrails or safety end treatments may be needed to enhance safety at culvert crossings. The AASHTO roadside design guide should be consulted for culvert designs under and adjacent to roadways

4.2.4 Design Procedures

4.2.4.1 Types of Flow Control

There are two types of flow conditions for culverts that are based upon the location of the control section and the critical flow depth (See Figure 4.2-1):

Inlet Control – Inlet control occurs when the culvert barrel is capable of conveying more flow than the inlet will accept. This typically happens when a culvert is operating on a steep slope. The control section of a culvert is located just inside the entrance. Critical depth occurs at or near this location, and the flow regime immediately downstream is supercritical.

Outlet Control – Outlet control flow occurs when the culvert barrel is not capable of conveying as much flow as the inlet opening will accept. The control section for outlet control flow in a culvert is located at the barrel exit or further downstream. Either subcritical or pressure flow exists in the culvert barrel under these conditions.

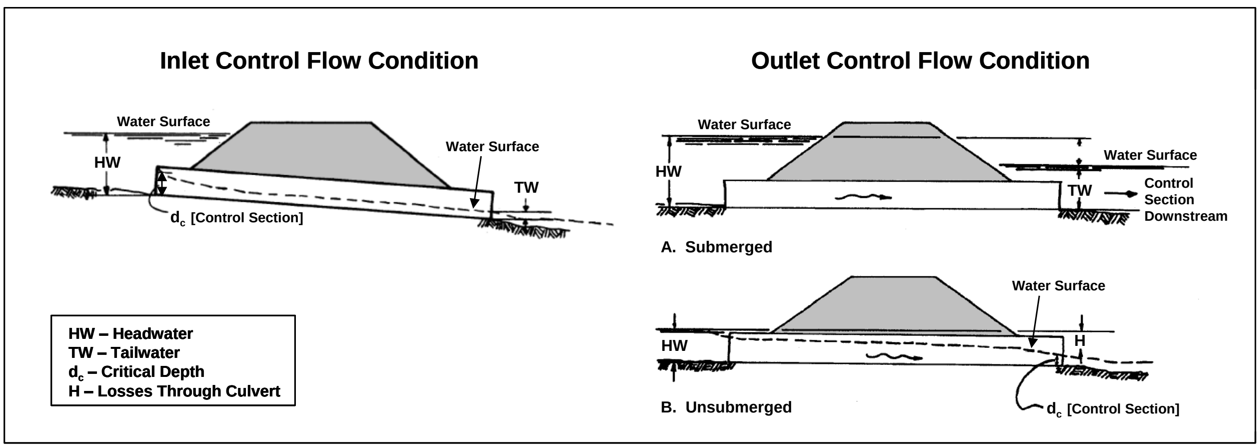


Figure 4.2-1 Culvert Flow Conditions

(Adapted from: HDS-5, 2001)

Proper culvert design and analysis requires checking for both inlet and outlet control to determine which will govern particular culvert designs. For more information on inlet and outlet control, see the FHWA Hydraulic Design of Highway Culverts, HDS-5, 2001.

4.2.4.2 Procedures

The culvert design process includes the following basic stages:

1. Define the location, orientation, shape, and material for the culvert to be designed. In many instances, consider more than single shape and material.
2. With consideration of the site data, establish allowable outlet velocity and maximum allowable depth of barrel.
3. Based upon subject discharges, associated tailwater levels, and allowable headwater level, define an overall culvert configuration to be analyzed (culvert hydraulic length, entrance conditions, and conduit shape and material).
4. Determine the flow type (supercritical or subcritical) to establish the proper path for determination of headwater and outlet velocity.
5. Optimize the culvert configuration.
6. Treat any excessive outlet velocity separately from headwater.

There are three procedures for designing culverts: inlet control design equations, manual use of inlet and outlet control nomographs, and the use computer programs such as HY8. It is recommended that the HY8 computer model or equivalent be used for culvert design. The computer software package HYDRAIN, which includes HY8, uses the theoretical basis from the nomographs to size culverts. In addition, this software can evaluate improved inlets, route hydrographs, consider road overtopping, and evaluate outlet streambed scour. By using water surface profiles, this procedure is more accurate in predicting backwater effects and outlet scour.

4.2.4.3 Inlet Control Design Equations

This section contains explanations of the equations and methods used to develop the design charts in HDS No. 5, where those equations and methods are not fully described in the main text. The following topics are discussed: the design equations for the unsubmerged and submerged inlet control nomographs, the dimensionless design curves for culvert shapes and sizes without nomographs, and the dimensionless critical depth charts for long span culverts and corrugated metal box culverts.

Inlet Control Nomograph Equations: The design equations used to develop the inlet control nomographs are based on the research conducted by the National Bureau of Standards (NBS) under the sponsorship of the Bureau of Public Roads (now the Federal Highway Administration). Seven progress reports were produced as a result of this research. Of these, the first and fourth through seventh reports dealt with the hydraulics of pipe and box culvert entrances, with and without tapered inlets (4, 7, to 10). These reports were one source of the equation coefficients and exponents, along with other references and unpublished FHWA notes on the development of the nomographs (56 and 57).

The two basic conditions on inlet control depend upon whether the inlet end of the culvert is or is not submerged by the upstream headwater. If the inlet is not submerged, the inlet performs as a weir. If the inlet is submerged, the inlet performs as an orifice. Equations are available for each of the above conditions.

Between the unsubmerged and the submerged conditions, there is a transition zone for which the NBS research provided only limited information. The transition zone is defined empirically by drawing a curve between and tangent to the curves defined by the unsubmerged and submerged equations. In most cases, the transition zone is short and the curve is easily constructed.

Table 4.2-4 contains the unsubmerged and submerged inlet control design equations. Note that there are two forms of the unsubmerged equation. Form (1) is based on the specific head at critical depth, adjusted with two correction factors. Form (2) is an exponential equation similar to a weir equation. Form (1) is preferable from a theoretical standpoint, but Form (2) is easier to apply and is the only documented form of equation for some of the inlet control nomographs.

The constants and the corresponding equation form are given in Table 4.2-5. Table 4.2-5 is arranged in the same order as the design nomographs in section 4.2.4.4, and provides the unsubmerged and submerged equation coefficients for each shape, material, and edge configuration. For the unsubmerged equations, the form of the equation is also noted.

The equations may be used to develop design curves for any conduit shape or size. Careful examination of the equation constants for a given form of equation reveals that there is very little difference between the constants for a given inlet configuration. Therefore, given the necessary conduit geometry for a new shape from the manufacturer, a similar shape is chosen from Table 4.2-5, and the constants are used to develop new design curves. The curves may be quasi-dimensionless, in terms of $Q/AD^{0.5}$ and HW_i/D, or dimensional, in terms of Q and HW_i for a particular conduit size. To make the curves truly dimensionless, $Q/AD^{0.5}$ must be divided by $g^{0.5}$, but this results in small decimal numbers. Note that coefficients for rectangular (Box) shapes should not be used for nonrectangular (circular, arch, pipe-arch, etc.) shapes and vice-versa. A constant slope value of 2 percent (0.02) is usually selected for the development of design curves. This is because the slope effect is small and the resultant headwater is conservatively high for sites with slopes exceeding 2 percent (except for mitered inlets).

Table 4.2-4 Inlet Control Design Equations		
UNSUBMERGED*		
Form (1)	$\frac{HW_i}{D} = \frac{H_c}{D} + K\left(\frac{K_u Q}{AD^{0.5}}\right)^M - 0.5S$ ***	**(4.2.1)**
Form (2)	$\frac{HW_i}{D} = K\left(\frac{K_u Q}{AD^{0.5}}\right)^M$	**(4.2.2)**
SUBMERGED**		
	$\frac{HW_i}{D} = c\left(\frac{K_u Q}{AD^{0.5}}\right)^2 + Y - 0.5S$ ***	**(4.2.3)**
Definitions		
HW_i	Headwater depth above inlet control section invert, m (ft)	
D	Interior height of culvert barrel, m (ft)	
H_c	Specific head at critical depth ($d_c + V_c^2/2g$), m^2 (ft^2)	
Q	Discharge, m^3/s (ft^3/s)	
A	Full cross sectional area of culvert barrel, m^2 (ft^2)	
S	Culvert barrel slope, m/m (ft/ft)	
K, M, c, Y	Constants from Table 4.2-5	
K_u	1.811 SI (1.0 English)	
* Equations 4.2.1 and 4.2.2 (unsubmerged) apply to about $Q/AD^{0.5}$ = 1.93 (3.5 English)		
** Equation 4.2.3 (submerged) above applies to about $Q/AD^{0.5}$ = 2.21 (4.0 English)		
*** For mitered inlets use +0.7 S instead of -0.5 S as the slope correction factor.		

Table 4.2-5 Constants for Inlet Control Design Equations

Chart No.	Shape and Material	Nomograph Scale	Inlet Edge Description	Equation Form	Unsubmerged K	Unsubmerged M	Submerged c	Submerged Y	References*
1	Circular Concrete	1	Square edge w/ headwall	1	.0098	2.0	.0398	.67	56/57
		2	Groove end w/ headwall		.0018	2.0	.0292	.74	
		3	Groove end projecting		.0045	2.0	.0317	.69	
2	Circular CMP	1	Headwall	1	.0078	2.0	.0379	.69	56/57
		2	Mitered to slope		.0210	1.33	.0463	.75	
		3	Projecting		.0340	1.50	.0553	.54	
3	Circular	A	Beveled ring, 45º bevels	1	.0018	2.50	.0300	.74	57
		B	Beveled ring, 33.7º bevels		.0018	2.50	.0243	.83	
8	Rectangular Box	1	30º to 75º wingwall flares	1	.026	1.0	.0347	.81	56
		2	90º and 15º wingwall flares		.061	.75	.0400	.80	56
		3	0º wingwall flares		.061	.75	.0423	.82	8
9	Rectangular Box	1	45º wingwall flare d = .043D	2	.510	.667	.0309	.80	8
		2	18º to 33.7º wingwall flare d = .083D		.486	.667	.0249	.83	
10	Rectangular Box	1	90º headwall w/ 3/4” chamfers	2	.515	.667	.0375	.79	8
		2	90º headwall w/ 45º bevels		.495	.667	.0314	.82	
		3	90º headwall w/ 33.7º bevels		.486	.667	.0252	.865	
11	Rectangular Box	1	3/4” chamfers; 45º skewed headwall	2	.545	.667	.04505	.73	8
		2	3/4” chamfers; 30º skewed headwall		.533	.667	.0425	.705	
		3	3/4” chamfers; 15º skewed headwall		.522	.667	.0402	.68	
		4	45º bevels; 10º-45º skewed headwall		.498	.667	.0327	.75	
12	Rectangular Box 3/4” chamfers	1	45º non-offset wingwall flares	2	.497	.667	.0339	.803	8
		2	18.4º non-offset wingwall flares		.493	.667	.0361	.806	
		3	18.4º non-offset wingwall flares 30º skewed barrel		.495	.667	.0386	.71	
13	Rectangular Box Top Bevels	1	45º wingwall flares - offset	2	.497	.667	.0302	.835	8
		2	33.7º wingwall flares - offset		.495	.667	.0252	.881	
		3	18.4º wingwall flares - offset		.493	.667	.0227	.887	
16-19	CM Boxes	2	90º headwall	1	.0083	2.0	.0379	.69	57
		3	Thick wall projecting		.0145	1.75	.0419	.64	
		5	Thin wall projecting		.0340	1.5	.0496	.57	
29	Horizontal Ellipse Concrete	1	Square edge w/ headwall	1	.0100	2.0	.0398	.67	57
		2	Groove end w/ headwall		.0018	2.5	.0292	.74	
		3	Groove end projecting		.0045	2.0	.0317	.69	

Table 4.2-5 Constants for Inlet Control Design Equations

Chart No.	Shape and Material	Nomograph Scale	Inlet Edge Description	Equation Form	Unsubmerged		Submerged		References*
					K	M	c	Y	
30	Vertical Ellipse Concrete	1	Square edge w/ headwall	1	.0100	2.0	.0398	.67	57
		2	Groove end w/ headwall		.0018	2.5	.0292	.74	
		3	Groove end projecting		.0095	2.0	.0317	.69	
34	Pipe Arch 18" Corner Radius CM	1	90º headwall	1	.0083	2.0	.0379	.69	57
		2	Mitered to slope		.0300	1.0	.0463	.75	
		3	Projecting		.0340	1.5	.0496	.57	
35	Pipe Arch 18" Corner Radius CM	1	Projecting	1	.0300	1.5	.0496	.57	56
		2	No Bevels		.0088	2.0	.0368	.68	
		3	33.7º Bevels		.0030	2.0	.0269	.77	
36	Pipe Arch 31" Corner Radius CM	1	Projecting	1	.0300	1.5	.0496	.57	56
			No Bevels		.0088	2.0	.0368	.68	
			33.7º Bevels		.0030	2.0	.0269	.77	
41-43	Arch CM	1	90º headwall	1	.0083	2.0	.0379	.69	57
		2	Mitered to slope		.0300	1.0	.0463	.75	
		3	Thin wall projecting		.0340	1.5	.0496	.57	
55	Circular	1	Smooth tapered inlet throat	2	.534	.555	.0196	.90	3
		2	Rough tapered inlet throat		.519	.64	.0210	.90	
56	Elliptical Inlet Face	1	Tapered inlet-beveled edges	2	.536	.622	.0368	.83	3
		2	Tapered inlet-square edges		.5035	.719	.0478	.80	
		3	Tapered inlet-thin edge projecting		.547	.80	.0598	.75	
57	Rectangular	1	Tapered inlet throat	2	.475	.667	.0179	.97	3
58	Rectangular Concrete	1	Side tapered-less favorable edges	2	.56	.667	.0446	.85	3
		2	Side tapered-more favorable edges		.56	.667	.0378	.87	
59	Rectangular Concrete	1	Slope tapered-less favorable edges	2	.50	.667	.0446	.65	3
			Slope tapered-more favorable edges		.50	.667	.0378	.71	

* These references are cited in FHWA, 2001, HYD-5. They can be accessed at the Federal Highway Administration web site: www.fhwa.dot.gov/bridge/hydpub.htm.

4.2.4.4 Nomographs

(Nomographs are not allowed by Town of Copper Canyon for final sizing of culverts with drainage areas greater than 10 acres. The use of nomographs for final design of culverts with drainage areas greater than 10 acres requires approval of the TOWN ENGINEER. A backwater analysis using HEC-RAS is required for culverts with areas greater than 10 acres.)

The use of culvert design nomographs requires a trial and error solution. Nomograph solutions provide reliable designs for many applications. It should be remembered that velocity, hydrograph routing, roadway overtopping, and outlet scour require additional, separate computations beyond what can be obtained from the nomographs. Figures 4.2-2(a) and (b) show examples of an inlet control and outlet control nomographs for the design of concrete pipe culverts. For other culvert designs, refer to the complete set of nomographs in FHWA Hydraulic Design of Highway Culverts, HDS-5, 2001, Second Edition.

This section presents design guidance for culverts originally published in HEC-12, Drainage of Highway Pavements and AASHTO's Model Drainage Manual.

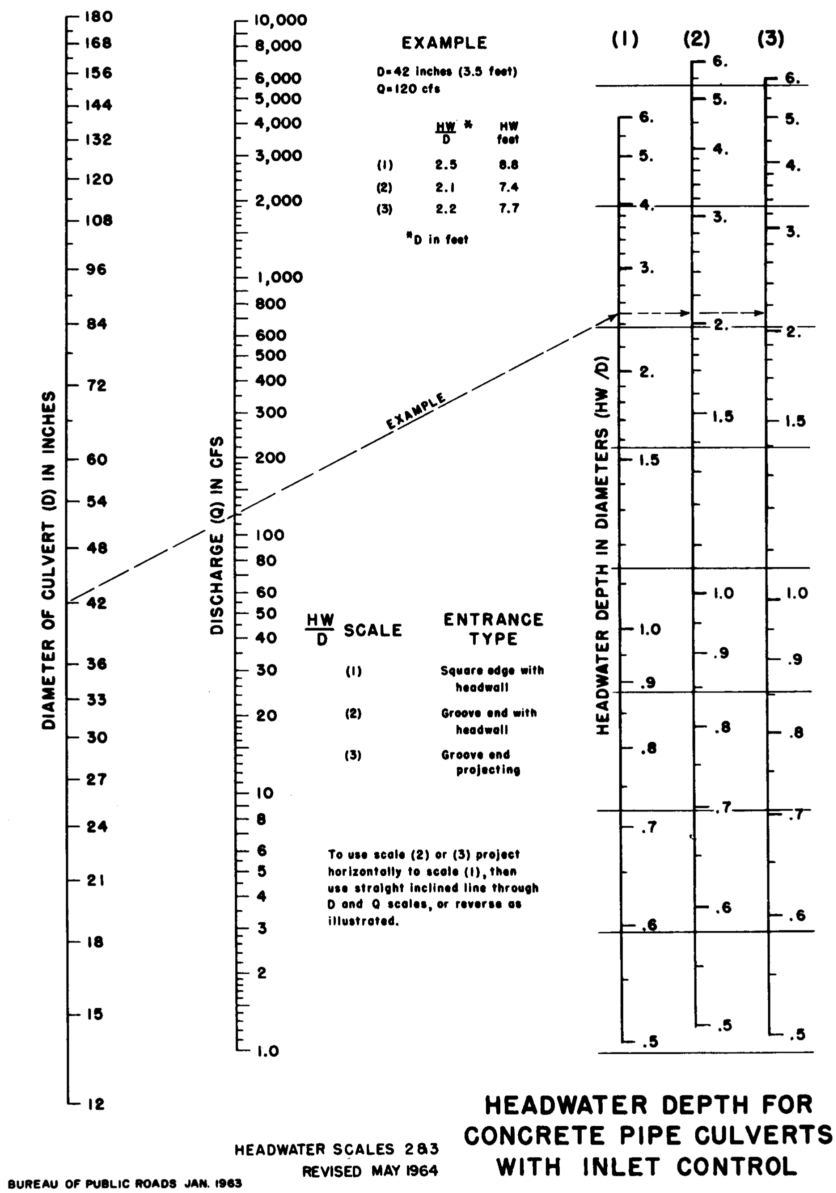


Figure 4.2-2(a)   Headwater Depth for Concrete Pipe Culvert with Inlet Control

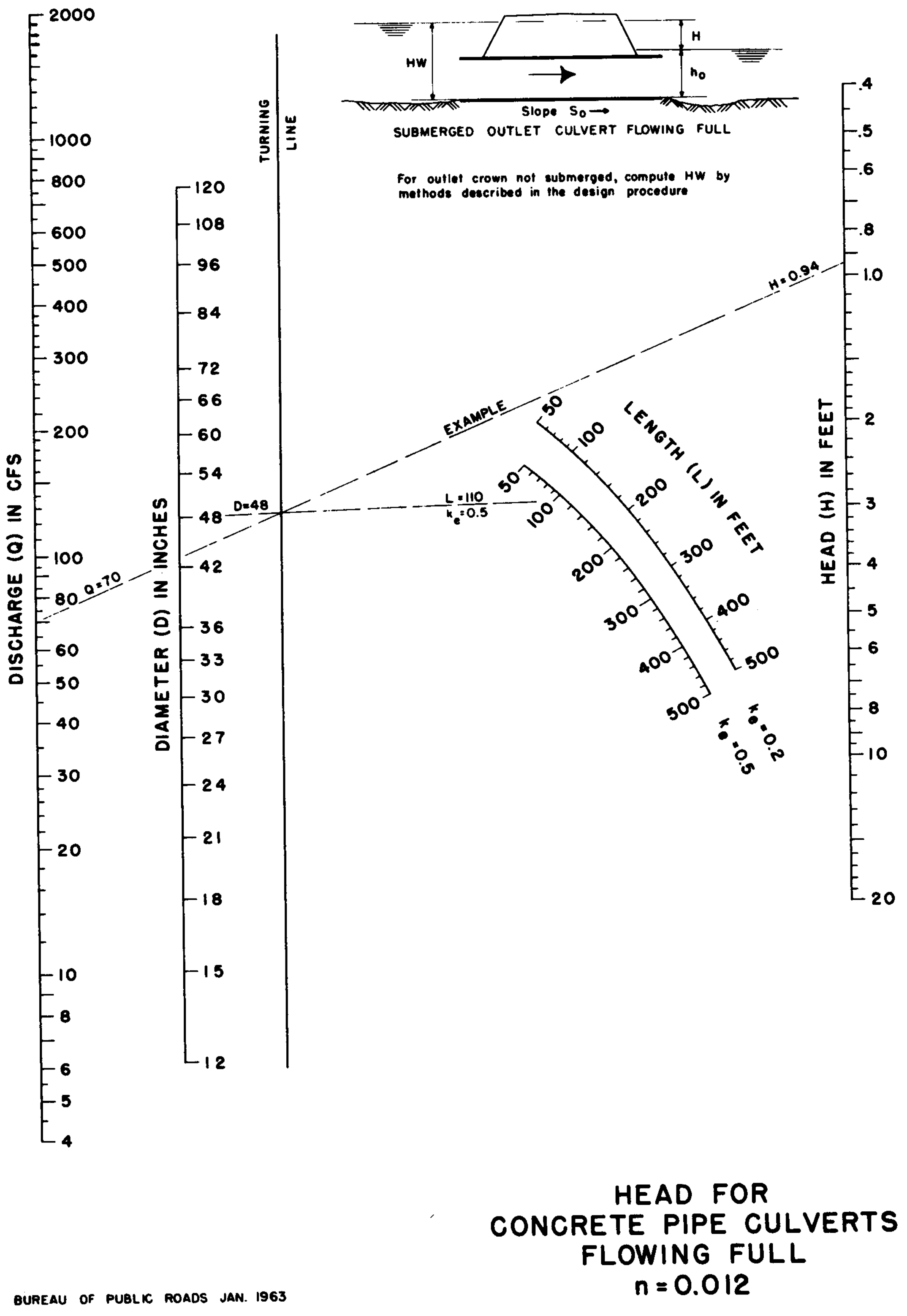


Figure 4.2-2(b) Head for Concrete Pipe Culverts Flowing Full

4.2.4.5 Design Procedure

The following design procedure requires the use of inlet and outlet nomographs.

Step 1 List design data:

- Q = discharge (cfs)
- L = culvert length (ft)
- S = culvert slope (ft/ft)
- TW = tailwater depth (ft)
- V = velocity for trial diameter (ft/s)
- K_e = inlet loss coefficient
- HW = allowable headwater depth for the design storm (ft)

Step 2 Determine trial culvert size by assuming a trial velocity of 3 to 5 ft/s and computing the culvert area, A = Q/V. Determine the culvert diameter (inches).

Step 3 Find the actual HW for the trial size culvert for both inlet and outlet control.

- For inlet control, enter inlet control nomograph with D and Q and find HW/D for the proper entrance type.
- Compute HW and, if too large or too small, try another culvert size before computing HW for outlet control.
- For outlet control enter the outlet control nomograph with the culvert length, entrance loss coefficient, and trial culvert diameter.
- To compute HW, connect the length scale for the type of entrance condition and culvert diameter scale with a straight line, pivot on the turning line, and draw a straight line from the design discharge through the turning point to the head loss scale H. Compute the headwater elevation HW from the equation:

$$\mathbf{HW = H + h_o - LS} \qquad \mathbf{(4.2.4)}$$

where:

- h_o = ½ (critical depth + D), or tailwater depth, whichever is greater
- L = culvert length
- S = culvert slope

Step 4 Compare the computed headwaters and use the higher HW nomograph to determine if the culvert is under inlet or outlet control.

- If inlet control governs, then the design is complete and no further analysis is required.
- If outlet control governs and the HW is unacceptable, select a larger trial size and find another HW with the outlet control nomographs. Since the smaller size of culvert had been selected for allowable HW by the inlet control nomographs, the inlet control for the larger pipe need not be checked.

Step 5 Calculate exit velocity and if erosion problems might be expected, refer to Section 4.7 for appropriate energy dissipation designs. Energy dissipation designs may affect the outlet hydraulics of the culvert.

4.2.4.6 Performance Curves - Roadway Overtopping

A performance curve for any culvert can be obtained from the nomographs by repeating the steps outlined above for a range of discharges that are of interest for that particular culvert design. A graph is then plotted of headwater versus discharge with sufficient points so that a curve can be drawn through the range of interest. These curves are applicable through a range of headwater, velocities, and scour depths versus discharges for a length and type of culvert. Usually charts with length intervals of 25 to 50 feet are satisfactory for design purposes. Such computations are made much easier by the use of computer programs.

To complete the culvert design, roadway overtopping should be analyzed. A performance curve showing the culvert flow as well as the flow across the roadway is a useful analysis tool. Rather than using a trial and error procedure to determine the flow division between the overtopping flow and the culvert flow, an overall performance curve can be developed.

The overall performance curve can be determined as follows:

Step 1 Select a range of flow rates and determine the corresponding headwater elevations for the culvert flow alone. The flow rates should fall above and below the design discharge and cover the entire flow range of interest. Both inlet and outlet control headwaters should be calculated.

Step 2 Combine the inlet and outlet control performance curves to define a single performance curve for the culvert.

Step 3 When the culvert headwater elevations exceed the roadway crest elevation, overtopping will begin. Calculate the equivalent upstream water surface depth above the roadway (crest of weir) for each selected flow rate. Use these water surface depths and Equation 4.2.5 to calculate flow rates across the roadway.

$$Q = C_d L (HW)^{1.5} \qquad \textbf{(4.2.5)}$$

where:

Q = overtopping flow rate (ft^3/s)
C_d = overtopping discharge coefficient
L = length of roadway (ft)
HW = upstream depth, measured from the roadway crest to the water surface upstream of the weir drawdown (ft)

Note: See Figure 4.2-3 for guidance in determining a value for C_d. For more information on calculating overtopping flow rates see pages 38 - 44 in HDS No. 5, 2001.

Step 4 Add the culvert flow and the roadway overtopping flow at the corresponding headwater elevations to obtain the overall culvert performance curve.

4.2.4.7 Storage Routing

A significant storage capacity behind a highway embankment attenuates a flood hydrograph. Because of the reduction of the peak discharge associated with this attenuation, the required capacity of the culvert, and its size, may be reduced considerably. If significant storage is anticipated behind a culvert, the design should be checked by routing the design hydrographs through the culvert to determine the discharge and stage behind the culvert. See subsection 4.2.7 and Section 2.2 for more information on routing. Additional routing procedures are outlined in Hydraulic Design of Highway Culverts, Section V - Storage Routing, HDS No. 5, 2001, Federal Highway Administration, pages 123 - 142.

Note: Storage should be taken into consideration only if the storage area will remain available for the life of the culvert as a result of purchase of ownership or right-of-way or an easement has been acquired.

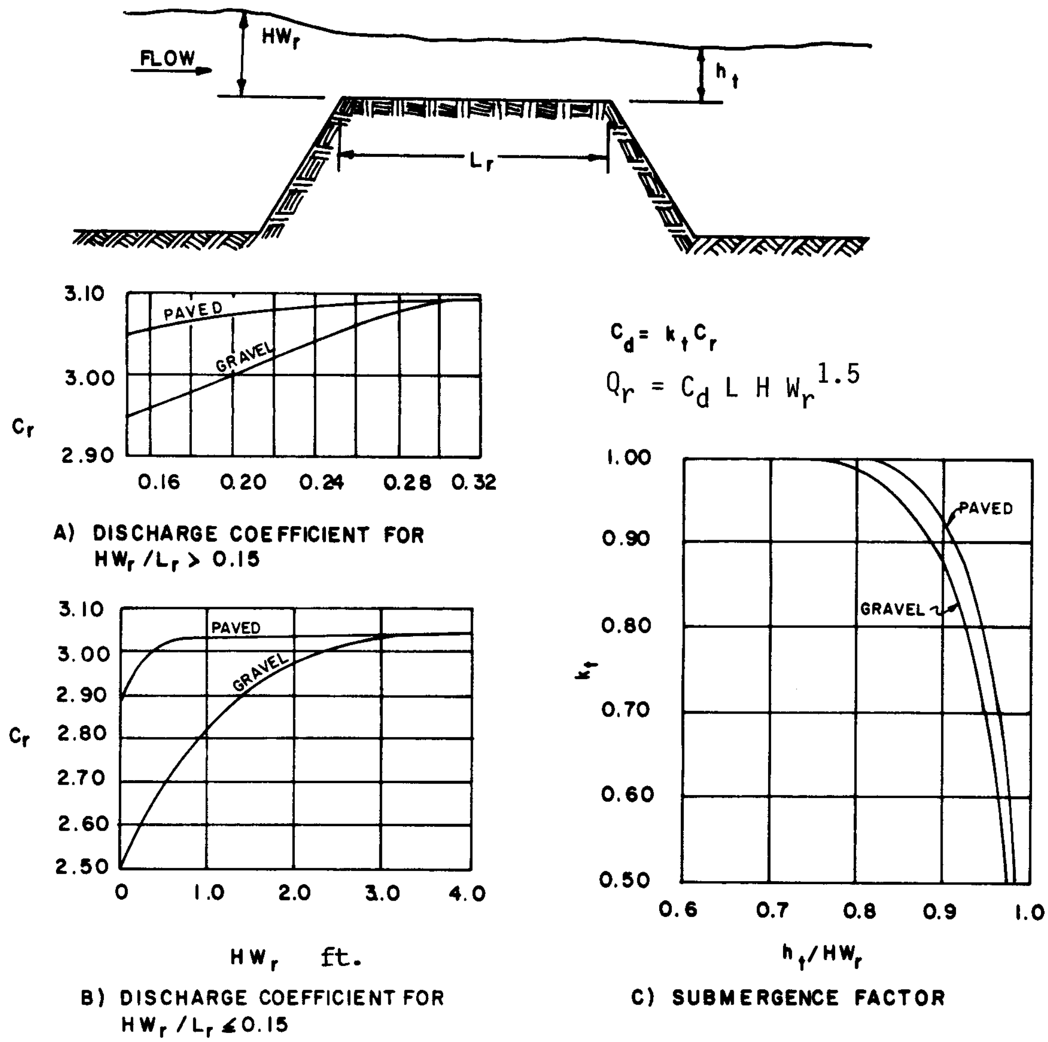


Figure 4.2-3 Discharge Coefficients for Roadway Overtopping
(Source: HDS No. 5, 2001)

4.2.5 – Culvert Design Example

(This procedure is acceptable for preliminary sizing of all culverts and final sizing of culverts with drainage areas of 10 acres or less unless approved by the TOWN ENGINEER.)

4.2.5.1 Introduction

The following example problem illustrates the procedures to be used in designing culverts using the nomographs.

4.2.5.2 Example

Size a culvert given the following example data, which were determined by physical limitations at the culvert site and hydraulic procedures described elsewhere in this handbook.

4.2.5.3 Example Data

<u>Input Data</u>

Discharge for 2-yr flood = 35 cfs

Discharge for 25-yr flood = 70 cfs

Allowable H_w for 25-yr discharge = 5.25 ft

Length of culvert = 100 ft

Natural channel invert elevations - inlet = 15.50 ft, outlet = 14.30 ft

Culvert slope = 0.012 ft/ft

Tailwater depth for 25-yr discharge = 3.5 ft

Tailwater depth is the normal depth in downstream channel

Entrance type = Groove end with headwall

4.2.5.4 Computations

1. Assume a culvert velocity of 5 ft/s. Required flow area = 70 cfs/5 ft/s = 14 ft^2 (for the 25-yr recurrence flood).
2. The corresponding culvert diameter is about 48 in. This can be calculated by using the formula for area of a circle: Area = $(3.14D^2)/4$ or D = $(\text{Area times } 4/3.14)^{0.5}$. Therefore: D = $((14 \text{ sq ft} \times 4)/3.14)^{0.5} \times 12$ in/ft) = 50.7 in
3. A grooved end concrete culvert with a headwall is selected for the design. Using the inlet control nomograph (Figure 4.2-2(a)), with a pipe diameter of 48 inches and a discharge of 70 cfs; read a HW/D value of 0.93.

4. The depth of headwater (HW) is (0.93) x (4) = 3.72 ft, which is less than the allowable headwater of 5.25 ft. Since 3.72 ft is considerably less than 5.25 try a small culvert.

5. Using the same procedures outlined in steps 4 and 5 the following results were obtained.

   42-inch culvert – HW = 4.13 ft

   36-inch culvert – HW = 5.04 ft

   Select a 36-inch culvert to check for outlet control.

6. The culvert is checked for outlet control by using Figure 4.2-2(b).

   With an entrance loss coefficient K_e of 0.20, a culvert length of 100 ft, and a pipe diameter of 36 in., an H value of 2.8 ft is determined. The headwater for outlet control is computed by the equation: $HW = H + h_o - LS$

   Compute h_o

   $h_o = T_w$ or ½ (critical depth in culvert + D), whichever is greater.

   $h_o = 3.5$ ft or $h_o = ½ (2.7 + 3.0) = 2.85$ ft

   Note: critical depth is obtained from Figure 3.2-18(b).

   Therefore: $h_o = 3.5$ ft

   The headwater depth for outlet control is:

   $HW = H + h_o - LS = 2.8 + 3.5 - (100) \times (0.012) = 5.10$ ft

7. Since HW for outlet control (5.10 ft) is greater than the HW for inlet control (5.04 ft), outlet control governs the culvert design. Thus, the maximum headwater expected for a 25-year recurrence flood is 5.10 ft, which is less than the allowable headwater of 5.25 ft.

8. Estimate outlet exit velocity. Since this culvert is an outlet control and discharges into an open channel downstream with tailwater above culvert, the culvert will be flowing full at the flow depth in the channel. Using the design peak discharge of 70 cfs and the area of a 36-inch or 3.0-foot diameter culvert the exit velocity will be:

   $Q = VA$

   Therefore: $V = 70 / (3.14(3.0)^2)/4 = 9.9$ ft/s

   With this high velocity, consideration should be given to provide an energy dissipator at the culvert outlet. See Section 4.7 (*Energy Dissipation Design)*.

9. Check for minimum velocity using the 2-year flow of 35 cfs.

   Therefore: $V = 35 / (3.14(3.0)^2/4 = 5.0$ ft/s > minimum of 2.5 - OK

10. The 100-year flow should be routed through the culvert to determine if any flooding problems will be associated with this flood.

Figure 4.2-4 provides a convenient form to organize culvert design calculations.

PROJECT: ________________	STATION: ________________	CULVERT DESIGN FORM
________________	SHEET ______ OF ______	DESIGNER / DATE: ______ OF ______ REVIEWER / DATE: ______ OF ______

HYDROLOGICAL DATA

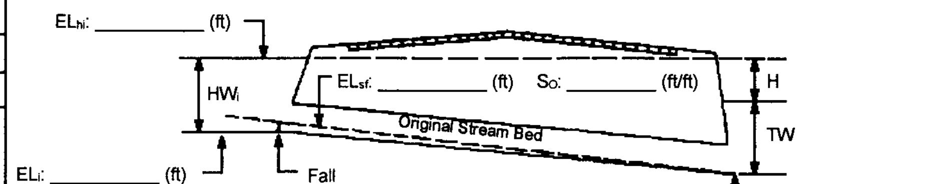
- ☐ METHOD: ________________
- ☐ DRAINAGE AREA: ________ ☐ STREAM SLOPE: ________
- ☐ CHANNEL SHAPE: ________________
- ☐ ROUTING: ________ ☐ OTHER: ________

SEE ADD'L SHTS.

DESIGN FLOWS/TAILWATER

R.I. (YEARS)	FLOW (cfs)	TW (ft)

ROADWAY ELEVATION: ____________ (ft)



$S = S_o - Fall/L_o$

S = ________ (ft/ft)

L_o = ________ (ft)

CULVERT DESCRIPTION: MATERIAL-SHAPE-SIZE-ENTRANCE	TOTAL FLOW Q (cfs)	FLOW PER BARREL Q/N (1)	HEADWATER CALCULATIONS: INLET CONTROL HW/D (2)	INLET CONTROL HW_i	INLET CONTROL FALL (3)	INLET CONTROL EL_{hi} (4)	OUTLET CONTROL TW (5)	OUTLET CONTROL d_c	OUTLET CONTROL $\frac{d_c + D}{2}$	OUTLET CONTROL h_o (6)	OUTLET CONTROL k_e	OUTLET CONTROL H (7)	OUTLET CONTROL EL_{ho} (8)	CONTROL HEADWATER ELEVATION	OUTLET VELOCITY	COMMENTS

TECHNICAL FOOTNOTES:

(1) USE Q/NB FOR BOX CULVERTS

(2) $HW_i/D = HW/D$ OR HW_f/D FROM DESIGN CHARTS

(3) FALL = $HW_i - (EL_{hd} - EL_{sf})$; FALL IS ZERO FOR CULVERTS ON GRADE

(4) $EL_{hi} = HW_i + EL_i$ (INVERT OF INLET CONTROL SECTION)

(5) TW BASED ON DOWN STREAM CONTROL OR FLOW DEPTH IN CHANNEL

(6) h_o = TW or $(d_c + D)/2$ (WHICHEVER IS GREATER)

(7) $H = [1+k_e+(29 n^2L)/R^{1.33}]\ V^2/2g$

(8) $EL_{ho} = EL_o + H + h_o$

SUSCRIPT DEFINITIONS:

a. Approximate
f. Culvert Face
hd. Design Headwater
hi. Headwater in Inlet Control
ho. Headwater in Outlet Control
i. Inlet Control Section
o. Outlet
sf. Streamed at Culvert Face
tw. Tailwater

COMMENTS / DISCUSSION:

CULVERT BARREL SELECTED:

SIZE: ________________

SHAPE: ________________

MATERIAL: ________________ n ______

ENTRANCE: ________________

Figure 4.2-4 Culvert Design Calculation Form

4.2.6 – Design Procedures for Beveled-Edged Inlets

(This procedure is acceptable for culverts with drainage areas less than 10 acres and preliminary sizing of culverts with larger drainage areas.)

4.2.6.1 Introduction

Improved inlets include inlet geometry refinements beyond those normally used in conventional culvert design practice. Several degrees of improvements are possible, including bevel-edged, side-tapered, and slope-tapered inlets. Those designers interested in using side- and slope-tapered inlets should consult the detailed design criteria and example designs outlined in the U. S. Department of Transportation publication Hydraulic Design Series No. 5 entitled, Hydraulic Design of Highway Culverts.

4.2.6.2 Design Figures

Four inlet control figures for culverts with beveled edges are found in Appendix D of HDS No. 5.

Chart	Page	Use for
3	D-3A & B	circular pipe culverts with beveled rings
10	D-10A & B	90º headwalls (same for 90 º wingwalls)
11	D-11A & B	skewed headwalls
112	D-12A & B	wingwalls with flare angles of 18 to 45 degrees

The following symbols are used in these figures:

B – Width of culvert barrel or diameter of pipe culvert

D – Height of box culvert or diameter of pipe culvert

H_f – Depth of pool or head, above the face section of invert

N – Number of barrels

Q – Design discharge

4.2.6.3 Design Procedure

The figures for bevel-edged inlets are used for design in the same manner as the conventional inlet design nomographs discussed earlier. Note that Charts 10, 11, and 12 in subsection 4.2.8 apply only to bevels having either a 33º angle (1.5:1) or a 45º angle (1:1).

For box culverts the dimensions of the bevels to be used are based on the culvert dimensions. The top bevel dimension is determined by multiplying the height of the culvert by a factor. The side bevel dimensions are determined by multiplying the width of the culvert by a factor. For a

1:1 bevel, the factor is 0.5 inch/ft. For a 1.5:1 bevel the factor is 1 inch/ft. For example, the minimum bevel dimensions for an 8 ft x 6 ft box culvert with 1:1 bevels would be:

Top Bevel = d = 6 ft x 0.5 inch/ft = 3 inches

Side Bevel = b = 8 ft x 0.5 inch/ft = 4 inches

For a 1.5:1 bevel computations would result in d = 6 and b = 8 inches.

4.2.6.4 Design Figure Limits

The improved inlet design figures are based on research results from culvert models with barrel width, B, to depth, D, ratios of from 0.5:1 to 2:1. For box culverts with more than one barrel, the figures are used in the same manner as for a single barrel, except that the bevels must be sized on the basis of the total clear opening rather than on individual barrel size.

For example, in a double 8 ft by 8 ft box culvert:

Top Bevel is proportioned based on the height of 8 feet, which results in a bevel of 4 in. for the 1:1 bevel and 8 in. for the 1.5:1 bevel.

Side Bevel is proportioned based on the clear width of 16 feet, which results in a bevel of 8 in. for the 1:1 bevel and 16 in. for the 1.5:1 bevel.

4.2.6.5 Multibarrel Installations

For multibarrel installations exceeding a 3:1 width to depth ratio, the side bevels become excessively large when proportioned on the basis of the total clear width. For these structures, it is recommended that the side bevel be sized in proportion to the total clear width, B, or three times the height, whichever is smaller.

The top bevel dimension should always be based on the culvert height.

The shape of the upstream edge of the intermediate walls of multibarrel installations is not as important to the hydraulic performance of a culvert as the edge condition of the top and sides. Therefore, the edges of these walls may be square, rounded with a radius of one-half their thickness, chamfered, or beveled. The intermediate walls may also project from the face and slope downward to the channel bottom to help direct debris through the culvert.

Multibarrel pipe culverts should be designed as a series of single barrel installations since each pipe requires a separate bevel.

4.2.6.6 Skewed Inlets

It is recommended that Chart 11 for skewed inlets not be used for multiple barrel installations, as the intermediate wall could cause an extreme contraction in the downstream barrels. This would result in underdesign due to a greatly reduced capacity. Skewed inlets (at an angle with the centerline of the stream) should be avoided whenever possible and should not be used with side- or slope-tapered inlets. It is important to align culverts with streams in order to avoid erosion problems associated with changing the direction of the natural stream flow.

4.2.7 – Flood Routing and Culvert Design

4.2.7.1 Introduction

Flood routing through a culvert is a practice that evaluates the effect of temporary upstream ponding caused by the culvert's backwater. By not considering flood routing it is possible that the findings from culvert analyses will be conservative. If the selected allowable headwater is accepted without flood routing, then costly over-design of both the culvert and outlet protection may result, depending on the amount of temporary storage involved. However, if storage is used in the design of culverts, consideration should be given to:

- The total area of flooding,
- The average time that bankfull stage is exceeded for the design flood up to 48 hours in rural areas or 6 hours in urban areas, and
- Ensuring that the storage area will remain available for the life of the culvert through the purchase of right-of-way or easement.

4.2.7.2 Design Procedure

The design procedure for flood routing through a culvert is the same as for reservoir routing. The site data and roadway geometry are obtained and the hydrology analysis completed to include estimating a hydrograph. Once this essential information is available, the culvert can be designed. Flood routing through a culvert can be time consuming. It is recommended that a computer program be used to perform routing calculations; however, an engineer should be familiar with the culvert flood routing design process.

A multiple trial and error procedure is required for culvert flood routing. In general:

Step 1 A trial culvert(s) is selected

Step 2 A trial discharge for a particular hydrograph time increment (selected time increment to estimate discharge from the design hydrograph) is selected

Step 3 Flood routing computations are made with successive trial discharges until the flood routing equation is satisfied

Step 4 The hydraulic findings are compared to the selected site criteria

Step 5 If the selected site criteria are satisfied, then a trial discharge for the next time increment is selected and this procedure is repeated; if not, a new trial culvert is selected and the entire procedure is repeated.

4.2.7.3 Comprehensive Design Guidance

Comprehensive design discussions and guidance may be found in the Federal Highway Administration, National Design Series No. 5, document entitled Hydraulic Design of Highway Culverts, Second Edition, published in 2001. This document is available from the FHWA as Publication Number NHI-01-020.
(http://www.fhwa.dot.gov/engineering/hydraulics/library_arc.cfm?pub_number=7&id=13).

Section 4.3 – Bridge Design

4.3.1 Overview

Bridges enable streams to maintain flow conveyance and to sustain aquatic life. They are important and expensive highway hydraulic structures vulnerable to failure from flood related causes. In order to minimize the risk of failure, the hydraulic requirements of a stream crossing must be recognized and considered during the development, construction, and maintenance highway phases.

This section addresses structures designed hydraulically as bridges, regardless of length. For economy and hydraulic efficiency, engineers should design culverts to operate with the inlet submerged during flood flows, if conditions permit. Bridges, on the other hand, are not covered with embankment or designed to take advantage of submergence to increase hydraulic capacity, even though some are designed to be inundated under flood conditions. This discussion of bridge hydraulics considers the total crossing, including approach embankments and structures on the floodplains.

The following subsections present considerations related to the hydraulics of bridges. It is generally excerpted from Chapter 9 of the Texas Department of Transportation (TxDOT) Hydraulics Design Manual dated March 2004.

4.3.1.1 Bridge Hydraulics Considerations

When beginning analysis for a cross-drainage facility, the flood frequency and stage-discharge curves should first be established, as well as the type of cross-drain facility. The choice is usually between a bridge and a culvert. Bridges are usually chosen if the discharge is significant or if the stream to be crossed is large in extent. Both types of facilities should be evaluated and a choice made based on performance and economics. If the stream crossing is wide with multiple concentrations of flow, a multiple opening facility may be in order.

