VOLUME 2
DESIGN METHODOLOGY ANDPROCREDURES

PINAL DRAINAGE ORGANIZATION

The drainage policies used in Pinal County are set forth in the Ordinance to Regulate Drainage in Pinal County. The Pinal County Drainage Manual sets forth design criteria, methodology and procedures and is comprised of two volumes; Volume 1: Policies and Design Criteria and Volume 2: Design Methodology and Procedures. The table of contents for both Volume 1 and 2 are included in each volume for easy accessibility.

Ordinance to Regulate Drainage

The ordinance document establishes general drainage policies and provides the minimum standards for the design of drainage and storm water management facilities within unincorporated Pinal County.

Drainage Manual Volume 1: Design Criteria

Volume 1 establishes minimum standards and criteria for the design of drainage and storm water management facilities within unincorporated Pinal County. It is desirable that the policies and standards set forth in this manual be adopted by local jurisdictional entities so that uniform drainage policies and practices will be established throughout the County. However, each entity has the authority to establish its own policies within its jurisdiction; therefore, the user is encouraged to review the policies and standards for the jurisdiction in which the project is located.

Drainage Manual Volume 2: Design Methodology & Procedures

Volume 2 is intended to serve as an aid in the design of drainage and stormwater management facilities. The manual provides a convenient source of technical information and presents methodologies and procedures acceptable to the County. However, the methodologies and procedures presented in the manual are not comprehensive and are not intended to replace or inhibit sound engineering judgment.

PINAL COUNTY DRAINAGE MANUAL
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VOLUME 2
DESIGN METHODOLOGY AND PROCREDURES

Chapter 1: Introduction

1.1 PURPOSE

The purpose of the Pinal County Drainage Manual (referred to as Manual in these documents) is to establish general drainage policies, provide the minimum criteria, and to serve as an aid in the design of drainage and stormwater management facilities within Pinal County. The Manual recommends design standards and criteria that if adopted by local jurisdictional entities will establish uniform drainage policies and practices throughout the County. The Manual comprises two volumes as described below.

It is the overall and primary objective of Pinal County to provide drainage design criteria which serve to protect the health, safety, and general welfare of the citizens of the community, with regards to flooding and drainage issues.

1.2 SCOPE

The Pinal County Drainage Manual must be used for any project being reviewed and approved by the County. This includes projects on County property as well as within County rights-of-way. Projects on private property that must be approved by the County must also follow the requirements set forth in this Manual.

Projects under the jurisdiction of towns or cities in Pinal County may also be subject to the provisions of this Manual. Check with the appropriate jurisdiction for specific requirements, as they may not use this Manual or they may have adopted local modifications to the Manual provisions.

This manual is not intended to conflict with any other Pinal County design standards or ordinances. If a conflict does arise, it is the intent of the County to require the more stringent or restrictive standard to apply.

This document provides general engineering guidelines and is not intended to be a substitute for sound engineering judgment when dealing with specific design problems. Specific engineering procedures and methodologies are not always dictated within this manual, but instead are often cited by reference to other widely accepted design manuals published by the U.S. Army Corps of Engineers, Arizona Department of Transportation, Federal Highway Administration and other regulatory agencies within Arizona. This approach is intended to provide the engineer with the flexibility to apply engineering methods most appropriate to an individual project. Other objectives of this manual include (1) minimizing the review time for drainage reports, (2) providing the design engineer with the County's drainage requirements prior to initiating a project; and, (3) providing for drainage infrastructure that is functional, durable and aesthetic.

1.2.1 Applicability

This manual is to be used by Civil Engineers in preparing drainage reports for stormwater planning, analysis and design within Pinal County, Arizona. Many procedures that are presented or referenced within this manual have a limited range of applicability. An attempt has been made in this manual to specify these ranges whenever possible. However, it is the responsibility of the practicing engineer to utilize sound engineering judgment and experience when applying any engineering methodology to a particular project.

1.2.2 Limitations of Liability

In any case, however, the Engineer performing stormwater analyses and preparing drainage reports for projects in Pinal County must assume the final responsibility for the appropriateness of their analysis and correctness of their results. This Manual is not intended to provide "lookup" answers to drainage questions or "one size fits all" methods. Proper and sound engineering judgment is required in all cases. The inappropriate use of and adherence to this Manual does not relieve the Engineer from the professional responsibility to provide an appropriate design.

Adherence to the provisions of this Manual and use of any method contained herein does not relieve any owner, Engineer, or designer of any present or future liability related to the design of works covered by this Manual. Pinal County is not liable for direct or consequential damages resulting from the construction of works covered by this Manual, whether the provisions of this Manual were followed or not.

1.2.3 Floodplain Regulations and Drainage Policies

The County is mandated to adopt and enforce regulations designed to protect health, safety, and general welfare of the citizens within the jurisdiction area of Pinal County and to minimize public and private losses due to flood conditions in specific areas.

The County is also mandated by the Federal Emergency Management Agency (FEMA) to regulate areas of special flood hazards. FEMA supplies the District with Floodway Maps and Flood Insurance Rate Maps which provide flood risk information and other technical data to be used in administering both floodplain management and insurance aspects of the National Flood Insurance Program (NFIP).

Requirements from both agencies have led to the adoption of the original Pinal County Drainage Ordinance on (date), and subsequent revision(s). The Ordinance requires that Pinal County regulate all activities within and along all watercourses within its jurisdiction.

1.2.4 Updates

Pinal County may choose to modify and update this Manual at any time, and anyone needing to perform design or construct works covered by this Manual must be sure that they are using the most current version. Check with the Pinal County Department of Public Works or the Pinal County Flood Control District for the current version.

VOLUME 2
DESIGN METHODOLOGY AND PROCREDURES

Chapter 2: Hydrology

2.1 INTRODUCTION

Hydrology for the design of stormwater management facilities in Pinal County can be obtained from methods used to determine the design hydrology depends upon the characteristics of the watershed and the intended application. For small (less than 160 acres) urban watersheds with fairly uniform land usage, the Rational Method may be utilized to estimate peak discharges.

2.2 METHODS

The methods used to determine the design hydrology depends upon the characteristics of the watershed and the intended application. For small (less than 160 acres) urban watersheds with fairly uniform land usage, the Rational Method may be utilized to estimate peak runoff and required detention storage. This method is not suitable for larger more complex watersheds or if a hydrograph or channel routing is required.

For larger more complex watersheds, a precipitation-runoff and routing simulation model should be developed. Although not necessarily required, the U.S Army Corps of Engineers (USACE) HEC-1 Flood Hydrograph Program and the USACE HEC Hydrologic Modeling System (HEC-HMS) are acceptable simulation models for complex watersheds that are public domain. Guidance in the development of the simulation models and the estimation of necessary input parameters are provided in the HEC-1 and HEC-HMS User's Manuals.

2.2.1 Rational Method

2.2.1.1 Limitations

Application of the Rational Method is appropriate for watersheds less than 160 acres in size. This is based on the assumption that the rainfall intensity is to be uniformly distributed over the drainage area at a uniform rate lasting for the duration of the storm.

2.2.1.2 Peak Discharge Values

The Rational Equation relates rainfall intensity, a runoff coefficient and the watershed size to the generated peak discharge. The following shows this relationship:

$$Q = CiA \tag{2.1}$$

Where:

Q = peak discharge (cfs)
c = runoff coefficient
i = rainfall intensity (in/hr) for the duration of T_c
A = drainage area (acres)
T_c = the time of concentration (hrs)

The Rational Equation is based on the concept that the application of a steady, uniform rainfall intensity will produce a peak discharge at such a time when all points of the watershed are contributing to the outflow at the point of design. Such a condition is met when the elapsed time is equal to the time of concentration, T_c, which is defined to be the floodwave travel time from the most remote part of the watershed to the point of design. The time of concentration should be computed by applying the following equation developed by Papadakis and Kazan (1987):

- The drainage area should not exceed 160 acres
- The time of concentration (T_c) should not exceed 60 minutes
- The design does not involve channel routing

The Rational Method parameters are discussed in further detail in the following sections.

Runoff coefficient, C

As rain falls to the ground it gathers and starts to move overland to points of lower elevation. Some water soaks into the ground and some runs away from the point of fall. The nature of the ground surface affects the division between those two components; the harder and more compact the ground surface the less will percolate into the soil.

Hydrologists use the concept of a coefficient to numerically represent the degree to which water will soak in or run off. This coefficient can have values from 0 to 1, with higher numbers representing a higher portion of the water running off. Table 2-1 shows typical values.

The runoff coefficient accounts for the infiltration and interception characteristics of the drainage area. Generally, the runoff coefficient is based upon the typical soil or land use characteristics of the drainage area. For the runoff coefficient, the more impervious the land surface, the less porous the soil, the higher the amount of drainage runoff and the higher the runoff coefficient. The runoff coefficient is also adjusted and increased for larger storm events to account for increased runoff arising from soil saturation and wetted surfaces. Table 2-1 provides runoff coefficients for typical land uses and return periods.

Table 2-1: Runoff Coefficients [1,2]

Land Use Category	2-10 Year		25 Year		50 Year		100 Year	
	min	max	min	max	min	max	min	max
Very Low Density Residential[3]	0.33	0.42	0.36	0.46	0.40	0.50	0.41	0.53
Low Density Residential[3]	0.42	0.48	0.46	0.53	0.50	0.58	0.53	0.60
Medium Density Residential[3]	0.48	0.65	0.53	0.72	0.58	0.78	0.60	0.82
Multiple Family Residential[3]	0.65	0.75	0.72	0.83	0.78	0.90	0.82	0.94
Industrial 1[3]	0.60	0.70	0.66	0.77	0.72	0.84	0.75	0.88
Industrial 2[3]	0.70	0.80	0.77	0.88	0.84	0.95	0.88	0.95
Commercial 1[3]	0.55	0.65	0.61	0.72	0.66	0.78	0.69	0.81
Commercial 2[3]	0.75	0.85	0.83	0.94	0.90	0.95	0.94	0.95
Pavement and Rooftops	0.75	0.85	0.83	0.94	0.90	0.95	0.94	0.95
Gravel Roadways & Shoulders	0.60	0.70	0.66	0.77	0.72	0.84	0.75	0.88
Agricultural	0.10	0.20	0.11	0.22	0.12	0.24	0.13	0.25
Lawns/Parks/Cemeteries	0.10	0.25	0.11	0.28	0.12	0.30	0.13	0.31
Desert Landscaping 1	0.55	0.85	0.61	0.94	0.66	0.95	0.69	0.95
Desert Landscaping 2	0.30	0.40	0.33	0.44	0.36	0.48	0.38	0.50
Undeveloped Desert Rangeland	0.30	0.40	0.33	0.44	0.36	0.48	0.38	0.50
Hillslopes, Sonoran Desert	0.40	0.55	0.44	0.61	0.48	0.66	0.50	0.69
Mountain Terrain	0.60	0.80	0.66	0.88	0.72	0.95	0.75	0.95

[1] Runoff coefficients for 25-, 50-, and 100-Year storm frequencies were derived using adjustment factors of 1.10, 1.20 and 1.25, respectively, applied to the 2-10 Year values with an upper limit of 0.95

[2] The ranges of runoff coefficients shown for urban land uses were derived from lot coverage standards specified in the zoning ordinances for Maricopa Pinal County

[3] Runoff coefficients for urban land uses are for lot coverage only and do not include the adjacent street and right-of-way, or alleys.

Weighted runoff coefficients

Many times the ground surface in a tributary area cannot be properly represented using a single runoff coefficient. This is usually because of differences in ground cover. For example, between the right-of-way and the inlet some ground may have vegetative cover while other ground is paved.

A weighted runoff coefficient can be computed by proportion to the subareas using Equation (2-2):

$$C_w = \frac{A_1 C_1 + A_2 C_2 + \ldots + A_n C_n}{A_1 + A_2 + \ldots + A_n} \tag{2-2}$$

Where:

A_i =Area in the i_{th} subarea, ac
C_i =Runoff coefficient for the i_{th} subarea

The weighted coefficient (C_w) is used in the Rational Formula just as would a single coefficient.

Rainfall Intensity, i, and Time of Concentration, T_c

The Rational Equation is based on the concept that steady uniform rainfall intensity will produce a peak discharge at such a time when the entire drainage area is contributing to the outflow. This condition is met when the rainfall duration is equal to the time of concentration, T_c, which is defined as the time it takes for water to travel from the most remote part of the drainage area to the point of design. The time of concentration is computed by applying the following equation:

$$T_c = 11.4 L^{0.5} K_b^{0.52} S^{-0.31} i^{-0.38} \qquad (2\text{-}2)$$

Where:

T_c = the time of concentration (hrs)
L = length of the longest flow path (miles)
K_b = watershed resistance coefficient
S = watercourse slope (ft/mi)
i = rainfall intensity (in/hr)

K_b is an approximation of the watershed “roughness” and accounts for its impact on the time it takes flow to travel across a particular type of land surface. The empirical equation for estimating K_b is:

$$K_b = m \log A + b \qquad (2\text{-}3)$$

Where:

K_b = watershed resistance coefficient
A = drainage area (acres)
m and b = empirical equation parameters (unitless)

Table 2-2 provides the equation parameters to determine K_b values for typical land use types.

Since the equation for the time of concentration has two unknowns, T_c and i, the following iterative process using Intensity-Duration-Frequency (I-D-F) curves is required to converge upon the values of T_c and i. I-D-F curves are presented in a later section of this chapter.

1) Determine all the initial parameters (L, K_b, and S) and select the design storm return frequency (2-yr, 5-yr, 10-yr, etc.)

2) Make an initial trial estimate for T_c and obtain the rainfall intensity, i, from the I-D-F curve for the return frequency using T_c as the storm duration.

3) Using the rainfall intensity, solve for T_c in the time of concentration equation.

4) Compare the estimated T_c to the calculated T_c. If the values are not comparable, repeat Steps 2 and 3 until the estimated of T_c and the calculated T_c converge or the difference between the values is less than 10%. Typically, the calculated T_c provides a good subsequent estimate of T_c in the iterative process.

5) Once the values T_c converge, the final rainfall intensity value is obtained from the I-D-F curve.

Table 2-2: Watershed Resistance Coefficients

Type	Description	Typical Applications	Equation Parameters	
			m	b
A	Minimal roughness: Relatively smooth and/or well-graded and uniform land surfaces. Surfaces runoff is sheet flow.	Commercial/industrial areas Residential area Parks and golf courses	-0.00625	0.04
B	Moderately low roughness: Land surfaces have irregularly spaced roughness elements that protrude from the surface but the overall character of the surface is relatively uniform. Surface runoff is predominately sheet flow around the roughness elements.	Agricultural fields Pastures Desert rangelands Undeveloped urban lands	-0.01375	0.08
C	Moderately high roughness: Land surfaces that have significant large to medium-sized roughness elements and/or poorly graded land surfaces that cause the flow to be diverted around the roughness elements. Surface runoff is sheet flow for short distances draining into meandering drainage paths	Hillslopes Brushy alluvial fans Hilly rangeland Disturbed land, mining, etc. Forests with underbrush	-0.02500	0.15
D	Maximum roughness: Rough land surfaces with torturous flow paths. Surface runoff is concentrated in numerous short flow paths that are often oblique to the main flow direction.	Mountains Some wetlands	-0.03000	0.20

Reference: Table 3.1, Drainage Design Manual for Maricopa County Vol. I - Hydrology, FCDMC

Application of the Rational Equation requires consideration of the following:

- The peak discharge rate corresponding to a given intensity would occur only if the rainfall duration is at least equal to the time of concentration
- The calculated runoff is directly proportional to the rainfall intensity

- The frequency of occurrence for the peak discharge is the same as the frequency for the rainfall producing that event
- The runoff coefficient increases as storm frequency decreases.

2.2.1.3 Multiple Basin Approach

The Rational Method can be used to compute peak discharges at intermediate locations within a drainage area less than 160 acres in size. A typical application of this approach is a local storm drain system where multiple subbasins are necessary to compute a peak discharge at each proposed inlet location. Consider a multiple basin problem where a peak discharge is needed for all three individual subareas, subareas A and B combined at Concentration Point 1 and subareas A-B and C combined at Concentration Point 2.

1. Compute the peak discharge for each individual subarea
2. Compute the arithmetically area-weighted value of C for subareas A and B.
3. Calculate the T_c for the combined area of subareas A and B at Concentration Point 1.
4. Compare the T_c values from subareas A and B to the T_c value for the combined area at Concentration Point 1. Compute the peak discharge at Concentration Point 1 using the i for the longest T_c from step 3. If the combined peak discharge is less than the discharges for the individual subareas, use the largest discharge as the peak discharge at Concentration Point 1. The design discharge SHOULD NOT INCREASE going downstream in a conveyance system unless storage facilities are used to attenuate peak flows.
5. Compute the arithmetically area-weighted value of C for subareas A, B, and C.
6. Calculate the T_c for the combined area at Concentration Point 2 using the following two methods:

    Method 1 - Calculate the T_c for the single basin composed of all three subareas.

    Method 2 - Compute the travel time from Concentration Point 1 to Concentration Point 2 using the continuity equation or other appropriate technique and hydraulic parameters for the conveyance path. Add the computed travel time for the conveyance path to the T_c from Concentration Point 1.

7. Compare the T_c values from Methods 1 and 2 as well as the T_c from subarea C and calculate the peak discharge at Concentration Point 2 as follows:

a. If the T_c value from Method 1 is the longest, compute the total peak discharge using the Method 1 intensity, the arithmetically area-weighted value of C for all three subareas and the total contributing drainage area at Concentration Point 2.

b. If the T_c value from Method 2 is the longest, determine i directly from the D-D-F statistics. Compute the total peak discharge at Concentration Point 2 using the arithmetically area-weighted value of C for all three subareas and the total contributing drainage area at Concentration Point 2.

c. If the T_c from subarea C is the longest, compute the total peak discharge using the i for subarea C, the arithmetically area-weighted value of C for all three subareas and the total contributing drainage area at Concentration Point 2.

8. As an alternative to the above procedure, the DDMSW program may be used to calculate the peak discharge at intermediate locations.

2.2.1.4 Detention/Retention Storage Volume Calculations

For purposes of determining on-site detention requirements utilizing the Rational Method, the following equation can be used:

$$V = C(\frac{P}{12})A \qquad (2\text{-}4)$$

Where:

V = Storage Volume (acre-ft)
C = Watershed Runoff Coefficient
P = 100-Year, 2-Hour Precipitation (inches)
A = Drainage Area (acres)

In the case of volume calculations for stormwater storage facility design, P equals the 100-year, 2-hour depth, in inches.

2.2.1.5 Flood Hydrographs

The procedure described within this section should be used in conjunction with the Rational Method for developing hydrographs from small watersheds, for the design of stormwater detention/retention basins, and for other stormwater routing analyses. As with the Rational Method, this hydrograph synthesis should not be used for watershed areas greater that 160 acres. This procedure was taken from Hickok, et. al, 1959, which presented a method of hydrograph synthesis for small watersheds within the Southwest.

A flood hydrograph is developed based on the curvilinear, dimensionless hydrograph shown in tabular form within Table 2-3. The symbols used in Table 2-3 are defined as follows:

T = Cumulative time from beginning of runoff, in minutes.
Tr = Rise time of the hydrograph, in minutes, calculated from the following equations:

$$T_r = 545 \frac{V}{Q_p} \qquad (2\text{-}5)$$

and

$$V = \frac{CAP_1}{12} \qquad (2\text{-}6)$$

Where:

- P_1 = One hour rainfall value for the return period storm under investigation, in inches.
- A = Watershed area in acres.
- C = Runoff coefficient.
- v = Accumulated runoff volume at time t, in acre-feet.
- V = Total runoff volume of storm event, in acre-feet.
- Q_p = Peak discharge, in cfs.
- Q = Discharge at time t/T_r, in cfs.

Table 2-3: Curvilinear, Dimensionless Hydrograph

t/T_r	Q/Q_p	v/V	t/T_r	Q/Q_p	v/V
0.0	0.000	0.000	1.6	0.545	0.671
0.1	0.025	0.002	1.7	0.482	0.707
0.2	0.087	0.007	1.8	0.424	0.742
0.3	0.160	0.020	1.9	0.372	0.773
0.4	0.243	0.036	2.0	0.323	0.799
0.5	0.346	0.063	2.2	0.241	0.841
0.6	0.451	0.096	2.4	0.179	0.875
0.7	0.576	0.136	2.6	0.136	0.900
0.8	0.738	0.180	2.8	0.102	0.917
0.9	0.887	0.253	3.0	0.078	0.932
1.0	1.000	0.325	3.4	0.049	0.953
1.1	0.924	0.400	3.8	0.030	0.965
1.2	0.839	0.464	4.2	0.020	0.973
1.3	0.756	0.523	4.6	0.012	0.979
1.4	0.678	0.578	5.0	0.008	0.983
1.5	0.604	0.627	7.0	0.000	1.000

2.2.2 HEC-1 Flood Hydrographs

The U.S. Army Corps of Engineers rainfall runoff model should be used for modeling larger, more complex watersheds, or for drainage networks requiring routing procedures. The SCS Type II 24-hour storm distributions with antecedent moisture condition II are generally acceptable. The HEC-1 methodology presented within the Arizona Department of Transportation (ADOT) Highway Drainage Design Manual - Hydrology (latest revision) is acceptable for use on projects reviewed by Pinal County.

2.3 INTENSITY-DURATION-FREQUENCY CURVES

2.3.1 Storm Size Probability

Storm intensities used in hydrologic computations vary with the likelihood a storm event will occur. Storm events with a larger probability of occurring are smaller than storm events with a smaller probability of occurring. Put another way, larger storms are less likely to occur at any time than smaller storms. Storm events are classified by the probability of occurrence; a storm may have a 1% chance of a certain size or a 50% chance of a different size. Long ago, someone decided to use the reciprocal of the probability value and came up with the idea of classifying storm event sizes using a year label. For example, a storm event with a 1% chance of happening in any year is called a 100-year storm, while a storm with a 50% chance of happening in any year is called a 2-year storm. Unfortunately, the return period classification system stuck and has widespread use in the popular press. Unfortunately, because it can lead to serious misunderstanding by those who do not realize the probabilistic nature of storm event sizing. Those people believe that once a “100-year” storm event happens another of the same size will not occur for another 100-years.

2.3.2 Rainfall intensity Change During a Storm Event

Storm intensities used in hydrologic computations also vary with the time over which the storm is determined to act or with the length of time since the start of the storm. This is represented in a curve of rainfall intensity versus time for a storm event. A family of such curves representing a series of storm probability for the same location is referred to as Intensity-Duration-Frequency curves.

Table 2-4 shows storm sizes for various return periods and storm length for locations in Pinal County. This data is recorded by the National Oceanic and Atmospheric Agency (NOAA), a federal agency.

Table 2-4: Storm sizes for locations in Pinal County

Location	Storm Probability and Length			
	2-yr, 6-hr	2-yr, 24-hr	100-yr, 6-hr	100-yr, 24-hr
Apache Junction	1.2	1.4	3.2	3.8
Casa Grande	1.3	1.5	3.4	4.6
Coolidge	1.2	1.4	3.2	3.8
Eloy	1.4	1.6	3.4	4.4
Florence	1.4	1.6	3.2	4.0
Kearny	1.6	2.0	3.6	4.8
Mammoth	1.5	1.8	3.4	4.2
Superior	1.8	2.6	4.0	6.0

2.3.3 Rainfall Data for Pinal County Communities

Rainfall data for the communities listed in Table 2-4 is given in Tables 2-5 through 2-12. I-D-F and log I-D-F curves are provided for the same list of communities at the end of this chapter.

Table 2-5: Rainfall data for Apache Junction

PREFRE Program Input Data

2-yr, 6-hr =	1.20
2-yr, 24-hr =	1.40
100-yr, 6-hr =	3.20
100-yr, 24-hr =	3.80

Depth-Duration-Frequency (D-D-F)

Duration	Frequency						
	2-Yr	5-Yr	10-Yr	25-Yr	50-Yr	100-Yr	500-Yr
5 min	0.33	0.43	0.50	0.60	0.68	0.76	0.94
10 min	0.49	0.65	0.77	0.92	1.04	1.16	1.44
15 min	0.59	0.82	0.97	1.17	1.33	1.49	1.86
30 min	0.79	1.09	1.30	1.58	1.80	2.02	2.53
1-hr	1.00	1.35	1.61	1.97	2.25	2.53	3.17
2-hr	1.04	1.47	1.76	2.15	2.45	2.76	3.46
3-hr	1.10	1.55	1.85	2.27	2.59	2.91	3.65
6-hr	1.20	1.70	2.03	2.49	2.85	3.20	4.01
12-hr	1.30	1.85	2.22	2.72	3.11	3.50	4.40
24-hr	1.40	2.00	2.40	2.95	3.38	3.80	4.78

*D-D-F data obtained from PREFRE Program

Localized Intensity-Depth-Frequency (I-D-F)

Duration (minutes)	Frequency						
	2-Yr	5-Yr	10-Yr	25-Yr	50-Yr	100-Yr	500-Yr
5	3.96	5.16	6.00	7.20	8.16	9.12	11.28
10	2.94	3.90	4.62	5.52	6.24	6.96	8.64
15	2.36	3.28	3.88	4.68	5.32	5.96	7.44
30	1.58	2.18	2.60	3.16	3.60	4.04	5.06
60	1.00	1.35	1.61	1.97	2.25	2.53	3.17

Table 2-6: Rainfall data for Case Grande

PREFRE Program Input Data

2-yr, 6-hr =	1.30
2-yr, 24-hr =	1.50
100-yr, 6-hr =	3.40
100-yr, 24-hr =	4.60

Depth-Duration-Frequency (D-D-F)

	Frequency						
Duration	2-Yr	5-Yr	10-Yr	25-Yr	50-Yr	100-Yr	500-Yr
5 min	0.36	0.44	0.50	0.58	0.65	0.72	0.87
10 min	0.54	0.67	0.76	0.89	1.00	1.10	1.34
15 min	0.65	0.83	0.96	1.13	1.27	1.41	1.73
30 min	0.86	1.11	1.29	1.53	1.72	1.91	2.35
1-hr	1.05	1.37	1.60	1.91	2.15	2.39	2.95
2-hr	1.14	1.53	1.79	2.16	2.45	2.74	3.39
3-hr	1.19	1.63	1.92	2.33	2.65	2.97	3.69
6-hr	1.30	1.82	2.17	2.66	3.03	3.40	4.26
12-hr	1.40	2.06	2.49	3.09	3.54	4.00	5.05
24-hr	1.50	2.29	2.81	3.52	4.06	4.60	5.85

*D-D-F data obtained from PREFRE Program

Localized Intensity-Depth-Frequency (I-D-F)

Duration	Frequency						
(minutes)	2-Yr	5-Yr	10-Yr	25-Yr	50-Yr	100-Yr	500-Yr
5	4.32	5.28	6.00	6.96	7.80	8.64	10.44
10	3.24	4.02	4.56	5.34	6.00	6.60	8.04
15	2.60	3.32	3.84	4.52	5.08	5.64	6.92
30	1.72	2.22	2.58	3.06	3.44	3.82	4.70
60	1.05	1.37	1.60	1.91	2.15	2.39	2.95

Table 2-7: Rainfall data for Coolidge

PREFRE Program Input Data

2-yr, 6-hr =	1.20
2-yr, 24-hr =	1.40
100-yr, 6-hr =	3.20
100-yr, 24-hr =	3.80

Depth-Duration-Frequency (D-D-F)

	Frequency						
Duration	2-Yr	5-Yr	10-Yr	25-Yr	50-Yr	100-Yr	500-Yr
5 min	0.33	0.43	0.50	0.60	0.68	0.76	0.94
10 min	0.49	0.65	0.77	0.92	1.04	1.16	1.44
15 min	0.59	0.82	0.97	1.17	1.33	1.49	1.86
30 min	0.79	1.09	1.30	1.58	1.80	2.02	2.53
1-hr	1.00	1.35	1.61	1.97	2.25	2.53	3.17
2-hr	1.04	1.47	1.76	2.15	2.45	2.76	3.46
3-hr	1.10	1.55	1.85	2.27	2.59	2.91	3.65
6-hr	1.20	1.70	2.03	2.49	2.85	3.20	4.01
12-hr	1.30	1.85	2.22	2.72	3.11	3.50	4.40
24-hr	1.40	2.00	2.40	2.95	3.38	3.80	4.78

*D-D-F data obtained from PREFRE Program

Localized Intensity-Depth-Frequency (I-D-F)

Duration	Frequency						
(minutes)	2-Yr	5-Yr	10-Yr	25-Yr	50-Yr	100-Yr	500-Yr
5	3.96	5.16	6.00	7.20	8.16	9.12	11.28
10	2.94	3.90	4.62	5.52	6.24	6.96	8.64
15	2.36	3.28	3.88	4.68	5.32	5.96	7.44
30	1.58	2.18	2.60	3.16	3.60	4.04	5.06
60	1.00	1.35	1.61	1.97	2.25	2.53	3.17

Table 2-8: Rainfall data for Eloy

PREFRE Program Input Data

2-yr, 6-hr =	1.40
2-yr, 24-hr =	1.60
100-yr, 6-hr =	3.40
100-yr, 24-hr =	4.40

Depth-Duration-Frequency (D-D-F)

	Frequency						
Duration	2-Yr	5-Yr	10-Yr	25-Yr	50-Yr	100-Yr	500-Yr
5 min	0.39	0.47	0.53	0.61	0.68	0.74	0.90
10 min	0.58	0.71	0.80	0.93	1.04	1.14	1.38
15 min	0.71	0.88	1.01	1.19	1.32	1.46	1.78
30 min	0.94	1.18	1.36	1.60	1.79	1.98	2.42
1-hr	1.14	1.46	1.68	1.99	2.24	2.48	3.04
2-hr	1.23	1.61	1.87	2.23	2.51	2.79	3.44
3-hr	1.29	1.70	1.99	2.39	2.70	3.00	3.71
6-hr	1.40	1.89	2.22	2.68	3.04	3.40	4.22
12-hr	1.50	2.10	2.50	3.05	3.48	3.90	4.88
24-hr	1.60	2.30	2.77	3.41	3.91	4.40	5.54

*D-D-F data obtained from PREFRE Program

Localized Intensity-Depth-Frequency (I-D-F)

Duration	Frequency						
(minutes)	2-Yr	5-Yr	10-Yr	25-Yr	50-Yr	100-Yr	500-Yr
5	4.68	5.64	6.36	7.32	8.16	8.88	10.80
10	3.48	4.26	4.80	5.58	6.24	6.84	8.28
15	2.84	3.52	4.04	4.76	5.28	5.84	7.12
30	1.88	2.36	2.72	3.20	3.58	3.96	4.84
60	1.14	1.46	1.68	1.99	2.24	2.48	3.04

Table 2-9: Rainfall data for Florence

PREFRE Program Input Data

2-yr, 6-hr =	1.40
2-yr, 24-hr =	1.60
100-yr, 6-hr =	3.20
100-yr, 24-hr =	4.00

Depth-Duration-Frequency (D-D-F)

	Frequency						
Duration	2-Yr	5-Yr	10-Yr	25-Yr	50-Yr	100-Yr	500-Yr
5 min	0.39	0.46	0.52	0.60	0.66	0.73	0.87
10 min	0.58	0.70	0.79	0.92	1.02	1.12	1.35
15 min	0.71	0.88	1.00	1.17	1.30	1.43	1.74
30 min	0.94	1.17	1.34	1.57	1.76	1.94	2.37
1-hr	1.14	1.45	1.66	1.96	2.19	2.43	2.97
2-hr	1.23	1.58	1.82	2.16	2.43	2.69	3.30
3-hr	1.29	1.67	1.93	2.30	2.58	2.87	3.52
6-hr	1.40	1.83	2.13	2.55	2.88	3.20	3.95
12-hr	1.50	2.01	2.36	2.85	3.23	3.60	4.47
24-hr	1.60	2.19	2.59	3.14	3.57	4.00	4.98

*D-D-F data obtained from PREFRE Program

Localized Intensity-Depth-Frequency (I-D-F)

Duration	Frequency						
(minutes)	2-Yr	5-Yr	10-Yr	25-Yr	50-Yr	100-Yr	500-Yr
5	4.68	5.52	6.24	7.20	7.92	8.76	10.44
10	3.48	4.20	4.74	5.52	6.12	6.72	8.10
15	2.84	3.52	4.00	4.68	5.20	5.72	6.96
30	1.88	2.34	2.68	3.14	3.52	3.88	4.74
60	1.14	1.45	1.66	1.96	2.19	2.43	2.97

Table 2-10: Rainfall data for Kearny

PREFRE Program Input Data

2-yr, 6-hr =	1.60
2-yr, 24-hr =	2.00
100-yr, 6-hr =	3.60
100-yr, 24-hr =	4.80

Depth-Duration-Frequency (D-D-F)

	Frequency						
Duration	2-Yr	5-Yr	10-Yr	25-Yr	50-Yr	100-Yr	500-Yr
5 min	0.41	0.49	0.54	0.63	0.69	0.76	0.91
10 min	0.61	0.74	0.83	0.96	1.06	1.16	1.40
15 min	0.74	0.92	1.04	1.22	1.36	1.49	1.81
30 min	0.98	1.22	1.40	1.64	1.83	2.03	2.47
1-hr	1.19	1.51	1.73	2.04	2.29	2.53	3.09
2-hr	1.33	1.70	1.96	2.33	2.61	2.90	3.55
3-hr	1.43	1.83	2.12	2.52	2.83	3.14	3.86
6-hr	1.60	2.08	2.41	2.88	3.24	3.60	4.43
12-hr	1.80	2.38	2.78	3.34	3.77	4.20	5.13
24-hr	2.00	2.68	3.15	3.80	4.30	4.80	5.95

*D-D-F data obtained from PREFRE Program

Localized Intensity-Depth-Frequency (I-D-F)

Duration	Frequency						
(minutes)	2-Yr	5-Yr	10-Yr	25-Yr	50-Yr	100-Yr	500-Yr
5	4.92	5.88	6.48	7.56	8.28	9.12	10.92
10	3.66	4.44	4.98	5.76	6.36	6.96	8.40
15	2.96	3.68	4.16	4.88	5.44	5.96	7.24
30	1.96	2.44	2.80	3.28	3.66	4.06	4.94
60	1.19	1.51	1.73	2.04	2.29	2.53	3.09

Table 2-11: Rainfall data for Mammoth

PREFRE Program Input Data

2-yr, 6-hr =	1.50
2-yr, 24-hr =	1.80
100-yr, 6-hr =	3.40
100-yr, 24-hr =	4.20

Depth-Duration-Frequency (D-D-F)

	Frequency						
Duration	2-Yr	5-Yr	10-Yr	25-Yr	50-Yr	100-Yr	500-Yr
5 min	0.40	0.48	0.54	0.63	0.70	0.77	0.93
10 min	0.59	0.73	0.83	0.97	1.07	1.18	1.43
15 min	0.72	0.91	1.04	1.23	1.37	1.52	1.85
30 min	0.96	1.22	1.40	1.66	1.86	2.06	2.52
1-hr	1.17	1.50	1.73	2.06	2.32	2.57	3.16
2-hr	1.28	1.66	1.92	2.28	2.57	2.85	3.51
3-hr	1.36	1.76	2.04	2.43	2.74	3.04	3.75
6-hr	1.50	1.96	2.27	2.71	3.06	3.40	4.19
12-hr	1.65	2.17	2.53	3.03	3.41	3.80	4.69
24-hr	1.80	2.38	2.78	3.34	3.77	4.20	5.19

*D-D-F data obtained from PREFRE Program

Localized Intensity-Depth-Frequency (I-D-F)

Duration	Frequency						
(minutes)	2-Yr	5-Yr	10-Yr	25-Yr	50-Yr	100-Yr	500-Yr
5	4.80	5.76	6.48	7.56	8.40	9.24	11.16
10	3.54	4.38	4.98	5.82	6.42	7.08	8.58
15	2.88	3.64	4.16	4.92	5.48	6.08	7.40
30	1.92	2.44	2.80	3.32	3.72	4.12	5.04
60	1.17	1.50	1.73	2.06	2.32	2.57	3.16

Table 2-12: Rainfall data for Superior

PREFRE Program Input Data

2-yr, 6-hr =	1.80
2-yr, 24-hr =	2.60
100-yr, 6-hr =	4.00
100-yr, 24-hr =	6.00

Depth-Duration-Frequency (D-D-F)

	Frequency						
Duration	2-Yr	5-Yr	10-Yr	25-Yr	50-Yr	100-Yr	500-Yr
5 min	0.40	0.48	0.53	0.62	0.69	0.75	0.91
10 min	0.59	0.72	0.81	0.95	1.05	1.15	1.39
15 min	0.72	0.90	1.02	1.20	1.34	1.48	1.80
30 min	0.95	1.20	1.37	1.62	1.81	2.01	2.45
1-hr	1.16	1.48	1.70	2.02	2.26	2.51	3.07
2-hr	1.38	1.77	2.04	2.42	2.72	3.02	3.70
3-hr	1.53	1.96	2.27	2.69	3.03	3.36	4.12
6-hr	1.80	2.33	2.69	3.20	3.60	4.00	4.92
12-hr	2.20	2.87	3.34	3.99	4.50	5.00	6.16
24-hr	2.60	3.42	3.99	4.78	5.39	6.00	7.41

*D-D-F data obtained from PREFRE Program

Localized Intensity-Depth-Frequency (I-D-F)

Duration	Frequency						
(minutes)	2-Yr	5-Yr	10-Yr	25-Yr	50-Yr	100-Yr	500-Yr
5	4.80	5.76	6.36	7.44	8.28	9.00	10.92
10	3.54	4.32	4.86	5.70	6.30	6.90	8.34
15	2.88	3.60	4.08	4.80	5.36	5.92	7.20
30	1.90	2.40	2.74	3.24	3.62	4.02	4.90
60	1.16	1.48	1.70	2.02	2.26	2.51	3.07

2.4 REFERENCES

The material presented in this chapter was obtained or adapted from:

- Arizona Department of Transportation, Highway Drainage Design Manual - Hydrology
- Federal Highway Administration, Hydraulic Engineering Circular No. 22, Urban Drainage Design Manual
- Flood Control District of Maricopa County, Drainage Design Manual for Maricopa County, Arizona – Hydrology
- Flood Control District of Maricopa County, Drainage Design Manual for Maricopa County, Arizona – Hydraulics
- U.S. Department of Transportation, Federal Highway Administration, Hydraulic Engineering Circular No. 12, Drainage of Highway Pavements

VOLUME 2
DESIGN METHODOLOGY AND PROCREDURES

Chapter 3: Street Drainage

3.1 INTRODUCTION

Removal of stormwater from roadways during storm events helps to maintain a safe and efficient driving condition. Removal of stormwater from roads helps to reduce maintenance costs as well. This chapter provides guidelines and procedures for the handling and removal of stormwater flow from streets and roadways, and describes methodology that should be used for the estimation of street flow capacity, allowable spread, and catch basin design.

3.2 PROCEDURE

Design procedures for street drainage on a continuous grade are as follows:

1. For a given longitudinal street slope and cross slope at a location determine the flow rate that would provide a flow spread that is equal to the allowable spread.

2. Determine if the drainage area draining to the location used in Step 1 will generate the discharge determined in Step 1. If not choose a different location for Step 1. Continue the iterative process until the drainage area flow rate is consistent with the allowable spread flow rate.

3. Determine if there are conflicts with the placement of a catch basin at this location. Conflicts could be but are not limited to, side streets, driveways, utilities that would be costly to relocate, etc. Should there be conflicts, move the catch basin location upstream.

4. Size a catch basin to intercept the calculated flow. Determine the efficiency of the catch basin and determine the flow rate, if any, that will by pass the catch basin.

5. Choose a location downstream in which the drainage area contributing to the location will generate a flow rate that when added to the by pass flow rate determined in Step 4 is equal to the flow rate that would generate a spread that is equal to the allowable spread.

6. Continue steps 3 through 5 to termination of the project.

3.3 APPLICATIONS

3.3.1 Street Capacity

Manning's equation as expressed in Equation (3.1) shall be used for estimating the total capacity of a roadway (curb to curb or sidewalk to sidewalk).

$$Q = A\left(\frac{1.49}{n}\right)R^{0.67}S^{0.5} \tag{3.1}$$

Where:

Q = Total flow, cfs
n = Manning's roughness coefficient. An n-value of 0.015 or 0.016 is typically used for paved streets unless special conditions exist.
A = Flow area, sq ft
R = Hydraulic radius, ft
S = Slope of energy grade line, assumed equal to longitudinal street slope, ft/ft

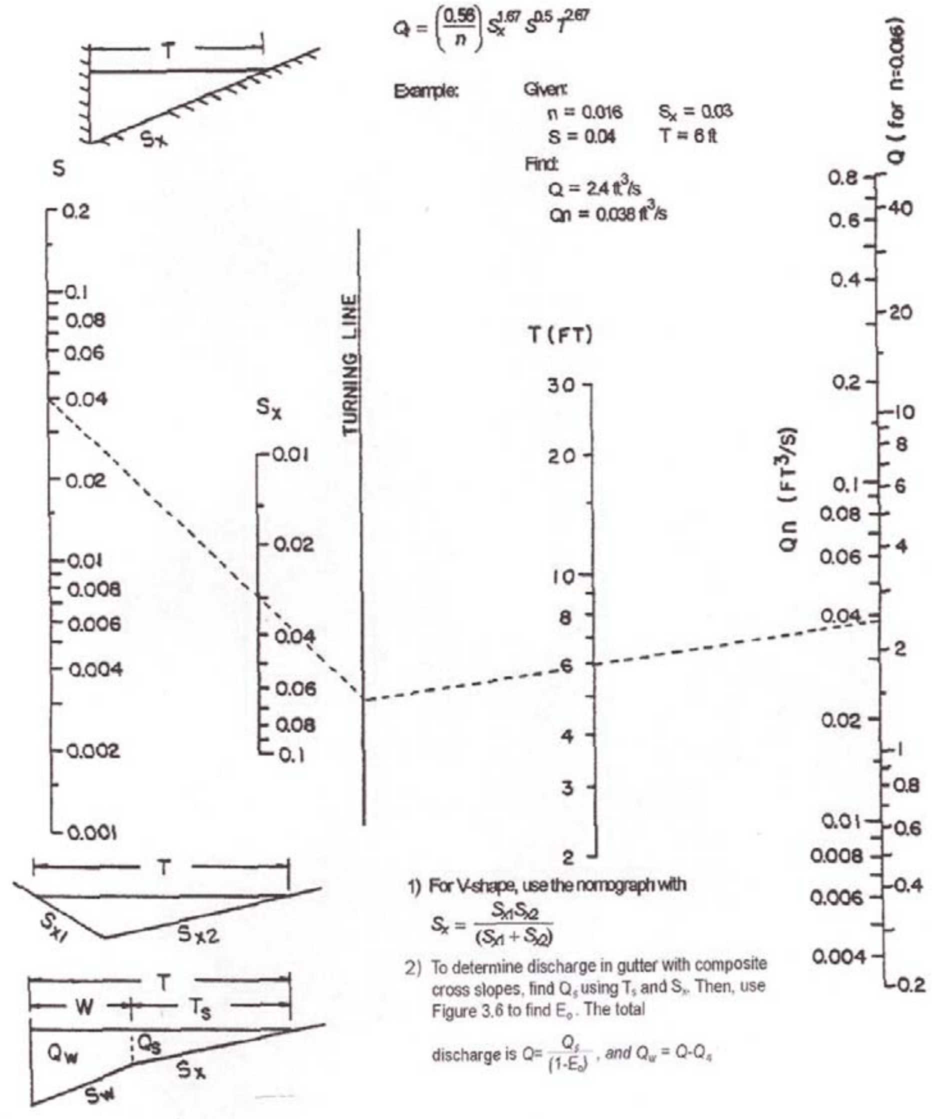
The theoretical gutter carrying capacity based on allowable pavement spread shall be computed using the modified Manning's formula as expressed in Equation (3.2) or shown on Figure 3-1.

$$Q_t = \left(\frac{0.56}{n}\right)S_x^{1.67}S^{0.5}T^{2.67} \tag{3.2}$$

Where:

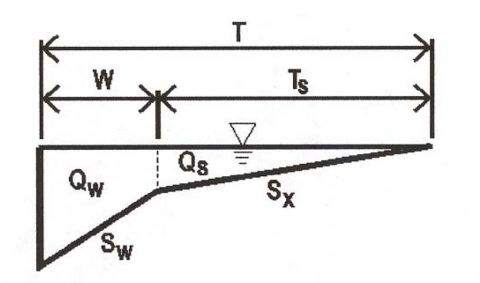
Q_t = Theoretical gutter carrying capacity, cfs
T = Spread of flow on pavement, ft
S_x = Pavement cross slope, ft/ft
S = Longitudinal slope, ft/ft

Figure 3.1: Nomograph for Triangular Gutters



Pavement spread for gutters with composite cross-slopes is determined using the relationships presented in Figure 3.2.

Figure 3.2: Composite Cross-slope Gutter Section



A multi-step analysis is required to determine discharge in a gutter with a composite cross-slope. First, find Q_s using Equation (3.3). Next, determine the ratio of flow in the depressed section to total gutter flow using Equation (3.4). Then, find the total gutter flow (Q) using Equation (3.5) or Figure 3.3. Gutter flow (Q_w) can then be determined using Equation (3.6).

$$Qs = \left(\frac{0.56}{n}\right) S_x^{\ 1.67} S^{0.5} T_s^{\ 2.67} \tag{3.3}$$

Where:

Q_s = Flow rate in paved area, cfs
T_s = Spread of flow on pavement for a composite section, ft
S = Longitudinal slope, ft/ft
S_x = Pavement cross-slope, ft/ft

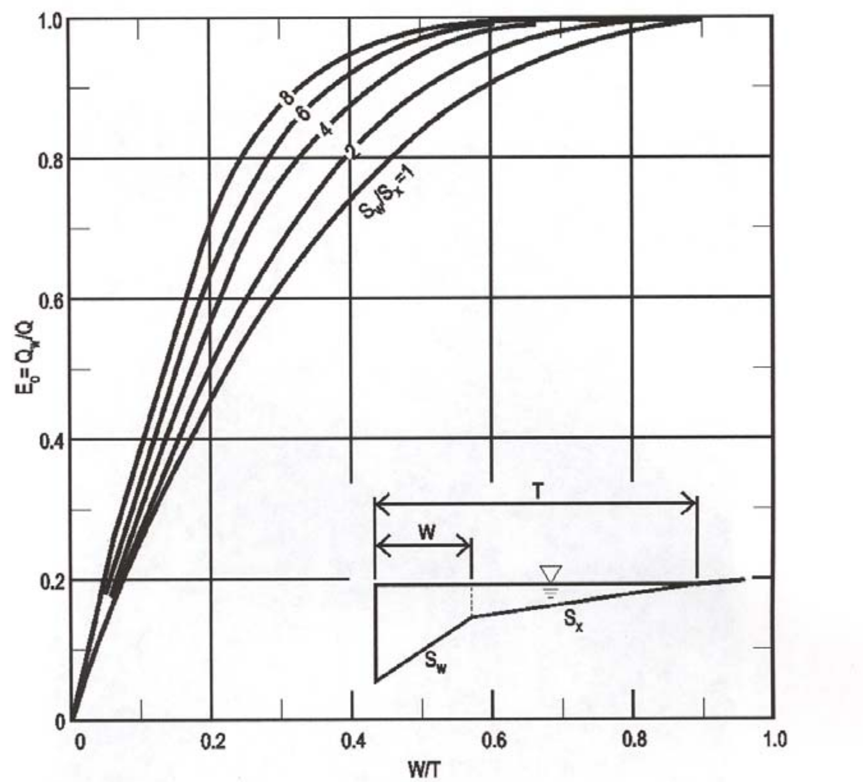
$$E_o = 1 / \left[1 + \frac{S_w / S_x}{\left(1 + \frac{S_w / S_x}{\left(\frac{T}{W}\right) - 1}\right)^{2.67} - 1} \right] \tag{3.4}$$

Where:

E_o = Ratio of flow in the depressed section to total gutter flow
S_x = Pavement cross-slope, ft/ft
W = Width of gutter, ft
T = Width of flow spread, ft
S_w = Cross-slope of a depressed gutter $\left(Sx + \frac{gutter\,depression}{W}\right)$, ft/ft

(Eqn. 3.4 Ref: US DOT, FHWA, 1996, HEC-22, Eqn 4-4)

Figure 3.3: Ratio of Frontal Flow to Total Gutter Flow



$$Q = \frac{Q_s}{(1 - Eo)} \tag{3.5}$$

$$Q_w = Q - Q_s \tag{3.6}$$

Where:

Q_w = Flow rate in depressed section of gutter, cfs
Q_s = Flow rate in paved area, cfs
Q = Total gutter flow rate, cfs

3.3.2 Catch Basins

3.3.2.1 Catch Basin Selection

Catch basins used for drainage can be divided into four main categories, curb-opening catch basins, grated catch basins, combination catch basins, and slotted drain catch basins. Typical catch basin inlets are shown in Figure 3.4.

Catch basins may be further classified as being on a continuous grade or in a sump. The continuous grade condition exists where the street grade is continuous past the catch basin and the water can flow past. The sump condition exists where water is restricted to the catch basin area because the catch basin is located at a low point. This may be due to a change in grade of the street from positive to negative or due to the crown slope of a cross street where the catch basin is located at an intersection.

Curb-opening catch basins

Curb-opening catch basins are effective in the drainage of roadways, 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, individuals using mechanical aids for commuting, or bicyclists. A depressed curb opening is hydraulically more efficient than an undepressed curb opening.

Grated or gutter catch basins

Grated or gutter catch basins refer to an opening in the gutter covered by one or more grates through which water falls. As with other catch basins, grated catch basins may be depressed or undepressed. Grated catch basins are more efficient than curb-opening catch basins where located on a continuous grade. Where grated catch basins are used, the engineer should design them to optimize hydraulic efficiency, bicycle and pedestrian safety, and structural adequacy. Grated catch basins should not extend into traffic lanes.

Combination catch basin

A combination catch basin consists of a curb-opening and grate placed side by side. The interception capacity of a combination catch basin on a continuous grade is not appreciably greater than that of the grate alone, so the grate alone should be used when computing the interception capacity for this situation. A curb opening longer than the grate does provide additional capacity for a combination catch basin, however. The curb opening in such an installation also intercepts debris which might otherwise clog the grate and is termed a "sweeper". A combination inlet with a curb opening upstream of the grate has an interception capacity equal to the sum of the two inlets, except that the frontal flow reaching the grate, and thus what it will intercept, is reduced by interception by the curb opening.

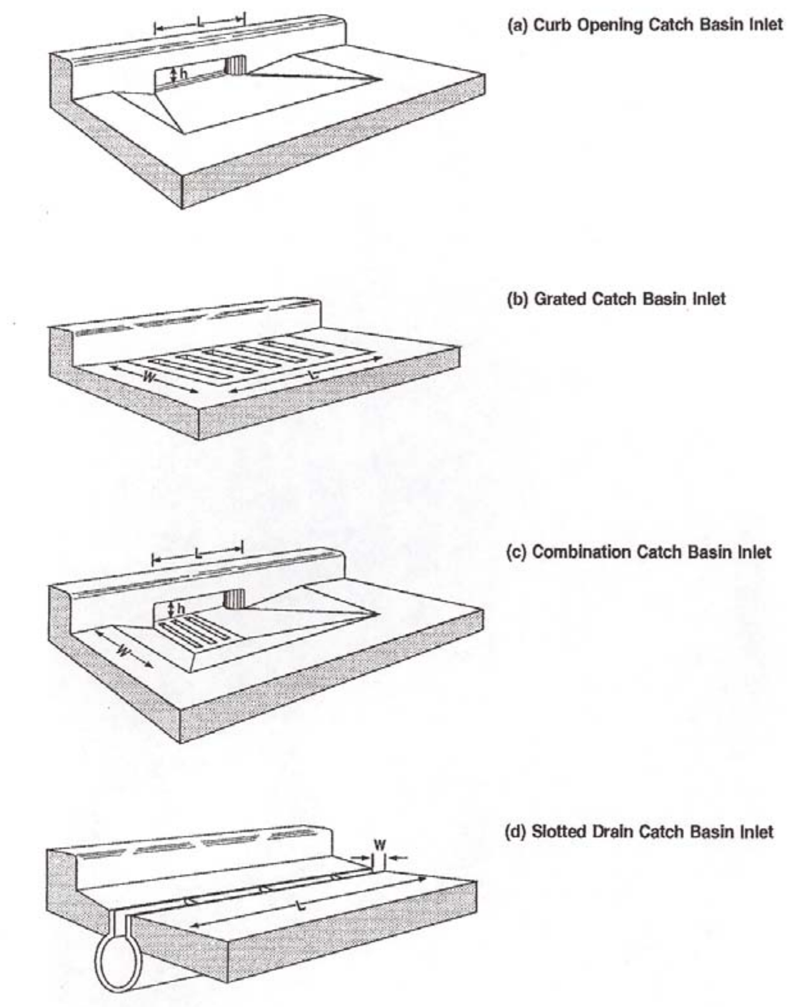
Combination inlets are very desirable in a sump because the curb opening provides a relief if the grate should become clogged.

Slotted drains

A slotted drain is a slotted opening in the pavement which intercepts sheet flow and conveys it into a pipe (normally corrugated steel) to which the slotted drain is attached. Slotted drains are most effective where street slopes are shallow.

Slotted drains can be used on curbed or uncurbed sections and offer little interference to traffic operations.

Figure 3. 4: Catch Basin Inlets



3.3.2.2 Curb-Opening Catch Basins

On-Grade

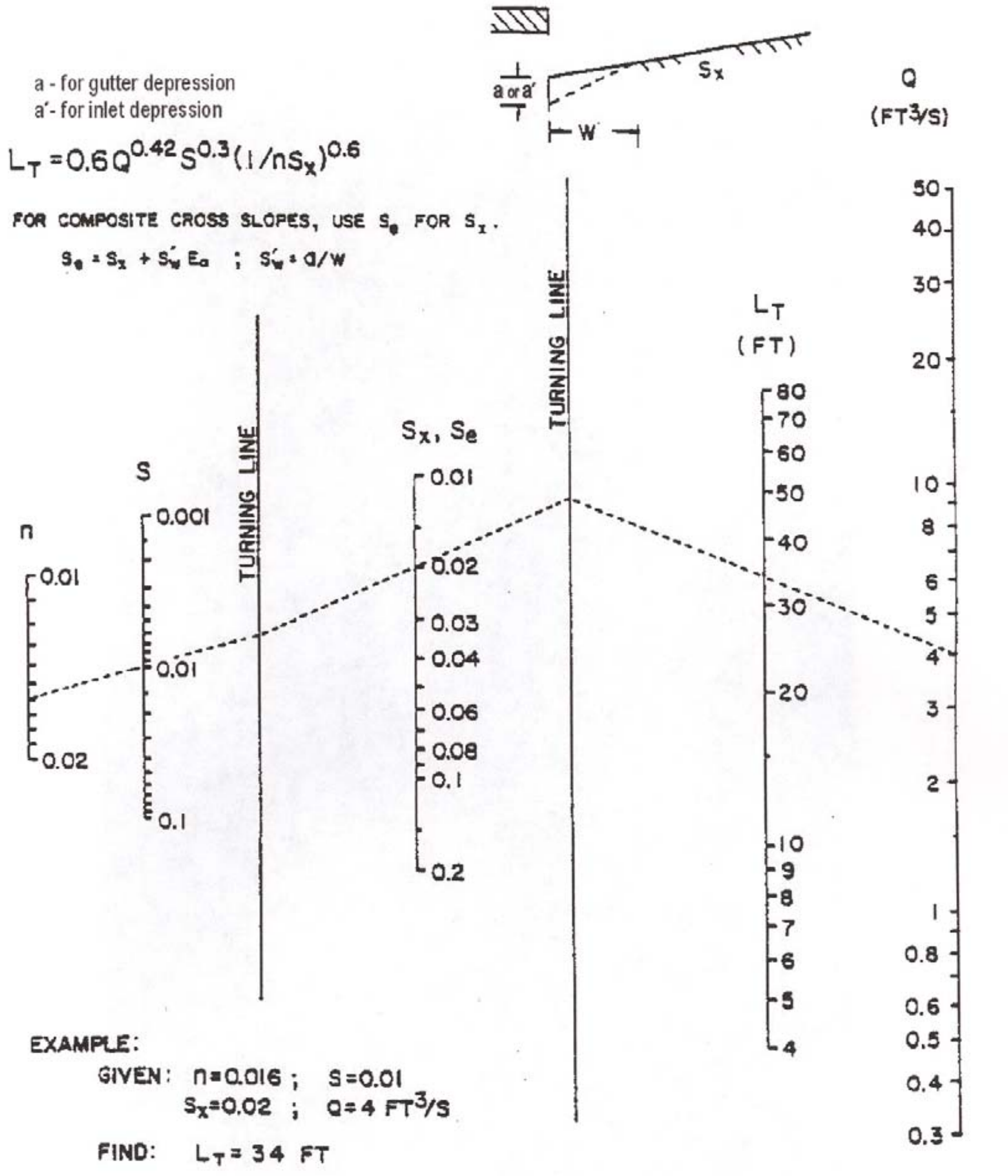
The length (L_r) of curb opening catch basin required for total interception of gutter flow on a pavement section with a straight cross slope can be determined using Equation (3.7) or Figure 3.5:

$$L_t = 0.6Q^{0.42}S^{0.3}\left(\frac{1}{nS_x}\right)^{0.6} \qquad (3.7)$$

Where:

Q = Total gutter flow rate, cfs
S = Longitudinal slope, ft/ft
S_x = Pavement cross-slope, ft/ft
n = Manning's roughness coefficient

Figure 3. 5: Curb Opening and Slotted Drain Inlet Length for Total Interception



The efficiency (E) of curb-opening catch basins shorter than the length required for total interception is computed using Equation (3.8):

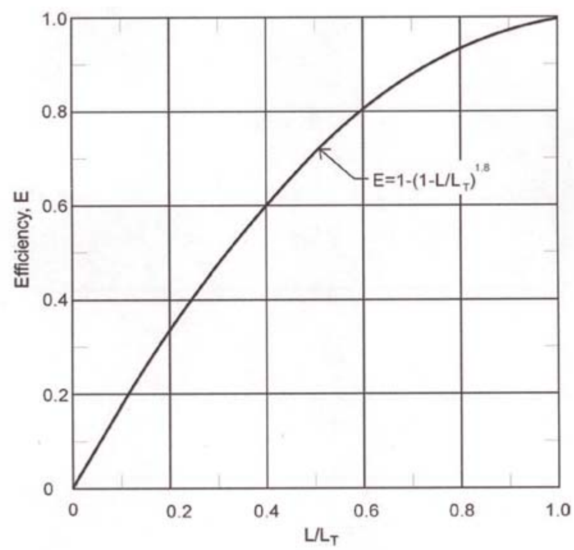
$$E = 1 - \left(1 - \frac{L}{L_t}\right)^{1.8} \quad (3.8)$$

Where:

L = Length of curb opening, grate or slot, ft
L_t = Curb opening length required to intercept 100% of the gutter flow, ft

Figure 3-6 provides a solution of Equation (3.8) and the equation is applicable with either straight cross slopes or compound cross slopes.

Figure 3. 6: Curb Opening and Slotted Drain Inlet Interception Efficiency



The length of catch basin required for total interception by depressed curb-opening catch basins or curb openings in depressed gutter sections can be found by using an equivalent cross slope, S_e. S_e can be calculated using Equation (3.9).

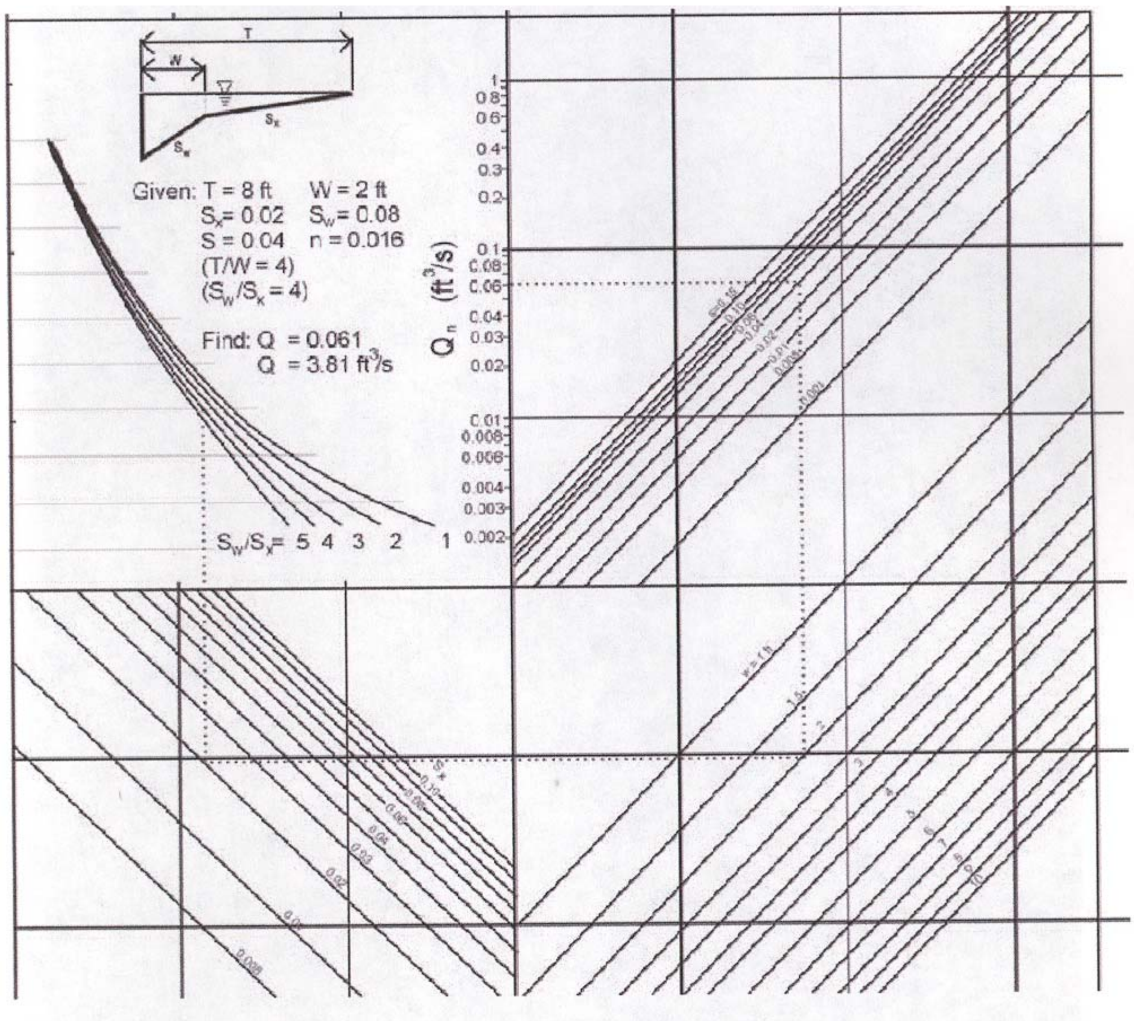
$$S_e = S_x + S'_w E_o \quad (3.9)$$

Where:

S'_w = Cross slope of the gutter (at the inlet) measured from the cross slope of the pavement, ft/ft ($S'_w = a/12W$; see Figure 3.7)
E_o = Ratio of flow in the depressed section to total gutter flow
S_x = Pavement cross-slope, ft/ft

E_o is the ratio of flow in the depressed section to the total gutter flow, and S'_w is the cross slope of the gutter measured from the cross slope of the pavement, S_x. Figure 3.7 can be used to determine the spread, and then Figure 3-3 can be used to determine E_o.

Figure 3. 7: Flow in Composite Gutter Sections



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.

Using the equivalent cross slope. S_e Equation (3.7) becomes Equation (3.10):

$$L_t = 0.6Q^{0.42}S^{0.3}\left(\frac{1}{nS_e}\right)^{0.6} \quad (3.10)$$

Figures 3.5 and 3.6 are applicable to depressed curb-opening catch basins using S_e as well as S'_x.

Sumps

The capacity of a curb-opening catch basin in a sump depends on water depth at the curb, the curb opening length, and the height of the curb opening. The catch basin operates as a weir for depths of water up to the curb-opening height and as an orifice at depths greater than 1.4 times the opening height. Flow is in a transition stage at water depths between 1.0 and 1.4 times the opening height.

The weir location for a depressed curb-opening catch basin is at the edge of the gutter, and the effective weir length is dependent on the width of the depressed gutter and the length of the curb opening. The weir location for a curb opening catch basin that is not depressed is at the lip of the curb-opening, and its length is equal to that of the curb-opening catch basin.

The equation for the interception capacity of a depressed curb opening-catch basin operating as a weir is as shown in Equation (3.11):

$$Q_i = C_w (L + 1.8W) d^{1.5} \tag{3.11}$$

Where:

Q_i = Amount of street flow intercepted by inlet, cfs
C_w = Weir coefficient = 2.3
W = Width of grate or depressed gutter, ft
d = Depth of flow, ft (measure from water surface to projected cross slope)
L = Length of curb opening or slot, ft

The weir equation is applicable to depths at the curb approximately equal to the height of the opening plus the depth of the depression. Thus, the limitation on the use of Equation (3.11) for a depressed curb opening catch basin is shown in Equation (3.12):

$$d < h + \frac{a'}{12} \tag{3.12}$$

Where:

h = Height of curb opening catch basin, curb opening orifice, or orifice throat width, ft
a = Gutter depression, inches

Experiments have not been conducted for curb opening catch basins with a continuously depressed gutter, but it is reasonable to expect that the effective weir length would be as great as that for a catch basin in a local depression. Use of Equation (3.11) will yield conservative estimates of the interception capacity.

The weir equation for curb opening catch basins without depression (W = 0) becomes:

$$Q_i = C_w L d^{1.5} \tag{3.13}$$

Where:

C_w = 3.0
d = Depth of flow, ft
L = Length of curb opening or slot, ft

The depth limitation for operation as a weir becomes: $d \le h$.

Curb opening catch basins operate as orifices at depths greater than approximately 1.4*h*. The interception capacity can be computed by Equation (3.14):

$$Q_i = C_o h L (2 g d_o)^{0.5} \tag{3.14}$$

Where:

C_o = Orifice coefficient = 0.67
g = Acceleration due to gravity, 32.2 ft/sec^2
d_o = Effective depth at the center of the curb opening orifice, ft
h = Height of curb opening catch basin, curb-opening orifice, or orifice throat, ft
L = Length of curb opening, ft

Equation (3.14) applies to depressed and undepressed curb opening catch basins and the depth at the catch basin includes any gutter depression.

Height of the orifice in Equation (3.14) assumes a vertical orifice opening. As illustrated in Figure 3.10, other orifice throat locations can change the effective depth on the orifice and the dimension $\left(d_i - h/2\right)$. A limited throat width could reduce the capacity of the curb-opening catch basin by causing the catch basin to go into orifice flow at depths less than the height of the opening.

Figure 3. 8: Curb Opening Catch Basin Inlets

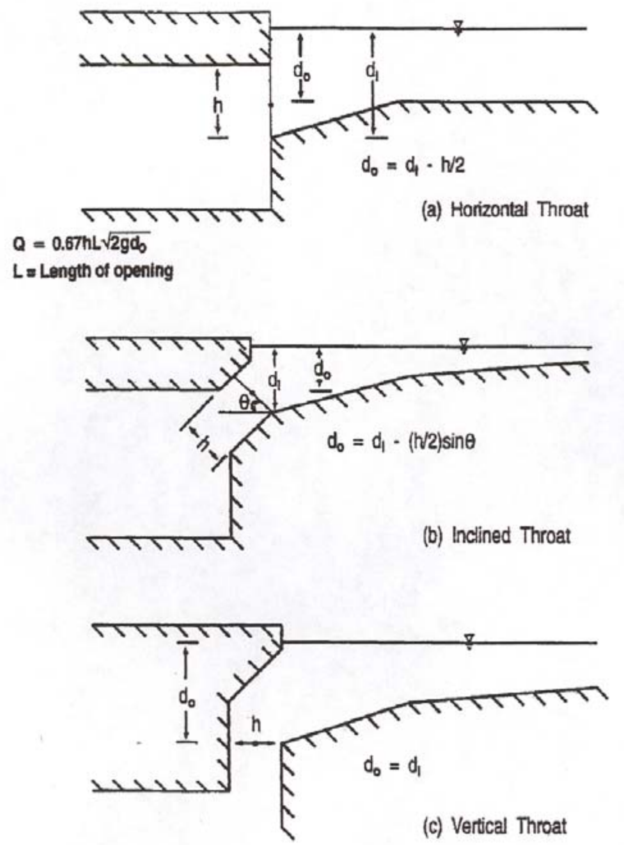


Figure 3.11 provides solutions for Equations (3.11) and (3.14) for depressed curb-opening catch basins, and Figure 3.10 provides solutions for Equations (3.13) and (3.14) for curb-opening catch basins without depression. Figure 3.11 is provided for use for curb openings with inclined or vertical orifice throats.

Figure 3. 9: Depressed Curb Opening Inlet Capacity in Sump Locations

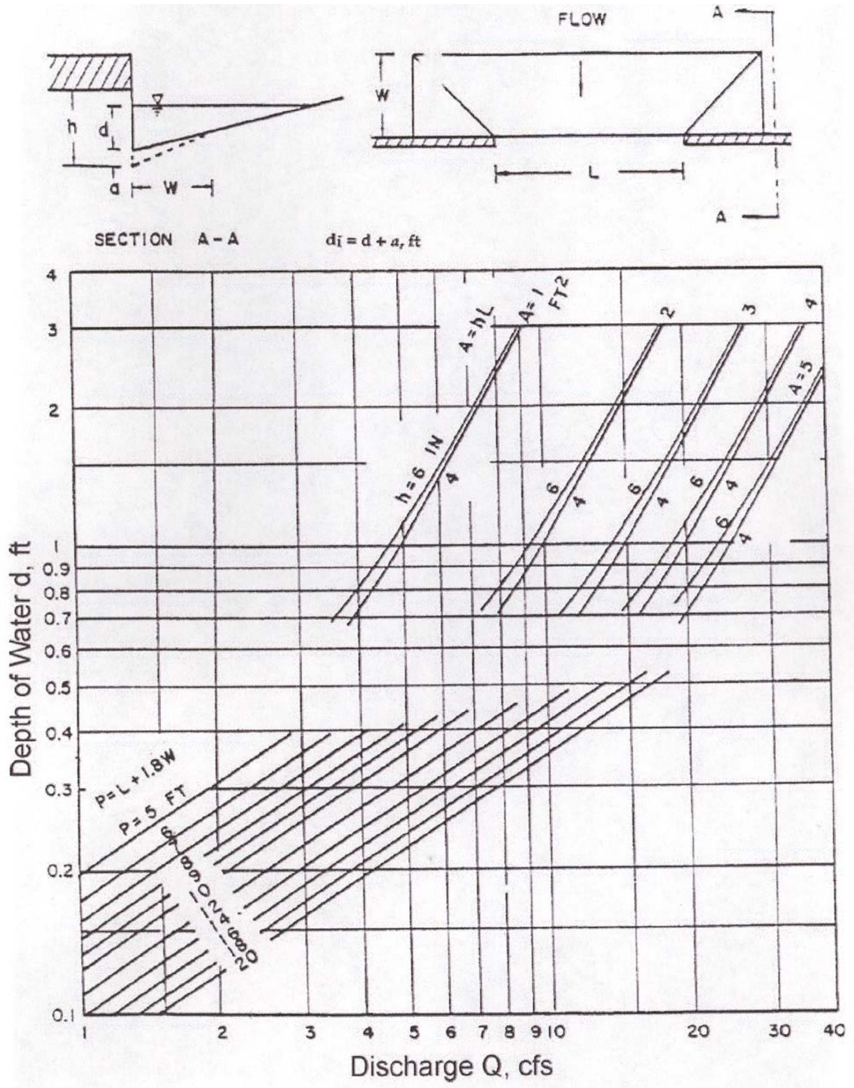


Figure 3. 10: Curb Opening Inlet Capacity in Sump Locations

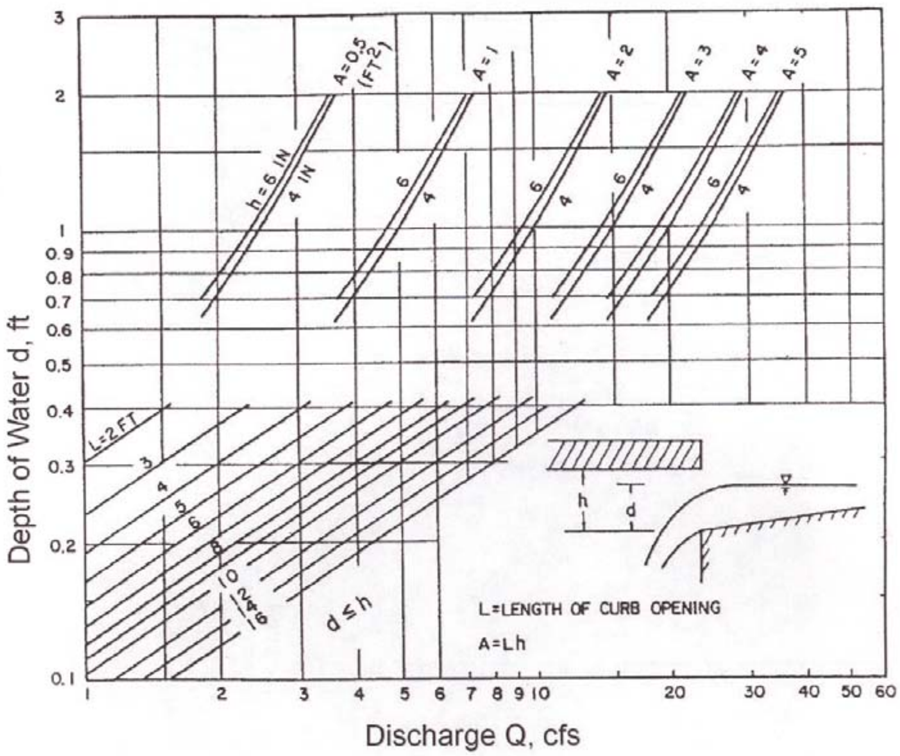
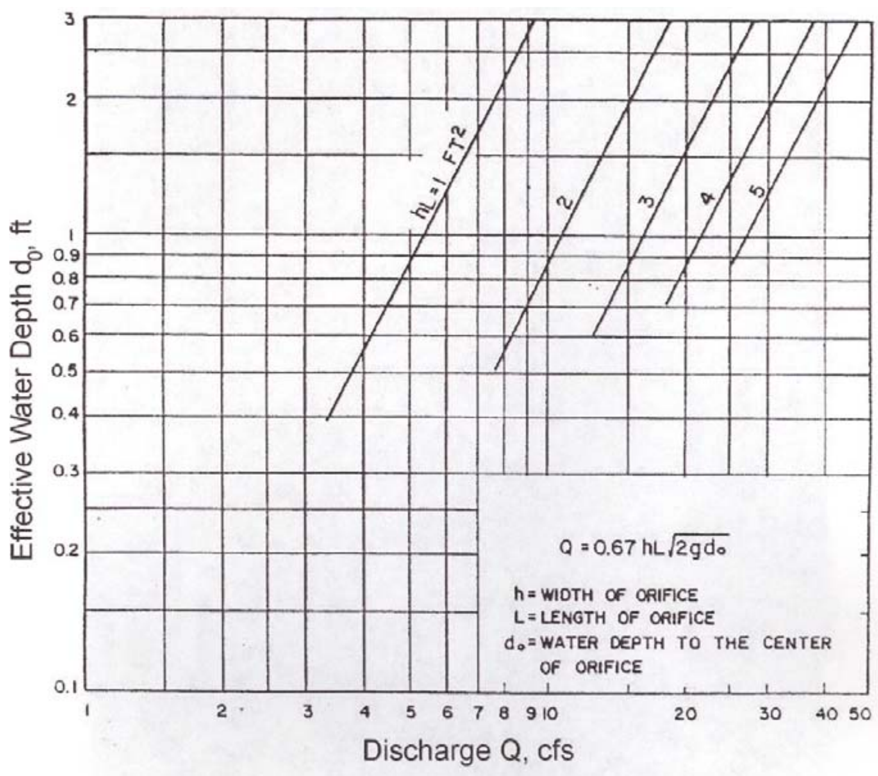


Figure 3. 11: Curb Opening Inlet Capacity for Inclined and Vertical Orifice Throats



3.3.2.3 Grated Catch Basins

On-Grade

Grated catch basins intercept all of the frontal flow until splash over (the velocity at which water begins to splash over the grate) is reached. At velocities greater than splash over, grate efficiency in intercepting frontal flow is diminished. Grates also intercept a portion of the flow along the length of the grate, or the side flow, dependent on the cross slope of the pavement, the length of the grate, and flow velocity.

The ratio of frontal flow to total gutter flow, E_o for a straight cross slope is:

$$E_o = \frac{Q_w}{Q} = 1 - \left(1 - \frac{W}{T}\right)^{2.67} \tag{3.15}$$

Where:

Q_w = Flow rate in width (W), cfs
Q = Total flow, cfs
W = Width of grate or gutter, ft
T = Spread of flow on the pavement, ft

Figure 3.3 provides a graphical solution of E_o for either straight cross slopes or depressed gutter sections.

The ratio of side flow, (Q_s) to total gutter flow (Q) is:

$$\frac{Q_s}{Q} = 1 - \frac{Q_w}{Q} = 1 - E_o \tag{3.16}$$

Where:

Q_s = Flow rate outside of width (W), cfs
Q_w = Flow rate in width of grate or gutter (W), cfs

The ratio of frontal flow intercepted to total frontal flow, R_f is expressed:

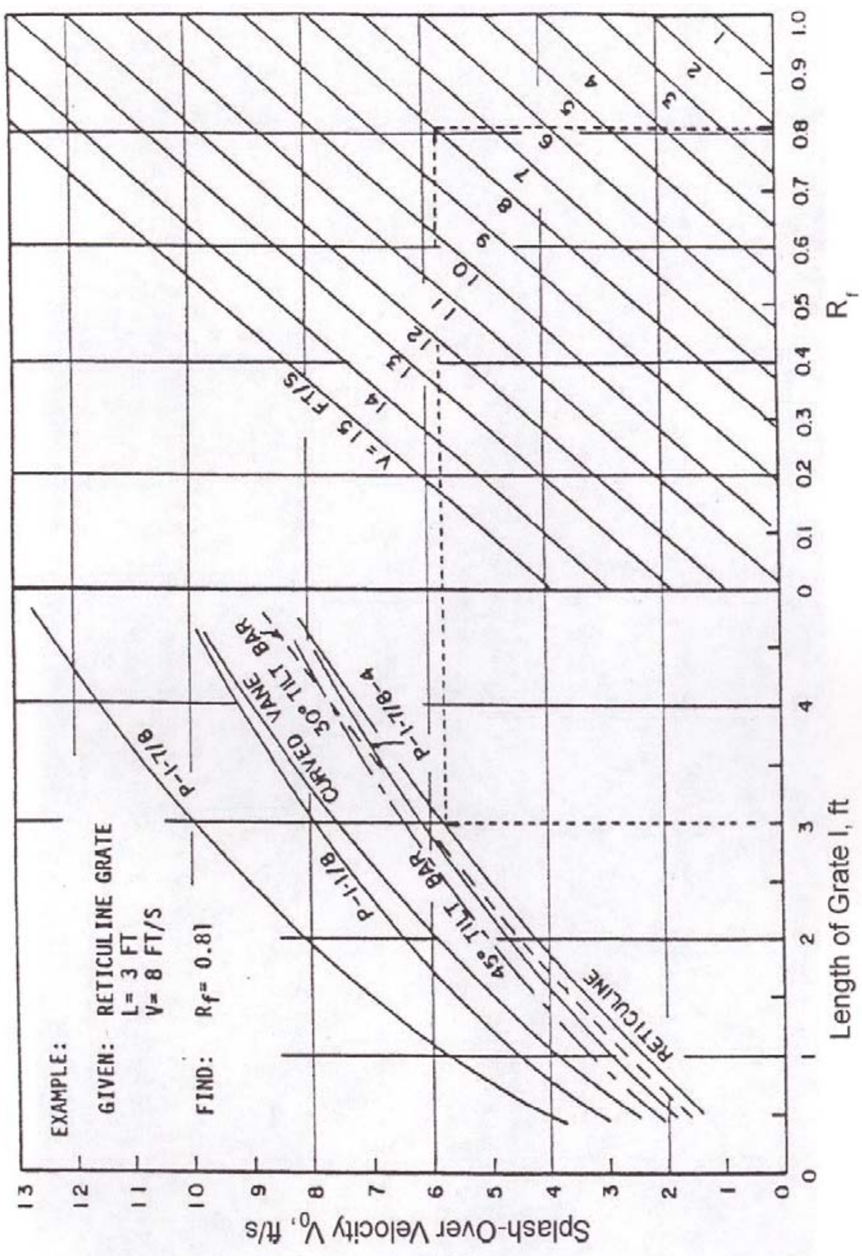
$$R_f = 1 - 0.09(V - V_o) \tag{3.17}$$

Where:

R_f = Ratio of frontal flow intercepted to total frontal flow
V = Velocity of flow in the gutter, ft/sec
V_o = Gutter velocity where splash over first occurs, ft/sec

This ratio is equivalent to frontal flow interception efficiency. Figure 3.12 provides a solution of Equation (3.17) which takes into account grate length, bar configuration and gutter velocity at which splash-over occurs. The gutter velocity needed to use Figure 3.12 is total gutter flow divided by the area of flow.

Figure 3. 12: Grate Inlet Flow Interception Efficiency



The ratio of side flow intercepted to total side flow, R_s, or side flow interception efficiency, is expressed:

$$R_s = \left[\frac{1}{\left(1 + \frac{0.15V^{1.8}}{S_x L^{2.3}}\right)} \right] \tag{3.18}$$

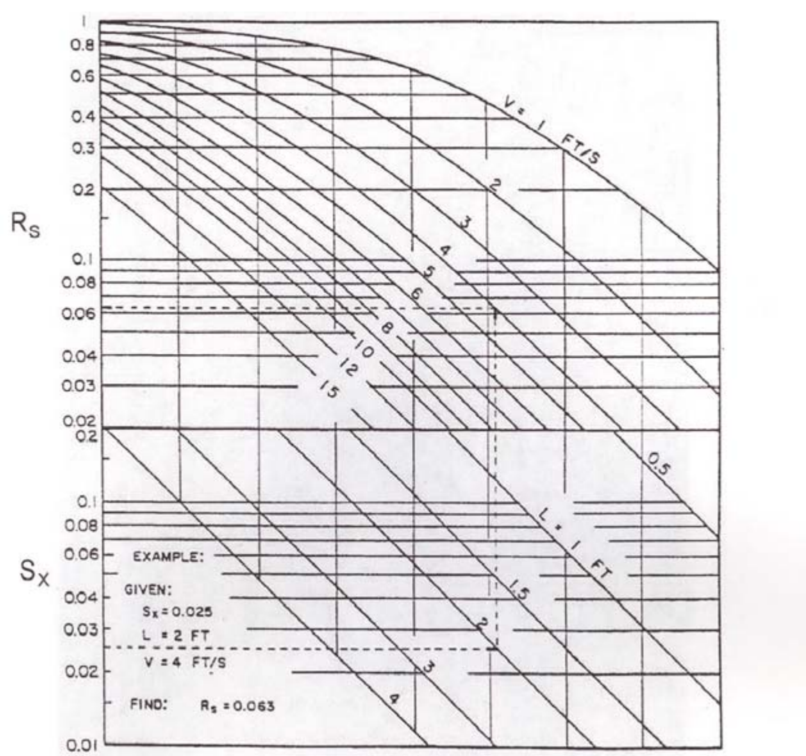
Where:

S_x = Pavement cross slope, ft/ft
L = Length of grate, ft
V = Velocity of flow in the gutter, ft/sec

Figure 3-13 provides a solution of Equation (3.18).

A deficiency in developing empirical equations and charts from experimental data is evident in Figure 3.13. The fact that a grate will intercept all or almost all of the side flow where the velocity is low and the spread only slightly exceeds the grate width is not reflected in the figure. Error due to this deficiency is very small. In fact, where velocities are high, side flow interception can be neglected entirely without significant error.

Figure 3. 13: Grate Inlet Side Flow Interception Efficiency



The efficiency, E, of a grate is:

$$E = R_f E_o + R_s (1 - E_o) \qquad (3.19)$$

The first term on the right side of Equation (3.19) is the ratio of intercepted frontal flow to total gutter flow, and the second term is the ratio of intercepted side flow to total side flow. The second term is insignificant with high velocities and short grates.

The interception capacity (Q_i) of a grate catch basin on grade is equal to the efficiency of the grate multiplied by the total gutter flow:

$$Q_i = EQ = Q\left[R_f E_o + R_s \left(1 - E_o\right)\right] \qquad (3.20)$$

Sumps

The efficiency of catch basins in passing debris is critical in sump locations because all runoff which enters the sump must be passed through the catch basin. Total or partial clogging of catch basins in these locations can result in hazardous ponding conditions. Grate catch basins alone are not recommended for use in sump locations because of the tendencies of grates to become clogged. Combination catch basins or curb-opening catch basins are recommended for use in these locations.

A grate catch basin in a sump location operates as a weir to depths dependent on the bar configuration and size of the grate and as an orifice at greater depths. Grates of larger dimension and grates with more open area, that is, with less space occupied by lateral and longitudinal bars, will operate as weirs to greater depths than smaller grates or grates with less open area.

The capacity of grate catch basins operating as weirs is:

$$Q_i = C_w P d^{1.5} \qquad (3.21)$$

Where:

C_w = Weir coefficient = 3.0
P = Perimeter of the grate, disregarding bars and side against curb, ft
d = Depth of flow at curb, ft

The capacity of a grate catch basin operating as an orifice is:

$$Q_i = C_o A_g (2gd)^{0.5} \qquad (3.22)$$

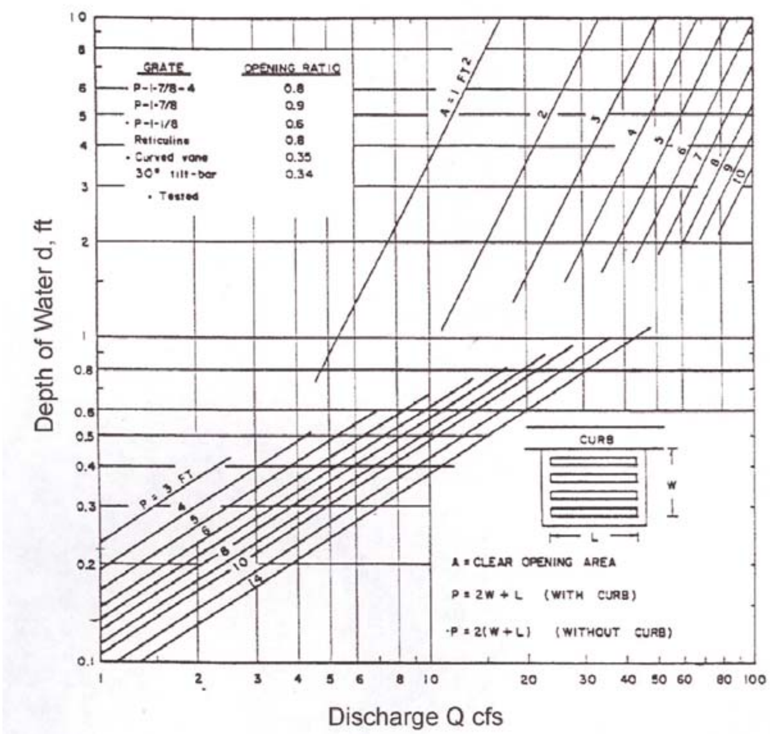
Where:

C_o = Orifice coefficient = 0.67
A_g = Clear opening area of the grate, sq ft
d = Depth of flow at curb, ft
g = Acceleration due to gravity, 32.2 ft/sec^2

Use of Equation (3.22) requires the clear opening area of the grate. Tests of three grates for the Federal Highway Administration showed that for flat bar grates, such as *P-1-7/8-4* and *P-1l-1/8* grates, the clear opening is equal to the total area of the grate less the area occupied by longitudinal and lateral bars.

Figure 3.14 is a plot of Equation (3.21) and Equation (3.22) for various grate sizes. The effects 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 an interception capacity that is less than that computed by either the weir or the orifice equation. This capacity can be approximated by drawing a curve between the lines representing the perimeter and net area of the grate to be used.

Figure 3. 14: Grate Inlet Capacity in Sump Conditions



3.3.2.4 Combination Catch Basins

On-Grade

The interception capacity of a combination catch basin consisting of a curb opening and grate placed side-by-side is not appreciably greater than that of the grate opening alone. Capacity is computed by neglecting the curb opening. A combination catch basin is sometimes used with the curb opening or part of the curb opening placed upstream of the grate. A combination catch basin with a curb opening extending upstream of the grate has an interception capacity equal to the sum of the grated catch basin and of the portion of the curb opening inlet upstream of the grate. The frontal flow, and thus the interception capacity of the grate, is reduced by the flow intercepted by the curb opening.

Sump

Combination catch basins consisting of a grate and a curb opening are considered advisable for use in sumps where hazardous ponding can occur. The interception capacity of the combination catch basin is essentially equal to that of a grate alone in weir flow unless the grate opening becomes clogged. In orifice flow, the capacity is equal to the capacity of the grate plus the capacity of the curb opening.

Equation (3.21) or Figure 3.16 can be used for weir flow in combination catch basins in sump locations. Assuming complete clogging of the grate, Equation (3.11), Equation (3.13), and Equation (3.14), or Figure 3.9, Figure 3.10 and Figure 3.11 for curb-opening catch basins are applicable.

Where depth at the curb is such that orifice flow occurs, the interception capacity of the catch basin is computed by adding Equation (3.22) and Equation (3.14):

$$Q_i = 0.67 A_g (2gd)^{0.5} + 0.67 hL(2gd_o)^{0.5} \qquad (3.23)$$

Where:

- Q_i = Amount of street flow intercepted by inlet, cfs
- A_g = Clear opening area of the grate, sq ft
- g = Acceleration due to gravity, 32.2 ft/sec^2
- d = Depth of flow at curb, ft
- h = Height of curb opening portion of catch basin, curb-opening orifice or orifice throat, ft
- L = Length of curb opening, ft
- d_o = Effective depth at the center of the curb opening orifice, ft

Trial and error solutions are necessary for depth at the curb for a given flow rate using Figure 3.9, Figure 3.10, Figure 3.11, or Figure 3.14 for orifice flow.

3.3.2.5 Slotted Drain Catch Basins

On-Grade

Wide experience with the debris-handling capabilities of slotted drain catch basins is not available. Deposition in the pipe is the problem most commonly encountered; however, the catch basin is accessible for cleaning with a high pressure water jet.

Flow interception by slotted drain catch basins and curb-opening catch basins is similar in that each is a side weir and the flow is subjected to lateral acceleration due to the cross slope of the pavement. Analysis of data from the HEC-12 tests of slotted drain catch basins with slot widths greater than or equal to 1.75 inches indicates that the length of the slotted drain catch basin required for total interception can be computed using Equation (3.7). Figure 3.5 is therefore applicable for both curb-opening catch basins and slotted drain catch basins. Similarly, Equation (3.8) is also applicable to slotted drain catch basins and Figure 3.6 can be used to obtain the catch basin efficiency for the selected length of the catch basin.

Using Figure 3.5 and Figure 3.6 for slotted drain catch basins is the same as using them for curb-opening catch basins. It should be noted, however, that it is much less expensive to add length to a slotted drain catch basin to increase interception capacity than it is to add length to a curb-opening catch basin.

Sump

Slotted drain catch basins in sump locations perform as weirs to depths of about 0.2 ft, dependent on slot width and length. At depths greater than about 0.4 ft, they perform as orifices. Between these depths, flow is in a transition stage. The interception capacity of a slotted drain catch basin operating as an orifice can be computed by.

$$Q_i = 0.8LW(2gd)^{0.5} \qquad (3.24)$$

Where:

Q_i = Amount of street flow intercepted by slotted inlet, cfs
L = Length of slotted inlet, ft
W = Width of slot, ft
d = Depth of water at slot, $d \geq 0.4$ ft
g = Acceleration due to gravity, 32.2 ft/sec^2

Equation (3.24) becomes:

$$Q_i = 0.94Ld^{0.5} \qquad (3.25)$$

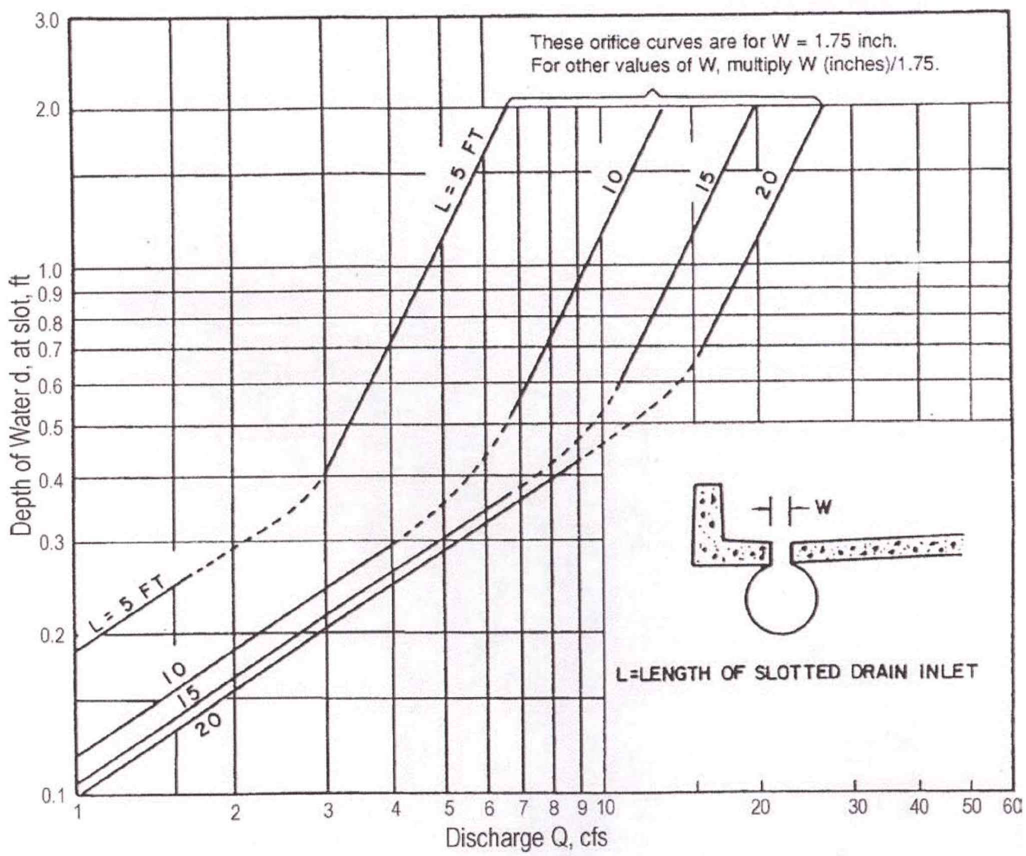
When:

W = 0.15ft (1.75 inches)

The interception capacity of slotted drain catch basins at depths between 0.2 and 0.4 feet can be computed by using the orifice equation. The orifice coefficient varies with depth, slot width, and the length of the slotted drain catch basin.

Figure 3-15 provides the solutions for weir flow, transition flow and orifice flow.

Figure 3.15: Slotted Drain Inlet Capacity in Sump Condition



3.3.2.6 Guidelines

Inlets in sumps are generally much more efficient and economically justifiable than inlets on a continuous grade, so the street designer should strive to adjust grades, when practical, to provide sumps for inlets. A sump is created at each intersection of a side street with a major street where the crown of the side street is extended at least to the quarter point of the major street. This provides an efficient pick up point. However, on the downstream side of the side street, incoming storm drainage will tend to flow on down the major street and bypass a catch basin. Therefore, where conditions permit, the side street may be depressed for a short distance upstream from the curb return to provide a second efficient pick up point, if the side street is bringing a large volume of runoff. Another alternative is multiple catch basins to intercept the excessive runoff. The most economical alternative shall be used.

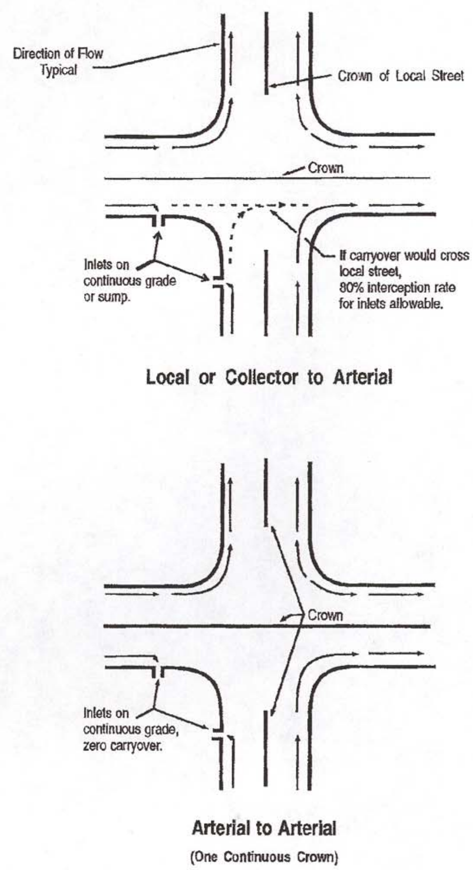
To account for a potential reduction of inflow capacity due to clogging, the design of the inlet should include a factor of safety. Here the area or length required is adjusted by clogging or reduction factors as set forth by the standards used by the jurisdictional entity. For Pinal County, clogging or reduction factors are set forth in Volume 1: Policies and Design Criteria.

3.3.3 Conveyance

3.3.3.1 Valley Gutters

Figure 3.16 shows some typical situations where local streets intersect arterial or collector streets. These examples show the minimum required inlets. Additional inlets may be necessary based upon allowable carrying capacity of gutters.

Figure 3. 16: Typical Street Intersection Drainage to Storm Drain System



The grades of the arterial or collector streets should be continued uninterrupted where local streets intersect arterial or collector streets. The grade of the more major street should be maintained as much as possible. No form of valley gutter for drainage purposes should be constructed across an arterial street. Occasionally, with agency approval, valley gutters may be considered on collector streets. Conventional valley gutters may be used to transport runoff across local streets where a storm drain system is not required and when approved by the governmental agency.

The valley gutter should be sufficient to transport the runoff across the intersection with lane encroachment limited to that allowed on the street. The theoretical carrying capacity of each gutter approaching an intersection shall be calculated based upon the effective slope, as outlined herein. Where the gutter slope will be continued across an intersection – as where valley gutters are used– use the slope of the gutter flow line crossing the street to calculate capacity.

Where the gutter flow must undergo a direction change at the intersection greater than 45 degrees, the slope used for calculating capacity shall be the effective gutter slope. This is defined as the average of the gutter slopes at 0 feet and 50 feet upstream from the point of direction change.

Where the gutter flow is intercepted by an inlet on a continuous grade with the intersection, the effective gutter slope shall be utilized for calculations. Under this condition, the points for averaging shall be 0 feet, 25 feet, and 50 feet upstream from the inlet.

Use walk-over curbs (where the pavement grade is raised to match the curb elevation at the crosswalk) where large volumes of pedestrian traffic are likely such as at intersections and other locations. Such a design may require two catch basins at nearly every corner if flow may not continue around the corner. The normal limitations on gutter flow and spread may need modification where concentrations of pedestrians occur. Ponding water and gutter flow wider than two feet can be difficult for pedestrians to negotiate and designing for pedestrian traffic is as important as designing for vehicular traffic.

3.3.3.2 Roadside Ditches

Roadside ditches are commonly used in rural areas to convey runoff from the highway pavement, and to intercept runoff from areas which drain toward the highway. Where practicable, the flow from major areas draining toward curbed highway pavements should be intercepted by ditches.

The following criteria pertain to the design of open channels along roadsides. For additional criteria for open channels, see Chapter 6.

Roadside ditches adjacent to public streets are discouraged in urban areas and require approval from the County Engineer. When they are allowed, adhere to the criteria outlined in this section.

The depth of flow in roadside ditches for the design storm shall be limited so that the adjacent roadway subgrade does not become saturated. Catch basins or scuppers should be provided as needed to drain the pavement into the drainage ditch where curbs exist and roadside ditches are used in lieu of storm drains.

Geometric considerations in the design of channel cross sections should incorporate hydraulic requirements for the design discharge, should be designed with safety in mind, should minimize the acquisition of right-of-way, should be designed for economy in construction and maintenance, and should have a good appearance.

Channel side slopes should be as mild as practical and should be no steeper than 4: 1 where terrain and right-of-way permit. Mild slopes reduce the potential for erosion and slides and the cost of maintenance, and improve the safety for errant vehicles. Safety considerations are subject to the requirements of the local jurisdiction.

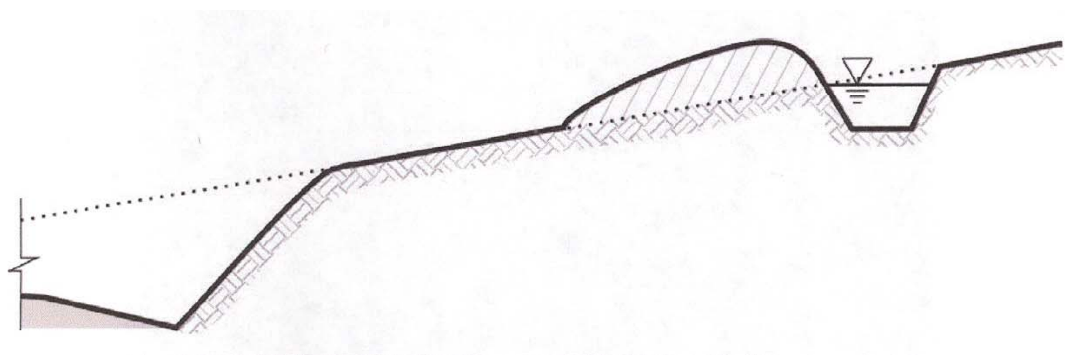
Trapezoidal channel bottoms should be a minimum of four feet wide for maintenance purposes. V-shaped channels may also be used where approved by the County Engineer.

Local soil conditions, flow depths, and velocities within the channel are usually the primary hydraulic considerations in channel geometric design; however, terrain and safety considerations have considerable influence. Steeper side slopes of rigid, lined channels may be more economical and will improve the hydraulic flow characteristics. The use of steeper slopes is normally limited to areas with limited right-of-way where the hazard to traffic can be minimized through the use of guardrails or parapets.

3.3.3.3 Rural Crown Ditch

In mountainous terrain where large cuts are required, crown ditches constructed on top of the cut embankment will intercept runoff and prevent it from eroding the face of the cut slope. A typical crown ditch is shown in Figure 3.17.

Figure 3. 17: Crown Ditch



3.3.4 Storage Facilities

Retention facilities may be used where no facilities exist to receive captured storm flows from a roadway. This is acceptable with approval from the appropriate governmental agency. Please refer to Volume 1 Chapter 3.10 for storage facility criteria.

3.4 REFRENCES

The procedures, equations, and nomographs in this section are adapted from the Federal Highway Administration, Hydraulic Engineering Circular No. 22 (HEC-22), Urban Drainage Design Manual (US DOT, FHWA, 1996) and U.S. Department of Transportation, Federal Highway Administration, March 1984, Hydraulic Engineering Circular No. 12, Drainage of Highway Pavements. Policies and Standards relative to Street Drainage are listed in the Policy and Standards Manual.

VOLUME 2
DESIGN METHODOLOGY AND PROCREDURES

Chapter 4: Storm Drains

4.1 INTRODUCTION

This chapter describes methodology that should be used for the hydraulic design of a stormdrain system. In this manual, a stormdrain system refers to a coordinated group of inlets, underground conduits, manholes, and various other appurtenances which are designed to collect stormwater runoff from the design storm and convey to a point of discharge into a major or regional drain outfall.

The size of a stormdrain system is based on a designated design storm, which has a specific storm duration and intensity. The design storm may vary from community to community, so the designer must determine the appropriate design storm from the County Engineer.

The designer should contact the County Engineer as soon as possible to explain the situation if the designer has to deviate from the requirements of this chapter. That way the designer and the County can agree on an acceptable solution and expedite the design process.

This chapter presents analysis and design methods that do not require the use of computers. However, there are many computer programs available to help in the design of stormdrain systems. These programs, however, may determine the various headlosses by methods different than those presented in this chapter. It is therefore recommended that the designer of any stormdrain system check with the County Engineer before using a particular program.

4.2 CONCEPTS

4.2.1 Pressure Flow vs. Open Channel Flow

Although not always feasible, the recommended procedure is to design stormdrains to flow under pressure because this maximizes conveyance while minimizing capital expenditure. However, the hydraulic grade line should fall below an inlet, ground, or manhole rim elevation. The freeboard for inlets is set forth in Chapter 3 of Volume 1. This freeboard sets the height below the inlet elevation for the HGL.

Losses at bends and junctions will frequently cause pressure flow to occur for some distance upstream of the “loss” area even though a conduit may be designed to carry stormwater as open-channel flow. Situations may also occur in steeper terrain where the flow often changes between open channel and pressure flows. If pressure flow is predicted in a pipe segment, changing to a larger pipe can result in open channel flow. Because it is not economical to size conduits to avoid pressure flow under all storm runoff and flow conditions, it follows that it is reasonable and even necessary to design the conduits as flowing full.

Often a closed conduit designed for open channel flow will operate as a pressure conduit. This may result when storm runoff exceeds that used for design purposes or simply because junction losses were underestimated or neglected in the design. In stormdrain systems, junctions in closed conduits can cause major losses in the energy grade line across the junction. If these losses are not included in the hydraulic design, the capacity of the conduit may not be adequate for the design flow.