4.3.1.2 Highway-Stream Crossing Analysis

The hydraulic analysis of a highway-stream crossing for a particular flood frequency involves:

- Determining the backwater associated with each alternative profile and waterway opening(s)
- Determining the effects on flow distribution and velocities
- Estimating scour potential

The hydraulic design of a bridge over a waterway involves the following such that the risks associated with backwater and increased velocities are not excessive:

- Establishing a location
- Bridge length

- Orientation
- Roadway and bridge profiles

A hydrologic and hydraulic analysis is recommended for designing all new bridges over waterways, bridge widening, bridge replacement, and roadway profile modifications that may adversely affect the floodplain, even if no structural modifications are necessary. Typically, this should include the following:

- An estimate of peak discharge (sometimes complete runoff hydrographs)
- Existing and proposed condition water surface profiles for design and check flood conditions
- Consideration of the potential for stream stability problems and scour potential.

4.3.1.3 Freeboard

Navigational clearance and other reasons notwithstanding, the low chord elevation is defined as the sum of the design normal water surface elevation (high water) and a *freeboard*. For on system TxDOT bridges, TxDOT recommends a minimum freeboard of 2 ft (0.6 m) to allow for passage of floating debris and to provide a safety factor for design flood flow. Higher freeboards may be appropriate over streams that are prone to heavy debris loads, such as large tree limbs, and to accommodate other clearance needs. Other constraints may make lower freeboards desirable, but the low chord should not impinge on the design high water. Generally, for off-system bridge replacement structures, the low chord should approximate that of the structure to be replaced, unless the results of a risk assessment indicate a different structure is the most beneficial option.

4.3.1.4 Roadway/Bridge Profile

A bridge is integrated into both the stream and the roadway and must be fully compatible with both. Therefore, the alignment of the roadway and the bridge are the same between the ends of the bridge. Hydraulically, the complete bridge profile can be any part of the structure that stream flow can strike or impact in its movement downstream. If the stream gets high enough to inundate the structure, then all parts of the roadway and the bridge become part of the complete bridge profile.

For TxDOT design, the roadway must not be inundated by the design flood, but inundation by the 100-year flood is allowed. Unless the route is an emergency escape route, it is often desirable to allow floods in excess of the design flood to overtop the road. This helps minimize both the backwater and the required length of structure.

Several vertical alignment alternatives are available for consideration, depending on site topography, traffic requirements, and flood damage potential. The alternatives range from crossings that are designed to overtop frequently to crossings that are designed to rarely or never overtop.

4.3.1.4 Crossing Profile

The horizontal alignment of a highway at a stream crossing should be taken into consideration when selecting the design and location of the waterway opening as well as the crossing profile. Every effort should be made to align the highway so that the crossing will be normal to the stream flow direction (highway centerline perpendicular to the streamline).

Often, this is not possible because of the highway or stream configuration. When a skewed structure is necessary, it should be ensured that substructure fixtures such as foundations, columns, piers, and bent caps offer minimum resistance to the stream flow.

Bent caps should be oriented as near to the skew of the streamlines at flood stage as possible. Headers should be skewed to minimize eddy-causing obstructions. A relief opening may be provided at the approximate location of point A to reduce the likelihood of trapped flow and minimize the amount of low that would have to travel up against the general direction of flow along the embankment.

4.3.1.5 Single Versus Multiple Openings

For a single structure, the flow will find its way to an opening until the roadway is overtopped. If two or more structures have flow area available, after accumulating a head, the flow will divide and proceed to the structures offering the least resistance. The point of division is called a stagnation point.

In usual practice, the TxDOT recommends that the flood discharge be forced to flow parallel to the highway embankment for no more than about 800 ft (240 m). If flow distances along the embankment are greater than recommended, an additional relief structure or opening should be considered. A possible alternative to the provision of an additional structure is a guide bank (spur dike) to control the turbulence at the header. Also, natural vegetation between the toe of slope and the right-of-way line is useful in controlling flow along the embankment. Therefore, special efforts should be made to preserve any natural vegetation in such a situation.

4.3.1.6 Factors Affecting Bridge Length

The discussions of bridge design assume normal cross sections and lengths (perpendicular to flow at flood stage). Usually one-dimensional flow is assumed, and cross sections and lengths are considered 90° to the direction of stream flow at flood stage.

If the crossing is skewed to the stream flow at flood stage, all cross sections and lengths should be normalized before proceeding with the bridge length design. If the skew is severe and the floodplain is wide, the analysis may need to be adjusted to offset the effects of elevation changes within the same cross section.

The following examples illustrate various factors that can cause a bridge opening to be larger than that required by hydraulic design.

- Bank protection may be placed in a certain location due to local soil instability or a high bank.
- Bridge costs may be cheaper than embankment costs.
- A highway profile grade line might dictate an excessive freeboard allowance. For sloping abutments, a higher freeboard will result in a longer bridge.
- High potential for meander to migrate, or other channel instabilities may warrant a longer opening.

4.3.2 – Symbols and Definitions

The hydraulics of bridge openings are basically the same as those of open channel flow. Therefore, the symbols and definitions are essentially the same as those of Section 4.4.2 presented in Table 4.4-1. There are other definitions unique to bridges which are presented here. They are defined in the TxDOT Hydraulic Design Manual.

4.3.2.1 Flow Zones and Energy Losses

Figure 4.3-1 shows a plan of typical cross section locations that establish three flow zones that should be considered when estimating the effects of bridge openings.

Zone 1 represents the area between the downstream face of the bridge and a cross section downstream of the bridge within which expansion of flow from the bridge is expected to occur. The distance over which this expansion occurs can vary depending on the flow rate and the floodplain characteristics. No detailed guidance is available, but a distance equal to about four times the length of the average embankment constriction is reasonable for most situations. Section 1 represents the effective channel flow geometry at the end of the expansion zone, which is also called the “exit” section. Cross sections 2 and 3 are at the toe of roadway embankment and represent the portion of unconstricted channel geometry that approximates the effective flow areas near the bridge opening as shown in Figure 4.3-2.

Zone 2 represents the area under the bridge opening through which friction, turbulence, and drag losses are considered. Generally, the bridge opening is obtained by superimposing the bridge geometry on cross sections 2 and 3.

Zone 3 represents an area from the upstream face of the bridge to a distance upstream where the contraction of flow must occur. A distance upstream of the bridge equal to the length of the average embankment constriction is a reasonable approximation of the location at which contraction begins. Cross section 4 represents the effective channel flow geometry where contraction begins. This is sometimes referred to as the “approach” cross section.

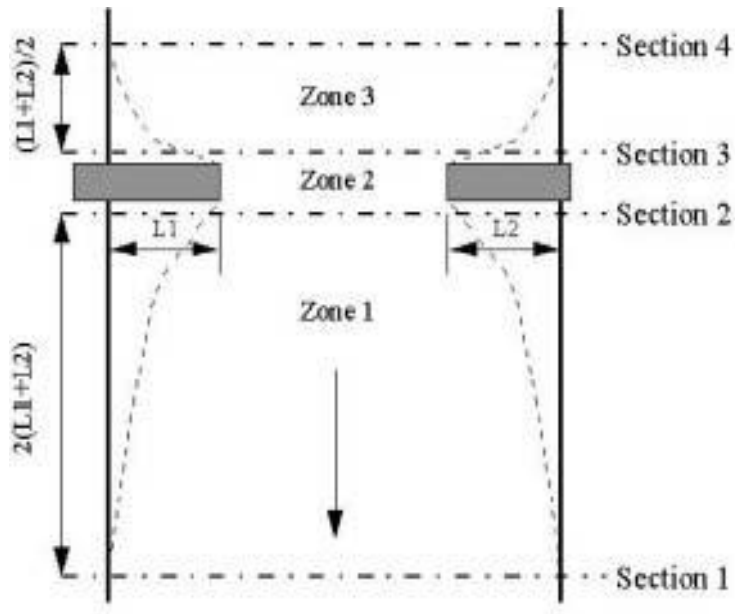


Figure 4.3-1 Flow Zones at Bridges

(TxDOT Hydraulic Design Manual)

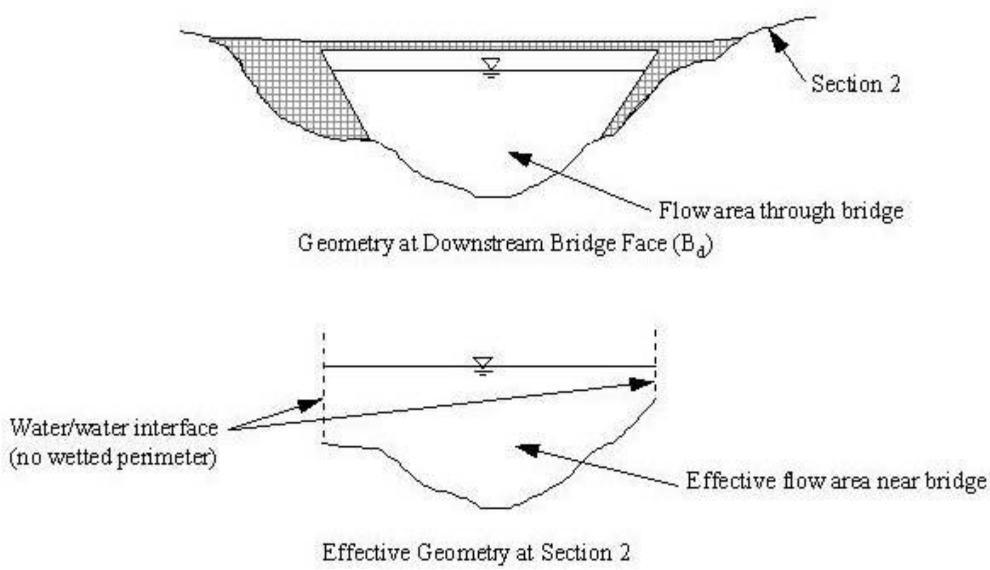


Figure 4.3-2 Effective Geometry for Bridge (Section 2 shown, Section 3 similar)

(TxDOT Hydraulic Design Manual)

4.3.2.2 Bridge Flow Class

The losses associated with flow through bridges depend on the hydraulic conditions of low or high flow.

Low flow describes hydraulic conditions in which the water surface between Zones 1, 2, and 3 is open to atmospheric pressure. That means the water surface does not impinge upon the superstructure. (This condition should exist for the design frequency of all new on-system bridges.) Low flow is divided into categories as described in the "Low Flow Classes" table below Type I is the most common in Texas, although severe constrictions compared to the flow conditions could result in Types IIA and IIB. Type III is likely to be limited to steep hills and mountainous regions.

Low Flow Class	Description
I	Subcritical flow through all Zones
IIA	Subcritical flow through Zones 1 and 3; flow through critical depth in Zone 2
IIB	Subcritical flow through Zone 3; flow through critical depth in Zone 2, hydraulic jump in Zone 1
III	Supercritical flow through all Zones

High flow refers to conditions in which the water surface impinges on the bridge superstructure:

- When the tailwater does not submerge the low chord of the bridge, the flow condition is comparable to a pressure flow sluice gate.
- At the tailwater, which submerges the low chord but does not exceed the elevation of critical depth over the road, the flow condition is comparable to orifice flow.
- If the tailwater overtops the roadway, neither sluice gate flow nor orifice flow is reasonable, and the flow is either weir flow or open flow.

4.3.3 – Design Criteria

The design of a bridge should take into account many different engineering and technical aspects at the bridge site and adjacent areas. The following design criteria should be considered for all bridge designs as applicable.

4.3.3.1 Frequency Flood

Design discharges chosen by TxDOT for bridges vary with the functional classification and structure type. For major river crossings, a return period of 50 years is recommended. Flow small bridges, the recommended return period is 25 years. <u>In all cases the check flood is for the 100-year return period.</u>

4.3.3.2 Freeboard

Typical freeboard, the length between the computed design water surface and the low chord, is two feet from the 100-yr discharge. In urban settings, it may be prudent to use the 100-year fully-developed discharge to check the bridge design.

4.3.3.3 Loss Coefficients

The contraction and expansion of water through the bridge opening creates hydraulic losses. These losses are accounted for through the use of loss coefficients. Table 4.3-1 gives recommended values for the Contraction (K_c) and Expansion (K_e) Coefficients.

Table 4.3-1 Recommended Loss Coefficients for Bridges

Transition Type	Contraction (Kc)	Expansion (Ke)
No Losses Computed	0.0	0.0
Gradual Transition	0.1	0.3
Typical Bridge	0.3	0.5
Severe Transition	0.6	0.8

4.3.4 – Design Procedures

The following is a general bridge hydraulic design procedure.

1. Determine the most efficient alignment of proposed roadway, attempting to minimize skew at the proposed stream crossing.
2. Determine design discharge from hydrologic studies or available data (Town, Federal Emergency Management Agency (FEMA), US Army Corp of Engineers (USACE), TxDOT, or similar sources).
3. If available, obtain effective FEMA hydraulic backwater model. It is assumed that if a bridge is required instead of a culvert, the drainage area would exceed one square mile and could already be included in a FEMA study. If an effective FEMA model or other model is not available, a basic hydrologic model and backwater analysis for the stream must be prepared. The HEC-RAS computer model is routinely used to compute backwater water surface profiles.
4. Using USACE or FEMA guidelines, compute or duplicate an existing conditions water surface profile for the design storm(s). Compute a profile for the fully-developed watershed, for use as a baseline for design of a new bridge/roadway crossing.
5. Use the design discharge to compute an approximate opening that will be needed to pass the design storm (for preliminary sizing, use a normal-depth design procedure, or simply estimate a required trapezoidal opening).
6. Prepare a bridge crossing data set in the hydraulic model to reflect the preliminary design opening, which includes the required freeboard and any channelization upstream or downstream to transition the floodwaters through the proposed structure.
7. Compute the proposed bridge flood profile and design parameters (velocities, flow distribution, energy grade, etc.). Review for criteria on velocities and freeboard, and revise model as needed to accommodate design flows.
8. Review the velocities and determine erosion control requirements downstream, through the structure, and upstream.

9. Finalize the design size and erosion control features, based on comparing the proposed model with the existing conditions profiles, impacts on other properties, FEMA guidelines, and Local Criteria.
10. Exceptions/Other Issues
    A. Conditional Letter of Map Amendment (CLOMR) may be needed for new crossings of streams studied by FEMA.
    B. If applicable, coordinate with USACE Regulatory Permit requirements.
    C. Evaluate the project with respect to the Town of Copper Canyon policy regarding downstream impacts.
    D. Design should be for fully developed watershed conditions. If the available discharges are from FEMA existing conditions hydrology, the following options are available: (1) Obtain new hydrology, (2) Extrapolate fully-developed from existing data, or (3) Variance from the local jurisdiction on design discharges
    E. Freeboard criteria may require an unusually expensive bridge or impracticable roadway elevation. A reasonable variance in criteria may be available with Town approval.

Section 4.4 – Open Channel Design

4.4.1 – Overview

4.4.1.1 Introduction

Open channel systems and their design are an integral part of storm water drainage design, particularly for development sites utilizing better site design practices and open channel structural controls. Open channels include drainage ditches, grass channels, dry and wet enhanced swales, stone riprap channels and concrete-lined channels.

The purpose of this section is to provide an overview of open channel design criteria and methods, including the use of channel design nomographs.

4.4.1.2 Open Channel Types

The three main classifications of open channel types according to channel linings are vegetated, flexible, and rigid. Vegetated linings include grass with mulch, sod and lapped sod, and wetland channels. Stone riprap and some forms of flexible man-made linings or gabions are examples of flexible linings, while rigid linings are generally concrete or rigid block.

<u>Vegetative Linings</u> – Vegetation, where practical, is the most desirable lining for an artificial channel. It stabilizes the channel body, consolidates the soil mass of the bed, checks erosion on the channel surface, provides habitat, and provides water quality benefits (see Section 1.4 for more details on using enhanced swales and grass channels for water quality purposes).

Conditions under which vegetation may not be acceptable include but are not limited to:

- High velocities
- Standing or continuously flowing water
- Lack of regular maintenance necessary to prevent growth of taller or woody vegetation
- Lack of nutrients and inadequate topsoil
- Excessive shade

Proper seeding, mulching, and soil preparation are required during construction to assure establishment of healthy vegetation.

<u>Flexible Linings</u> – Rock riprap, including rubble, is the most common type of flexible lining for channels. It presents a rough surface that can dissipate energy and mitigate increases in erosive velocity. These linings are usually less expensive than rigid linings and have self-healing qualities that reduce maintenance. However, they may require the use of a filter fabric depending

on the underlying soils, and the growth of grass, weeds, and trees may present maintenance problems.

Rigid Linings – Rigid linings are generally constructed of concrete and used where high flow capacity is required. Higher velocities, however, create the potential for scour at channel lining transitions and channel headcutting.

Normal Depth (Uniform Flow) vs. Backwater Profile Depths:

For uniform flow calculations, the theoretical channel dimensions, computed by the slope-area methods are generally to be used only for an initial dimension in the design of an improved channel. Exceptions will be for small outfall channels (with the approval of TOWN ENGINEER) meeting the following criteria:

1. Drainage area 10 acres or less.
2. Completely contained on the development site ;
3. No nearby downstream restrictions (no significant backwater effects).
4. Flow conditions consistent with uniform flow assumption.

Town of Copper Canyon requires a HEC-RAS backwater/frontwater analysis on any proposed open channel with a drainage area greater than 10 acres to determine the actual tailwater elevations, channel capacity and freeboard, and impacts on adjacent floodplains. If the current effective FEMA model for the stream is a HEC-2 model, the engineer has the option to either use that model, or convert to HEC-RAS for analysis of proposed conditions.

Supercritical Flow Regime

Supercritical flow will not be allowed except under unusual circumstances, with special approval of the TOWN ENGINEER. However, for lined channels the analysis should include a mixed-flow regime analysis, to make sure no supercritical flow occurs. Town of Copper Canyon requires that the computed flow depths in designed channels be outside of the range of instability, i.e. depth of flow should be at least 1.1 times critical depth.

Channel Transitions or Energy Dissipation Structures or Small Dams

A HEC-RAS model is a standard requirement for design of channel transitions (upstream and downstream), energy dissipation structures, and small dams. A backwater analysis will be required by the Town, to determine accurate tailwater elevation, headlosses, headwater elevations and floodplains affected by the proposed transition into and out of an improved channel, any on-stream energy dissipating structures, and small dams (less than 6 feet). If the current effective FEMA model for the stream is a HEC-2 model, the engineer has the option to either use that model, or convert to HEC-RAS for analysis of proposed conditions. For larger dams, a hydrologic routing will be required, as well as hydraulic analysis, to determine impacts of the proposed structure on existing floodplains and adjacent properties.

4.4.2 - Symbols and Definitions

To provide consistency within this section as well as throughout this Manual, the symbols listed in Table 4.4-1 will be used. These symbols were selected because of their wide use. In some cases, the same symbol is used in existing publications for more than one definition. Where this occurs in this section, the symbol will be defined where it occurs in the text or equations.

Table 4.4-1 Symbols and Definitions

Symbol	**Definition**	**Units**
α	Energy coefficient	-
A	Cross-sectional area	ft^2
b	Bottom width	ft
C_g	Specific weight correction factor	-
D or d	Depth of flow	ft
d	Stone diameter	ft
delta d	Super-elevation of the water surface profile	ft
d_x	Diameter of stone for which x percent, by weight, of the gradation is finer	ft
E	Specific energy	ft
Fr	Froude Number	-
g	Acceleration of gravity	32.2 ft/s^2
h_{loss}	Head loss	ft
K	Channel conveyance	-
k_e	Eddy head loss coefficient	ft
K_T	Trapezoidal open channel conveyance factor	-
L	Length of channel	ft
L_p	Length of downstream protection	ft
n	Manning's roughness coefficient	-
P	Wetted perimeter	ft
Q	Discharge rate	cfs
R	Hydraulic radius of flow	ft
R_c	Mean radius of the bend	ft
S	Slope	ft/ft
SW_s	Specific weight of stone	lbs/ft^3
T	Top width of water surface	ft
V or v	Velocity of flow	ft/s
w	Stone weight	lbs
y_c	Critical depth	ft
y_n	Normal depth	ft
z	Critical flow section factor	-

4.4.3 – Design Criteria

4.4.3.1 General Criteria

Earthen Channels

The Town of Copper Canyon encourages the preservation of natural drainageways or use of constructed vegetated or permeable channels designed to create a more natural environment.

1. An earthen channel shall have a trapezoidal shape with side slopes not steeper than a 4:1 ratio and a channel bottom at least eight (8) feet in width.
2. One (1) foot of freeboard must be provided, within drainage easements, above the 100-year fully developed water surface elevation at all locations along channels.
3. The side slopes and bottom of an earthen channel shall be smooth, free of rocks, and contain a minimum of six (6) inches of topsoil. The side slopes and channel bottom shall be re-vegetated with grass or other acceptable vegetative material. No channel shall be accepted by the Town until a uniform (e.g., evenly distributed, without large bare areas) vegetative cover at least 2" in height with a density of 80% has been established.
4. Each reach of a channel requiring vehicular access for maintenance must have a ramp. In general, reaches with maintenance access ramps should be located between bridges or culverts but individual situations may vary. Ramps shall be at least ten (10) feet wide and have 15% maximum grade. Twelve-foot (12') width is required if the ramp is bound by vertical walls.
5. Minimum channel slope is 0.0020 ft/ft unless approved by the TOWN ENGINEER.
6. Erosion protection to be provided at upper limits of improvements and outfall to the receiving stream.
7. All improved earthen channels shall include either " Composite Low Flow" channel or "Trickle" channel. Criteria for each of these channels is as follows:
    a. Low Flow Composite Channels-
        1) Drainage area greater than 300 acres.
        2) Minimum design discharge - 2% of fully developed 100 year peak discharge.
        3) Maximum depth - 5 feet. Maximum side slope 4:1 (H: V).
        4) Minimum bottom width- 8 feet unless approved by the TOWN ENGINEER.
        5) Lined with riprap or gabions if design velocity exceeds 5 feet/second (also see iSWM sections 4.4.3 and 4.4.4).
        6) Some meanders in alignment acceptable as long as width of shelf between top of bank of low flow channel and toe of slope of main channel is not less than 10 feet. Minimum lateral slope of shelf is 1%.

    b. Trickle Channels-
        1) Drainage area less than or equal to 300 acres.
        2) Design discharge - 2% of fully developed 100 year peak discharge.
        3) Concrete or permeable armor such as gabions, mat or interlocking block-lined.
        4) Minimum bottle width – 8 feet unless approved by the TOWN ENGINEER
        5) Maximum depth -5 feet. Maximum side slope dependent on type of lining.

8. The following guidelines shall be considered for buffer areas or zones along natural or constructed earthen channels:

a. A minimum Erosion Control Setback on each side of natural channels based on a 4:1 (H:V) slope from the bottom of the bank to the natural ground adjacent to the bank plus an additional 15 feet. See Figure 4.4.-1A.

b. Include adjacent delineated wetlands or critical habitats.

c. Other buffer widths will be considered if supported by specific engineering and environmental studies.

9. Landscaping shall be installed to allow earthen channels to evolve into a more natural environment. Tree or shrub plantings will be required to enhance habitat of channels by providing shade once mature plant growth has been reached. Mature plantings must be considered in setting design Manning's "n" values.

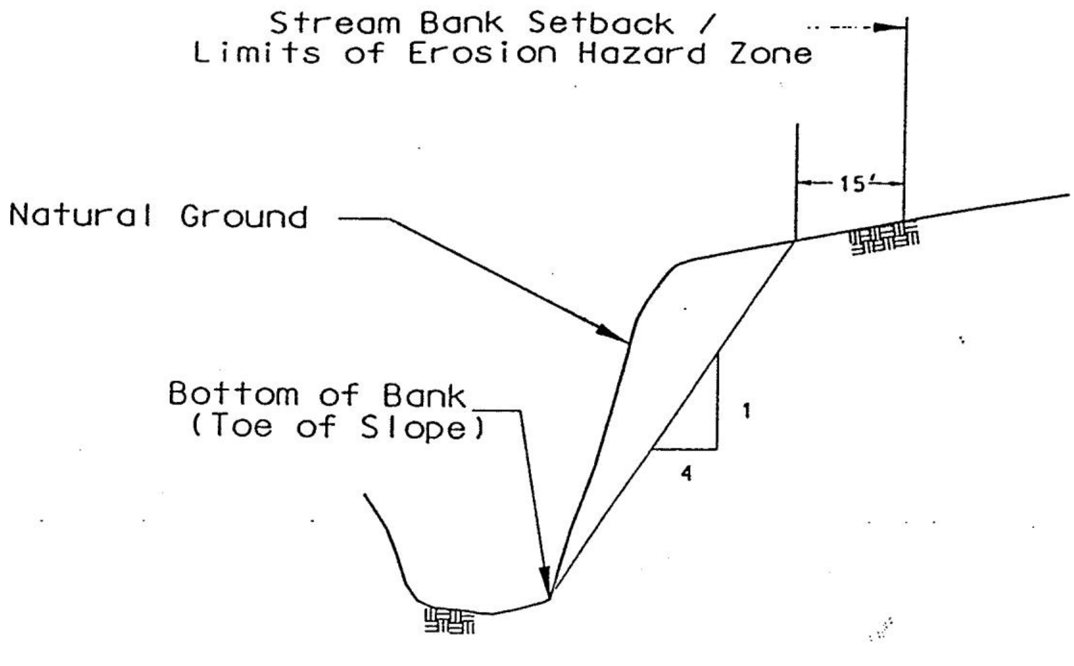


Figure 4.4.-1A Minimum Erosion Control Setback

Lined Channels

In general, lined channels are discouraged and must have approval of the TOWN ENGINEER.

1. Channels shall be trapezoidal in shape and lined with reinforced concrete (or flexible lining material as approved by the TOWN ENGINEER). Side slopes shall generally be no steeper than 1.5:1 unless approved by the TOWN ENGINEER as appropriate for the lining material. The lining shall extend to and include the water surface elevation of the 100 year fully developed storm plus one foot freeboard.

2. The channel bottom must be a minimum of 8' in width. (A minimum bottom width of 6 feet for overflow structures of storm sewer system sumps or where access is not a concern).

3. The maximum water flow velocity in a lined channel shall be fifteen (15) feet per second except that the water flow shall not be supercritical in an area from 100' upstream from a bridge to 25' downstream from a bridge. Hydraulic jumps shall not

be allowed from the face of a culvert to 50’ upstream from that culvert. In general channels having supercritical flow conditions are discouraged (See Section 4.4).

4. Whenever flow changes from supercritical to subcritical channel protection shall be provided to protect from the hydraulic jump that is anticipated (see comment in Item 3).

5. The design of the channel lining shall take into account the super elevation of the water surface around curves and other changes in direction.

6. A chain link fence six (6) feet in height or other fence as approved by the TOWN ENGINEER may be required on each side of a lined channel.

7. The TOWN ENGINEER may require a geotechnical study and /or an underground drainage system design option prior to approval of concrete lined channels.

Roadside Ditches

Design Considerations

1. The design storm for the roadside ditches shall be the 10-year storm. The 100-year flow shall not exceed the right-of-way capacity defined as the natural ground at the right-of-way line or top of roadside ditch.

2. For grass lined sections, the maximum design velocity shall be 6.0 feet per second during the 100-year design storm (Higher velocities justified by a sealed geotechnical study).

3. Minimum grades for roadside ditches shall be 0.0040 foot/foot (0.40%).

4. Manning’s roughness coefficient for analysis and design of roadside ditches are presented in Section 4.4.4.

5. Erosion protection will be provided at the upstream and downstream ends of all culverts.

6. Maximum depth will not exceed 4 feet from center-line of pavement except as specifically approved by TOWN ENGINEER.

7. If the ditch extends beyond the right-of-way line, an additional drainage easement shall be dedicated extending at least 2 feet beyond the top of bank. Utility easements must be separate and beyond any drainage easements.

8. Hydraulic analysis of roadside ditches will require a HEC-RAS analysis.

Culverts in Roadside Ditches

1. Culverts will be placed at all driveway and roadway crossings and other locations where appropriate.

2. Roadside culverts are to be sized based on drainage area, assuming inlet control. Calculations are to be provided for each block based on drainage calculations. The size of culvert used shall not create a head loss of more than 0.20 feet greater than the normal water surface profile without the culvert.

3. Roadside ditch culverts will be no smaller than 24 inches inside diameter or equivalent for roadway crossings and 18 inches for driveway culverts.

4. A driveway culvert schedule shall be included on the face of the plat. It shall include for each lot approximate culvert flowline depth below top of pavement, number and size of pipe required, and horizontal distance from edge of pavement to center of culvert (based on horizontal control requirements above).

4.4.3.2 Velocity Limitation

The final design of artificial open channels should be consistent with the velocity limitations for the selected channel lining. Recommended maximum velocity values for selected lining categories are presented in Table 4.4-2. Seeding and mulch should only be used when the design value does not exceed the allowable value for bare soil. Velocity limitations for vegetative linings are reported in Table 4.4-3. Vegetative lining calculations are presented in Section 4.4.7 and stone riprap procedures are presented in Section 4.4.8.

Typically, local design limits the velocity to 15 fps in concrete lined channels. For gabions typical design velocities range from 10 fps for 6-inch mattresses up to 15 fps for 1-foot mattresses. Some manufacturers indicate that velocities of 20 fps are allowable for basket installations. No specific velocity limit is appropriate for rock riprap. The design of stable riprap lining depends upon the intersection of the velocity (local boundary shear) and the size and gradation of the riprap material. In general, velocity limitations should be set by the local jurisdiction.

4.4.4 – Manning's n Values

The Manning's n value is an important variable in open channel flow computations. Variation in this variable can significantly affect discharge, depth, and velocity estimates. Since Manning's n values depend on many different physical characteristics of natural and man-made channels, care and good engineering judgment must be exercised in the selection process.

Recommended Manning's n values for artificial channels with rigid, unlined, temporary, and stone riprap linings are given in Table 4.4-4. Recommended values for vegetative linings should be determined using Figure 4.4-1B, which provides a graphical relationship between Manning's n values and the product of velocity and hydraulic radius for several vegetative retardance classifications (see Table 4.4-6). Figure 4.4-1B is used iteratively as described in Section 4.4.7. Recommended Manning's values for natural channels that are either excavated or dredged, and natural are given in Table 4.4-5. For natural channels, Manning's n values should be estimated using experienced judgment and information presented in publications such as the *Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains*, FHWA-TS-84-204, 1984, FHWA HEC-15, 1988, or Chow, 1959. Some of these values are given in Table 4.4-2 below.

Table 4.4-2 Roughness Coefficients (Manning's n) and Allowable Velocities for Natural Channels

Channel Description	**Manning's n**	**Maximum Permissible Channel Velocity (ft/s)**
MINOR NATURAL STREAMS		
Fairly regular section		
1. Some grass and weeds; little or no brush	0.030	3 to 6
2. Dense growth of weeds, depth of flow materially greater than weed height	0.035	3 to 6
3. Some weeds, light brush on banks	0.035	3 to 6
4. Some weeds, heavy brush on banks	0.050	3 to 6
5. Some weeds, dense willows on banks	0.060	3 to 6
For trees within channels with branches submerged at high stage, increase above values by	0.010	
Irregular section with pools, slight channel meander, increase above values by	0.010	
Floodplain – Pasture		
1. Short grass	0.030	3 to 6
2. Tall grass	0.035	3 to 6
Floodplain – Cultivated Areas		
1. No crop	0.030	3 to 6
2. Mature row crops	0.035	3 to 6
3. Mature field crops	0.040	3 to 6
Floodplain – Uncleared		
1. Heavy weeds scattered brush	0.050	3 to 6
2. Wooded	0.120	3 to 6
UNLINED VEGETATED CHANNELS		
Clays (Bermuda Grass)	0.035	5 to 6
Sandy and Silty Soils (Bermuda Grass)	0.035	3 to 5
UNLINED NON-VEGETATED CHANNELS		
Sandy Soils	0.030	1.5 to 2.5
Silts	0.030	0.7 to 1.5
Sandy Silts	0.030	2.5 to 3.0
Clays	0.030	3.0 to 5.0
Coarse Gravels	0.030	5.0 to 6.0
Shale	0.030	6.0 to 10.0
Rock	0.025	15

Table 4.4-3 Maximum Velocities for Vegetative Channel Linings		
Vegetation Type	**Slope Range (%)[1]**	**Maximum Velocity[2] (ft/s)**
Bermuda grass	0-5	6
Bahia		4
Tall fescue grass mixtures[3]	0-10	4
Kentucky bluegrass	0-5	6
Buffalo grass	5-10	5
	>10	4
Grass mixture	0-5[1]	4
	5-10	3
Sericea lespedeza, Weeping lovegrass, Alfalfa	0-5[4]	3
Annuals[5]	0-5	3
Sod		4
Lapped sod		5

[1] Do not use on slopes steeper than 10% except for side-slope in combination channel.

[2] Use velocities exceeding 5 ft/s only where good stands can be maintained.

[3] Mixtures of Tall Fescue, Bahia, and/or Bermuda

[4] Do not use on slopes steeper than 5% except for side-slope in combination channel.

[5] Annuals - used on mild slopes or as temporary protection until permanent covers are established.

Source: Manual for Erosion and Sediment Control in Georgia, 1996

Table 4.4-4 Manning's Roughness Coefficients (n) for Artificial Channels				
		Depth Ranges		
Category	***Lining Type***	**0-0.5 ft**	**0.5-2.0 ft**	**>2.0 ft**
Rigid	Concrete	0.015	0.013	0.013
	Grouted Riprap	0.040	0.030	0.028
	Stone Masonry	0.042	0.032	0.030
	Soil Cement	0.025	0.022	0.020
	Asphalt	0.018	0.016	0.016
Unlined	Bare Soil	0.023	0.020	0.020
	Rock Cut	0.045	0.035	0.025
Temporary*	Woven Paper Net	0.016	0.015	0.015
	Jute Net	0.028	0.022	0.019
	Fiberglass Roving	0.028	0.022	0.019
	Straw with Net	0.065	0.033	0.025
	Curled Wood Mat	0.066	0.035	0.028
	Synthetic Mat	0.036	0.025	0.021
Gravel Riprap	1-inch D_{50}	0.044	0.033	0.030
	2-inch D_{50}	0.066	0.041	0.034
Rock Riprap	6-inch D_{50}	0.104	0.069	0.035
	12-inch D_{50}	–	0.078	0.040

Note: Values listed are representative values for the respective depth ranges. Manning's roughness coefficients, n, vary with the flow depth.

*Some "temporary" linings become permanent when buried.

Source: HEC-15, 1988.

When designing open channels, the usual choice of Manning's roughness coefficients may be found in Table 4.4-5. The TOWN ENGINEER may choose to vary from these values.

Table 4.4-5 Manning's Roughness Coefficients for Design		
Lining Type	**Manning's n**	**Comments**
Grass Lined	0.035	Use for velocity check.
	0.050	Use for channel capacity check (freeboard check)
Concrete Lined	0.015	
Gabions	0.030	
Rock Riprap	0.040	$n = 0.0395 d_{50}^{1/6}$ where d_{50} is the stone size of which 50% of the sample is smaller
Grouted Riprap	0.028	FWHA

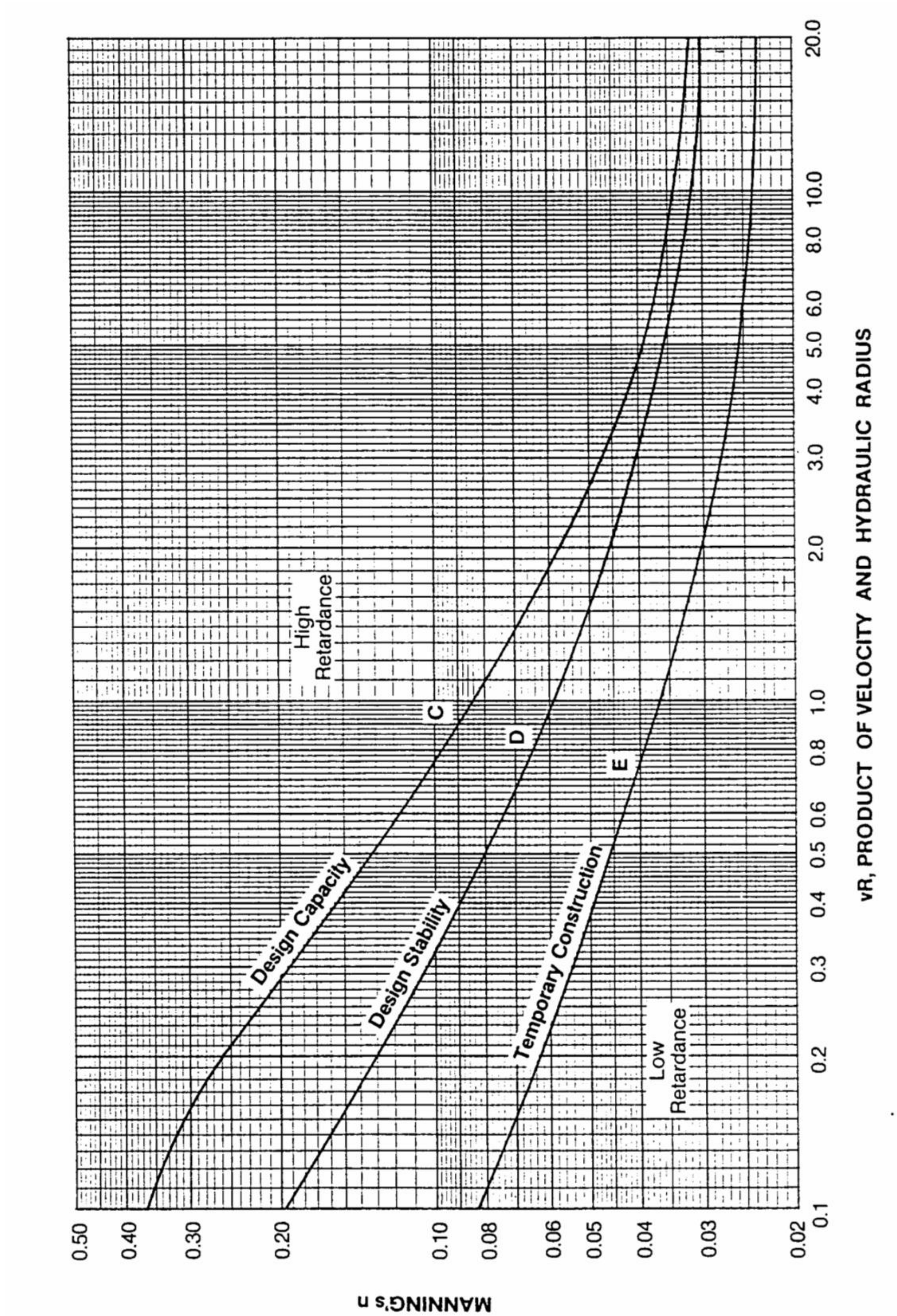


Figure 4.4-1B Manning's n Values for Vegetated Channels
(Source: USDA, TP-61, 1947)

Table 4.4-6 Classification of Vegetal Covers as to Degrees of Retardance

Retardance	Cover	Condition
A	Weeping Lovegrass	Excellent stand, tall (average 30")
	Yellow Bluestem Ischaemum	Excellent stand, tall (average 36")
B	Kudzu	Very dense growth, uncut
	Bermuda grass	Good stand, tall (average 12”)
	Native grass mixture Little bluestem, bluestem, blue gamma other short and long stem Midwest grasses	Good stand, unmowed
	Weeping lovegrass	Good stand, tall (average 24”)
	Laspedeza sericea	Good stand, not woody, tall (average 19”)
	Alfalfa	Good stand, uncut (average 11”)
	Weeping lovegrass	Good stand, unmowed (average 13”)
	Kudzu	Dense growth, uncut
	Blue gamma	Good stand, uncut (average 13”)
C	Crabgrass	Fair stand, uncut (10 – 48”)
	Bermuda grass	Good stand, mowed (average 6”)
	Common lespedeza	Good stand, uncut (average 11”)
	Grass-legume mixture: summer (orchard grass redtop, Italian ryegrass, and common lespedeza)	Good stand, uncut (6 – 8 “)
	Centipede grass	Very dense cover (average 6”)
	Kentucky bluegrass	Good stand, headed (6 – 12”)
D	Bermuda grass	Good stand, cut to 2.5”
	Common lespedeza	Excellent stand, uncut (average 4.5”)
	Buffalo grass	Good stand, uncut (3 – 6”)
	Grass-legume mixture: fall, spring (orchard grass, redtop, Italian ryegrass, and common lespedeza)	Good stand, uncut (4 – 5”)
	Lespedeza serices	After cutting to 2” (very good before cutting)
E	Bermuda grass	Good stand, cut to 1.5”
	Bermuda grass	Burned stubble

Note: Covers classified have been tested in experimental channels. Covers were green and generally uniform.

Source: HEC-15, 1988

4.4.5 – Uniform Flow Calculations

4.4.5.1 Design Charts

Following is a discussion of the equations that can be used for the design and analysis of open channel flow. The Federal Highway Administration has prepared numerous design charts to aid in the design of rectangular, trapezoidal, and triangular open channel cross sections. In addition, design charts for grass-lined channels have been developed. Examples of these charts and instructions for their use are given in subsection 4.4.12.

4.4.5.2 Manning's Equation

Manning's Equation, presented in three forms below, is recommended for evaluating uniform flow conditions in open channels:

$$\mathbf{v = (1.49/n)\, R^{2/3}\, S^{1/2}} \qquad \textbf{(4.4.1)}$$

$$\mathbf{Q = (1.49/n)\, A\, R^{2/3}\, S^{1/2}} \qquad \textbf{(4.4.2)}$$

$$\mathbf{S = [Q_n/(1.49\, A\, R^{2/3})]^2} \qquad \textbf{(4.4.3)}$$

where:

v = average channel velocity (ft/s)
Q = discharge rate for design conditions (cfs)
n = Manning's roughness coefficient
A = cross-sectional area (ft^2)
R = hydraulic radius A/P (ft)
P = wetted perimeter (ft)
S = slope of the energy grade line (ft/ft)

For prismatic channels, in the absence of backwater conditions, the slope of the energy grade line, water surface and channel bottom are assumed to be equal.

For a more comprehensive discussion of open channel theory and design, see the reference USACE, 1991/1994.

4.4.5.3 Geometric Relationships

Area, wetted perimeter, hydraulic radius, and channel top width for standard channel cross sections can be calculated from geometric dimensions. Irregular channel cross sections (i.e., those with a narrow deep main channel and a wide shallow overbank channel) must be subdivided into segments so that the flow can be computed separately for the main channel and overbank portions. This same process of subdivision may be used when different parts of the channel cross section have different roughness coefficients. When computing the hydraulic radius of the subsections, the water depth common to the two adjacent subsections is not counted as wetted perimeter.

4.4.5.4 Direct Solutions

When the hydraulic radius, cross-sectional area, and roughness coefficient and slope are known, discharge can be calculated directly from equation 4.4.2. The slope can be calculated using equation 4.4.3 when the discharge, roughness coefficient, area, and hydraulic radius are known.

Nomographs for obtaining direct solutions to Manning's Equation are presented in Figures 4.4-2 and 4.4-3. Figure 4.4-2 provides a general solution for the velocity form of Manning's Equation, while Figure 4.4-3 provides a solution of Manning's Equation for trapezoidal channels.

General Solution Nomograph

The following steps are used for the general solution nomograph in Figure 4.4-2:

Step 1 Determine open channel data, including slope in ft/ft, hydraulic radius in ft, and Manning's n value.

Step 2 Connect a line between the Manning's n scale and slope scale and note the point of intersection on the turning line.

Step 3 Connect a line from the hydraulic radius to the point of intersection obtained in Step 2.

Step 4 Extend the line from Step 3 to the velocity scale to obtain the velocity in ft/s.

Trapezoidal Solution Nomograph

The trapezoidal channel nomograph solution to Manning's Equation in Figure 4.4-3 can be used to find the depth of flow if the design discharge is known or the design discharge if the depth of flow is known.

Determine input data, including slope in ft/ft, Manning's n value, bottom width in ft, and side slope in ft/ft.

Given Q, find d.

a. Given the design discharge, find the product of Q times n, connect a line from the slope scale to the Qn scale, and find the point of intersection on the turning line.
b. Connect a line from the turning point from Step 2a to the b scale and find the intersection with the z = 0 scale.
c. Project horizontally from the point located in Step 2b to the appropriate z value and find the value of d/b.
d. Multiply the value of d/b obtained in Step 2c by the bottom width b to find the depth of uniform flow, d.