4.2.2 Hydraulic Grade Line

. The hydraulic grade line (HGL) represents the potential energy of the flowing water. Where the HGL falls below the pipe crown the pipe functions as an open channel. Where the HGL is above the pipe crown the pipe will operate as a pressure conduit. Calculations to check the pressure (hydraulic grade) of water surface elevations in the stormdrain system typically begin with a known hydraulic grade elevation at some downstream point. To this are added the various losses that can occur to determine the likely upstream hydraulic grade elevation. These losses are commonly referred to as *headlosses*. The procedures for calculating the various headlosses are presented in the Head Losses section of this chapter.

Where pressure flow occurs, the resulting HGL can rise above not only the pipe crown but also above the rim of manholes or catch basins. Under those conditions, the water in the system can escape it. Extreme conditions result in manhole covers “popping up” above the street. Stormwater systems are usually designed for open channel flow for such reasons.

Whether or not the final design assumes the pipe is flowing partially or completely full, a hydraulic grade line must be computed and displayed on a profile drawing of the conduit.

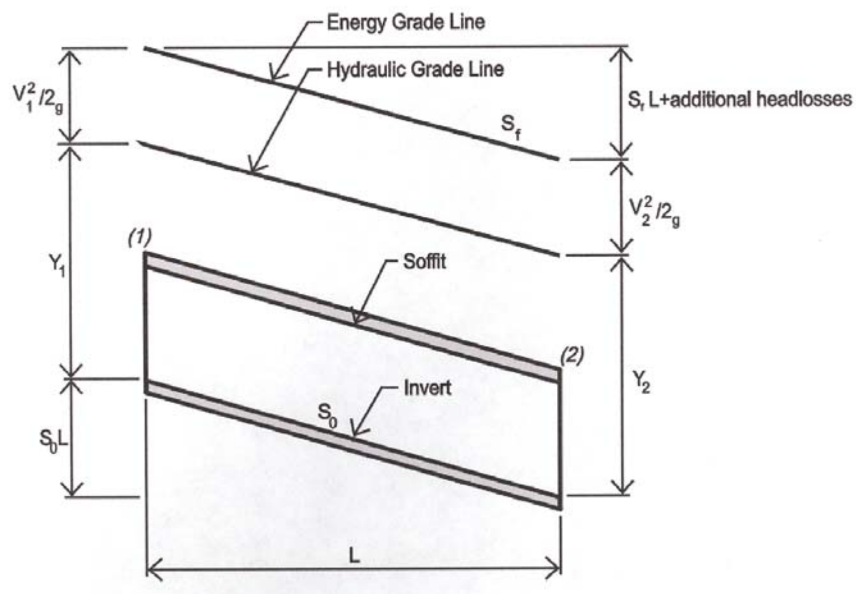
4.2.3 Energy Equation

Most procedures for calculating hydraulic grade line profiles are based on the energy equation and can be expressed as:

$$\frac{V_1^{\ 2}}{2g} + Y_1 + S_0 L = \frac{V_2^{\ 2}}{2g} + Y_2 + S_f L + headlosses \qquad (4.1)$$

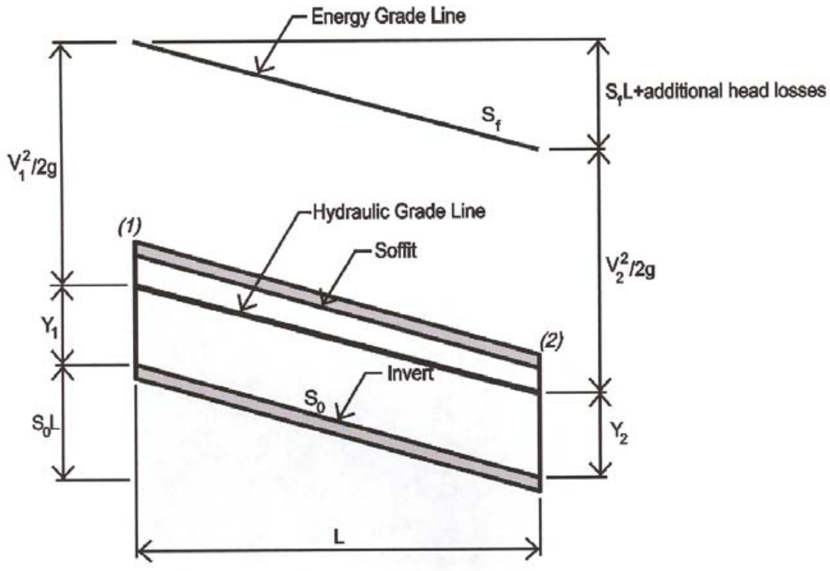
The various terms used in Equation (4.1) are identified in Figure 4.1 and Figure 4.2. Minor losses have been included in the energy equation because of their importance in calculating hydraulic grade line profiles.

Figure 4. 1: Strom Drain Profile Pressure Flow Conditions



As depicted, Y_1 and Y_2 include the pressure components since they are above the soffit of the pipe.

Figure 4. 2: Storm Drain Profile Open Flow Conditions



In this presentation of design methods, provision is made to identify pipes by use of numbered subscripts. The number one (1) is used to identify the upstream main pipe, the number two (2) is used to identify the downstream main pipe, and the number three (3) is used for incoming or branching flow.

The general procedure for the hydraulic calculations is to establish the downstream control elevation. From there the hydraulic calculations proceed upstream from point of interest to point of interest. For example, from one junction to another junction or from a junction to the beginning of a bend. At the lower end of each point of interest the pipe friction losses from the downstream section are added to the downstream hydraulic grade line. The losses through the point of interest are added at the upstream end of the point of interest. The procedures for calculating the various headlosses encountered in a stormdrain system are presented in the following Head Losses Section. The Hydraulic Grade Line Calculation Sheet (found at the end of this Chapter and as an electronic document and spreadsheet from Pinal County) may be used to assist in the accounting and computing of the losses.

Equation (4.2) is a simplification of a more complex equation and is a convenient method for locating the approximate point where pressure flow may cease (may become open channel flow). It is derived by substituting specific energy (E) for the quantity $V^2/2g+Y$ in Equation (4.1) and rearranging the results. For S_f use the average friction slope between the two points of interest.

$$L=\frac{E_2-E_1}{S_0-S_f} \tag{4.2}$$

4.2.3.1 Head Losses

The headlosses that need to be determined are: friction, transition, junction, manhole, bend, inlet, and exit. These losses are usually determined individually and then added together to determine the overall headloss for each segment of the stormdrain. The methods for determining the various headlosses presented in this section were selected for their wide acceptance and ease of use.

Friction losses

Friction losses for closed conduits carrying stormwater, including pump station discharge lines, will be calculated from Manning's equation or a derivation thereof. The Manning's equation is commonly expressed as follows:

$$Q = \frac{1.486}{n} AR^{2/3} S^{1/2} \tag{4.3}$$

The equation for determining pipe friction slope can be expressed as:

$$S_f = K \frac{V^2}{2gR^{4/3}} \tag{4.4}$$

Where:

V = Velocity, ft/sec
g = Acceleration due to gravity, 32.2 ft/sec^2

The value of K is dependent only upon the roughness coefficient (n) for the pipe. The Manning's n values for various pipe materials are given in Table 4.1. The value of K can be estimated using Equation (4.5).

$$K = \frac{2gn^2}{2.21} \tag{4.5}$$

Where:

g = Acceleration due to gravity, 32.2 ft/sec^2

Table 4- 1: Values of Roughness and Friction Formula Coefficients for Closed Conduits

Conduit Material	Manning's n
Asbestos Cement Pipe	0.013
Brick	0.015
Cast Iron Pipe	
Cement lined and seal coated	0.013
Concrete (monolithic)	
Smooth forms	0.013
Rough forms	0.017
Concrete Pipe	0.013
Corrugated Metal Pipe ($^1/_2$ x 2 $^2/_3$ in corrugations)	
Plain	0.024
Paved invert	0.020
Spun asphalt lined	0.013
Corrugated Polyethylene Pipe	
15" diameter	0.018
18 to 36" diameter	0.020
Plastic Pipe (smooth)	0.013
Vitrified Clay	
Pipes	0.013
Liner plates	0.013

The loss of head due to friction throughout the length of reach (L) is calculated by:

$$h_f = S_f L \tag{4.6}$$

Where:

h_f = Friction headloss, ft
L = Reach length, ft

Transition losses

There are two types of pipe transitions that can occur in a stormdrain system that would add headloss to the energy grade line. The transition types are expansion and contraction. Figure 4.3 shows the two types of transitions that can be encountered. The headloss due to the expansion of flow for a storm sewer flowing under open channel conditions is expressed as:

$$h_t = k_e \left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g} \right) \tag{4.7}$$

Where:

h_t = Transition headloss, ft
k_e = Coefficient for transition loss due to expansion

V_1 = Upstream velocity, ft/sec
V_2 = Downstream velocity, ft/sec
g = Acceleration due to gravity, 32.3 ft/sec^2

Note: V_1 is greater than V_2

The values for the transition coefficient, k_e, for enlargements are given in Table 4.2.

The headloss due to the contraction of flow under open channel flow conditions is expressed as:

$$h_t = k_c\left(\frac{V_1^{\,2}}{2g} - \frac{V_2^{\,2}}{2g}\right) \tag{4.8}$$

Where:

h_t = Transition headloss, ft
k_c = Coefficient for transition loss due to constriction
V_1 = Upstream velocity, ft/sec
V_2 = Downstream velocity, ft/sec
g = Acceleration due to gravity, 32.3 ft/sec^2
Note: V_1 is greater than V_2

Values for the transition loss coefficient, k_c, for contractions can also be found in Table 4.2.

Figure 4. 3: Transition Loss

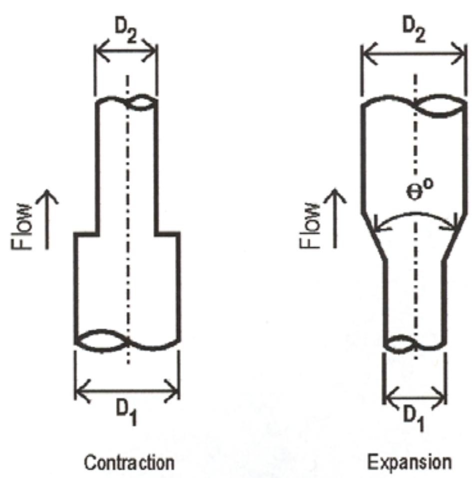


Table 4- 2: Storm Sewer Energy Loss Coefficients under Open Channel Conditions

(ASCE, 1992)

(a) Contractions (K_c)		(b) Expansion (K_e)		
$\frac{D_2}{D_1}$	K_c	θ	$\frac{D_2}{D_1}=3$	$\frac{D_2}{D_1}=1.5$
0.0	0.5	10	0.17	0.17
0.4	0.4	20	0.40	0.40
0.6	0.3	45	0.86	1.06
0.8	0.1	60	1.02	1.21
1.0	0	90	1.06	1.14
		120	1.04	1.07
		180	1.00	1.00

Under pressure flow conditions, the headloss due to contraction and expansion of flow can be expressed as:

$$h_t = k\frac{V^2}{2g} \tag{4.9}$$

Where:

h_t = Headloss due to a contraction or expansion, ft
k = Coefficient for contraction (k_c) or expansion (k_e), see below
V = Velocity of flow in the smallest diameter pipe, ft/sec

The values for the transition coefficient, k_e, for gradual enlargements are given in Table 4.3. For sudden enlargements, values for the transition coefficients are listed Table 4.4. Values for the transition loss coefficient, k_c, for sudden contractions can be found in Table 4.5.

Table 4- 3: Coefficient k_e for Gradual Enlargement Under Pressure Flow Conditions

(AISI, 1990)

$\frac{D_2}{D_1}$	Angle of Cone, degrees													
	2	4	6	8	10	15	20	25	30	35	40	45	50	60
1.1	0.01	0.01	0.01	0.02	0.03	0.05	0.10	0.13	0.16	0.18	0.19	0.20	0.21	0.23
1.2	0.02	0.02	0.02	0.03	0.04	0.09	0.16	0.21	0.25	0.29	0.31	0.33	0.35	0.37
1.4	0.02	0.03	0.03	0.04	0.06	0.12	0.23	0.30	0.36	0.41	0.44	0.47	0.50	0.53
1.6	0.03	0.03	0.04	0.05	0.07	0.14	0.26	0.35	0.42	0.47	0.51	0.54	0.57	0.61
1.8	0.03	0.04	0.04	0.05	0.07	0.15	0.28	0.37	0.44	0.50	0.54	0.58	0.61	0.65
2.0	0.03	0.04	0.04	0.05	0.07	0.16	0.29	0.38	0.46	0.52	0.56	0.60	0.63	0.68
2.5	0.03	0.04	0.04	0.05	0.08	0.16	0.30	0.39	0.48	0.54	0.58	0.62	0.65	0.70
3.0	0.03	0.04	0.04	0.05	0.08	0.16	0.31	0.40	0.48	0.55	0.59	0.63	0.66	0.71
	0.03	0.04	0.05	0.06	0.08	0.16	0.31	0.40	0.49	0.56	0.60	0.64	0.67	0.72

Table 4- 4: Coefficient k_e for Sudden Enlargement Under Pressure Flow Conditions

(AISI, 1990)

$\frac{D_2}{D_1}$	Velocity, V_l, ft/sec												
	2	3	4	5	6	7	8	10	12	15	20	30	40
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

Table 4- 5: Coefficient k_c for Sudden Contraction Under Pressure Flow Conditions

(AISI, 1990)

$\frac{D_2}{D_1}$	Velocity, V^2 ft/sec												
	2	3	4	5	6	7	8	10	12	15	20	30	40
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	0.10	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	0.25	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	0.31	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

Junction losses

A junction occurs where one or more lateral pipes enter the main stormdrain, at a formed junction, prefabricated fitting, or at a manhole. Multiple pipes coming together at a junction should flow together smoothly to avoid high headlosses. Figure 4.4, Figure 4.5, and Figure 4.6 show typical junctions in plan and profile.

Junction headloss for a single lateral can be determined by applying the Energy Equation and the Thompson Equation (California Department of Transportation, 1985).

The Energy Equation (Equation (4.1)) at a junction (as displayed in Figure 4.4 through Figure 4.6) is expressed as:

$$\frac{V_1^{\ 2}}{2g} + Y_1 + Z_1 = \frac{V_2^{\ 2}}{2g} + Y_2 + Z_2 + headlosses \qquad (4.10)$$

Where:

headlosses = h_j (junction loss) + h_T (transition loss) + h_F (friction loss)

$\frac{V_1^{\ 2}}{2g}$ = Main line velocity head upstream of junction, ft

$\frac{V_2^{\ 2}}{2g}$ = Main line velocity head downstream of junction, ft

Y_1 = Upstream hydraulic gradient elevation measure from invert, ft

Y_2 = Downstream hydraulic gradient elevation measure from invert, ft

Z_1 = Elevation at location 1, ft

Z_2 = Elevation at location 2, ft

Equation (4.1) can be rewritten to solve for headlosses

$$\frac{V_1^{\ 2}}{2g} - \frac{V_2^{\ 2}}{2g} + Y_1 - Y_2 + Z_1 - Z_2 = headlosses$$

Substitute HG_1 for $Y_1 + Z_1$ and HG_2 for $Y_2 + Z_2$

$$\frac{V_1^{\ 2}}{2g} - \frac{V_2^{\ 2}}{2g} + HG_1 - HG_2 = headlosses$$

$$\frac{V_1^{\ 2}}{2g} - \frac{V_2^{\ 2}}{2g} + \Delta HG = headlosses$$

The Thompson Equation (Equation (4.10a)), a form of the momentum equation, is used to determine the change in flow depth across a junction.

$$\Delta HG \frac{A_1 + A_2}{2} = \frac{Q_2 V_2 - Q_1 V_1 - Q_3 V_3 \cos\theta}{g} \qquad (4\text{-}15)$$

or

$$\Delta HG = \frac{\dfrac{Q_2V_2 - Q_1V_1 - Q_3V_3\cos\theta}{g}}{\dfrac{A_1 + A_2}{2}} \tag{4-16}$$

Where:

ΔHG = Difference in upstream and downstream hydraulic grade line elevations, ft
A_1 = Upstream flow area, sf
A_2 = Downstream flow area, sf
Q_1 = Upstream flow rate, cfs
Q_2 = Downstream flow rate, cfs
Q_3 = Lateral flow rate, cfs
V_1 = Upstream flow velocity, fps
V_2 = Downstream flow velocity, fps
V_3 = Lateral flow velocity, fps
θ = Angle between lateral and main line stormdrain (See Figure 4.7), degrees

To determine junction headloss h_j, substitute the Thompson Equation into the rewritten Equation (4.1), assuming transition and friction losses at the junction are negligible.

$$hj = \frac{2(Q_2V_2 - Q_1V_1 - Q_3V_3\cos\theta)}{(A_1 + A_2)g} + \frac{V_1^{\ 2}}{2g} - \frac{V_2^{\ 2}}{2g} \tag{4-17}$$

Should friction losses be determined not to be negligible Equation (4.18) should be used:

$$hj = \frac{2(Q_2V_2 - Q_1V_1 - Q_3V_3\cos\theta)}{(A_1 + A_2)g} + \frac{V_1^{\ 2}}{2g} - \frac{V_2^{\ 2}}{2g} + \left(\frac{S_{f1} + S_{f2}}{2}\right)L \tag{4.14}$$

Where:

S_{f1} = Upstream friction slope, ft
S_{f2} = Downstream friction slope, ft
L = Length of transition, ft

Should transition losses be determined not to be negligible but friction losses are negligible, then Equation (4.15) should be used for computing junction loss h_j.

$$hj = \frac{2(Q_2V_2 - Q_1V_1 - Q_3V_3\cos\theta)}{(A_1 + A_2)g} + \frac{V_1^{\ 2}}{2g} - \frac{V_2^{\ 2}}{2g} + k_{je}\left(\frac{V_1^{\ 2}}{2g} - \frac{V_2^{\ 2}}{2g}\right) \tag{4.15}$$

Where:

K_{je} = Coefficient for transition loss due to expansion at a junction

$$k_{je} = 3.50(\tan\frac{\theta}{2})^{1.22}$$ (California Department of Transportation, 1985)

See Figure 4.4 through 4.7c for location of θ angle.

V_1 = Upstream velocity, ft/sec

V_2 = Downstream velocity, ft/sec

g = Acceleration due to gravity, 32.2 ft/sec^2

Figure 4. 4: Formed or PreFab Storm Drain Junction

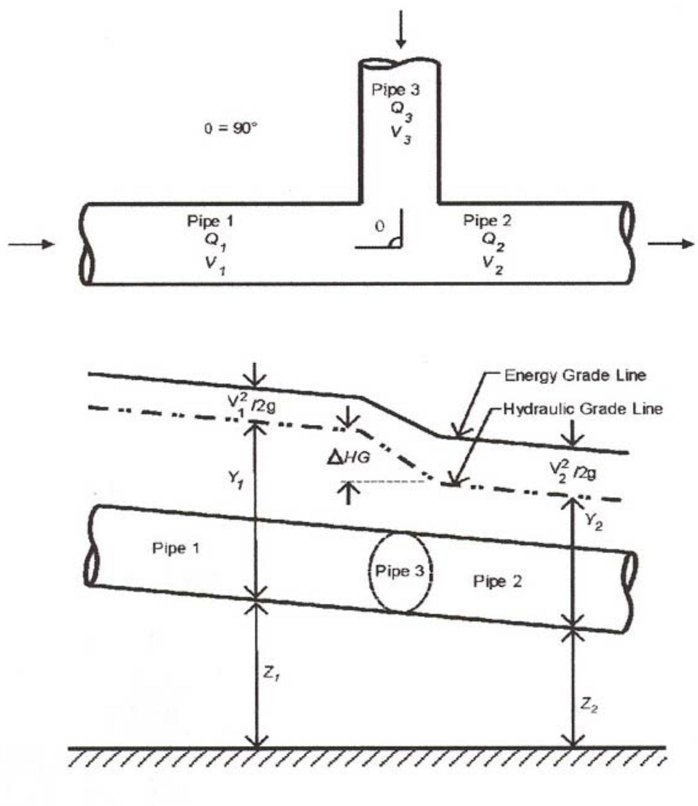


Figure 4. 5: Strom Drain Function at Manhole with Aligned Crowns under Pressure Flow

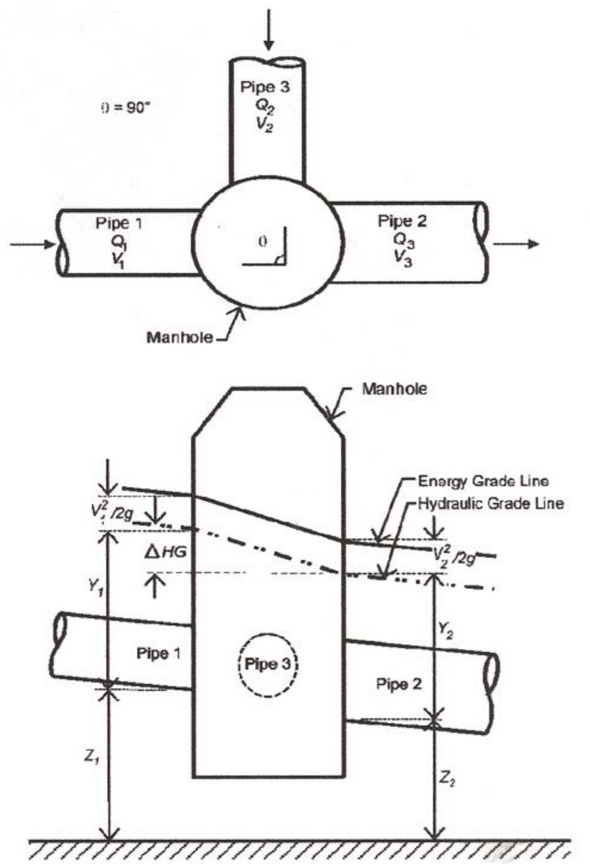
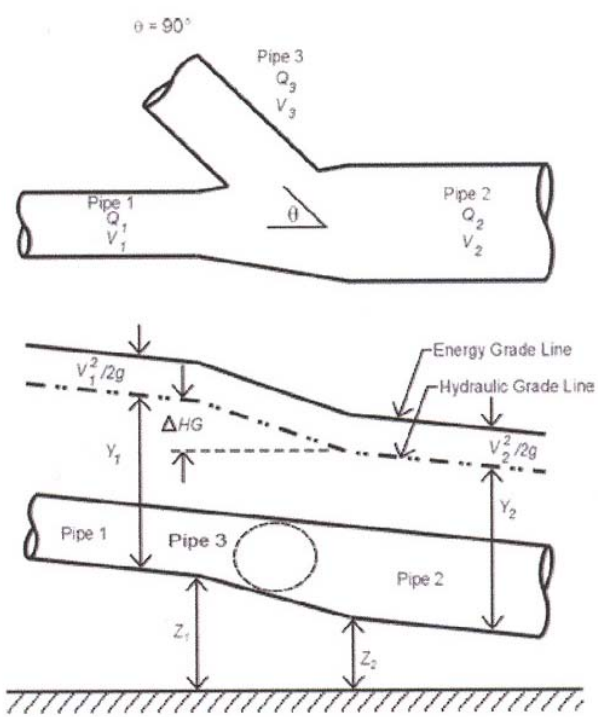


Figure 4. 6: Formed Strom Drain Junction with Aligned Crowns Under Pressure Flow



In situations where crowns at a junction are not matching, a pressure momentum approach for solving headloss is suggested

Straight-through manhole losses (no laterals)

In a straight-through manhole where there is no change in pipe size or rate of flow, the loss can be estimated by Equation (4.16):

$$h_{mh} = 0.05\frac{V^2}{2g} \qquad (4.16)$$

Where:

h_{mh}= Headloss due to a manhole, ft
V = Velocity, ft/sec

Bend losses at manholes (no laterals)

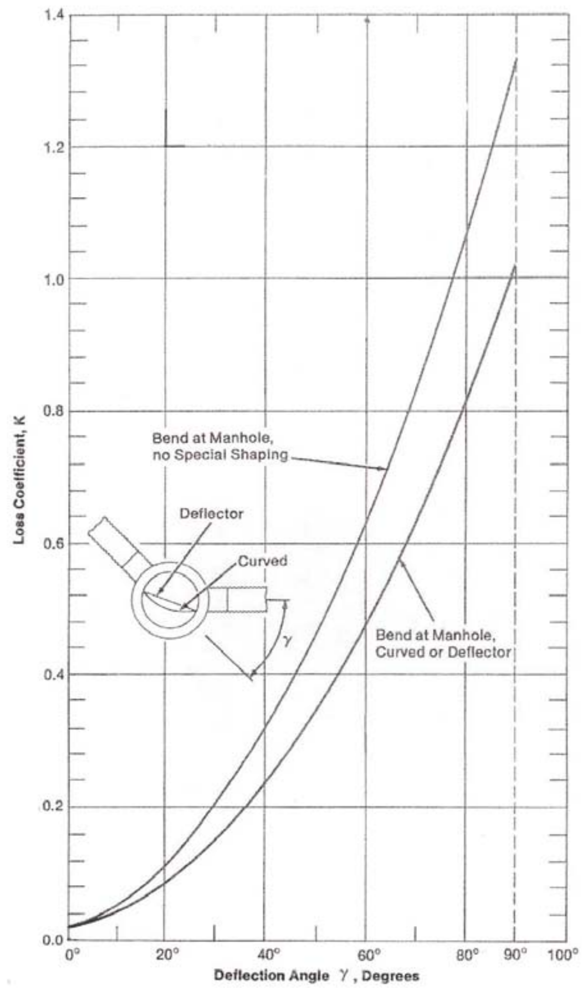
The bend loss at a manhole is determined using Equation (4.20). The bend loss coefficient, k_b, can be determined using Figure 4.7.

$$h_{mh} = k_b\frac{V^2}{2g} \qquad (4.17)$$

Where:

hmh = Headloss due to a manhole, ft
kb = Bend loss coefficient
V = Velocity of flow, ft/sec
g = Acceleration due to gravity, 32.2 ft/sec2

Figure 4. 7: Bend Loss Coefficient



Bend losses at curved sewer

For bend loss at a curved sewer, the loss is calculated using Equation (4.18).

$$h_b = k_b \frac{V^2}{2g} \tag{4.18}$$

Where:

h_b = Headloss due to a bend, ft
k_b = Bend headloss coefficient
V = Velocity of flow, ft/sec
g = Acceleration due to gravity, 32.2 ft/sec^2

The value of the bend loss coefficient, k_b, depends upon the angle of the bend. It can be estimated from Equation (4.19) (USDOT, 2001).

$$k_b = 0.0033\Delta \tag{4.19}$$

Where:

k_b = Bend headloss coefficient
Δ = Angle of curvature or deflection, degrees

Bend losses should be included for all closed conduits, those flowing partially full as well as those flowing full.

Inlet losses

A stormdrain inlet will operate the same as a culvert inlet at open inlets to a stormdrain system. Under inlet control, the hydraulic grade line at the entrance can be estimated by using the appropriate procedures and figures presented in the Culvert Chapter. Under outlet control, entrance losses can be calculated using Equation (4.20).

$$h_i = k_{en} \frac{V^2}{2g} \tag{4.20}$$

Where:

h_i = Headloss at inlet, ft
k_{en} = Entrance loss coefficient

The k_{en} in the equation is equivalent to k_e values listed in Table 4.6.

In addition to the entrance loss, losses associated with a protection barrier or trashrack over the inlet should be taken into consideration. Procedures to estimate headlosses due to barriers or trashracks can be found in Trashracks and Access Barriers section of Chapter 5.

Table 4- 6: Entrance Loss Coefficients

Outlet Control, Full or Partly Full Entrance Head Loss

(USDOT, FHWA, HDS-5, 1985)

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 (grove-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, 33.7° or 45° bevels	0.2
Side-or slope-tapered inlet	0.2
Pipe, or Pipe-Arch, Corrugated Metal	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls square-edge	0.5
Mitered to conform to fill slope, paved or unpaved slope	0.7
End-Section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° 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 on sides	0.2
Wingwalls at 30° to 75° to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of 1/12 barrel dimension, or beveled top edge	0.2
Wingwalls at 10° to 25° 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

Outlet losses

Additional headloss occurs due to the change in velocity and changes in flow direction where a stormdrain outfalls to a retention basin, lake, or open channel. The exit headloss at stormdrain outlets is expressed as in Equation (4.21) (Clark County Regional Flood Control District, 1990):

$$h_o = 1.0 \frac{V_0^{\ 2}}{2g} \tag{4.21}$$

Where:

h_o = Headloss at outlet, ft
V_o = Average outlet velocity, ft/sec

4.3 DESIGN PROCEDURE

The general process for the design or analysis of a stormdrain system is:

- Layout the stormdrain system in a sketch or graphic view, identifying and characterizing each inlet, connector pipe, manhole, catch basin, main pipe, and discharge point.
- Compute the tributary flows at each inlet.
- Starting from the upstream end and working downstream, size the main line and connector pipes based on hydraulic capacity.
- Starting from the downstream end, compute the hydraulic grade line (HGL) including provisions for losses. Check for where the HGL exceeds the limits given in Section 3.5.8 of Volume 1, Chapter 3
- Adjust pipe sizes and slopes as necessary to achieve the desired design.

The general procedure for establishing the quantity of flow is the same for a pipe flowing either as an open channel or as a pressure conduit. However, because of the nature of flow in circular conduits real open channel flow occurs only if the flow depth is less than 80 percent of the conduit diameter.

4.4 APPLICATION

4.4.1 Pipe Sizing

The Pipe Sizing Table shown at the end of this chapter can be used to track the entries and organize the data. It is available in a spreadsheet format as well from Pinal County.

4.4.1.1 Initial Pipe Slope Selection

The initial pipe slope can be set using any criterion desired. One common way is to simply use the roadway longitudinal gradient. Another is to assume an initial hydraulic grade line elevation at both ends and compute the slope from the difference in elevation and the distance along the pipe route. A typical approach using the HGL is to assume that the HGL at the upstream and downstream ends of the pipe segment is one foot below the ground surface. Since the goal of design is to keep the HGL below the surface, this will give good starting values for slope.

4.4.1.2 Compute Inflow to an Inlet

Use the Rational Formula to compute the inflow to the most upstream inlets as described above. If two or more inlets connect at the upstream end of the first pipe, combine the inflows from all. Use the design storm frequency and the longest time of concentration to select the rainfall intensity; use 10 minutes if the actual value is less than that.

4.4.1.3 Size Stormdrain Pipe

Calculate the initial size of the stormdrain pipe using Equation (4.22):

$$D = 1.33\left(\frac{nQ}{\sqrt{S}}\right)^{3/8} \qquad (4.22)$$

Where:

n = Manning's roughness coefficient for the pipe
Q = the inflow to the upstream end of the pipe, cfs
S = the pipe slope, ft/ft

Equation (4-2) is based on the pipe flowing full.

Round up the pipe size to the next larger nominal size for the type of pipe being used. Also, round up to the minimum size (see Chapter 3.6.2 of Volume 1 for minimum pipe sizes).

4.4.1.4 Check Velocity of Flow

The velocity of flow in the pipe under the peak flow conditions is computed using Equation (4.23):

$$V = \frac{Q}{A} \tag{4.23}$$

Where:

Q = the computed flow rate, cfs
A = the area of flow, sf

The actual pipe inside area is used for pipes flowing full. When flowing less than full, the area is that occupied by the water.

Chapter 3 of Volume 1 provides the minimum velocities required for stormdrain systems.

Minimum velocities are used to create a condition where the pipe is “self-cleansing”, which means that under the design event flow any sediments entering the pipe will be carried on through and any sediments deposited previously in the pipe will be swept away.

Two minimum velocities are usually used:

- A velocity for flowing full
- A velocity at one-half of the peak discharge

If the computed velocity is less than the minimum full-flow velocity, a check should be made to determine if the velocity is greater than the one-half peak flow velocity. If less than the one-half peak velocity:

- Steepen the pipe to increase velocity

A check should also be made against the maximum permissible velocity, found in Chapter 3 referenced above. If the maximum velocity is exceeded:

- Change pipe material to one with a higher allowable maximum velocity
- Flatten the pipe run so that full-flow Q, and therefore full-flow velocity, are reduced

4.4.1.5 Set Pipe Elevation

Set the upstream pipe invert elevation based on the pipe size selected and the HGL assumptions made. Set the downstream pipe invert elevation using the pipe slope selected.

4.4.1.6 Compute Time of Travel

Compute the time of travel within this pipe segment using the velocity from the pipe design above and the length of the pipe segment using Equation (4.24):

$$T_{cd} = \frac{L}{60V} \tag{4.24}$$

Where:

T_{cd} = time in segment, min
L = segment length, ft
V = computed velocity, fps

4.4.1.7 Add Junction

If the segment ends at a manhole, set the invert elevation for the next pipe using the standard fall through the manhole as given in Section 3.5 of Volume 1, Chapter 3.

If the segment ends at a pipe junction, continue the pipe through the junction at the same slope and size.

4.4.1.8 Design Next Segment

If inflows are added at the junction, use the process above to compute the added inflows. Add the T_{cd} computed above to the T_c used for the previous segment to arrive at a new T_c for this segment. Use this new T_c and the design storm frequency to derive a new rainfall intensity, then use this new *i* to compute the runoff for the next tributary area(s).

Continue with the design steps above for this and the remaining segments of the stormdrain.

4.4.1.9 Setting Pipe Elevations

Where pipe size changes at a manhole, set the downstream pipe invert elevation so that the crown of the pipes entering and leaving have the fall given in Section 3.5 of Volume 1, Chapter 3.

Where pipe size changes at a non-manhole junction, match the crowns of the two pipes.

4.4.1.10 Adjusting Pipe Segments

Adjust the slope of one or more upstream pipe segments if the pipe elevation at the most downstream point of the stormdrain segment does not meet the desired elevation. Then revise the pipe sizing table entries to reflect the new conditions.

4.4.2 Evaluate Hydraulic Grade Line

Once preliminary pipe sizes are selected and pipe slopes determined, the stormdrain design must be checked to be sure the HGL falls within the guidelines and limits set forth in Section 3.5.8 of Volume 1, Chapter 3. The following items describe the energy and headloss considerations for stormdrains.

The HGL and the energy line are related at all times by the velocity head. The difference in elevation between the HGL and the higher energy line is given by $V^2/(2g)$, where V is the velocity of the flow in feet per second (ft/sec) and g represents the acceleration of gravity, which value is 32.2 ft/sec^2. Losses in head are related to the velocity, so computing the HGL requires computing and working with the associated energy line

Changes in HGL and energy line are caused by the slope and length of the pipe and by losses that occur as the flowing water encounter expansions, contractions, bends, and other situations.

Once the stormdrain has been initially designed working downstream for capacity and pipe size using the process steps described above, evaluation of the HGL is done working upstream from the discharge end.

4.4.2.1 Step-by-step Process

The HGL analysis moves upstream one segment at a time, computing the new energy line and HGL elevations at each upstream end. These HGL elevations are compared with the criteria to determine if changes in pipe size or slope need to be made. Changes in the pipe size or slope are then carried back into the hydraulic calculations so that the two analyses proceed together.

4.4.2.2 Starting HGL

The starting HGL at the discharge end is determined by the situation the discharge is entering. If to an impoundment or a relatively still body of water, the starting HGL is at the water surface elevation. If the discharge joins flow in a channel, the starting HGL will be that of that flow being joined.

Starting at the most downstream point, the HGL and energy line elevation are computed (or estimated).

4.4.2.3 Gain for Pipe Segment

Ignoring losses, the HGL and energy line elevations at the upstream end of a pipe run are higher than the HGL and energy line elevations at the downstream end by the product of the pipe slope and the length of the run.

The upstream energy line elevation may be even higher yet if there are losses between the two points. These losses may be estimated using the procedures in the sections that follow.

The upstream HGL elevation is lower than the energy line elevation by the velocity head, $V^2/(2g)$.

4.4.2.4 Discharge Hydraulic Grade Line

A stormdrain system may discharge into one of the following:

- A body of water such as a storage facility, reservoir, or lake.
- A natural watercourse or open channel (either improved or unimproved).
- Another closed conduit.

The controlling water surface elevation at the point of discharge is commonly referred to as the tailwater elevation. The tailwater elevation at the stormdrain outfall must be considered carefully. Evaluation of the hydraulic grade line for a stormdrain system begins at the system outfall with the tailwater elevation.

The tailwater elevation at the stormdrain outlet should be considered the same as the water surface elevation within the receiving channel or facility which has the same return period as the stormdrain design discharge, unless otherwise approved by the County Engineer. In general the two types of tailwater conditions are:

1. Tailwater elevation is above the crown elevation. In such situations the control shall conform to the following criteria:

    a. In the case of a conduit discharging into a storage basin, the control shall be the storage basin water surface elevation coinciding with the design peak flow to the storage basin.

    b. In the case of a conduit discharging into an open channel, the tailwater elevation shall be the water surface elevation of the channel coinciding with same return period as the stormdrain design peak discharge.

    c. In the case of a conduit discharging into another conduit, the control shall be the highest hydraulic grade line elevation of the outlet conduit immediately upstream or down-stream of the confluence.

2. Tailwater elevation is at or below the crown elevation. The tailwater shall be the crown elevation at the point of discharge.

4.4.2.5 Connector Pipe Hydraulic Grade Line

Connector pipes connecting catch basins to stormdrains can be sized and/or evaluated by estimating headlosses due to friction and inlet losses at catch basin. The designer should consider the catch basin connector pipes to be flowing full. The headloss due to friction can be estimated by using Equation (4.2). The headlosses at the inlet of the connector pipe can be estimated by using Equation (4.17). Equation (4.17) is modified from Equation (4.15):

4.4.3 Manhole Design

In stormdrain systems, junctions in closed conduits can cause major headlosses across the junction. If these losses are not included in the hydraulic design, the capacity of the conduit may not be adequate for the desired design flow. For a straight flow-through condition at a manhole, pipes should be positioned vertically so that the crowns are aligned. An offset in the plan is allowable provided the projected area of the smaller pipe falls within that of the larger. Aligning the crowns of the pipes is the most hydraulically efficient. When two inflowing laterals intersect in a manhole, the horizontal alignment of those laterals is important. For example, if two lateral pipes are aligned opposite each other such that the outflows impinge directly upon each other, the magnitude of the losses can be extremely high. If the installation of directly opposed inflow laterals is necessary, the installation of a deflector, as shown in Figure 4.1will result in significantly reduced losses. The research conducted on this type deflector is limited to the ratios of D_o/D_i = 1.25. The tests indicate that it would be conservative to assume the coefficient of pressure change at 1.6 for all flow ratios and pipe diameter ratios when no catch basin is considered, and 1.8 when the catch basin flow is more than 10 percent of Q_o. Lateral connector pipes should not be located directly opposite; rather, their centerlines should be separated laterally by at least the sum of the two lateral pipe diameters. Some jurisdictions require greater separation, and therefore, the design engineer should check jurisdiction specific standards. Studies have shown that this reduces headlosses as compared with directly opposed laterals, even with deflectors. Sufficient data has not been collected to determine the effect of off-setting laterals vertically.

Jets issuing from the upstream and lateral pipes must be considered when attempting to shape the inside of manholes. Tests for full flow revealed that very little, if anything, is gained by shaping the bottom of a manhole to conform to the pipe invert. Shaping of the invert may even be detrimental when lateral flows are involved, as the shaping tends to deflect the jet upwards, causing unnecessary headloss. From a practical point of view, limited shaping of the invert is necessary in order to handle low flows and to reduce sedimentation. Figure 4.8

details several types of deflector devices that have been found efficient in reducing losses at junctions and bends. In all cases, the bottoms are flat or only slightly rounded, to handle low flows. Numerous other types of deflectors or shaping of the manhole interiors were tested by the University of Missouri. Some of these devices were found inefficient and are shown in Figure 4.9. The fact that several of these inefficient devices would appear to be improvements indicates that special shapings deviating from those in Figure 4.8 should be used with caution, possibly only after model tests. Tests indicate that rounding entrances or the use of pipe socket entrances do not have the effect on reducing losses that might be expected. Once again, the effect of the jet from the upstream pipe must be considered. Specific reductions to the pressure change factors are indicated with each design figure.

Figure 4. 8: Efficient Manhole Shaping

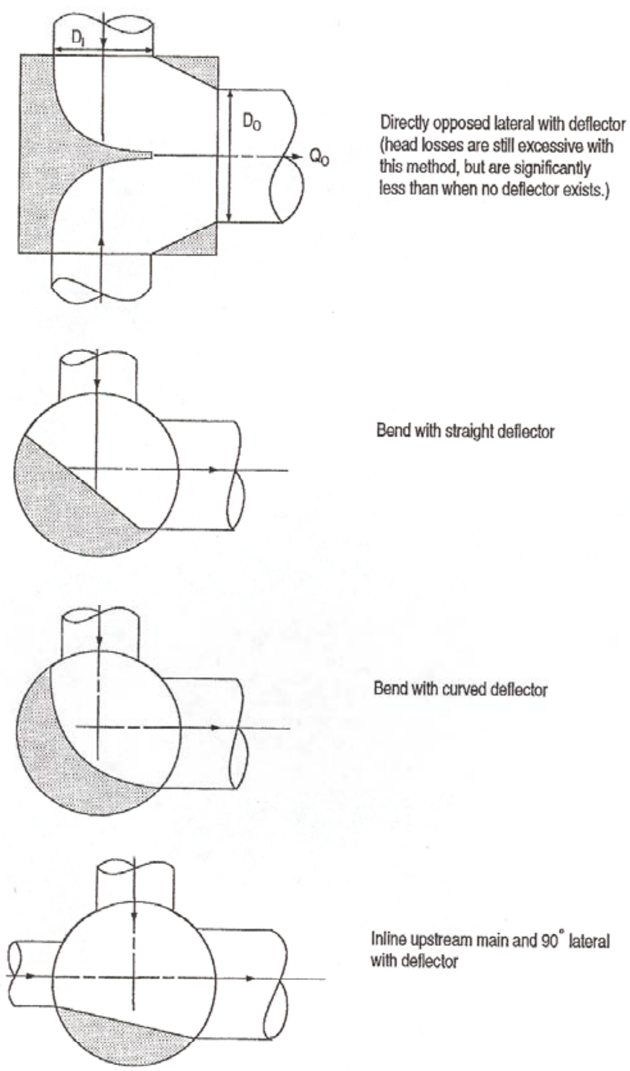
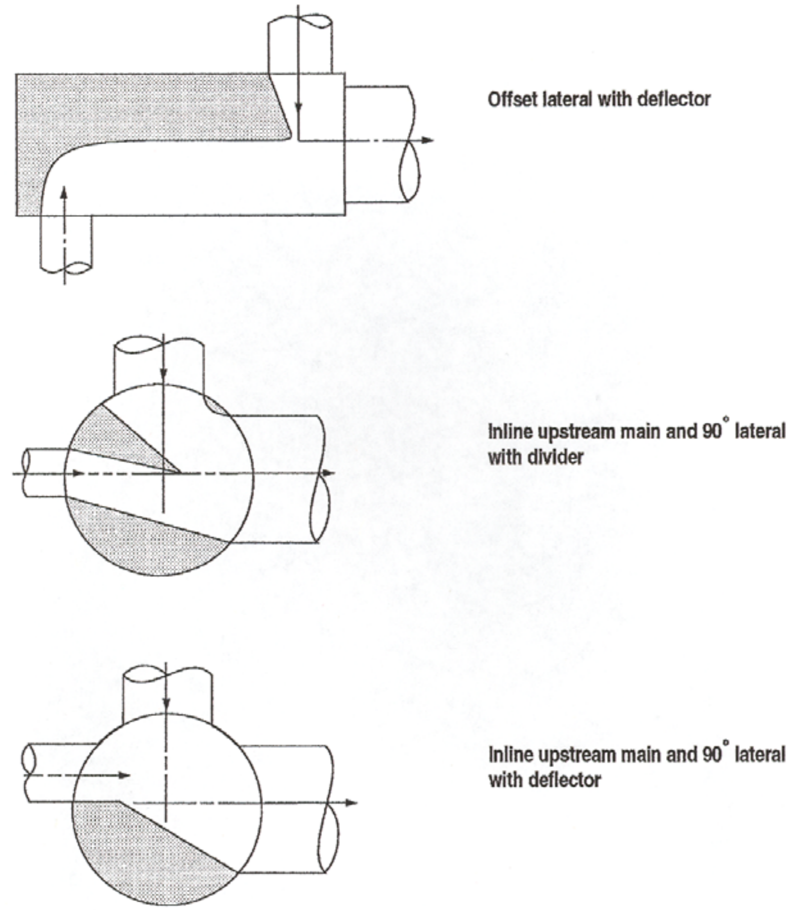


Figure 4. 9: Inefficient Manhole Shaping



VOLUME 2
DESIGN METHODOLOGY AND PROCREDURES

Chapter 5: Culverts, Bridges, and At-Grade Drainage Crossings

5.1 INTRODUCTION

Culverts and bridges are structures that convey storm water under roads. Their purpose is to prevent water from the more frequent storm events from overtopping and crossing the road as such conditions inhibit safe passage of vehicles. The intent of this chapter is to provide guidance for the design of culverts. This includes the necessary design aids and guidance for treatment of culvert inlets and outlets. Some brief guidelines are presented to follow when using inverted siphons. The design of bridges requires special training and experience in regard to hydraulic analyses, design of flow training works, and estimates of pier and abutment scour. Therefore, only an overview of the hydraulic analyses for bridge openings is presented.

5.2 CULVERTS

Culverts are primarily used for conveying runoff through a roadway embankment. They are normally aligned with a watercourse or engineered drainage channel. Culverts are typically used for smaller drainageways. They may also serve as outfall structures for stormdrain systems. Bridges are generally used for larger drainageways such as large washes and rivers.

The charts and procedures for culvert design used in this manual are taken from the Federal Highway Administration, Hydraulic Design Series Number 5, Hydraulic Design of Highway Culverts (USDOT, FHWA, HDS-5, 1985). Culvert designers use this reference liberally as it is the result of years of research and experience in culvert design and at this time represents the state of the art.

5.2.1 Design Procedure

This design method provides a convenient and organized procedure for designing culverts, considering inlet and outlet control; however, it is recommended that this procedure only be applied by individuals possessing a solid understanding of culvert hydraulics.

The first step in the design process is to summarize all known data for the culvert at the top of the Culvert Design Form (Figure 5.1). This includes establishing a maximum design headwater elevation, considering roadway overflow, roadway subgrade elevation, the finished floor elevation of any upstream structures, right-of-way or easement requirements for the backwater ponding elevation, and any potential flow diversions. This information will have been collected or calculated prior to performing the actual culvert design. The next step is to select a preliminary culvert material, shape, size, and entrance type. The user then enters the design flow rate and proceeds with the inlet control calculations.

5.2.1.1 Inlet Control

The inlet control calculations determine the headwater elevation required to pass the design flow through the selected culvert configuration if the culvert is operating in inlet control. The inlet control nomographs in Section 5.2.3 are used in the design process. For the following discussion, refer to the schematic inlet control nomograph shown in Figure 5.2.

1. Locate the selected culvert size (point 1) and flow rate (point 2) on the appropriate scales of the inlet control nomograph. (Note that for box culverts, the flow rate per foot of barrel width is used.)

2. Using a straightedge, extend a straight line from the culvert size (point 1) through the flow rate (point 2) and mark a point on the first headwater/culvert height (HW/D) scale (point 3). The first HW/D scale is also a turning line.

3. If another HW/D scale is required, extend a horizontal line from the first HW/D scale (the turning line) to the desired scale and read the result.

4. Multiply HW/D by the culvert height, D, to obtain the required headwater (HW) from the invert of the control section to the energy grade line. HW equals the required headwater depth. If trashracks are used, add trashrack losses to Hw:

5. Calculate the inlet control headwater elevation.

$$EL_{hi} = EL_i + HW \tag{5.1}$$

   Where:

   EL_i is the invert elevation at the inlet.

6. If the inlet control headwater elevation exceeds the design headwater elevation determined in the first step and tabulated on Figure 5.1, a new culvert configuration must be selected and the process repeated. Improvements to the inlet may suffice, or an enlarged barrel may be necessary, particularly if the outlet control headwater elevation calculated in the following section also exceeds the design headwater elevation.

5.2.1.2 Outlet Control

The outlet control calculations result in the headwater elevation required to convey the design discharge through the selected culvert if the culvert is operating in outlet control. The critical depth charts and outlet control nomographs of Section 5.2.3 are used in the design process. For illustration, refer to the schematic critical depth chart and outlet control nomograph shown in Figure 5.3 and Figure 5.4, respectively.

Figure 5. 1: Culvert Design Form

PROJECT : ______	STATION : ______ SHEET ____ OF ____	CULVERT DESIGN FORM DESIGNER / DATE : ______ / ______ REVIEWER / DATE : ______ / ______

HYDROLOGICAL DATA

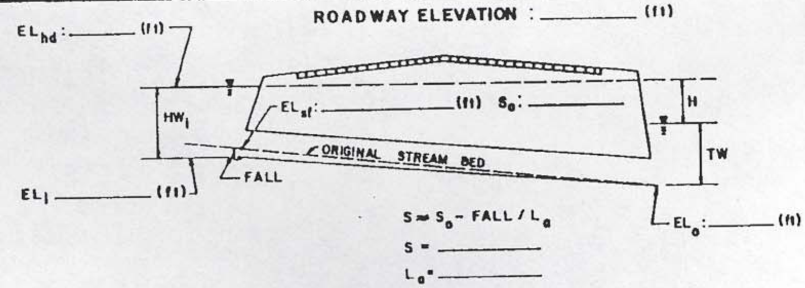
SEE ADD'L SHTS.

- ☐ METHOD : ______
- ☐ DRAINAGE AREA : ______ ☐ STREAM SLOPE : ______
- ☐ CHANNEL SHAPE : ______
- ☐ ROUTING : ______ ☐ OTHER : ______

DESIGN FLOWS/TAILWATER

R.I. (YEARS)	FLOW (cfs)	TW (ft)

ROADWAY ELEVATION : ______ (ft)



$S = S_o - FALL / L_a$

S = ______

L_a = ______

CULVERT DESCRIPTION: MATERIAL - SHAPE - SIZE - ENTRANCE	TOTAL FLOW Q (cfs)	FLOW PER BARREL Q/N (1)	HEADWATER CALCULATIONS: INLET CONTROL HW$_i$/D (2)	HW$_i$	FALL (3)	EL hi (4)	OUTLET CONTROL TW (5)	d_c	$\frac{d_c+D}{2}$	h_o (6)	k_e	H (7)	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_i/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) $H = \left[1 + k_e + (29 n^2 L)/R^{1.33}\right] V^2/2g$

(8) $EL_{ho} = EL_o + H + h_o$

SUBSCRIPT 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. STREAMBED AT CULVERT FACE
- tw. TAILWATER

COMMENTS / DISCUSSION :

CULVERT BARREL SELECTED :

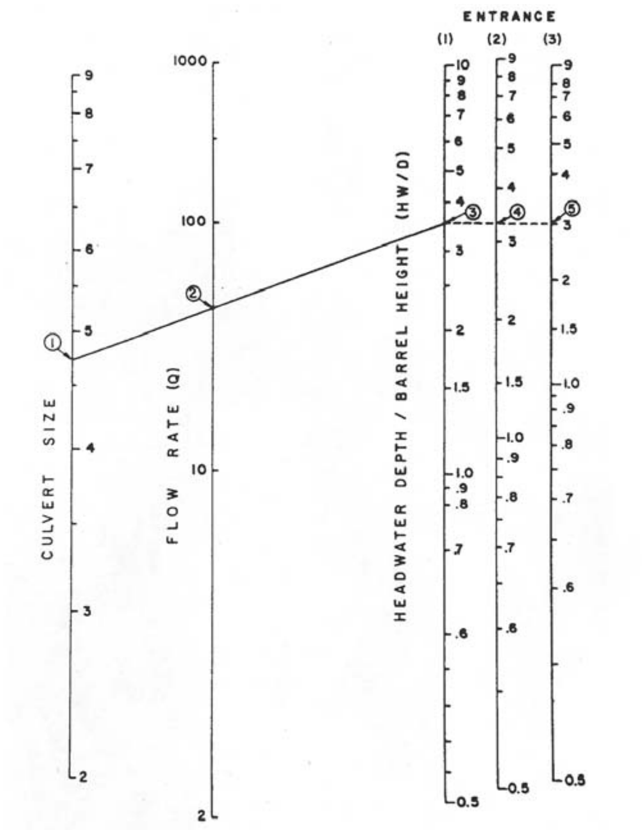
SIZE : ______

SHAPE : ______

MATERIAL : ______ n ______

ENTRANCE : ______

Figure 5. 2: Inlet Control Nomograph (Schematic)



1. Determine the tailwater depth above the outlet invert (TW) at the design flow rate. This is obtained from backwater or normal depth calculations of the downstream channel, or from field observations. Field observations are important in determining tailwater depths. The area downstream of the culvert should be examined for features that may create backwater effects, i.e., channel control, another culvert, etc. If such features are found, appropriate backwater analysis techniques should be employed to determine the tailwater depth. When culverts are in series, the headwater elevation from the downstream culvert should be checked to make sure that it doesn't back up water affecting the outlet conditions of the upstream culvert.

2. Enter the appropriate critical depth chart (Figure 5.3) with the flow rate and read the critical depth (d_c). If the computed d_c is greater than D, use D for critical depth. d_c cannot exceed the top of the culvert.

   (Note: The d_c curves are truncated for convenience when they converge. If an accurate d_c is required for d_c much greater than O.9D, consult the Handbook of Hydraulics by Brater and King, 1976, or other hydraulic references.)

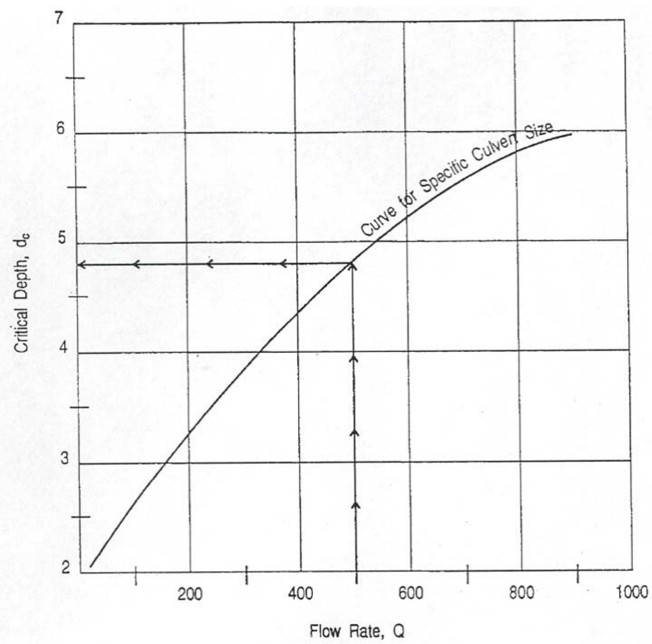
3. Calculate $(d_c + D)/2$

4. Determine the depth from the culvert outlet invert to the hydraulic grade line (h_o).

   h_o = TW or (d_c + D)/2, whichever is larger (5.2)

   From Table 5.7 obtain the appropriate entrance loss coefficient, K_e, for the culvert inlet configuration.

Figure 5. 3: Critical Depth Chart (Schematic)



5. Determine the losses through the culvert barrel, H, using the outlet control nomograph (Figure 5.4) or appropriate equations if outside the range of the nomograph.

   a) If the Manning's n value given in the outlet control nomograph is different than the Manning's n for the culvert, adjust the culvert length using the equation:

$$L_1 = L\left(\frac{n_1}{n}\right)^2 \tag{5.3}$$

   Then use L_1 rather than the actual culvert length when using the outlet control nomograph.

   b) Using a straightedge, connect the culvert size (point 1) with the culvert length on the appropriate K_e scale (point 2). This defines a point on the turning line (point 3).

c) Again using the straightedge, extend a line from the discharge (point 4) through the point on the turning line (point 3) to the Barrel Losses (H) scale. Read H, which is the energy loss through the culvert, including entrance, friction, and outlet losses.

d) All other applicable losses should be added to H.

6. Calculate the outlet control headwater elevation.

$$EL_{ho} = EL_o + H + h_o \tag{5.4}$$

Where:

EL_o is the invert elevation at the outlet.

7. If the outlet control headwater elevation exceeds the design headwater elevation determined in the first step, and tabulated on Figure 5.10, a new culvert configuration must be selected and the process repeated. Generally, an enlarged barrel will be necessary since inlet improvements are of limited benefit in outlet control.

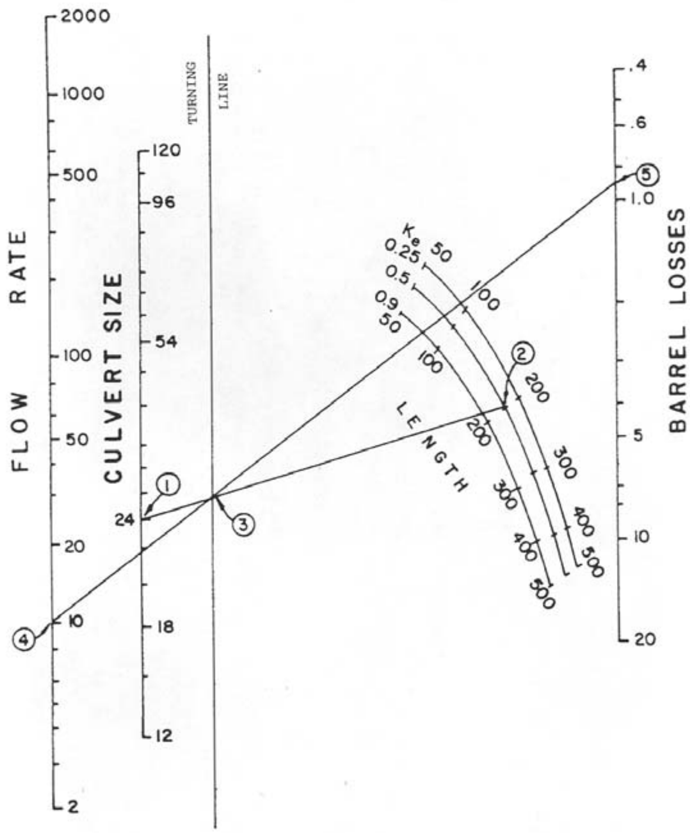
5.2.1.3 Evaluation of Results

Compare the headwater elevations calculated for inlet and outlet control. The higher of the two is designated the controlling headwater elevation. The culvert can be expected to operate with that higher headwater for at least part of the time.

The outlet velocity is calculated as follows:

1. If the controlling headwater is based on inlet control, determine the normal depth and velocity in the culvert barrel. The velocity at normal depth is assumed to be the outlet velocity (Figure 5.5). Normal depth for circular and rectangular culverts can be found using Figure 5.19.

Figure 5. 4: Outlet Control Nomographs



2. If the controlling headwater is in outlet control, determine the area of flow and velocity at the outlet based on the barrel geometry (see Figure 5.6) and the following:

   a) Critical depth, if the tailwater is below critical depth.

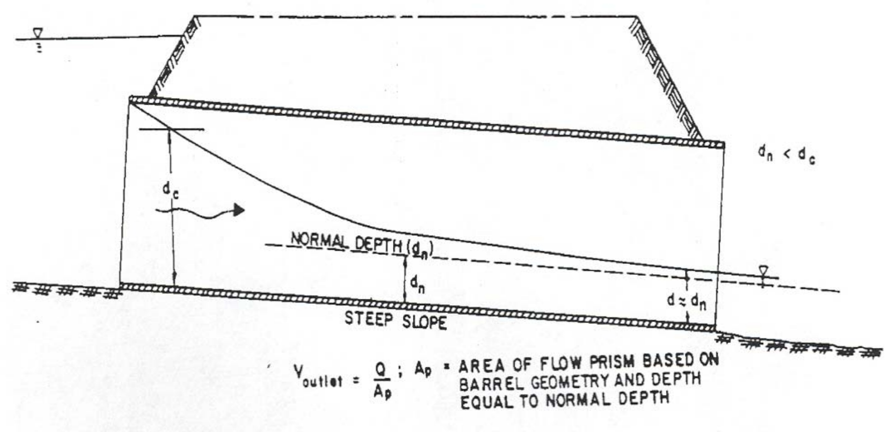
   b) The tailwater depth if the tailwater is between critical depth and the top of the barrel.

   c) The height of the barrel if the tailwater is above the top of the barrel.

Repeat the design process until an acceptable culvert configuration is determined. Once the barrel is selected it must be fitted into the roadway cross section. The culvert barrel must have adequate cover, the length should be close to the approximate length, and the headwalls and wingwalls must be dimensioned.

If outlet control governs and the headwater depth (referenced to the inlet invert) is less than 1.2D, it is possible that the barrel flows partly full through its entire length. In this case, caution should be used in applying the approximate method of setting the downstream elevation based on the greater of tailwater or (dc + D)/2. If an accurate headwater is necessary, backwater calculations should be used to check the result from the approximate method. If the headwater depth falls below O.75D, the approximate method should not be used.

Figure 5. 5: Outlet Velocity - Inlet Control



If the selected culvert will not fit the site, return to the culvert design process and select another culvert. After a selected culvert is found to meet the design conditions, document the design to this point. Culvert design documentation shall include a performance curve which displays culvert behavior over a range of discharges. Development of performance curves is presented later in this section.

Additional design considerations including stage discharge ratings, roadway overtopping, and performance curves, are discussed in the following sections.

5.2.1.4 Stage Discharge Ratings

All reservoir routing procedures require three basic data inputs:

1. an inflow hydrograph
2. a stage versus storage relationship
3. a stage versus discharge relationship.

Stage, that is elevation above some base datum, is the parameter which relates storage to discharge providing the key to the storage routing solution.

Stage versus discharge data can be computed from culvert data and the roadway geometry as described under Performance Curves. Discharge values for the selected culvert and overtopping flows are tabulated with reference to elevation. The combined discharge is utilized in the formulation of a performance curve.

Culverts are frequently used for detention basin outlet structures. The culvert design methods presented in this section can be used to develop the stage-discharge relationship for these structures. If the detention basin discharges into a stormdrain system, procedures from Section 4.3.2 should be used to establish the hydraulic grade line for that stormdrain to check for outlet control

5.2.1.5 Performance Curves

Performance curves are representations of flow rate versus headwater depth or stage for a culvert. Because a culvert has several possible control sections (inlet, outlet, throat), a given installation will have a performance curve for each control section and one for roadway overtopping. The overall culvert performance curve is made up of the controlling portions of the individual performance curves for each control section.

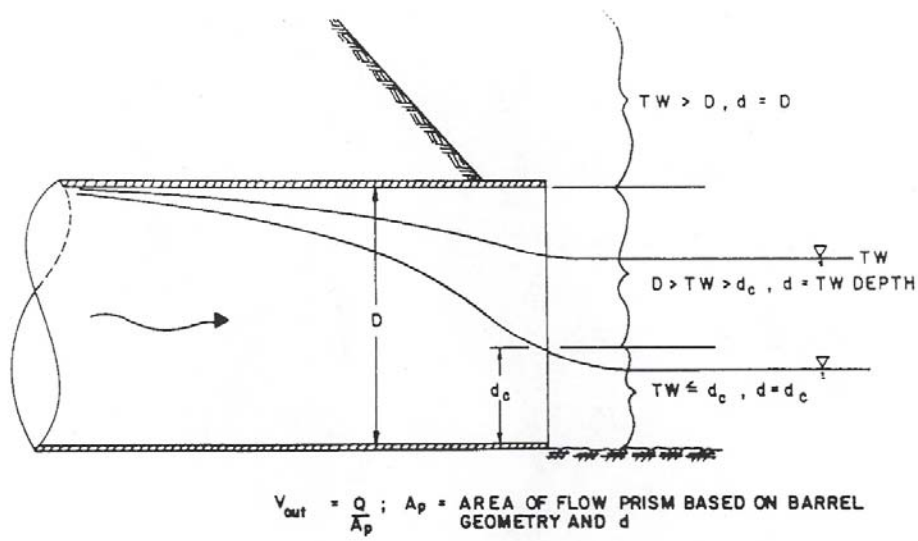
Inlet Control

The inlet control performance curves are developed using the inlet control nomographs of Section 5.2.3. The headwaters corresponding to the series of flow rates are determined and then plotted. The transition zone is inherent in the nomographs.

Outlet Control

The outlet control performance curves are developed using the outlet control nomographs of Section 5.2.3. Flows bracketing the design flow are selected. For these flows, the total losses through the barrel are calculated or read from the outlet control nomographs. The losses are added to the elevation of the hydraulic grade line at the culvert outlet to obtain the headwater.

Figure 5. 6: Outlet Velocity - Outlet Control



If backwater calculations are performed beginning at the downstream end of the culvert, friction losses are accounted for in the calculations. Adding the inlet loss to the energy grade line in the barrel at the inlet results in the headwater elevation for each flow rate.

5.2.2 Application

5.2.2.1 Criteria

Refer to Volume 1, Chapter 3, for criteria for culverts.

5.2.2.2 Skewed Channels

The angle from the culvert face to a line normal to the culvert barrel is referred to as the inlet skew angle (Figure 5.8). The structural integrity of circular sections is compromised when the inlet is skewed due to the loss of a portion of the full circular section where the culvert barrel extends beyond the full section. Although concrete headwalls help stabilize the pipe section, structural considerations should not be overlooked in the design of skewed inlets.

When high velocities exist, inlet losses resulting from turning the flow into the culvert should be considered. If backwater computations are not employed and the approach channel velocity is 6 feet per second or greater, the following equation should be used to estimate the loss. The loss should be added to the other inlet losses in the culvert design computation, if they aren't included in the appropriate nomographs.

$$H_t = \left(\frac{V_a 2}{2g}\right)\sin a \tag{5.5}$$

Figure 5. 7: Barrel Skew Angle

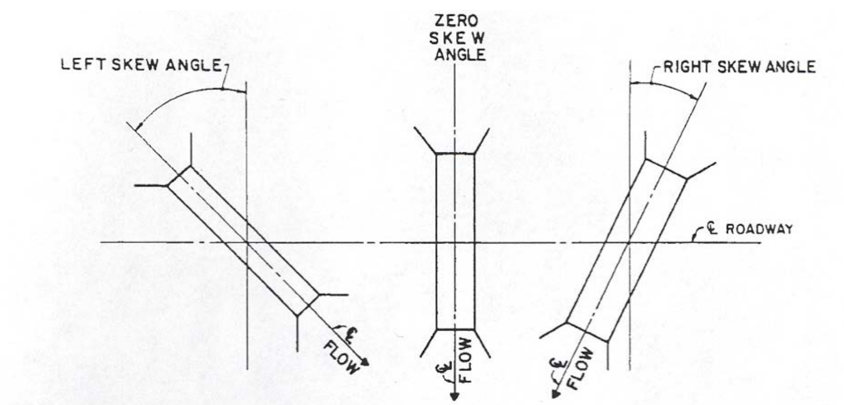


Figure 5. 8: Inlet Skew Angle

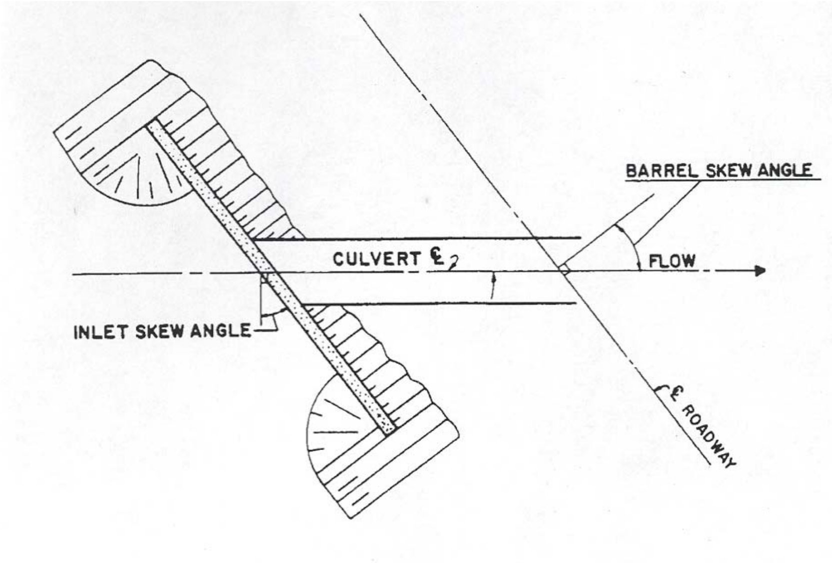
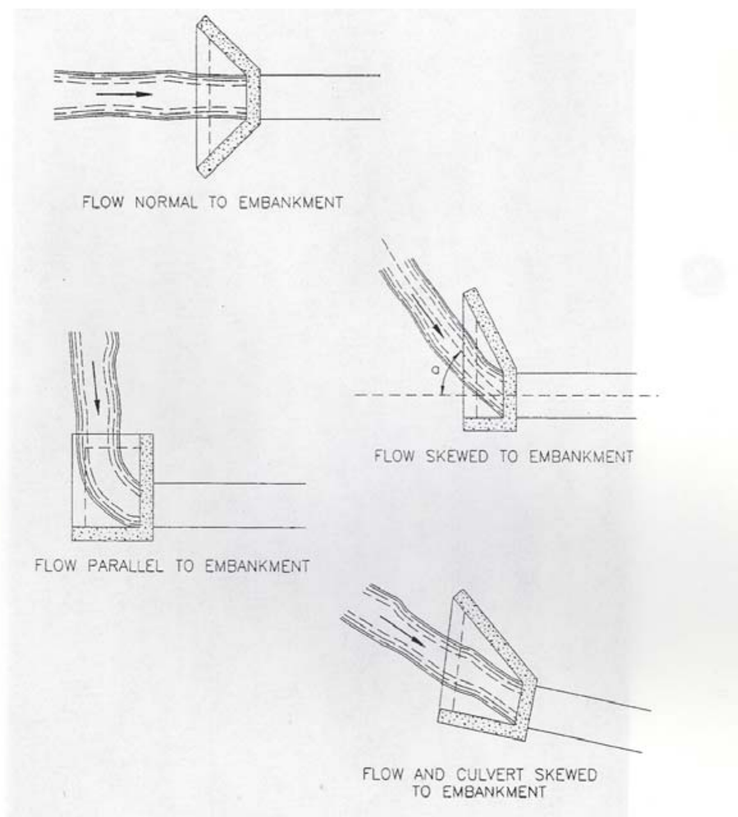


Figure 5. 9: Typical Headwall/Wingwall Configurations for Skewed Channels



5.2.2.3 Bends

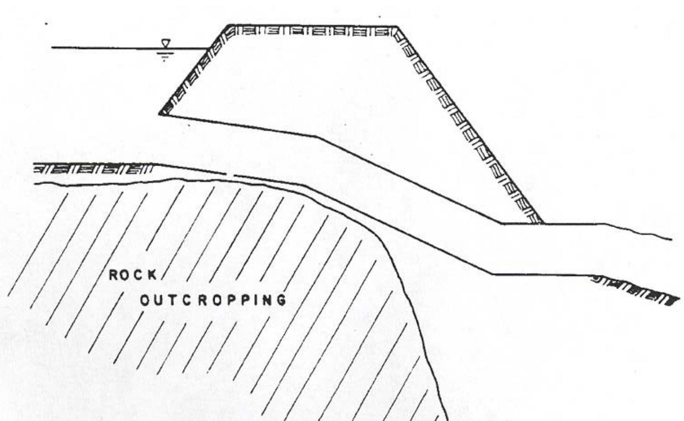
If the conditions listed in the criteria of Volume 1, Chapter 3, for bends cannot be met, analysis of bend losses is required. Bend losses are a function of the velocity head in the culvert barrel. To calculate bend losses, use the following equation:

$$H_b = K_b \frac{V^2}{2g} \tag{5.6}$$

H_b is added to the other outlet losses. See Chapter 4, Storm Drains, to determine loss coefficients (K_b) for bend losses in conduits flowing full.

The broken back culvert, shown in Figure 5.10, has four possible control sections: the inlet, the outlet, and the two bends. The upstream bend may act as a control section, with the flow passing through critical depth just upstream of the bend. In this case, the upstream section of the culvert operates in outlet control and the downstream section operates in inlet control. Outlet control calculation procedures can be applied to the upstream barrel, assuming critical depth at the bend, to obtain a headwater elevation. This elevation is then compared with the inlet and outlet control headwater elevations for the overall culvert. The controlling flow condition produces the highest headwater elevation. Control at the lower bend is very unlikely. That possible control section can be ignored except for the bend losses in outlet control.

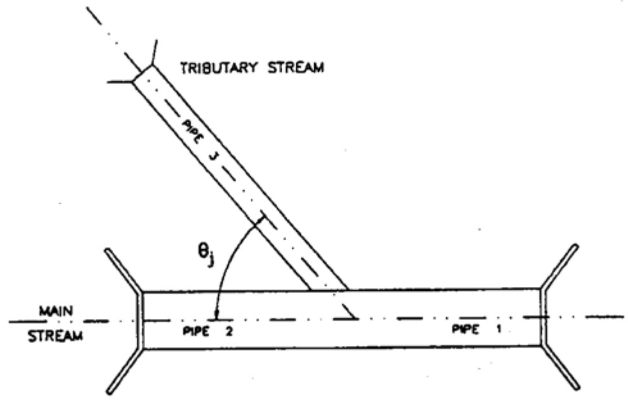
Figure 5. 10: "Broken Back" Culvert



5.2.2.4 Junctions

Flow from two or more separate culverts or stormdrains may be combined at a junction into a single culvert barrel. For example, a tributary and a main stream intersecting at a roadway crossing can be accommodated by a culvert junction (Figure 5.11).

Figure 5. 11: Culvert Junction



Loss of head may be important in the hydraulic design of a culvert containing a junction. Attention should be given to streamlining the junction to minimize turbulence and head loss. Also, timing of peak flows from the two branches should be considered in analyzing flow conditions and control. When possible, the tributary flow should be released downstream of the culvert barrel. When this is not practical, the following procedure should be used to estimate the losses.

For a culvert barrel operating in outlet control and flowing full, the junction loss is calculated using the equations given below. The loss is then added to the other outlet control losses.

$$H_j = y' + H_{v1} - H_{v2} \tag{5.7}$$

The equation for y' is based on momentum considerations and is as follows:

$$y' = \frac{Q_2 V_2 - Q_1 V_1 - Q_3 V_3 \cos\theta_j}{0.5(A_1 + A_2)g} \tag{5.8}$$

The subscripts 1, 2, and 3 refer to the outlet pipe, the upstream pipe, and the lateral pipe respectively.

5.2.2.5 Trashracks and Access Barriers

Refer to Volume 1, Chapter 3, for criteria related to trashracks and access barriers.

For trash racks with approach velocities less than 3 feet per second, it is not necessary to include a head loss for the trash rack; however, for velocities greater than 3 feet per second, such computations are required. See Hydraulic Structures, Chapter 8, Section 8.1.5.

5.2.2.6 Flotation and Anchorage

Refer to Volume 1, Chapter 3, for criteria related to flotation and anchorage.

5.2.2.7 Safety

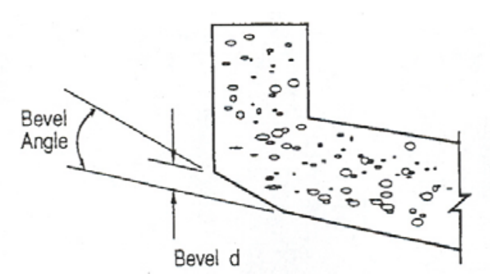
Culverts shall be designed to conform to the safety protocols identified in Volume 1, Chapter 1.

5.2.2.8 Inlets

Culvert inlets are used to transition the flow from a headwater condition upstream of the culvert into the culvert barrel. Losses caused by the inlets have been studied extensively for several types of inlets. The inlet control nomographs in Section 5.2.3 give the required headwater depth to pass the design discharge through several types of culvert entrances. The hydraulic capacity of a culvert may be improved by appropriate inlet selection. Since the channel is usually wider than the culvert barrel, the culvert inlet edge represents a flow contraction and may be the primary flow control. The provision of a more gradual flow transition will lessen the energy loss and thus create a more hydraulically efficient inlet condition. Design charts for improved inlets are contained in Hydraulic Design of Highway Culverts (USDOT, FHWA, HDS No.5, September 1985). It should be noted that improving culvert inlets will cause the greatest increase in culvert capacity when the culvert is operating in inlet control. The hydraulic performance of culverts operating in inlet control can be improved by changing the inlet geometry of the headwall. Improvements include bevel-edged, side-tapered, and slope-tapered inlets. The advantage of these improvements is to convert an inlet control culvert closer to outlet control by using more of the barrel capacity.

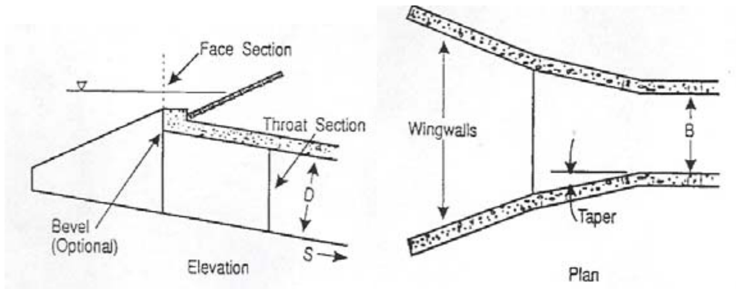
A beveled-edge provides a decrease in flow contraction losses at the inlet and the entrance loss coefficient, K_e is normally reduced to 0.2, which can increase the culvert capacity by as much as 20 percent. Bevels are required on all culverts with headwalls and should be constructed as shown in Figure 5.12.

Figure 5. 12: Inlet Bevel Detail



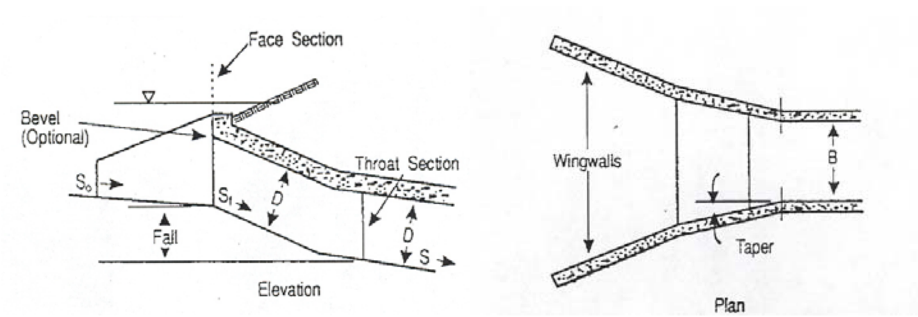
Side-tapered inlets have an enlarged face area accomplished by tapering sidewalls as shown in Figure 5.13. It provides an increase in flow capacity of 25 to 40 percent over square-edged inlets. There are two types of control sections for side-tapered inlets; face and throat control. The advantages of side-tapered inlets under throat control are; reduced flow contraction at the throat and increased head at the throat control section.

Figure 5. 13: Side-Tapered Inlet



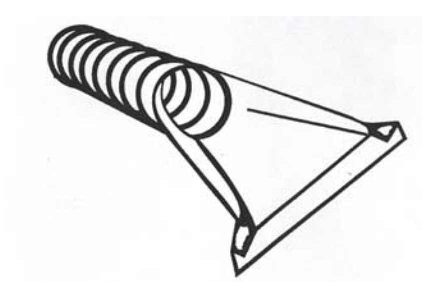
Slope-tapered inlets provide additional head at the throat section as shown in Figure 5.14. This type of inlet can have over 100 percent greater capacity than a conventional culvert with square edges. The degree of increased capacity depends upon the drop between the face and the throat section. Both the face and the throat are possible control sections. The inlet face should be designed with a greater capacity than the throat to promote flow control at the throat and therefore greater potential capacity of the culvert. This type of inlet may not be appropriate for flows containing high sediment loads; caution should be excised for this design condition.

Figure 5. 14: Slope-Tapered Inlet



Prefabricated steel inlet end sections (Figure 5.15) are available for corrugated steel pipe that perform about as well as a square-edged headwall inlet with an entrance loss coefficient of 0.5.

Figure 5. 15: Prefabricated Culvert End Section



When there is a potential for inlet uplift failure or inlet damage from other sources, concrete headwalls are recommended. In some cases, such as when concrete encasement of the pipe is utilized, metal end sections such as the one shown in Figure 5.15 may be acceptable.

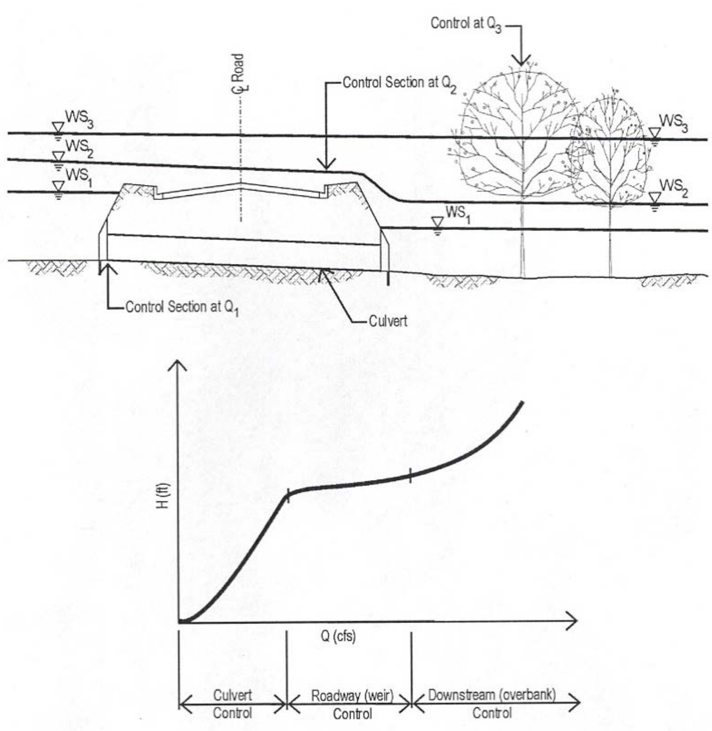
5.2.2.9 Outlets

Refer to Volume 1, Chapter 3, for criteria related to culverts. Culvert outlet designs are presented in Section 5.3. Energy dissipation structures, if needed are presented in Chapter 8 Hydraulic Structures, Section 8.3.

5.2.2.10 Roadway Overtopping

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 performance curve depicts the sum of the flow through the culvert and the flow across the roadway.

Figure 5. 16: Culvert Performance Curve with Roadway Overtopping



The overall performance curve can be determined by performing the following steps:

1. Select a range of flow rates and determine the corresponding headwater elevations for the culvert flow alone. These 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. It is recommended that the 2-, 10-, 50- and 100-year flow rates be included in the range of flow rates considered.

2. Combine the inlet and outlet control performance curves to define a single performance curve for the culvert based on the controlling stage for each discharge.

3. When the culvert headwater stages 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 (5.9) or Equation (5.10) to calculate flow rates across the roadway.

4. Add the culvert flow and the roadway overtopping flow at the corresponding headwater elevations to obtain the overall culvert performance curve.

Using the combined culvert performance curve, it is an easy matter to determine the headwater stage for any flow rate, or to visualize the performance of the culvert installation over a range of flow rates. When roadway overtopping begins, the rate of headwater increase will diminish. The headwater will rise very slowly from that point on. Figure 5.16 depicts an overall culvert performance curve with roadway overtopping. The 100-year discharge should be identified on the performance curve and the corresponding depth of flow over the roadway.

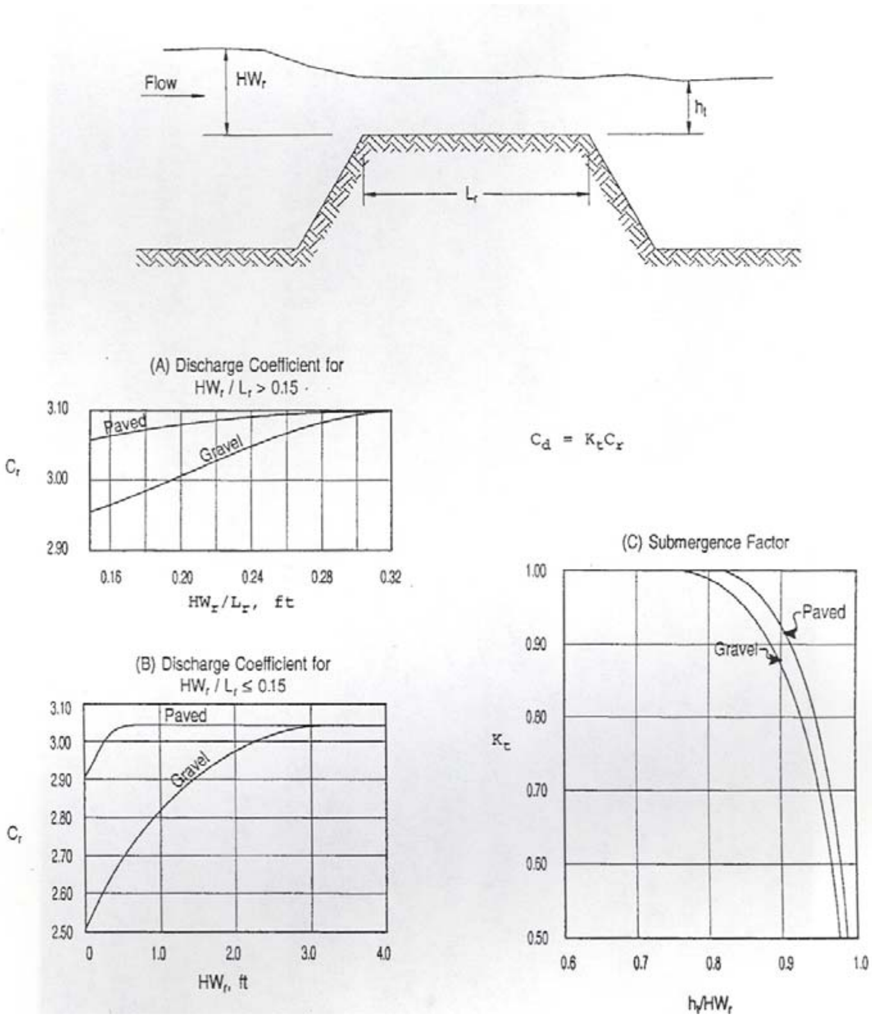
The Federal Highway Administration's computer program, HY8 (USDOT, 1999), can be used in the development of performance curves. HY8 automates the design methods described in HDS-5 (USDOT, 1985), and HEC-14 (USDOT, 1983). The U.S. Army Corps of Engineers HEC-2 (USACOE, 1990) and HEC-RAS computer programs (USACOE, 2001a and 2001b) are also capable of analyzing culverts. The use of HY8 is preferred for design of culverts that are not subject to backwater conditions. HEC-RAS is preferred for modeling and design of culverts in river systems where backwater effects are of concern.

Roadway overtopping will begin as the headwater rises to the elevation of the lowest point of the roadway. This type of flow is similar to flow over a broad crested weir. The length of the weir can be taken as the horizontal length along the roadway. The flow across the roadway is calculated from the broad crested weir equation:

$$Q_o = K_t C_r L_x (HW_r)^{1.5} \qquad (5.9)$$

The charts in Figure 5.17 provide estimates of the correction factors K_t and C_r.

Figure 5. 17: Discharge Coefficient and Submerge Factor for Roadway Overtopping



If the elevation of the roadway crest varies, for instance where the crest is defined by a roadway sag vertical curve, the vertical curve can be approximated as a series of horizontal segments. The flow over each is calculated separately and the total flow across the roadway is the sum of the incremental flows for each segment (Figure 5.18). If the assumption of horizontal segments is invalid ($HW_{ra} > 1.5\ HW_{rb}$), the following formula may be used, assuming the value of C_r remains constant:

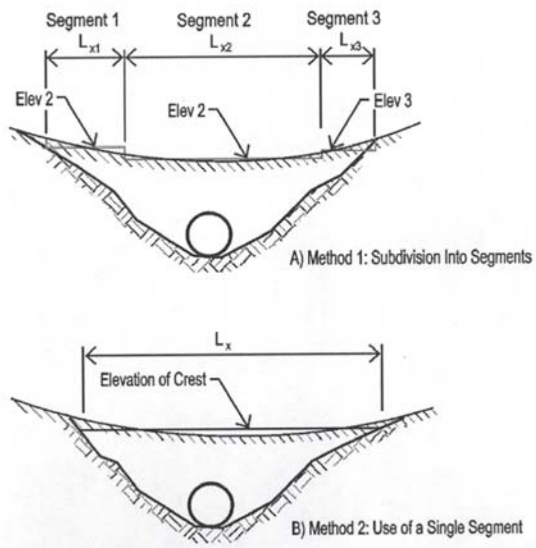
$$Q_o = \frac{2K_t C_r L_x (HW_{rb}^{\ 5/2} - HW_{ra}^{\ 5/2})}{5(HW_{rb} - HW_{ra})} \tag{5.10}$$

Where:

HW_{ra} = flow depth above roadway at the high end of the weir segment, ft.
HW_{rb} = flow depth above roadway at the low end of the weir segment, ft.
Adapted from Hulsing (1968).

The total flow across the roadway then equals the sum of the roadway overflow plus the culvert flow. A performance curve must be plotted including both culvert flow and road overflow. The headwater depth for a specific discharge, such as the 100-year discharge can then be read from the curve.

Figure 5. 18: Weir Crest Length Determination for Roadway Overtopping



5.2.3 Design Aids

Computer programs for culvert design are acceptable provided they are based on US DOT, FHWA, HDS-5, 1985.

The Culvert Design Form (Figure 5.1) has been formulated to guide the user through the design process. Summary blocks are provided at the top of the form for the project description, and the designer's identification. Summaries of hydrologic data are also included. At the top right is a small sketch of a culvert with blanks for inserting important dimensions and elevations.

The central portion of the design form contains lines for inserting the trial culvert description and calculating the inlet control and outlet control headwater elevations. Space is provided at the lower center for comments and at the lower right for a description of the culvert barrel selected. The design chart should be completely filled out, including consideration of inlet and outlet control. Table 5.1 and Figure 5.19 through Figure 5.38 should facilitate completion of the Culvert Design Form.

Table 5- 1: Entrance Loss Coefficients

Outlet Control, Full or Partly Full Entrance Head Loss
(USDOT, FHWA, HDS-5, 1985)

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 (grove-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, 33.7° or 45° bevels	0.2
Side-or slope-tapered inlet	0.2
Pipe, or Pipe-Arch, Corrugated Metal	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls square-edge	0.5
Mitered to conform to fill slope, paved or unpaved slope	0.7
End-Section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° 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 on sides	0.2
Wingwalls at 30° to 75° to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of 1/12 barrel dimension, or beveled top edge	0.2
Wingwalls at 10° to 25° 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

Figure 5. 19: Curves for Determining the Normal Depth

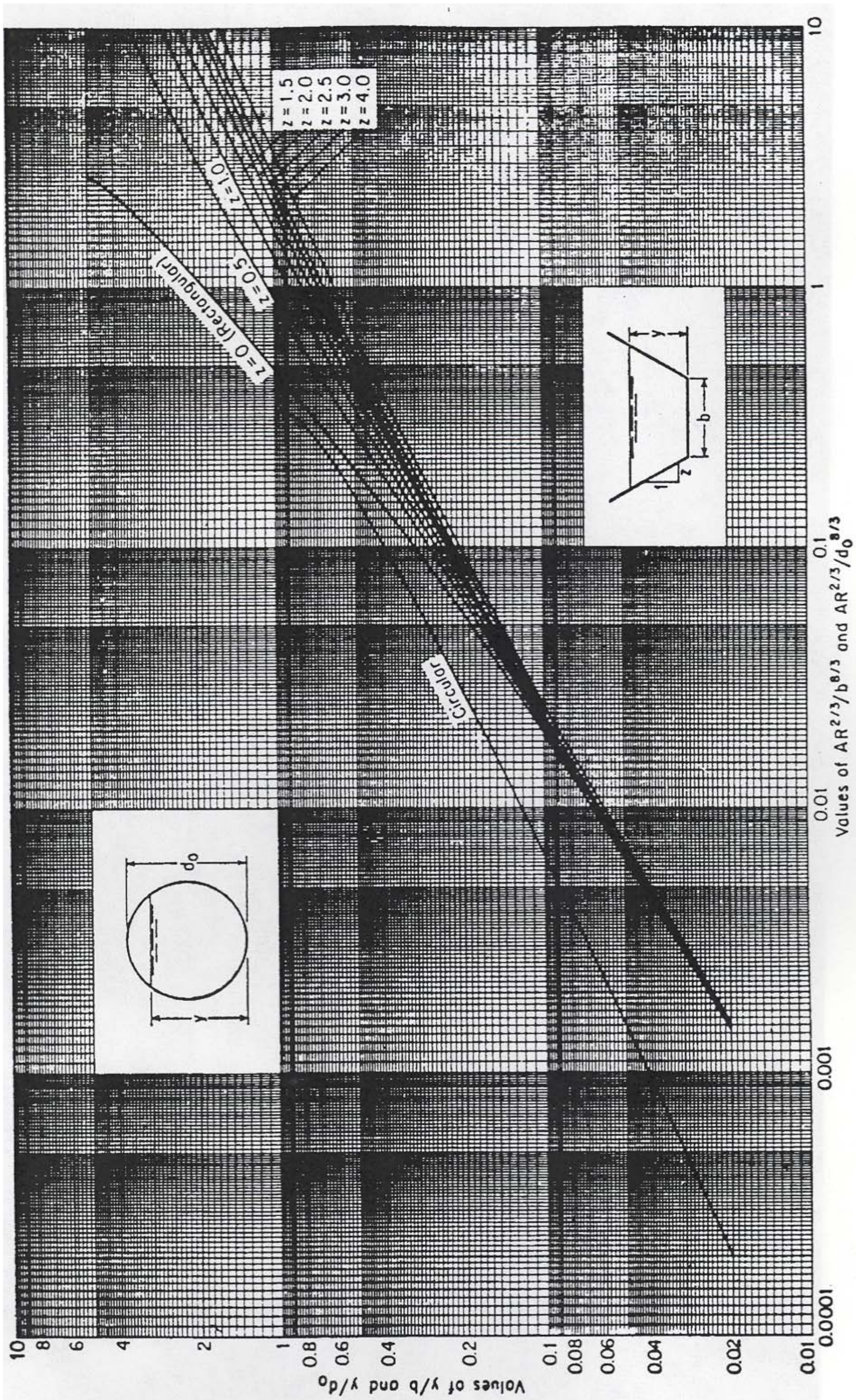


Figure 5. 20: Inlet Control Headwater Depth for Concrete Pipe Culverts

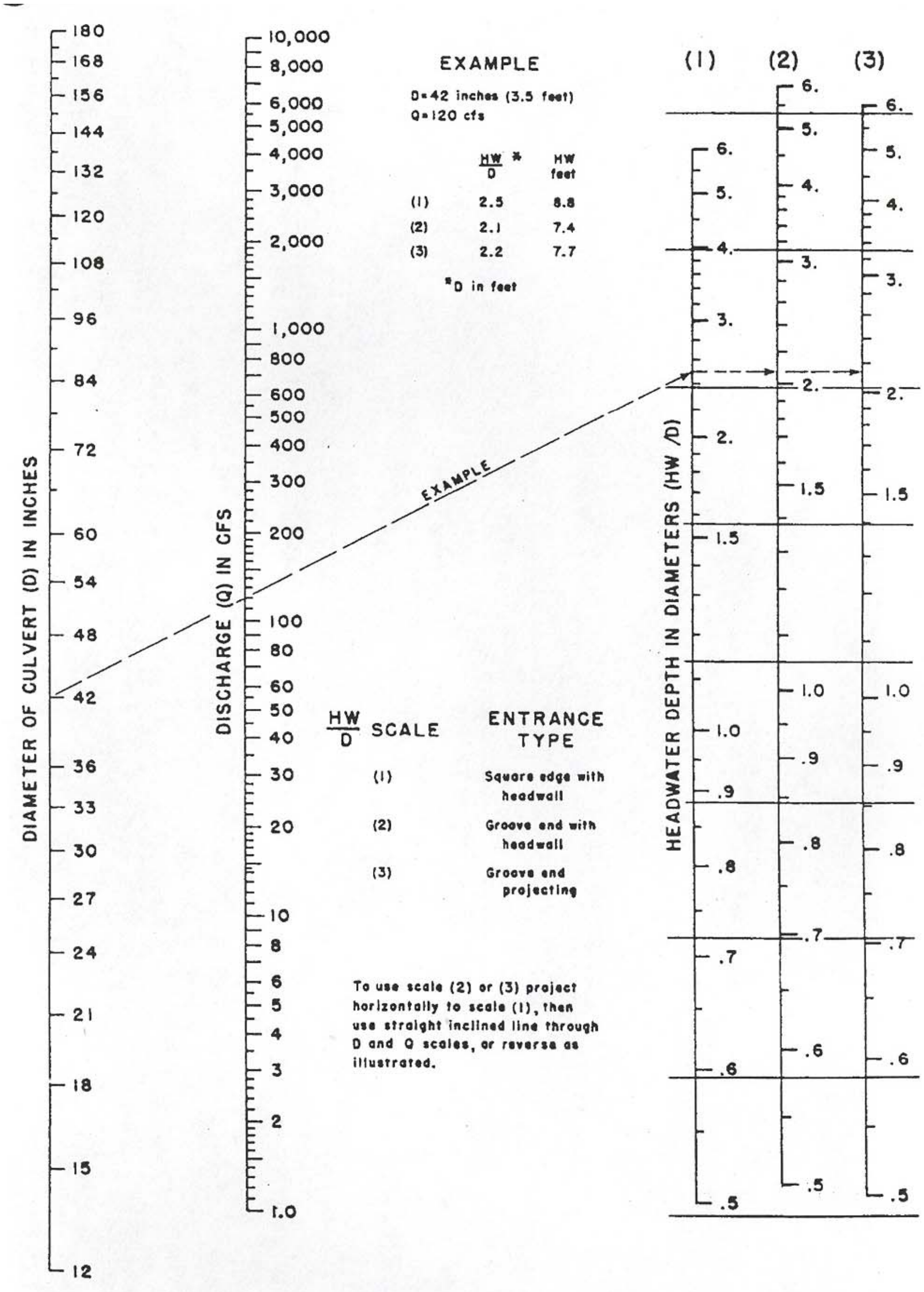


Figure 5. 21: Inlet Control Headwater Depth for C.M. Pipe

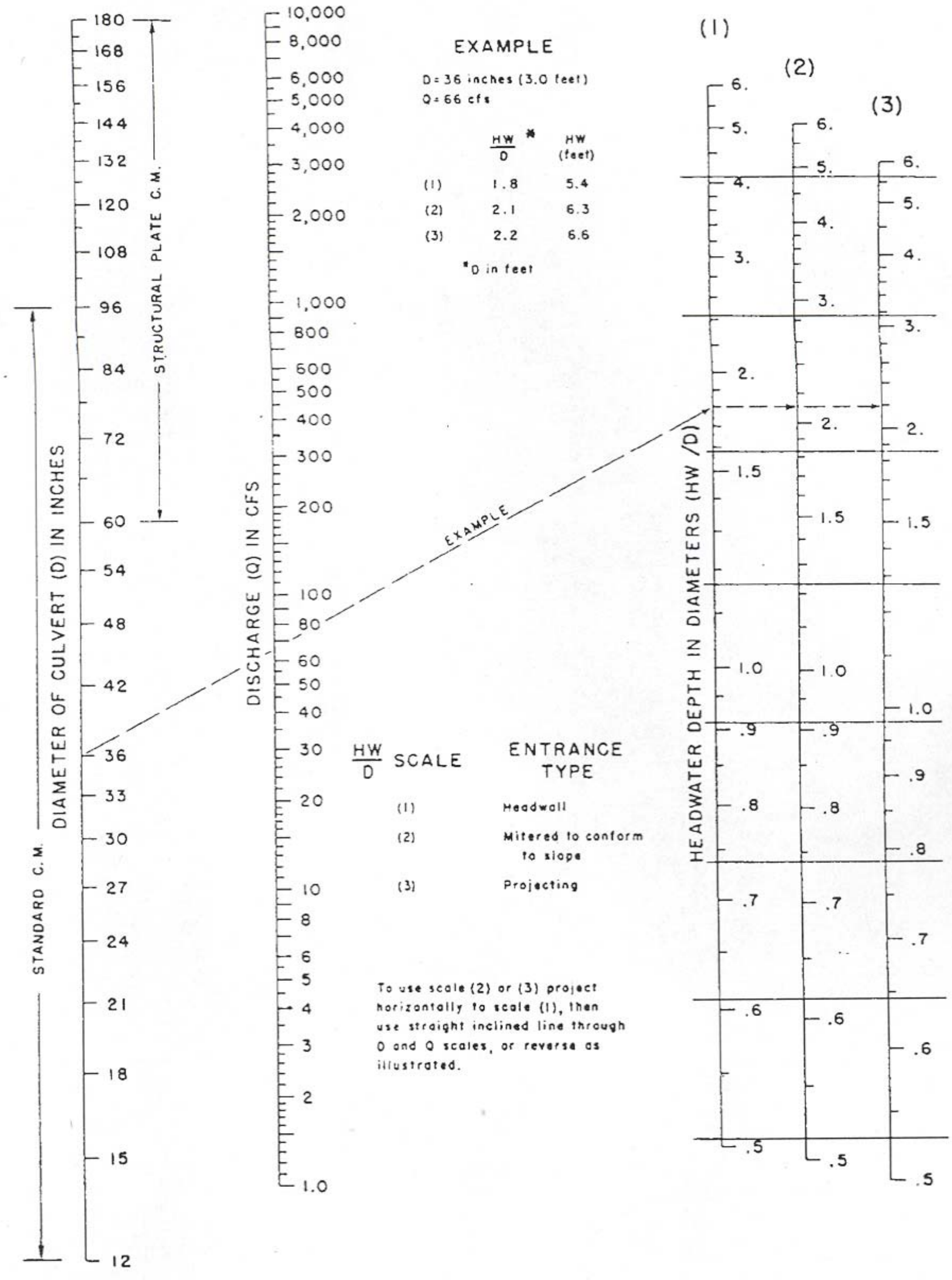


Figure 5. 22: Inlet Control Headwater Depth for Circular Pipe Culverts with Beveled Ring