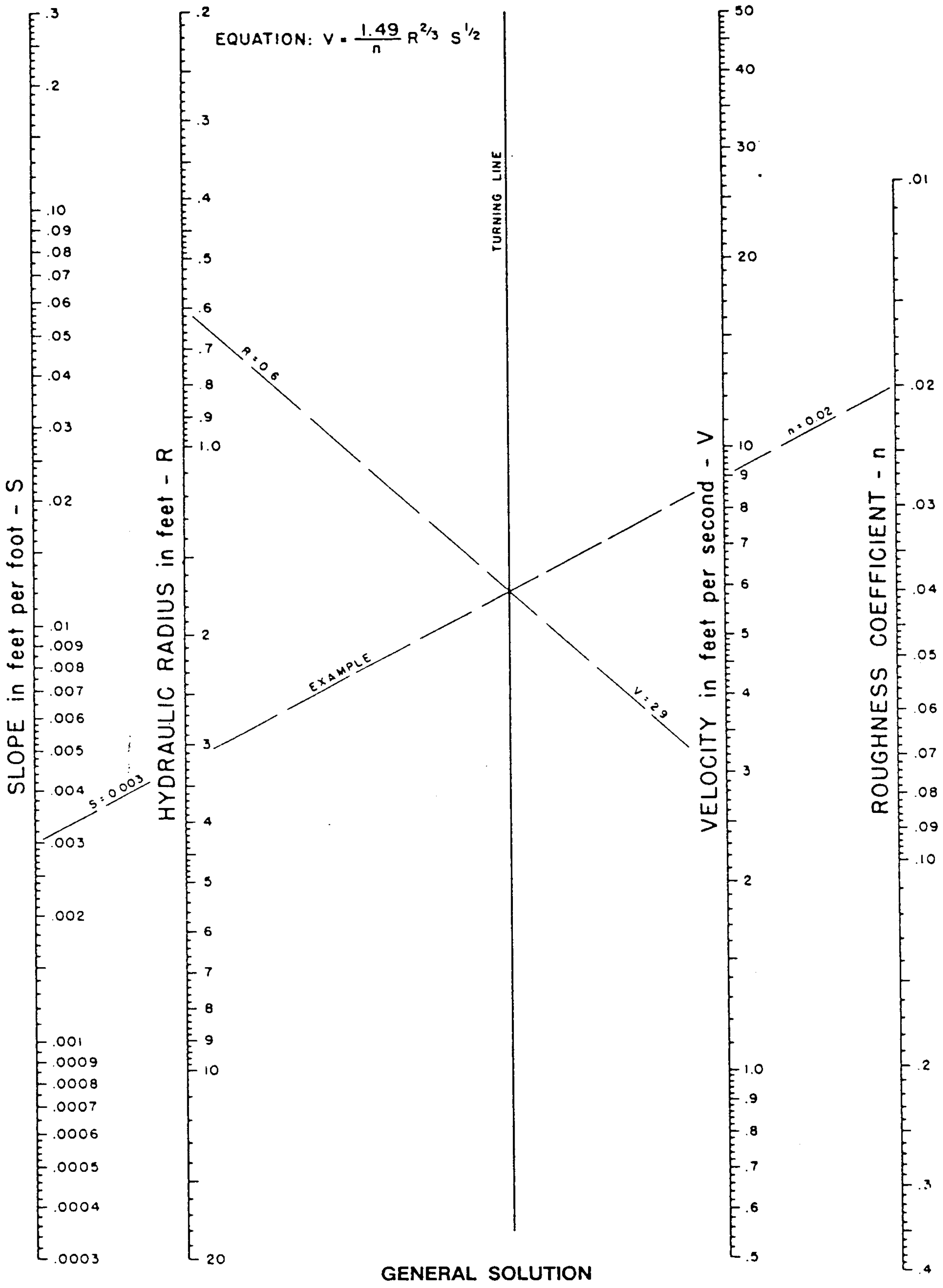
Given d, find Q

Given the depth of flow, find the ratio d divided by b and project a horizontal line from the d/b ratio at the appropriate side slope, z, to the z = 0 scale.

Connect a line from the point located in Step 3a to the b scale and find the intersection with the turning line.

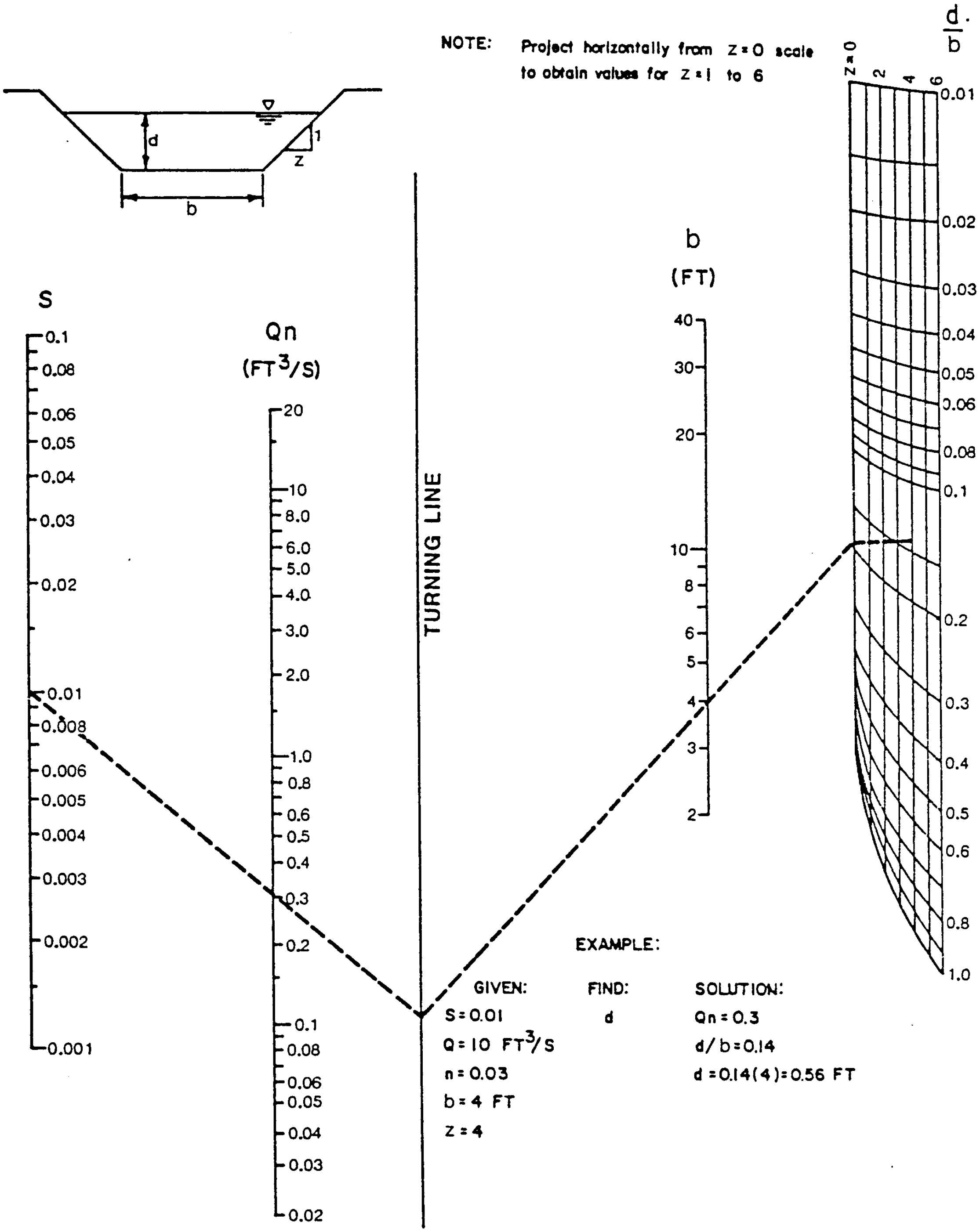
Connect a line from the point located in Step 3b to the slope scale and find the intersection with the Q_n scale.

Divide the value of Qn obtained in Step 3c by the n value to find the design discharge, Q.



Reference: USDOT, FHWA, HDS-3 (1961).

Figure 4.4-2 Nomograph for the Solution of Manning's Equation



Reference: USDOT, FHWA, HEC-15 (1986).

Figure 4.4-3 Solution of Manning's Equation for Trapezoidal Channels

4.4.5.5 Trial and Error Solutions

A trial and error procedure for solving Manning's Equation is used to compute the normal depth of flow in a uniform channel when the channel shape, slope, roughness, and design discharge are known. For purposes of the trial and error process, Manning's Equation can be arranged as:

$$\mathbf{AR^{2/3} = (Qn)/(1.49\ S^{1/2})} \qquad \mathbf{(4.4.4)}$$

where:

A = cross-sectional area (ft)
R = hydraulic radius (ft)
Q = discharge rate for design conditions (cfs)
n = Manning's roughness coefficient
S = slope of the energy grade line (ft/ft)

To determine the normal depth of flow in a channel by the trial and error process, trial values of depth are used to determine A, P, and R for the given channel cross section. Trial values of $AR^{2/3}$ are computed until the equality of equation 4.4.4 is satisfied such that the design flow is conveyed for the slope and selected channel cross section.

Graphical procedures for simplifying trial and error solutions are presented in Figure 4.4-4 for trapezoidal channels. Computer programs are also available for these calculations.

Step 1 Determine input data, including design discharge, Q, Manning's n value, channel bottom width, b, channel slope, S, and channel side slope, z.

Step 2 Calculate the trapezoidal conveyance factor using the equation:

$$\mathbf{K_T = (Qn)/(b^{8/3}S^{1/2})} \qquad \mathbf{(4.4.5)}$$

where:

K_T = trapezoidal open channel conveyance factor
Q = discharge rate for design conditions (cfs)
n = Manning's roughness coefficient
b = bottom width (ft)
S = slope of the energy grade line (ft/ft)

Step 3 Enter the x-axis of Figure 4.4-4 with the value of K_T calculated in Step 2 and draw a line vertically to the curve corresponding to the appropriate z value from Step 1.

Step 4 From the point of intersection obtained in Step 3, draw a horizontal line to the y-axis and read the value of the normal depth of flow over the bottom width, d/b.

Step 5 Multiply the d/b value from Step 4 by b to obtain the normal depth of flow.

Note: If bends are considered, refer to equation 4.4.11

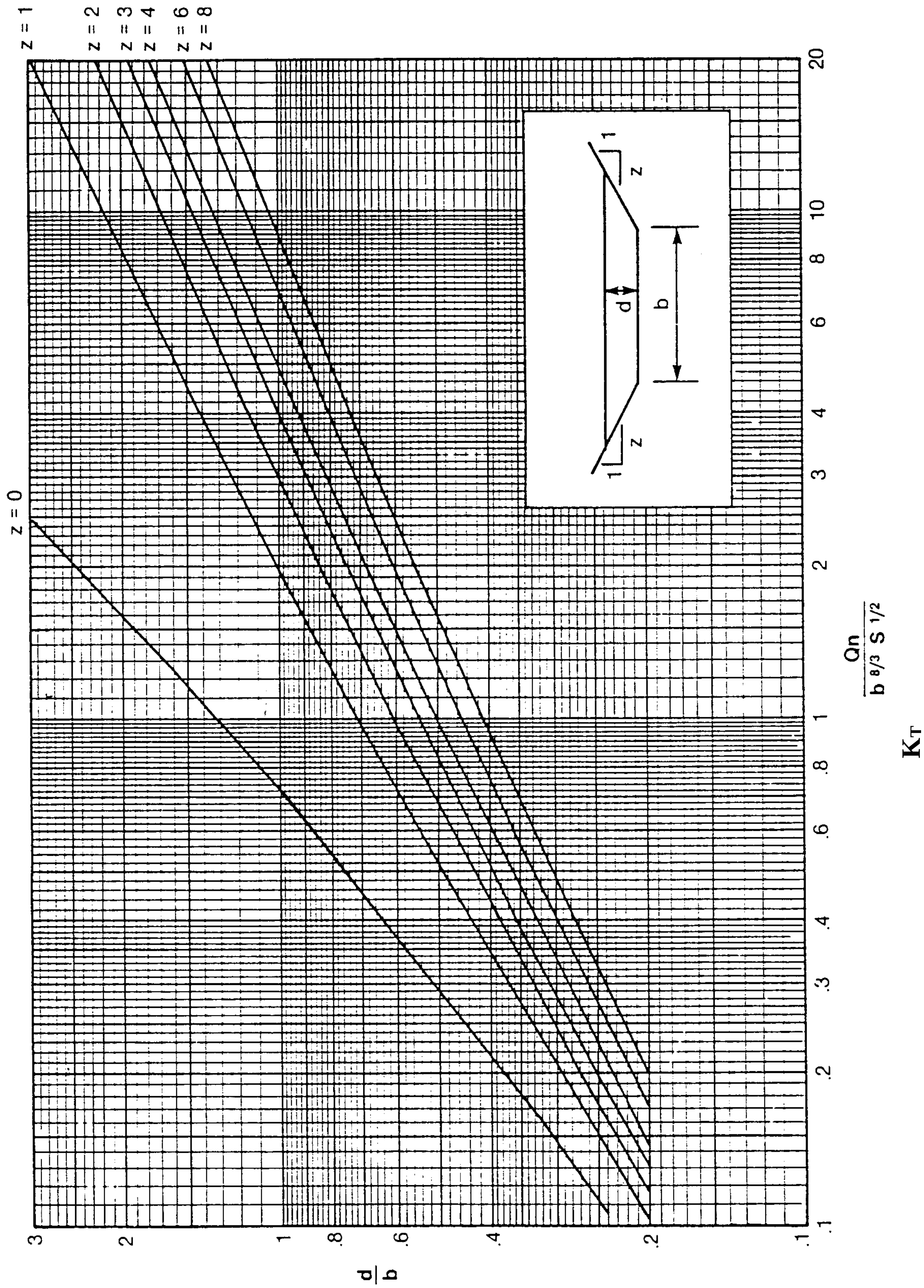


Figure 4.4-4   Trapezoidal Channel Capacity Chart
(Source: Nashville Storm Water Management Manual, 1988)

4.4.6 – Critical Flow Calculations

4.4.6.1 Background

In the design of open channels, it is important to calculate the critical depth in order to determine if the flow in the channel will be subcritical or supercritical. If the flow is subcritical it is relatively easy to handle the flow through channel transitions because the flows are tranquil and wave action is minimal. In subcritical flow, the depth at any point is influenced by a downstream control, which may be either the critical depth or the water surface elevation in a pond or larger downstream channel. In supercritical flow, the depth of flow at any point is influenced by a control upstream, usually critical depth. In addition, the flows have relatively shallow depths and high velocities. Hydraulic jumps are possible under these conditions and consideration should be given to stabilizing the channel.

Critical depth depends only on the discharge rate and channel geometry. The general equation for determining critical depth is expressed as:

$$\mathbf{Q^2/g = A^3/T} \qquad \mathbf{(4.4.6)}$$

where:

Q = discharge rate for design conditions (cfs)
g = acceleration due to gravity (32.2 ft/sec^2)
A = cross-sectional area (ft^2)
T = top width of water surface (ft)

Note: A trial and error procedure is needed to solve equation 4.4-6.

4.4.6.2 Semi-Empirical Equations

Semi-empirical equations (as presented in Table 4.4-7) or section factors (as presented in Figure 4.4-5) can be used to simplify trial and error critical depth calculations. The following equation is used to determine critical depth with the critical flow section factor, Z:

$$\mathbf{Z = Q/(g^{0.5})} \qquad \mathbf{(4.4.7)}$$

where:

Z = critical flow section factor
Q = discharge rate for design conditions (cfs)
g = acceleration due to gravity (32.3 ft/sec^2)

The following guidelines are given for evaluating critical flow conditions of open channel flow:

1. A normal depth of uniform flow within about 10% of critical depth is unstable and should be avoided in design, if possible.
2. If the velocity head is less than one-half the mean depth of flow, the flow is subcritical.
3. If the velocity head is equal to one-half the mean depth of flow, the flow is critical.
4. If the velocity head is greater than one-half the mean depth of flow, the flow is supercritical.

Note: The head is the height of water above any point, plane, or datum of reference. The velocity head in flowing water is calculated as the velocity squared divided by 2 times the gravitational constant ($V^2/2g$).

The Froude number, Fr, calculated by the following equation, is useful for evaluating the type of flow conditions in an open channel:

$$Fr = v/(gA/T)^{0.5} \qquad (4.4.8)$$

where:

- Fr = Froude number (dimensionless)
- v = velocity of flow (ft/s)
- g = acceleration of gravity (32.2 ft/sec^2)
- A = cross-sectional area of flow (ft^2)
- T = top width of flow (ft)

If Fr is greater than 1.0, flow is supercritical; if it is under 1.0, flow is subcritical. Fr is 1.0 for critical flow conditions.

Table 4.4-7 Critical Depth Equations for Uniform Flow in Selected Channel Cross Sections

Channel Type[1]	Semi-Empirical Equations[2] for Estimating Critical Depth	Range of Applicability
1. Rectangular[3]	$d_c = [Q^2/(gb^2)]^{1/3}$	N/A
2. Trapezoidal[3]	$d_c = 0.81[Q^2/(gz^{0.75}b^{1.25})]^{0.27} - b/30z$	$0.1 < 0.5522\ Q/b^{2.5} < 0.4$ For $0.5522\ Q/b^{2.5} < 0.1$, use rectangular channel equation
3. Triangular[3]	$d_c = [(2Q^2)/(gz^2)]^{1/5}$	N/A
4. Circular[4]	$d_c = 0.325(Q/D)^{2/3} + 0.083D$	$0.3 < d_c/D < 0.9$
5. General[5]	$(A^3/T) = (Q^2/g)$	N/A

where:

- d_c = critical depth (ft)
- Q = design discharge (cfs)
- g = acceleration due to gravity (32.3 ft/s^2)
- b = bottom width of channel (ft)
- z = side slopes of a channel (horizontal to vertical)
- D = diameter of circular conduit (ft)
- A = cross-sectional area of flow (ft^2)
- T = top width of water surface (ft)

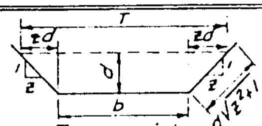
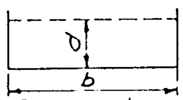
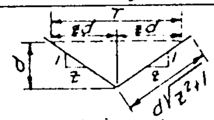
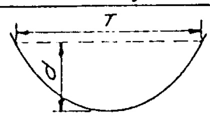
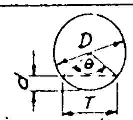
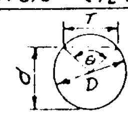
[1] See Figure 4.4-5 for channel sketches

[2] Assumes uniform flow with the kinetic energy coefficient equal to 1.0

[3] Reference: French (1985)

[4] Reference: USDOT, FHWA, HDS-4 (1965)

[5] Reference: Brater and King (1976)

Section	Area A	Wetted Perimeter P	Hydraulic Radius R	Top Width T	Critical Depth Factor, Z
 Trapezoid	$bd+zd^2$	$b+2d\sqrt{z^2+1}$	$\frac{bd+zd^2}{b+2d\sqrt{z^2+1}}$	$b+2zd$	$\frac{[(b+zd)d]^{1.5}}{\sqrt{b+2zd}}$
 Rectangle	bd	$b+2d$	$\frac{bd}{b+2d}$	b	$bd^{1.5}$
 Triangle	zd^2	$2d\sqrt{z^2+1}$	$\frac{zd}{2\sqrt{z^2+1}}$	$2zd$	$\frac{\sqrt{2}}{2}zd^{2.5}$
 Parabola	$\frac{2}{3}dT$	$T+\frac{8d^2}{3T}$ 1	$\frac{2dT^2}{3T^2+8d^2}$ 1	$\frac{3a}{2d}$	$\frac{2}{9}\sqrt{6}\,Td^{1.5}$
 Circle - < ½ full 2	$\frac{D^2}{8}\left(\frac{\pi\theta}{180}-\sin\theta\right)$	$\frac{\pi D\theta}{360}$	$\frac{45D}{\pi\theta}\left(\frac{\pi\theta}{180}-\sin\theta\right)$	$D\sin\frac{\theta}{2}$ or $2\sqrt{d(D-d)}$	$a\sqrt{\frac{a}{D\sin\frac{\theta}{2}}}$
 Circle - > ½ full 3	$\frac{D^2}{8}\left(2\pi-\frac{\pi\theta}{180}+\sin\theta\right)$	$\frac{\pi D(360-\theta)}{360}$	$\frac{45D}{\pi(360-\theta)}\left(2\pi-\frac{\pi\theta}{180}+\sin\theta\right)$	$D\sin\frac{\theta}{2}$ or $2\sqrt{d(D-d)}$	$a\sqrt{\frac{a}{D\sin\frac{\theta}{2}}}$

1 Satisfactory approximation for the interval $0 < \frac{d}{T} \leq 0.25$
When $d/T > 0.25$, use $p = \frac{1}{2}\sqrt{16d^2+T^2} + \frac{T^2}{8d}\sinh^{-1}\frac{4d}{T}$

2 $\theta = 4\sin^{-1}\sqrt{d/D}$
3 $\theta = 4\cos^{-1}\sqrt{d/D}$
Insert θ in degrees in above equations

Note: Small z = Side Slope Horizontal Distance
Large Z = Critical Depth Section Factor

Reference: USDA, SCS, NEH-5 (1956).

Figure 4.4-5 Open Channel Geometric Relationships for Various Cross Sections

4.4.7 – Vegetative Design

4.4.7.1 Introduction

A two-part procedure is an alternative for design of temporary and vegetative channel linings. **This procedure is only allowed with the approval of the TOWN ENGINEER.** Part 1, the design stability component, involves determining channel dimensions for low vegetative retardance conditions, using Class D as defined in Table 4.4-6. Part 2, the design capacity component, involves determining the depth increase necessary to maintain capacity for higher vegetative retardance conditions, using Class C as defined in Table 4.4-6. If temporary lining is to be used during construction, vegetative retardance Class E should be used for the design stability calculations.

If the channel slope exceeds 10%, or a combination of channel linings will be used, additional procedures not presented below are required. References include HEC-15 (USDOT, FHWA, 1986) and HEC-14 (USDOT, FHWA, 1983).

4.4.7.2 Design Stability

The following are the steps for design stability calculations:

Step 1 Determine appropriate design variables, including discharge, Q, bottom slope, S, cross section parameters, and vegetation type.

Step 2 Use Table 4.4-3 to assign a maximum velocity, v_m based on vegetation type and slope range.

Step 3 Assume a value of n and determine the corresponding value of vR from the n versus vR curves in Figure 4.4-1B. Use retardance Class D for permanent vegetation and E for temporary construction.

Step 4 Calculate the hydraulic radius using the equation:

$$\mathbf{R = (vR)/v_m} \qquad \textbf{(4.4.9)}$$

where:

R = hydraulic radius of flow (ft)
vR = value obtained from Figure 4.4-1B in Step 3
v_m = maximum velocity from Step 2 (ft/s)

Step 5 Use the following form of Manning's Equation to calculate the value of vR:

$$\mathbf{vR = (1.49\, R^{5/3}\, S^{1/2})/n} \qquad \textbf{(4.4.10)}$$

where:

vR = calculated value of vR product
R = hydraulic radius value from Step 4 (ft)
S = channel bottom slope (ft/ft)
n = Manning's n value assumed in Step 3

Step 6 Compare the vR product value obtained in Step 5 to the value obtained from Figure 4.4-1B for the assumed n value in Step 3. If the values are not reasonably close, return to Step 3 and repeat the calculations using a new assumed n value.

Step 7 For trapezoidal channels, find the flow depth using Figures 4.4-3 or 4.4-4, as described in Section 4.4.4.4. The depth of flow for other channel shapes can be evaluated using the trial and error procedure described in Section 4.4.4.5.

Step 8 If bends are considered, calculate the length of downstream protection, L_p, for the bend, using Figure 4.4-6. Provide additional protection, such as gravel or riprap in the bend and extending downstream for length, L_p.

Design Capacity

The following are the steps for design capacity calculations:

Step 1 Assume a depth of flow greater than the value from Step 7 above and compute the waterway area and hydraulic radius (see Figure 4.4-5 for equations).

Step 2 Divide the design flow rate, obtained using appropriate procedures from Chapter 2, by the waterway area from Step 1 to find the velocity.

Step 3 Multiply the velocity from Step 2 by the hydraulic radius from Step 1 to find the value of vR.

Step 4 Use Figure 4.4-1B to find a Manning's n value for retardance Class C based on the vR value from Step 3.

Step 5 Use Manning's Equation (equation 4.4.1) or Figure 4.4-2 to find the velocity using the hydraulic radius from Step 1, Manning's n value from Step 4, and appropriate bottom slope.

Step 6 Compare the velocity values from Steps 2 and 5. If the values are not reasonably close, return to Step 1 and repeat the calculations.

Step 7 Add an appropriate freeboard to the final depth from Step 6. Generally, 20% is adequate.

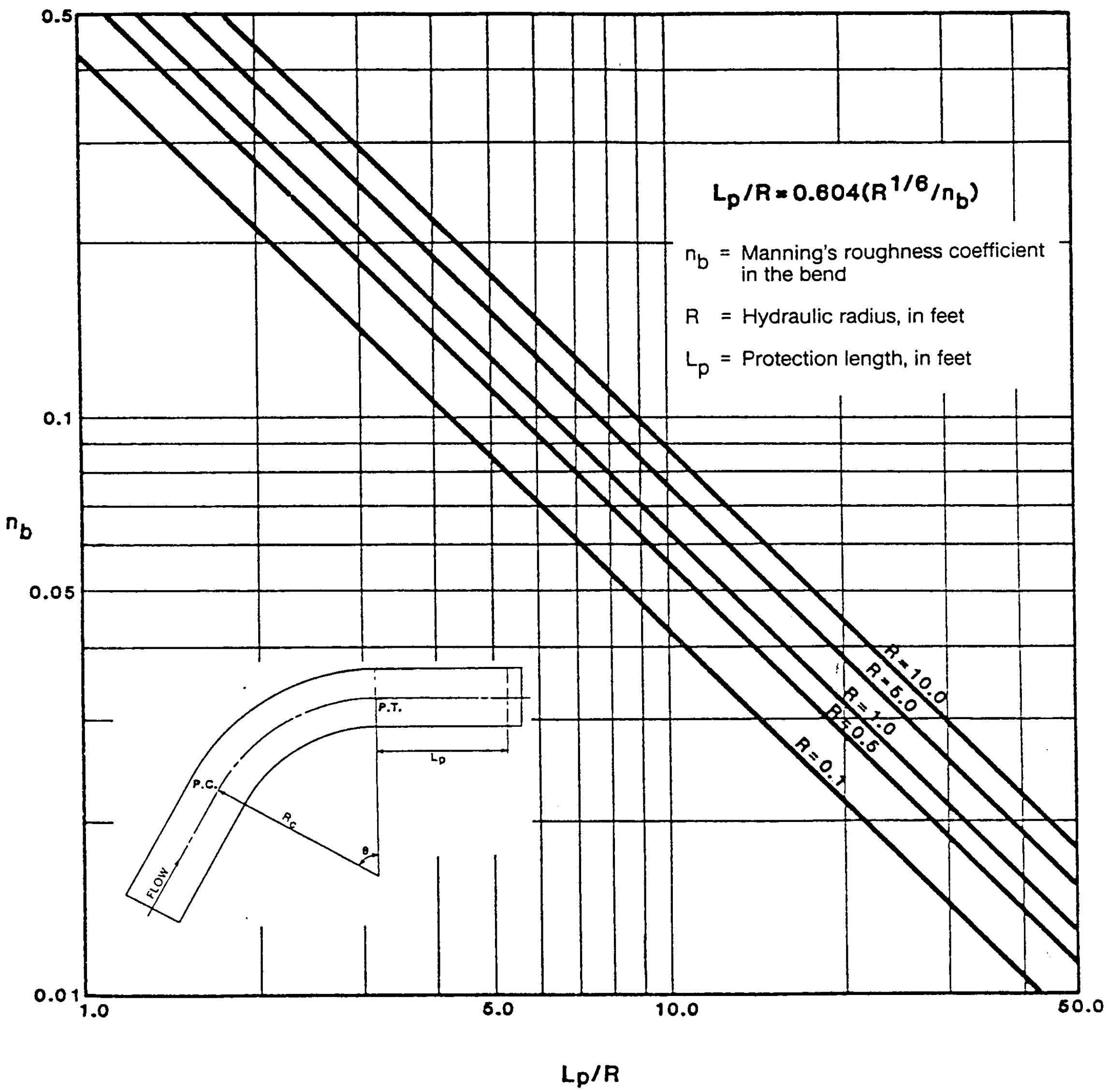
Step 8 If bends are considered, calculate super-elevation of the water surface profile at the bend using the equation:

$$\mathbf{d = (v^2T)/(gR_c)} \qquad \mathbf{(4.4.11)}$$

where:

d = super-elevation of the water surface profile due to the bend (ft)
v = average velocity from Step 6 (ft/s)
T = top width of flow (ft)
g = acceleration of gravity (32.2 ft/sec^2)
R_c = mean radius of the bend (ft)

Note: Add freeboard consistent with the calculated □d.



Reference: USDOT, FHWA, HEC-15 (1986).

Figure 4.4-6 Protection Length, Lp, Downstream of Channel Bend

4.4.8 – Stone Riprap Design

A number of agencies and researchers have studied and developed empirical equations to estimate the required size of rock riprap to resist various hydraulic conditions, including the U.S. Army Corps of Engineers, Soil Conservation Service and Federal Highway Administration. As with all empirical equations based on the results of laboratory experiments, they must be used with an understanding of the range of data on which they are based.

A paper prepared by Garry Gregory in June of 1987 has been widely used in Texas for riprap design. He recommends estimating $D_{50} = \tau/[0.04(\gamma_s-\gamma)]$, including similar adjustments for bends and channel side slopes. Excerpts from this paper are presented below. Also see Figure 4.4-7 through 4.4-12 and stone riprap sections Figures 4.4-15 and 4.4-16. Regardless of computed thickness the minimum allowable riprap thickness is 12 inches. A properly designed geotextile is required under the bedding layer.

Design Criteria

Natural or Construction Channel Protection:

1. Calculate boundary shear (tractive stress or tractive force) by:

$$T_o=\gamma RS \qquad \textbf{(4.4.12)}$$

Where: T_o=average tractive stress on channel bottom, PSF
γ=unit weight of water (62.4pcf)
R=hydraulic radius of channel
S=slope of energy gradient

$$T_o^1 = T_o*(1-(\sin^2 \varphi/\sin^2 \theta))^{0.5} \qquad \textbf{(4.4.13)}$$

Where: T_o^1=average tractive stress on channel side slopes, PSF
T_o=same as in equation (Eq. 4.4.12)
φ =angle of side slope with the horizontal
θ =angle of repose of riprap (approx. 40°)

The greater value of T_o or T_o^1 governs.

2. Determine the tractive stress in a bend in the channel by:

$$T_b=T \times 3.15\ (r/w)^{-0.5} \qquad \textbf{(4.4.14)}$$

Where: T_b=local tractive stress in the bend, PSF
T=the greater of T_o or T_o^1 from equations **(4.4.12) & (4.4.13)**
r=center-line radius of the bend, feet
w=water surface width at upstream end of bend, feet

3. Determine D_{50} size of riprap stone required from:

$$D_{50}=T/0.04\ (\gamma_5-\gamma) \qquad \textbf{(4.4.15)}$$

Where: D_{50}=required average size of riprap stone, feet
(size at which 50% of the gradation is finer weight)
T=the greater of T_o or T_o^1 from equations **(4.4.12)** & **(4.4.13)**
or for a bend in the channel
a=a constant = 0.04
γ_S=saturated surface dry (SSD) specific weight of stone
γ=unit weight of water (62.4pcf)

4. Select minimum riprap thickness required from GRAIN SIZE CURVES, Figures 4.4-7 through 4.4-12. Select from smaller side of band at 50% finer gradation.

5. Select RIPRAP GRADATIONS table (Figure 4.4-13 and 4.4-14) based upon riprap thickness selected in step 4.

6. Select bedding thickness from the GRAIN SIZE CURVES, Figures 4.4-7 through 4.4-12 which was used to select the riprap thickness in step 4. NOTE: the bedding thicknesses included on Figures 4.4-7 through 4.4-12 are based upon using a properly designed geotextile underneath the bedding. If a geotextile is not used the bedding thickness must be increased to a minimum of 9 inches for 24 inch and 30 inch thickness of riprap and a minimum of 12 inches for the 36 inch thickness of riprap.

7. To provide stability in the riprap layer the riprap gradations should meet the following criteria for GRADATION INDEX:

$$\text{GRADATION INDEX: } [D_{85}/D_{50} + D_{50}/D_{15}] \leq 5.5 \qquad \textbf{(4.4-16)}$$

Where: D85, D50 and D15 are the riprap grain sizes in MM of which 85%, 50% and 15% respectively are finer by weight.
The mid-band gradations of Plates 1 through 6 meet this criteria.

8. To provide stability of the bedding layer the bedding should meet the following filter criteria with respect to the riprap

$$D_{15}/d_{85}<5<D_{15}/d_{15}<40 \qquad \textbf{(4.4-17)}$$

$$D_{50}/d_{50}<40 \qquad \textbf{(4.4-18)}$$

Where: D refers to riprap sizes in MM
d refers to bedding sizes in MM
The mid-band gradations of Plates 1 through 6 meet this criteria.

9. The geotextile underneath the bedding should be designed as a filter to the soil.

10. Figures 4.4-17 and 4.4-18 present typical riprap design sections. These figures are from EM1110-2-160 by the U.S. Army Corps of Engineers.

4.4.8.4 Culvert Outfall Protection

The following procedure can be used to design stone riprap protection for culvert outfalls.

1. Determine the D_{50} size of riprap required from:

$$D_{50}=\sqrt{V}[C\ [2g\ (\gamma_5-\gamma_w/\gamma_w)]]^{½} \qquad \textbf{(4.4-19)}$$

Where: D_{50}=Required average size of riprap stone, feet
V=water velocity at culvert outlet, FPS
g=acceleration of gravity, 32.2 feet per sec/sec
γ_5=saturated surface dry (SSD) specific weight of stone
γ_w=unit weight of water, 62.4 pcf
C=a stability coefficient determined by the author to be 1.8
For culvert outlets based upon experience and observation

NOTE: For a SSD specific weight of stone of 160 pcf and C=1.8 equation (8) reduces to:

$$D_{50} = (V/18)^{1/2} \qquad \textbf{(4.4.20)}$$

2. Select riprap and bedding from Figures 4.4-7 through 4.4-12 D_{50} determined from equation (Eq. 4.4.19) or (Eq. 4.4.20).

3. Select gradations from gradation tables (Figures 4.4-17 and 4.4-18).

Grouted Riprap – The Town of Copper Canyon will allow grouted stone riprap as an erosion control feature. However, the design thickness of the stone lining will not be reduced by the use of grout. See the Corps' design manual ETL 1110-2-334 on design and construction of grouted riprap which should be an available option for certain applications.