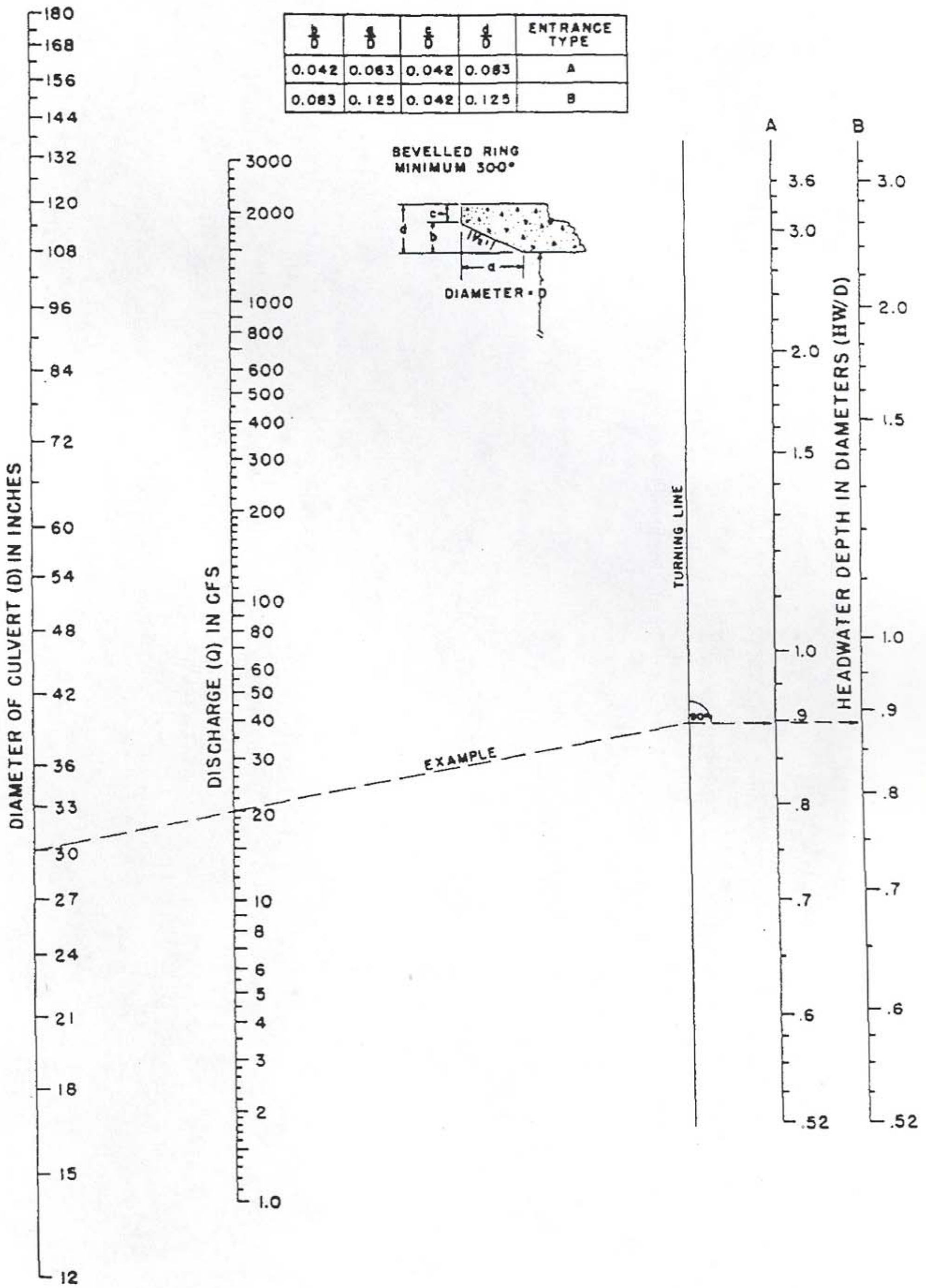


Figure 5. 23: Critical Depth for Circular Pipe

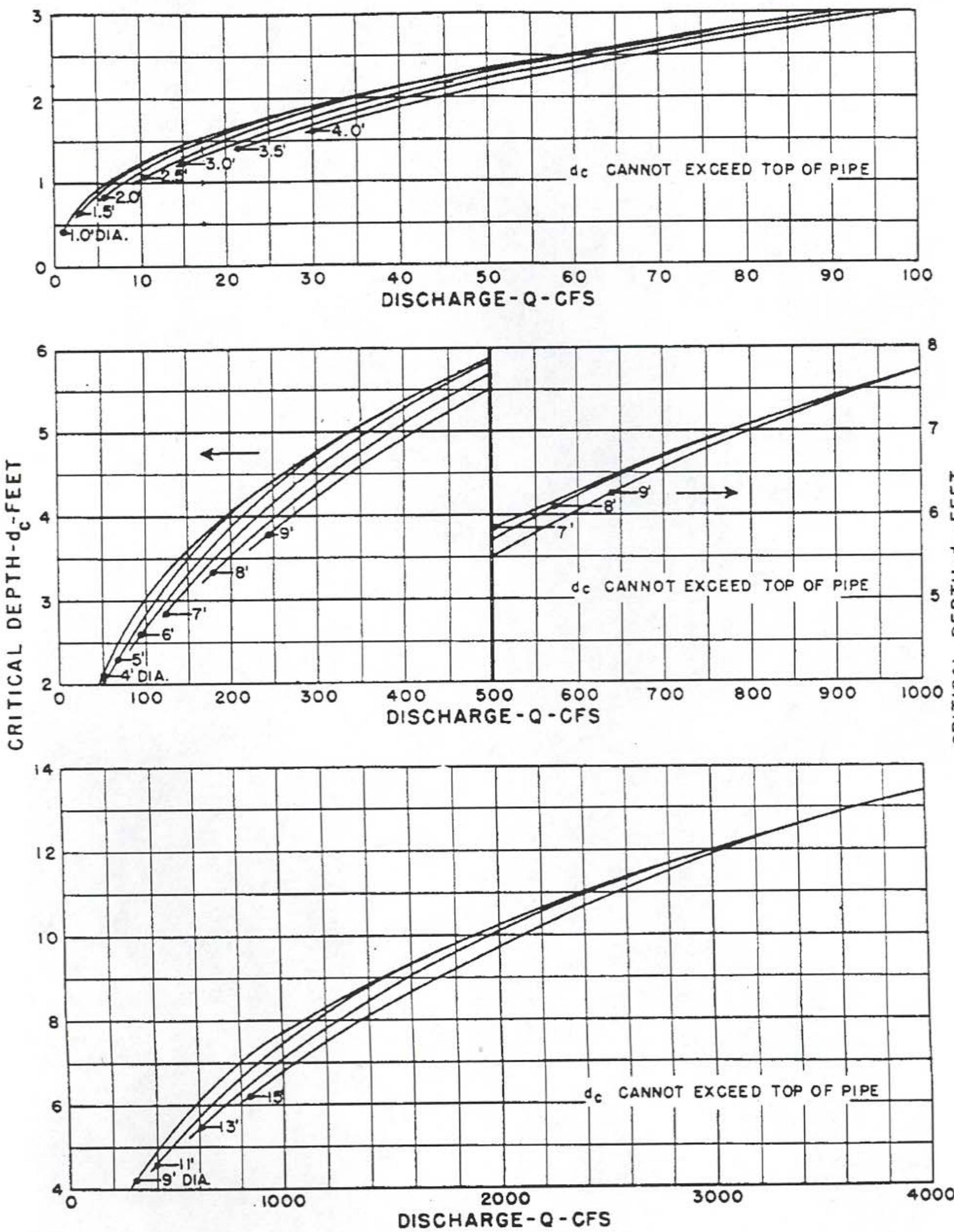


Figure 5. 24: Head for Concrete Pipe Culverts Flowing Full

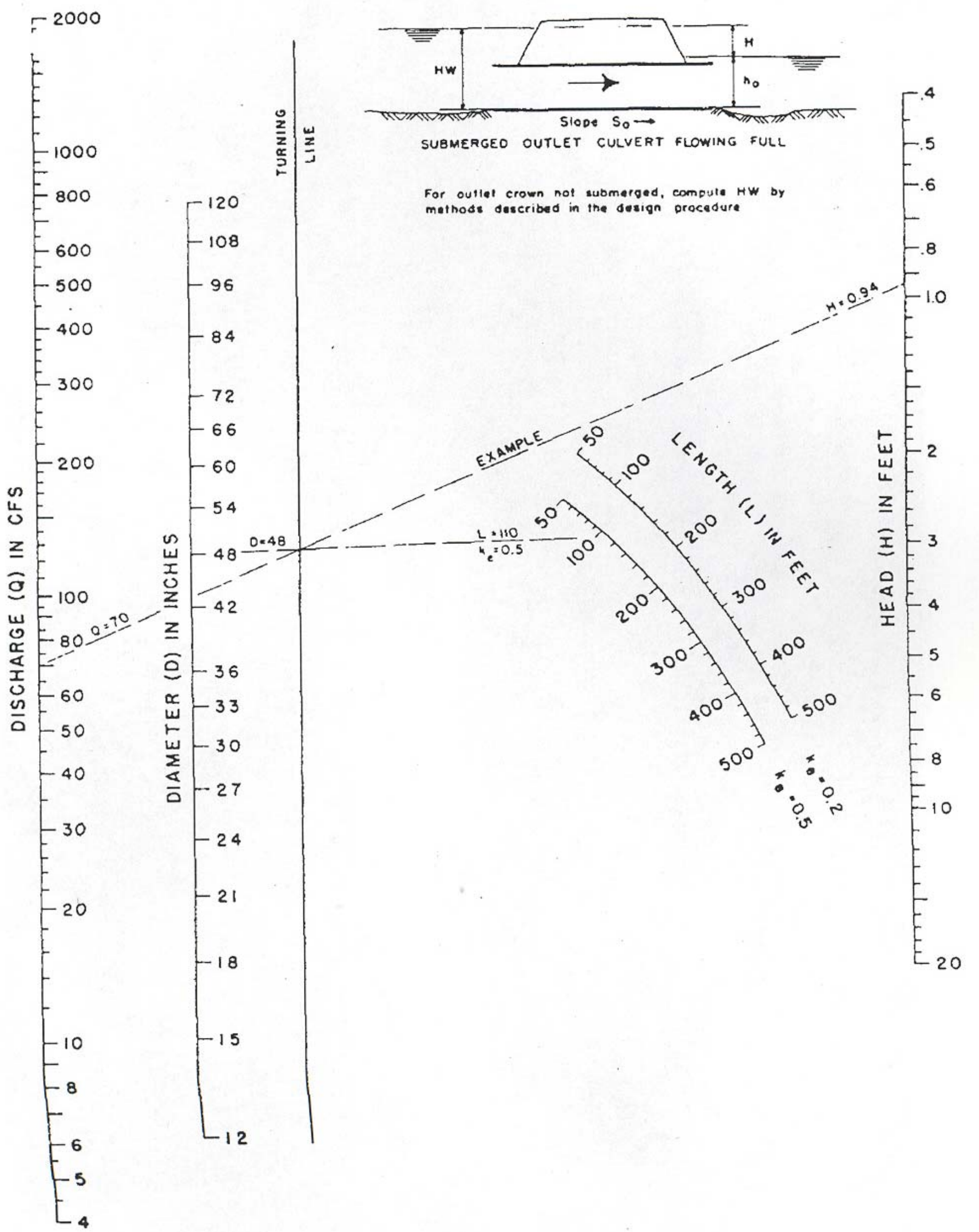


Figure 5. 25: Head for C.M. Pipe Culverts Flowing Full

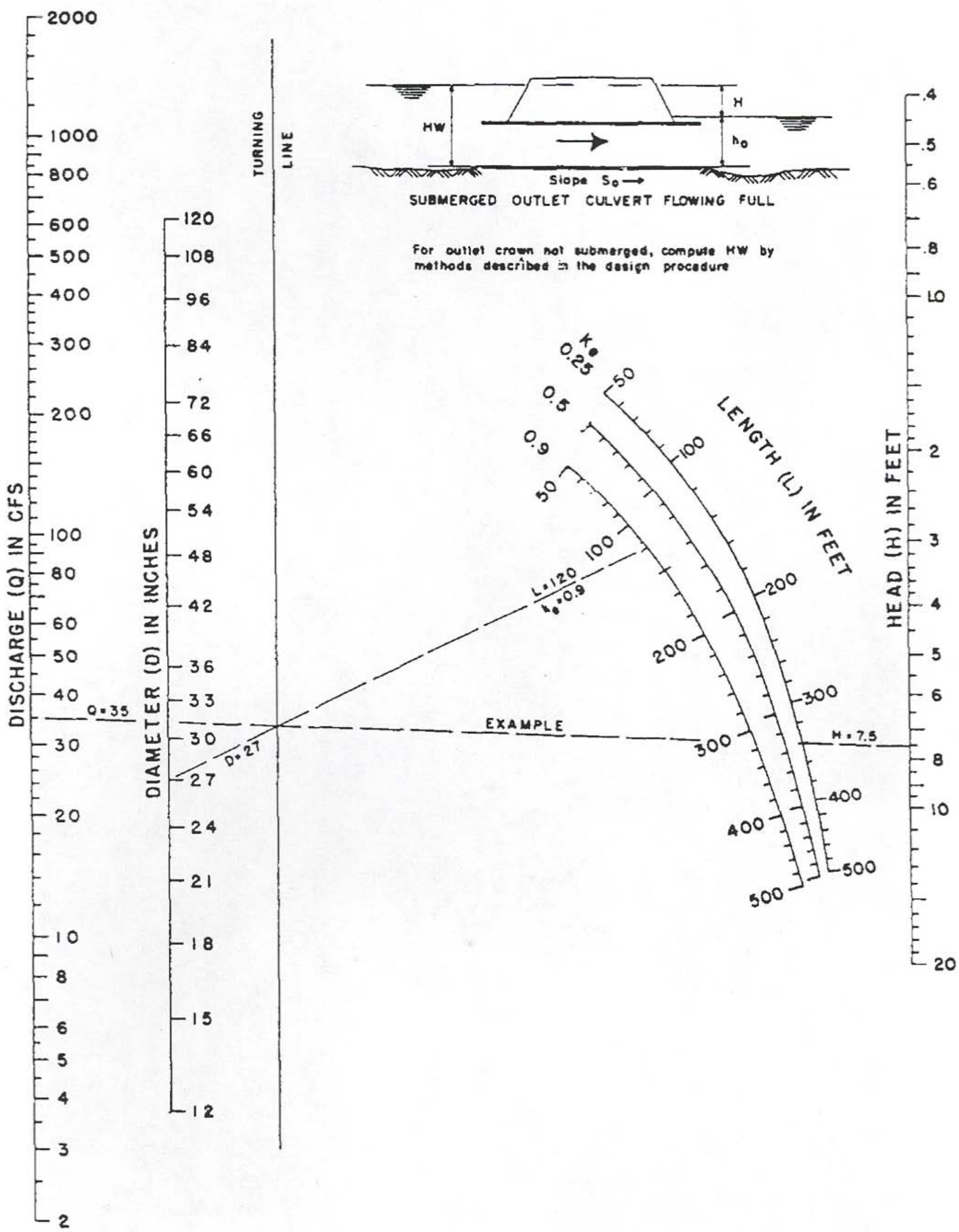


Figure 5. 26: Inlet Control Headwater Depth for Box Culverts

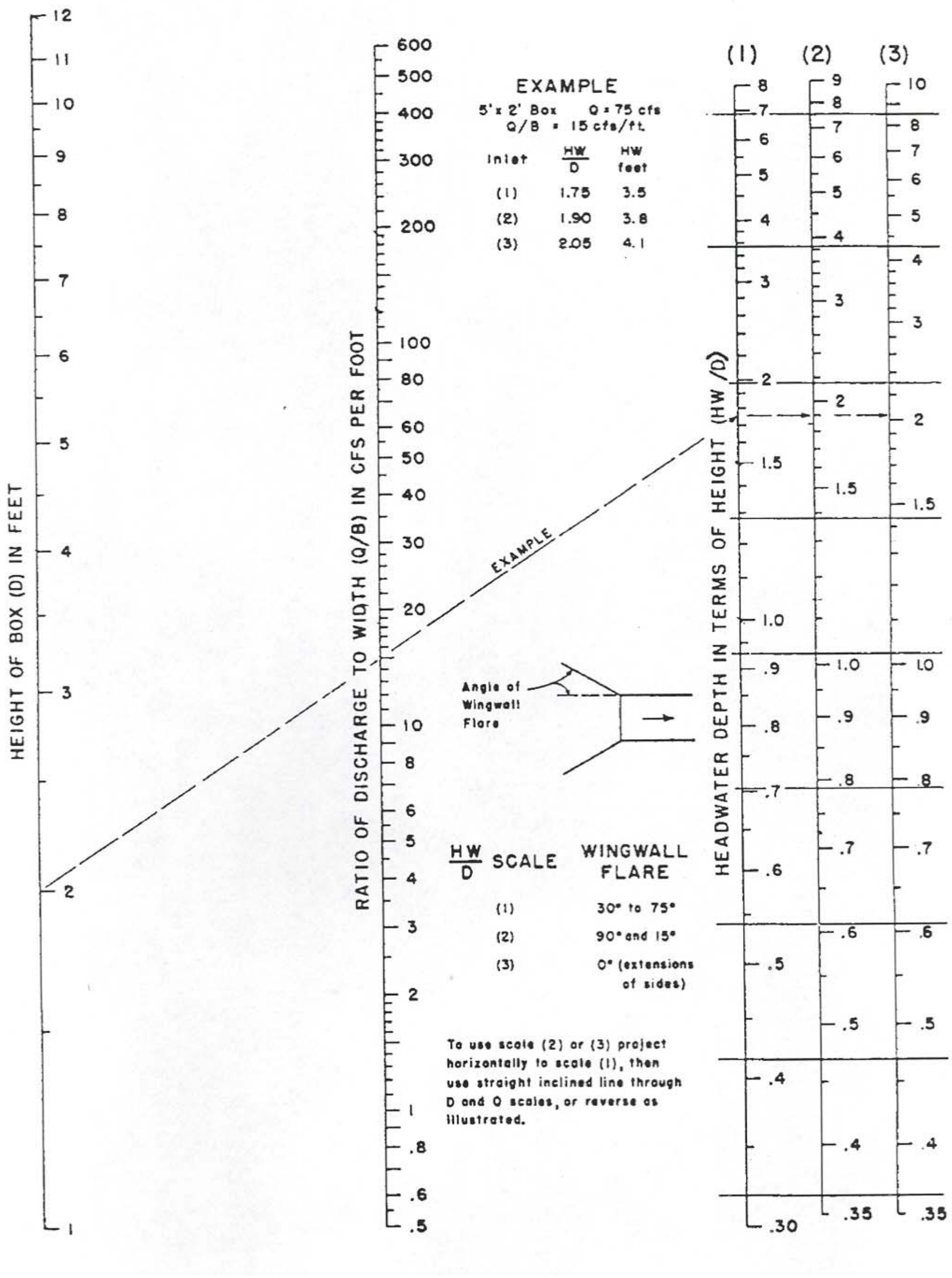


Figure 5. 27: Inlet Control Headwater Depth for Rectangular Box Culvert

Flared Wingwalls (18° to 33.7° and 45°) and Beveled Edge at Top of Inlet

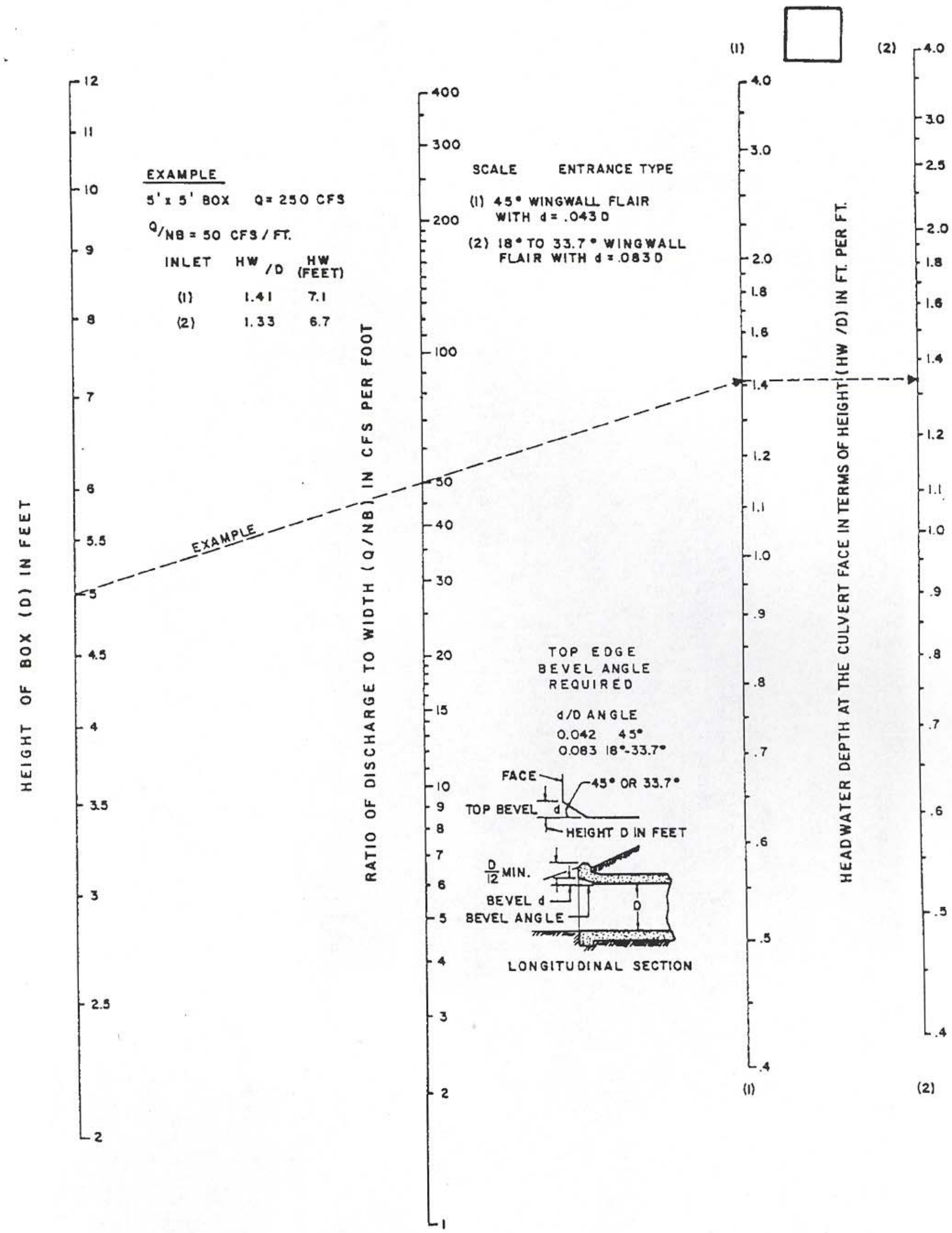


Figure 5. 28: Inlet Control for Rectangular Box Culvert (90° Headwall)

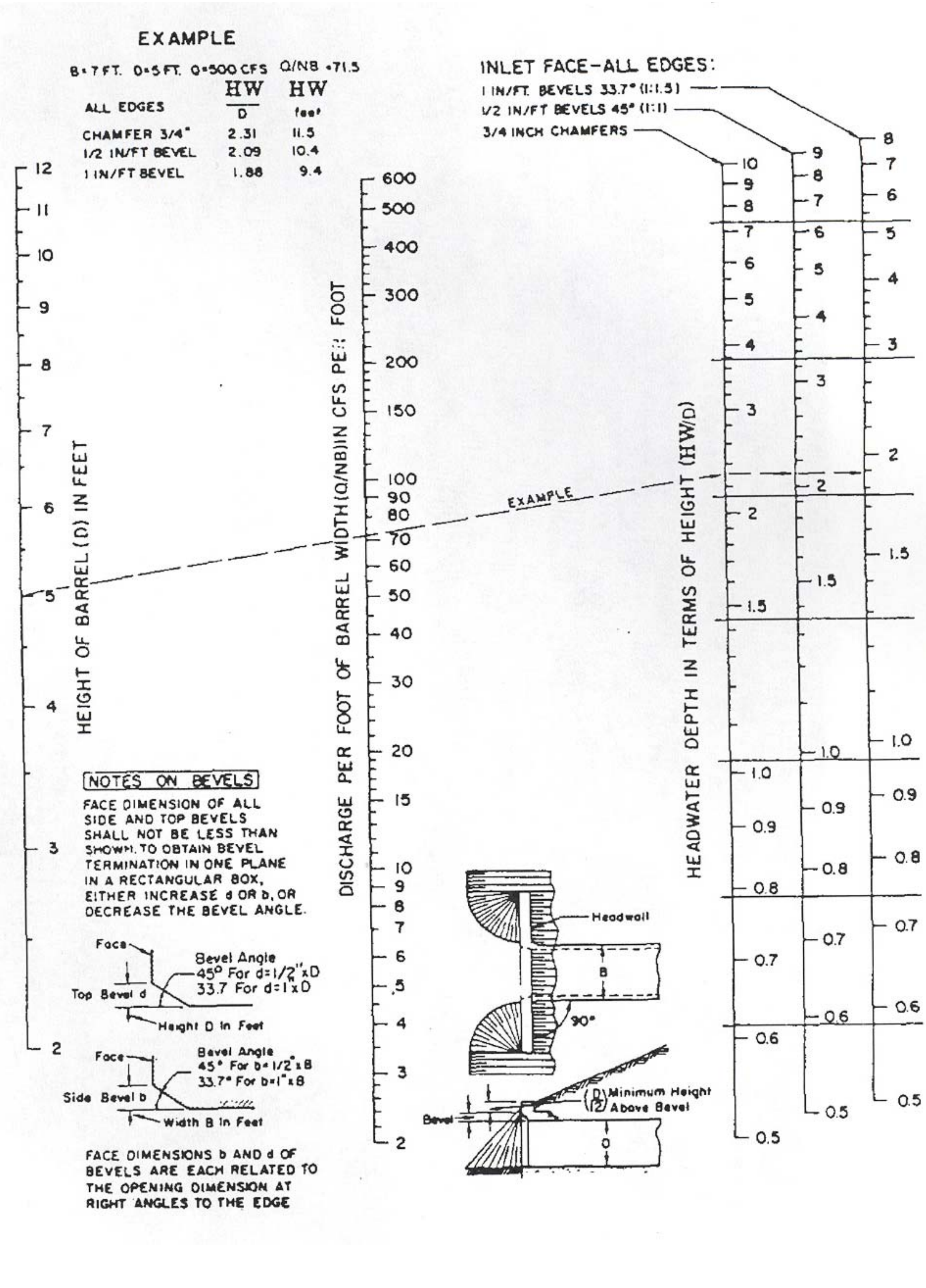


Figure 5. 29: Critical Depth Rectangula Section

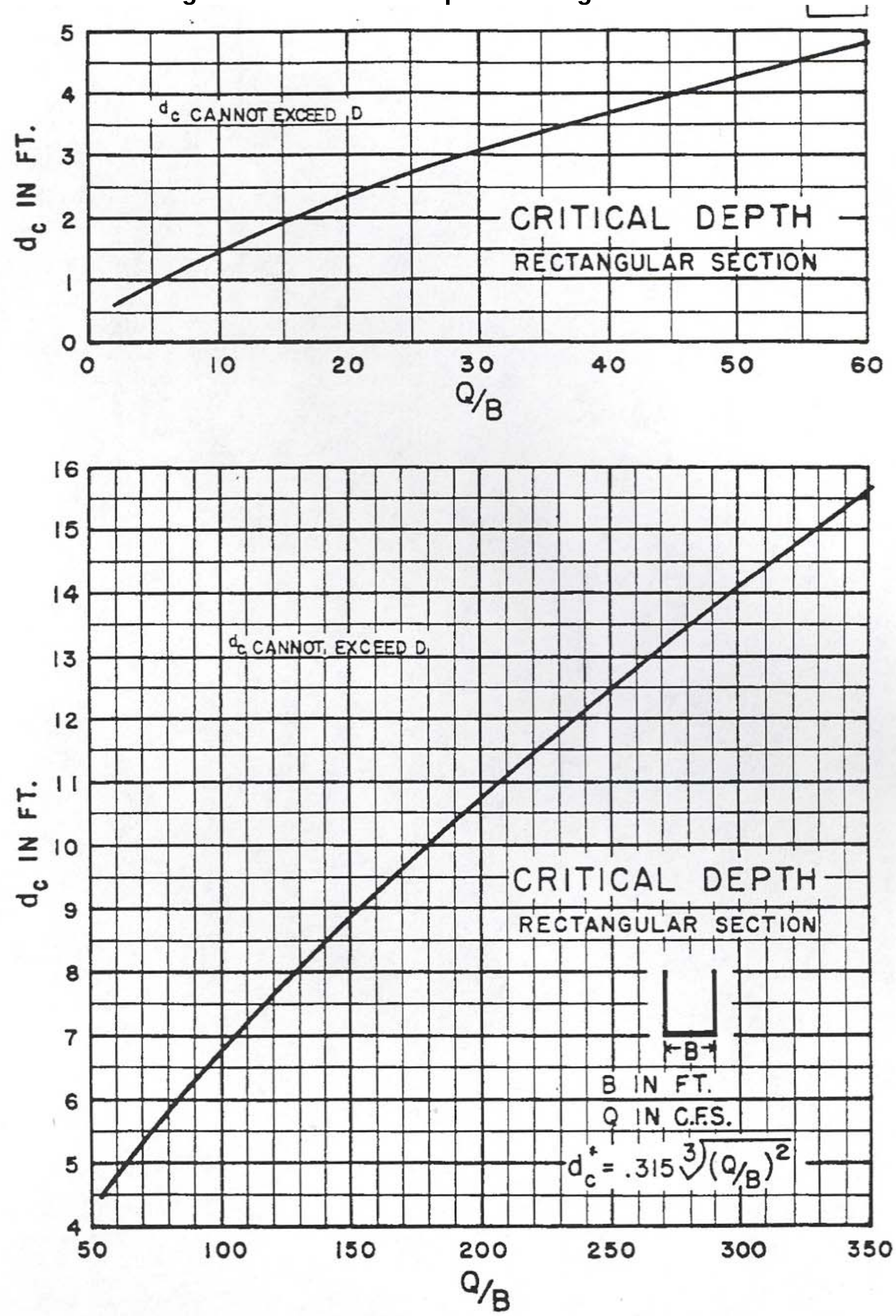


Figure 5. 30: Head for Concrete Box Culverts Flowing Full

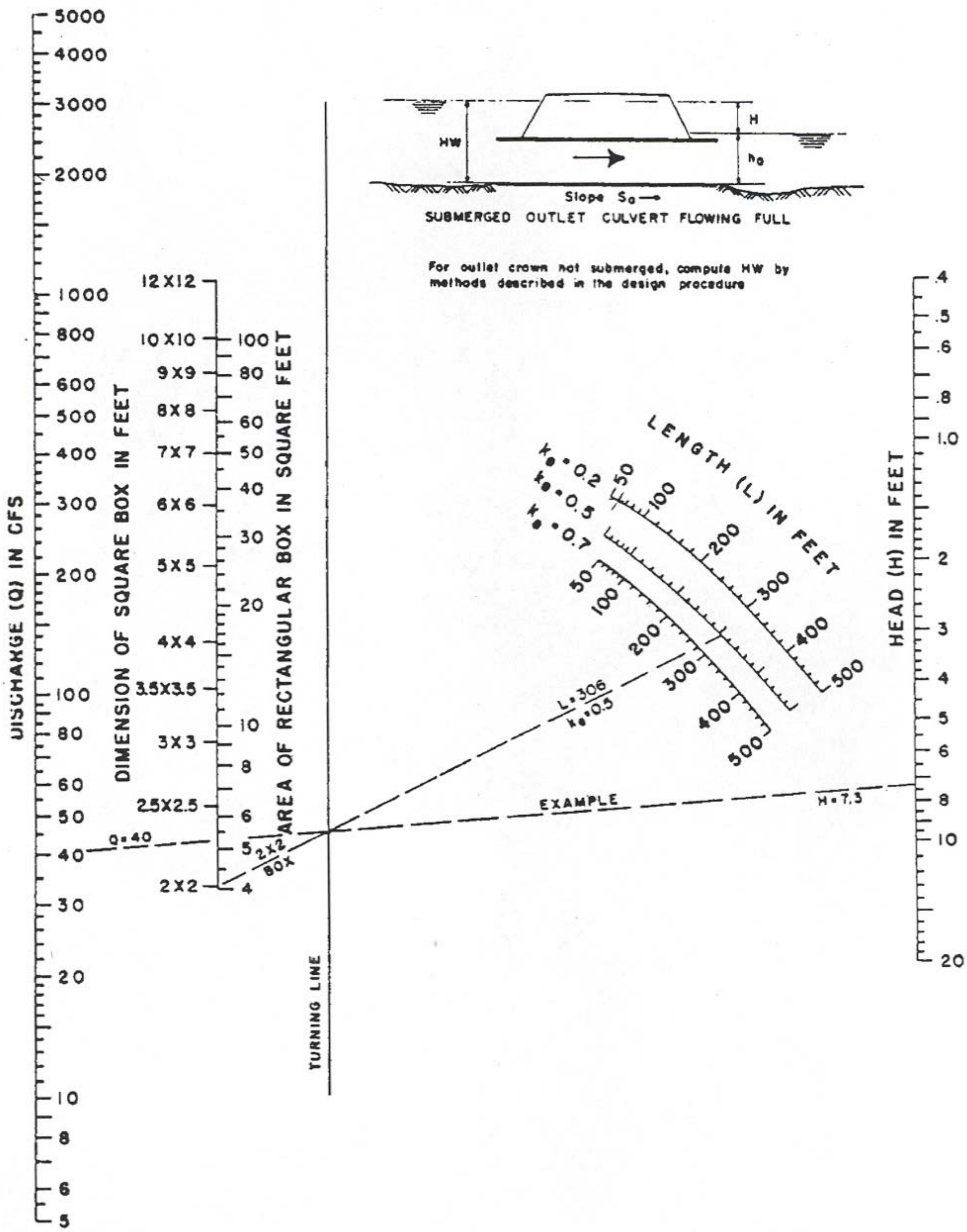


Figure 5. 31: Inlet Control Headwater Depth for Oval Concrete Pipe - Long Axis Horizontal

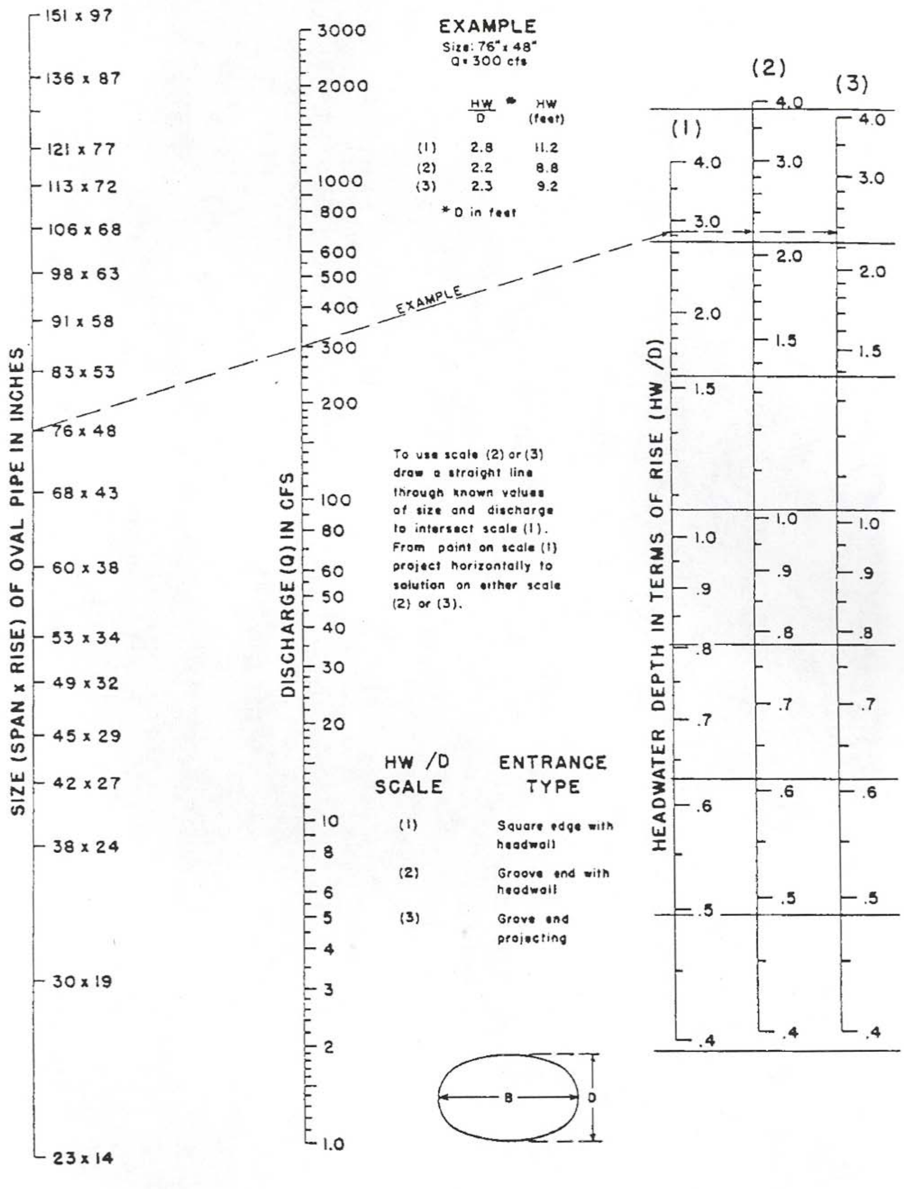


Figure 5. 32: Inlet Control Headwater Depth for Oval Concrete Pipe - Long Axis Vertical

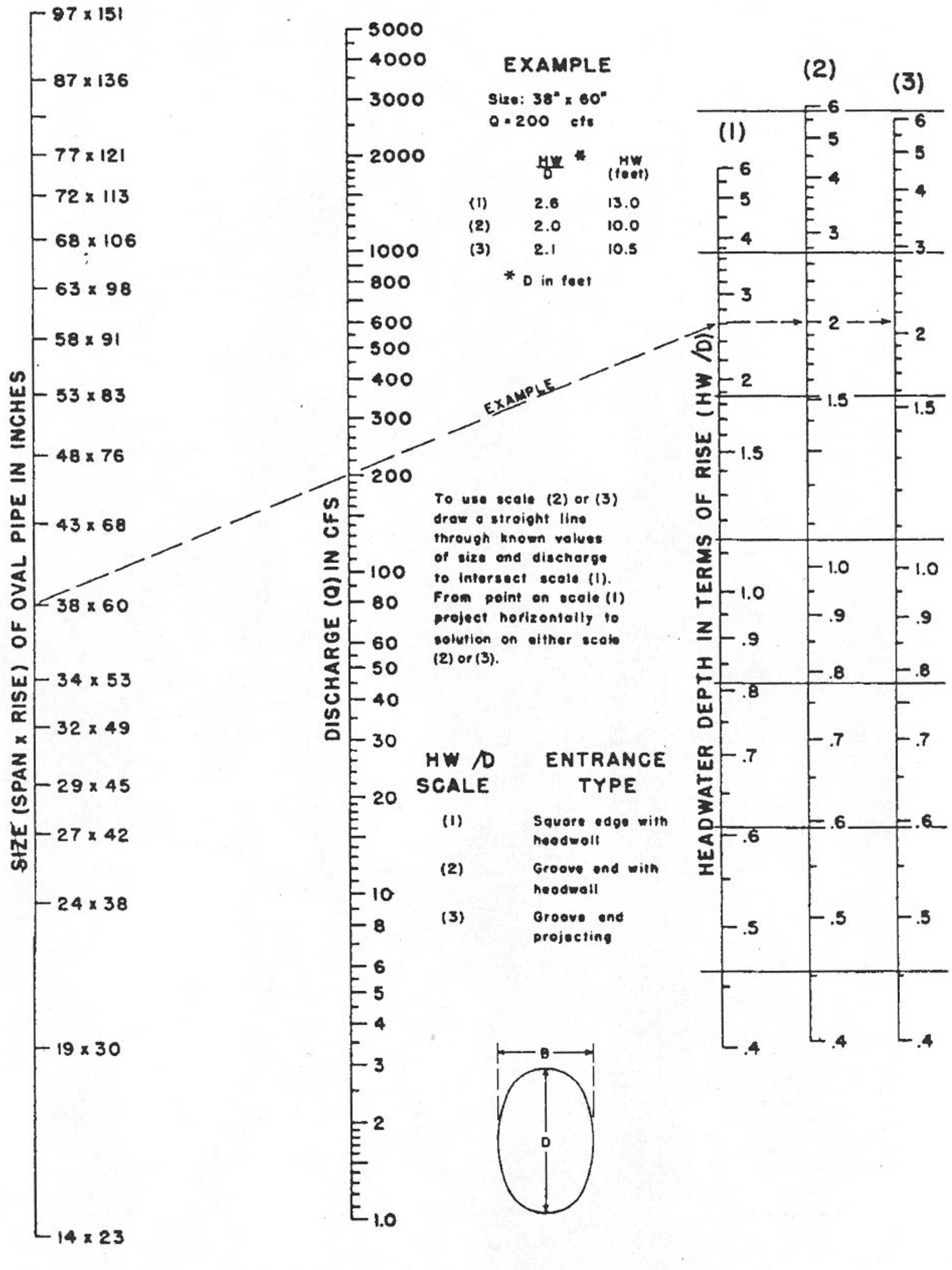


Figure 5. 33: Critical Depth for an Oval Concrete Pipe – Long Axis Horizontal

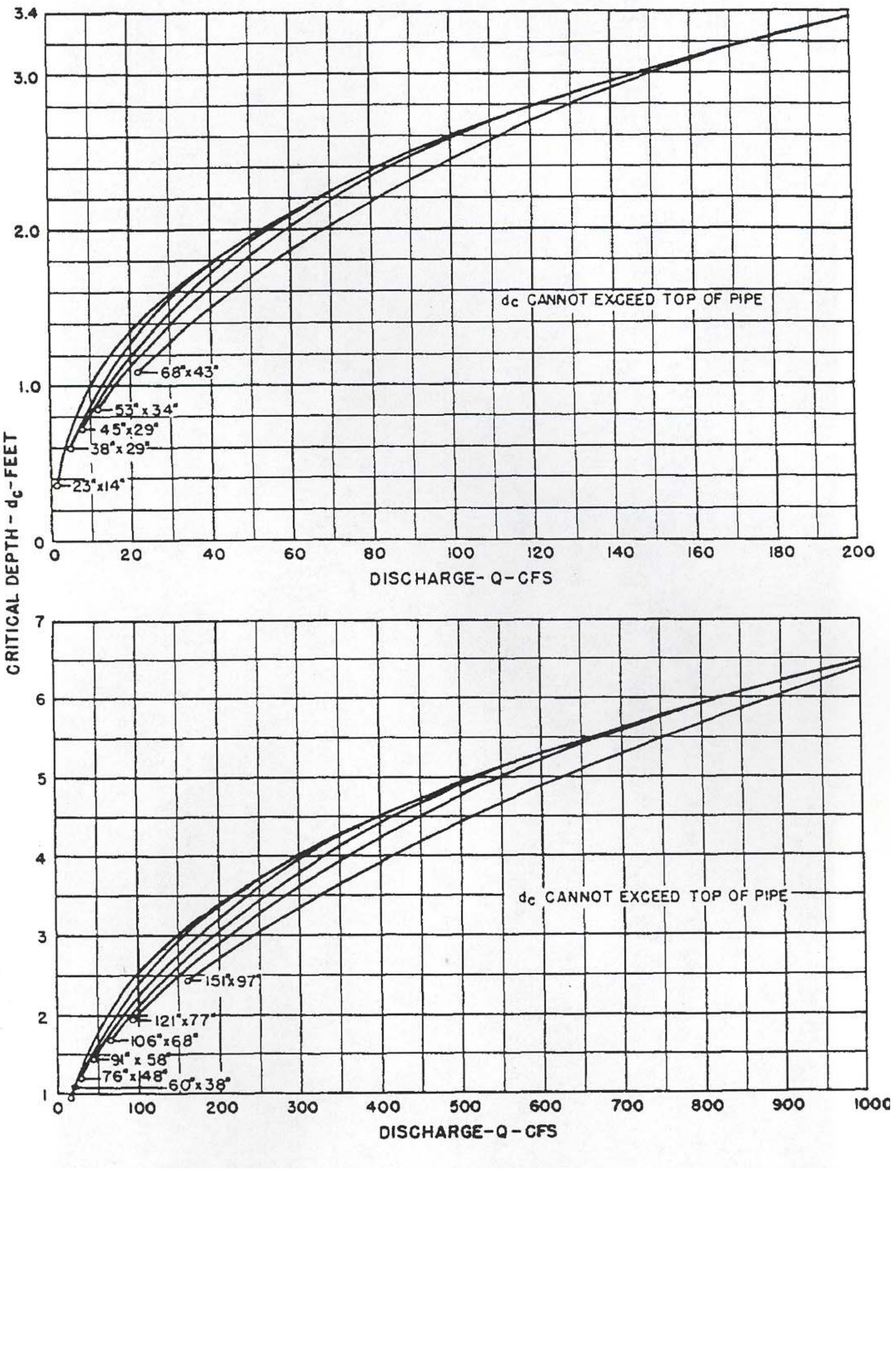


Figure 5. 34: Critical Depth for an Oval Concrete Pipe- Long Axis Vertical

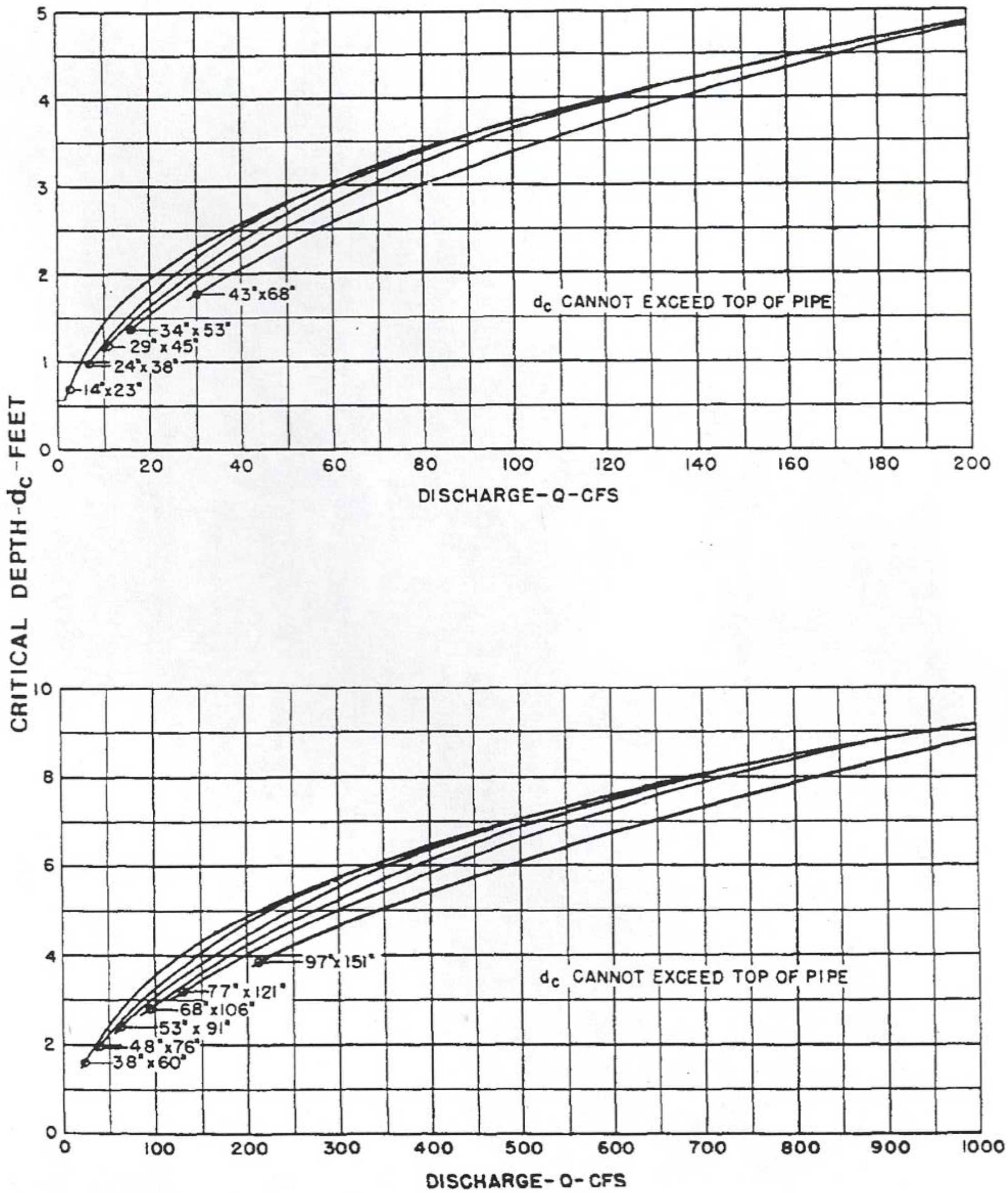


Figure 5. 35: Head for Concrete Pipe Flowing Full – Long Axis Horizontal or Vertical

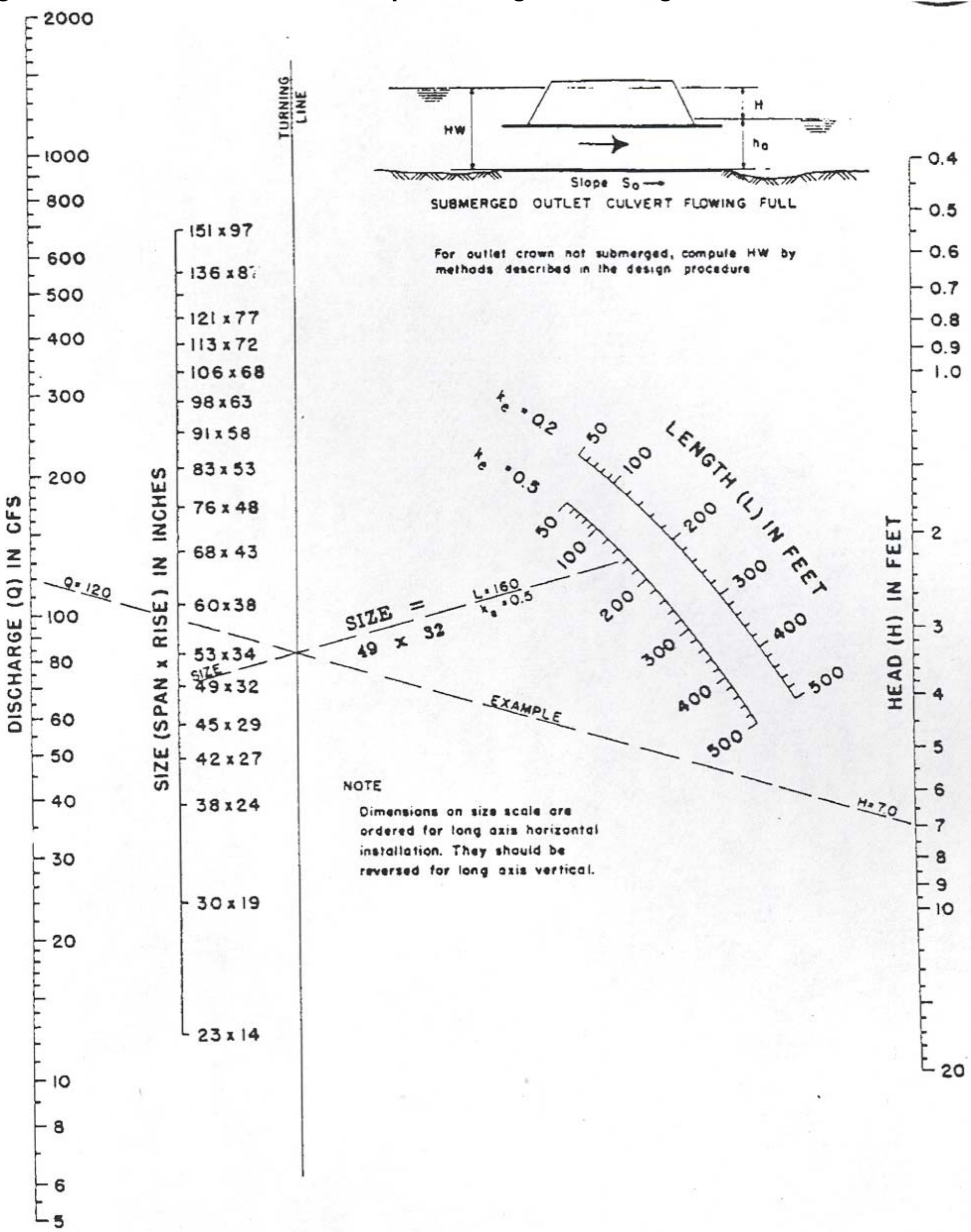


Figure 5. 36: Headwater Depth for C.M. Pipe - Arch Culvert with Inlet Control

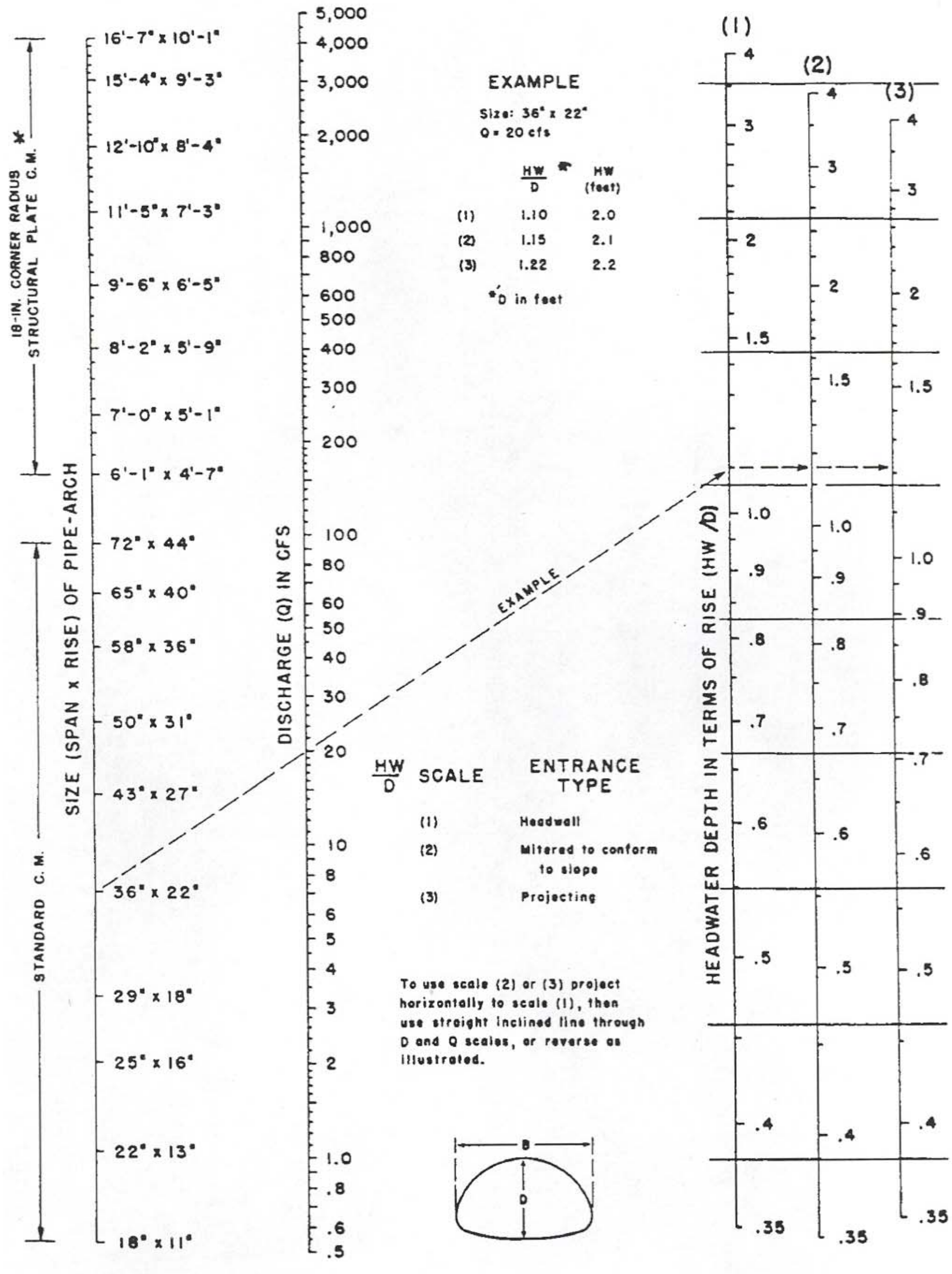
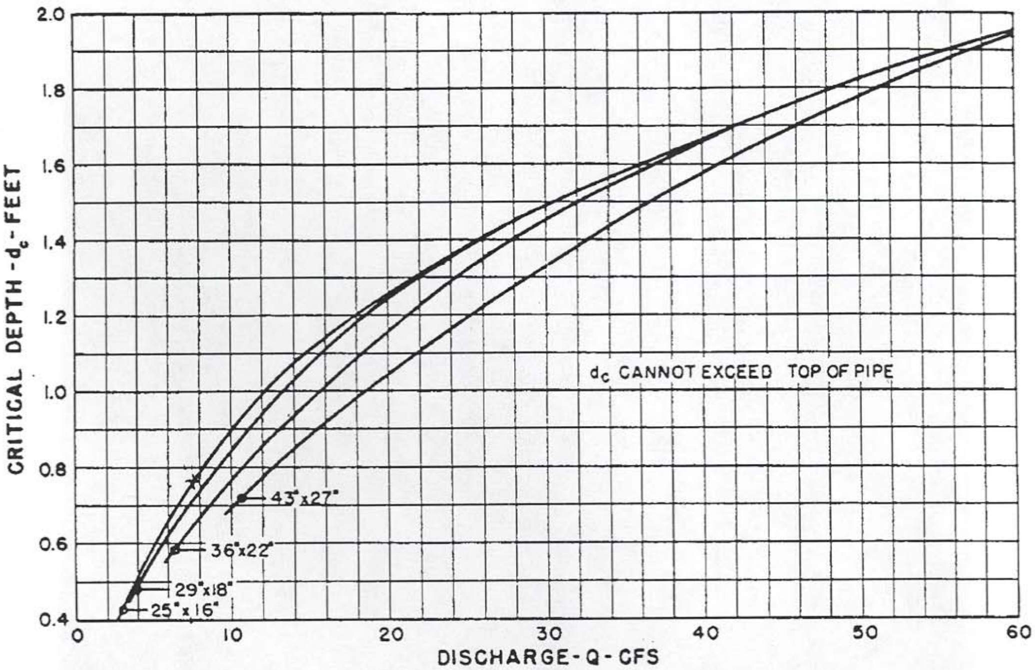


Figure 5. 37: Critical Depth for Standard C.M. Pipe - Arch



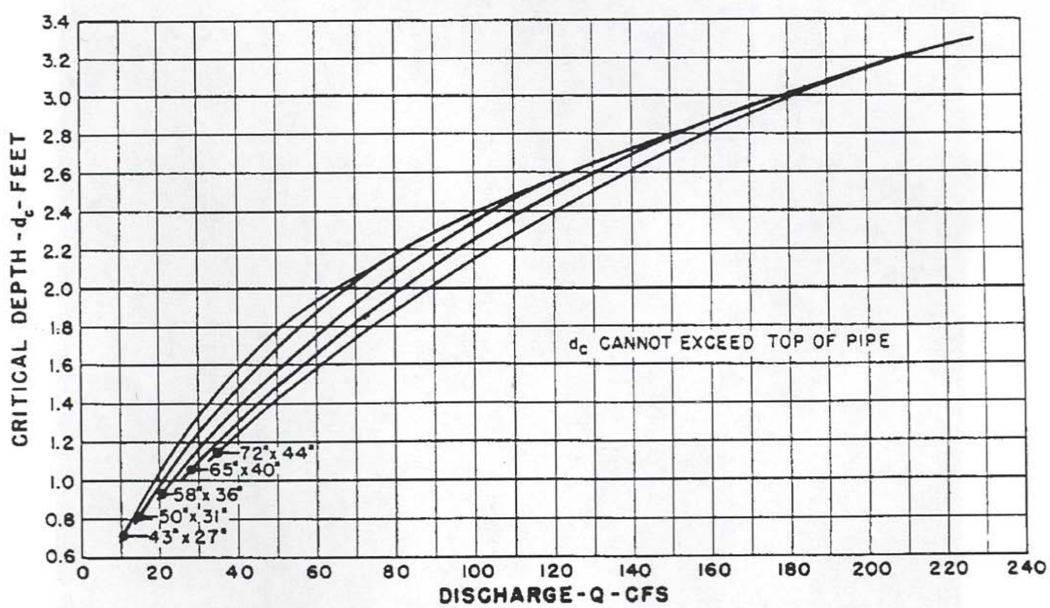
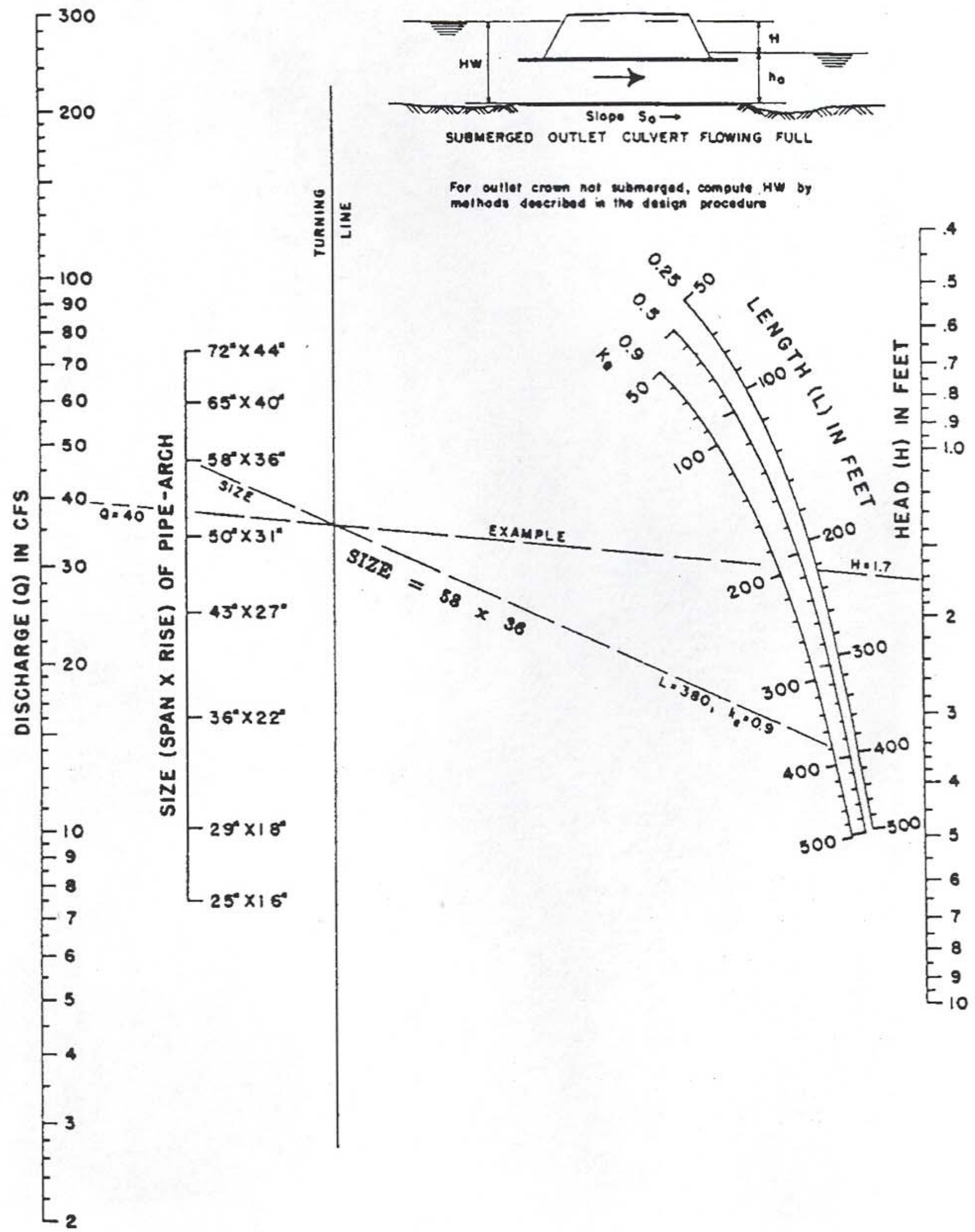


Figure 5. 38: Head for Standard C.M. Pipe - Arch Culverts Flowing Full



5.3 INLETS AND OUTLETS FOR CULVERTS

This section provides guidelines for design of culvert type inlets and outlets to closed conduit systems. Runoff entering and exiting closed conduits may require transitions into and out of the conduit to minimize entrance losses and protect adjacent property and drainage facilities from possible erosion. Pavement drainage inlets that allow runoff to drop into catch basins are discussed in Chapter 3, of this Manual and are not addressed here.

5.3.1 Interaction with Other Systems

Closed conduit inlets and outlets provide transitions from a ponded or channelized condition upstream into the closed conduit and then back to a natural or channelized condition downstream. Additional channel bank protection may be required in the vicinity of the inlet or outlet to complete the transition to the design velocity and flow depth of the receiving channel. The design of inlets and outlets should take into account all conditions in the upstream and downstream direction to the location where the inlet, outlet, and closed conduit have no effect on predesign flow conditions.

When an open channel or stormwater storage basin drains into a stormdrain system, culvert type inlets are frequently used. The stormdrain hydraulic grade line must be considered when estimating the inlet capacity for culvert type inlets. The stormdrain hydraulic grade line at the inlet, with the appropriate entrance loss added, should be substituted for the outlet control headwater elevation normally used for outlet control computations. To determine the controlling headwater, the computed outlet control headwater elevation should be compared with the inlet control headwater elevation obtained from the standard inlet control nomograph.

5.3.2 Special Criteria

5.3.2.1 Bank Protection

Roadway embankments with culverts passing through them should be protected from potential damage caused by roadway overtopping during a runoff event in excess of the culvert design capacity. When a planned flow over the road has damage potential, such as when the 100-year discharge causes flow over the roadway, the embankment for both upstream and downstream sides may need to be protected by use of paving, grouted riprap, or other means of permanent stabilization.

5.3.2.2 Entrance Structures and Transitions

Criteria for culvert entrances are contained Volume 1 Chapter 3. The same criteria apply to culvert type entrances for stormdrains. Design considerations include aligning the culvert with the natural channel profile, protection against inlet failure due to buoyant forces, and safety considerations for the public.

Culvert performance can be improved by providing a smooth and gradual transition at the entrance. Improved inlet designs have been developed for culverts operating in inlet control and are presented in Volume 1 Chapter 3.

Supercritical flow transitions at inlets require special design consideration. For design of supercritical flow contractions, refer to Hydraulic Design of Energy Dissipators for Culverts and Channels (USDOT, FHWA, HEC-14, 1983).

5.3.2.3 Outlet Structures

Standard measures for scour protection at conduit outlets include cutoff walls, wingwalls with aprons, and grouted or ungrouted riprap. These measures should be used as appropriate such that the velocity entering the receiving channel is within the allowable range of velocities for the channel outlet condition. Outlet conditions are classified as follows:

1. Natural channel outlets where the existing natural channel is modified only to transition to and from the culvert.

2. Artificial channel outlets where the culvert is part of an overall drainage plan and discharges into an improved, artificial channel.

3. Side channel outlets where a conduit drains into a larger receiving channel from the side at some angle of confluence.

It is not always desirable to totally restrict the movement of natural channels at the culvert outlet. Limited downstream scour and channel movement may be allowed in some cases. However, for artificial channel and side channel outlets, scour and bed movement should not be permitted. The following criteria shall be used in determining the type of outlet protection required based on the outlet condition.

5.3.2.4 Protection at Culvert Outlets

Riprap aprons placed downstream of culverts provide protection against scour immediately around the culvert as well as providing for the uniform spreading of the flow and decreasing the flow velocity, thus mitigating downstream damages.

5.3.2.5 Natural Channel Outlets

Natural channel outlet protection is based on the ratio of the culvert outlet velocity to the average natural stream velocity.

1. Culverts with outlet velocities less than or equal to 1.3 times the average natural stream velocity for the design discharge should have a cutoff wall

as a minimum for protection. Design criteria for cutoff walls are presented below.

2. Where the outlet velocity is greater than 1.3 times the natural stream velocity, but less than 2.5 times, a riprap apron should be provided..

3. When outlet velocities exceed 2.5 times the natural stream velocity, an energy dissipator should be provided. Several energy dissipators are described in Chapter 8, Hydraulic Structures.

5.3.2.6 Artificial Channel and Side Channel Outlets

Artificial channel and side channel outlet protection is based on the ratio of the culvert outlet velocity to the allowable velocity for the channel lining material. High velocity flow from the outlet must be transitioned to reduce the velocity to the allowable. Allowable velocities for several channel lining materials are shown in Chapter 6.

1. Conduits with outlet velocity less than or equal to the allowable require no outlet protection.

2. Conduits with outlet velocity greater than one and less than 2.5 times the allowable velocity should be provided with a riprap, concrete, or other suitable apron to transition the flow to the allowable channel velocity.

3. When outlet velocities exceed 2.5 times the allowable channel velocity, an energy dissipator should be provided. Several energy dissipators are described in Chapter 8, Hydraulic Structures.

5.3.2.7 Cutoff Walls

A cutoff wall placed at the culvert outlet in a natural wash provides adequate protection of the downstream end of the culvert when the outlet velocity does not exceed 1.3 times the average natural stream velocity for the design discharge. Cut-off walls are appropriate where the development of a scour hole will not undermine nearby structures or result in other harmful effects.

Depth of scour for cohesion less materials (0.2mm<=D_{50}<=2.0mm) downstream of culvert structures may be estimated using the following equation from Hydraulic Design of Energy Dissipators for Culverts and Channels (USDOT, FHWA, HEC-14, 1983).

$$\frac{d_s}{y_e} = \alpha\left(\frac{Q}{\sqrt{g}\,{y_e}^{2.5}}\right)^{\beta}\left(\frac{t}{t_0}\right)^{\theta} \qquad (5.11)$$

Where:

d_s = Depth of scour hole, ft
y_e = $\left(A/2\right)^{0.5}$ = Equivalent depth, ft
A = Flow area, sq ft
Q = Discharge, cfs
g = Acceleration due to gravity, ft/sec^2
t = Time of scour, set at 30 minutes
t0 = Base time (=316 minutes) used in the experiments in deriving the coefficients α, β, and θ
DI = Discharge intensity

$$DI = \left(\frac{Q}{\sqrt{g}\, y_e^{\;2.5}} \right) \tag{5.12}$$

For uniform sand with D_{50} = 0.2 mm (fine sand), values of the coefficients α, β, and θ are:

α = 2.72, β = 2.72, θ = 0.100

For uniform sand with D_{50} = 2.0 mm (very coarse sand), values of the coefficients α, β, and θ are:

α = 1.86, β = 0.45, θ = 0.09

The following guidelines are applicable to cutoff walls and are based on the computed scour depth analysis identified above.

1. The depth of the cutoff wall should be equal to or greater than the maximum depth of scour.

2. The depth of the cutoff wall should not normally exceed 6 feet. Where a deeper wall is necessary to meet the above guidelines. either another form of protection should be employed or an analysis will be required to substantiate the walls structural stability. Typically, some combination of cutoff wall and erosion protection such as riprap is used at culvert outlets.

Topics on scour are presented in Chapter 7, Sedimentation.

5.3.2.8 Safety

Inlets and outlets to closed conduits may present dangers to the public when access is not controlled. Refer to Volume 1, Chapter 1 for the safety requirements related to conduit inlets and outlets.

5.4 INVERTED SIPHONS

Because of the resulting physical conditions, inverted siphons are rarely used in urban drainage and should be avoided where possible. Due to the flat topography and a large number of canals in Pinal County, however, the designer may have to consider using an inverted siphon.

Inverted siphons are used to convey water by gravity under canals, roads, railroads, other structures, and depressions. An inverted siphon is a closed conduit designed to run full and under pressure. When flowing at design capacity, the structure should operate without excess head.

For canal structures, inverted siphons are economical, easily designed and built, and have proven to be a reliable means of water conveyance. However, because of sediment and debris present in stormwater, maintenance can be a significant negative factor. In addition, canals run more or less continually and can be drained between periods of use, but inverted siphons for stormwater do not operate on a regular cycle. If water is left to stand, significant health hazards could result. Inverted siphons shall be considered only when absolutely necessary, and permitted by the jurisdictional agency.

5.4.1 Design Procedure

A design procedure with examples is contained in Design of Small Canal Structures (USBR, 1974). Taking into consideration conditions that are more specific to urban drainage described before, this publication can be used for most applications in Pinal County.

All pipes should be designed for watertight joints. Velocity in the conduit should be a minimum of 5.0 ft/sec to prevent sedimentation. The cover over the conduit should exceed the minimum cover necessary to meet its loading classification. Inlet and outlet structures are required, and the facility shall meet the requirements for safety described in Volume1, Chapter 1. Pipe collars and blow-off structures may be required as determined by the jurisdictional agency. Air vents, after the entrance, should be used unless the agency agrees with eliminating the vents.

At a minimum, the designer should compute losses for the entrance and outlet (including trash racks), pipe friction, and losses at bends and transitions.

5.5 BRIDGES

This section presents a brief overview of the hydraulic analyses for bridge crossings over open channels. A general discussion of scour is also presented. Comprehensive guidelines and criteria for hydraulic analyses of bridge crossings are beyond the scope of this manual. The reader should refer to appropriate texts and technical handbooks for further information on this subject.

Roadways must often cross open channels in urban areas; therefore, sizing the bridge openings is of paramount importance. In general, bridges should be designed to have as little effect as possible upon the flow passing beneath them. If possible, bridges over natural or man-made channels should be designed so that there is no disturbance to the flow whatsoever. Whenever piers are used, they need to be oriented parallel to flow. Impacts upon channels and floodplains created by bridges usually take the form of increased flow velocities through and downstream of the bridges, increased scour and upstream ponding due to backwater effects. These impacts can cause flood damage to the channel, to adjacent property and to the bridge structure itself.

A new or replacement bridge should not be permitted to create a rise in the existing water surface elevation, to cause an increase in lateral extent of the floodplain, or to otherwise worsen existing conditions for discharges up to and including the 100-year discharge, unless appropriate measures are taken to mitigate the effects of such increases.

5.5.1 Hydraulic Analysis

The hydraulic analyses of pre- and post-bridge conditions can be performed using a computerized step-backwater model. The HEC-RAS program developed by the U.S. Army Corps of Engineers (USACE, 2001) is the most common backwater computation software available and is used nationwide. HEC-RAS is the preferred computer software for one-dimensional hydraulic analyses for studies of this type in Pinal County. The Corps older HEC-2 program may also be used for analyzing bridges, but is not preferred.

Bridge analysis requires meticulous input preparation for proper analysis, and care should be taken to review input data and to examine results thoroughly for reasonableness. Analyses of this type should only be undertaken by an engineer with a solid understanding of hydraulic fundamentals.

If there is a good possibility of debris collecting on the piers, it may be advisable to use a value greater than the physical pier width to account for debris blockage. Some agencies require the pier width to be modeled as twice its width while others require 1 foot added to each side of the pier. Thus, modeling requirements of debris blockage should be reviewed with the jurisdictional agency. For criteria guidance, refer to Volume 1.

5.5.2 Design Considerations

Additional factors to be considered in the design of a bridge crossing include flow regime (i.e., subcritical or supercritical flow), anticipated scour effects, and freeboard.

5.5.2.1 Freeboard

Freeboard at a bridge is the vertical distance between the design water surface elevation and the low chord of the bridge. The bridge low-chord is the lowest portion of the bridge deck superstructure. The purpose of freeboard is to provide room for the passage of floating debris, to provide extra area for conveyance in the event that debris build-up on the piers reduces hydraulic capacity of the bridge, and to provide a factor of safety against the occurrence of waves or floods larger than the design flood. Freeboard should be provided as required by jurisdictional standards.

A minimum freeboard of 2 feet for the 100-year event is recommended. The structural design of the bridge should take into account the possibility of debris and/or flows impacting the bridge.

In certain cases, site conditions or other circumstances may limit the amount of freeboard at a particular bridge crossing. An example would be the replacement of a "perched" bridge across a natural watercourse where major flows overtop the roadway approaches. In general, variances to the minimum freeboard requirement will be evaluated on a case by case basis by the jurisdictional agency.

5.5.2.2 Supercritical Flow

For the special condition of supercritical flow within a lined channel, the bridge structure should not affect the flow at all. That is, there should be no projections, piers, etc. in the channel area. The bridge opening should be clear and permit the flow to pass unimpeded and unchanged in cross section.

5.5.2.3 Scour

The issue of scour analysis at a bridge is beyond the scope of this chapter. The following discussion touches upon the subject matter to provide the interested designer an indication of the issues. Local pier and abutment scour, contraction scour, and long-term scour must be investigated when designing a bridge. Refer to Chapter 7, Sedimentation for guidance and insight into sedimentation and scour.

General scour from a contraction usually occurs when the normal flow area of a stream is decreased by a bridge. The contraction of the flow by the bridge can be caused by a decrease in flow area of the stream channel by the abutments projecting into the channel and/or the piers taking up a large portion of the flow area. Also, the contraction can be caused by approaches to the bridge that cut off the overland flow that normally goes across the floodplain during high flow. This latter case also can cause clear water scour at the bridge section because overland flow normally does not transport any significant bed material sediments. This clear water picks up additional sediment from the bed when it returns to the bridge crossing. In addition, if floodwater returns to the stream channel at an abutment it increases the local scour there. A guide bank at an abutment decreases the risk from scour of that abutment from returning overbank flow. Also, relief bridges in the approaches reduce general scour by decreasing the amount of flow returning to the natural channel, which then decreases the scour problem. See Chapter 7, Sedimentation for scour analysis protocol.

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VOLUME 2
DESIGN METHODOLOGY AND PROCREDURES

Chapter 6: Open Channels

6.1 INTRODUCTION

An open channel is a conveyance system in which water flows with a free surface at the water atmosphere interface. The channel may be either a natural watercourse or an artificial, "engineered" conveyance. Natural streams typically consist of a main flow channel, often termed the thalweg, and adjacent floodplains. Artificial channels are used for a wide variety of applications varying in scale from modest roadside ditches to large conveyance facilities that can be up to several hundred feet wide. Design guides are provided for the analysis of both natural and engineered channels.

This chapter is intended to provide design guidelines for use by engineers in the design of public infrastructure projects. More detailed explanations and further information are available from the technical resources listed at the end of this chapter. Readers are strongly encouraged to review the reference list and consider adding some of those publications to their design library.

6.1.1 Limitations

This chapter assumes that all channel boundaries are rigid, i.e., the channel cross section remains unaffected by erosion and the channel gradient remains constant for all flows. In this respect, this chapter is limited to channels where erosion, transportation, and deposition of sediment are not critical design considerations. For channels requiring consideration of non-rigid boundaries and/or sedimentation, see Chapter 7, Sedimentation.

Recommendations in this chapter address only channels designed to sustain subcritical or mildly supercritical flow regimes. Supercritical flows with Froude numbers greater than 1.13 require design procedures outside the scope of this chapter. If a designer determines that flows in the supercritical regime are unavoidable because of unique physical conditions, they should consult the technical staff of the jurisdiction involved for appropriate guidance. Section 6.2.5 contains discussion of the calculation of the Froude number and the determination of flow regime.

The design guidelines in Section 6.4 of this chapter for channel side slopes, lining materials, and allowable velocities have been put forth to protect the health and welfare of the public while minimizing societal costs. Designers are strongly encouraged to stay within these guidelines, unless alternative analytic procedures, guidelines, etc. can be substantiated.

6.2 CONCEPTS

6.2.1 Control Sections

A quantitatively definitive relationship between the stage and discharge of flow in an open channel exists at a control section. The control section regulates the hydraulic properties of flow in such a way as to restrict the transmission of the effects of changes in flow condition either in the upstream or downstream direction depending on the flow regime in the channel. These sections are ideal beginning points for calculation of water surface profiles. A control is in any section where depth of flow is known, such as critical depth, depth upstream of a culvert, depth of flow over a weir and depth of flow under a gate.

6.2.2 Continuity

For any flow, the discharge, Q, at a channel section is expressed by:

$$Q = AV \tag{6.1}$$

Where:

V = is the mean velocity (ft/sec)

A = is the cross sectional area of the flow measured normal to the direction of flow (sq ft).

Under steady flow conditions, the discharge is constant and:

$$Q = A_1 V_1 = A_2 V_2 \tag{6.2}$$

The subscripts denote different channel sections. Equation (6.2) is known as the Continuity Equation and is applicable to the flow conditions addressed in this chapter.

Obviously, Equation (6.2) is invalid for unsteady flow conditions in which discharge increases or decreases along the course of flow. Examples of unsteady flow are flood waves, bores, roadside gutters, side-channel spillways, wash water troughs in filters, and effluent channels around sewage treatment tanks. Precise treatment of unsteady flow is mathematically complicated and beyond the scope of this chapter.

6.2.3 Roughness Coefficients

Roughness coefficients (Manning's n values) vary considerably according to depth of flow, and type and quality of the surface material. Estimates of n values should include consideration that roughness may vary with flood stage, depending on such factors as the width-depth ratio of the watercourse; presence of vegetation in the main channel; the types of materials making up the channel bed; and the degree of meandering. Additional information concerning Manning's roughness coefficients can be found in Phillips and Ingersoll (1998), Thomsen and Hjalmarson (1991), Davidian (1984), Aldridge and Garret (1973) and Barnes (1967).

Typical values of roughness coefficients are given in Table 6.1. For each material and/or construction method listed, three possible values of n are given. These values should be interpreted as follows:

- minimum = new construction
- normal = good maintenance
- maximum = deteriorated and/or poor maintenance

The hydraulic design of a channel should be based upon the maximum n value anticipated during the life of the structure. The maximum n value for a particular channel material as listed in Table 6.1 is representative of this design-life condition. Channel design based on the maximum n value results in a conservative estimation of flow depth. Likewise, use of the minimum n value results in estimation of the maximum velocity of flow in the channel.

The minimum n values as listed in Table 6.1 represent newly constructed conditions. Maximum expected channel velocity should be a consideration in the analysis of supercritical flow, hydraulic jumps, and forces on structures, among others. In addition, Simons and Richardson (1966) observed that on natural sand-bed streams, resistance to flow might decrease significantly whenever large flows occur. As the depth of flow increases, resistance to flow due to bed roughness decreases. Flows that are sufficient to damage vegetation also reduce resistance to flow.

Table 6- 1: Manning's Roughness Coefficients

From: Simons, Li and Associates, 1988. Adapted from Chow (1959) and Aldridge and Garret (1973)

Channel Material	**Roughness Coefficient (n)**		
Concrete			
Trowel finish	0.011	0.013	0.015
Float finish	0.013	0.015	0.016
Unfinished	0.014	0.017	0.020
Shotcrete, good section	0.016	0.019	0.023
Shotcrete, wavy section	0.018	0.022	0.025
Soil cement	0.018	0.020	0.025
Constructed channels with earthen bed			
Clean earth; straight	0.018	0.022	0.025
Earth with grass and forbs	0.020	0.025	0.030
Earth with sparse trees and shrubs	0.024	0.032	0.040
Shotcrete	0.018	0.022	0.025
Soil cement	0.022	0.025	0.028
Concrete	0.017	0.020	0.024
Riprap	0.023	0.032	0.036

It is recommended that both maximum and minimum n values be applied in the design of channels to check for sufficient hydraulic capacity and stability of channel linings, respectively.

6.2.4 Flow Types

6.2.4.1 Uniform Flow

Manning's Equation

The most commonly used equations for analysis of open channel flow express mean velocity of flow as a function of the roughness of the channel, the hydraulic radius, and the slope of the energy gradient. They are empirical equations in which the values of constants and exponents have been derived from experimental data. Manning's equation is one of the most widely accepted and commonly used of the open channel equations:

$$V = \frac{1.486}{n} R^{2/3} S_f^{\ 1/2} \tag{6.3}$$

Substituting Equation (6.1) and rearranging yields the familiar form of Manning's equation:

$$Q = \frac{1.486}{n} A R^{2/3} S_f^{\ 1/2} \tag{6.4}$$

Where:

n=	Roughness coefficient (see Table 6.1)
A	=Flow area, sq ft
R	=Hydraulic radius, ft
S_f	=Friction slope, ft/ft

The Manning's roughness coefficient (n value) is a measure of the frictional resistance exerted by a channel on the flow. The n value can also reflect other energy losses such as those resulting from unsteady flow, extreme turbulence, and transport of suspended material and debris that are difficult or impossible to isolate and quantify. The reader is referred to Barnes (1967) and Thomsen and Hjalmarson (1991) for discussion of the estimation of n values for natural and composite channels.

The most common error in the application of Manning's equation is to substitute the bed slope of the channel, S_o' for the slope of the energy gradient, S_f. This substitution is correct only when the two gradients are parallel, as in the case of uniform flow. For a given condition of n, Q, and S_o' uniform flow is maintained only at normal depth. Normal depth rarely occurs in nature, and it is primarily a theoretical concept that simplifies the computation and analysis of uniform flow.

Table 6.2 lists the algebraic expressions for computing the hydraulic geometry for typical channel sections.

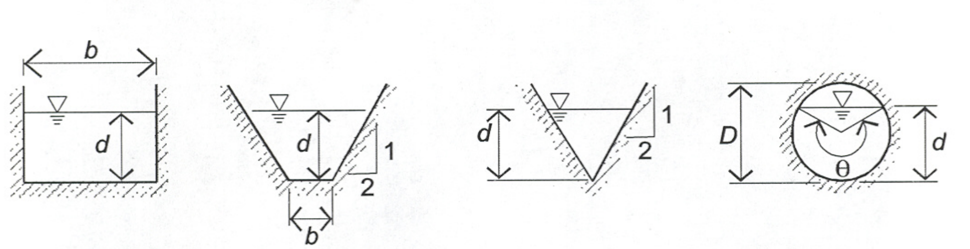
Table 6- 3: Elements of Channel Sections

Channel Section	**Area**	Wetted Perimeter	Hydraulic Radius	Top Width
Rectangle	bd	$b + 2d$	$\frac{bd}{b+2d}$	b
Trapezoid	$bd + zd^2$	$b+2d\sqrt{z^2+1}$	$\frac{bd+zd^2}{b+2d\sqrt{z^2+1}}$	$b+2zd$
Triangle	zd^2	$2d\sqrt{z^2+1}$	$\frac{zd}{2\sqrt{z^2+1}}$	$2\ zd$
Circular < 1/2 full (2)	$\frac{D^2}{8}\left(\frac{\pi\theta}{180}-\sin\theta\right)$	$\frac{\pi D\theta}{360}$		$D\sin\theta$ *or* $2\sqrt{d(D-d)}$
Circular > 1/2 full (3)	$\frac{D^2}{8}\left(2\pi-\frac{\pi\theta}{180}+\sin\theta\right)$	$\frac{\pi D(360-\theta)}{360}$	$\left(\frac{45D}{\pi(360-\theta)}\right)^* \left(2\pi-\frac{\pi\theta}{180}+\sin\theta\right)$	$D\sin\theta$ *or* $2\sqrt{d(D-d)}$

(1) After USDA Soil Conservation Service ES-33

(2) $\theta = 4\sin^{-1}\sqrt{d/D}$ Insert θ in degrees

(3) $\theta = 4\cos^{-1}\sqrt{d/D}$ Insert θ in degrees



Rectangle Trapezoid Triangle Circular

Composite Channels

The cross section of a natural or artificial watercourse or a street right-of-way may be composed of several distinct subsections, with each subsection having different hydraulic characteristics such as hydraulic roughness and average flow depth. For example, a natural alluvial channel may have a primary sand-bed channel which is bounded on both sides by densely-vegetated overbank floodplains, or an urban flooded street section may be bounded on both sides by landscaped front yards having shallower flow depths and slower flow velocities.

In composite channels like these, the discharge is computed for each subsection having distinct and different hydraulic characteristics, and the total computed discharge is set equal to the sum of the individual discharges. Similarly, the mean velocity for the entire flow cross section is assumed to be equal to the total discharge divided by the total water area. Open Channel Hydraulics (Chow, 1959), provides an example of computing flow in channels having composite roughness.

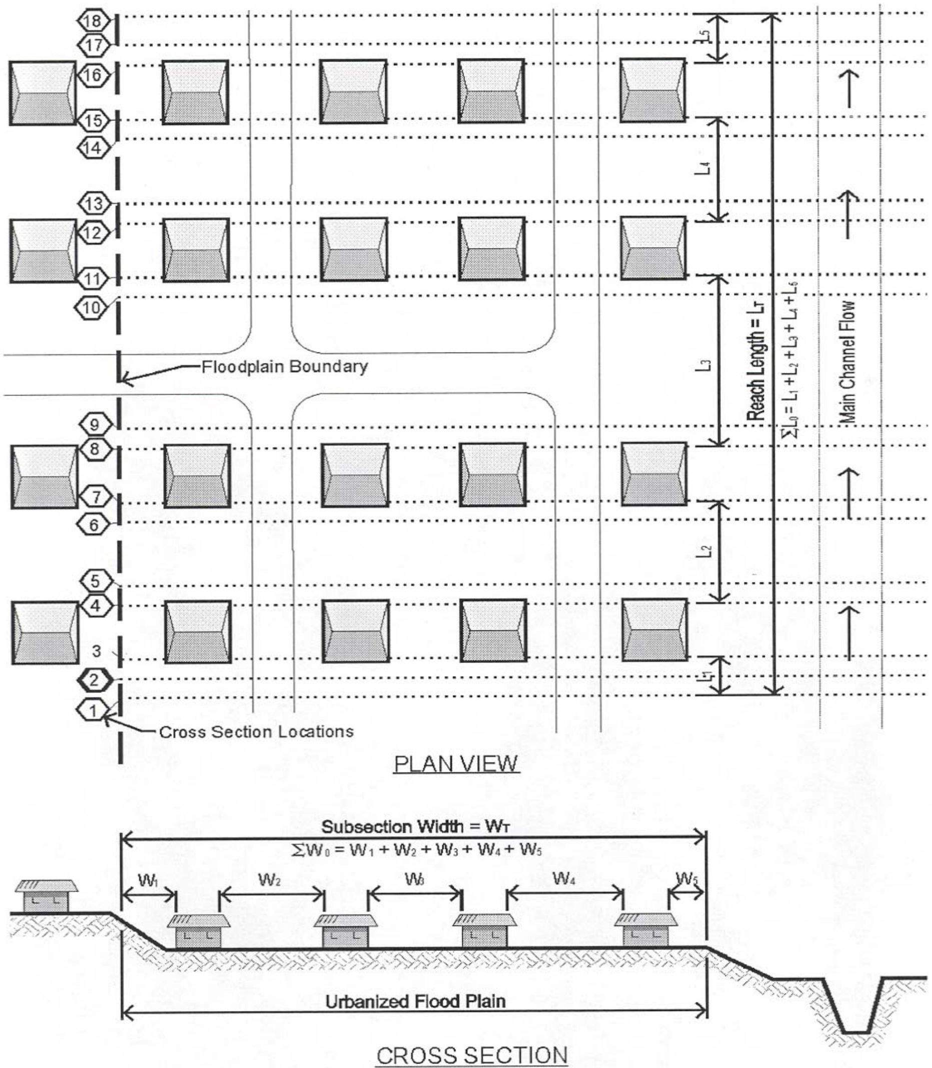
In the urban setting, it is not unusual for buildings and other structures to occupy a significant portion of any given hydraulic cross section. Under these circumstances, it is often difficult to estimate both the effective width of the cross-section and the Manning's roughness coefficient for the overbank areas. Given this situation, the engineer should eliminate the portion of the cross section occupied by the building.

Where only an estimate of the computed water surface elevation is needed, a second option may be selected. An adjusted urban roughness coefficient, n_u, may be computed and applied to the total cross-sectional area (Hejl 1977). See Figure 6.1.

$$n_u = n_0 \left[1.5 \left(\frac{W_T}{\sum W_0} \right) + \left(1 - \frac{W_T}{\sum W_0} \right) \frac{\sum L_0}{L_T} - 0.5 \right] \tag{6.4}$$

See Figure 6.1.

Figure 6. 1: Diagram of Idealized Urban Floodplain



6.2.4.2 Gradually Varied Flow

Classification of Water Surface Profiles

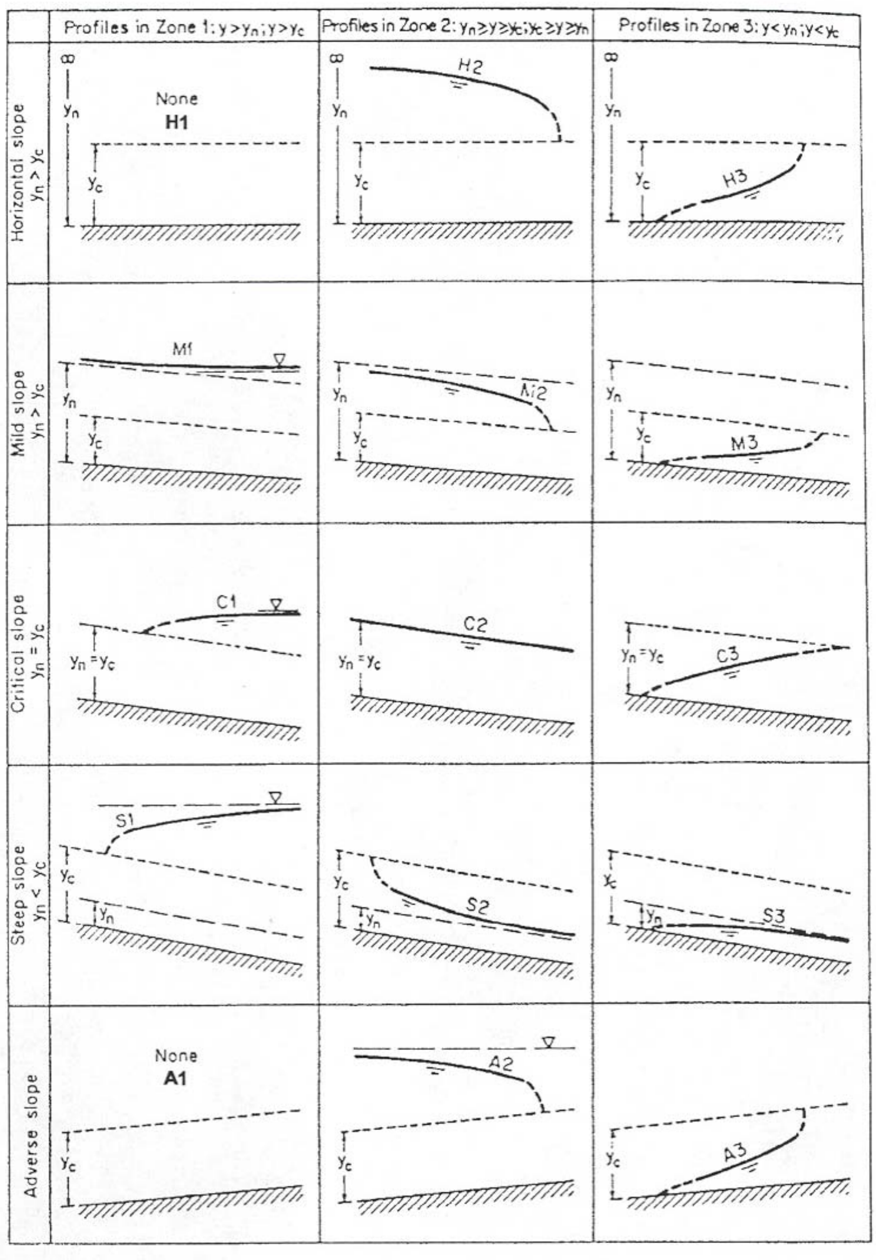
Chow (1959) describes the classification of these flow profiles into fifteen different types according to the nature of the channel slope and the zone in which the flow surface for a given discharge lies. These water surface profile types are designated according to an alphanumeric protocol, as follows:

- The letter is descriptive of the slope, i.e., *H* for horizontal, *M* for mild, *C* for critical, *S* for steep (supercritical), and *A* for adverse slope
- The numeral represents the zone number, where
  .Zone 1 -water surface above both normal and critical depths.
  .Zone 2 -water surface between normal and critical depths.
  .Zone 3 -water surface below both normal and critical depths.

These types are designated as H1, H2, H3; M1, M2, M3; C1, C2, C3; 81, 82, 83; and A1, A2, A3 as shown in Figure 6.2.

Flow profile analysis enables the designer to predict the general shape of the flow profile for a given channel layout. This step is a significant part of the open channel design process and it should not be omitted. Flow profile analysis will serve to identify control sections and to provide a work plan for more detailed design calculations.

Figure 6. 2: Classification of Flow Portion of Gradually Varied Flow



Calculation of Water Surface Profiles

The methods for calculating of normal depth assume uniform flow. However, sudden changes in discharge, bed slope, and cross sectional area and/or form will produce additional energy losses that are not accounted for in Manning's equation. This may be particularly true in cases of sudden contractions and expansions of the channel cross-section.

In those instances where an upstream or downstream hydraulic control section exists, the Standard Step Method should be used for evaluating water surface profiles. The procedure used for Standard Step calculations is presented in several of the technical references listed at the end of this chapter. The designer can perform the Standard Step calculations either manually using standard forms, or digitally using readily available and well-documented computer programs such as HEC-2 (USACE, 1990) or HEC-RAS (USACE, 2001a and b). These programs were developed by the U.S. Army Corps of Engineers and are available through the Corps WEB site at: www.hec.usace.army.mil .

One advantage of the Standard Step Method is the ability to converge an actual water surface profile for the study reach without needing to know the precise starting water surface elevation. If the computation is started at an assumed elevation that is incorrect for the given discharge, the resulting flow profile will approach the correct water surface elevation with each succeeding cross section evaluated within a study reach. If no accurate elevation is known within or near the reach under consideration, an arbitrary elevation may be assumed at a cross section far enough away from the "starting" cross section in the study reach to compensate for any initial error.

The step computations should be carried upstream if the flow is subcritical and downstream if the flow is supercritical. Otherwise, step computations carried in the wrong direction will result in a profile that diverges from the actual water surface profile.

For natural streams flowing under supercritical conditions, critical depth should be used for determining the water surface profile. Using the critical depth will produce higher and thus more conservative water surface elevations for design purposes. Velocities computed for the supercritical profile will be higher and more conservative and, therefore, should be used to evaluate scour potential and other velocity critical design features such as superelevation and freeboard.

The reader is referred to the technical references listed at the end of the chapter for more information regarding application of the standard step method and/or use of computer models such as HEC-2 and HEC-RAS for computation of water surface profiles. Specific references most instructive in this subject include Chow (1959) and USACE (1990, 2001 a, 2001 b), among others.

6.2.5 Flow Condition

The state of open channel flow is governed by the effects of viscosity and gravity relative to the inertial forces of the flow. The effect of gravity on the state of flow is represented by a ratio of inertial forces to gravity forces. This ratio is given by the Froude number, defined as:

$$F_r = \frac{V}{\sqrt{gd}} \tag{6.5}$$

Where:

V =The mean velocity, ft/sec
g =Acceleration due to gravity, ft/sec^2
d =Hydraulic depth, ft = A/T
A =Cross sectional area of water, sq ft
T =Width of free surface, ft

When F_r is equal to 1, the flow is in the critical state. This flow condition is unstable and flow depths at or near critical depth should be avoided. If F_r is less than 1, the flow is subcritical and gravity forces dominate. When F_r is greater than 1, the flow is supercritical and inertial forces predominate.

6.2.5.1 Subcritical Flow

Flows producing Froude numbers less than 1.0 are subcritical and have the following general characteristics relative to critical depth:

- Slower velocities
- Greater depths
- Lower hydraulic losses
- Less erosive power
- Less sediment carrying capacity
- Behavior easily described by relatively simple mathematical equations
- Surface waves propagate upstream

6.2.5.2 Supercritical Flow

Flows with Froude numbers greater than 1.0 are supercritical and have the following general characteristics relative to critical depth:

- Higher velocities
- Shallower depths
- Higher hydraulic losses
- More erosive power
- More sediment carrying capacity

- With few exceptions, behavior can't be easily predicted mathematically
- Surface waves propagate downstream only

6.3 DESIGN PROCEDURE

6.3.1 Route Considerations

The design of a safe and economical drainage system should be one of the first steps in the land development process. Drainage system requirements may determine the character of the development, and often dictate the layout of streets and lots. Attention to drainage requirements during the first phases of planning will result in better land use decisions and lower maintenance costs.

A drainage system that is well planned and designed incorporates several features. The pro- posed drainage system should be aligned with any existing and proposed structures, such as bridges and culverts, and be designed in such a manner that subcritical flow is maintained throughout (except at designed drop structures). The design should incorporate uniform channel properties, such as gradient and cross sectional geometry, as much as possible. Sharp and closely spaced curves should be avoided. Uncontrolled local runoff should not be allowed to enter the channel; rather, it should be collected and discharged into the channel through a structure specifically designed for that purpose. In all cases, the issue of wet and dry weather safety should be a paramount consideration in route and right-of-way determinations.

6.3.2 Layout

Unless special exception is made by the County Engineer, all artificial channels must begin and end where, historically, runoff has flowed.

The alignment of new drainage channels should follow existing washes, swales, and depressions whenever possible. The water must be collected and discharged at the same point and in the same manner as prior to the construction of the new channel. This means that the design of the new drainage features must account for runoff entering the property in the same location and manner as it historically flowed, and collect the water and transition it into the new channel for conveyance through the project site. At the downstream end of the channel, the drainage design must provide a transition from the on-site channel to return the runoff to its historic location prior to leaving the property. This requirement applies to the hydraulic geometry and velocity of the water, and the elevation of the water surface.

6.3.3 Grade Control

Regardless of the size of watershed, a key design element, including conceptual layout, is establishing whether or not grade control exists below the design section. General degradation and aggradation is beyond the scope of this manual; however, references are provided in the references listed at the end of this chapter.

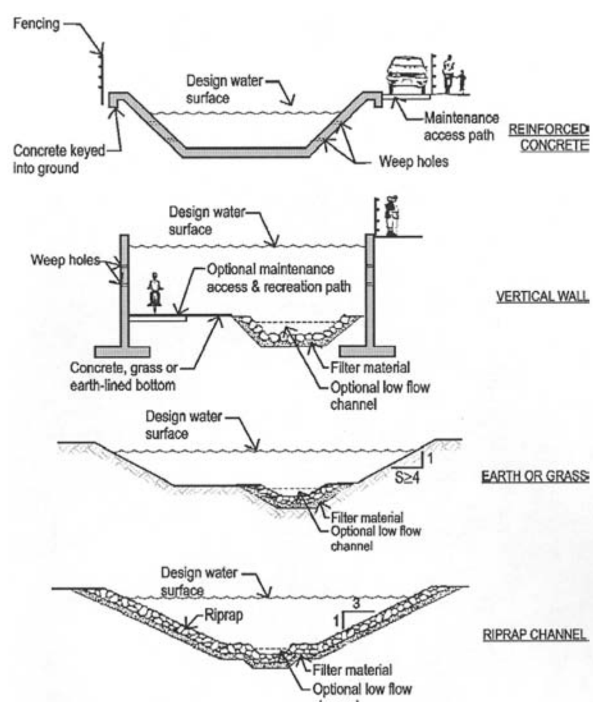
Grade control is a critical factor in the long-term behavior of non-rigid channels. By definition, grade control is any natural or man-made structure within a channel that limits or prevents vertical movement of the channel bed, either degradation or aggradation. Examples include rock outcroppings, culverts under embankments, drop structures, and bridges; however, not all drop structures, culverts, or bridges can be considered as grade control structures.

Grade control and channel slope are interrelated. In the design of grade control structures, the stability of the study reach must be assessed in context of the equilibrium of the entire system. The benefits of establishing grade control within a specific channel reach are minimal when the adjacent channel reach is either in a degradational or aggradational mode. When designing artificial channels, the designer needs to assess the stability of the reach immediately downstream from the segment under design. If there is evidence of ongoing downstream degradation, a grade control structure may be required. At a minimum, the grade control structure should extend to a depth sufficient to prevent upstream migration of the headcut. For each alternative investigated, the longitudinal spacing of grade control structures and the design slope of the channel should result in a stable channel.

6.3.4 Channel Linings

Artificial channel linings vary with the shape of the section and with the velocity of the water. Typical channel linings include concrete, soil cement, rock, earth (natural), and grass. These linings can be used alone or in combination with other linings. Typical linings and sections are shown in Figure 6.3.

Figure 6. 3: Typical Channel Sections



The type of stabilization that may be best suited for a particular purpose will depend upon a variety of factors, including hydraulic conditions, economic factors, soil conditions, material availability, aesthetics, maintenance and compatibility with existing improvements. The order of preference for subcritical flow conditions is natural channels with periodic grade-control structures, channels with vegetal linings, compound channels, channels lined with riprap, or its variations, channels lined with soil cement, and concrete-lined channels. Where supercritical flow conditions occur, only acceptable structurally sound channel linings such as concrete and shotcrete are recommended.

6.3.4.1 Earth Lined Channels

This category includes both bare earth and naturally vegetated channels in Pinal County. Subsequent to construction, some revegetation will naturally occur, or landscaping practices may be used to establish growth of indigenous plant materials. For Pinal County, this growth will be desert-like, with few grasses and a sparse spacing of other plants.

Earth lined channels are to be designed for subcritical flow regimes. Normally, these channels are relatively small and do not require low flow channels. If earth lining is used for larger channels, an armored low flow channel is required to control meandering and sediment deposition during low flow events. The low flow design should be checked for the effect that less frequent storms may have on sediment or scour, in terms of maintenance and aesthetic implications.

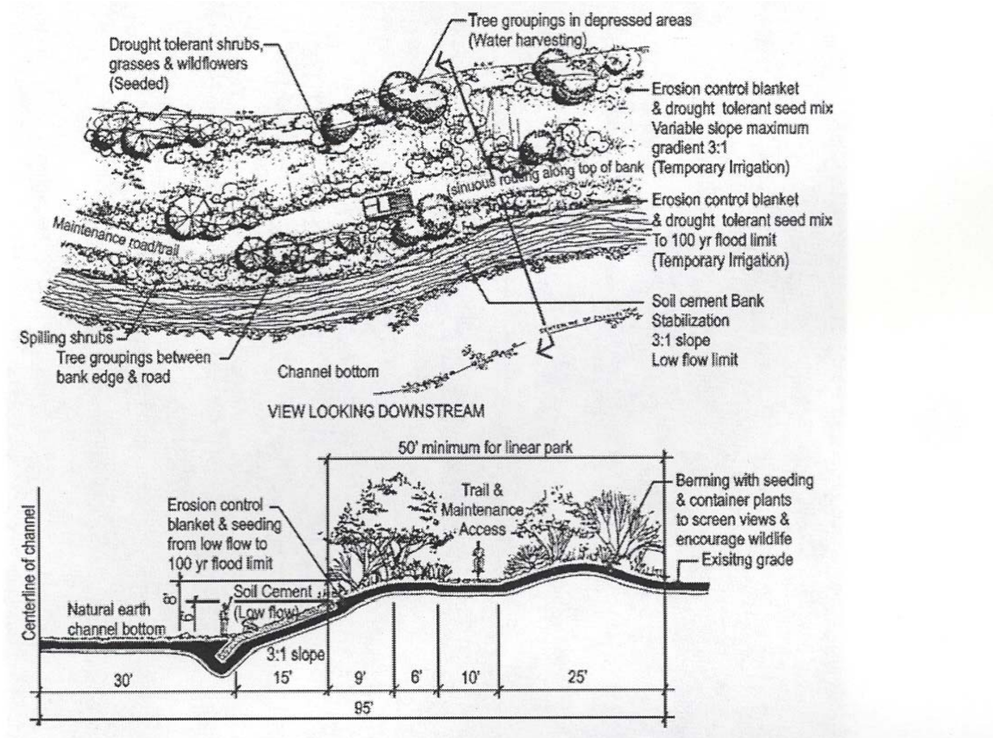
6.3.4.2 Grass Lined Channels

In a desert environment such as Pinal County, there is not enough natural rainfall to maintain a grass lined channel without irrigation. Therefore, only those channels where an irrigation system is provided and maintenance can be performed are candidates for grass lining.

6.3.4.3 Compound Channels with Multi-Use Opportunities

A channel with a compound or contoured cross section typically contains a smaller, interior channel that isolates frequent low-flows from upper portions of the channel. The upper portions of the channel which are only inundated during the less frequent storm events (typically, 100-year event), may then be utilized for landscaping and recreation opportunities (such as trails and bike paths). See Figure 6.4. Bank protection can extend from the channel bottom to the top of the low- flow channel; or it can extend the full height of the channel sides to the top of the high-flow portion of the channel, depending on the hydraulic characteristics of the channel.