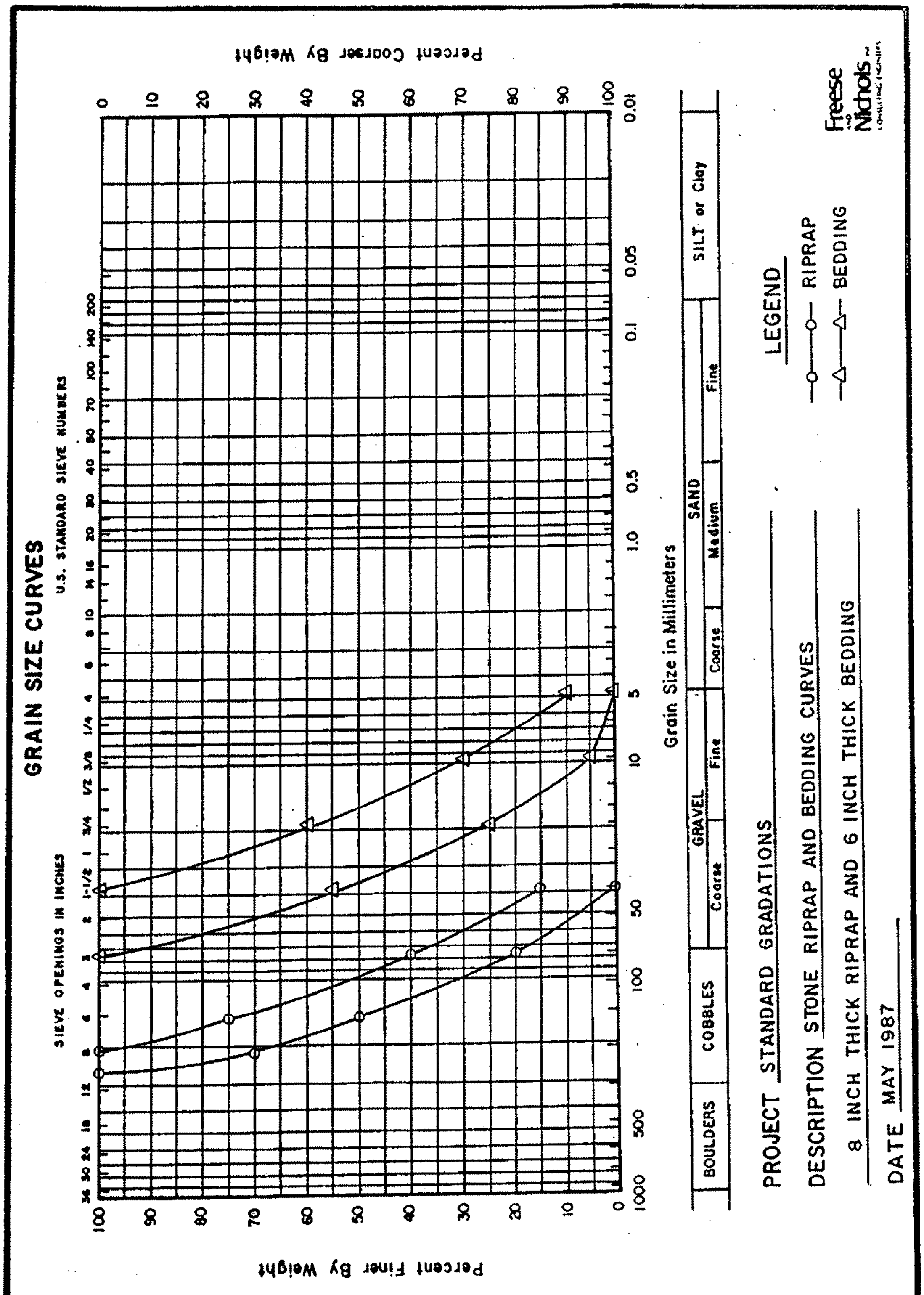


Figure 4.4-7 Grain Size Curve for 8" Riprap and 6" Bedding

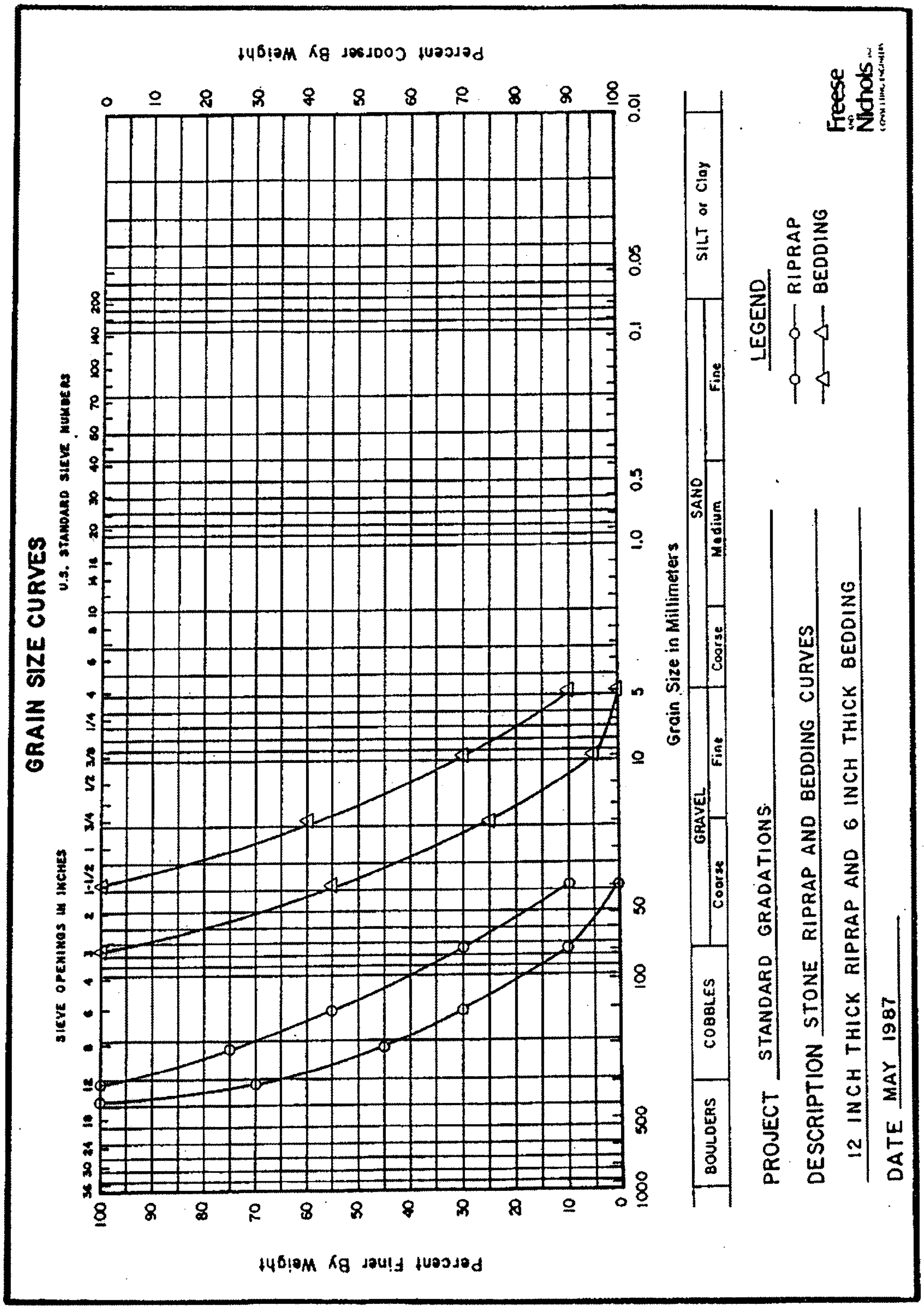


Figure 4.4-8 Grain Size Curve for 12" Riprap and 6" Bedding

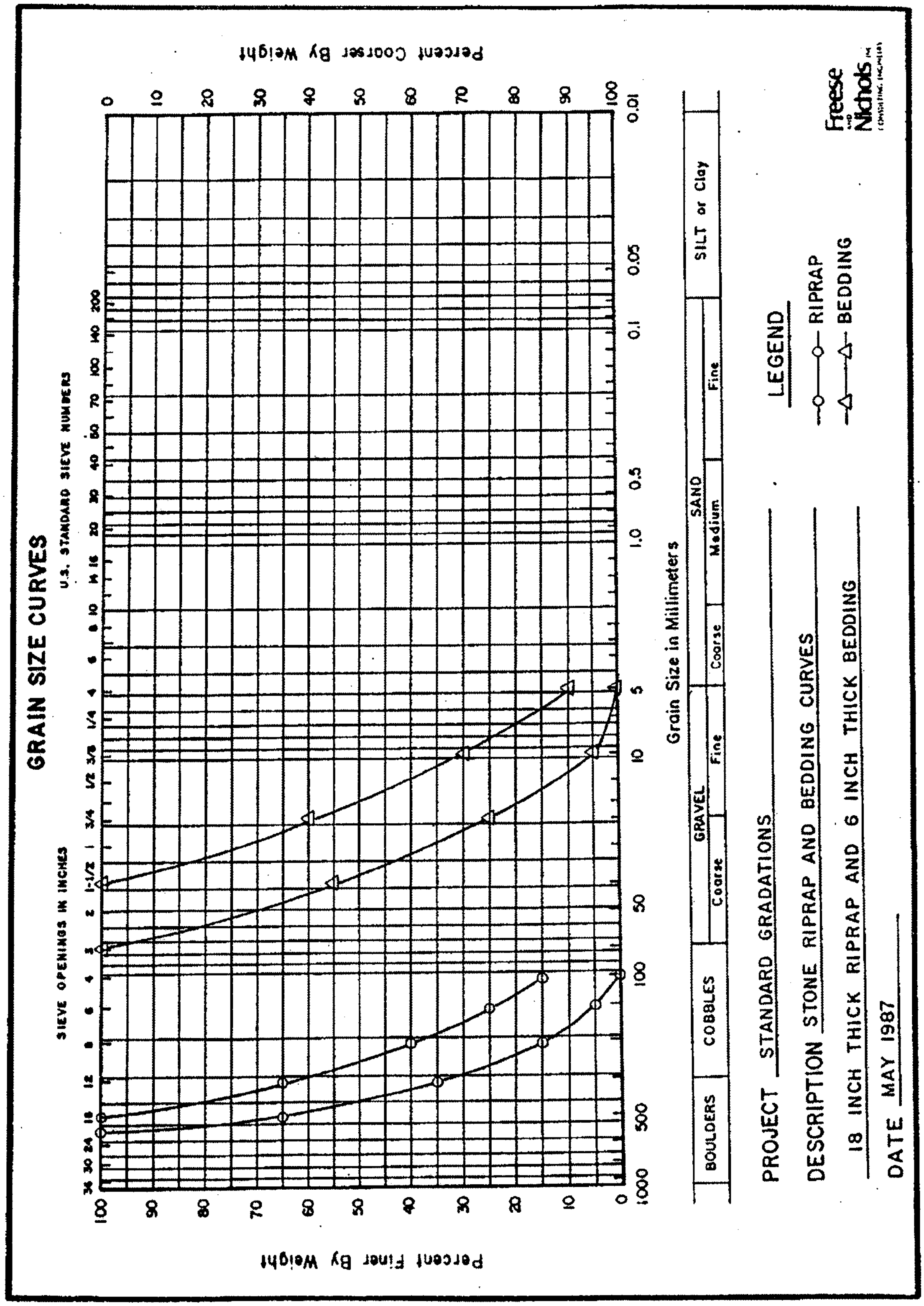


Figure 4.4-9 Grain Size Curve for 18" Riprap and 6" Bedding

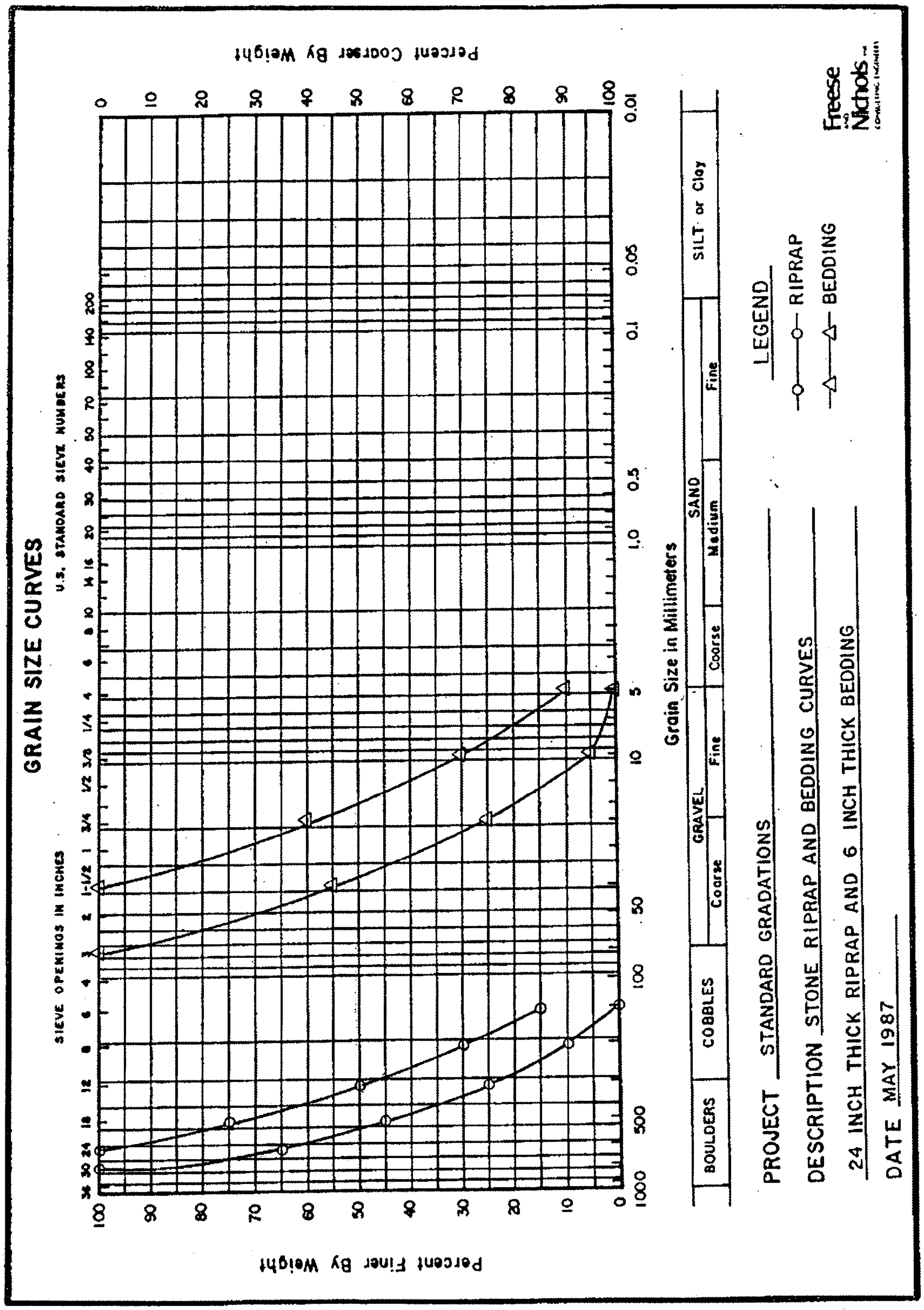


Figure 4.4-10 Grain Size Curve for 24" Riprap and 6" Bedding

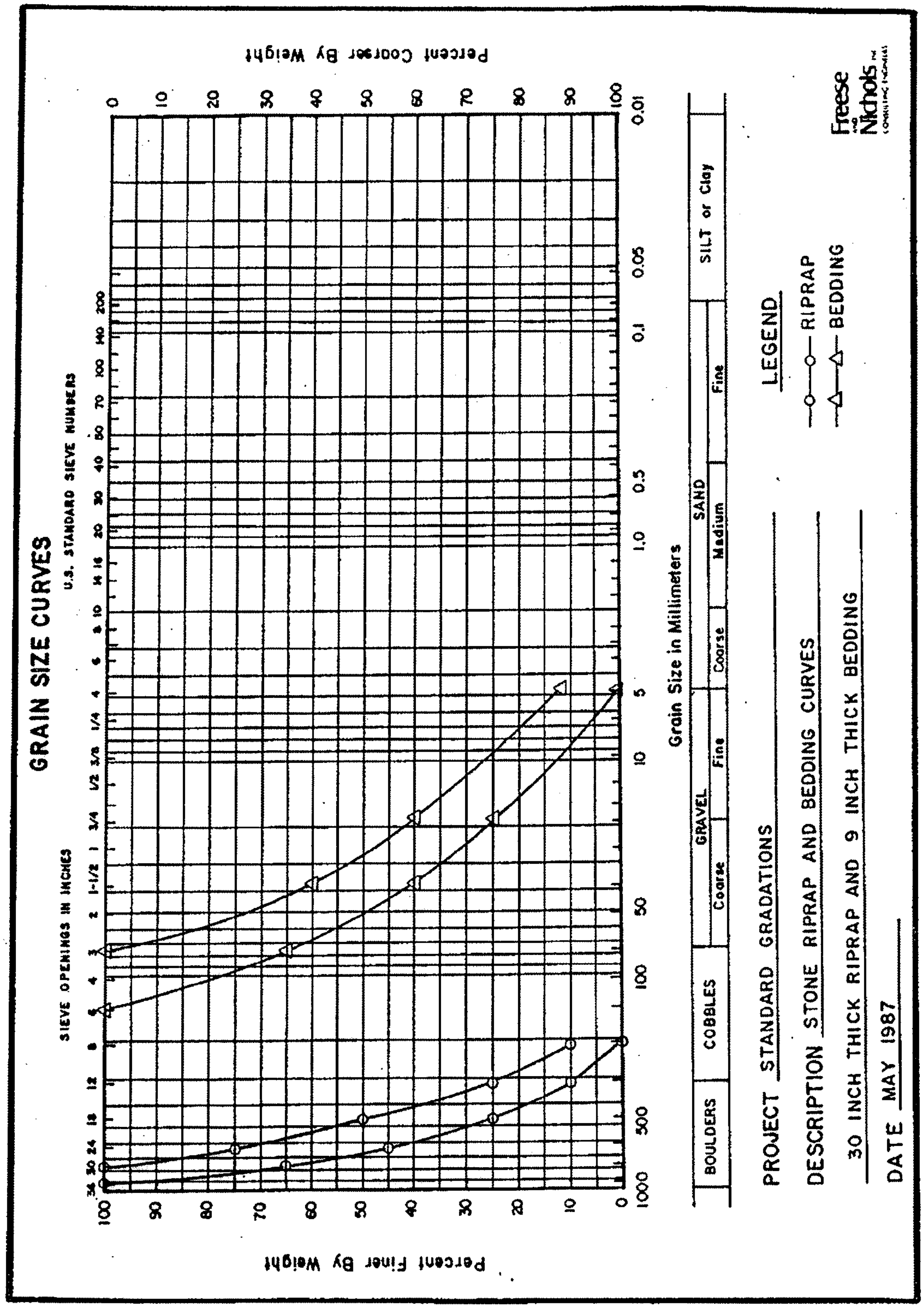


Figure 4.4-11 Grain Size Curve for 30" Riprap and 9" Bedding

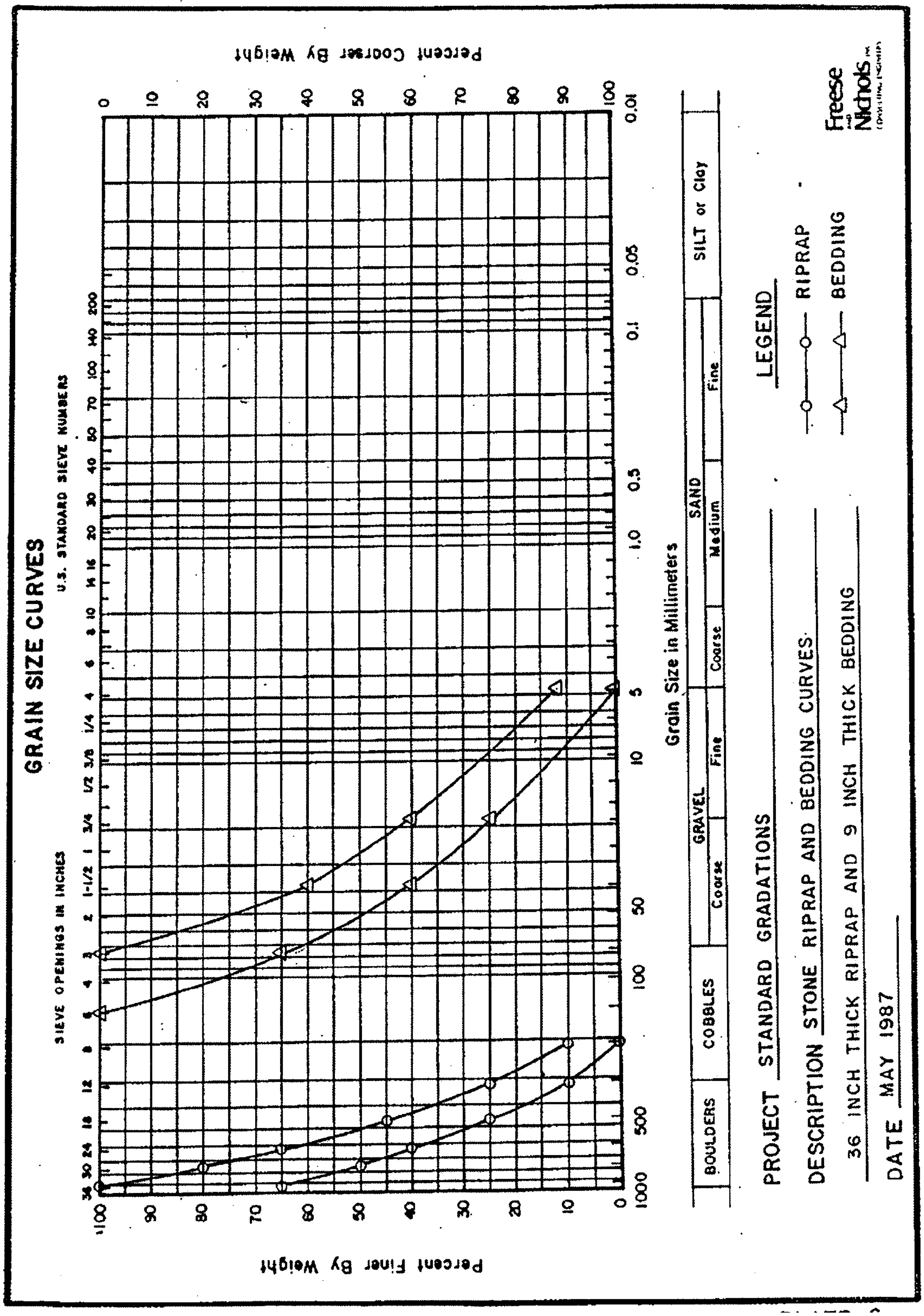


Figure 4.4-12 Grain Size Curve for 36' Riprap and 9" Bedding

RIPRAP GRADATIONS

12" THICKNESS OF RIPRAP

SIEVE SIZE SQUARE MESH	PERCENT PASSING
15 INCH	100
12 INCH	70 – 100
8 INCH	45 – 75
6 INCH	30 – 55
3 INCH	10 – 30
1-1/2 INCH	0 – 10

RIPRAP GRADATIONS

8" THICKNESS OF RIPRAP

SIEVE SIZE SQUARE MESH	PERCENT PASSING
10 INCH	100
8 INCH	70 – 100
6 INCH	50 – 75
3 INCH	20 – 40
1-1/2 INCH	0 – 15

BEDDING GRADATIONS

6" THICKNESS OF BEDDING

SIEVE SIZE SQUARE MESH	PERCENT PASSING
3 INCH	100
1-1/2 INCH	55 – 100
3/4 INCH	25 – 60
3/8 INCH	5 – 30
No.4	0 – 10

BEDDING GRADATIONS

9" THICKNESS OF BEDDING

SIEVE SIZE SQUARE MESH	PERCENT PASSING
6 INCH	100
3 INCH	65 – 100
1-1/2 INCH	40 – 60
3/4 INCH	25 – 40
No.4	0 – 12

Figure 4.4-13 Riprap Gradation Tables for 6", 8", 9" and 12" Thickness of Riprap

RIPRAP GRADATIONS

36" THICKNESS OF RIPRAP

SIEVE SIZE SQUARE MESH	PERCENT PASSING
44 INCH	100
36 INCH	65 – 100
30 INCH	50 – 80
18 INCH	25 – 45
12 INCH	10 – 25
8 INCH	0 – 10

RIPRAP GRADATIONS

30" THICKNESS OF RIPRAP

SIEVE SIZE SQUARE MESH	PERCENT PASSING
36 INCH	100
30 INCH	65 – 100
24 INCH	45 – 75
18 INCH	25 – 50
12 INCH	10 – 25
8 INCH	0 – 10

RIPRAP GRADATIONS

24" THICKNESS OF RIPRAP

SIEVE SIZE SQUARE MESH	PERCENT PASSING
30 INCH	100
24 INCH	65 – 100
18 INCH	45 – 75
12 INCH	25 – 50
8 INCH	10 – 30
6 INCH	0 – 15

RIPRAP GRADATIONS

18" THICKNESS OF RIPRAP

SIEVE SIZE SQUARE MESH	PERCENT PASSING
21 INCH	100
18 INCH	65 – 100
12 INCH	35 – 65
8 INCH	15 – 40
6 INCH	5 – 25
4 INCH	0 – 15

Figure 4.4-14 Riprap Gradation Tables for 18", 24", 30" and 36" Thickness of Riprap

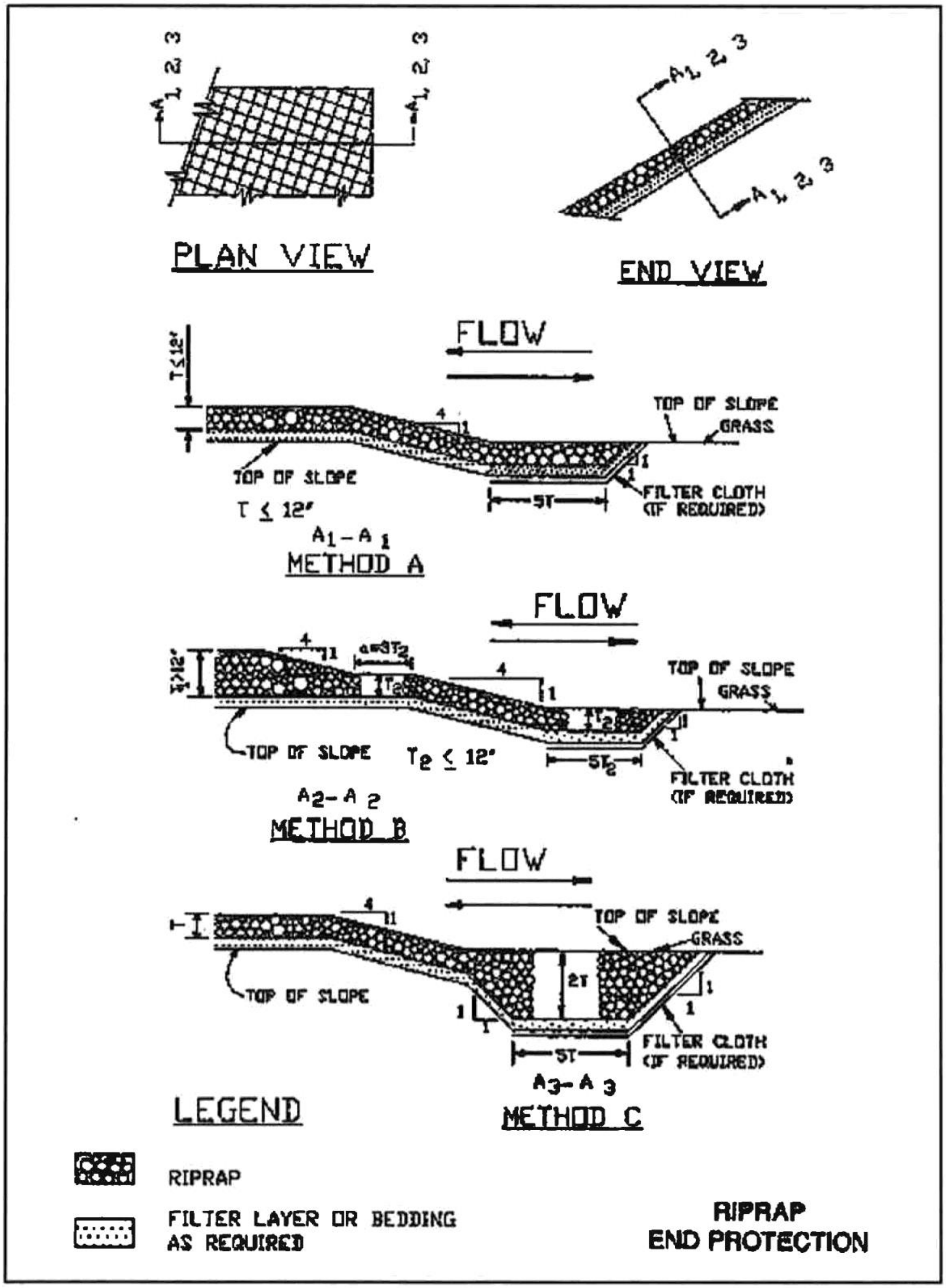


Figure 4.4-15 Typical Riprap Design Cross Sections

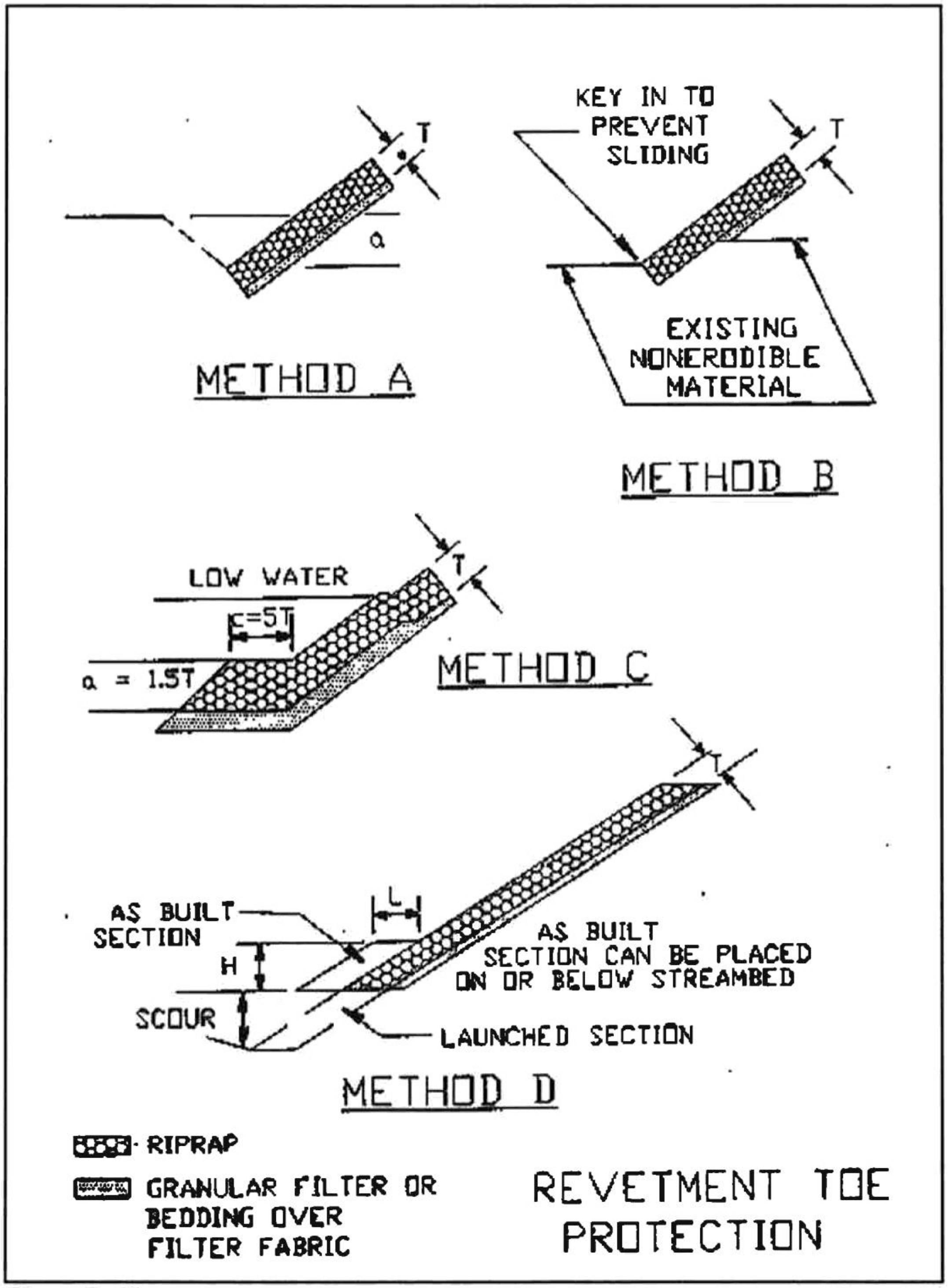


Figure 4.4-16 Typical Riprap Design for Revetment Toe Protection

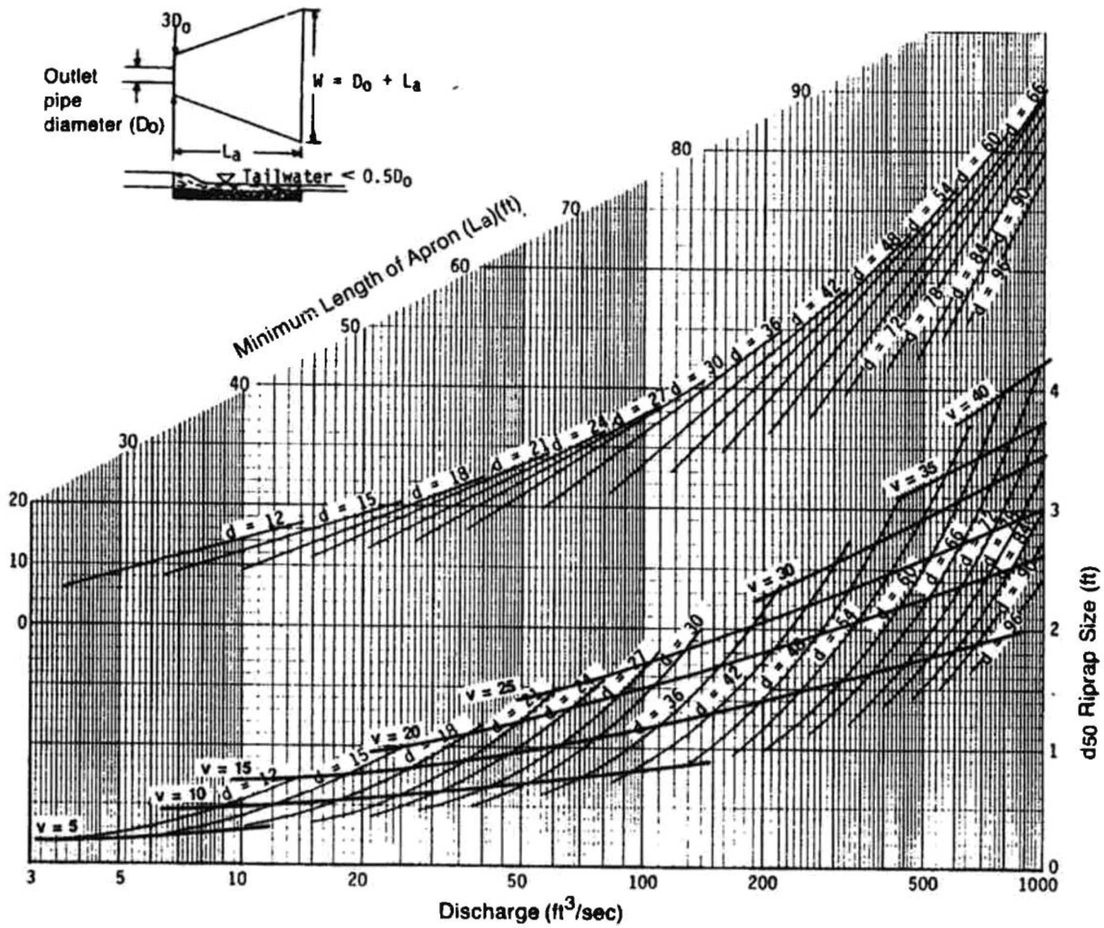
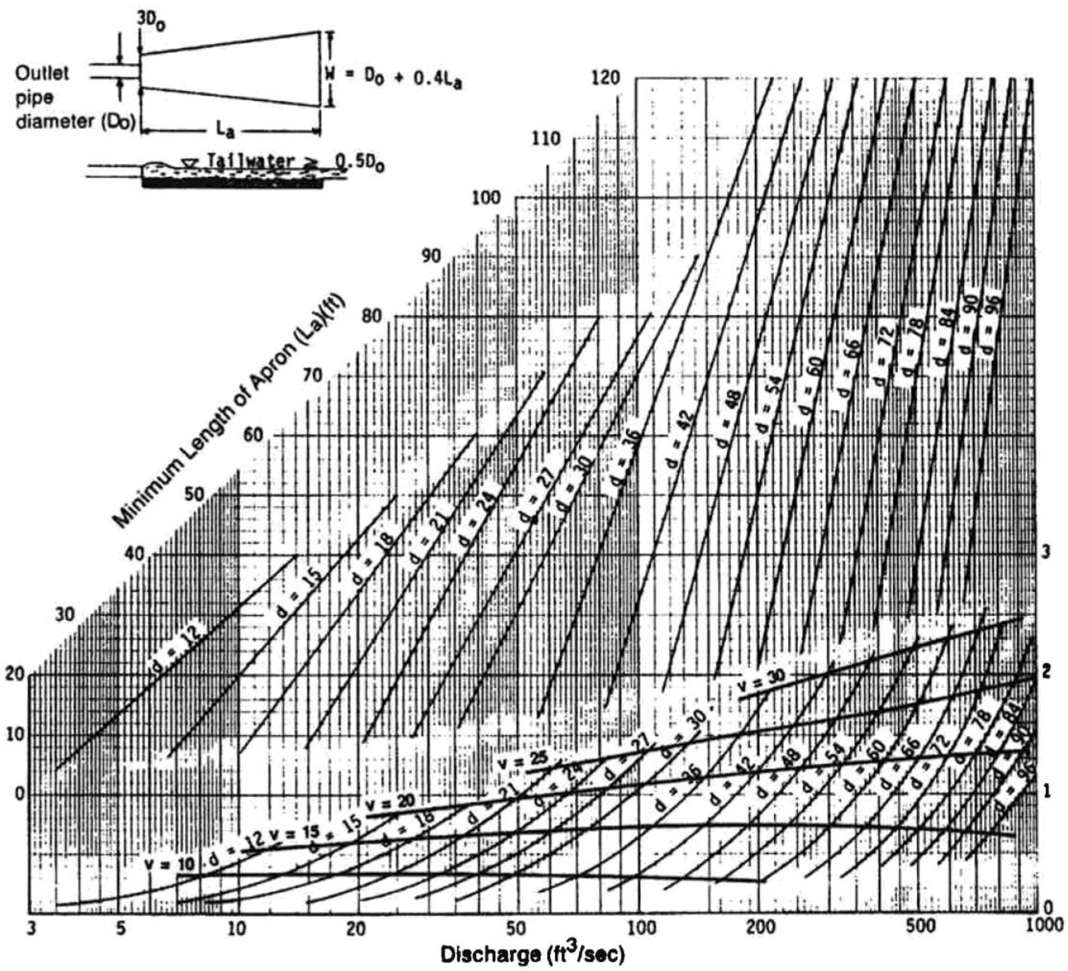


Figure 4.4-17 Design of Riprap Apron Under Minimum Tailwater Conditions
(Source: USDA, SCS, 1975)



Curves may not be extrapolated.

Figure 4.4-18 Design of Riprap Apron Under Maximum Tailwater Conditions
(Source: USDA, SCS, 1975)

4.4.9 – Gabion Design

This section is excerpted from “Gabions for Streambank Erosion Control” EMRR Technical Notes Collection (ERDC TN-EMRRP-SR-22), U.S. Army Engineer Research and Development Center, Vicksburg, MS, 2000.

4.4.9.1 OVERVIEW

Gabions come in three basic forms, the gabion basket, gabion mattress, and sack gabion. All three types consist of wire mesh baskets filled with cobble or small boulder material. The baskets are used to maintain stability and to protect streambanks and beds.

The difference between a gabion basket and a gabion mattress is the thickness and the aerial extent of the basket. A sack gabion is, as the name implies, a mesh sack that is filled with rock material. The benefit of gabions is that they can be filled with rocks that would individually be too small to withstand the erosive forces of the stream. The gabion mattress is shallower (0.5 to 1.5 ft) than the basket and is designed to protect the bed or banks of a stream against erosion.

Gabion baskets are normally much thicker (about 1.5 to 3 ft) and cover a much smaller area. They are used to protect banks where mattresses are not adequate or are used to stabilize slopes, construct drop structures, pipe outlet structures, or nearly any other application where soil must be protected from the erosive forces of water. References to gabions in this article refer generally to both mattresses and baskets.

Gabion baskets can be made from either welded or woven wire mesh. The wire is normally galvanized to reduce corrosion but may be coated with plastic or other material to prevent corrosion and/or damage to the wire mesh containing the rock fill. New materials such as Tensar, a heavy-duty polymer plastic material, have been used in some applications in place of the wire mesh. If the wire baskets break, either through corrosion, vandalism, or damage from debris or bed load, the rock fill in the basket can be lost and the protective value of the method endangered.

Gabions are often used where available rock size is too small to withstand the erosive and tractive forces present at a project site. The available stone size may be too small due to the cost of transporting larger stone from remote sites, or the desire to have a project with a smoother appearance than obtained from riprap or other methods. Gabions also require about one third the thickness of material when compared to riprap designs. Riprap is often preferred, however, due to the low labor requirements for its placement.

The science behind gabions is fairly well established, with numerous manufacturers providing design methodology and guidance for their gabion products. Dr. Stephen T. Maynord of the U.S. Army Engineer Research and Development Center in Vicksburg, Mississippi, has also conducted research to develop design guidance for the installation of gabions. Two general methods are typically used to determine the stability of gabion baskets in stream channels, the critical shear stress calculation and the critical velocity calculation. A software package known as CHANLPRO has been developed by Dr. Maynord (Maynord et al. 1998).

Manufacturers have generated extensive debate regarding the use and durability of welded wire baskets versus woven wire baskets in project design and construction. Project results seem to indicate that performance is satisfactory for both types of mesh.

The rocks contained within the gabions provide substrates for a wide variety of aquatic organisms. Organisms that have adapted to living on and within the rocks have an excellent home, but vegetation may be difficult to establish unless the voids in the rocks contained within the baskets are filled with soil.

If large woody vegetation is allowed to grow in the gabions, there is a risk that the baskets will break when the large woody vegetation is uprooted or as the root and trunk systems grow. Thus, it is normally not acceptable to allow large woody vegetation to grow in the baskets. The possibility of damage must be weighed against the desirability of vegetation on the area protected by gabions and the stability of the large woody vegetation.

If large woody vegetation is kept out of the baskets, grasses and other desirable vegetation types may be established and provide a more aesthetic and ecologically desirable project than gabions alone.

4.4.9.2 DESIGN

Primary design considerations for gabions and mattresses are: 1) foundation stability; 2) sustained velocity and shear-stress thresholds that the gabions must withstand; and 3) toe and flank protection. The base layer of gabions should be placed below the expected maximum scour depth. Alternatively, the toe can be protected with mattresses that will fall into any scoured areas without compromising the stability of the bank or bed protection portion of the project. If bank protection does not extend above the expected water surface elevation for the design flood, measures such as tiebacks to protect against flanking should be installed.

The use of a filter fabric behind or under the gabion baskets to prevent the movement of soil material through the gabion baskets is an extremely important part of the design process. This migration of soil through the baskets can cause undermining of the supporting soil structure and failure of the gabion baskets and mattresses.

4.4.9.3 PRIMARY DESIGN CONSIDERATIONS

The major consideration in the design of gabion structures is the expected velocity at the gabion face. The gabion must be designed to withstand the force of the water in the stream.

Since gabion mattresses are much shallower and more subject to movement than gabion baskets, care should be taken to design the mattresses such that they can withstand the forces applied to them by the water. However, mattresses have been used in application where very high velocities are present and have performed well. But, projects using gabion mattresses should be carefully designed.

The median stone size for a gabion mattress can be determined from the following equation:

$$d_m = S_f C_s C_v d\left[\left(\frac{\gamma_w}{\gamma_s - \gamma_w}\right)^{0.5} \frac{V}{\sqrt{gdK_1}}\right]^{2.5} \qquad (1) \qquad \textbf{(4.4.21)}$$

The variables in the above equation are defined as:

C_s = stability coefficient (use 0.1)
C_v = velocity distribution coefficient
= 1.283-0.2 log *(R/W)* (minimum
of 1.0) and equals 1.25 at end

of dikes and concrete channels d_m = average rock diameter in gabions
d = local flow depth at V
9 = acceleration due to gravity $K_,$ = side slope correction factor (Table 4.4-8)
R = centerline bend radius of main
channel flow
Sf = safety factor (1.1 minimum) V = depth-averaged velocity
W = water surface width of main channel
,¥s = unit weight of stone ,¥w = unit weight of water

Table 4.4-8 K1 Versus Side Slope Angle

Side Slope	K1
1V : 1H	0.46
1V : 1.5H	0.71
1V : 2H	0.88
1V : 3H	0.98
<1V : 4H	1.0

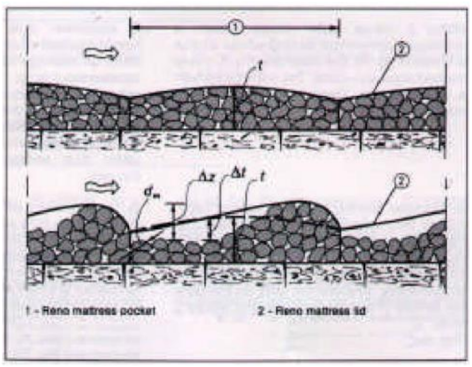


Figure 4.4-22 Gabion mattress showing deformation of mattress pockets under high velocities (courtesy Maccaferri Gabions)

Equation 4.4.21 was developed to design stone size such that the movement of filler stone in the mattresses is prevented. This eliminates deformation that can occur when stone sizes are not large enough to withstand the forces of the water. The result of mattress deformation (Figure 4.4-22) is stress on the basket wire and increases in resistance to flow and the likelihood of basket failure. The upper portion of Figure 4.4-22 shows an undeformed gabion, while the lower portion shows how gabions deform under high-velocity conditions.

Maccaferri Gabions offers a table in their materials giving guidance on sizing stone and allowable velocities for gabion baskets and mattresses. This is shown in Table 4.4-9.

Table 4.4-9 Stone Sizes and Allowable Velocities for Gabions (courtesy of and adapted from Maccaferri Gabions)

Type	Thickness (ft)	Filling Stone Range	D50	Critical* Velocity	Limit** Velocity
Mattress	0.5	3 - 4"	3.4"	11.5	13.8
	0.5	3 - 6"	4.3"	13.8	14.8
	0.75	3 - 4"	3.4"	14.8	16
	0.75	3 - 6"	4.7"	14.8	20
	1	3 - 5"	4"	13.6	18
	1	4 - 6"	5"	16.4	21
Basket	1.5	4 - 8"	6"	19	24.9
	1.5	5 - 10"	7.5"	21	26.2

When the data in Table 4.4-9 are compared to Equation 4.4.21, if $V = 11.5$, $C_s = 0.1$, $C_v = 1.0$, $K_1 = 0.71$, $\gamma_w = 150$ and $Sf = 1.1$, the local flow depth must be on the order of 25 ft in order to arrive at the stone diameter of 3.4 in. shown in Table 4.4-9. Designers should use Equation 4.4.21 to take the depth of flow into account. Table 4.4-9 does, however, give some general guidelines for fill sizes and is a quick reference for maximum allowable velocities.

Maccaferri also gives guidance on the stability of gabions in terms of shear stress limits. The following equation gives the shear for the bed of the channel:

$$\tau_b = \gamma_w S d \qquad \textbf{(4.4.22)}$$

with the bank shear τ_m taken as 75 percent of the bed shear, i.e. $\tau_m = 0.75\tau_b$. (S is the bed or water surface slope through the reach.) These values are then compared to the critical stress for the bed calculated by the following equation:

$$\tau_c = 0.10(\gamma_s - \gamma_w)d_m \qquad \textbf{(4.4.23)}$$

with critical shear stress for the banks given as:

$$\tau_s = \tau_c \sqrt{1 - \frac{\sin^2\theta}{0.4304}} \qquad \textbf{(4.4.24)}$$

where θ = the angle of the bank rotated up from horizontal.

A design is acceptable if $\tau_b < \tau_c$ and $\tau_m < \tau_s$. if either $\tau_b > \tau_c$ or $\tau_m > \tau_s$, then a check must be made to see if they are less than 120 percent of τ_b and τ_s. If the values are less than 120 percent of τ_b and τ_s, the gabions will not be subject to more than what Maccaferri defines as "acceptable" deformation. However, it is recommended that stone size be increased to limit deformation if possible.

Research has indicated that stone in the gabion mattress should be sized such that the largest stone diameter is not more than about two times the diameter of the smallest stone diameter and the mattress should be at least twice the depth of the largest stone size. The size range should, however, vary by about a factor of two to ensure proper packing of the stone material into the gabions. Since the mattresses normally come in discrete sizes, i.e. 0.5, 1.0, and 1.5 ft in depth, normal practice is to size the stone and then select the basket depth that is deep enough to be at least two times the largest stone diameter. The smallest stone should also be sized such that it cannot pass through the wire mesh.

4.4.9.4 Stability of Underlying Bed and Bank Materials

Another critical consideration is the stability of the gabion foundation. This includes both geotechnical stability and the resistance of the soil under the gabions to the erosive forces of the water moving through the gabions. If there is any question regarding the stability of the foundation, i.e. possibility of rotational failures, slip failures, etc., a qualified geotechnical engineer should be consulted prior to and during the design of the bank/channel protection. Several manufacturers give guidance on how to check for geotechnical failure (see Maccaferri Gabions brochure as an example).

Stacked gabion baskets used for bank stability should be tilted towards the soil they are protecting by a minimum of about 6 deg from vertical. Gabions are stacked using two methods. These are shown in Figure 4.4-23. While the gabions can be stacked with no tilt, it is recommended that some tilt into the soil being protected be provided.

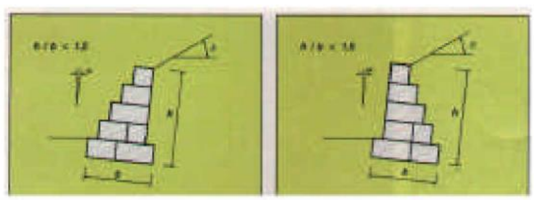


Figure 4.4-23 Front step and rear step gabion layout (courtesy of Maccaferri Gabions)

One of the critical factors in determining stability is the velocity of the water that passes through the gabions and reaches the soil behind the gabion. The water velocity under the filter fabric, i.e. water that moves through the gabions and filter fabric, is estimated to be one-fourth to one-half of the velocity at the mattress/filter interface. (Simons, Chen, and Swenson 1984) The velocity at the mattress/filter interface *(Vb)* is estimated to be

$$V_b = \frac{1.486}{n_f}\left(\frac{d_m}{2}\right)^{2/3} S^{1/2} \qquad \textbf{(4.4.25)}$$

where *n,* = 0.02 for filter fabric, 0.022 for gravel filter material and S is the water surface slope (or bed slope) through the reach. If the underlying soil material is not stable, additional filter material must be installed under the gabions to ensure soil stability. Maccaferri also provides guidance on the stability of soil under the gabions in terms of velocity criteria.

The limit for velocity on the soil is different for each type of soil. The limit for cohesive soils is obtained from a chart, while maximum allowable velocities for other soil types are obtained by calculating V_e, the maximum velocity allowable at the soil interface, and comparing it to V'' the residual velocity on the bed, i.e. under the gabion mattress and under the filter fabric. V_e for loose soils is equal to 16.1 $d^{1/2}$ while *V,* is calculated by:

$$V_f = \frac{1.486}{n_f}\left(\frac{d_m}{2}\right)^{2/3} SV_a^{\,1/2}$$

(4.4.26)

where *Va* is the average channel velocity and d_m is the average rock diameter.

If *V,* is larger than two to four times V_e, a gravel filter is required to further reduce the water velocity at the soil interface under the gabions until *V,* is in an acceptable range. To check for the acceptability of the filter use the average gravel size for d_m in Equation 4.4.22. If the velocity *V,* is still too high, the gravel size should be reduced to obtain an acceptable value for *V.*

Other Design Considerations

It may be possible to combine gabions with less harsh methods of bank protection on the upper bank and still achieve the desired result of a stable channel. Provisions for large woody vegetation and a more aesthetically pleasing project may also be used on the upper banks or within the gabions However, the stability of vegetation or other upper bank protection should be carefully analyzed to ensure stability of the upper bank area. A failure in the upper bank region can adversely affect gabion stability and lead to project failure.

4.4.10 – Uniform Flow - Example Problems

Example 1 -- Direct Solution of Manning's Equation

Use Manning's Equation to find the velocity, v, for an open channel with a hydraulic radius value of 0.6 ft, an n value of 0.020, and slope of 0.003 ft/ft. Solve using Figure 4.4-2:

1. Connect a line between the slope scale at 0.003 and the roughness scale at 0.020 and note the intersection point on the turning line.
2. Connect a line between that intersection point and the hydraulic radius scale at 0.6 ft and read the velocity of 2.9 ft/s from the velocity scale.

Example 2 -- Grassed Channel Design Stability

A trapezoidal channel is required to carry 50 cfs at a bottom slope of 0.015 ft/ft. Find the channel dimensions required for design stability criteria (retardance Class D) for a grass mixture.

1. From Table 4.4-3, the maximum velocity, v_m, for a grass mixture with a bottom slope less than 5% is 4 ft/s.
2. Assume an n value of 0.035 and find the value of vR from Figure 4.4-1, vR = 5.4
3. Use equation 4.4.9 to calculate the value of R: R = 5.4/4 = 1.35 ft
4. Use equation 4.4.10 to calculate the value of vR:

vR = [1.49 $(1.35)^{5/3}$ $(0.015)^{1/2}$]/0.035 = 8.60

5. Since the vR value calculated in Step 4 is higher than the value obtained from Step 2, a higher n value is required and calculations are repeated. The results from each trial of calculations are presented below:

Assumed		vR	R	vR
a.	n Value	(Figure 4.4-1)	(4.4.9)	(4.4.10)
b.	0.035	5.40	1.35	8.60
c.	0.038	3.8	0.95	4.41
d.	0.039	3.4	0.85	3.57
e.	0.040	3.2	0.80	3.15

Select n = 0.040 for stability criteria.

6. Use Figure 4.4-3 to select channel dimensions for a trapezoidal shape with 3:1 side slopes.

   Qn = (50) (0.040) = 2.0,   S = 0.015

   For b = 10 ft,   d = (10) (0.098) = 0.98 ft,   b = 8 ft,   d = (8) (0.14) = 1.12 ft

   Select:

   b = 10 ft, such that R is approximately 0.80 ft
   z = 3
   d = 1 ft
   v = 3.9 ft/s (equation 4.4.1)
   Fr = 0.76 (equation 4.4.8)
   Flow is subcritical

   Design capacity calculations for this channel are presented in Example 3 below.

Example 3 -- Grassed Channel Design Capacity

Use a 10-ft bottom width and 3:1 side-slopes for the trapezoidal channel sized in Example 2 and find the depth of flow for retardance Class C.

Assume a depth of 1.0 ft and calculate the following (see Figure 4.4-5):

$A = (b + zd)\, d = [10 + (3)\,(1)]\,(1) = 13.0$ square ft

$R = [(b + zd)\, d]/\{b + [2d(1 + z^2)^{0.5}]\} = \{[10+(3)(1)]1\}/\{10+[(2)(1)(1+3^2)^{0.5}]\}$

$R = 0.796$ ft

Find the velocity: v = Q/A = 50/13.0 = 3.85 ft/s

Find the value of vR: vR = (3.85) (0.796) = 3.06

Using the vR product from Step 3, find Manning's n from Figure 4.4-1 for retardance Class C (n = 0.047)

Use Figure 4.4-2 or equation 4.4.1 to find the velocity for S = 0.015, R = 0.796, and n = 0.047: <u>v = 3.34 ft/s</u>

Since 3.34 ft/s is less than 3.85 ft/s, a higher depth is required and calculations are repeated.

Results from each trial of calculations are presented below:

Assumed Depth (ft)	Area (ft^2)	R (ft)	Velocity Q/A (ft/sec)	vR	Manning's n (Fig. 4.4-1)	Velocity (4.4.1)
1.0	13.00	0.796	3.85	3.06	0.047	3.34
1.05	13.81	0.830	3.62	3.00	0.047	3.39
1.1	14.63	0.863	3.42	2.95	0.048	3.45
1.2	16.32	0.928	3.06	2.84	0.049	3.54

Select a depth of 1.1 with an n value of 0.048 for design capacity requirements. Add at least 0.2 ft for freeboard to give a design depth of 1.3 ft. Design data for the trapezoidal channel are summarized as follows:

Vegetation lining = grass mixture, v_m = 4 ft/s

Q = 50 cfs

b = 10 ft, d = 1.3 ft, z = 3, S = 0.015

Top width = (10) + (2) (3) (1.3) = 17.8 ft

n (stability) = 0.040, d = 1.0 ft, v = 3.9 ft/s, Froude number = 0.76 (equation 4.4.8)

n (capacity) = 0.048, d = 1.1 ft, v = 3.45 ft/s, Froude number = 0.64 (equation 4.4.8)

4.4.11 – Gradually Varied Flow

The most common occurrence of gradually varied flow in storm drainage is the backwater created by culverts, storm sewer inlets, or channel constrictions. For these conditions, the flow depth will be greater than normal depth in the channel and the water surface profile should be computed using backwater techniques.

Many computer programs are available for computation of backwater curves. The most general and widely used programs are, HEC-RAS, developed by the U.S. Army Corps of Engineers and Bridge Waterways Analysis Model (WSPRO) developed for the Federal Highway Administration. These programs can be used to compute water surface profiles for both natural and artificial channels.

For prismatic channels, the backwater calculation can be computed manually using the direct step method (TxDOT, 2002). For an irregular nonuniform channel, the standard step method is recommended, although it is a more tedious and iterative process. The use of HEC-RAS is recommended for standard step calculations.

Cross sections for water surface profile calculations should be normal to the direction of flood flow. The number of sections required will depend on the irregularity of the stream and flood plain. In general, a cross section should be obtained at each location where there are significant changes in stream width, shape, or vegetal patterns. Sections should usually be no more than 4 to 5 channel widths apart or 100 feet apart for ditches or streams and 500 feet apart for floodplains, unless the channel is very regular.

4.4.12 – Rectangular, Triangular and Trapezoidal Open Channel Design

4.4.12.1 Introduction

The Federal Highway Administration has prepared numerous design figures to aid in the design of open channels. Copies of these figures, a brief description of their use, and several example design problems are presented. For design conditions not covered by the figures, a trial and error solution of Manning's Equation must be used. However, it is anticipated that available software programs will be the first choice for solving these design computations.

4.4.12.2 Description of Figures

Figures given in FHWA, HDS No. 3, 1973 and Atlanta Regional Commission, 2001 are for the direct solution of the Manning's Equation for various sized open channels with rectangular, triangular, and trapezoidal cross sections. Each figure (except for the triangular cross section) is prepared for a channel of given bottom width and a particular value of Manning's n.

The figures for rectangular and trapezoidal cross section channels are used the same way. The abscissa scale of discharge in cubic feet per second (cfs), and the ordinate scale is velocity in feet per second (ft/s). Both scales are logarithmic. Superimposed on the logarithmic grid are steeply inclined lines representing depth (ft), and slightly inclined lines representing channel slope (ft/ft). A heavy dashed line on each figure shows critical flow conditions. Auxiliary abscissa and ordinate scales are provided for use with other values of n and are explained in the example problems. In the figures, interpolations may be made not only on the ordinate and abscissa scales but also between the inclined lines representing depth and slope.

The chart for a triangular cross section (see Figure 3.2-1) is in nomograph form. It may be used for street sections with a vertical (or nearly vertical) curb face. The nomograph also may be used for shallow V-shaped sections by following the instructions on the chart.

4.4.12.3 Instructions for Rectangular and Trapezoidal Figures

Figures in such as Figure 4.4-24 provide a solution of the Manning equation for flow in open channels of uniform slope, cross section, and roughness, provided the flow is not affected by backwater and the channel has a length sufficient to establish uniform flow.

For a given slope and channel cross section, when n is 0.015 for rectangular channels or 0.03 for trapezoidal channels, the depth and velocity of uniform flow may be read directly from the figure for that size channel. The initial step is to locate the intersection of a vertical line through the discharge (abscissa) and the appropriate slope line. At this intersection, the depth of flow is read from the depth lines, and the mean velocity is read on the ordinate scale.

The procedure is reversed to determine the discharge at a given depth of flow. Critical depth, slope, and velocity for a given discharge can be read on the appropriate scale at the intersection of the critical curve and a vertical line through the discharge.

Auxiliary scales, labeled Qn (abscissa) and Vn (ordinate), are provided so the figures can be used for values of n other than those for which the charts were basically prepared. To use these scales, multiply the discharge by the value of n and use the Qn and Vn scales instead of the Q and V scales, except for computation of critical depth or critical velocity. To obtain normal velocity V from a value on the Vn scale, divide the value by n. The following examples will illustrate these points.

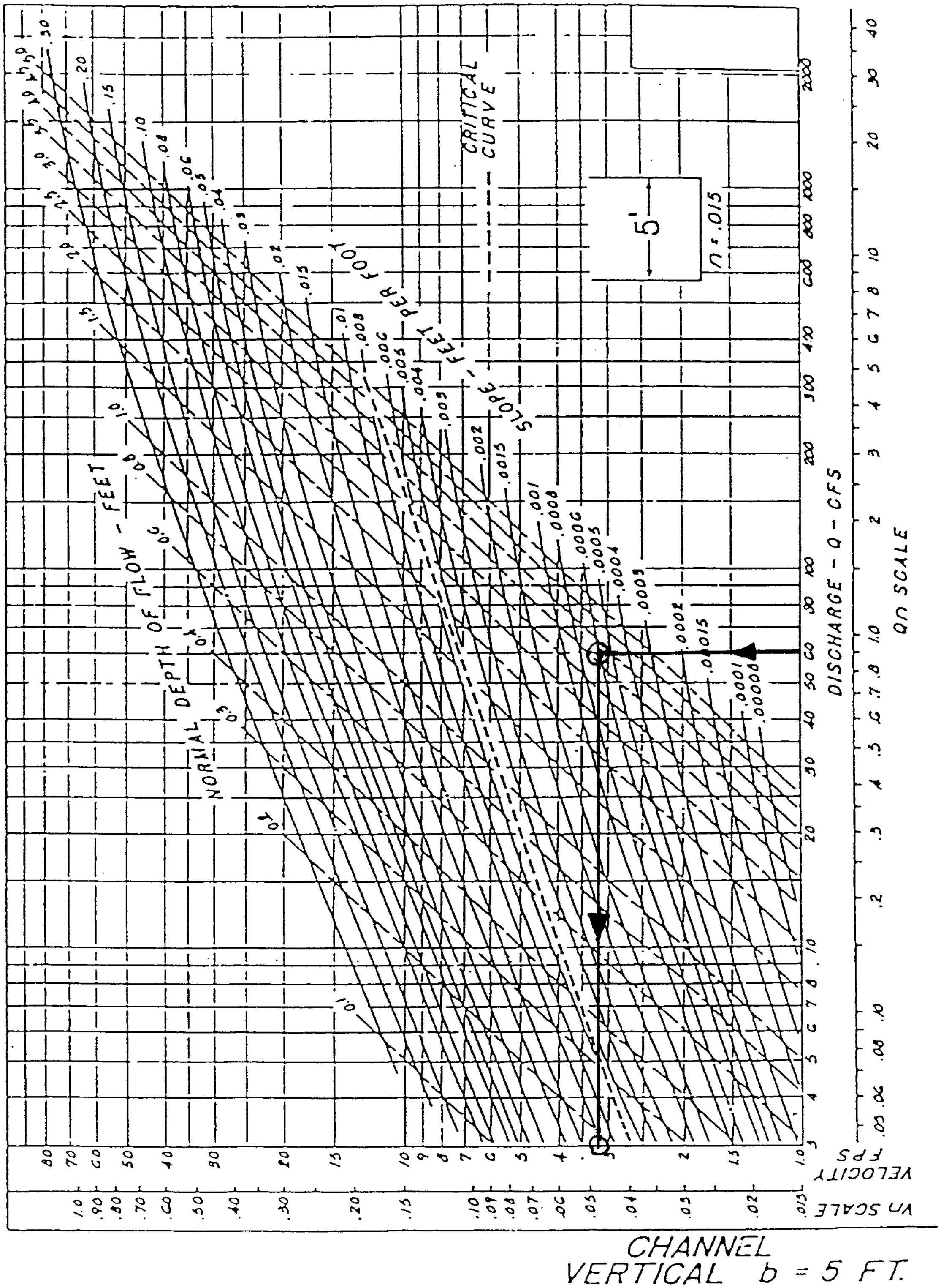
Example Design Problem 1

Given: A rectangular concrete channel 5 ft wide with n = 0.015, .06 percent slope (S = .0006), discharging 60 cfs.

Find: Depth, velocity, and type of flow

Procedure:

1. From subsection 4.4.12, select the rectangular figure for a 5-ft width (Figure 4.4-11).
2. From 60 cfs on the Q scale, move vertically to intersect the slope line S = .0006, and from the depth lines read d_n = 3.7 ft.
3. Move horizontally from the same intersection and read the normal velocity, V = 3.2 ft/s, on the ordinate scale.
4. The intersection lies below the critical curve, and the flow is therefore in the subcritical range.



Source: Federal Highway Administration

Figure 4.4-24 Example Nomograph #1

Example Design Problem 2

Given: A trapezoidal channel with 2:1 side slopes and a 4 ft bottom width, with n = 0.030, 0.2% slope (S = 0.002), discharging 50 cfs.

Find: Depth, velocity, type flow.

Procedure:

1. Select the trapezoidal figure for b = 4 ft (see Figure 4.4-25).
2. From 50 cfs on the Q scale, move vertically to intersect the slope line S = 0.002 and from the depth lines read d_n = 2.2 ft.
3. Move horizontally from the same intersection and read the normal velocity, V = 2.75 ft/s, on the ordinate scale. The intersection lies below the critical curve, and the flow is therefore subcritical.

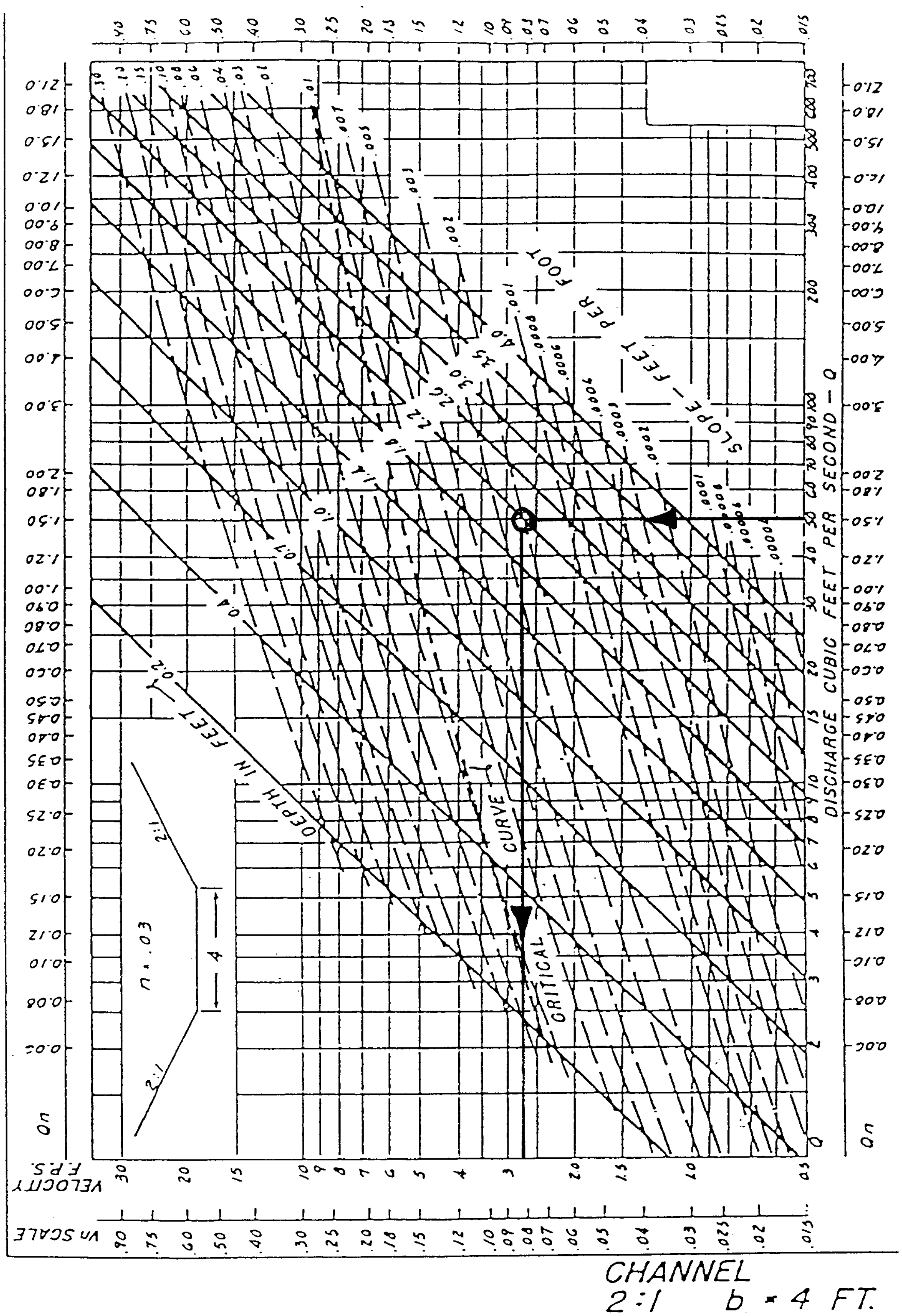
Example Design Problem 3

Given: A rectangular cement rubble masonry channel 5 ft wide, with n = 0.025, 0.5% slope (S = 0.005), discharging 80 cfs.

Find: Depth velocity and type of flow

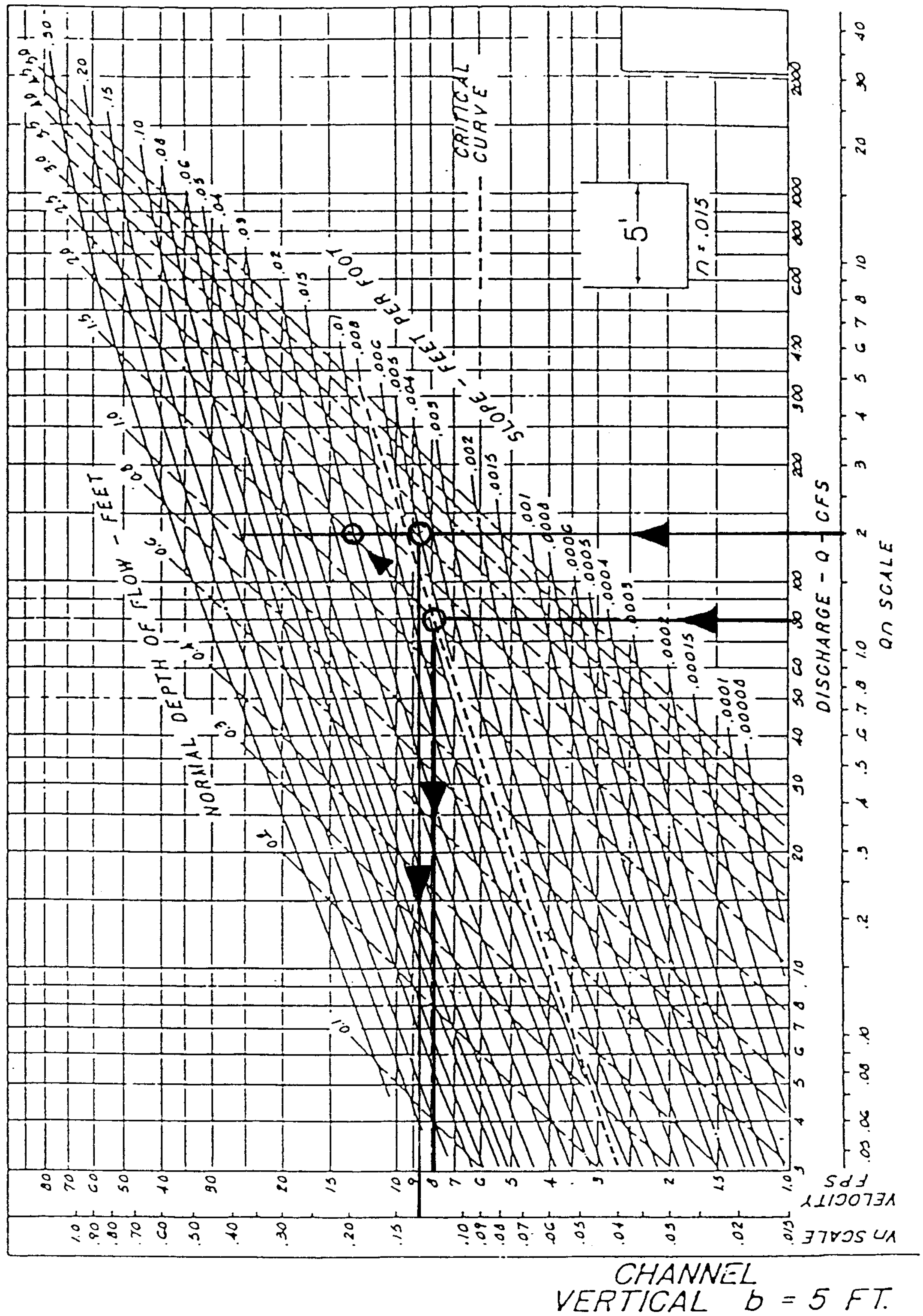
Procedure:

1. Select the rectangular figure for a 5 ft width (Figure 4.4-26).
2. Multiply Q by n to obtain Qn: 80 x 0.025 = 2.0.
3. From 2.0 on the Qn scale, move vertically to intersect the slope line, S = 0.005, and at the intersection read d_n = 3.1 ft.
4. Move horizontally from the intersection and read Vn = .13, then Vn/n = 0.13/0.025 = 5.2 ft/s.
5. Critical depth and critical velocity are independent of the value of n so their values can be read at the intersection of the critical curve with a vertical line through the discharge. For 80 cfs, on Figure 4.4-13, d_c = 2.0 ft and V_c = 7.9 ft/s. The normal velocity, 5.2 ft/s (from step 4), is less than the critical velocity, and the flow is therefore subcritical. It will also be noted that the normal depth, 3.0 ft, is greater than the critical depth, 2.0 ft, which also indicates subcritical flow.
6. To determine the critical slope for Q = 80 cfs and n = 0.025, start at the intersection of the critical curve and a vertical line through the discharge, Q = 80 cfs, finding d_c (2.0 ft) at this point. Follow along this d_c line to its intersection with a vertical line through Qn = 2.0 (step 2), at this intersection read the slope value S_c = 0.015.



Source: Federal Highway Administration

Figure 4.4-25 Example Nomograph #2



Source: Federal Highway Administration

Figure 4.4-26 Example Nomograph #3

4.4.12.4 Grassed Channel Figures

The Manning equation can be used to determine the capacity of a grass-lined channel, but the value of n varies with the type of grass, development of the grass cover, depth, and velocity of flow. The variable value of n complicates the solution of the Manning equation. The depth and velocity of flow must be estimated and the Manning equation solved using the n value that corresponds to the estimated depth and velocity. The trial solution provides better estimates of the depth and velocity for a new value of n and the equation is again solved. The procedure is repeated until a depth is found that carries the design discharge.

To prevent excessive erosion, the velocity of flow in a grass-lined channel must be kept below some maximum value (referred to as permissible velocity). The permissible velocity in a grass-lined channel depends upon the type of grass, condition of the grass cover, texture of the soil comprising the channel bed, channel slope, and to some extent the size and shape of the drainage channel. To guard against overtopping, the channel capacity should be computed for taller grass than is expected to be maintained, while the velocity used to check the adequacy of the protection should be computed assuming a lower grass height than will likely be maintained.

To aid in the design of grassed channels, the Federal Highway Administration has prepared numerous design figures. Copies of these figures are in subsection 4.4.14. Following is a brief description of general design criteria, instructions on how to use the figures, and several example design problems. For design conditions not covered by the figures, a trial-and-error solution of the Manning equation must be used.

4.4.12.5 Description of Figures

A set of figures in FHWA, NDS No. 3, 1973 and Atlanta Regional Commission, 2001 are designed for use in the direct solution of the Manning equation for various channel sections lined with grass. The figures are similar in appearance and use to those for trapezoidal cross sections described earlier. However, their construction is much more difficult because the roughness coefficient (n) changes as higher velocities and/or greater depths change the condition of the grass. The effect of velocity and depth of flow on n is evaluated by the product of velocity and hydraulic radius V times R. The variation of Manning's n with the retardance (Table 4.4-6) and the product V times R is shown in Figure 4.4-1. As indicated in Table 4.4-6, retardance varies with the height of the grass and the condition of the stand. Both of these factors depend upon the type of grass, planting conditions, and maintenance practices. Table 4.4-6 is used to determine retardance classification.

The grassed channel figures each have two graphs, the upper graph for retardance Class D and the lower graph for retardance Class C. The figures are plotted with discharge in cubic feet per second on the abscissa and slope in feet per foot on the ordinate. Both scales are logarithmic.

Superimposed on the logarithmic grid are lines for velocity in feet per second and lines for depth in feet. A dashed line shows the position of critical flow.

4.4.12.5 Instructions for Grassed Channel Figures

The grassed channel figures like those in Figure 4.4-11 provide a solution of the Manning equation for flow in open grassed channels of uniform slope and cross section. The flow should not be affected by backwater and the channel should have length sufficient to establish uniform flow. The figures are sufficiently accurate for design of drainage channels of fairly uniform cross section and slope, but are not appropriate for irregular natural channels.

The design of grassed channels requires two operations: (1) selecting a section that has the capacity to carry the design discharge on the available slope and (2) checking the velocity in the channel to ensure that the grass lining will not be eroded. Because the retardance of the channel is largely beyond the control of the designer, it is good practice to compute the channel capacity using retardance Class C and the velocity using retardance Class D. The calculated velocity should then be checked against the permissible velocities listed in Tables 4.4-2 and 4.4-3. The use of the figures is explained in the following steps:

Step 1 Select the channel cross section to be used and find the appropriate figure.

Step 2 Enter the lower graph (for retardance Class C) on the figure with the design discharge value on the abscissa and move vertically to the value of the slope on the ordinate scale. As this intersection, read the normal velocity and normal depth and note the position of the critical curve. If the intersection point is below the critical curve, the flow is subcritical; if it is above, the flow is supercritical.

Step 3 To check the velocity developed against the permissible velocities (Tables 4.4-2 and 4.4-3), enter the upper graph on the same figure and repeat Step 2. Then compare the computed velocity with the velocity permissible for the type of grass, channel slope, and erosion resistance of the soil. If the computed velocity is less, the design is acceptable. If not, a different channel section must be selected and the process repeated.

Example Design Problem 1

Given: A trapezoidal channel in easily eroded soil, lined with a grass mixture with 4:1 side slopes, and a 4 ft bottom width on slope of 0.02 ft per foot (S=.02), discharging 20 cfs.

Find: Depth, velocity, type of flow, and adequacy of grass to prevent erosion

Procedure:

1. From subsection 4.4.13 select figure for 4:1 side slopes (see Figure 4.4-27).
2. Enter the lower graph with Q = 20 cfs, and move vertically to the line for S=0.02. At this intersection read d_n = 1.0 ft, and normal velocity V_n 2.6 ft/s.
3. The velocity for checking the adequacy of the grass cover should be obtained from the upper graph, for retardance Class D. Using the same procedure as in step 2, the velocity is found to be 3.0 ft/s. This is about three-quarters of that listed as permissible, 4.0 ft/s in Table 4.4-3.

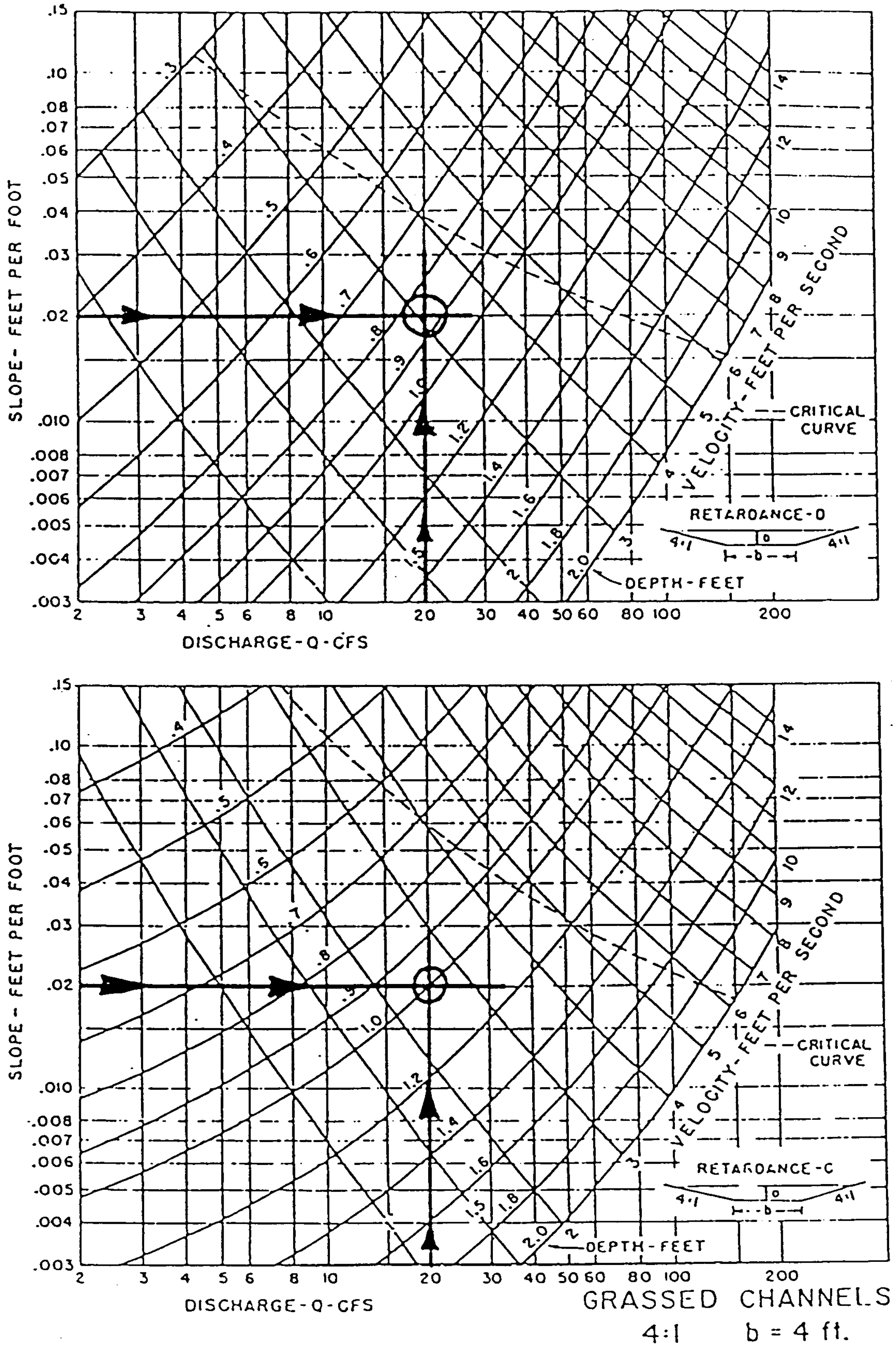
Example Design Problem 2

Given: The channel and discharge of Example 1.

Find: The maximum grade on which the 20 cfs could safely be carried

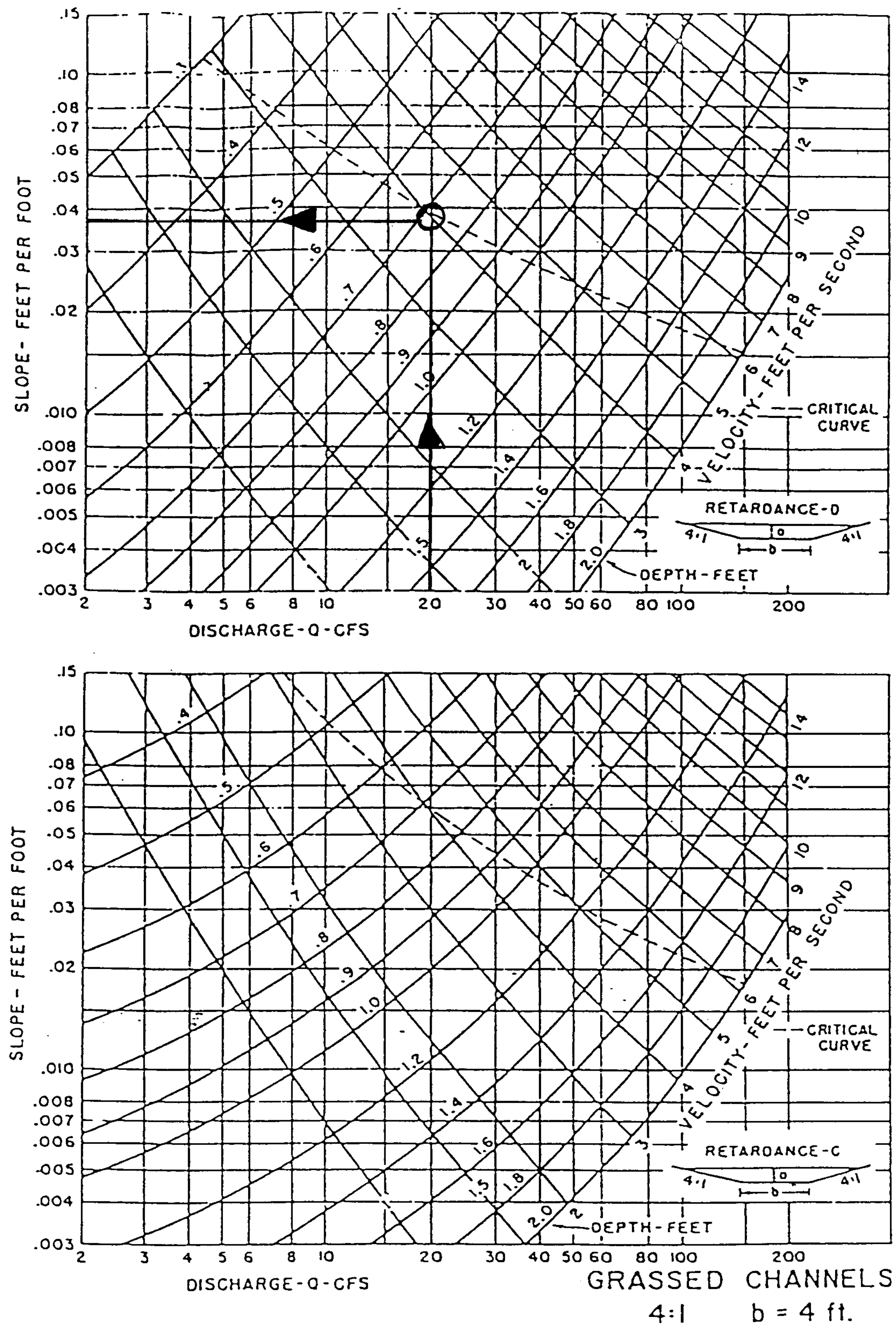
Procedure:

With an increase in slope (but still less than 5%), the allowable velocity is estimated to be 4 ft/s (see Table 4.4-3). On the upper graph of Figure 4.4-28 for short grass, the intersection of the 20 cfs line and the 4 ft/s line indicates a slope of 3.7% and a depth of 0.73 ft.



Source: Federal Highway Administration

Figure 4.4-27 Example Nomograph #4



Source: Federal Highway Administration

Figure 4.4-28 Example Nomograph #5

Section 4.5 – Storage Design

4.5.1 – General Storage Concepts

4.5.1.1 Introduction

This section provides general guidance on storm water runoff storage for meeting storm water management control objectives.

Storage of storm water runoff within a storm water management system is essential to providing detention of flows for water quality enhancement and downstream streambank protection, as well as for peak flow attenuation of larger flows for flood protection. Runoff storage can be provided within an on-site system through the use of structural storm water controls and/or nonstructural features and landscaped areas. Figure 4.5-1 illustrates various storage facilities that can be considered for a development site.

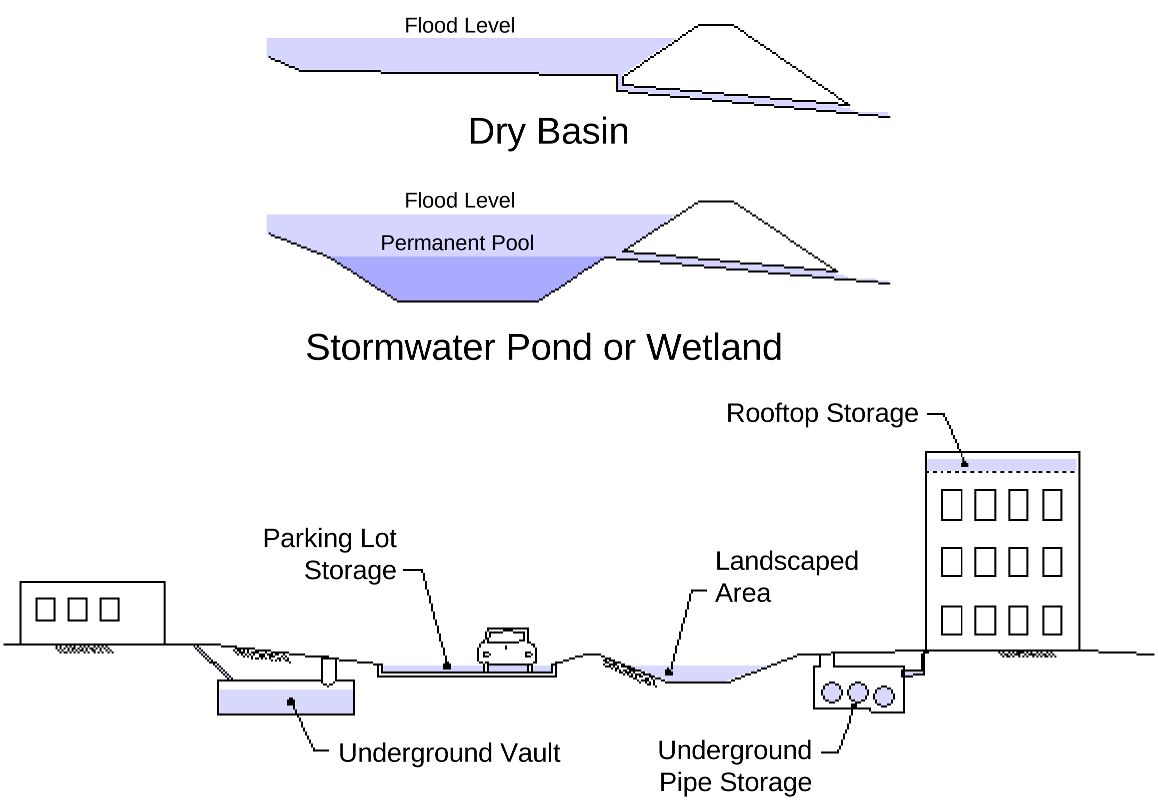


Figure 4.5-1 Examples of Typical Storm Water Storage Facilities

4.5.1.2 Storage Classification

Storm water storage(s) can be classified as either detention, extended detention or retention. Some facilities include one or more types of storage.

Storm water *detention* is used to reduce the peak discharge and detain runoff for a specified short period of time. Detention volumes are designed to completely drain after the design storm has passed. Detention is used to meet streambank protection criteria, and flood criteria where required.

Extended detention (ED) is used to drain a runoff volume over a specified period of time, typically 24 hours, and is used to meet streambank protection criteria. Some structural control designs (wet ED pond, micropool ED pond, and shallow ED marsh) also include extended detention storage of a portion of the water quality protection volume. **(NOT currently required by Copper Canyon).**

Retention facilities are designed to contain a permanent pool of water, such as storm water ponds and wetlands, which is used for water quality protection.

Storage facilities are often classified on the basis of their location and size. *On-site* storage is constructed on individual development sites. *Regional* storage facilities are constructed at the lower end of a subwatershed and are designed to manage storm water runoff from multiple projects and/or properties. A discussion of regional storm water controls is found in Appendix G.

Storage can also be categorized as *on-line* or *off-line*. On-line storage uses a structural control facility that intercepts flows directly within a conveyance system or stream. Off-line storage is a separate storage facility to which flow is diverted from the conveyance system. Figure 4.5-2 illustrates on-line versus off-line storage.

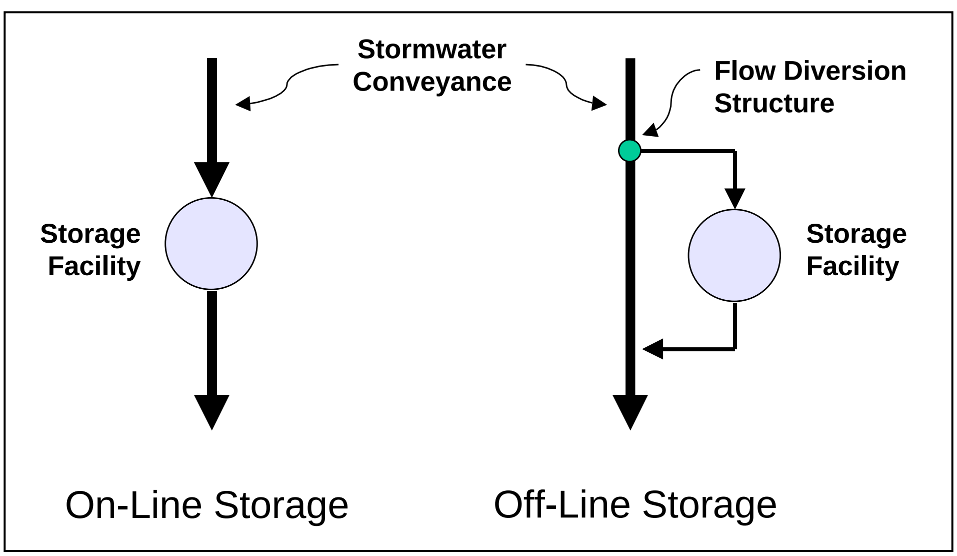


Figure 4.5-2 On-Line versus Off-Line Storage

4.5.1.3 Stage-Storage Relationship

A stage-storage curve defines the relationship between the depth of water and storage volume in a storage facility (see Figure 4.5-3). The volume of storage can be calculated by using simple geometric formulas expressed as a function of depth.

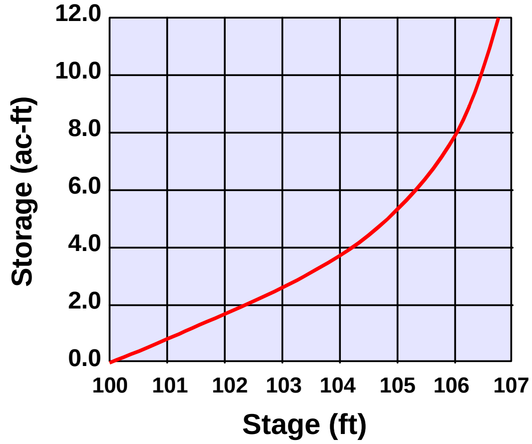


Figure 4.5-3 Stage-Storage Curve

The storage volume for natural basins may be developed using a topographic map and the double-end area, frustum of a pyramid, prismoidal or circular conic section formulas.

The double-end area formula (see Figure 4.5.1-4) is expressed as:

$$V_{1,2} = [(A_1 + A_2)/2]d \qquad \textbf{(4.5.1)}$$

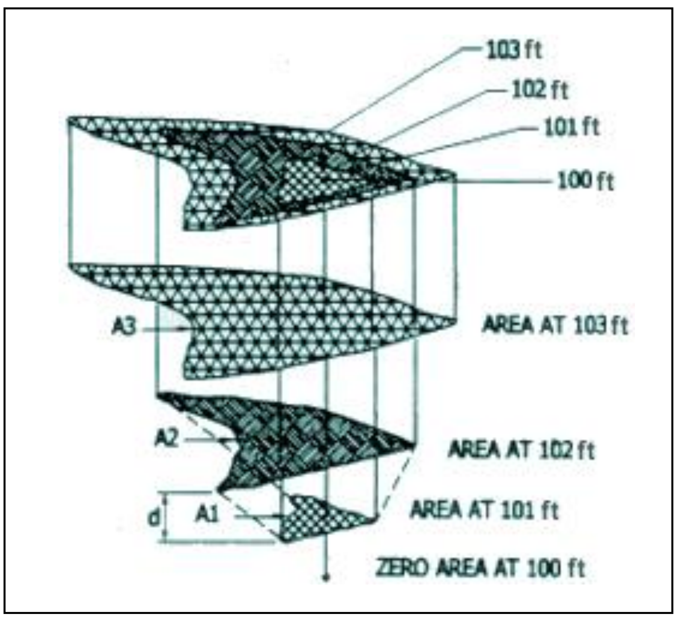


Figure 4.5-4 Double-End Area Method

where:

$V_{1,2}$ = storage volume (ft^3) between elevations 1 and 2

A_1 = surface area at elevation 1 (ft^2)

A_2 = surface area at elevation 2 (ft^2)

d = change in elevation between points 1 and 2 (ft)

The frustum of a pyramid formula is expressed as:

$$\mathbf{V = d/3\ [A_1 + (A_1 \times A_2)^{0.5} + A_2]/3} \qquad \mathbf{(4.5.2)}$$

where:

V = volume of frustum of a pyramid (ft^3)

d = change in elevation between points 1 and 2 (ft)

A_1 = surface area at elevation 1 (ft^2)

A_2 = surface area at elevation 2 (ft^2)

The prismoidal formula for trapezoidal basins is expressed as:

$$\mathbf{V = LWD + (L + W)\ ZD^2 + 4/3\ Z^2\ D^3} \qquad \mathbf{(4.5.3)}$$

where:

V = volume of trapezoidal basin (ft^3)

L = length of basin at base (ft)

W = width of basin at base (ft)

D = depth of basin (ft)

Z = side slope factor, ratio of horizontal to vertical

The circular conic section formula is:

$$\mathbf{V = 1.047\ D\ (R_1^2 + R_2^2 + R_1R_2)} \qquad \mathbf{(4.5.4)}$$

$$\mathbf{V = 1.047\ D\ (3\ R_1^2 + 3ZDR_1 + Z_2D^2)} \qquad \mathbf{(4.5.5)}$$

where:

R_1, R_2 = bottom and surface radii of the conic section (ft)

D = depth of basin (ft)

Z = side slope factor, ratio of horizontal to vertical

4.5.1.4 Stage-Discharge Relationship

A stage-discharge curve defines the relationship between the depth of water and the discharge or outflow from a storage facility (see Figure 4.5-5). A typical storage facility has two outlets or spillways: a principal outlet and a secondary (or emergency) outlet. The principal outlet is usually designed with a capacity sufficient to convey the design flows without allowing flow to enter the emergency spillway. A pipe culvert, weir, or other appropriate outlet can be used for the principal spillway or outlet.

The emergency spillway is sized to provide a bypass for floodwater during a flood that exceeds the design capacity of the principal outlet. This spillway should be designed taking into account the potential threat to downstream areas if the storage facility were to fail. The stage-discharge curve should take into account the discharge characteristics of both the principal spillway and the emergency spillway. For more details, see Section 4.6, *Outlet Structures*.

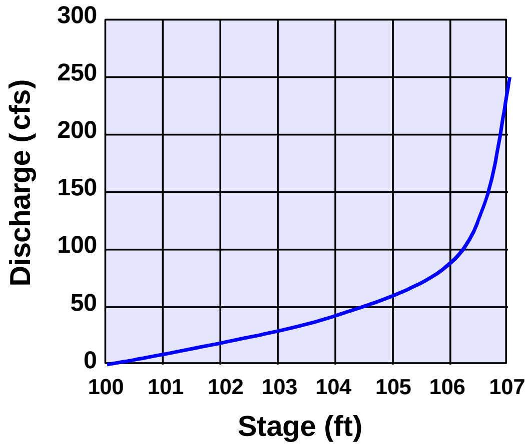


Figure 4.5-5 Stage-Discharge Curve

The purpose of the mitigation is to minimize downstream flooding impacts or streambank erosion from upstream development. In some instances, detention may be shown to exacerbate potential flooding conditions downstream. Therefore, the "Zone of Influence" criteria (Reference Section 2.1.8.2 shall be applied in addition to these criteria.)

4.5.2 – Symbols and Definitions

To provide consistency within this section as well as throughout this Manual, the symbols listed in Table 4.5.2-1 will be used. These symbols were selected because of their wide use in technical publications. In some cases, the same symbol is used in existing publications for more than one definition. Where this occurs in this section, the symbol will be defined where it occurs in the text or equations.