Figure 6. 4: Compound Channel



6.3.4.4 Riprap Lined Channels

Rock lined channel lining includes both common riprap (graded rock) and gabion basket linings. Both types require a gravel filter layer and/or filter fabric between the rock layer and the natural ground. Excluding applications for hydraulic structures, gabion riprap is normally used when rock of sufficient size for common riprap is unavailable, poorly shaped, and/or overly expensive for a project. Normally, rock linings are used for channels where right-of-way is limited (considering maximum side slope requirements) and subcritical flow can be maintained. These linings are also used immediately upstream and downstream of hydraulic structures. Refer to Section 6.4.3.

6.3.4.5 Soil Cement

Soil cement linings are composed of a thick layer (4-foot minimum) of unreinforced soil cement and are used successfully in many locations. Soil cement is subject to weathering and abrasion and, thus, may not function satisfactorily long-term when used in the bottom of channels. Soil cement can withstand relatively high velocities for short periods of time and, therefore, is most appropriate for channels with limited right-of-way or as a bank lining near bridges and culverts where local velocities tend to be high. Refer to Section 6.4.2.

6.3.4.6 Concrete Lined Channels

Concrete lined channels may be constructed of reinforced concrete or shotcrete. They are used primarily where right-of-way is limited and may be designed for either subcritical or supercritical flow. Concrete lined channels generally have steep side slopes because of the limited right-of- way. Inherently, these channels present public safety problems both in wet and dry weather.

The anticipated structural loads and the clearance requirements of the reinforcing steel will dictate the thickness of the concrete lining. Weep holes and subdrains are required to prevent uplift pressures from hydrostatic force in saturated conditions. Reinforced tie-ins are required at the top of the lining. Designers are cautioned against copying these details directly without first evaluating the design conditions for their specific project.

Concrete and shotcrete lined channels are discouraged in residential and recreational areas. If concrete channels are needed in these areas, the designer should contact the technical staff of the appropriate jurisdiction. Refer to Section 6.4.1.

6.3.5 Low Flow Channels

Some of the sections shown in Figure 6.3 have an optional low flow channel. Low flow channels are provided to minimize lateral meandering and sedimentation during low flow events. They also permit the incorporation of recreational amenities by preventing these facilities from being flooded during high frequency, low discharge flow events in compound channels.

Many large drainage basins have small base flows resulting from irrigation returns, treatment plant effluent, or urban cooling water. In addition, the most frequent runoff events are considerably smaller in magnitude than the storm for which the channel was designed. In the long term, such high frequency, low magnitude flows will deposit considerable amounts of sediment in the channel. Sediment deposition can cause redirection of flow into the channel banks resulting in erosion and/or a meandering low flow channel in the channel bottom. Earth and grass lined channels are particularly susceptible to this problem. It is recommended that low flow channels be provided whenever the following condition exists:

$$\frac{b}{V_y} \geq 1.40 \qquad (6.6)$$

6.3.6 Safety

Deep channels with steep side-slopes and high flow velocities can be a hazard to the health, safety, and welfare of the general public. Therefore, the design engineer must always consider the safety aspects of any design. The reader is referred to Volume 1 of this manual.

6.3.7 Maintenance

The design engineer must also consider maintenance issues associated with any design. At a minimum, a 16-ft maintenance access lane with access ramps is recommended to be provided on one side of a channel for publicly maintained channels. Refer to the jurisdictions Policies and Standards Manual for specific criteria. To minimize maintenance; paths, walkways, play areas, and irrigation systems should be located in less frequently inundated levels of channels. Bottom widths of channels should be designed in consideration of maintenance requirements for the channel lining, and will be no narrower than 8 feet unless otherwise approved by the jurisdictional entity.

6.3.8 Design Factors

Good design practice requires that several issues be addressed. Unless exempted by the County Engineer, water surface profiles must be computed for all channels during final design and clearly shown on a copy of the final drawings. Computation of the water surface profile should use standard step backwater methods (see Section 6.2.4). These computations must account for all losses due to changes in velocity, drops, bridge openings, and other factors. Computations should begin at a known point and extend in an upstream direction for subcritical flow regimes, and in a downstream direction for supercritical regimes. Concrete lined channels with supercritical flow regimes should be analyzed as described in Section 6.4.1. The energy gradient must be shown on all preliminary drawings to help check for errors; however, it is optional for final drawings. Open channel flow in urban drainage is usually non-uniform due to bridge openings, channel curves, and hydraulic structures, therefore backwater computations must be used for all final channel design work.

6.3.8.1 Minimum Velocity

Very low velocities encourage sedimentation and undesirable plant growth, which decreases channel carrying capacity and promotes nuisance ponding. Channels must be designed with respect to sedimentation issues elaborated in Chapter 7.

6.3.8.2 Maximum Velocity

For earthen or grass lined channels, maximum permissible velocities should be governed by Table 6.3 and Table 6.4, respectively. If the natural channel slope would cause excessive velocity, employ drop structures, checks, riprap (USDOT, FHWA HEC-11), or other suitable velocity control design features.

Table 6- 4: Max Permissible Velocities for Roadside Drainage Channels with Erodible Linings
(USDOT, FHWA, 1961 and 1988)

Soils Type of Lining (Earth, No Vegetation)	Permissible Velocity [1] [2], ft/ sec
Fine Sand (noncolloidal)	2.5
Sandy Loam (noncolloidal)	2.5
Silt Loam (non colloidal)	3.0
Ordinary Firm Loam	3.5
Fine Gravel	5.0
Stiff Clay (very colloidal)	5.0
Graded, Loam to Cobbles (noncolloidal)	5.0
Graded, Silt to Cobbles (noncolloidal)	5.5
Alluvial Silts (non colloidal)	3.5
Alluvial Silts (colloidal)	5.0
Coarse Gravel (non colloidal)	6.0
Cobbles and Shingles	5.5
Shales and Hard Pans	6.0

(1) For sinuous channels multiply permissible velocity by:
0.95 for slightly sinuous;
0.90 for moderately sinuous; and
0.80 for highly sinuous

(2) Higher velocities may be allowed for design of unlined channels, for the 1 OO-year design event in particular, based on sediment balance considerations defined using the guidelines in Chapter 10. However, sufficient setback allowance should be provided for expected bank erosion during the 100-year event, or a series of annualized events over a 50-year period. Higher velocities may also be acceptable for 100-year peak flow design with approved engineering justification based on a tractive force analysis (USDOT, FHWA HEC-11).

Table 6- 5: Roadside Channels with Uniform Grass Cover and Well Maintained
(Adapted from USDOT, FHWA 1961 and 1988) [1] [2] [3]

Cover	Permissible Velocity, ft/sec
Bermuda Grass	6.0
Desert Salt Grass	5.0
Vine Mesquite	
Lehman Lovegrass	3.5
Big Galleta	
Purple Threeawn	
Sand Dropseed	

(1) Use velocities over 5 ft/sec only where good covers and proper maintenance can be obtained.
(2) Grass is accepted only if an irrigation system is provided.
(3) Grass lined channels not recommended for slopes greater than 5%.

6.3.8.3 Freeboard

Freeboard is the distance between the calculated water surface and the top of the channel lining or bank. The minimum freeboard is calculated as follows:

$$FB = 0.25\left(y + \frac{V^2}{2g}\right) \tag{6.7}$$

In subcritical channels, the minimum required freeboard is the larger of one foot or that calculated using Equation (6.7). In supercritical channels, the required freeboard is the larger of two feet or the results of Equation (6.7). In all instances, the freeboard required is additive to any increases in water surface due to superelevation or channel curvature. Freeboard for levees must meet FEMA freeboard requirements (3, 3.5, or 4 feet minimum depending on location relative to end of levee, and to other structures).

6.3.8.4 Channel Curvature

The minimum radius of a curved channel, measured to the channel centerline, carrying subcritical flows is recommended to be three times greater than the width of the water surface. That is:

$$r_c \geq 3T \tag{6.8}$$

If the channel is carrying supercritical flows, the recommended minimum radius is:

$$r_c = \frac{4V^2 T}{gy} \tag{6.9}$$

6.3.8.5 Superelevation

Curves in a channel cause the maximum flow velocity to shift toward the outside of the bend. Along the outside of the curve, the depth of flow is at a maximum. The consequent rise in the water surface is referred to as superelevation. Under subcritical conditions, the following equation is recommended to estimate the magnitude of the superelevation:

$$y = \frac{0.5V^2 T}{gr_c} \tag{6.28}$$

Readers are cautioned to avoid curves in channels with supercritical flows. The shift in the velocity distribution may cause cross-waves to form, which will persist downstream and could severely limit the hydraulic capacity of the channel. Advanced design criteria or physical model studies beyond the scope of this chapter may be required.

6.3.8.6 Toe Protection

Toe protection failures result when the foundation of the bank protection measure is undermined by scour at the toe resulting from local scour and/or general channel bed degradation. Proper design of protection from toe scour involves two parameters. First, an estimate must be made of the maximum scour expected to occur over the design life of the structure. Second, a means of protection must be provided for the maximum scour. The first parameter, scour depth estimation, requires specialized analysis techniques by a qualified engineer. References for scour and sediment transport analysis are included in Chapter 7. Mitigation measures for providing protection for the maximum scour are presented in this section.

The two methods of providing toe protection in erodible channels are:

1. To extend protection to the maximum estimated depth of scour

2. To provide protection that adjusts to the scour as it occurs

The first method is the preferred technique because the protection is initially placed to a known depth and the designer does not have to depend on uncertainties associated with the method that adjusts to the scour. This method requires extension of the bank protection into the excavated channel bed and is primarily used for placement in dry conditions because of the expense and uncertainties of deep excavation that can frequently encounter groundwater.

The main advantage of the second method is the elimination of relatively deep excavation and related water control. The most frequently used material for providing adjustable toe protection is riprap placed at the toe of the bank in a weighted riprap configuration. The riprap moves downs- lope, as scour occurs, to form a protective cover. Figure 6.5a shows the desirable configuration for a weighted riprap toe. Other materials utilized are gabion mattresses (see Figure 6.8b). These mattresses are anchored to the bank protection and their riverward ends are allowed to lower as scour occurs. Studies by Linder (1976) and the U.S. Army Corps of Engineers (1981) on riprap toe protection arrived at the following conclusions:

1. Volume of rock in the weighted riprap toe is probably the most significant factor in determining the success of the weighted riprap toe.

2. Toe shape has a definite influence on performance. Thin toes do not release rock fast enough, which results in poor slope coverage. Thick toes release rock at a greater rate than is needed. The thickness of the recommended toe ranges from two to three times the thick- ness of the riprap bank protection. The recommended toe shape is shown in Figure 6.8a.

3. Complex toe designs that are difficult to construct are not necessary.

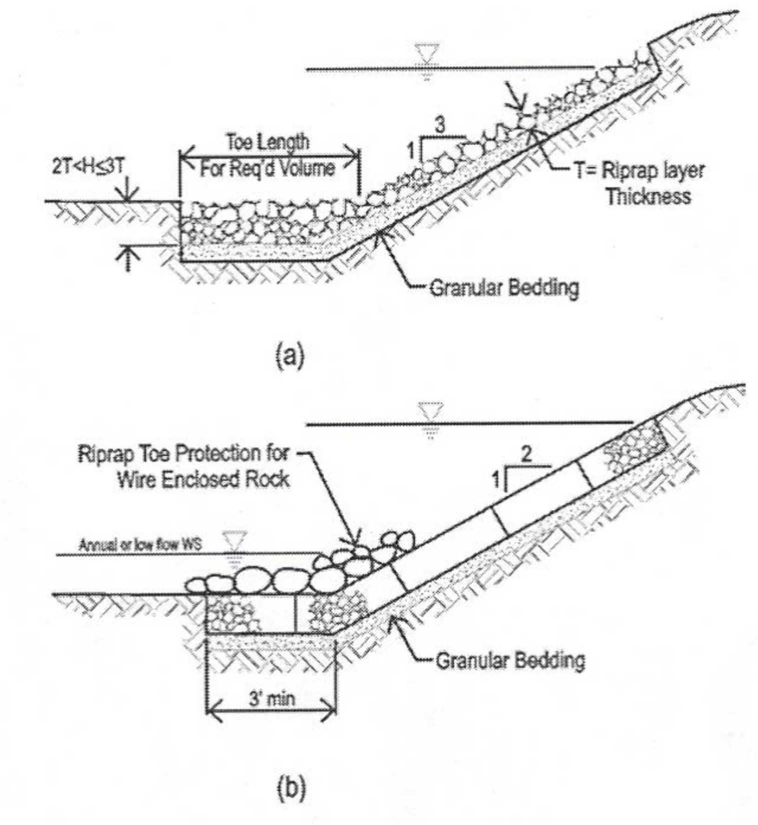
4. Downslope rock movement occurred without significant movement in the downstream direction.

5. Results from modeling and the subsequent prototypes show that the recommended weighted toe designs launch at a slope slightly steeper than 2:1.

6. Toe volume in the physical model was approximately equal to the volume needed to extend the bank protection to the maximum scour depth at a 2:1 slope. Linder (1976) recommends a toe volume equal to 1.5 times the volume of extending the bank protection to the maximum scour depth.

Weighted riprap toes have been used successfully for many years. However, success has not been universal. A common factor among the failures appears to be the presence of impinged flow on the bank. Therefore, the guidelines herein apply chiefly to flow conditions parallel to the bank. Where impinged flow is likely, then analyses must be made to determine an appropriate additional level of protection for such flow conditions.

Figure 6. 5: Toe Protection Channel Lining



6.4 APPLICATION

6.4.1 Concrete Lined Channels

Reinforced concrete and shotcrete are alternative lining materials for channels with limited right of way and/or high velocity flow. The most common problems of concrete lined channels are due to bedding and liner failures. Typical failures are:

1. liner cracking due to settlement of the subgrade

2. liner cracking due to the removal of bed and bank material by seepage force

3. liner cracking and floating due to hydrostatic back pressure from high groundwater

Lack of maintenance can result in vegetation growth through the concrete lining and sediment deposition in the channel that will increase the flow resistance. This reduction in channel capacity can cause overflow at design discharges and, consequently, permit the erosion of overbank material and failure of concrete lining.

Concrete lined channels are usually designed for supercritical flow conditions and/or when velocities exceed five feet per second for earth lined channels. Froude Numbers for supercritical flow shall be greater than 1.13 and less than 2.0. Unstable flow conditions occur when the Froude number falls between 0.86 and 1.13 and must be avoided.

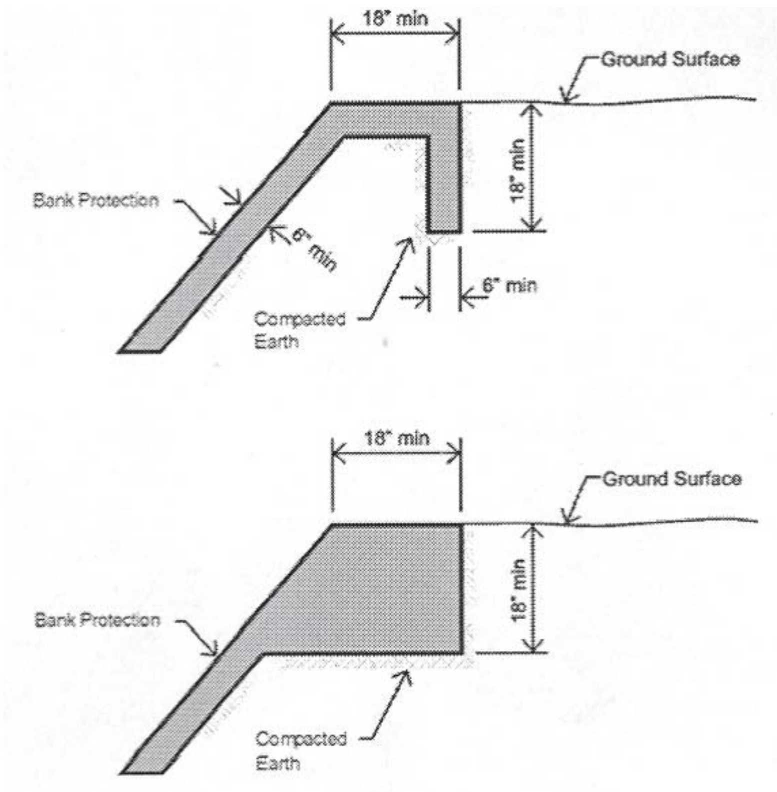
Supercritical flow in an open channel in an urbanized area creates certain hazards that the designer must take into consideration. From a practical standpoint it is generally unwise to have any curvature in a supercritical channel. Careful attention must be taken to prevent or control excessive oscillatory waves that may extend the entire length of the channel from only minor obstructions upstream. Imperfections at joints may rapidly cause a deterioration of the joints, in which case a complete failure of the channel can readily occur. High velocity flow can enter cracks or joints and create uplift forces by the conversion of velocity head to pressure head causing damage to the channel lining. It is evident that when designing a lined channel with supercritical flow, the designer must use utmost care and consider all relevant factors.

All concrete lined channels must have continuous reinforcement extending both longitudinally and laterally. For channels carrying supercritical flow, there shall be no reduction in cross sectional area at bridges or culverts, or any obstructions in the flow path.

Bridges or other structures crossing the channel must be anchored satisfactorily to withstand the full dynamic load that might be imposed upon the structure in the event of major debris blockage. Tributary stormdrain pipelines must not protrude into the channel flow area.

Generally, if side slopes steeper than 2:1 are used, then safety and structural requirements become a primary concern. To determine the thickness of the lining refer to ADOT (1989). Design of the lining should also include consideration of anticipated vehicular loading from maintenance equipment. Joints in the lining should be designed in accordance with standard structural analysis procedures with consideration of the size of the channel, thickness of the lining and anticipated construction techniques. The concrete lining must be keyed into the adjacent over- banks as shown in Figure 6.6.

Figure 6. 6: Typical Bank-Protection Key-Ins



The roughness coefficient for a concrete lining can vary from 0.011 for a troweled finish to 0.020 for a very rough or unfinished surface. For shotcrete, roughness coefficients can vary from 0.016 to 0.025. The accumulation of sediment and debris must be taken into account when determining the roughness coefficient.

Long-term stability of concrete lined channels depends in part on proper bedding. Undisturbed soils often are satisfactory for a foundation for lining without further treatment. Expansive clays are usually an extreme hazard to concrete lining and should be avoided. A filter underneath the lining is recommended to protect fine material from creeping along the lining. A well-graded gravel filter should be placed over the channel bed prior to lining the channel with concrete.

Since concrete-lined channels are often used at locations where excessive seepage exists or smaller channel cross sections are required, transitions will be required both upstream and downstream of the concrete lined channel. Such transitions are intended to prevent undermining of the lining and to reduce turbulence. Transitions should be lined with concrete or other scour resistant material to reduce scour potential.

Cutoff walls should be incorporated with transitions at both the upstream and downstream end of the concrete lined channel to reduce seepage forces and prevent lining failure due to scour, undermining, and piping. The depth of cutoff walls should extend below the expected scour depth. Determination of expected total scour depth requires analyses as discussed in Chapter 10.

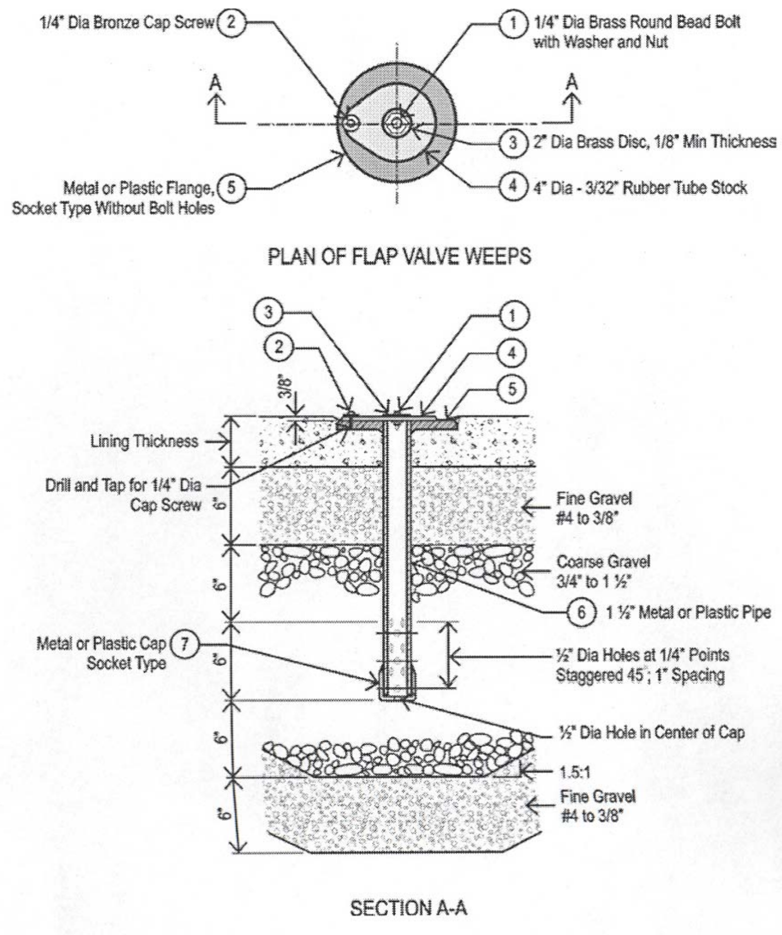
The probability of damaging the concrete lining due to hydrostatic back pressure and subgrade erosion can be greatly reduced by providing underdrains. There are two types of artificial drainage installations. One type consists of 4- or 6-inch diameter perforated pipelines placed in gravel-filled trenches along one or both toes of the inside slopes. These longitudinal drains are either connected to transverse cross drains which discharge the water below the channel or to pump pits, or extend through the lining and connect to outlet boxes on the floor of the channel. The outlet boxes are equipped with one-way flap valves that prevent backflow and relieve any external pressure that is greater than the water pressure on the upper surface of the channel bottom. The second type consists of a permeable gravel blanket of selected material or sand and gravel pockets, drained into the channel at frequent intervals (10 to 20 feet) by flap valves in the channel invert. Figure 6.7 shows a drawing of a flap valve for use without tile pipe and in a fine gravel and sand subgrade. Both the tile and pipe system and the unconnected flap valve type must be encased in a filter that will prevent piping of subgrade material into the pipe or through the valve. For detailed information on underdrains refer to Lining for Irrigation Canals (USBR, undated).

Where a lesser degree of seepage control is warranted, weep holes spaced at appropriate intervals may be used. When embankment stability may be compromised or when ground water levels may be raised by back drainage from the lined channel, weep holes may be equipped with flap valves or other measures that allow seepage relief but prevent backflow or introduction of surface water behind the lining.

The shotcrete process has become an important and widely used technique. Shotcrete is mortar or concrete pneumatically projected at high velocities onto a surface. In the past, the term 'gunite' was commonly used to designate dry-mix mortar shotcrete. The term is currently outdated and 'shotcrete' has become the trade name for all pneumatically applied dry-mix or wet-mix concrete or mortar.

ACI 506R (1985) discusses the properties, applications, materials, reinforcement, equipment, shotcrete crews, proportioning, batching, placement, and quality control of the shotcrete process.

Figure 6. 7: Flap Valve Installation for a Channel Underdrain



As a channel lining, shotcrete is an acceptable method of applying concrete with a general improvement in density, bonding, and decreased permeability. The same design considerations discussed for concrete channels apply in the design of shotcrete channels. Shotcrete linings are to be designed to the same thickness and reinforcement as required for concrete linings. Given the limitations of construction, the minimum slope for concrete and shotcrete channels is 0.0015 ft/ft.

6.4.2 Soil Cement Lined Channels

Soil cement has been shown to be an effective and economical method for slope protection and channel lining in many Arizona areas.

6.4.2.1 Materials

A wide variety of soils can be used to make durable soil cement. For maximum economy and most efficient construction, it is recommended that:

1. The soil contains no material retained on a 3-inch (75 mm) sieve

2. Between 40 percent and 80 percent pass the No.4 (4.75 mm) sieve
3. Between 2 percent and 10 percent pass the No. 200 (0.074 mm) sieve
4. The Plasticity Index (PI) of the fines should not exceed 10

If the onsite material does not meet these guidelines, the addition of import material may be necessary. Standard laboratory tests are available to determine the required proportions of cement and moisture to produce durable soil cement. The design of most soil cement for water control projects is based on the cement content indicated by ASTM testing procedures and increased by a suitable factor to account for direct exposure, erosion or abrasion forces.

The Portland cement should comply with one of the following specifications: ASTM C150, CSA A5, or AASHTO M85 for Portland cement of the type specified; or ASTM C595 or AASHTO M240 for Portland blast-furnace slag or Portland pozzolan cement, excluding slag cements Types S and SA.

It is important that testing to establish required cement content be done with the specific cement type, soil, and water that will be used in the project.

Typically, soil cement linings are constructed by the central-plant method, where selected onsite soil materials, or soils borrowed from nearby areas, are mixed with Portland cement and water and transported to the site for placement and compaction.

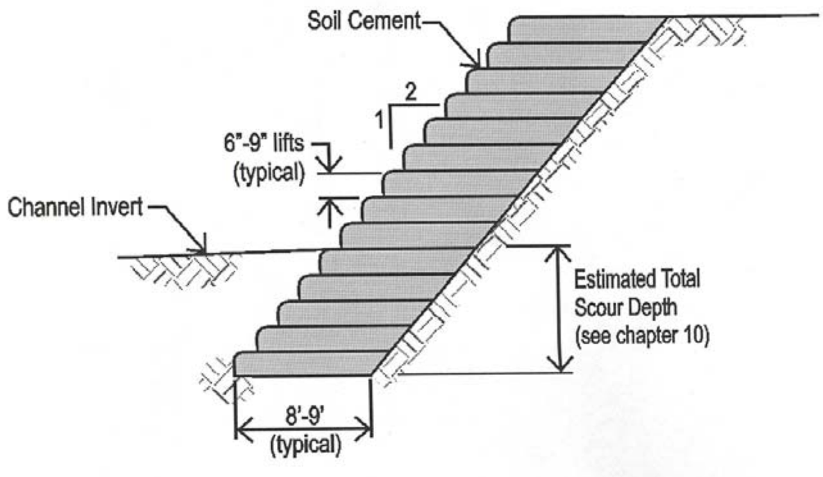
6.4.2.2 Design of Soil Cement Linings

Figure 6.8 shows a composite channel consisting of an earth bottom with soil cement stabilization along the banks. On side slopes, the soil cement is often constructed by placing and compacting the material in horizontal layers stair-stepped up the slope. The rounded step facing results from ordinary placement and compaction methods. Generally, an 8 to 9 foot minimum working width is required for placement and compaction of the soil cement layers by standard highway construction equipment. A width of 9-feet is preferred for maintenance and safety reasons. Figure 6.9 shows the relationship between slope of facing, thickness of compacted horizontal layer, horizontal layer width and minimum facing thickness measured normal to slope. For a horizontal working width of 9 feet, a side slope of 2: 1 and 6-inch thick layers, the resulting minimum thickness of facing would be about 4 feet, measured normal to the slope. The sideslope can vary from 1: 1 to 3: 1 depending on the soil type and natural angle of repose. Side slopes steeper than 2:1 are not recommended, due to safety issues, but may be allowed when right-of- way is a problem. Soil cement may be placed on slopes 3:1 or flatter at a minimum thickness of eight to twelve inches, depending upon the mixing technique. This would be done without the stair-step layer approach, where a lesser level of protection is permissible.

An important consideration in the design of the soil cement facing is to provide that all extremities of the facing are tied into non-erodible sections or abutments.

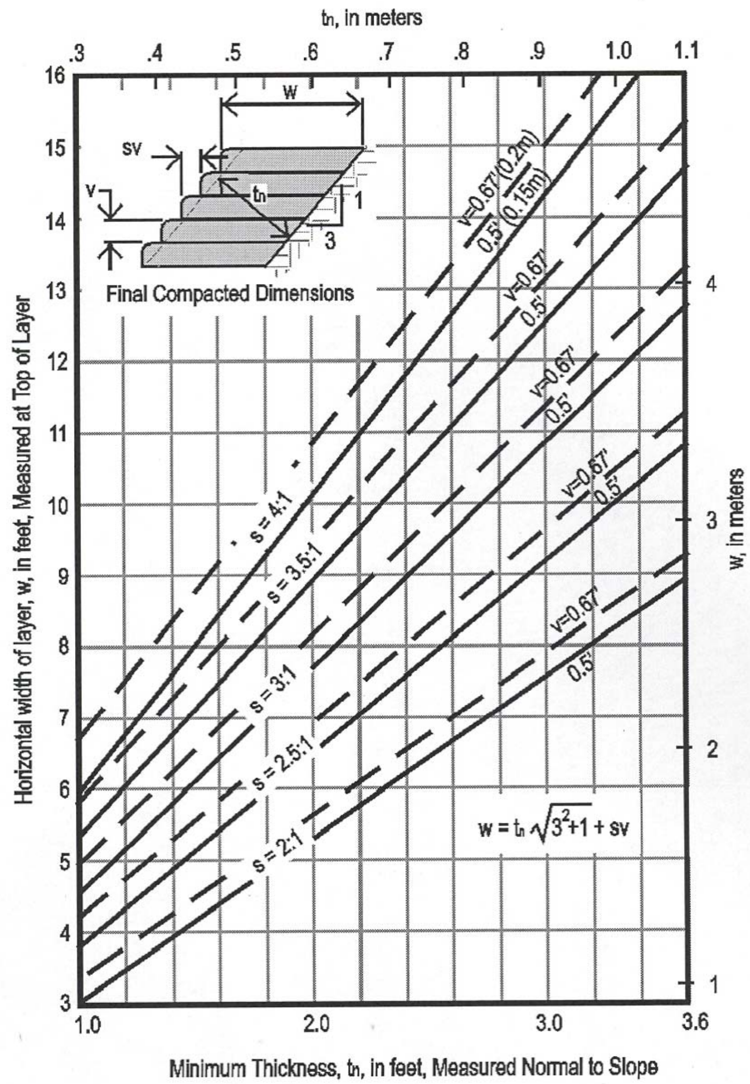
The upstream and downstream ends of the facing should terminate smoothly into the natural channel banks. A buried cutoff wall normal to the slope or other measures may be necessary to prevent undermining of the soil cement facing by flood flows.

Figure 6. 8: Soil Cement Placement Detail



The top of the lining should be keyed into the ground to protect against erosion of the backside of the soil cement layer by lateral inflows, as shown in Figure 6.6. As with any impervious channel lining system, seepage and related uplift forces should be considered and, if required, appropriate counter-measures provided, such as weep holes or subdrains. Tributary stormdrain pipelines can normally be accommodated by placing and compacting the soil cement by hand, using small power tools, or by using a lean mix concrete. For earthen channels with soil cement side slope protection, the lining should be designed to extend to the anticipated depth of total scour. Further design information may be found in ACI 230.1, State Of The Art Report on Soil Cement. Additional information on design and construction is available from the Portland Cementment Association, Skokie, IL.

Figure 6. 9: Relationships for Soil Cement Lining, Slope, Facing Thickness, Layer Thickness, and Horizontal Layer Width



6.4.3 Riprap Lined Channels

Common riprap can be an effective lining material if properly designed and constructed. The choice of riprap usually depends on the availability of graded rock with suitable material properties and at a cost that is competitive with alternative lining systems.

Riprap design involves the evaluation of five performance areas. These areas include the evaluation of:

- riprap quality
- riprap layer characteristics
- hydraulic requirements

- site conditions
- river conditions

In Arizona, site requirements and river conditions are important factors in the protection of bridge structures and flood control channels.

6.4.3.1 Riprap Quality

Riprap quality determination refers to the physical characteristics of the rock particles that make up the bank protection. Qualities determined to be most important include density, durability, and shape. Requirements for each of these properties are summarized in this section.

Specific Gravity (Density)

The design stone size for a channel depends on the particle weight, which is a function of the density or specific gravity of the rock material. A typical value of specific gravity in Pinal County is 2.4. All stones composing the riprap should have a specific gravity equal to or exceeding 2.4, following the standard test ASTM C127.

Durability

Durability addresses the in-place performance of the individual rock particles, and also the transportation of riprap to the construction site. In-place deterioration of rock particles can occur due to cycles of freezing and thawing, or can occur during transportation to the site. The rock particles must have sufficient strength to withstand abrasive action without reducing the gradation below specified limits. Qualitatively, a stone that is hard, dense, and resistant to weathering and water action should be used. Rocks derived from igneous and metamorphic sources provide the most durable riprap.

Laboratory tests should be conducted to document the quality of the rock. Specified tests that should be used to determine durability include: the durability index test and absorption test (see ASTM C127). Based on these tests, the durability absorption ratio (DAR) is computed as follows:

$$DAR = \frac{DurabilityIndex}{PercentAbsorption + 1} \tag{6.11}$$

The following specifications are used to accept or reject material:

1. DAR greater than 23, material is accepted
2. DAR less than 10, material is rejected
3. DAR 10 through 23
    a. Durability index 52 or greater, material is accepted
    b. Durability index 51 or less, material is rejected.

6.4.3.2 Shape

There are two basic shape criteria. First, the stones should be angular. Angular stones with relatively flat faces will form a mass having an angle of internal friction greater than rounded stones, and therefore will be less susceptible to slope failures. Second, not more than 25 percent of the stones should have a length more than 2.5 times the breadth. The shape of the riprap stone should be cubical, rather than elongated. Cubical stones nest together, and are more resistant to movement. The length is the longest axis through the stone, and the breadth is the shortest axis perpendicular to the length. Angularity is a qualitative parameter which is assessed by visual inspection. No standard tests are used to evaluate this specification. If the engineer is faced with a supply of rounded river rock without a crusher to create angular rock, stone size should be increased 25% and side slopes decreased (USACE, 1995).

6.4.3.3 Riprap Layer Characteristics

The major characteristics of the riprap layer include: characteristic size; gradation; thickness; and filter-blanket requirements.

Characteristic Size

The characteristic size in a riprap gradation is the d_{50}. This size represents the average diameter of a rock particle for which 50 percent of the gradation is finer, by weight.

Gradation

To form an interlocked mass of stones, a range of stone sizes must be specified. The object is to obtain a dense, uniform mass of durable, angular stones with no apparent voids or pockets. The recommended maximum stone size is 2 times the d_{50} and the recommended minimum size is one-third of the d_{50}.

The gradation coefficient, G, should equal 1.5.

$$G = 0.5\left({}^{d_{84}}\!/_{d_{50}} + {}^{d_{50}}\!/_{d_{16}} \right) \qquad (6.12)$$

Table 6.5 provides design gradations for riprap. As a practical matter, the designer should check with local quarries and suppliers regarding the classes and quality of riprap available near the site.

Table 6- 6: RIPRAP GRADATION LIMITS
(US DOT, FHWA, HEC-11)

Stone Size Range (ft.)	Stone Weight Range (lb)	Percent of Gradation Smaller Than
1.5 d_{50} to 1.7 d_{50}	3.0 W_{50} to 5.0 W_{50}	100
1.2 d_{50} to 1.4 d_{50}	2.0 W_{50} to 2.75 W_{50}	85
1.0 d_{50} to 1.15 d_{50}	1.0 W_{50} to 1.5 W_{50}	50
0.4 d_{50} to 0.6 d_{50}	0.1 W_{50} to 0.2 W_{50}	15

Thickness

The riprap-layer thickness shall be the greater of 1.0 times the d_{100} value, or 1.5 times the d_{50} value. But the thickness need not exceed twice the d_{100} value. The thickness is measured perpendicular to the slope upon which the riprap is placed.

6.4.3.4 Filter Blanket Requirements

The purpose of granular filter blankets underlying riprap is two-fold. First, they protect the underlying soil from washing out; and, second, they provide a base on which the riprap will rest. The need for a filter blanket is a function of particle-size ratios between the riprap and the underlying soil which comprise the channel bank. The inequalities that must be satisfied are as follows:

$$\frac{(d_{15})_{filter}}{(d_{85})_{base}} < 5 < \frac{(d_{15})_{filter}}{(d_{15})_{base}} < 40 \qquad (6.13)$$

$$\frac{(d_{50})_{filter}}{(d_{50})_{base}} < 40 \qquad (6.14)$$

In these relationships, "filter" refers to the overlying material and "base" refers to the underlying material. The relationships must hold between the filter blanket and base material and between the riprap and filter blanket (USDOT, 1988 and 1989).

If the inequalities are satisfied by the riprap itself, then no filter blanket is required. If the difference between the base material and the riprap gradations are very large, then multiple filter layers may be necessary. To simplify the use of a gravel filter layer, Table 6.6 outlines recommended standard gradations.

The Type-I and Type-II bedding specifications shown in Table 6.6 were developed using the criteria given in Equation (6.13) and Equation (6.14), considering that very fine grained, silty, non-cohesive soils can be protected with the same bedding gradation developed for a mean grain size of 0.045 mm. The Type-I bedding in Table 6.6 is designed to be the lower layer in a two-layer filter for protecting fine grained soils. When the channel is excavated in coarse sand and gravel (i.e., 50 percent or more by weight retained on the No. 40 sieve), only the Type-II filter is required. Otherwise, two bedding layers (Type-I topped by Type-II) are required. For the required bedding thickness, see Table 6.7.

Table 6- 7: Gradation for Gravel Bedding
(Simons, Li and Associates, 1989)

Standard Sieve Size	Type I [(1)]	Type II [(1)]
3 inches	-	90 to 100
1-1/2 inches	-	-
3/4 inch	-	20 to 90
3/8 inch	100	-
#4 (4.75 mm)	95 to 100	0 to 20
#16 (1.18 mm)	45 to 80	-
#50 (0.30 mm)	10 to 30	-
#100 (0.15 mm)	2 to 10	-
#200 (0.075 mm)	0 to 2	0 to 3

(1) Percent passing by weight

Table 6- 8: Thickness Requirements for Gravel Bedding

Riprap Size Classification, inches	Minimum Bedding Thickness, inches		
	Fine Grain Native Soils		Coarse Grain Native Soils
	Type I	Type II	Type III
6, 8	4	4	6
12	4	4	6
18	4	6	8
24	4	6	8
30	4	8	10
36	4	8	10

Filter Fabric Requirements

The design criteria for filter fabric are a function of the permeability of the fabric and the effective opening size. The permeability of the fabric must exceed the permeability of the underlying soil, and the apparent opening size (AOS) must be small enough to retain the soil.

The criteria for apparent opening size are as follows:

1. For soil with less than 50 percent of the particles, by weight, passing a No. 200 sieve, the AOS should be less than 0.6 mm (a No. 30 sieve).

2. For soil with more than 50 percent of the particles, by weight, passing a No. 200 sieve, the AOS should be less than 0.3 mm (a No. 50 sieve).

Filter fabric is not a complete substitute for granular bedding. Filter fabric provides filtering action only perpendicular to the fabric and has only a single equivalent pore opening between the channel bed and the riprap. Filter fabric has a relatively smooth surface which provides less resistance to stone movement. Tears in the fabric greatly reduce its effectiveness so that direct dumping of riprap on the filter fabric is not allowed and due care must be exercised during construction. The site conditions and specific application and installation procedures must be care- fully considered in evaluating filter fabric as a replacement for granular bedding material. Filter fabric can provide an adequate bedding for channel linings along uniform mild sloping channels where leaching forces are primarily perpendicular to the fabric.

Numerous failures have occurred because of the improper installation of filter fabric. Therefore, when using filter fabric it is critical that the manufacture's guidelines for installing it be followed.

6.4.3.5 Hydraulic Design Requirements

Channel linings constructed of placed, graded riprap or gabions to control channel erosion have been found to be cost effective where channel reaches are relatively short and where a nearby source of quality rock is available.

Situations where riprap or gabion basket linings may be appropriate are:

1. Major flows are found to produce channel velocities in excess of allowable non-eroding values

2. Channel side slopes at 3:1 for riprap and 2:1 for gabion mattresses

3. Where rapid changes in channel geometry occur, such as channel bends and transitions.

This section presents design requirements for common riprap, while Section 6.6.4 contains additional design considerations specifically related to gabions. Both sections are valid only for subcritical flow conditions where the Froude Number is 0.86 or less.

Riprap Sizing

Several reference sources are available for design procedures. Two recommended sources are:

1. Design of Riprap Revetment (Federal Highway Administration, Hydraulic Engineering Circular No. 11, Publication No. FHWA-IP-89-016, March 1989)

2. Hydraulic Design of Flood Control Channels (Corps of Engineers, EM-1110-2-1601,1991)

The riprap sizing method presented here is from HEC-11 (for a complete discussion on this method the designer is referred to the above referenced documents). This method is based on tractive force (shear stress) theory but with velocity as its primary design parameter. This is a blend between the two approaches of permissible velocity and permissible tractive force. The hydraulic assumptions are uniform, steady, subcritical flow. However, adjustments to the design equation are provided for other regimes and conditions such as gradually varying flow and approaching rapidly varying flow. In this method, the riprap size is selected such that the flow induced tractive force does not exceed the critical shear stress of the riprap. The critical shear is based on Shield's relationship, a function of specific weight of water, specific weight of the riprap material, the median rock size (d_{50}), Shields parameter, and a factor that is a function of the bank angle and riprap's material angle of repose. The average shear stress or tractive force exerted by flowing water is the product of unit weight of water, energy grade line slope and hydraulic radius. These two equations are combined to develop the design tractive force relationship in terms of a stability factor (SF). The stability factor is defined as the ratio of the average tractive force exerted by the flow field and the riprap materials critical shear stress. Therefore if the stability factor is greater than 1.0, the critical shear stress is greater than the flow induced tractive stress and the riprap is considered stable.

For the HEC-11 method the d_{50} (ft) is determined by:

$$d_{50} = \frac{0.001 V_a^{\ 3}}{d_{avg}^{\ \ 0.5} K_1^{\ 1.5}} \tag{6.15}$$

Where V_a (ft/sec) is the average velocity in the main channel, d_{avg} (ft) is the average flow depth in the main channel, and K_1 is the bank angle correction factor. The bank angle correction factor is determined using Equation (6.16).

$$K_1 = \left[1 - \frac{\sin^2\theta}{\sin^2\phi}\right]^{0.5} \tag{6.16}$$

Where θ is the bank angle with the horizontal, Φ is the riprap material's angle of repose. The bank angle correction factor can also be determined using Figure 6.10. The riprap material's angle of repose can be determined using Figure 6.11.

Figure 6. 10: Bank Angle Correction Factor, K1

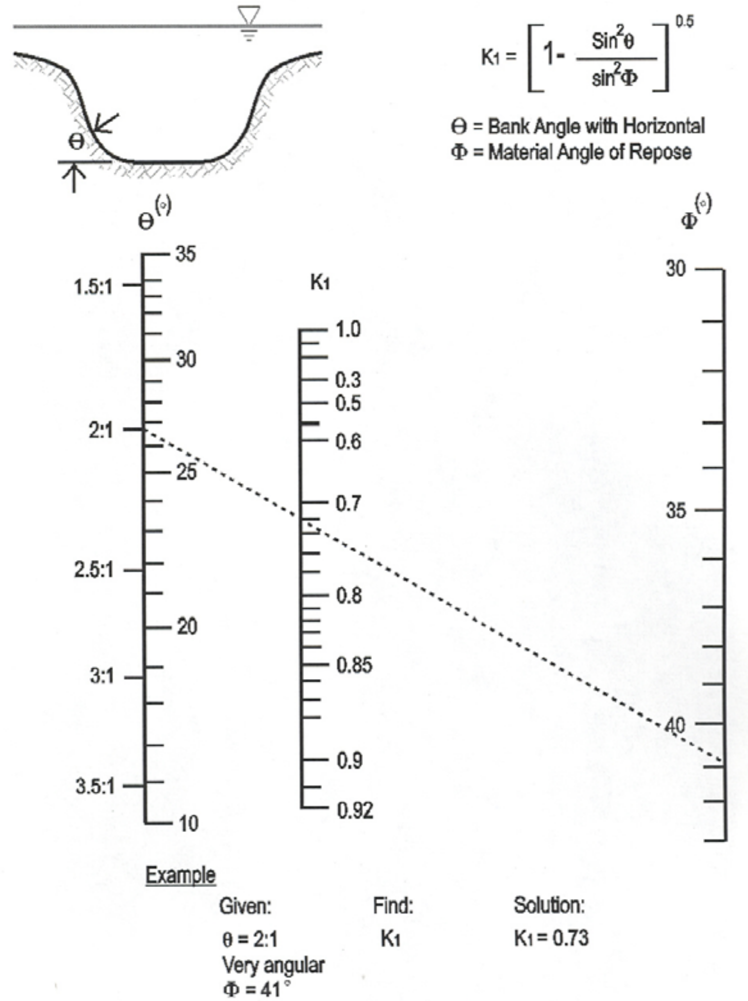
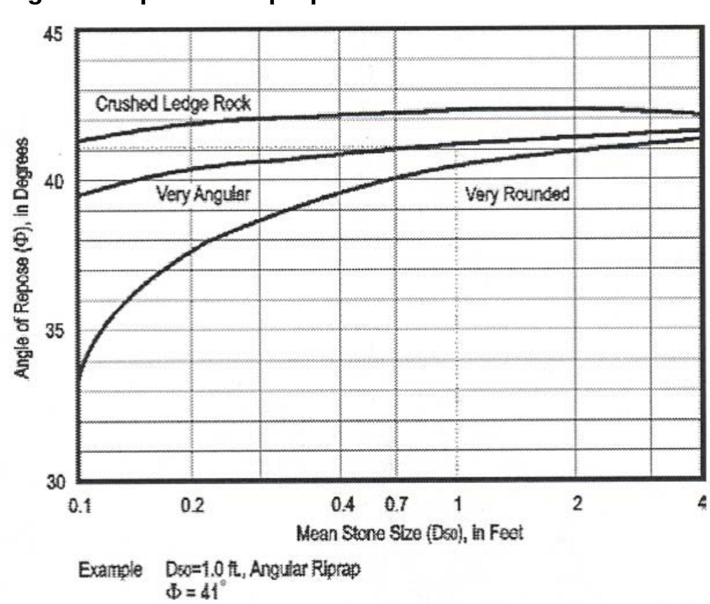


Figure 6. 11: Angle of Repose of Riprap in Terms of Mean Size and Shape of Stones



Equation (6.15) is based on a rock riprap specific gravity of 2.65, and a stability factor of 1.2. Equation (6.17) and Equation (6.36) present correction factors for other specific gravities and stability factors.

$$C_{sg} = \frac{2.12}{(S_s - 1)^{1.5}} \tag{6.17}$$

Where:

S_s is the specific gravity of the rock riprap.

$$C_{sf} = \left(\frac{SF}{1.2} \right)^{1.5} \tag{6.18}$$

Where (SF) is the stability factor to be applied. Table 6.8 presents guidelines for the selection of an appropriate value for the stability factor.

The correction factors computed using Equation (6.16) and Equation (6.17) are multiplied together to form a single correction factor C. This correction factor is then multiplied by the riprap size computed from Equation (6.15) to arrive at a stable riprap size.

The stability factor is used to reflect the uncertainty in the hydraulic conditions at a particular site. Equation (6.15) is based on the assumption of uniform or gradually varying flow. In many instances, this assumption is violated or other uncertainties come to bear. For example, debris and/or ice impacts, or the cumulative effect of high shear stresses and forces from wind and/or boat generated waves. The stability factor is used to increase the design rock size when these conditions must be considered. Typically, the minimum thickness of riprap linings should be the greater of 1 x d_{100} or 1.5 x d_{50}.

Table 6- 9: Stability Factors
(USDOT, FHWA, HEC-11, 1989)

Condition	Stability Factor
Uniform Flow: Straight or mildly curving reach (curve radius/channel width> 30); Impact from wave action and floating debris is minimal; Little or no uncertainty in design parameters.	1.0 -1.2
Gradually Varying Flow: Moderate bend curvature (30 > curve radius/channel width> 10); Impact from wave action and floating debris is moderate.	1.3 -1.6
Approaching rapidly varying flow: Sharp bend curvature (10 > curve radius/channel width); Significant impact potential from floating debris and/or ice; Significant wind and/or boat generated waves (1-2 ft); High flow turbulence; Turbulently mixing flow at bridge abutments; Significant uncertainty in design parameters.	1.6 -2.0

6.4.3.6 Grouted Rock

Grouted rock is a structural lining comprised of a blanket of rock that is interlocked and bound together by means of concrete grout injected into the void spaces to form a monolithic revetment. The grout must extend the full thickness of the rock blanket, with the face rocks exposed for a maximum of one-fourth to one-third of their depth.

This lining type is often suggested as a substitute for adequately sized riprap. It is not an equivalent product because it is neither rigid nor flexible. Any movement or settlement of the subgrade immediately results in cracks in the matrix that, in turn, allows water to enter behind the lining and greatly accelerate the lining's destruction. Some jurisdictions do not accept this alternative and its use is discouraged with two exceptions; riprap designed by the guidelines contained herein can be grouted to 1) minimize vandalism and/or 2) to inhibit the growth of volunteer vegetation and to aid in maintenance.

6.4.4 Gabion lined Channels

Gabions refer to rocks that are confined by a wire basket so that they act as a single unit. The wire mesh enclosed rock units are also known as gabion baskets or gabion mattresses. One of the major advantages of wire-enclosed rock is that it provides an alternative in situations where available rock sizes are too small for common riprap. Another advantage is the versatility that results from the regular geometric shapes of wire-enclosed rock. The rectangular blocks and mats can be fashioned into almost any shape that can be formed with concrete. The durability of wire-enclosed rock is generally limited by the service life of the galvanized binding wire, which under normal conditions here in the arid southwest, is considered to be about 35 years. In applications where the gabions are subjected to frequent wet conditions, the life span diminishes to about 15 years (Myers, 2000). Water carrying silt, sand or gravel can reduce the service life of the wire. Also, water that rolls or otherwise moves cobbles and large stones breaks the wire with a hammer and anvil action and considerably shortens the life of the wire. The wire has been found to be susceptible to corrosion by various chemical agents and is particularly affected by high sulfate soils. If corrosive agents are known to be in the water or soil, a plastic coated wire should be specified. The designer should verify site specific conditions and coordinate with a qualified manufacturer to properly specify gabion wire. See ASTM A-974 and ASTM A-975.

Gabions are not maintenance free and must be periodically inspected to determine whether the wire is sound. If breaks are found while they are still relatively small, they may be patched by weaving new strands of wire into the wire cage. Wire enclosed rock installations have been found to attract vandalism. Flat mattress surfaces seem to be particularly susceptible to having wires cut and stones removed. It is recommended that, where possible, mattress surfaces be buried, where they are less prone to vandalism. Wire enclosed rock installations should be inspected at least once a year under the best circumstances and may require inspection every three months in vandalism prone areas in conjunction with a regular maintenance program. They should also be inspected after high flow events. Under high flow velocity conditions, mattresses on sloping surfaces must be securely anchored to the surface of the soil as dig-cussed previously.

6.4.4.1 Materials

Rock and Wire Enclosure Requirements

Rock filler for the wire baskets should meet the rock property requirements for common riprap. Rock sizes and basket characteristics should meet ASTM A-974 and ASTM A-975. The minimum rock size do should be equal to the size of the gabion mesh opening. The maximum rock size d1oo should be less than the gabion thickness.

Bedding Requirements

Long term stability of gabion (and common riprap) erosion protection is strongly influenced by proper bedding conditions. A large percentage of all riprap failures are directly attributable to bedding failures, which is particularly disturbing in light of the fact that over half of all riprap installations experience some degree of failure within 10 years of construction. Refer to Section 6.4.3 for gravel bedding or filter design. Non-woven, 8-ounce filter fabric has been found acceptable in many applications. The design engineer should check with the manufacturer for its given application.

6.4.4.2 Design Considerations

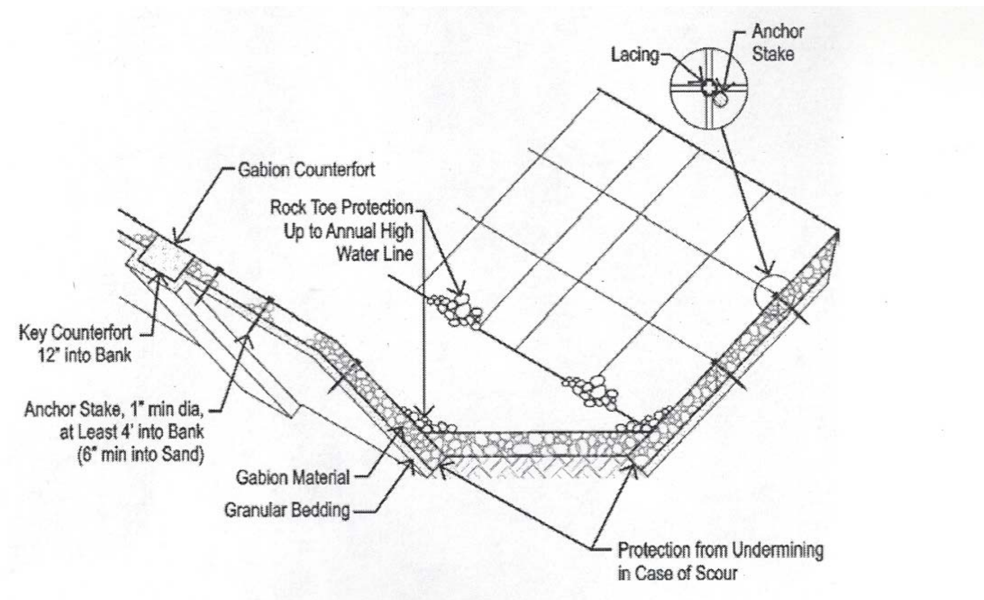
The geometric properties of gabions permit placement in areas where common riprap is either difficult or impractical to place. Proper design and construction is important to successful operation and lifetime performance. Twisted wire mesh has been found to be more tolerant to settlement than welded wire mesh (See ASTM A-975).