Table 4.5.2-1 Symbols and Definitions

Symbol	Definition	Units
A	Cross sectional or surface area	ft^2
A_m	Drainage area	mi^2
C	Weir coefficient	-
d	Change in elevation	ft
D	Depth of basin or diameter of pipe	ft
t	Routing time period	sec
g	Acceleration due to gravity	ft/s^2
H	Head on structure	ft
H_C	Height of weir crest above channel bottom	ft
K	Coefficient	-
I	Inflow rate	cfs
L	Length	ft
Q, q	Peak inflow or outflow rate	cfs, in

R	Surface Radii	ft
S, V_S	Storage volume	ft^3
t_b	Time base on hydrograph	hrs
T_I	Duration of basin inflow	hrs
t_P	Time to peak	hrs
V_S, S	Storage volume	ft^3, in, acre-ft
V_r	Volume of runoff	ft^3, in, acre-ft
W	Width of basin	ft
Z	Side slope factor	-

4.5.3 – General Storage Design Procedures

4.5.3.1 Introduction

This section discusses the general design procedures for designing storage to provide standard detention of storm water runoff for flood control (Q_f).

The design procedures for all structural control storage facilities are the same whether or not they include a permanent pool of water. In the latter case, the permanent pool elevation is taken as the "bottom" of storage and is treated as if it were a solid basin bottom for routing purposes.

It should be noted that the location of structural storm water controls is very important as it relates to the effectiveness of these facilities to control downstream impacts. In addition, multiple storage facilities located in the same drainage basin will affect the timing of the runoff through the conveyance system, which could decrease or increase flood peaks in different downstream locations. Therefore, a downstream peak flow analysis should be performed as part of the storage facility design process (see subsection 2.1.8).

In multi-purpose multi-stage facilities such as storm water ponds, the design of storage must be integrated with the overall design for water quality protection objectives. See Appendix G for further guidance and criteria for the design of structural storm water controls.

"Dry" Detention Basins

1. Detention Basins shall be required when downstream facilities within the "Zone of Influence" are not adequately sized to convey a design storm based on current Town criteria for hydraulic capacity. Detention basins may not be required if downstream improvements that will result in sufficient hydraulic capacity are proposed by the Town within a relatively short period of time.
2. Calculated proposed storm water discharge from a site shall not exceed the calculated discharges from existing conditions, unless sufficient downstream capacity above existing discharge conditions is available.
3. The Preliminary Detention Calculation Method (Section 4.5.4) is allowed for planning and conceptual design for watersheds of 100 acres and less. For final design purposes this Method is allowed only for watersheds of 10 acres and less (see Table 2.1.1-2).
4. Detention Basins draining watersheds over 25 acres shall be designed using a detailed unit hydrograph method acceptable to the Town of Copper Canyon. The preferred unit

hydrograph method is the SCS Dimensionless Unit Hydrograph Method. The Snyder's Unit Hydrograph Method may be accepted with approval of the TOWN ENGINEER.

5. Detention Basins shall be designed for the 2-year, 10-year and 100-year storm for the critical storm duration (i.e. 3-hour, 6-hour, or 24-hour storm duration) that results in the maximum (or near maximum) peak flow.
6. Detention Basins shall be designed with access for tracked earthwork equipment with a 10-foot crown width on any embankment.
7. Earthen (grassed) embankment slopes shall NOT exceed 4:1. Concrete lined or structural embankment can be steeper with the approval of the TOWN ENGINEER.
8. A calculation summary shall be provided on construction plans. For detailed calculations of unit hydrograph studies, a separate report shall be provided to the Town for review and referenced on the construction plans. Stage-storage-discharge values shall be tabulated and flow calculations for discharge structures shall be shown on the construction plans.
9. An emergency spillway shall be provided at the 100-year maximum storage elevation with sufficient capacity to convey the fully urbanized 100-year storm assuming blockage of the closed conduit portion outlet works with six inches of freeboard. Spillway requirements must also meet all appropriate state and Federal criteria.
10. Design calculations will be provided for all spillways.
11. All detention basins shall be stabilized against significant erosion and include a maintenance plan.
12. State rules and regulations regarding impoundments shall be observed including 30 TAC Chapter 299, Dams and Reservoirs (TCEQ).
13. In accordance with Texas Water Code §11, all surface impoundments not used for domestic or livestock purposes must obtain a water rights permit from the TCEQ. A completed permit for the proposed use, or written documentation stating that a permit is not required, must be obtained. All detention facility designs shall include a landscaping plan

"Wet" Detention Basins and Amenity Ponds

Wet detention basins maintain a permanent pool with additional storage capacity to detain storm water. Amenity ponds may or may not include this additional storage. The depth of a wet or amenity pond is generally seven (7) to ten (10) feet to prevent algal growth, although greater depths are possible with artificial mixing. The objective is to avoid thermal stratification that could result in odor problems or recycling of nutrients. Gentle artificial mixing may be needed in small ponds because they are effectively sheltered from the wind. If properly designed, constructed, and maintained, wet ponds will not only reduce peak storm water flows, but also improve water quality and can be an attractive feature of a development.

Below are guidelines for wet detention basins in addition to those presented under "Dry" Detention Basins.

1. Must be appropriately aerated according to normal pool size unless specifically approved by TOWN Engineer.
2. Provisions shall be made to ensure that normal water surface elevation is maintained through the use of ground wells or the Domestic water supply unless surface water supply can be justified based on drainage area to pond. (general requirement is 5 acres of drainage area for every acre-foot of normal pool storage).
3. Ten-foot (10') wide maintenance access shall be provided with a slope of 6:1 or flatter.
4. A debris filter must be provided for all outlet structures.
5. Design shall provide adequate capacity for trapped sediment for five (5) years.
6. To minimize short-circuiting, the inlet and outlet should be placed at opposite ends of the pond or baffling shall be installed to direct the water to the opposite end before returning to the outlet. Dead space should be avoided.

7. To limit water loss by infiltration through the bottom of the pond either an artificial liner or a clay liner may be used. Natural material may be used if a geotechnical report is provided to assure it will not leach out the bottom or sides of the pond.
8. Reference Appendix G, Section 5.2.21 “Storm Water Ponds” for additional guidance on the design of Wet Ponds. The water quality and streambank protection criteria described in this Appendix section are not currently required by the Town.

4.5.3.2 Data Needs

The following data are needed for storage design and routing calculations:

- Inflow hydrograph for all selected design storms
- Stage-storage curve for proposed storage facility
- Stage-discharge curve for all outlet control structures

4.5.3.3 Design Procedure

A general procedure for using the above data in the design of storage facilities is presented below.

Step 1 Compute inflow hydrograph for runoff from the 2 year design storms using the hydrologic methods outlined in Section 2.1. Both existing- and post-development hydrographs are required for the design storm.

Step 2 Perform preliminary calculations to evaluate detention storage requirements for the hydrographs from Step 1 (see subsection 4.5.4).

Step 3 Determine the physical dimensions necessary to hold the estimated volume from Step 2, including freeboard. The maximum storage requirement calculated from Step 2 should be used. From the selected shape determine the maximum depth in the pond.

Step 4 Select the type of outlet and size the outlet structure. The estimated peak stage will occur for the estimated volume from Step 2. The outlet structure should be sized to convey the allowable discharge at this stage.

Step 5 Perform routing calculations using inflow hydrographs from Step 1 to check the preliminary design using a storage routing computer model. If the routed post-development peak discharges from the 10-year design storm exceed the existing-development peak discharges, then revise the available storage volume, outlet device, etc., and return to Step 3.

Step 6 Perform routing calculations using the 100-year hydrograph to determine if any increases in downstream flows from this hydrograph will cause damages and/or drainage and flooding problems. If problems will be created (e.g., flooding of habitable dwellings, property damage, or public access and/or utility interruption) then the storage facility must be designed to control the increased flows from the 100-year storm. If not then consider emergency overflow from runoff due to the 100-year (or larger) design storm and established freeboard requirements.

Step 7 Evaluate the downstream effects of detention outflows for the 10- and 100-year storms to ensure that the routed hydrograph does not cause downstream flooding problems. The exit hydrograph from the storage facility should be routed though the downstream channel system until a confluence point is reached where the drainage area being analyzed represents 10% of the total drainage area (see subsection 2.1.11).

Step 8 Evaluate the control structure outlet velocity and provide channel and bank stabilization if the velocity will cause erosion problems downstream.

Routing of hydrographs through storage facilities is critical to the proper design of these facilities. Although storage design procedures using inflow/outflow analysis without routing have been developed, their use in designing detention facilities has not produced acceptable results in many areas of the country, including North Central Texas.

Although hand calculation procedures are available for routing hydrographs through storage facilities, they are very time consuming, especially when several different designs are evaluated. Many standard hydrology and hydraulics textbooks give examples of hand-routing techniques. For this Manual, it assumed that designers will be using one of the many computer programs available for storage routing and thus other procedures and example applications will not be given here.

4.5.4 – Preliminary Detention Calculations

4.5.4.1 Introduction

Procedures for preliminary detention calculations are included here to provide a simple method that can be used to estimate storage needs and also provide a quick check on the results of using different computer programs. Standard routing should be used for actual (final) storage facility calculations and design.

4.5.4.2 Storage Volume

For small drainage areas, a preliminary estimate of the storage volume required for peak flow attenuation may be obtained from a simplified design procedure that replaces the actual inflow and outflow hydrographs with the standard triangular shapes shown in Figure 4.5.4-1.

The required storage volume may be estimated from the area above the outflow hydrograph and inside the inflow hydrograph, expressed as:

$$V_S = 0.5T_i\,(Q_i - Q_O) \qquad \textbf{(4.5.6)}$$

where:

V_S = storage volume estimate (ft^3)
Q_i = peak inflow rate (cfs)
Q_O = peak outflow rate (cfs)
T_i = duration of basin inflow (s)

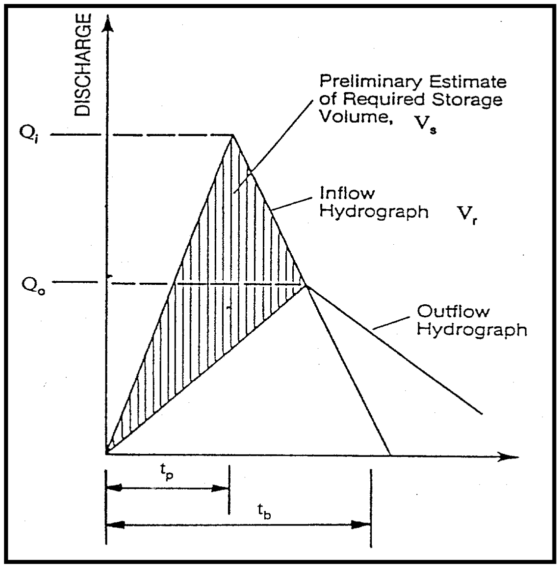


Figure 4.5-6 Triangular-Shaped Hydrographs (For Preliminary Estimate of Required Storage Volume)

4.5.4.3 Alternative Method

An alternative preliminary estimate of the storage volume required for a specified peak flow reduction can be obtained by the following regression equation procedure (Wycoff and Singh, 1976).

Determine input data, including the allowable peak outflow rate, Q_O, the peak flow rate of the inflow hydrograph, Q_i, the time base of the inflow hydrograph, t_b, and the time to peak of the inflow hydrograph, t_p.

Calculate a preliminary estimate of the ratio V_S/V_r using the input data from Step 1 and the following equation:

$$\frac{V_S}{V_R} = \frac{1.291\left(1 - \frac{Q_O}{Q_I}\right)^{0.753}}{\left(\frac{t_p}{T_B}\right)^{0.753}} \qquad (4.5.7)$$

where:

V_S = volume of storage (in)
V_r = volume of runoff (in)
Q_O = outflow peak flow (cfs)
Q_i = inflow peak flow (cfs)
t_b = time base of the inflow hydrograph (hr) [Determined as the time from the beginning of rise to a point on the recession limb where the flow is 5% of the peak]
t_p = time to peak of the inflow hydrograph (hr)

Multiply the volume of runoff, V_r, times the ratio V_S/V_r, calculated in Step 2 to obtain the estimated storage volume V_S.

4.5.4.4 Peak Flow Reduction

A preliminary estimate of the potential peak flow reduction for a selected storage volume can be obtained by the following procedure.

Determine volume of runoff, V_r, peak flow rate of the inflow hydrograph, Q_i, time base of the inflow hydrograph, t_b, time to peak of the inflow hydrograph, t_p, and storage volume V_S.

Calculate a preliminary estimate of the potential peak flow reduction for the selected storage volume using the following equation (Wycoff and Singh, 1976):

$$Q_O/Q_i = 1 - 0.712(V_S/V_r)^{1.328}(t_b/t_p)^{0.546} \qquad \textbf{(4.5.8)}$$

where:

Q_O = outflow peak flow (cfs)
Q_i = inflow peak flow (cfs)
V_S = volume of storage (in)
V_r = volume of runoff (in)
t_b = time base of the inflow hydrograph (hr) [Determined as the time from the beginning of rise to a point on the recession limb where the flow is 5 percent of the peak]
t_p = time to peak of the inflow hydrograph (hr)

Multiply the peak flow rate of the inflow hydrograph, Q_i, times the potential peak flow reduction calculated from Step 2 to obtain the estimated peak outflow rate, Q_O, for the selected storage volume.

Section 4.6 – Outlet Structures

4.6.1 – Symbols and Definitions

To provide consistency within this section as well as throughout this Manual, the symbols listed in Table 4.6.1-1 will be used. These symbols were selected because of their wide use in technical publications. In some cases, the same symbol is used in existing publications for more than one definition. Where this occurs in this section, the symbol will be defined where it occurs in the text or equations.

Table 4.6.1-1 Symbols and Definitions

Symbol	Definition	Units
A, a	Cross sectional or surface area	ft^2
A_m	Drainage area	mi^2
B	Breadth of weir	ft
C	Weir coefficient	-
d	Change in elevation	ft
D	Depth of basin or diameter of pipe	ft
g	Acceleration due to gravity	ft/s^2
H	Head on structure	ft
H_C	Height of weir crest above channel bottom	ft
K, k	Coefficient	-
L	Length	ft
n	Manning’s n	-
Q, q	Peak inflow or outflow rate	cfs, in
V_u	Approach velocity	ft/s
WQ_v	Water quality protection volume	ac ft
w	Maximum cross sectional bar width facing the flow	in
x	Minimum clear spacing between bars	in
θ	Angle of v-notch	degrees
θ_g	Angle of the grate with respect to the horizontal	degrees

4.6.2 – Primary Outlets

4.6.2.1 Introduction

Primary outlets provide the critical function of the regulation of flow for structural storm water controls. There are several different types of outlets that may consist of a single stage outlet structure, or several outlet structures combined to provide multi-stage outlet control.

For a single stage system, the storm water facility can be designed as a simple pipe or culvert. For multistage control structures, the inlet is designed considering a range of design flows.

A stage-discharge curve is developed for the full range of flows that the structure would experience. The outlets are housed in a riser structure connected to a single outlet conduit. An alternative approach would be to provide several pipe or culvert outlets at different levels in the basin that are either discharged separately or are combined to discharge at a single location.

This section provides an overview of outlet structure hydraulics and design for storm water storage facilities. The design engineer is referred to an appropriate hydraulics text for additional information on outlet structures not contained in this section.

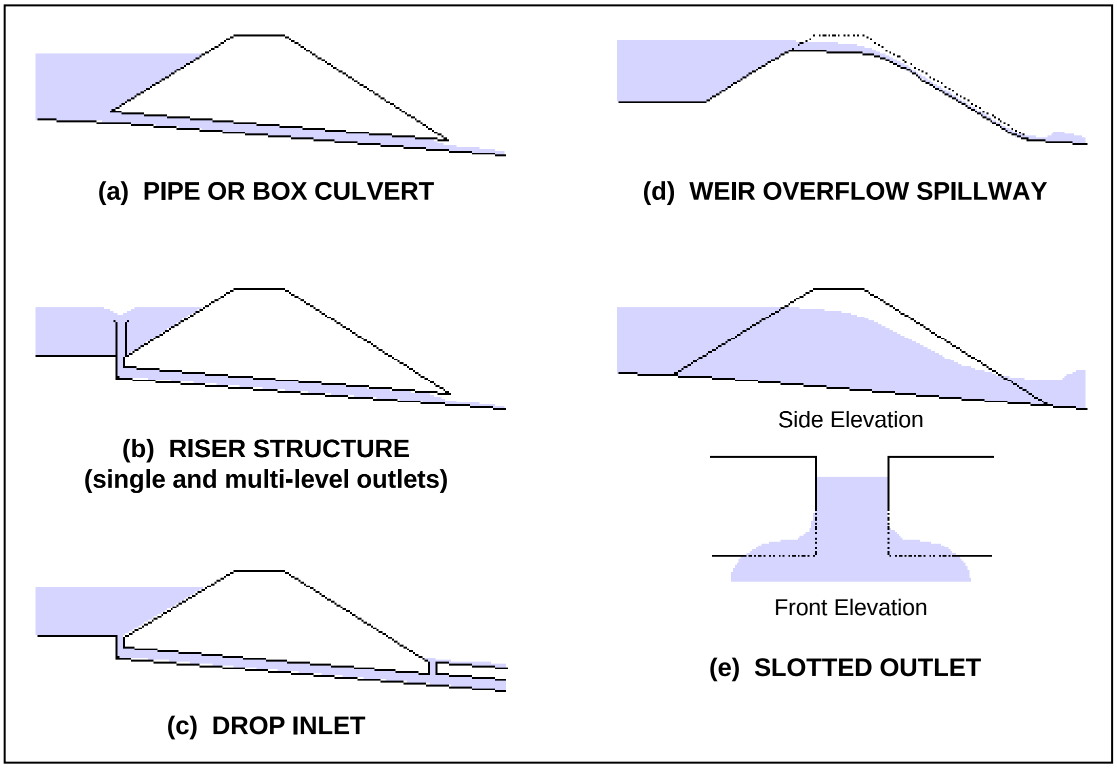


Figure 4.6-1 Typical Primary Outlets

4.6.2.2 Outlet Structure Types

There are a wide variety of outlet structure types, the most common of which are covered in this section. Descriptions and equations are provided for the following outlet types for use in storm water facility design:

- Orifices
- Perforated risers
- Pipes / Culverts
- Sharp-crested weirs
- Broad-crested weirs
- V-notch weirs
- Proportional weirs
- Combination outlets

The design professional must pay attention to material types and construction details when designing an outlet structure or device. Non-corrosive material and mounting hardware are key to device longevity, ease of operation, and low cost maintenance. Special attention must also be paid to not placing dissimilar metal materials together where a cathodic reaction will cause deterioration and destruction of metal parts.

Protective coatings, paints, and sealants must also be chosen carefully to prevent contamination of the storm water flowing through the structure/device. This is not only important while they are being applied, but also as these coating deteriorate and age over the functional life of the facility.

Each of these outlet types has a different design purpose and application:

- Water quality and streambank protection flows are normally handled with smaller, more protected outlet structures such as reverse slope pipes, hooded orifices, orifices located within screened pipes or risers, perforated plates or risers, and V-notch weirs.

- Larger flows, such as flood flows, are typically handled through a riser with different sized openings, through an overflow at the top of a riser (drop inlet structure), or a flow over a broad crested weir or spillway through the embankment. Overflow weirs can also be of different heights and configurations to handle control of multiple design flows.

4.6.2.3 Orifices

An orifice is a circular or rectangular opening of a prescribed shape and size. The flow rate depends on the height of the water above the opening and the size and edge treatment of the orifice.

For a single orifice, as illustrated in Figure 4.6-2(a), the orifice discharge can be determined using the standard orifice equation below.

$$\mathbf{Q = CA\,(2gH)^{0.5}} \qquad \textbf{(4.6.1)}$$

where:

Q = the orifice flow discharge (cfs)
C = discharge coefficient
A = cross-sectional area of orifice or pipe (ft^2)
g = acceleration due to gravity (32.2 ft/s^2)
D = diameter of orifice or pipe (ft)
H = effective head on the orifice, from the center of orifice to the water surface

If the orifice discharges as a free outfall, then the effective head is measured from the center of the orifice to the upstream (headwater) surface elevation. If the orifice discharge is submerged, then the effective head is the difference in elevation of the headwater and tailwater surfaces as shown in Figure 4.6-2(b).

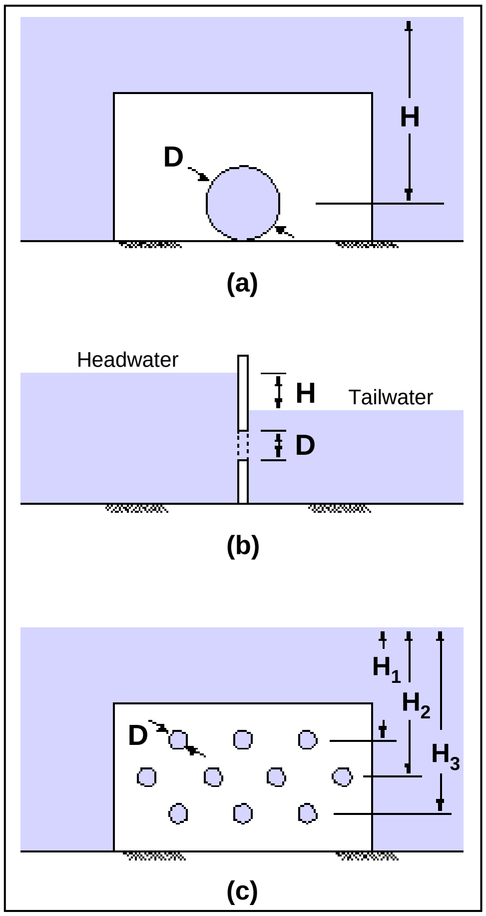


Figure 4.6-2 Orifice Definitions

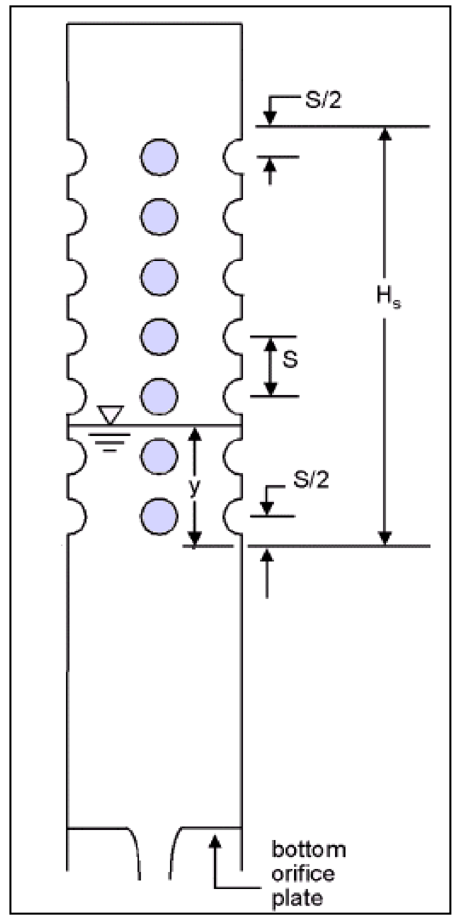


Figure 4.6-3 Perforated Riser

When the material is thinner than the orifice diameter, with sharp edges, a coefficient of 0.6 should be used. For square-edged entrance conditions the generic orifice equation can be simplified:

$$\mathbf{Q = 0.6A\ (2gH)^{0.5} = 3.78D^2H^{0.5}} \qquad \mathbf{(4.6.2)}$$

where:

D = diameter of orifice or pipe (ft)

When the material is thicker than the orifice diameter a coefficient of 0.80 should be used. If the edges are rounded, a coefficient of 0.92 can be used.

Flow through multiple orifices, such as the perforated plate shown in Figure 4.6-2(c), can be computed by summing the flow through individual orifices. For multiple orifices of the same size and under the influence of the same effective head, the total flow can be determined by multiplying the discharge for a single orifice by the number of openings.

Perforated orifice plates for the control of discharge can be of any size and configuration. However, the Denver Urban Drainage and Flood Control District has developed standardized dimensions that have worked well. Table 4.6.2-1 gives appropriate dimensions. The vertical spacing between hole centerlines is always 4 inches.

Table 4.6.2-1 Circular Perforation Sizing

Hole Diameter (in)	MINIMUM COLUMN HOLE CENTERLINE SPACING (IN)	Flow Area per Row (in^2)		
		1 column	2 columns	3 columns
1/4	1	0.05	0.1	0.15
5/16	2	0.08	0.15	0.23
3/8	2	0.11	0.22	0.33
7/16	2	0.15	0.3	0.45
1/2	2	0.2	0.4	0.6
9/16	3	0.25	0.5	0.75
5/8	3	0.31	0.62	0.93
11/16	3	0.37	0.74	1.11
3/4	3	0.44	0.88	1.32
13/16	3	0.52	1.04	1.56
7/8	3	0.6	1.2	1.8
15/16	3	0.69	1.38	2.07
1	4	0.79	1.58	2.37
1 1/16	4	0.89	1.78	2.67
1 1/8	4	0.99	1.98	2.97
1 3/16	4	1.11	2.22	3.33
1 1/4	4	1.23	2.46	3.69
1 5/16	4	1.35	2.7	4.05
1 3/8	4	1.48	2.96	4.44
1 7/16	4	1.62	3.24	4.86
1 1/2	4	1.77	3.54	5.31
1 9/16	4	1.92	3.84	5.76
1 5/8	4	2.07	4.14	6.21
1 11/16	4	2.24	4.48	6.72
1 3/4	4	2.41	4.82	7.23
1 13/16	4	2.58	5.16	7.74
1 7/8	4	2.76	5.52	8.28
1 15/16	4	2.95	5.9	8.85
2	4	3.14	6.28	9.42
Number of columns refers to parallel columns of holes				
Minimum plate thickness		1/4”	5/16”	3/8”

Source: Urban Drainage and Flood Control District, Denver, CO

For rectangular slots the height is normally 2 inches with variable width. Only one column of rectangular slots is allowed.

Figure 4.6-4 provides a schematic of an orifice plate outlet structure for a wet extended detention pond (Not currently required in Copper Canyon) showing the design pool elevations and the flow control mechanisms.

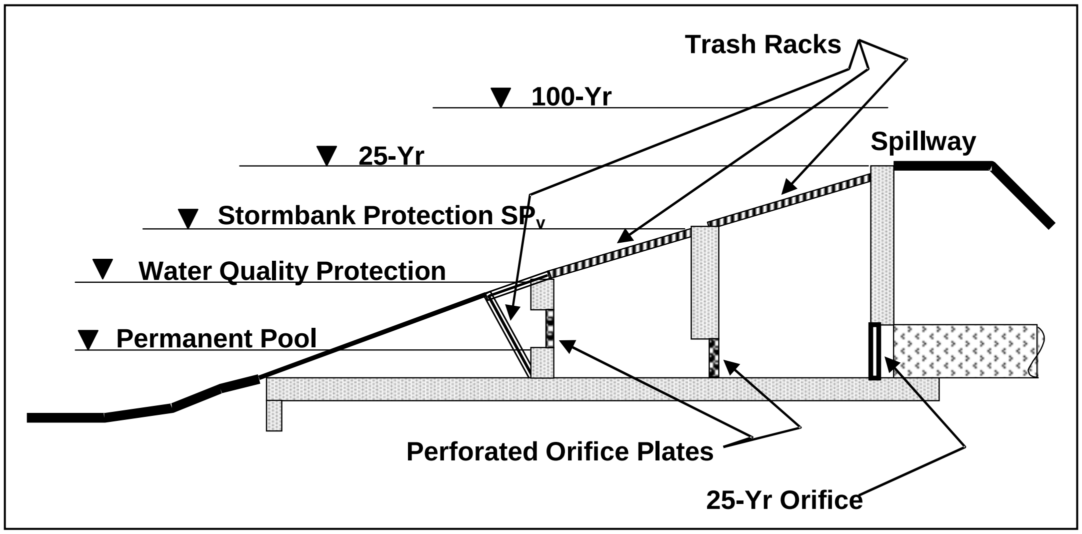


Figure 4.6-4 Schematic of Orifice Plate Outlet Structure

4.6.2.4 Perforated Risers

A special kind of orifice flow is a perforated riser as illustrated in Figure 4.6-3. In the perforated riser, an orifice plate at the bottom of the riser, or in the outlet pipe just downstream from the elbow at the bottom of the riser, controls the flow. It is important that the perforations in the riser convey more flow than the orifice plate so as not to become the control.

Referring to Figure 4.6-3, a shortcut formula has been developed to estimate the total flow capacity of the perforated section (McEnroe, 1988):

$$Q = C_p \frac{2A_p}{3H_s} \sqrt{2g} H^{3/2} \qquad \textbf{(4.6.3)}$$

where:

- Q = discharge (cfs)
- C_p = discharge coefficient for perforations (normally 0.61)
- A_p = cross-sectional area of all the holes (ft^2)
- H_s = distance from S/2 below the lowest row of holes to S/2 above the top row (ft)

4.6.2.4 Pipes and Culverts

Discharge pipes are often used as outlet structures for storm water control facilities. The design of these pipes can be for either single or multi-stage discharges. A reverse-slope underwater pipe is often used for water quality or streambank protection outlets.

Pipes smaller than 12 inches in diameter may be analyzed as a submerged orifice as long as H/D is greater than 1.5. Note: For low flow conditions when the flow reaches and begins to overflow the pipe, weir flow controls (see subsection 4.6.2.6). As the stage increases the flow will transition to orifice flow.

Pipes greater than 12 inches in diameter should be analyzed as a discharge pipe with headwater and tailwater effects taken into account. The outlet hydraulics for pipe flow can be determined from the outlet control culvert nomographs and procedures given in Section 4.3, *Culvert Design*, or by using equation 4.6.4 (NRCS, 1984).

The following equation is a general pipe flow equation derived through the use of the Bernoulli and continuity principles.

$$\mathbf{Q = a[(2gH) / (1 + k_m + k_pL)]0.5} \qquad \mathbf{(4.6.4)}$$

where:

- Q = discharge (cfs)
- a = pipe cross sectional area (ft^2)
- g = acceleration of gravity (ft/s^2)
- H = elevation head differential (ft)
- k_m = coefficient of minor losses (use 1.0)
- k_p = pipe friction coefficient = $5087n^2/D^{4/3}$
- L =pipe length (ft)

4.6.2.5 Sharp-Crested Weirs

If the overflow portion of a weir has a sharp, thin leading edge such that the water springs clear as it overflows, the overflow is termed a *sharp-crested* weir. If the sides of the weir also cause the through flow to contract, it is termed an *end-contracted* sharp-crested weir. Sharp-crested weirs have stable stage-discharge relations and are often used as a measurement device. A sharp-crested weir with compensation for end contractions is illustrated in Figure 4.6-5(a). The discharge equation for this configuration is (Chow, 1959) which can also be used for circular pipe risers:

$$\mathbf{Q = [(3.27 + 0.4(H/H_C)] LH^{1.5}} \qquad \mathbf{(4.6.5)}$$

where:

- Q = discharge (cfs)
- H = head above weir crest excluding velocity head (ft)
- H_C = height of weir crest above channel bottom (ft)
- L = horizontal weir length (ft)

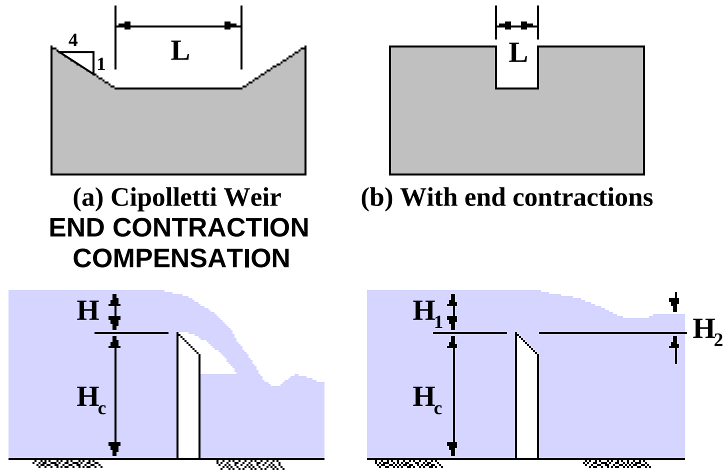


Figure 4.6-5 Sharp-Crested Weir

The discharge equation for the Cipolletti Weir is Q = 3.367 $LH^{1/2}$

A sharp-crested weir with two end contractions is illustrated in Figure 4.6-5(b). The discharge equation for this configuration is (Chow, 1959) which can also be used for circular pipe risers:

$$\mathbf{Q = [(3.27 + 0.4(H/H_C)] (L - 0.2H) H^{1.5}} \qquad \textbf{(4.6.6)}$$

where:

- Q = discharge (cfs)
- H = head above weir crest excluding velocity head (ft)
- H_C = height of weir crest above channel bottom (ft)
- L = horizontal weir length (ft)

A sharp-crested weir will be affected by submergence when the tailwater rises above the weir crest elevation. The result will be that the discharge over the weir will be reduced. The discharge equation for a sharp-crested submerged weir is (Brater and King, 1976):

$$\mathbf{Q_S = Q_f (1 - (H_2/H_1)^{1.5})^{0.385}} \qquad \textbf{(4.6.7)}$$

where:

- Q_S = submergence flow (cfs)
- Q_f = free flow (cfs)
- H_1 = upstream head above crest (ft)
- H_2 = downstream head above crest (ft)

4.6.2.7 Broad-Crested Weirs

A weir in the form of a relatively long raised channel control crest section is a *broad-crested* weir. The flow control section can have different shapes, such as triangular or circular. True broad-crested weir flow occurs when upstream head above the crest is between the limits of about 1/20 and 1/2 the crest length in the direction of flow. For example, a thick wall or a flat stop log can act like a sharp-crested weir when the approach head is large enough that the flow springs from the upstream corner. If upstream head is small enough relative to the top profile length, the stop log can act like a broad-crested weir (USBR, 1997).

The equation for the broad-crested weir is (Brater and King, 1976):

$Q = CLH^{1.5}$ (4.6.8)

where:

- Q = discharge (cfs)
- C = broad-crested weir coefficient
- L = broad-crested weir length perpendicular to flow (ft)
- H = head above weir crest (ft)

If the upstream edge of a broad-crested weir is so rounded as to prevent contraction and if the slope of the crest is as great as the loss of head due to friction, flow will pass through critical depth at the weir crest; this gives the maximum C value of 3.087. For sharp corners on the broad-crested weir, a minimum C value of 2.6 should be used. Information on C values as a function of weir crest breadth and head is given in Table 4.6.2-2.

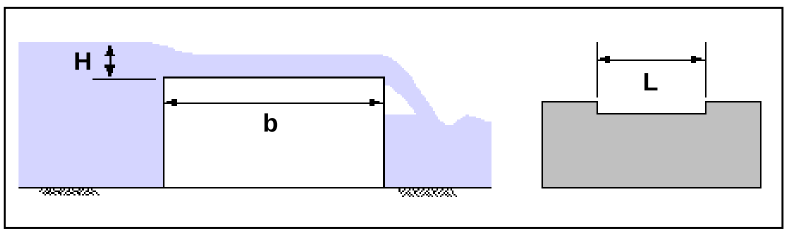


Figure 4.6-6
Broad-Crested Weir

Table 4.6.2-2 Broad-Crested Weir Coefficient (C) Values											
Measured Head (H)*	**WEIR CREST BREADTH (B) IN FEET**										
In feet	0.50	0.75	1.00	1.50	2.00	2.50	3.00	4.00	5.00	10.00	15.00
0.2	2.80	2.75	2.69	2.62	2.54	2.48	2.44	2.38	2.34	2.49	2.68
0.4	2.92	2.80	2.72	2.64	2.61	2.60	2.58	2.54	2.50	2.56	2.70
0.6	3.08	2.89	2.75	2.64	2.61	2.60	2.68	2.69	2.70	2.70	2.70
0.8	3.30	3.04	2.85	2.68	2.60	2.60	2.67	2.68	2.68	2.69	2.64
1.0	3.32	3.14	2.98	2.75	2.66	2.64	2.65	2.67	2.68	2.68	2.63
1.2	3.32	3.20	3.08	2.86	2.70	2.65	2.64	2.67	2.66	2.69	2.64
1.4	3.32	3.26	3.20	2.92	2.77	2.68	2.64	2.65	2.65	2.67	2.64
1.6	3.32	3.29	3.28	3.07	2.89	2.75	2.68	2.66	2.65	2.64	2.63
1.8	3.32	3.32	3.31	3.07	2.88	2.74	2.68	2.66	2.65	2.64	2.63
2.0	3.32	3.31	3.30	3.03	2.85	2.76	2.72	2.68	2.65	2.64	2.63
2.5	3.32	3.32	3.31	3.28	3.07	2.89	2.81	2.72	2.67	2.64	2.63
3.0	3.32	3.32	3.32	3.32	3.20	3.05	2.92	2.73	2.66	2.64	2.63
3.5	3.32	3.32	3.32	3.32	3.32	3.19	2.97	2.76	2.68	2.64	2.63
4.0	3.32	3.32	3.32	3.32	3.32	3.32	3.07	2.79	2.70	2.64	2.63
4.5	3.32	3.32	3.32	3.32	3.32	3.32	3.32	2.88	2.74	2.64	2.63
5.0	3.32	3.32	3.32	3.32	3.32	3.32	3.32	3.07	2.79	2.64	2.63
5.5	3.32	3.32	3.32	3.32	3.32	3.32	3.32	3.32	2.88	2.64	2.63

* Measured at least 2.5H upstream of the weir.
Source: Brater and King (1976)

4.6.2.8 V-Notch Weirs

The discharge through a V-notch weir (Figure 4.6-7) can be calculated from the following equation (Brater and King, 1976).

$Q = 2.5 \tan(\theta/2) H^{2.5}$ **(4.6.9)**

where:

Q = discharge (cfs)

θ = angle of V-notch (degrees)

H = head on apex of notch (ft)

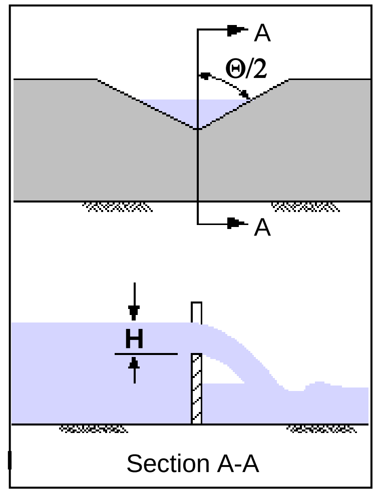


Figure 4.6-7 V-Notch Weir

4.6.2.9 Proportional Weirs

Although it may be more complex to design and construct, a proportional weir may significantly reduce the required storage volume for a given site. The proportional weir is distinguished from other control devices by having a linear head-discharge relationship achieved by allowing the discharge area to vary nonlinearly with head. A typical proportional weir is shown in Figure 4.6-8. Design equations for proportional weirs are (Sandvik, 1985):

$Q = 4.97 a^{0.5} b (H - a/3)$ **(4.6.10)**

$x/b = 1 - (1/3.17) (\arctan (y/a)^{0.5})$ **(4.6.11)**

where:

Q = discharge (cfs)

Dimensions a, b, H, x, and y are shown in Figure 4.6.2-8

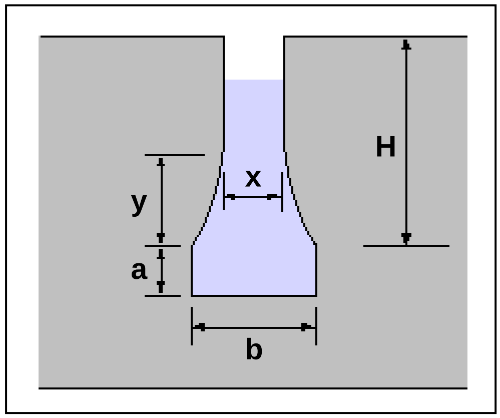


Figure 4.6-8 Proportional Weir Dimensions

4.6.2.10 Combination Outlets

Combinations of orifices, weirs, and pipes can be used to provide multi-stage outlet control for different control volumes within a storage facility (i.e., water quality protection volume, streambank protection volume, and flood control volume).

They are generally two types of combination outlets: shared outlet control structures and separate outlet controls. Shared outlet control is typically a number of individual outlet openings (orifices), weirs, or drops at different elevations on a riser pipe or box which all flow to a common larger conduit or pipe. Figure 4.6-9 shows an example of a riser designed for a wet extended detention pond. The orifice plate outlet structure in Figure 4.6-4 is another example of a combination outlet.

Separate outlet controls are less common and may consist of several pipe or culvert outlets at different levels in the storage facility that are either discharged separately or are combined to discharge at a single location.

The use of a combination outlet requires the construction of a composite stage-discharge curve (as shown in Figure 4.6-10) suitable for control of multiple storm flows. The design of multi-stage combination outlets is discussed later in this section.

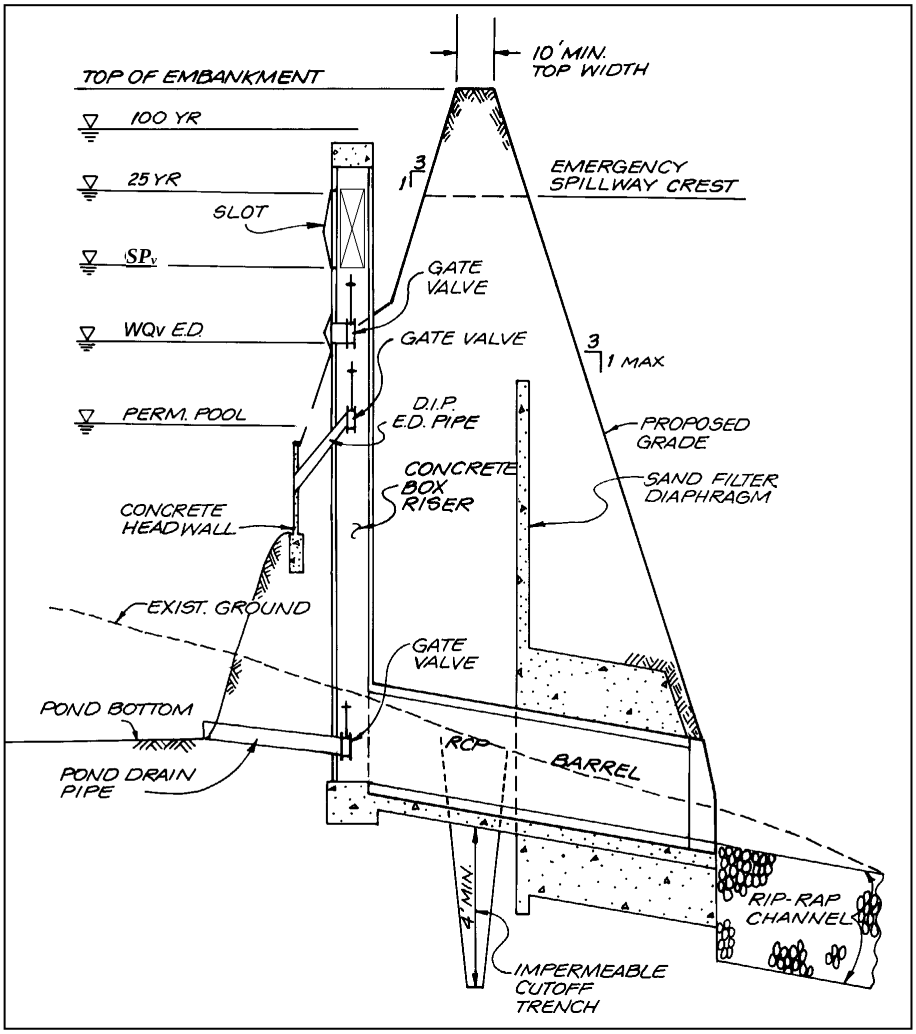


Figure 4.6-9 Schematic of Combination Outlet Structure

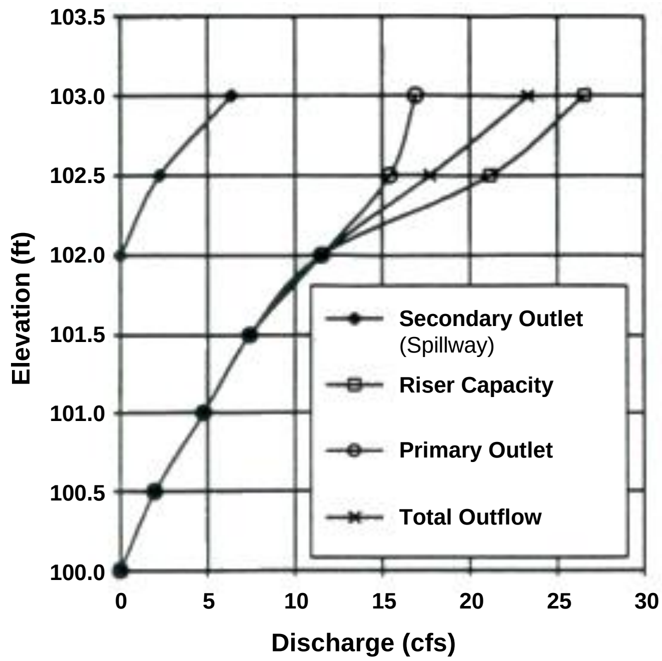


Figure 4.6-10 Composite Stage-Discharge Curve

4.6.3 – Extended Detention (Water Quality and Streambank Protection) Outlet Design

(This type of structure is not currently required by the Town of Copper Canyon but is included in this Manual for situations where the developer may choose to implement this type of facility)

4.6.3.1 Introduction

Extended detention (ED) orifice sizing is required in design applications that provide extended detention for downstream streambank protection or water quality control.

(The following procedures are based on the water quality outlet design procedures included in the Virginia Stormwater Management Handbook, 1999)

The outlet hydraulics for peak control design (flood control) is usually straightforward in that an outlet is selected to limit the peak flow to some predetermined maximum. Since volume and the time required for water to exit the storage facility are not usually considered, the outlet design can easily be calculated and routing procedures used to determine if quantity design criteria are met.

In an extended detention facility for water quality protection or downstream streambank protection, however, the storage volume is detained and released for each over a specified amount of time (e.g., 24-hours). The release period is a "brim" drawdown time, beginning at the time of peak storage of the WQ_v or SP_v until the entire calculated volume drains out of the basin. This assumes the brim volume is present in the basin prior to any discharge. In reality, however, water is flowing out of the basin prior to the full or brim volume being reached. Therefore, the extended detention outlet can be sized using either of the following methods:

- Using the maximum hydraulic head associated with the brim storage volume and maximum discharge, calculate the orifice size needed to achieve the required drawdown time. Route the volume through the basin to verify the actual storage volume used and the drawdown time.
- Approximate the orifice size using the average hydraulic head associated with the storage volume and the required drawdown time.

These two procedures are outlined in the examples below and can be used to size an extended detention orifice for water quality and/or streambank protection.

4.6.3.2 Method 1: Maximum Hydraulic Head with Routing

A wet ED pond sized for the required water quality protection volume will be used here to illustrate the sizing procedure for an extended-detention orifice.

Given the following information, calculate the required orifice size for water quality protection design.

Given: Water Quality Protection Volume (WQ_v) = 0.76 ac ft = 33,106 ft^3

Maximum Hydraulic Head (H_{max}) = 5.0 ft (from stage vs. storage data)

Step 1 Determine the maximum discharge resulting from the 24-hour drawdown requirement. It is calculated by dividing the Water Quality Protection Volume (or Streambank Protection Volume) by the required time to find the average discharge, and then multiplying by two to obtain the maximum discharge.

Q_{avg} = 33,106 ft^3 / (24 hr)(3,600 s/hr) = 0.38 cfs

Q_{max} = 2 * Q_{avg} = 2 * 0.38 = 0.76 cfs

Step 2 Determine the required orifice diameter by using the orifice equation (4.6.8) and Q_{max} and H_{max}:

$Q = CA(2gH)^{0.5}$, or $A = Q / C(2gH)^{0.5}$
$A = 0.76 / 0.6[(2)(32.2)(5.0)]^{0.5} = 0.071\ ft^2$

Determine pipe diameter from $A = 3.14d^2/4$, then $d = (4A/3.14)^{0.5}$
$D = [4(0.071)/3.14]^{0.5}$ = 0.30 ft = 3.61 in

Use a 3.6-inch diameter water quality protection orifice.

Routing the water quality protection volume of 0.76 ac ft through the 3.6-inch water quality protection orifice will allow the designer to verify the drawdown time, as well as the maximum hydraulic head elevation. The routing effect will result in the actual drawdown time being less than the calculated 24 hours. Judgment should be used to determine whether the orifice size should be reduced to achieve the required 24 hours.

4.6.3.2 Method 2: Average Hydraulic Head and Average Discharge

Using the data from the previous example (4.6.3.2) use Method 2 to calculate the size of the outlet orifice.

Given: Water Quality Protection Volume (WQ_v) = 0.76 ac ft = 33,106 ft^3

Average Hydraulic Head (h_{avg}) = 2.5 ft (from stage vs storage data)

Step 1 Determine the average release rate to release the water quality protection volume over a 24-hour time period.

Q = 33,106 ft^3 / (24 hr)(3,600 s/hr) = 0.38 cfs

Step 2 Determine the required orifice diameter by using the orifice equation (4.6.8) and the average head on the orifice:

$Q = CA(2gH)^{0.5}$, or $A = Q / C(2gH)^{0.5}$
$A = 0.38 / 0.6[(2)(32.2)(2.5)]^{0.5} = 0.05\ ft^3$

Determine pipe diameter from $A = 3.14r^2 = 3.14d^2/4$, then $d = (4A/3.14)^{0.5}$

$D = [4(0.05)/3.14]^{0.5} = 0.252$ ft = 3.03 in

Use a 3-inch diameter water quality protection orifice.

Use of Method 1, utilizing the maximum hydraulic head and discharge and routing, results in a 3.6-inch diameter orifice (though actual routing may result in a changed orifice size) and Method 2, utilizing average hydraulic head and average discharge, results in a 3.0-inch diameter orifice

4.6.4 – Multi-Stage Outlet Design

4.6.4.1 Introduction

A combination outlet such as a multiple orifice plate system or multi-stage riser is often used to provide adequate hydraulic outlet controls for the different design requirements for storm water ponds, storm water wetlands and detention-only facilities. Separate openings or devices at different elevations are used to control the rate of discharge from a facility during multiple design storms. Figures 4.6-4 and 4.6-9 are examples of multi-stage combination outlet systems.

A design engineer may be creative to provide the most economical and hydraulically efficient outlet design possible in designing a multi-stage outlet. Many iterative routings are usually required to arrive at a minimum structure size and storage volume that provides proper control. The stage-discharge table or rating curve is a composite of the different outlets that are used for different elevations within the multi-stage riser (see Figure 4.6-10)

4.6.4.2 Multi-Stage Outlet Design Procedure

(Please note that detention of Water Quality Volume and Streambank Protection Volume <u>is not currently required</u> but control of 2-year, 10-year and 100-year peak discharges <u>is required</u>.)

Below are the steps for designing a multi-stage outlet. Note that if a structural control facility will not control one or more of the required storage volumes (WQ_v, SP_v, and Q_f), then that step in the procedure is skipped.

Step 1 <u>Determine Storm Water Control Volumes</u>. Using the procedures from Sections 2.1 and 2.2, estimate the required storage volumes for water quality protection (WQ_v), streambank protection (SP_v), and flood control (Q_f).

Step 2 <u>Develop Stage-Storage Curve</u>. Using the structure geometry and topography, develop the stage-storage curve for the facility in order to provide sufficient storage for the control volumes involved in the design.

Step 3 <u>Design Water Quality Protection Outlet</u>. Design the water quality protection extended detention (WQ_v-ED) orifice using either Method 1 or Method 2 outlined in subsection 4.6.3. If a permanent pool is incorporated into the design of the facility, a portion of the storage volume for water quality protection will be above the elevation of the permanent pool. The outlet can be protected using either a reverse slope pipe, a hooded protection device, or another acceptable method (see subsection 4.6.5).

Step 4 Design Streambank Protection Outlet. Design the streambank protection extended detention outlet (SP_v-ED) using either method from subsection 4.6.3. For this design, the storage needed for streambank protection will be greater than the water quality protection volume storage elevation determined in Step 3. The total stage-discharge rating curve at this point will include the water quality protection orifice and the outlet used for streambank protection. The outlet should be protected in a manner similar to that for the water quality protection orifice.

Step 5 Design Flood Control Outlet. The storage needed for flood control will be greater than the water quality protection and streambank protection storage. Establish the Q_f maximum water surface elevation using the stage-storage curve and subtract the SP_v elevation to find the maximum head. Select an outlet type and calculate the initial size and geometry based upon maintaining the predevelopment peak discharge rate. Develop a stage-discharge curve for the combined set of outlets (WQ_v, SP_v and Q_f).

Step 6 Check Performance of the Outlet Structure. Perform a hydraulic analysis of the multi-stage outlet structure using reservoir routing to ensure that all outlets will function as designed. Several iterations may be required to calibrate and optimize the hydraulics and outlets that are used. Also, the structure should operate without excessive surging, noise, vibration, or vortex action at any stage. This usually requires that the outlet structure have a larger cross-sectional area than the outlet conduit.

The hydraulic analysis of the design must take into account the hydraulic changes that will occur as depth of storage changes for the different design storms. As shown in Figure 4.6-11, as the water passes over the rim of a riser, the riser acts as a weir. However, when the water surface reaches a certain height over the rim of a riser, the riser will begin to act as a submerged orifice. The designer must compute the elevation at which this transition from riser weir flow control to riser orifice flow control takes place for an outlet where this transition will occur. Also note in Figure 4.6-11that as the elevation of the water increases further, the control can change from barrel inlet flow control to barrel pipe flow control. Figure 4.6.-12 shows another condition where weir flow can change to orifice flow, which must be taken into account in the hydraulics of the rating curve as different design conditions results in changing water surface elevations.

Step 7 Size the Emergency Spillway. It is recommended that all storm water impoundment structures have a vegetated emergency spillway (see subsection 4.6.7). An emergency spillway provides a degree of safety to prevent overtopping of an embankment if the primary outlet or principal spillway should become clogged, or otherwise inoperative. The 100-year storm should be routed through the outlet devices and emergency spillway to ensure the hydraulics of the system will operate as designed. Also check the dam safety requirements to be sure of an adequate design.

Step 8 Design Outlet Protection. Design necessary outlet protection and energy dissipation facilities to avoid erosion problems downstream from outlet devices and emergency spillway(s). See Subsection 4.7, *Energy Dissipation Design*, for more information.

Step 9 Perform Buoyancy Calculations. Perform buoyancy calculations for the outlet structure and footing. Flotation will occur when the weight of the structure is less than or equal to the buoyant force exerted by the water.

Step 10 Provide Seepage Control. Seepage control should be provided for the outflow pipe or culvert through an embankment. The two most common devices for controlling seepage are (1) filter and drainage diaphragms and (2) anti-seep collars.

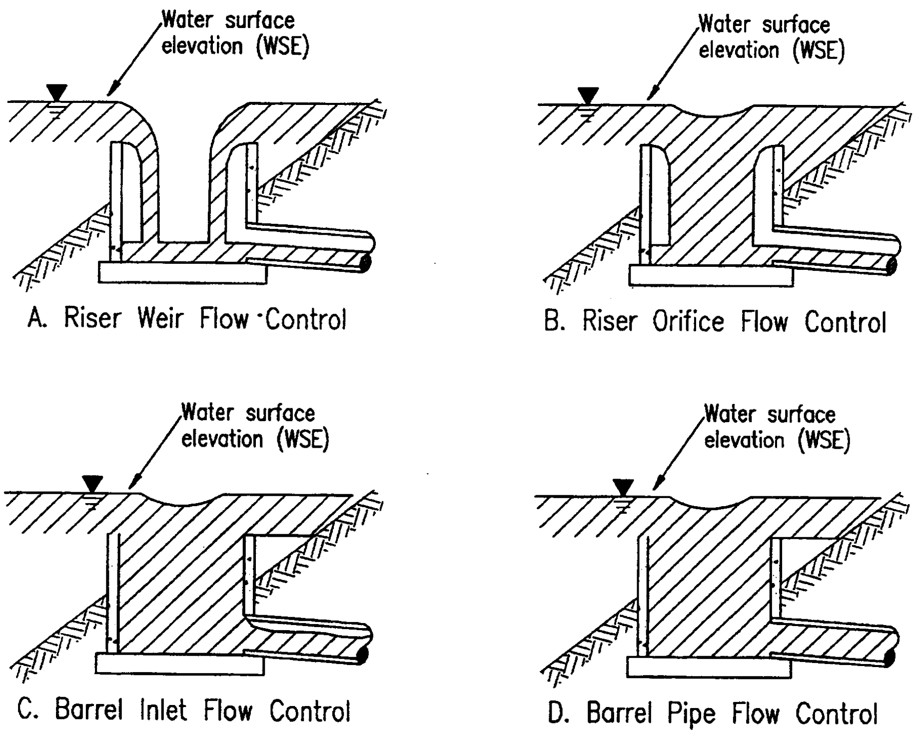


Figure 4.6-11 Riser Flow Diagrams
(Source: VDCR, 1999)

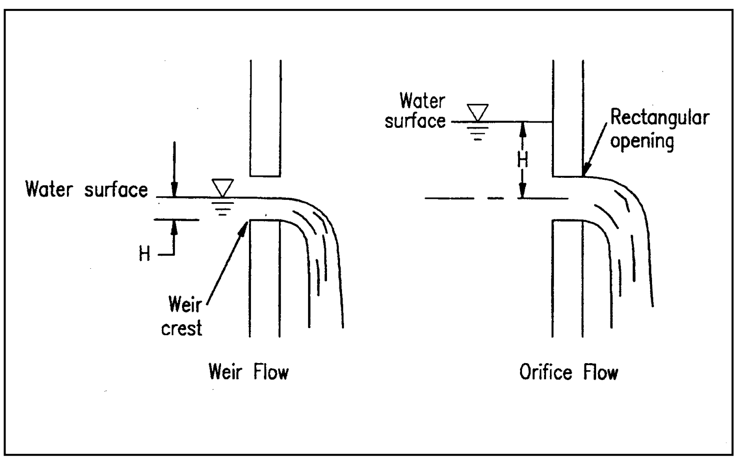


Figure 4.6-12 Weir and Orifice Flow
(Source: VDCR, 1999)

4.6.4.3 Extended Detention Outlet Protection

(This type of structure is not currently required by the Town of Copper Canyon by is included in this Manual for situations where the developer may chose to implement this type of facility)

Small low flow orifices such as those used for extended detention applications can easily clog, preventing the structural control from meeting its design purpose(s) and potentially causing adverse impacts. Therefore, extended detention orifices need to be adequately protected from clogging. There are a number of different anti-clogging designs, including:

The use of a reverse slope pipe attached to a riser for a storm water pond or wetland with a permanent pool (see Figure 4.6-13). The inlet is submerged a minimum of 1 foot below the elevation of the permanent pool to prevent floatables from clogging the pipe and to avoid discharging warmer water at the surface of the pond.

The use of a hooded outlet for a storm water pond or wetland with a permanent pool (see Figures 4.6-14 and 4.6-15).

Internal orifice protection through the use of an over-perforated vertical stand pipe with ½-inch orifices or slots that are protected by wirecloth and a stone filtering jacket (see Figure 4.6-16).

Internal orifice size requirements may be attained by the use of adjustable gate valves to achieve an equivalent orifice diameter.

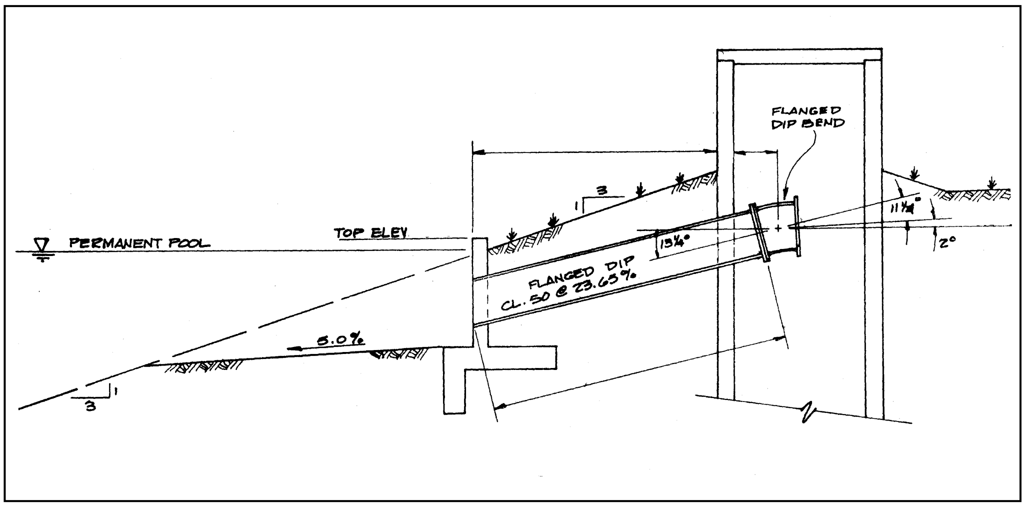


Figure 4.6-13 Reverse Slope Pipe Outlet

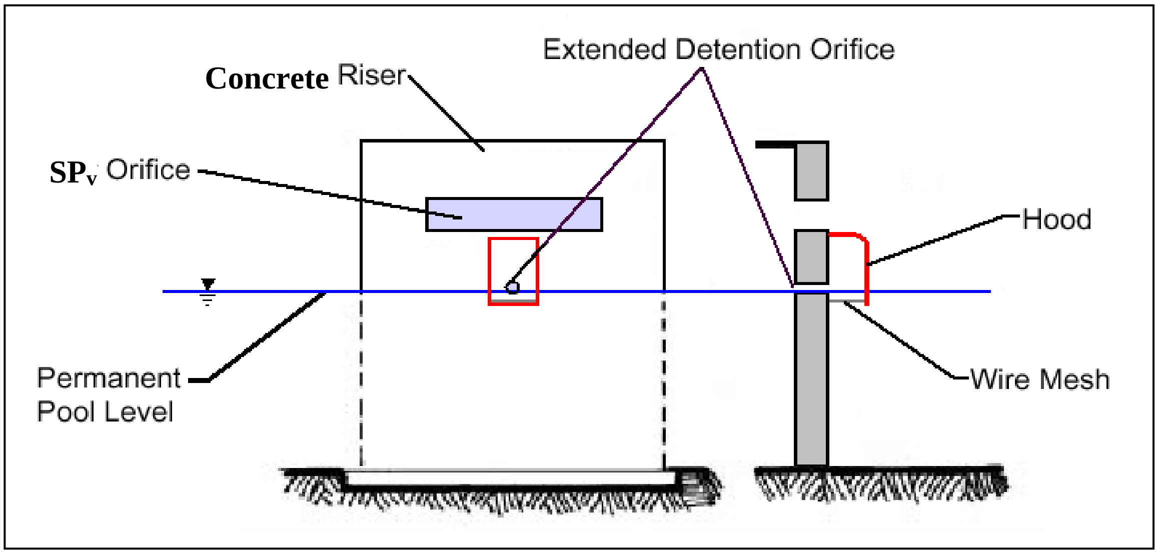


Figure 4.6-14 Hooded Outlet

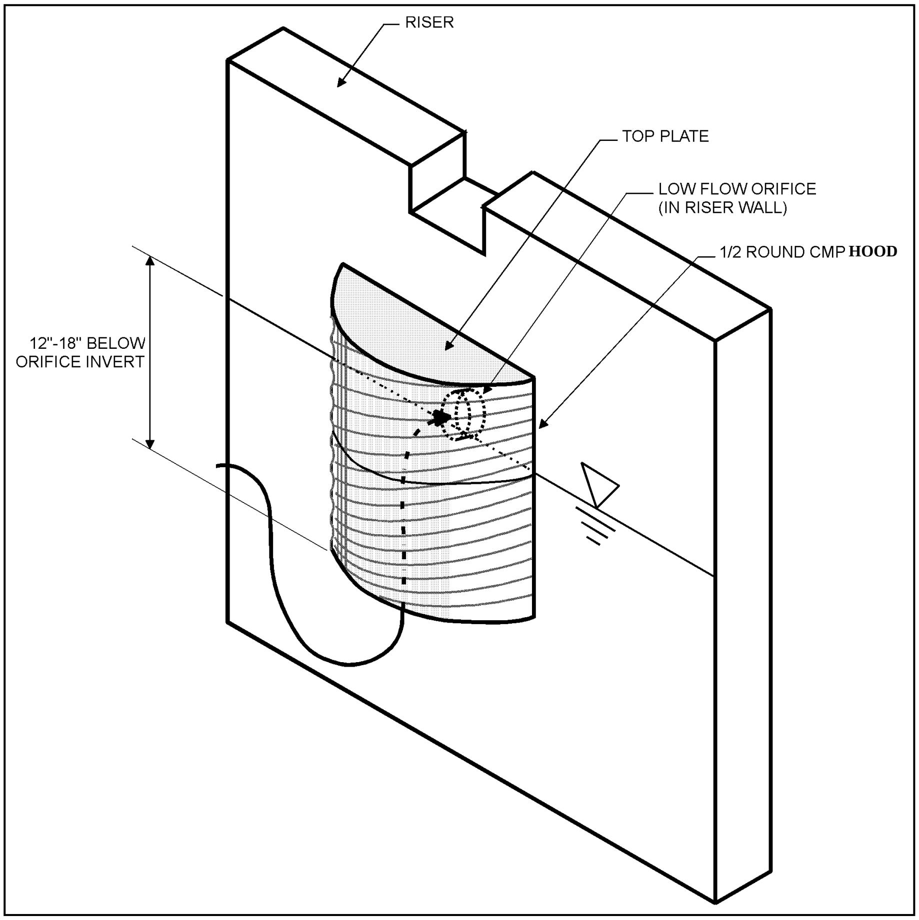


Figure 4.6-15   Half-Round CMP Orifice Hood

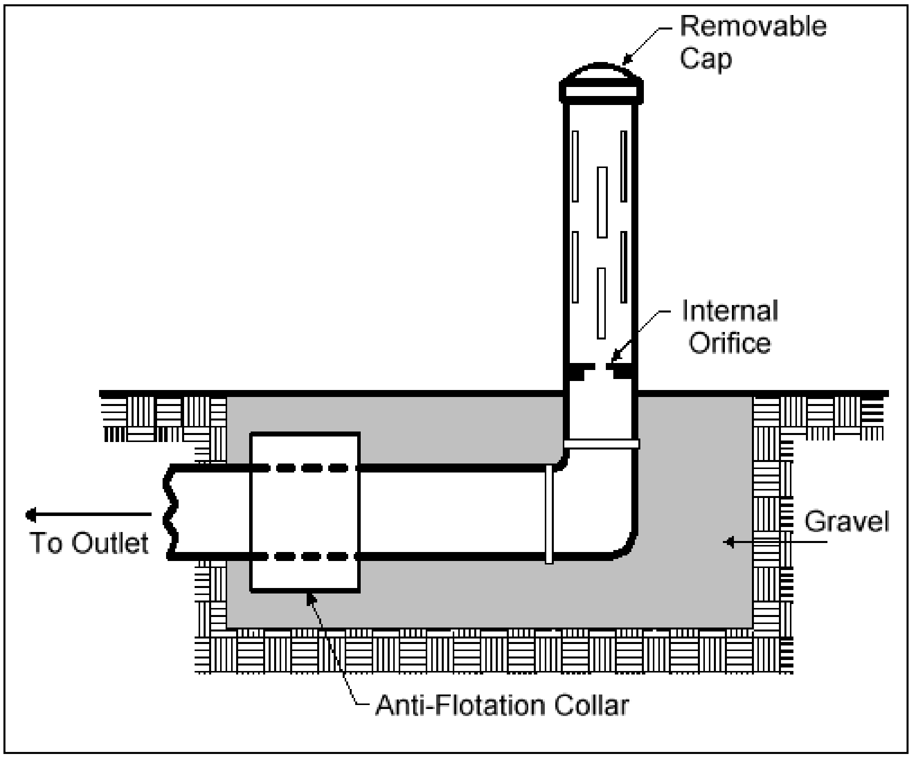


Figure 4.6-16   Internal Control for Orifice Protection

4.6.5 – Trash Racks and Safety Grates

4.6.6.1 Introduction

The susceptibility of larger inlets to clogging by debris and trash needs to be considered when estimating their hydraulic capacities. In most instances trash racks will be needed. Trash racks and safety grates are a critical element of outlet structure design and serve several important functions:

- **Keeping debris away from the entrance to the outlet works where they will not clog the critical portions of the structure**
- Capturing debris in such a way that relatively easy removal is possible
- Ensuring that people and large animals are kept out of confined conveyance and outlet areas
- Providing a safety system that prevents anyone from being drawn into the outlet and allows them to climb to safety

When designed properly, trash racks serve these purposes without interfering significantly with the hydraulic capacity of the outlet (or inlet in the case of conveyance structures) (ASCE, 1985; Allred-Coonrod, 1991). The location and size of the trash rack depends on a number of factors, including head losses through the rack, structural convenience, safety and size of outlet. Well-designed trash racks can also have an aesthetically pleasing appearance.

An example of trash racks used on a riser outlet structure is shown in Figure 4.6-17. Additional track rack design can be found in Appendix G. The inclined vertical bar rack is most effective for lower stage outlets. Debris will ride up the trash rack as water levels rise. This design also allows for removal of accumulated debris with a rake while standing on top of the structure.

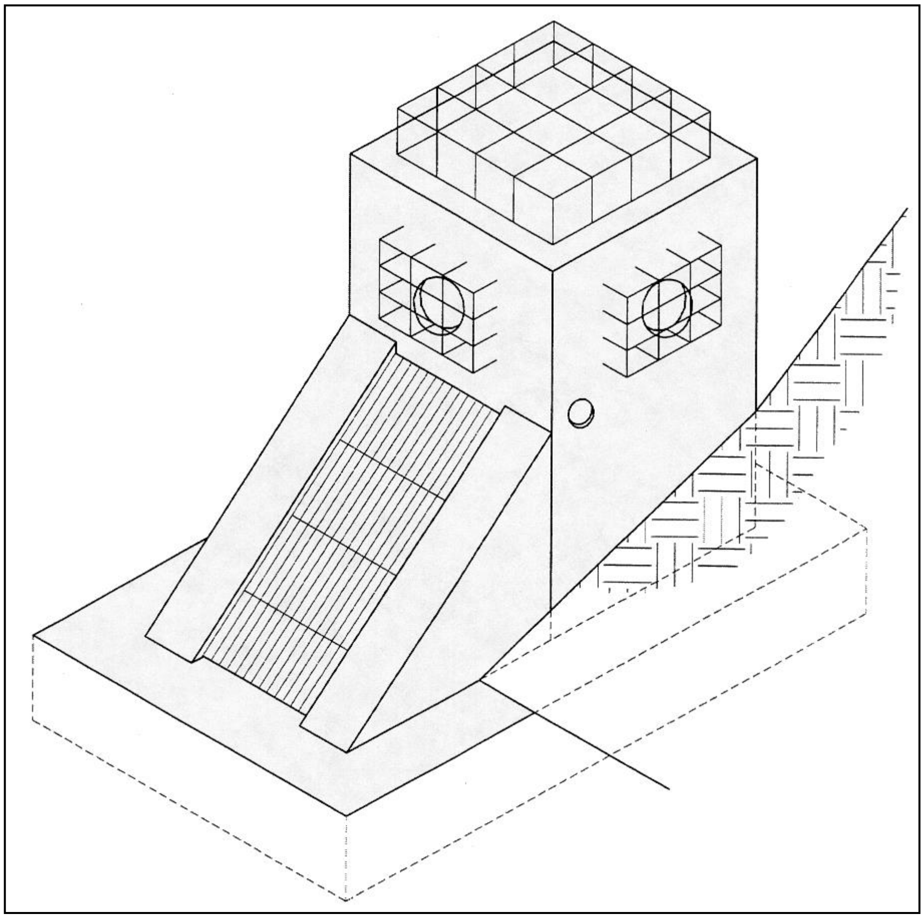


Figure 4.6-17 Example of Various Trash Racks Used on a Riser Outlet Structure

(Source: VDCR, 1999)

4.6.6.2 Trash Rack Design

Trash racks must be large enough so that partial plugging will not adversely restrict flows reaching the control outlet. There are no universal guidelines for the design of trash racks to protect detention basin outlets, although a commonly used "rule-of-thumb" is to have the trash rack area at least ten times larger than the control outlet orifice.

The surface area of all trash racks should be maximized and the trash racks should be located a suitable distance from the protected outlet to avoid interference with the hydraulic capacity of the outlet. The spacing of trash rack bars must be proportioned to the size of the smallest outlet protected. However, where a small orifice is provided, a separate trash rack for that outlet should be used, so that a simpler, sturdier trash rack with more widely spaced members can be used for the other outlets. Spacing of the rack bars should be wide enough to avoid interference, but close enough to provide the level of clogging protection required.

To facilitate removal of accumulated debris and sediment from around the outlet structure, the racks should have hinged connections. If the rack is bolted or set in concrete it will preclude removal of accumulated material and will eventually adversely affect the outlet hydraulics.

Since sediment will tend to accumulate around the lowest stage outlet, the inside of the outlet structure for a dry basin should be depressed below the ground level to minimize clogging due to sedimentation. Depressing the outlet bottom to a depth below the ground surface at least equal to the diameter of the outlet is recommended.

Trash racks at entrances to pipes and conduits should be sloped at about 3H:1V to 5H:1V to allow trash to slide up the rack with flow pressure and rising water level — the slower the approach flow, the flatter the angle. Rack opening rules-of-thumb are found in literature. Figure 4.6.6-2 gives opening estimates based on outlet diameter (UDFCD, 1992). Judgment should be used in that an area with higher debris (e.g., a wooded area) may require more opening space.

The bar opening space for small pipes should be less than the pipe diameter. For larger diameter pipes, openings should be 6 inches or less. Collapsible racks have been used in some places if clogging becomes excessive or a person becomes pinned to the rack.

Alternately, debris for culvert openings can be caught upstream from the opening by using pipes placed in the ground or a chain safety net (USBR, 1978; UDFCD, 1999). Racks can be hinged on top to allow for easy opening and cleaning.

The control for the outlet should not shift to the grate, nor should the grate cause the headwater to rise above planned levels. Therefore head losses through the grate should be calculated. A number of empirical loss equations exist though many have difficult to estimate variables. Two will be given to allow for comparison.

Metcalf & Eddy (1972) give the following equation (based on German experiments) for losses. Grate openings should be calculated assuming a certain percentage blockage as a worst case to determine losses and upstream head. Often 40 to 50% is chosen as a working assumption.

$$\mathbf{H_g = K_{g1}\,(w/x)^{4/3}\,(V_u^2/2g)\sin\theta_g} \qquad \textbf{(4.6.12)}$$

Where:

H_g = head loss through grate (ft)

K_{g1} = bar shape factor:

2.42 - sharp edged rectangular

1.83 - rectangular bars with semicircular upstream faces

1.79 - circular bars

1.67 - rectangular bars with semicircular up- and downstream faces

w = maximum cross-sectional bar width facing the flow (in)

x = minimum clear spacing between bars (in)
V_u = approach velocity (ft/s)
g = acceleration due to gravity (32.2 ft/s^2)
θ_g = angle of the grate with respect to the horizontal (degrees)

The Corps of Engineers (HDC, 1988) has developed curves for trash racks based on similar and additional tests. These curves are for vertical racks but presumably they can be adjusted, in a manner similar to the previous equation, through multiplication by the sine of the angle of the grate with respect to the horizontal.

$$\mathbf{H_g = \frac{K_{g2}V_u^2}{2g}} \qquad \textbf{(4.6.13)}$$

Where:

K_{g2} is defined from a series of fit curves as:

- sharp edged rectangular (length/thickness = 10)
  $K_{g2} = 0.00158 - 0.03217\ A_r + 7.1786\ A_r^2$
- sharp edged rectangular (length/thickness = 5)
  $K_{g2} = -0.00731 + 0.69453\ A_r + 7.0856\ A_r^2$
- round edged rectangular (length/thickness = 10.9)
  $K_{g2} = -0.00101 + 0.02520\ A_r + 6.0000\ A_r^2$
- circular cross section
  $K_{g2} = 0.00866 + 0.13589\ A_r + 6.0357\ A_r^2$

and A_r is the ratio of the area of the bars to the area of the grate section.

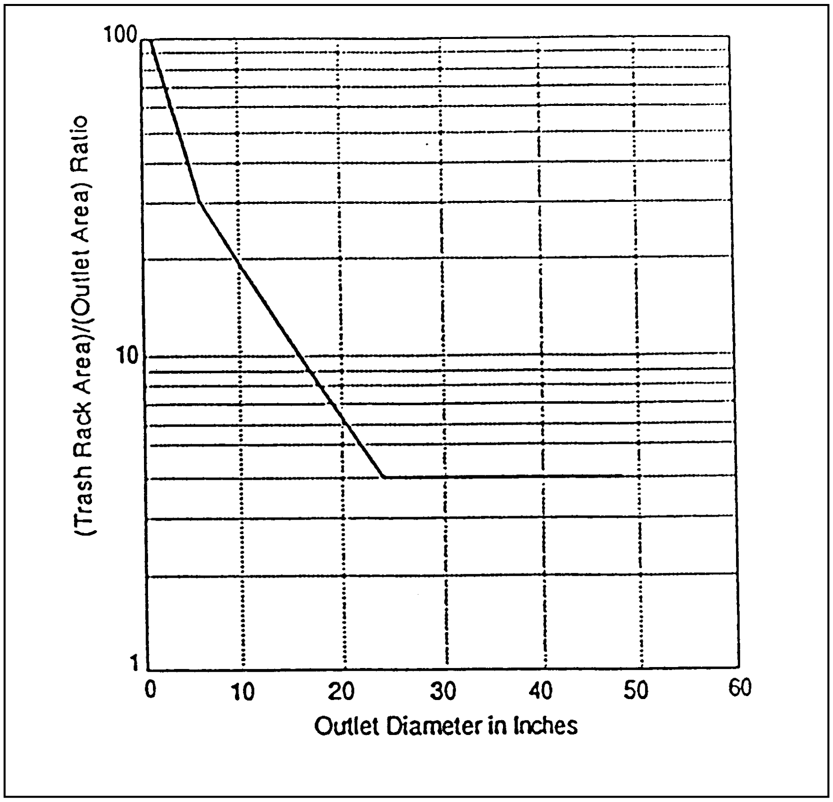


Figure 4.6-18 Minimum Rack Size vs. Outlet Diameter
(Source: UDCFD, 1992)

4.6.7 – Secondary Outlets

4.6.7.1 Introduction

The purpose of a secondary outlet (emergency spillway) is to provide a controlled overflow for flows in excess of the maximum design storm for a storage facility. Figure 4.6-19 shows an example of an emergency spillway.

In many cases, on-site storm water storage facilities do not warrant elaborate studies to determine spillway capacity. While the risk of damage due to failure is a real one, it normally does not approach the catastrophic risk involved in the overtopping or breaching of a major reservoir. By contrast, regional facilities with homes immediately downstream could pose a significant hazard if failure were to occur, in which case emergency spillway considerations are a major design factor.

4.6.7.2 Emergency Spillway Design

Emergency spillway designs are open channels, usually trapezoidal in cross section, and consist of an inlet channel, a control section, and an exit channel (see Figure 4.6-19). The emergency spillway is proportioned to pass flows in excess of the design flood (typically the 100-year flood or greater) without allowing excessive velocities and without overtopping of the embankment. Flow in the emergency spillway is open channel flow (see Section 4.4, *Open Channel Design*, for more information). Normally, it is assumed that critical depth occurs at the control section.

NRCS (SCS) manuals provide guidance for the selection of emergency spillway characteristics for different soil conditions and different types of vegetation. The selection of degree of retardance for a given spillway depends on the vegetation. Knowing the retardance factor and the estimated discharge rate, the emergency spillway bottom width can be determined. For erosion protection during the first year, assume minimum retardance. Both the inlet and exit channels should have a straight alignment and grade. Spillway side slopes should be no steeper the 3:1 horizontal to vertical.

The most common type of emergency spillway used is a broad-crested overflow weir cut through original ground next to the embankment. The transverse cross section of the weir cut is typically trapezoidal in shape for ease of construction. Such an excavated emergency spillway is illustrated below.

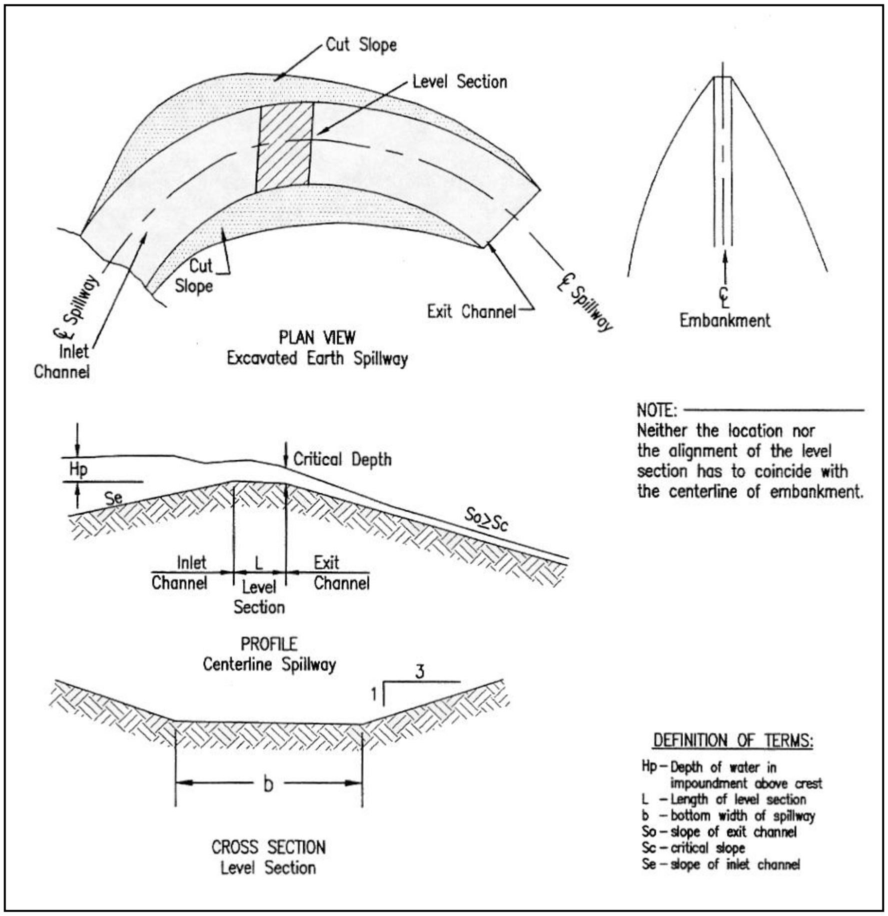


Figure 4.6-19 Emergency Spillway

Section 4.7 – Energy Dissipation

Section 4.7.1 – Overview

4.7.1.1 Introduction

The outlets of pipes and lined channels are points of critical erosion potential. Storm water transported through man-made conveyance systems at design capacity generally reaches a velocity that exceeds the capacity of the receiving channel or area to resist erosion. To prevent scour at storm water outlets, protect the outlet structure and minimize the potential for downstream erosion, a flow transition structure is needed to absorb the initial impact of flow and reduce the speed of the flow to a non-erosive velocity.

Energy dissipators are engineered devices such as rip-rap aprons or concrete baffles placed at the outlet of storm water conveyances for the purpose of reducing the velocity, energy and turbulence of the discharged flow.

4.7.1.2 General Criteria

Erosion problems at culvert, pipe and engineered channel outlets are common. Determination of the flow conditions, scour potential, and channel erosion resistance shall be standard procedure for all designs.

Energy dissipators shall be employed whenever the velocity of flows leaving a storm water management facility exceeds the erosion velocity of the downstream area channel system.

Energy dissipator designs will vary based on discharge specifics and tailwater conditions.

Outlet structures should provide uniform redistribution or spreading of the flow without excessive separation and turbulence.

4.7.1.3 Recommended Energy Dissipators

For many designs, the following outlet protection devices and energy dissipators provide sufficient protection at a reasonable cost:

- Riprap apron
- Riprap outlet basins
- Baffled outlets
- Grade Control Structures

This section focuses on the design on these measures. The reader is referred to the Federal Highway Administration Hydraulic Engineering Circular No. 14 entitled, Hydraulic Design of

Energy Dissipators for Culverts and Channels, for the design procedures of other energy dissipators.

Channel Transitions, Energy Dissipation Structures, or Small Dams

A backwater analysis is required by the Town of Copper Canyon, using HEC-RAS, to determine accurate tailwater elevation and velocities, headlosses, headwater elevations, velocities and floodplains affected by the proposed transition into and out of 1) An improved channel, 2) Any on-stream energy dissipating structures, and 3) Small dams (less than 6 feet). If the current effective FEMA model for the stream is a HEC-2 model, the engineer has the option to either use that model, or convert to HEC-RAS for analysis of proposed conditions. For larger dams, a hydrologic routing will be required, as well as hydraulic analysis, to determine impacts of the proposed structure on existing floodplains and adjacent properties.

Exceptions may be granted for small outfall channels (with the approval of TOWN ENGINEER) with drainage areas of 10 acres or less and no nearby downstream restrictions.

Examples of Open Channel Transition Structures

Details and Specifications and application guidance for Harris County Flood Control District Straight Drop Structure and Bureau of Reclamation Baffled Chute (Basin IX) can be found in Harris County Flood Control District Policy Criteria& Procedure Manual (See references section for description). A computer program associated with FHWA Hydraulic Engineering Circular No. 14 is “HY8Energy” dated May 2000. This program provides guidance in the selection and sizing of a broad range of energy dissipaters including some of those listed in Section 4.7.

4.7.2 – Symbols and Definitions

To provide consistency within this section as well as throughout this Manual, the symbols listed in Table 4.7-1 will be used. These symbols were selected because of their wide use. In some cases, the same symbol is used in existing publications for more than one definition. Where this occurs in this section, the symbol will be defined where it occurs in the text or equations.

Table 4.7-1 Symbols and Definitions

Symbol	Definition	Units
A	Cross-sectional area	ft^2
D	Height of box culvert	ft
d_{50}	Size of riprap	ft
d_w	Culvert width	ft
Fr	Froude Number	-
g	Acceleration of gravity	ft/s^2
h_s	Depth of dissipator pool	ft
L	Length	ft
L_a	Riprap apron length	ft
L_B	Overall length of basin	ft
L_s	Length of dissipator pool	ft
PI	Plasticity index	-
Q	Rate of discharge	cfs
S_v	Saturated shear strength	lbs/in^2
t	Time of scour	min.
t_c	Critical tractive shear stress	lbs/in^2
TW	Tailwater depth	ft
V_L	Velocity L feet from brink	ft/s
V_o	Normal velocity at brink	ft/s
V_o	Outlet mean velocity	ft/s
V_s	Volume of dissipator pool	ft^2
W_o	Diameter or width of culvert	ft
W_s	Width of dissipator pool	ft
y_e	Hydraulic depth at brink	ft
y_o	Normal flow depth at brink	ft

4.7.3 – Design Guidelines

If outlet protection is required, choose an appropriate type. Suggested outlet protection facilities and applicable flow conditions (based on Froude number and dissipation velocity) are described below:

a. Riprap aprons may be used when the outlet Froude number (Fr) is less than or equal to 2.5. In general, riprap aprons prove economical for transitions from culverts to overland sheet flow at terminal outlets, but may also be used for transitions from culvert sections to stable channel sections. Stability of the surface at the termination of the apron should be considered.

b. Riprap outlet basins may also be used when the outlet Fr is less than or equal to 2.5. They are generally used for transitions from culverts to stable channels. Since riprap outlet basins function by creating a hydraulic jump to dissipate energy, performance is impacted by tailwater conditions.

c. Baffled outlets have been used with outlet velocities up to 50 feet per second. Practical application typically requires an outlet Fr between 1 and 9. Baffled outlets may be used at both terminal outlet and channel outlet transitions. They function by dissipating energy through impact and turbulence and are not significantly affected by tailwater conditions.

When outlet protection facilities are selected, appropriate design flow conditions and site-specific factors affecting erosion and scour potential, construction cost, and long-term durability should be considered.

If outlet protection is not provided, energy dissipation will occur through formation of a local scourhole. A cutoff wall will be needed at the discharge outlet to prevent structural undermining. The wall depth should be slightly greater than the computed scourhole depth, h_s. The scourhole should then be stabilized. If the scourhole is of such size that it will present maintenance, safety, or aesthetic problems, other outlet protection will be needed.

Evaluate the downstream channel stability and provide appropriate erosion protection if channel degradation is expected to occur. Figure 4.7-1 provides the riprap size recommended for use downstream of energy dissipators.

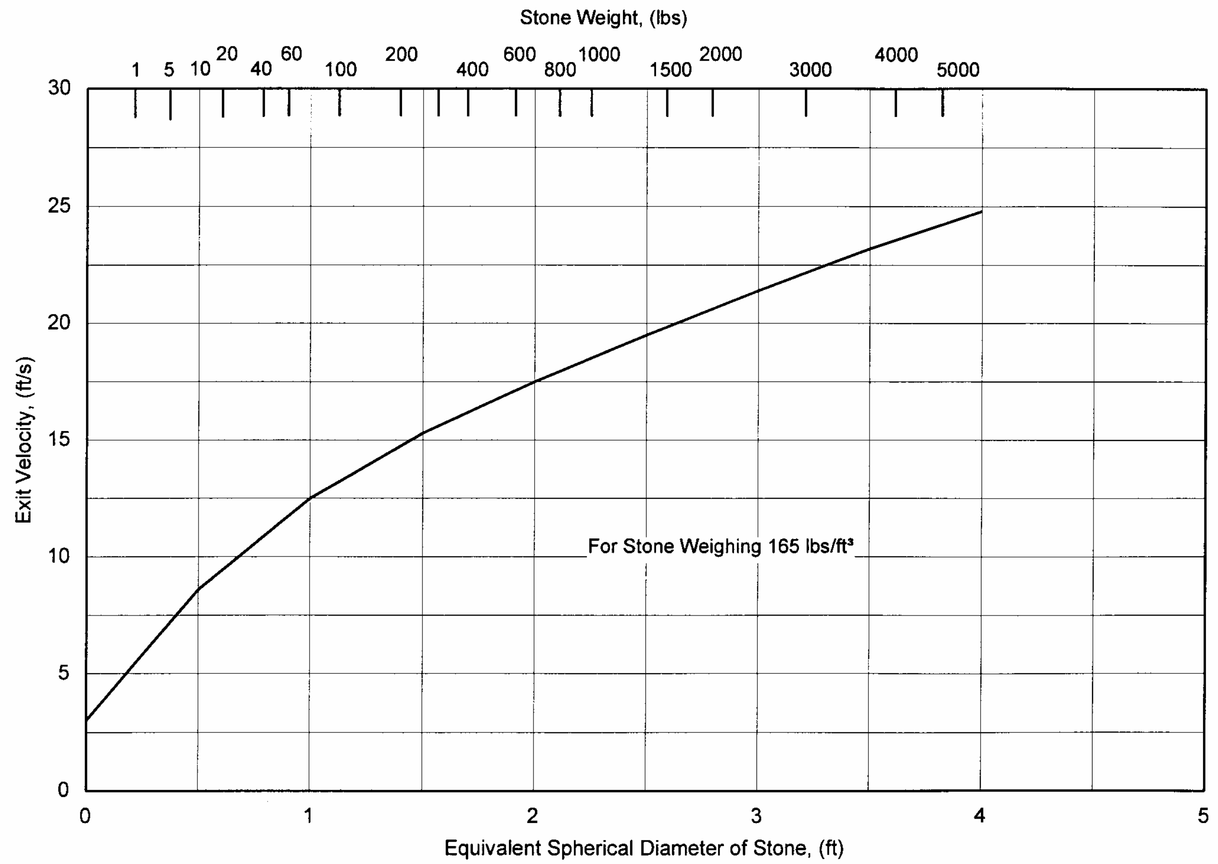


Figure 4.7-1 Riprap Size for Use Downstream of Energy Dissipator
(Source: Searcy, 1967)

4.7.4 – Riprap Aprons

4.7.4.1 Description

A riprap-lined apron is a commonly used practice for energy dissipation because of its relatively low cost and ease of installation. A flat riprap apron can be used to prevent erosion at the transition from a pipe or box culvert outlet to a natural channel. Protection is provided primarily by having sufficient length and flare to dissipate energy by expanding the flow. Riprap aprons are appropriate when the culvert outlet Fr is less than or equal to 2.5.See Section 4.4.8.4 for design procedures for this type of structure.

4.7.5 – Riprap Basins

4.7.5.1 Description

Another method to reduce the exit velocities from storm water outlets is through the use of a riprap basin. A riprap outlet basin is a preshaped scourhole lined with riprap that functions as an energy dissipator by forming a hydraulic jump.

4.7.5.2 Basin Features

General details of the basin recommended in this section are shown in Figure 4.7-5. Principal features of the basin are:

The basin is preshaped and lined with riprap of median size (d_{50}).

The floor of the riprap basin is constructed at an elevation of h_s below the culvert invert. The dimension h_s is the approximate depth of scour that would occur in a thick pad of riprap of size d_{50} if subjected to design discharge. The ratio of h_s to d_{50} of the material should be between 2 and 4.

The length of the energy dissipating pool is 10 x h_s or 3 x W_o, whichever is larger. The overall length of the basin is 15 x h_s or 4 x W_o, whichever is larger.

4.7.5.3 Design Procedure

The following procedure should be used for the design of riprap basins.

Estimate the flow properties at the brink (outlet) of the culvert. Establish the outlet invert elevation such that $TW/y_o \leq 0.75$ for the design discharge.

For subcritical flow conditions (culvert set on mild or horizontal slope) use Figure 4.7-6 or Figure 4.7-7 to obtain y_o/D, then obtain V_o by dividing Q by the wetted area associated with y_o. D is the height of a box culvert. If the culvert is on a steep slope, V_o will be the normal velocity obtained by using the Manning equation for appropriate slope, section, and discharge.

For streambank protection, compute the Froude number for brink conditions with $y_e = (A/2)^{1.5}$. Select d_{50}/y_e appropriate for locally available riprap (usually the most satisfactory results will be obtained if $0.25 < d_{50}/y_e < 0.45$). Obtain h_s/y_e from Figure 4.7-8, and check to see that $2 < h_s/d_{50} < 4$. Recycle computations if h_s/d_{50} falls out of this range.

Size basin as shown in Figure 4.7-5.

Where allowable dissipator exit velocity is specified:

a. Determine the average normal flow depth in the natural channel for the design discharge.

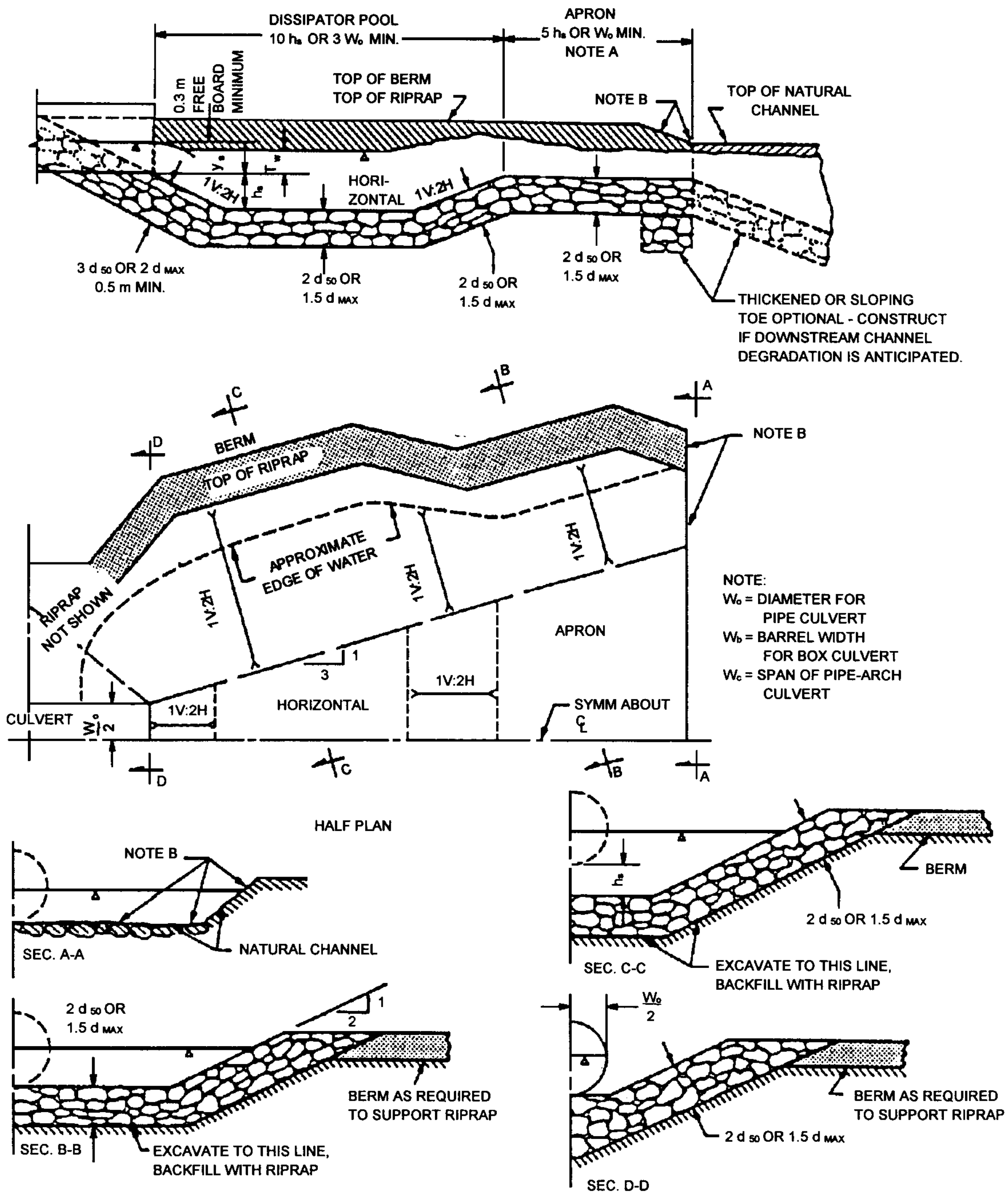
b. Extend the length of the energy basin (if necessary) so the width of the energy basin at section A-A, Figure 4.7-5, times the average normal flow depth in the natural channel is approximately equal to the design discharge divided by the specified exit velocity.

In the exit region of the basin, the walls and apron of the basin should be warped (or transitioned) so the cross section of the basin at the exit conforms to the cross section of the natural channel. Abrupt transition of surfaces should be avoided to minimize separation zones and resultant eddies.

If high tailwater is a possibility and erosion protection is necessary for the downstream channel, the following design procedure is suggested:

- Design a conventional basin for low tailwater conditions in accordance with the instructions above.
- Estimate centerline velocity at a series of downstream cross sections using the information shown in Figure 4.7-9.
- Shape downstream channel and size riprap using Figure 4.7-1 and the stream velocities obtained above.

Material, construction techniques, and design details for riprap should be in accordance with specifications in the Federal Highway publication HEC No. 11 entitled Use of Riprap For Bank Protection.



NOTE A - IF EXIT VELOCITY OF BASIN IS SPECIFIED, EXTEND BASIN AS REQUIRED TO OBTAIN SUFFICIENT CROSS-SECTIONAL AREA AT SECTION A-A SUCH THAT Q/(CROSS SECTION AREA AT SEC. A-A) = SPECIFIED EXIT VELOCITY.

NOTE B - WARP BASIN TO CONFORM TO NATURAL STREAM CHANNEL. TOP OF RIPRAP IN FLOOR OF BASIN SHOULD BE AT THE SAME ELEVATION OR LOWER THAN NATURAL CHANNEL BOTTOM AT SEC. A-A.

Figure 4.7-5 Details of Riprap Outlet Basin
(Source: HEC-14, 1983)

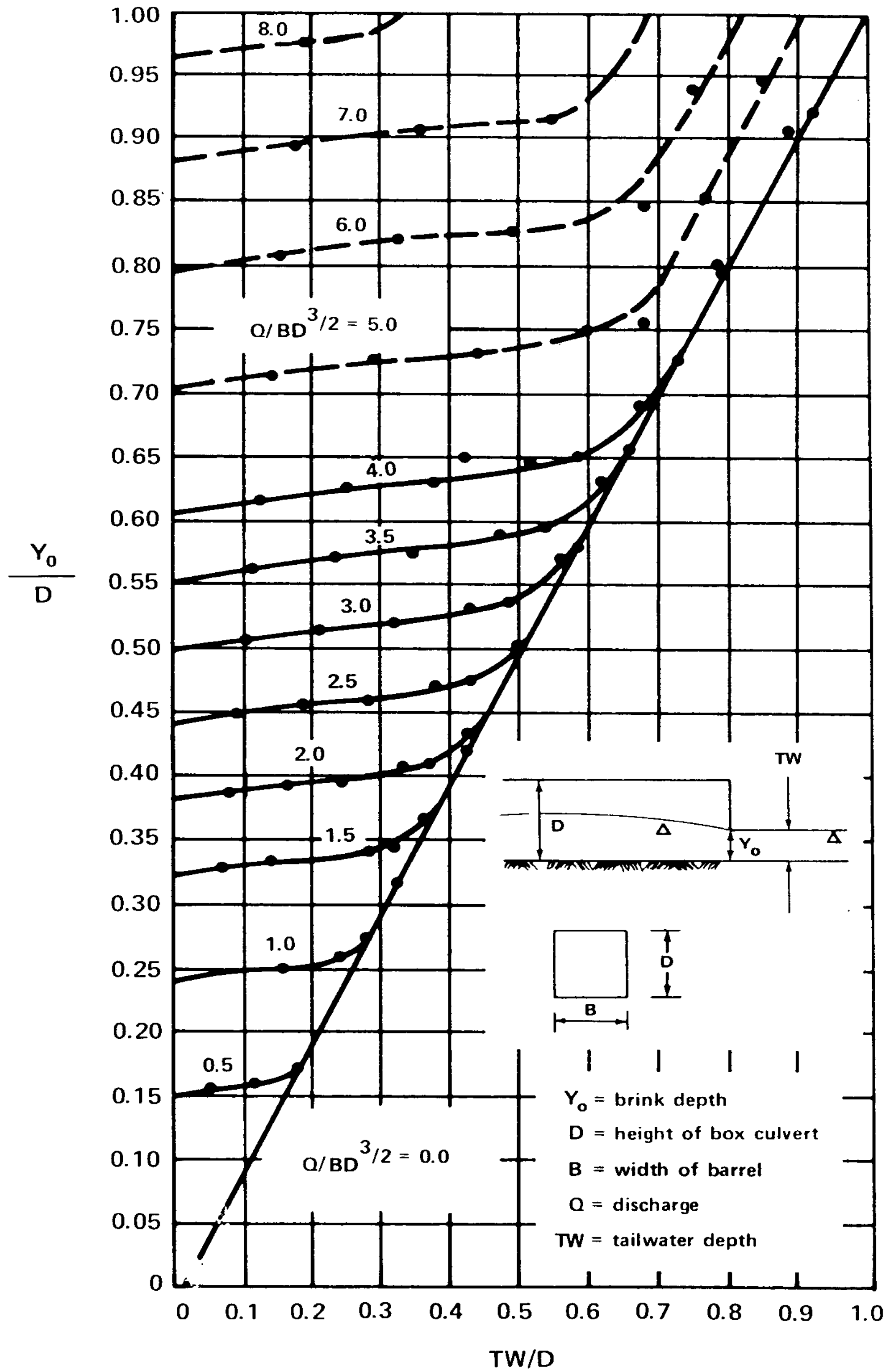


Figure 4.7-6 Dimensionless Rating Curves for the Outlets of Rectangular Culverts on Horizontal and Mild Slopes
(Source: USDOT, FHWA, HEC-14, 1983)

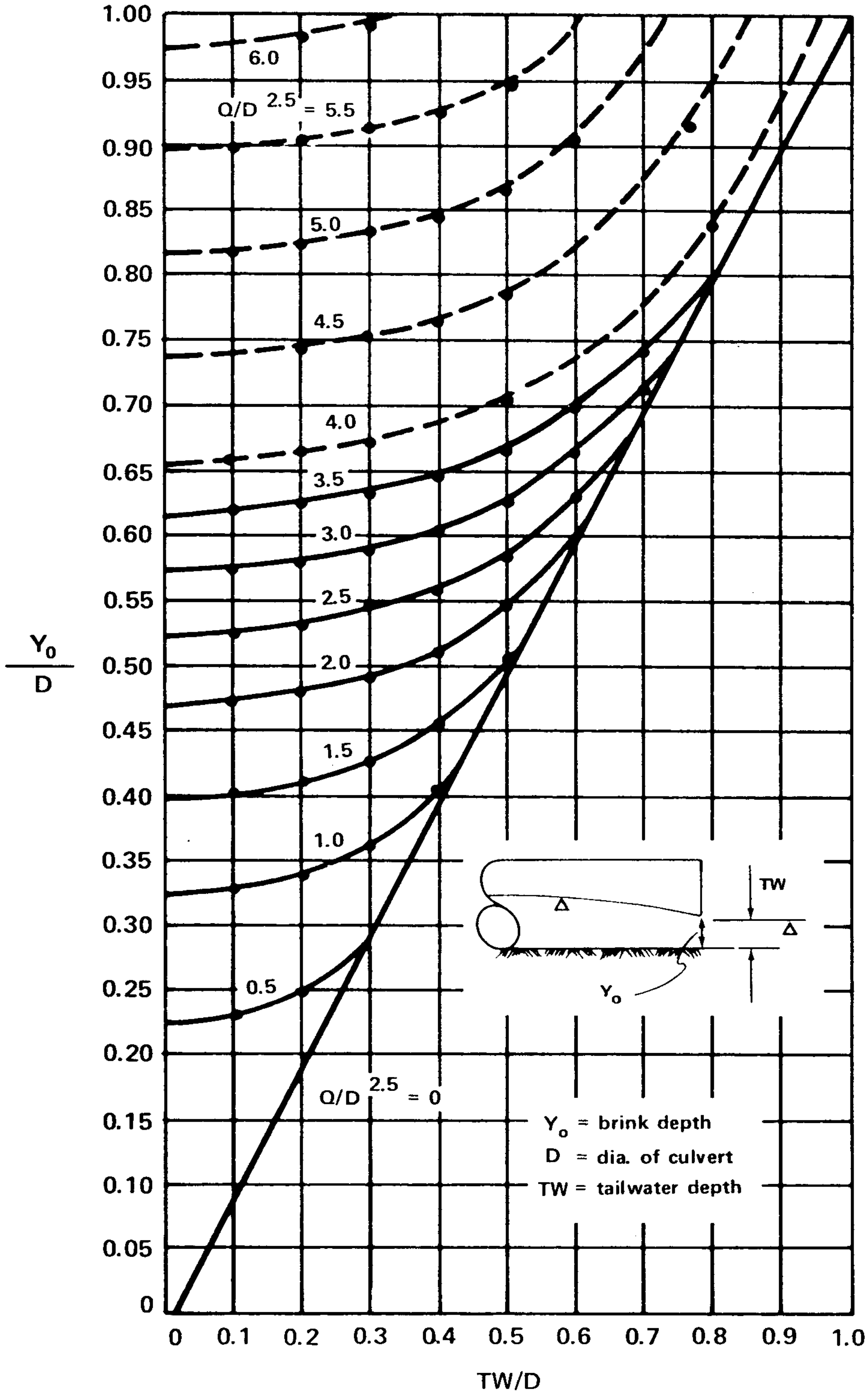


Figure 4.7-7 Dimensionless Rating Curves for the Outlets of Circular Culverts on Horizontal and Mild Slopes
(Source: USDOT, FHWA, HEC-14, 1983)

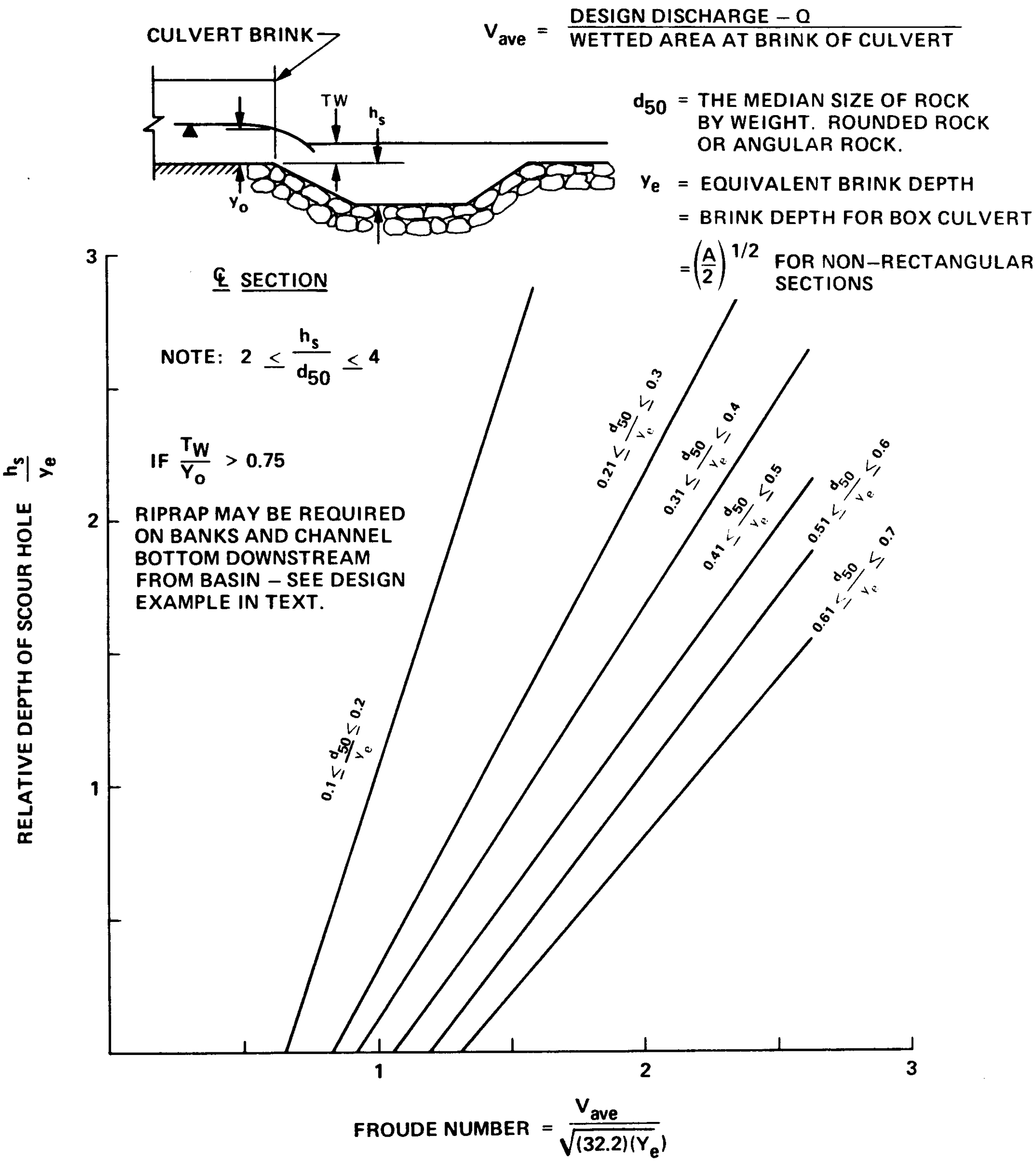


Figure 4.7-8 Relative Depth of Scour Hole Versus Froude Number at Brink of Culvert with Relative Size of Riprap as a Third Variable
(Source: USDOT, FHWA, HEC-14, 1983)

4.7.5.4 Design Considerations

Riprap basin design should include consideration of the following:

The dimensions of a scourhole in a basin constructed with angular rock can be approximately the same as the dimensions of a scourhole in a basin constructed of rounded material when rock size and other variables are similar.

When the ratio of tailwater depth to brink depth, TW/y_o, is less than 0.75 and the ratio of scour depth to size of riprap, h_s/d_{50}, is greater than 2.0, the scourhole should function very efficiently as an energy dissipator. The concentrated flow at the culvert brink plunges into the hole, a jump forms against the downstream extremity of the scourhole, and flow is generally well dispersed leaving the basin.

The mound of material formed on the bed downstream of the scourhole contributes to the dissipation of energy and reduces the size of the scourhole; that is, if the mound from a stable scoured basin is removed and the basin is again subjected to design flow, the scourhole will enlarge.

For high tailwater basins (TW/y_o greater than 0.75), the high velocity core of water emerging from the culvert retains its jet-like character as it passes through the basin and diffuses similarly to a concentrated jet diffusing in a large body of water. As a result, the scourhole is much shallower and generally longer. Consequently, riprap may be required for the channel downstream of the rock-lined basin.

It should be recognized that there is a potential for limited degradation to the floor of the dissipator pool for rare event discharges. With the protection afforded by the $2(d_{50})$ thickness of riprap, the heavy layer of riprap adjacent to the roadway prism, and the apron riprap in the downstream portion of the basin, such damage should be superficial.

See Section 4.4.8 or FHWA HEC No. 11 for details on riprap materials and use of filter fabric.

Stability of the surface at the outlet of a basin should be considered using the methods for open channel flow as outlined in Section 4.4, *Open Channel Design.*

4.7.5.5 Example Designs

Following are some example problems to illustrate the design procedures outlined.

Example 1

Given:	Box culvert - 8 ft by 6 ft	Design Discharge Q = 800 cfs
	Supercritical flow in culvert	Normal flow depth = brink depth
	Y_o = 4 ft	Tailwater depth TW = 2.8 ft

Find: Riprap basin dimensions for these conditions

Solution: Definition of terms in Steps 1 through 5 can be found in Figures 4.7-5 and 4.7-8.

$y_o = y_e$ for rectangular section; therefore, with y_o given as 4 ft, y_e = 4 ft.

$V_o = Q/A = 800/(4 \times 8) = 25$ ft/s

Froude Number = $Fr = V/(g \times y_e)^{0.5}$ ($g = 32.2$ ft/s^2)

$Fr = 25/(32.2 \times 4)^{0.5} = 2.20 < 2.5$ O.K.

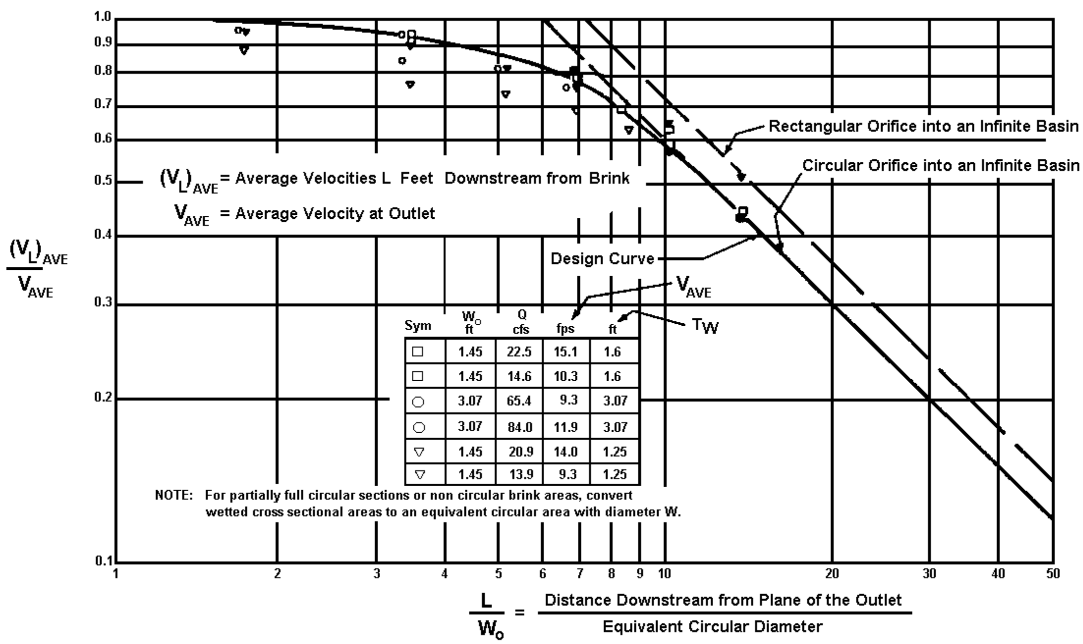


Figure 4.7-9 Distribution of Centerline Velocity for Flow from Submerged Outlets to Be Used for Predicting Channel Velocities Downstream from Culvert Outlet Where High Tailwater Prevails

(Source: USDOT, FHWA, HEC-14, 1983)

$TW/y_e = 2.8/4.0 = 0.7$ Therefore, $TW/y_e < 0.75$ OK

Try $d_{50}/y_e = 0.45$, $d_{50} = 0.45 \times 4 = 1.80$ ft

From Figure 4.7-8, $h_s/y_e = 1.6$, $h_s = 4 \times 1.6 = 6.4$ ft

h_s/d_{50} = 6.4/1.8 = 3.6 ft, 2 < h_s/d_{50} < 4   OK

L_s = 10 x h_s = 10 x 6.4 = 64 ft (L_s = length of energy dissipator pool)

L_s min = 3 x W_o = 3 x 8 = 24 ft; therefore, use L_s = 64 ft

L_B = 15 x h_s= 15 x 6.4 = 96 ft (L_B = overall length of riprap basin)

L_B min = 4 x W_o = 4 x 8 = 32 ft; therefore, use L_B = 96 ft

Thickness of riprap: On the approach = 3 x d_{50} = 3 x 1.8 = 5.4 ft

Remainder = 2 x d_{50} = 2 x 1.8 = 3.6 ft

Other basin dimensions designed according to details shown in Figure 4.7-5.

Example 2

Given: Same design data as Example 1 except:

Tailwater depth TW = 4.2 ft

Downstream channel can tolerate only 7 ft/s discharge

Find: Riprap basin dimensions for these conditions

Solutions: Note -- High tailwater depth, TW/y_o = 4.2/4 = 1.05 > 0.75

From Example 1: d_{50} = 1.8 ft, h_s = 6.4 ft, L_s = 64 ft, L_B = 96 ft.

Design riprap for downstream channel. Use Figure 4.7-9 for estimating average velocity along the channel. Compute equivalent circular diameter D_e for brink area from:

$A = 3.14D_e^2/4 = y_o \times W_o = 4 \times 8 = 32\ ft^2$

$D_e = ((32 \times 4)/3.14)^{0.5} = 6.4$ ft

V_o = 25 ft/s (From Example 1)

Set up the following table:

L/D_e	**L (ft)**	**V_L/V_o**	**v_1 (ft/s)**	***Rock Size*** **d_{50} (ft)**
(Assume)	(Compute)	(Fig. 4.7-9)	(Fig. 4.7-1)	$D_e = W_o$
10	64	0.59	14.7	1.4
15*	96	0.37	9.0	0.6
20	128	0.30	7.5	0.4

21	135	0.28	7.0	0.4

*L/W_o is on a logarithmic scale so interpolations must be done logarithmically.

Riprap should be at least the size shown but can be larger. As a practical consideration, the channel can be lined with the same size rock used for the basin. Protection must extend at least 135 ft downstream from the culvert brink. Channel should be shaped and riprap should be installed in accordance with details shown in the HEC No. 11 publication.

Example 3

Given: 6-ft diameter CMC

Design discharge Q = 135 cfs

Slope channel $S_o = 0.004$

Manning's n = 0.024

Normal depth in pipe for Q = 135 cfs is 4.5 ft

Normal velocity is 5.9 ft/s

Flow is subcritical

Tailwater depth TW = 2.0 ft

Find: Riprap basin dimensions for these conditions.

Solution:

Determine y_o and V_o

$Q/D^{2.5} = 135/6^{2.5} = 1.53$

TW/D = 2.0/6 = 0.33

From Figure 4.7-7, $y_o/D = 0.45$

$y_o = .45 \times 6 = 2.7$ ft

$TW/y_o = 2.0/2.7 = 0.74$ $TW/y_o < 0.75$ O.K.

Determine Brink Area (A) for $y_o/D = 0.45$

From Uniform Flow in Circular Sections Table (from Section 4.3)

For $y_o/D = d/D = 0.45$

$A/D^2 = 0.3428$; therefore, $A = 0.3428 \times 6^2 = 12.3\ ft^2$

$V_o = Q/A = 135/12.3 = 11.0$ ft/s

For Froude number calculations at brink conditions,

$y_e = (A/2)^{1/2} = (12.3/2)^{1/2} = 2.48$ ft

Froude number = $Fr = V_o/(32.2 \times y_e)^{1/2} = 11/(32.2 \times 2.48)^{1/2} = 1.23 < 2.5$ OK

For most satisfactory results, $0.25 < d_{50}/y_e < 0.45$

Try $d_{50}/y_e = 0.25$

$d_{50} = 0.25 \times 2.48 = 0.62$ ft

From Figure 4.7-8, $h_s/y_e = 0.7$; therefore, $h_s = 0.7 \times 2.48 = 1.74$ ft

Uniform Flow in Circular Sections Flowing Partly Full (From Section 4.3)

Check: $h_s/d_{50} = 1.74/0.62 = 2.8$, $2 < h_s/d_{50} < 4$ OK

$L_s = 10 \times h_s = 10 \times 1.74 = 17.4$ ft or $L_s = 3 \times W_o = 3 \times 6 = 18$ ft;

therefore, use $L_s = 17.4$ ft

$L_B = 15 \times h_s = 15 \times 1.74 = 26.1$ ft or $L_B = 4 \times W_o = 4 \times 6 = 24$ ft;

therefore, use $L_B = 26.1$ ft

$d_{50} = 0.62$ ft or use $d_{50} = 8$ in

Other basin dimensions should be designed in accordance with details shown on Figure 4.7-5. Figure 4.7-10 is provided as a convenient form to organize and present the results of riprap basin designs.

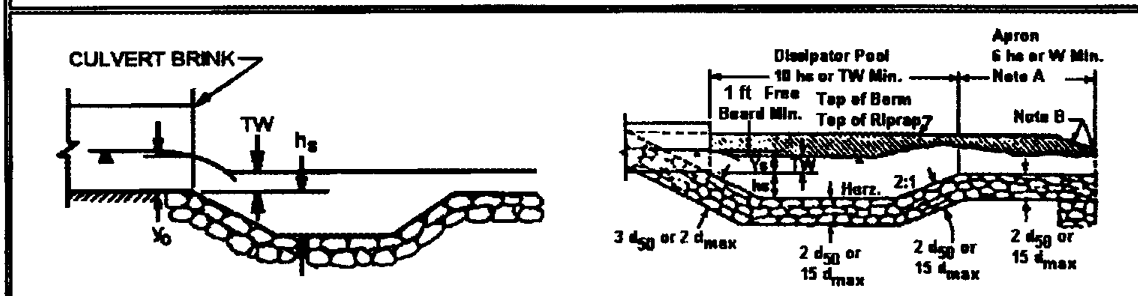
Note: When using the design procedure outlined in this section, it is recognized that there is some chance of limited degradation of the floor of the dissipator pool for rare event discharges. With the protection afforded by the 3 x d_{50} thickness of riprap on the approach and the 2 x d_{50} thickness of riprap on the basin floor and the apron in the downstream portion of the basin, the damage should be superficial.

RIPRAP BASIN

Project No. ____________________

Designer ____________________ Date ________

Reviewer ____________________ Date ________



DESIGN VALUES	TRIAL 1	FINAL TRIAL
Equi. Depth, d_E		
D_{50}/d_E		
D_{50}		
Froude No., Fr		
h_s/d_E		
h_s		
h_s/D_{50}		
$2 < h_s/D_{50} < 4$		

BASIN DIMENSIONS		FEET	
Pool length is the larger of:	$10h_s$		
	$3W_o$		
Basin length is the larger of:	$15h_s$		
	$4W_o$		
Approach Thickness	$3D_{50}$		
Basin Thickness	$2D_{50}$		

TAILWATER CHECK	
Tailwater, TW	
Equivalent depth, d_E	
TW/d_E	
IF $TW/d_E > 0.75$, calculate riprap downstream	
$D_E=(4A_c/\pi)^{0.5}$	

DOWNSTREAM RIPRAP				
L/D_E	L	V_L/V_o	V_L	D_{50}

Figure 4.7-10 Riprap Basin Design Form
(Source: USDOT, FHWA, HEC-14, 1983)

4.7.6 – Baffled Outlets

4.7.6.1 Description

The baffled outlet (also known as the Impact Basin - USBR Type VI) is a boxlike structure with a vertical hanging baffle and an end sill, as shown in Figure 4.7-11. Energy is dissipated primarily through the impact of the water striking the baffle and, to a lesser extent, through the resulting turbulence. This type of outlet protection has been used with outlet velocities up to 50 feet per second and with Froude numbers from 1 to 9. Tailwater depth is not required for adequate energy dissipation, but a tailwater will help smooth the outlet flow.

4.7.6.2 Design Procedure

The following design procedure is based on physical modeling studies summarized from the U.S. Department of Interior (1978). The dimensions of a baffled outlet as shown in Figure 4.7-11 should be calculated as follows:

Determine input parameters, including:

h = Energy head to be dissipated, in ft (can be approximated as the difference between channel invert elevations at the inlet and outlet)

Q = Design discharge (cfs)

v = Theoretical velocity (ft/s = 2gh)

A = Q/v = Flow area (ft^2)

d = $A^{0.5}$= Representative flow depth entering the basin (ft) *assumes square jet*

Fr = $v/(gd)^{0.5}$ = Froude number, dimensionless

Calculate the minimum basin width, W, in ft, using the following equation.

$W/d = 2.88Fr^{0.566}$ or $W = 2.88dFr^{0.566}$ (4.7.2)

Where:

W = minimum basin width (ft)

d = depth of incoming flow (ft)

Fr = $v/(gd)^{0.5}$ = Froude number, dimensionless

The limits of the W/d ratio are from 3 to 10, which corresponds to Froude numbers 1 and 9. If the basin is much wider than W, flow will pass under the baffle and energy dissipation will not be effective.

Calculate the other basin dimensions as shown in Figure 4.7-11, as a function of W. Construction drawings for selected widths are available from the U.S. Department of the Interior (1978).

Calculate required protection for the transition from the baffled outlet to the natural channel based on the outlet width. A riprap apron should be added of width W, length W (or a 5-foot minimum), and depth f (W/6). The side slopes should be 1.5:1, and median rock diameter should be at least W/20.

Calculate the baffled outlet invert elevation based on expected tailwater. The maximum distance between expected tailwater elevation and the invert should be b + f or some flow will go over the baffle with no energy dissipation. If the tailwater is known and fairly controlled, the baffled outlet invert should be a distance, b/2 + f, below the calculated tailwater elevation. If tailwater is uncontrolled, the baffled outlet invert should be a distance, f, below the downstream channel invert.

Calculate the outlet pipe diameter entering the basin assuming a velocity of 12 ft/s flowing full.

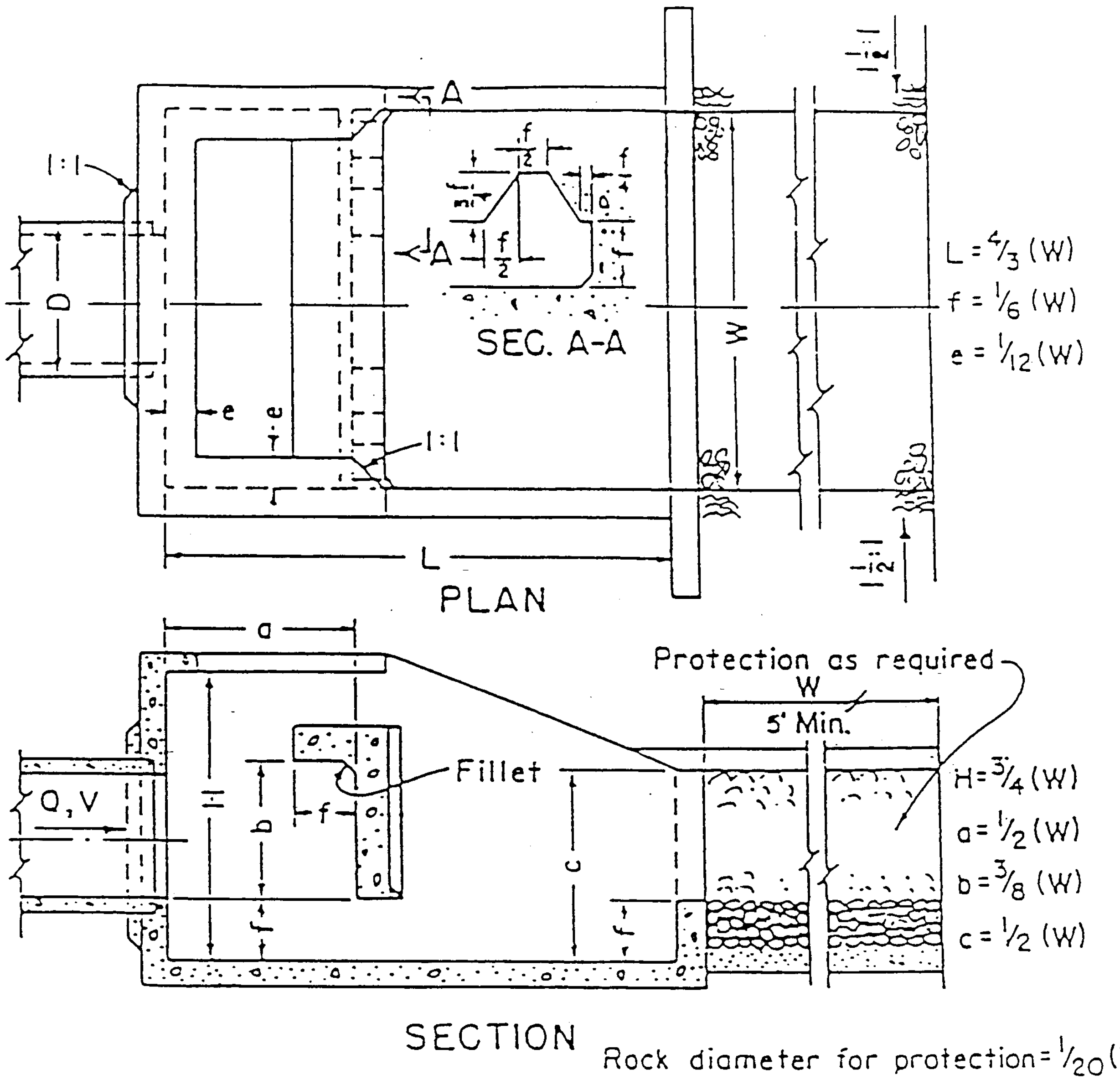


Figure 4.7-11 Schematic of Baffled Outlet
(Source: U.S. Dept. of the Interior, 1978)

If the entrance pipe slopes steeply downward, the outlet pipe should be turned horizontal for at least 3 ft before entering the baffled outlet.

If it is possible that both the upstream and downstream ends of the pipe will be submerged, provide an air vent approximately 1/6 the pipe diameter near the upstream end to prevent pressure fluctuations and possible surging flow conditions.

4.7.6.3 Example Design

A cross-drainage pipe structure has a design flow rate of 150 cfs, a head, h, of 15 ft from invert of pipe, and a tailwater depth, TW, of 3 ft above ground surface. Find the baffled outlet basin dimensions and inlet pipe requirements.

1. Compute the theoretical velocity from

   $v = (2gh)^{0.5} = [2(32.2 \text{ ft/sec}^2)(15 \text{ ft})]^{0.5} = 31.1$ ft/s

   This is less than 50 ft/s, so a baffled outlet is suitable.

2. Determine the flow area using the theoretical velocity as follows:

   $A = Q/v = 150 \text{ cfs}/31.1 \text{ ft/sec} = 4.8 \text{ ft}^2$

3. Compute the flow depth using the area from Step 2.

   $d = (A)^{0.5} = (4.8 \text{ ft}^2)^{0.5} = 2.12$ ft

4. Compute the Froude number using the results from Steps 1 and 3.

   $Fr = v/(gd)^{0.5} = 31.1 \text{ ft/sec}/[(32.2 \text{ ft/sec}^2)(2.12 \text{ ft})]^{0.5} = 3.8$

5. Determine the basin width using equation 4.7.2 with the Froude number from Step 4.

   $W = 2.88\ dFr^{0.566} = 2.88\ (2.12)\ (3.8)^{0.566} = 13.0$ ft (minimum)

   Use 13 ft as the design width.

6. Compute the remaining basin dimensions (as shown in Figure 4.7-11):

   L = 4/3 (W) = 17.3 ft, use L = 17 ft, 4 in
   f = 1/6 (W) = 2.17 ft, use f = 2 ft, 2 in
   e = 1/12 (W) = 1.08 ft, use e = 1 ft, 1 in
   H = 3/4 (W) = 9.75 ft, use H = 9 ft, 9 in
   a = 1/2 (W) = 6.5 ft, use a = 6 ft, 6 in
   b = 3/8 (W) = 4.88 ft, use b = 4 ft, 11 in
   c = 1/2 (W) = 6.5 ft, use c = 6 ft, 6 in

Baffle opening dimensions would be calculated as shown in Figure 4.7-11.

7. Basin invert should be at b/2 + f below tailwater, or

   (4 ft, 11 in)/2 + 2 ft, 2 in = 4.73 ft

   Use 4 ft 8 in; therefore, invert should be 2 ft, 8 in below ground surface.

8. The riprap transition from the baffled outlet to the natural channel should be 13 ft long by 13 ft wide by 2 ft, 2 in deep (W x W x f). Median rock diameter should be of diameter W/20, or about 8 in.

9. Inlet pipe diameter should be sized for an inlet velocity of about 12 ft/s.

   $(3.14d)^2 /4 = Q/v$; $d = [(4Q)/3.14v)]^{0.5} = [(4(150 \text{ cfs})/3.14(12 \text{ ft/sec})]^{0.5} = 3.99$ ft

   Use 48-in pipe. If a vent is required, it should be about 1/6 of the pipe diameter or 8 in.

4.7.7 – Grade Control Structures

When channels are relocated through non-stable soils and stream gradients are increased, the stream bottom may degrade or dig itself deeper. This can cause bank instability, increased upstream scouring, and sloughing of natural slopes. The U.S. Soil Conservation Services (SCS) requires that streambed stability be maintained in any of its stream projects. This can be accomplished by grade stabilization structures; in essence a series of low-head weirs.

If designed and constructed with ecological values in mind, these structures can double as habitat enhancement devices. If improperly planned however, they can atually degrade habitat values. The most productive method of installing these structures is to use low weirs that pool water just a short distance (approximately 100 feet) upstream. A plunge pool will form just below the structures, and a riffle area should develop below this pool. The next structure should be located downstream a sufficient distance to avoid impounding the riffle area below the pool at the base of the upstream weir.

Specific construction requirements and techniques can be obtained from the SCS or other agencies upon request. The intent of this general discussion of grade stabilization structures is to promote consideration of such measures early in the planning process.

Source: US Army Corp of Engineers, Nashville District, *“Mitigating the Impacts of Stream Alterations”*, unkn.

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CHAPTER 5 - STORM WATER CONTROLS

Appendix G contains an exhaustive discussion and detailed examples of storm water controls that can be implemented in land development to meet the goals of protecting water quality, minimizing streambank erosion, and reducing flood volumes. It is an excellent planning and design resource document and has valuable design examples that the Town of Copper Canyon encourages local developers to consider in their site planning. Although it is primarily oriented toward water quality issues, these storm water controls bring additional and valuable benefits for flood control and streambank protection. Many of the listed storm water control features and techniques enhance the aesthetics and value of land developments, as well as providing a drainage function.

The Town of Copper Canyon does not mandate the use of any of these storm water controls, but recognizes the inherent values of their application in overall storm water management.