Slope Mattress Lining

Figure 6.12 shows a typical configuration for a gabion slope mattress channel lining. The long side of the gabion basket should be aligned parallel with the channel for applications on banks steeper than 2: 1. Channel linings should be tied to the channel banks with gabion counterforts (thickened gabion sections that extend into the channel bank) at the upstream edge of the lining. Counterfort spacing shall be per manufacturer's recommendations.

Mattresses and flat gabions on channel side slopes need to be tied to the banks. The ties should be metal stakes no less than 4 feet in length (sandy soils warrant longer lengths). These should be located at the inside corners of basket diaphragms along an upslope (highest) basket wall, so that the metal stakes are an integral part of the basket. The exact spacing of the stakes depends upon the configuration of the baskets, however the following is the suggested minimum spacing: stake every 6 feet along and down the slope for 2: 1 slopes or steeper. Channel linings should be tied to the channel banks with gabion counterforts (thickened gabion sections that extend into the channel bank) at the upstream edge of the lining. For most applications, mattresses should be a minimum of 9 inches thick.

Figure 6. 12: Slope Matress Lining



6.4.5 Design Documentation Requirements for Major Watercourses

The following guidelines should be used for all watercourses subject to submittal for FCDMC and FEMA review. These are primarily for watercourses with flows in excess of 2000 cfs.

6.4.5.1 Open Channel Hydraulics

HEC-RAS or HEC-2 shall be used to perform water surface profile calculations. Alternative methods require approval. A hard copy and floppy disk/CD-ROM with input and output files shall be submitted for County review. The HEC input and output files shall be prepared in a format suitable for submittal to FEMA, using Requirements for Flood Study Technical Documentation, ADWR 1997.

The starting water-surface elevations for profile computations for mainstreams and tributaries should be based on FEMA requirements (FEMA, 2002). In general, the starting water-surface elevations chosen for profile computations should be based on normal depth (or slope-area), unless known water-surface elevations are available from other sources. When using normal depth on the main stream, the model should be started several cross sections downstream of the beginning of the study reach. For starting conditions on tributaries, normal depth should be used unless a coincident peak situation is assumed, or the tributary flow depths are higher than the corresponding main stream events. The assumption of coincident peaks may be appropriate if a) the ratio of the drainage areas lies between 0.6 and 1.4, b) the times of peak flows are similar for the two combining watersheds, and c) the likelihood of both watersheds being covered by the storm being modeled are high. If gage records are available for the basin, guidance for coincidence of peak flows should be taken from them.

The Consultant shall estimate blockage due to debris at bridge piers based on field conditions. As a minimum, use the greater of 2 times the diameter of the pier or 1 foot on each side of the pier.

Freeboard for levees shall, as a minimum, comply with FEMA freeboard criteria: 3 feet of freeboard at the 100-year peak stage plus one foot additional at bridges. Refer to the local jurisdiction Policies and Standards Manual for possible more stringent conditions.

Locations of cross sections used in the water surface profile calculations shall be provided on a scaled map and also in a tabular format. The cross section labels on the maps shall reflect cross sections in the models (ADWR, 1997).

6.4.5.2 Channel Stabilization Design

Channel stability based on permissible velocity shall only be used for preliminary design purposes. The tractive shear stress approach shall be used to confirm unlined channel stability.

Provide calculations to show that the type of bank protection (common riprap, gabions, concrete, etc.) is suitably sized to resist hydraulic forces (tractive shear, impingement, buoyancy, etc.) at the design frequency peak flow.

Appropriate hydraulics and structural calculations should be provided for review. Refer to the local jurisdiction Policies and Standards Manual for requirements.

Consideration shall be given to how the upstream and downstream floodplain conditions will impact the proposed channel. The effects of existing and potential mining and fill operations shall be addressed. Overbank flooding upstream of the channelization shall be analyzed to demonstrate that design flows enter and are contained within the improved channelization. The design and analysis shall address the potential impacts of future modifications proposed by others. Gradual transition of the existing floodplain/floodway upstream and downstream of the channelization is required.

The minimum factor of safety applied to hydraulic forces on structural components shall be 1.5, based on the 100-year frequency peak flow.

The analysis shall address sediment transport, scour, lateral migration, and river mechanics as discussed in Chapter 7.

Plans submitted for review shall include profiles showing the top of levee protection, toe-down, hydraulic grade line, existing and design invert elevations at the thalweg, and the low chord elevations for bridges. Also, road and railway crossing locations must be shown on plans and profiles.

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VOLUME 2
DESIGN METHODOLOGY AND PROCREDURES

Chapter 7: Erosion and Sedimentation

7.1 INTRODUCTION

Sedimentation and the fluvial processes associated with sediment transport play an important role in the long-term conveyance capacity of a drainage system as well as the on-going cost of maintenance. Sedimentation is a very complex subject and not all drainage designs need to consider it as a primary design criterion.

This chapter provides some basic concepts of sedimentation and sediment transport. It provides some analytical methods, but perhaps more importantly, directs the interested reader to publications dealing with the processes involved. Specific references are provided throughout this chapter. Some useful general references in the topic of erosion and sedimentation include ASCE (1975), ADWR (1996), Richardson and others (2001), Simons and Senturk (1992), ADWR (1985), Guy (1970), Henderson (1987), Schumm (1977), SCS (1977), and U.S. Army Corps of Engineers (1994).

For those versed in this subject, this chapter identifies a checklist of issues for consideration during the design of drainage facilities, a three-tier approach to analysis, and the specific requirements of Pinal County. Those readers should turn to Section 7.5.1.

7.2 CONCEPTS

7.2.1 Erosion and Sedimentation Concerns

7.2.1.1 Watercourse Stabilization

Any watercourse modifications affecting the flow direction, depth, velocity or duration of discharge may result in erosion and sedimentation. This applies not only to flood control and drainage facilities, but any structural works to a watercourse may have discernable and potentially deleterious impacts regarding erosion and sedimentation to the watercourse. The following is a partial list of watercourse modifications and potential impacts:

- Channel straightening, will generally increase channel gradient and flow velocity, and may initiate channel erosion.
- Channel constriction increases flow velocities and often flow depth, thus increasing sediment transport capacity and may initiate channel erosion.
- Lowering the bed elevation of a watercourse may prompt degradation in the mainstem of the watercourse and its tributaries.
- Raising the bed elevation or reducing the slope of the energy grade line may result in sediment deposition upstream due to reduced transport capacity.
- Bank lining may increase flow velocity and increase erosion and/or bank attack where banks are left unprotected.

Alluvial channels are often in a balanced state of dynamic equilibrium (see Section 7.2.4) and even subtle changes in water discharge, watercourse hydraulic characteristics or sediment properties may result in erosion or sedimentation that is initiated promptly and proceeds rapidly. Accelerated erosion and sedimentation results in the destruction of natural conditions in the watercourse, may seriously depreciate land values by unsightly erosion and lost land, may decrease flood conveyance capacity by sediment deposition, and often requires expensive structural mitigation measures to restore the aesthetics and function of the watercourse.

7.2.1.2 Stormwater Storage

Stormwater storage facilities must be planned and analyzed in regard to their impact on watercourses. Those impacts are generally viewed within the context of the following:

- Sediment deposition in the impoundment and upstream backwater of the impoundment. This may result in decreased upstream conveyance capacity, the potential for breakout flows due to sediment deposits, and maintenance requirements in regard to sediment deposits.

- Release of “clear water” downstream of the impoundment. This may result in local scour and/or degradation of the downstream watercourse. Reduction of the continual replenishment of finer sediments may result in a changed character of the watercourse including the bed becoming more “cobbly” and loss of riparian vegetation.

- Although the peak discharges are usually reduced downstream of the storage facility, the duration of high flows often increases. This may increase the opportunity for scour and the flushing of finer sediments through the system.

7.2.1.3 Water Quality Issues

Sediment in water is often viewed as an undesired element for municipal and industrial water. The undesirable characteristics are often due to its quantity (volumetric) and its abrasive nature (size gradation and hardness).Chemicals such as pesticides, herbicides, organic compounds such as nitrogen and phosphorus, pathogens, and waste products can become attached to sediment particles and thereby be transported and stored along with sediment. Sediment can also be an asset such as for irrigated agriculture and for certain riparian habitats.

7.2.2 Channel Processes

Watercourses are either erodible or nonerodible, and either natural or artificial. In general, this section considers erodible channels, although sedimentation of nonerodible channels must also be considered as discussed in the next paragraph. Natural channels are those for which the form and dimensions are the result of a natural process. Artificial channels are constructed to predetermined dimensions and alignment. Natural channels are often analyzed for erosion and sedimentation under changed conditions, such as increased future flood flows. Artificial channels are typically designed based on stable dimensions and balance of sediment transport. Occasionally the question arises as to the design of modifications to natural channels such as local scour protection for structures built within the watercourse. Both natural and artificial channels must be considered in regard to sedimentation.

Although sedimentation, as defined herein, is limited to erodible channels, constructed channels of nonerodible material must be analyzed under reasonable conditions of sedimentation and sediment transport. For example, a fully lined concrete channel may have uncontrolled local runoff that can introduce a large amount of sediment load into the channel. Deposition of sediment in the channel can diminish the flow area, and also increase the resistance to flow. Under such conditions, the conveyance capacity of a channel can be reduced significantly. Therefore, constructed, nonerodible channels in Pinal County often must be analyzed and designed under conditions of sediment transport.

7.2.2.1 Regime

Many natural watercourses are in equilibrium with the major hydraulic parameters of width, depth, velocity and slope remaining constant even though erosion and sedimentation is occurring. Those watercourses are in dynamic equilibrium with realignments and bank erosion maintaining an overall system balance. Sediment inflow to a reach equals, on the average, sediment outflow. Such watercourses are said to be "in regime." Any factors or activities that would tend to upset these balance relations may result in the watercourse seeking a new regime.

The "in regime" approach is a means to describe watercourse form in terms of time-average magnitudes. A historic perspective of regime equations is provided by Schumm (1971), and practical guidance for selected regime equations are provided by Leopold and Maddock (1953) and Mahmood and Shen (1971). The regime concept is based on the premise that water discharge is the controlling factor in watercourse form. However, other factors such as sediment yield, upstream channelization, land use, vegetation and climate changes affect watercourse form.

Furthermore, the applicability of regime equations must be made with care since they are empirical. Regime equations should be applied only to watercourses that are "in regime." Stevens and others (1975) suggest that watercourses can be classified as "in regime" or not based on flood hydrology; watercourses with a low ratio of peak-flood discharge to the average annual peak-flood discharge can be in regime and regime-type equations applied. However, if the ratio is large, then the watercourse should exhibit nonequilibrium form for which regime equations do not apply. Stevens and others (1975) illustrate major watercourse widening and narrowing of the Gila River in the Safford Valley, Arizona, for a 125 year period, and during that time the ratio of peak-flood discharge to average annual peak-flood discharge was as large as 10. Based on streamform data, they conclude that the Gila River, where such conditions occur, is not in quasi or dynamic equilibrium and is not "in regime." Most watercourses in Pinal County, except possibly those for which flood peaks are controlled by upstream regulation, are expected to have large ratios of peak-flood discharges to average annual peak-flood discharge. Therefore, regime equations should be applied carefully, if at all.

7.2.2.2 Aggrading and Degrading Watercourses

Aggradation is the raising (filling) of the bed of a watercourse over some reach length as a result of incoming volume of sediment to the reach exceeding the outgoing volume of sediment. Degradation is the lowering (cutting) of the bed due to outgoing volume of sediment exceeding the incoming volume of sediment.

The aggradation or degradation process will continue until a new dynamic equilibrium is established and the watercourse is back “in regime.”

Long term bed elevation changes (aggradation or degradation) may be the natural trend of the watercourse or may be the result of some modification to the watercourse or watershed condition. Factors that affect long term bed elevation changes are; dams and reservoirs (upstream or downstream), changes in watershed land use (urbanization, deforestation, etc.), channelization, cutoff of a meander bend (natural or manmade), changes in the downstream base level (control), gravel mining, diversion of water into or out of the watercourse, natural lowering of the total system, and lateral watercourse movement such as bank erosion or migration.

7.2.2.3 Stream Forms

Watercourses are classified as straight, braided or meandering. Descriptions of these stream forms and quantitative criteria are presented for each in ASCE (1975).

Straight channels have the following characteristics:

- Essentially a straight alignment (low sinuosity).
- May have very flat slopes with nonerodible velocities.
- May have very steep slopes with high momentum that resists alignment changes.

Braided channels have the following characteristics:

- Formed by random interconnected channels separated by sand or gravel bars.
- Braided channels are often aggrading and may occur on flat, steep or moderate slopes.
- Often have high bed material transport during floods.
- Deposits form bars that are often vegetated.
- Distributary networks are types of braided stream form that occur on alluvial fans.

Flooding, erosion and sedimentation on alluvial fans is beyond the scope of this manual. The reader should consult the Maricopa County Flood Control District’s Piedmont Flood Hazard Assessment for Flood Plain Management for Maricopa County (Hjalmarson, 1998) and the National Research Council (1996).

Meandering channels have the following characteristics:

- Follow a winding course.
- Alignment tends to shift continuously by local erosion and opposing bank building.

7.2.3 Sediment Properties

Sedimentation is a function of the flow in the watercourse and of the properties of the sediment itself. The flow is a factor of the discharge and hydraulics of the conveyance system. The sediment properties of interest to typical fluvial sedimentation are; particle size, particle size distribution, fall velocity, specific gravity of sediment particles and specific weight of sediment deposits.

7.2.3.1 Sediment Particle Size

The commonly used sediment particle size scale and sieve number for sands and smaller gravel are shown in Table 7.1.

Table 7- 1: Sediment Grade Scale
(Lane, 1947; ASCE 1975)

Class name	**Size Range**		**Approximate Sieve Mesh Openings per inch**	
	Millimeters	***Inches***	***Tyler***	***United States Standard***
Very large boulders	4,096-2,048	160-80		
Large boulders	2,048-1,024	80-40		
Medium boulders	1,024-512	40-20		
Small boulders	512-256	20-10		
Large cobbles	256-128	10-5		
Small cobbles	128-64	5-2.5		
Very coarse gravel	64-32	2.5-1.3		
Coarse gravel	32-16	1.3-0.6		
Medium gravel	16-8	0.6-0.3	2-1/2	
Fine gravel	8-4	0.3-0.16	5	5
Very fine gravel	4-2	0.16-0.08	9	10
Very coarse sand	2.0-1.00		16	18
Coarse sand	1.0-0.5		32	35
Medium sand	0.5-0.25		60	60
Fine sand	0.25-0.125		115	120
Very fine sand	0.125-0.062		250	230
Coarse silt	0.062-0.031			
Medium silt	0.031-0.016			
Fine silt	0.016-0.008			
Very fine silt	0.008-0.004			
Coarse clay	0.004-0.002			
Medium clay	0.002-0.001			
Fine clay	0.001-0.0005			
Very fine clay	0.0005-0.00024			

Sediment size is measured by a length scale or "diameter." Two commonly used sediment size diameters as defined by ASCE (1975) are:

- Sieve diameter -The length of the side of a square sieve opening through which the given particle will just pass, and

- Sedimentation diameter - The diameter of a sphere of the same specific weight and the same terminal fall velocity as the given particle in the same sedimentation fluid.

The size of sand and larger sediment particles is usually expressed as sieve diameter. The size of silts and clays is generally expressed as a sedimentation diameter.

7.2.3.2 Size-Frequency Distributions

Natural sediments are made up of grains with wide ranges of size. Statistical methods are used to describe the size distribution. The size distribution of sand and larger particles is obtained by mechanical sieve analyses. For silt and smaller particle sizes a fall velocity method is typically used. Size gradation is usually presented as cumulative size-frequency curves, where the fraction or percentage by weight of sediment that is smaller or larger than a given size is plotted against the size. Figure 7.1 is a typical size distribution graph. The data are plotted on a semi-logarithmic graph. The median size, d_{50}, that is, the size for which 50 percent of the material is finer, can be read from the curve. Other values of interest are d_{16}, d_{84}, d_5 and d_{95}, defined similarly as d_{50}. The geometric mean size, d_g, is estimated as:

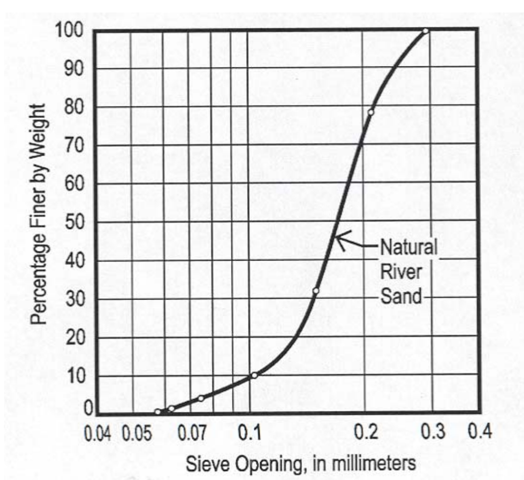
$$d_g = (d_{84} d_{16})^{1/2} \tag{7.1}$$

and the geometric standard deviation

$$\sigma_g = \left(\frac{d_{84}}{d_{16}} \right)^{1/2} \tag{7.2}$$

These statistics and others (see ASCE, 1975) are commonly used to describe the size-frequency distribution of sediment.

Figure 7- 1: Cumulative Semilogarithmic Size-Frequency Curve

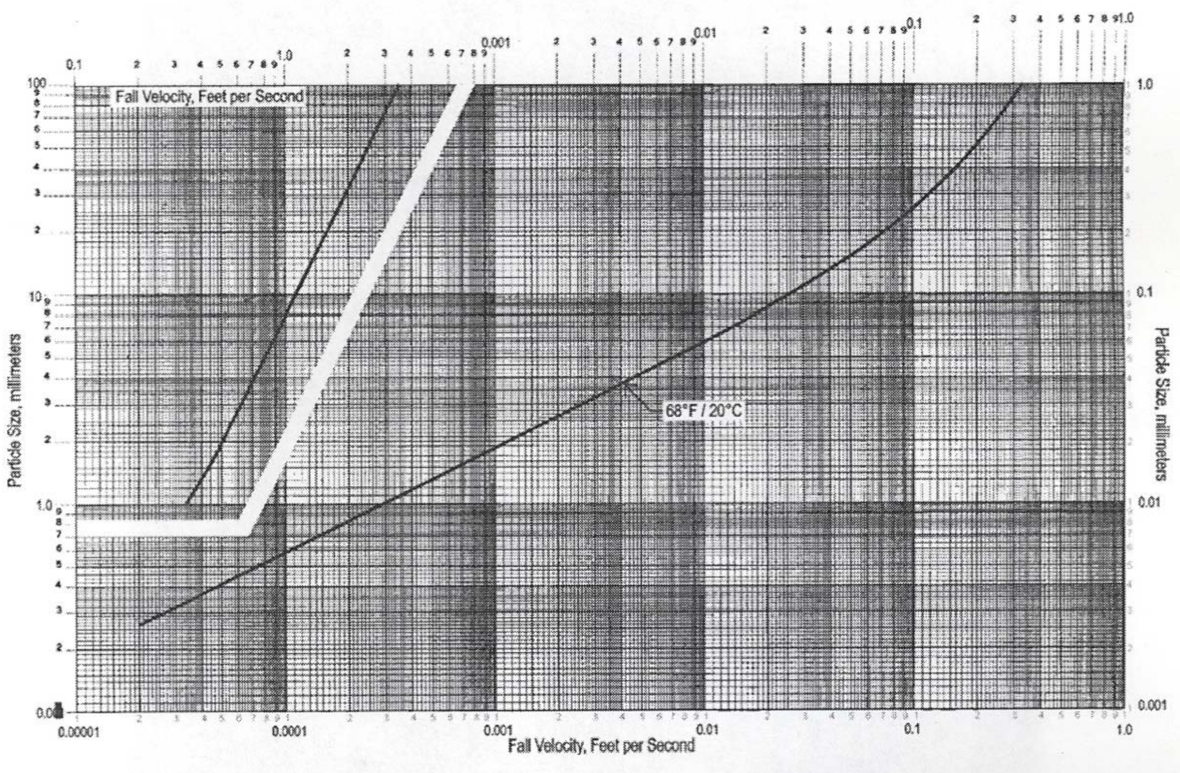


7.2.3.3 Fall Velocity

The fall velocity of sediment is a function of particle size and shape, specific gravity, water temperature, and concentration of sediment in the water. For typical engineering sedimentation studies, the fall velocity can be estimated by Figure 7.2. It is noted that fall velocity is usually expressed for quiescent fluid conditions and there is a tendency for the fall velocity to decrease in turbulent flows.

Figure 7- 2: Sediment Fall Velocity Curve for Typical Sediments in Water at 68°F

(Pemberton and Lara, 1971)



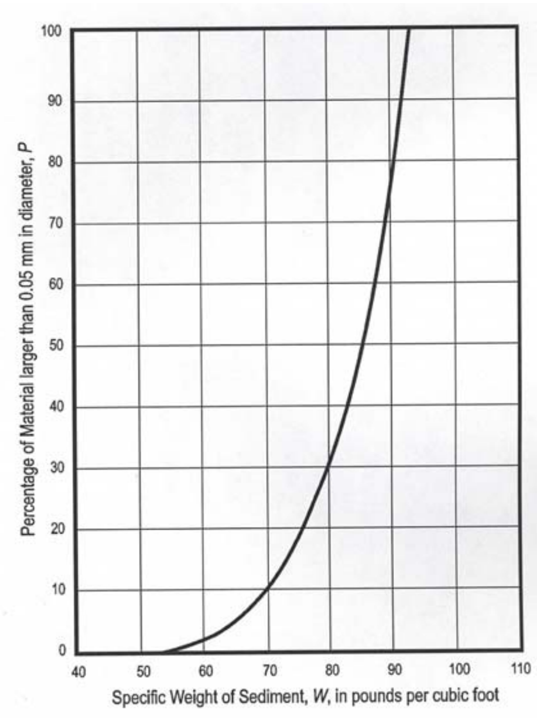
7.2.3.4 Specific Gravity of Sediment Particles

Sediment is rock material that is produced by weathering and abrasion. Less competent material in the rock decomposes more readily into smaller sizes. Coarser sediments decompose more slowly and may be pieces of the parent rock. Sand is most often composed of small quartz particles. Quartz has a specific gravity of 2.65 and that value is often used for sediment. For most applications a specific gravity of 2.65 can be assumed for sediment; however, the specific gravity can be less or much higher for heavy minerals. Specific gravity is important to the sedimentation process and should be measured if an untypical sediment source is suspected.

7.2.3.5 Specific Weight of Sediment Deposits

Specific weight of a sediment deposit is the dry weight of the sediment within a unit volume (including pore space of the sediment mass), in pounds per cubic foot. Specific weight is of particular interest when estimating depletion of storage volume in stormwater storage facilities. Specific weight varies over a large range due to sediment properties and hydraulic factors. The age of the sediment deposit can also be a factor for deposits of fine material, but generally is unaffected by time for deposits of coarse sand and larger particles. A relation for specific weight of sediment deposited to percentage of sand is provided in Figure 7.3.

Figure 7- 3: Relation of Specific Weight of Sediment Deposits to Percentage of Sand
(Modified from Lane and Koelzer, 1953; ASCE, 1975)



7.2.4 Equilibrium Concept

Watercourses tend to adjust their physical characteristics toward a state of dynamic equilibrium such that their ability to transport sediment is in balance with the amount of water and sediment that is delivered to the watercourse. Dynamic equilibrium is achieved when the amount of sediment entering a reach is equal to the amount leaving. The concept of dynamic equilibrium must be understood within the context of long-term trends. That is, any particular discharge event within a watercourse may not result in balanced sediment inflow and outflow; however, if the watercourse is in dynamic equilibrium, that balance is achieved during a reasonable time span of discharge events. Adjustments to the watercourse to achieve dynamic equilibrium are achieved in several ways, including bank erosion or fill, change in bed material size gradation and/or changes in bed slope (aggradation or degradation).This concept is illustrated by Lane's balance (Lane, 1955) as shown in Figure 7.4, and the qualitative equation:

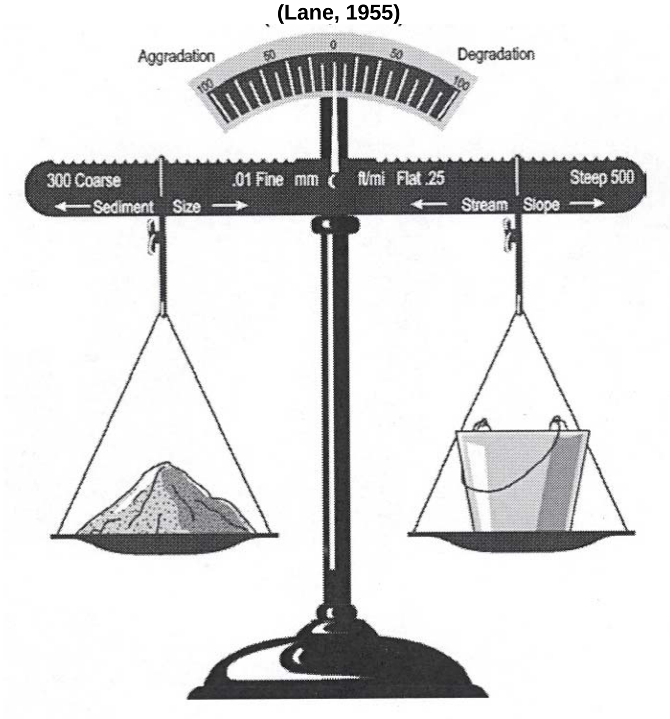
$$Q_w S \propto Q_s d_s \tag{7.3}$$

Where:

Q_w = water discharge
S = longitudinal slope of the watercourse
Q_s = bed material discharge
d_s = characteristic bed material particle size

Alteration of one variable, as illustrated by the "balance" of Figure 7.4, will indicate the effect upon the others. In practice, it is often useful to consider two of the variables to remain constant while considering the quantitative effect of one variable by a change in the fourth variable.

Figure 7- 4: Table Channel Balance
(Lane, 1955)



7.3 ANALYSIS

Table 7.2 presents a checklist of potential problems relating to channel movement/scour and the causative factors that should be examined.

Table 7- 2: Checklist of Potential Problems Relating to Channel Movement and Scour

Potential Problem	Yes	No
Long term degradation or aggradation		
Reservoirs		
Mining		
Urbanization		
Watershed changes		
General scour		
Downstream variable water surface relationship		
Contraction and expansion		
Bed configuration and movement		
Live-bed scour		
Clearwater scour		
Bends		
Natural stream constriction		
Floodplain encroachment		
Berms from sediment deposits		
Island or bar formations		
Debris		
Growth of vegetation in floodplain or channel		
Bed and sediment characteristics		
Armoring		
Lateral migration		

7.3.1 Three Tier Approach for Sediment Transport Analysis

The following table provides a three tiered approach to undertaking sediment transport analysis. The degree of effort increases from top to bottom and it is assumed that the tasks of a preceding tier are undertaken prior to starting the more comprehensive tasks.

The first tier represents a qualitative approach. This is the least sophisticated which requires the lowest effort. The tasks listed herein provide background information for more detailed analysis. Many of these tasks would be undertaken as part of a basic drainage study. The bed and bank material analysis consists of identifying the material present for the project area and the areas immediately upstream and downstream of the site. Planform characteristics relate to the geometry of the watercourse bed, such as presence of dunes or antidunes. Information about land use changes provides insight as to the past and present supply of sediment and changes in runoff characteristics due to increased imperviousness and changes in hydraulic travel time. Review of past and present aerial photographs provides information as to the extent of streambed migration and the stability of the fluvial system. Coupled with information about flood history, this provides the designer with helpful information about watershed responses to flood events. Hydrologic analysis of the onsite and offsite contributing drainage areas provides the designer with the magnitude and frequency of runoff expected. Finally, as explained previously in this chapter, the information gathered from above is used with rudimentary geomorphologic relationships to provide insight as to the expectation of sedimentation or scour. The second tier identifies quantitative, end result methods of analysis consistent with the methods presented in this chapter. The final level of analysis is dynamic modeling of sediment transport that requires extensive knowledge of the sediment transport process. That level is well beyond the scope of this chapter. Furthermore, this list is not all-inclusive. Additional methods may be necessary depending upon the specifics of the project under study. Interested readers should review the references cited at the end of this chapter for further guidance.

Table 7- 3: Task List for Sediment Transport Investigations

Sediment Transport Analysis (Qualitative)
Determination of Planform Characteristics
Lane Relation and other Geomorphic Relationships
Aerial Photograph Interpretations
Bed and Bank Material Analysis (visual inspection)
Land Use Changes
Flood History
Rainfall/Runoff Relationships
Sediment Transport Analysis (Quantitative)
Watershed Sediment Yield
Detailed Bed and Bank Material Analysis
Profile Analysis
Incipient Motion Analysis
Armoring Potential
Sediment Transport Capacity
Equilibrium Slope Analysis
Sediment Continuity Analysis
Quantification of Vertical and Horizontal Channel Response
Bend Scour
Low Flow Channel Incisement
Gravel Mining Impacts
Contraction Scour
Local Abutment Scour
Local Pier Scour
Cumulative Channel Adjustment
Lateral Migration [a]
Sediment Transport Analysis (Sediment Routing in Time & Space)
Data Inventory Modeling
Watershed Sediment Modeling
Instream Mining Response
Single Event Stream Bed Modeling
Long Term Bed Modeling

See - State Standard for Watercourse System Sediment Balance, Guideline 1, Lateral Migration Setback Allowance for Riverine Floodplains in Arizona, SSA 5-96, Arizona Department of Water Resources, September 1996.

7.4 SEDIMENTATION

7.4.1 Sediment Transport

The magnitude of sediment transport is dependent upon the ability of the flowing water to transport incoming sediment and/or to erode the material making up the bed and/or banks of the watercourse. Watercourses composed predominately of sand-sized material will respond to virtually the entire range of flows to which it is subjected. However, watercourses composed of significant quantities of coarser (gravel, cobble and boulder) material will be limited to adjustments only during large flow events.

A basic understanding of sediment transport mechanics is fundamental in qualitative and quantitative sediment transport analyses. Inherent in that understanding are the concepts of incipient motion and armoring. Incipient motion analysis provides a means to estimate the largest size of sediment particle that can be transported during a given flow event. In cases where there is a sufficient quantity of coarse sediment, an armor layer may form that can act as a complete or partial control to sediment transport. The application and limitation of the numerous sediment transport equations must be understood and appreciated when performing sediment transport analyses and quantitative studies.

Shallow flow over roadway initially causes headcutting into road subgrade and pavement

7.4.1.1 Bed Form

Sediment transport is highly dependent upon the resistance to flow, and resistance to flow in an alluvial channel is strongly related to the physical shape of the bed. The physical elements that comprise the shape of the bed are called bed form. For a more thorough discussion of bed form and its impact on flow resistance see Simons and Senturk (1992). Those bed forms in common occurrence in alluvial channels are briefly described:

- Plane bed - A flat or nearly-flat and smooth surface of the bed.
- Ripples - Small bed forms that are typically less than a foot long and less than 1 1/2 inches high. They occur in lower regime flow.
- Bars - Large bed forms that have lengths of the same order as channel width and heights about the same as flow depth. There are several kinds of bars, such as point bars, alternate bars, tributary bars and middle bars.
- Dunes - Bed forms that are larger than ripples and smaller than bars. Size is a function of the geometry of the watercourse. It indicates higher transport rates than ripples.

- Antidunes - Bed forms in upper regime flow that are often in trains. They are often called standing waves. They exhibit surface waves that are in phase with the antidunes.
- Chutes and Pools - Bed forms of large elongated chutes of high slope and high velocity flow separated by low velocity pools. These represent very high sediment transport rates.

Watercourse exhibiting potential for large bed load discharge

Bed form is often associated with regime of flow. (Note: This is a different concept than the regime of Section 7.2.2.). Plane bed, ripples and dunes are typically in lower flow regime where the Froude number is usually less than 0.4. The transition to washed-out dunes and a return to plane bed (with high bed load transport) represent the transition regime where the Froude number is typically between about 0.4 to 0.7. Antidunes with standing waves or with violent breaking waves and chute and pool are in upper flow regime where the Froude number is typically greater than 0.7 (Guy, 1970).

7.4.1.2 Incipient Motion

Incipient motion occurs when the hydrodynamic forces acting on a grain of sediment of given size is equal to the forces resisting movement. Incipient motion is often analyzed using the Shields relation:

$$d_c = \frac{\tau_0}{F_*(\gamma_s - \gamma)} \tag{7.4}$$

Where:

Dc = the sediment diameter at incipient motion in feet,
τ_0 = the bed shear stress in pounds per square foot
γ_s = the sediment specific weight, typically 165 pounds per cubic foot,
γ = the water specific weight, 62.4 pounds per cubic foot
F_* = the dimensionless shear stress, often referred to as the Shields parameter. F_* ranges from 0.03 to 0.06 and a value of 0.047 is often used (AMAFCA, 1994).

The bed shear stress in pounds per square foot, is calculated by

$$\tau_0 = \gamma RS \tag{7.5}$$

Where:

R = hydraulic radius, in feet
S = channel friction slope, in ft/ft.

Incipient analysis, as presented herein, does not cover all aspects of incipient motion. For a discussion of applications, limitations and modifications see AMAFCA (1994), ASCE (1975), Richardson and others (2001), Simons and Senturk (1992), Yang (1973), ADWR (1985), Chang (1988), and Shen (1971, 1972 and 1973).Application of incipient motion analysis may provide information on the magnitude of discharge required to move the particles lining the watercourse bed and/or banks. These analyses are generally most reliable and useful for gravel or cobble bed watercourses. When applied to sand bed systems, incipient motion results usually show that the sediment particles are in motion, even at small discharges.

7.4.1.3 Armoring

Armoring occurs when material finer than the incipient motion size is eroded and transported away leaving a layer of coarser, immobile (for a given discharge) material on the surface. If the watercourse is in a degradational mode, this process can continue over a range of discharge events, each larger event removing the increasing larger particle sizes. Armoring is effective only to a given magnitude of flood event; flows exceeding that magnitude may disrupt the armor layer causing bed scour and degradation.

Armoring analysis normally requires the application of incipient motion analysis and data on bed material size gradation within the anticipated depth of scour. In application, the d95 particle size is considered to be the maximum size for armor formation. Therefore, armoring (for a given discharge) can be expected when the computed incipient motion size is equal to or smaller than the d95 size of the bed material.

The depth of scour (Y_s) necessary to establish an armor layer can be estimated by Pemberton and Lara (1984).

$$Y_s = Y_a\left(\frac{1}{P_c} - 1\right) \tag{7.6}$$

Where:

Y_a = desired thickness of the armor layer (normally assumed to be 2 to 3 times the critical particle size, dc,

P_c = decimal fraction of bed material coarser than the armoring size.

7.4.1.4 Sediment Transport Methods

The planning and design of drainage and flood control facilities often requires the analysis of sediment transport. Often those analyses are performed using sediment transport methods. Those methods may be mathematical or graphical and can be theoretically or empirically based. Often the method is some combination of all of the above. Some of the more popular sediment transport methods are the Einstein bed load function, the Meyer-Peter, Muller equation, the Yang unit stream power concept and the Colby relations. However, there are virtually dozens of sediment transport relations in the literature. A problem for the engineer is to select one or more of these relations for use in solving a particular problem. When selecting a sediment transport method, the data base (sediment size, flow condition, mode of transport process, etc.) used to develop each method must be understood. The selection, however, is not straightforward and often it is not possible to determine which one is best for a particular application. Often the selection process indicates that no one method is best and two or more methods may need to be used and the respective results evaluated. The results by different methods often differ drastically. It is absolutely imperative that the application and limitation of the various methods be understood when using those to estimate sediment transport. The engineer must use experience and judgment in both the selection of the sediment transport method and in the interpretation of the results. See AMAFCA (1994), ASCE (1975), Yang (1973), ADWR (1985), Chang (1988), Richardson and others (2001), Shen (1971, 1972, 1973), Sheppard (1960), and Simons and Senturk (1992) for further discussions of sediment transport methods.

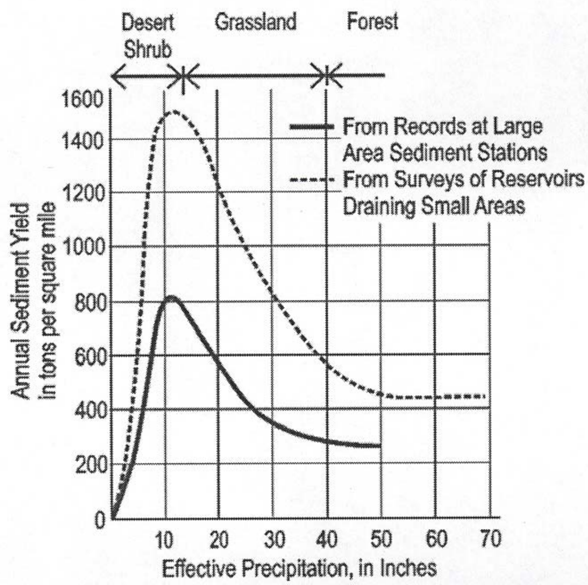
7.4.2 Watershed Sediment Yield

Sediment yield is a measure of the sediment production from a watershed exiting the watershed at some point in the drainage network. It is usually measured in units of weight (tons), volume (acre-feet) or uniformly eroded depth of soil (inches or millimeters). Since sediment yield increases with increasing time duration, the yield is usually expressed in terms of annual average or per specific flood event or flood duration. An aid to analyzing sediment yield data is to convert it to a value per unit of drainage area (acre-feet per square mile per year).Sediment yield is dependent upon the rate of total erosion within the watershed and the efficiency of transport of those eroded sediments through the drainage network. Erosion and transport factors are widely variable, and therefore, measures of sediment yield are broadly generalized.

7.4.2.1 Deposit of Sediment

Sediment yield is highly dependent upon vegetation cover and precipitation. Langbein, and Schumm (1958) illustrates in Figure 7.5 a trend of increasing sediment yield with increasing annual precipitation, until increased precipitation results in improved vegetation cover. Beyond that point, sediment yield then decreases with increasing precipitation. Maximum sediment yield occurs in the 10 to 15 inches of annual precipitation range. Notice in Figure 7.5 that the sediment yield is considerably higher when data for small watersheds is used. Smaller watersheds typically have higher unit sediment yields because of the influence of high intensity rainfalls that can impact the entire watershed.

Figure 7- 5: Sediment Yield, as Affected by Climate
(Langbein and Schumm, 1958)



7.4.2.2 Analytic Methods to Estimate Sediment Yield

Numerous methods are available for estimating sediment yield by analytic methods; see for example, Pacific Southwest Inter-Agency Committee (1974). A commonly used procedure is the Revised Universal Soil Loss Equation (RUSLE) (Soil and Water Conservation Society, 1995, USDA, 1997, and Toy and Osterkamp, 1995). Flaxman (1972 and 1974) provides a procedure more applicable to the Western United States. The use of any of these methods is subjective to the selection of the input parameters. In practice, more than one analytic method may be used and the results compared. The use of empirical data or regional familiarity should be used in accepting results of these analytic methods.

Equations of mean annual soil loss like RUSLE do not account for climate changes that may produce episodic changes in channel processes such as gullies. For example, in southeastern Arizona there is geologically recent headcutting of the San Pedro River and its tributaries. The sediment yield from gullies and channel enlargement is more than 30 times the sediment yield from rill and interrill processes estimated by RUSLE (Toy and Osterkamp, 1995). Renard and Stone (1981) report sediment yield increases of nearly four times that estimated by the universal soil loss equation (USLE) as a result of channel and bank erosion at two small watersheds in the San Pedro Basin. Headcutting and gully erosion, and their influence on sediment yield, is discussed by Leopold and others (1966).

(Recent headcutting is apparent in the Cave Creek basin especially near the main channel of Cave Creek. Channel incision also is apparent in the Indian

Bend Wash basin such as Lost Dog Wash at the southern end of the McDowell Mountains.)

For watersheds larger than a few acres that have defined channels, mean annual soil loss may or may not be a large part of the sediment yield. The proportion of sediment yielded from the soil and from watercourse beds and banks is difficult to estimate.

The above examples serve as a reminder that large amounts of sediment can be derived from the watercourses of small desert watersheds. Large amounts of sediment can be derived from rill development, gully formation and watercourse bed and bank erosion where concentrated runoff from urban development crosses unprotected soil.

7.4.2.3 Sediment Yield Data

Sediment yield data for watersheds in Arizona, New Mexico, and California that may be applicable to conditions in Pinal County are shown in Table 7.4

Table 7- 4: Measured Sediment Yield from Representative Watersheds

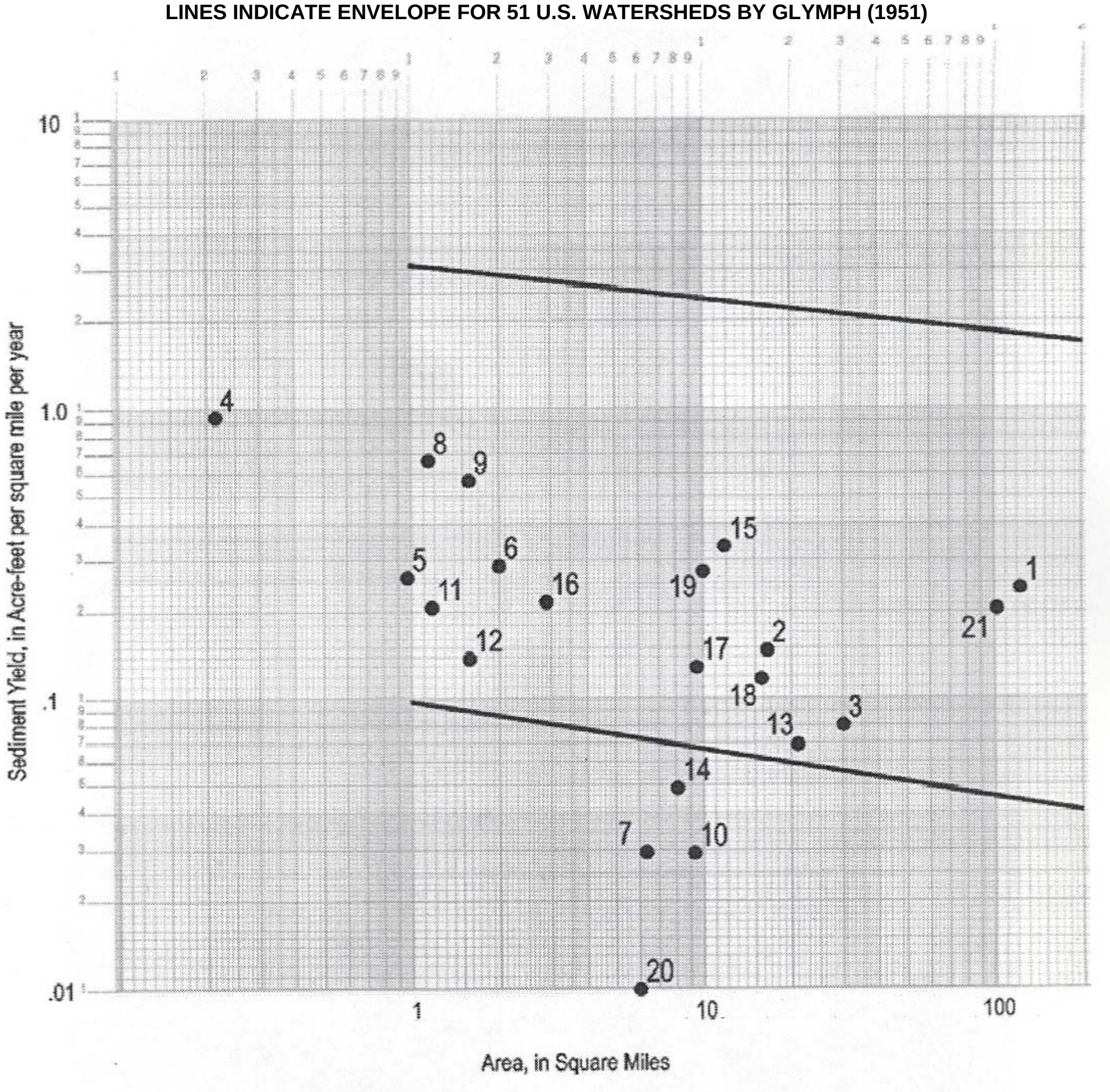
No.	Location	Drainage Area, sq mi	Sediment Yield, ac-ft/sq mi/yr	Reference
1	Cave Creek Dam, AZ	121.00	0.24	A
2	Spook Hill FRS, AZ	16.40	0.15	B
3	Saddleback FRS, AZ	30.00	0.08	B
4	Davis Tank, AZ	0.21	0.96	C
5	Kennedy Tank, AZ	0.97	0.27	C
6	Juniper Tank, AZ	2.00	0.29	C
7	Alhambra Tank, AZ	6.61	0.03	C
8	Black Hills Tank, AZ	1.14	0.68	C
9	Black Hills Tank, AZ	1.56	0.58	D
10	Mesquite Tank, AZ	9.00	0.03	C
11	Tank 76, AZ	1.17	0.21	C
12	Camp Marston, CA	1.59	0.14	B
13	Embudo Arroyo, NM	20.68	0.07	E
14	La Cueva Arroyo, NM	8.00	0.05	E
15	Baca Arroyo, NM	11.55	0.34	E
16	North Pino Arroyo, NM	2.82	0.22	E
17	South Pino, Arroyo, NM	9.33	0.13	E
18	Bear Arroyo, NM	15.50	0.12	E
19	Vinyard Arroyo, NM	0.98	0.28	E
20	Hahn Arroyo, NM	5.80	0.01	E
21	N. Diversion Channel, NM	101.0	10.21	E
		Average =	0.24	
		Median =	0.21	
		AZ Average =	0.32	
		AZ Median =	0.24	

A U.S. Army Corps of Engineers, 1974
B USDA, Natural Resources Conservation Service file data
C Peterson, 1962
D Langbein and others, 1951
E Albuquerque Metropolitan Arroyo Flood Control Authority, 1994

The sediment yield data from Table 7.4 are plotted in Figure 7.6 along with an envelope of sediment yield for 51 watersheds in the United States (Glymph, 1951). It is noted that the U.S. Army Corps of Engineers used a sediment yield of 0.30 acre-feet per square mile per year for the design of Cave Buttes Dam in Maricopa County, Arizona. Although at the time (1970), sediment yield for the immediately upstream Cave Creek Dam was only 0.24 acre-feet per square mile per year. The larger value (0.30) was used for design purposes to account for large sediment inflow during the September 1970 flood that is not reflected in the 0.24 acre-feet per square mile per year measurement.

Figure 7- 6: Regional Sediment Yield as a function of Drainage Area

LINES INDICATE ENVELOPE FOR 51 U.S. WATERSHEDS BY GLYMPH (1951)



The USACE (1974) indicate a range of sediment yield of 0.009 to 1.33 acre-feet per square mile per year for watersheds in Arizona and New Mexico. The U.S. Department of Agriculture (Alonso, 1997), reports sediment yield of 0.12 to 0.4

acre-feet per square mile per year for the Walnut Gulch Experimental Watershed near Tombstone, Arizona.

The wide range of sediment yield is explained by soil conditions, precipitation, and watercourse conditions among other things. For example, the relatively small yield of 0.08 acre-feet per square mile per year from the 30 square mile basin above Saddleback Flood Retarding Structure in Maricopa County, Arizona, is due to the well-developed soil covered with desert pavement. The differences of sediment yield are also related to climate differences. For example, certain watersheds in San Diego County, CA reflect yields of only 0.07 and 0.13 acre-feet per square mile per year due to the low annual precipitation of only 3 inches. Some sites with a large sediment yield such as Davis Tank, AZ are known to have watercourse bed and bank erosion. Lastly, other sites with relatively high yield such as Black Hills Tank, AZ may have experienced a large flood during a short period of data collection.

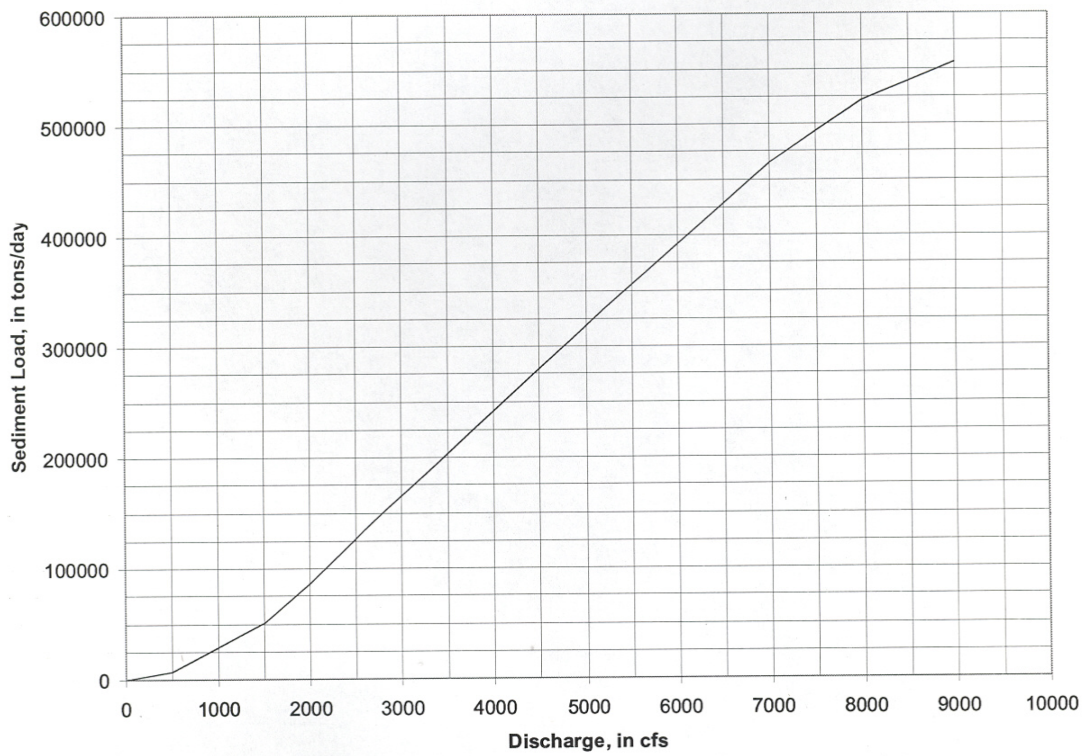
Runoff and sediment yield data were collected at the Black Hills Tank, near Cave Creek, Arizona, from 1945 to 1948 (Langbein and others, 1951, and Peterson, 1962). The precise location of the site is uncertain but it was near the northern end of the McDowell Mountains on a granite pediment at an elevation of about 2,600 feet. Vegetation was mountain-brush type consisting mainly of snakeweed, yucca, creosote bush, and cactus, with small palo verde and mesquite trees along the channels. According to Langbein and others (1951), the approximately 2.5 mile long drainage basin was 1.56 square miles in area, headed at 3,200 feet elevation, and was drained by a network of 0.5 to 2 feet deep watercourses at a slope of about 2 percent. The granitic rock is capped with a thin veneer of coarse residual soil. The watershed sediment yield was 0.9 acre-feet per year or 0.58 acre-feet per square mile per year based on capacity surveys at the beginning and end of the data collection. A field examination of the 1948 flood reportedly showed coarse sediment with uprooted mesquite trees deposited in a fan at the entrance to the tank. There was no spill during the period. According to Peterson (1962) the drainage basin is only 1.14 square miles and the watershed sediment yield is 0.78 acre-feet per year or 0.68 acre-feet per square mile per year. The difference in reported sediment yield for the same watershed is not significant. However, the reported large flood in 1948 is significant because unusually large amounts of sediment were deposited in the tank. The reported average annual sediment yield in Table 7.3 for Black Hills Tank for the 4-year period probably is too high because of the 1948 flood. However, that data does indicate the magnitude of sediment that can be produced from a single intense runoff event

7.4.3 Sediment Discharge

7.4.3.1 Sediment Discharge Rating Curves

For Pinal County, there is a scarcity or complete lack of adequate sediment discharge and corresponding storm runoff data. Therefore, it is often necessary to estimate sediment yield from the watershed and to investigate the sedimentation aspects of the drainage design or study by analytic or indirect empirical methods. Techniques of sediment transport modeling, such as with HEC-6 (USACE, 1991), are often used in such analyses. Sediment inflow relations are generally required for such analyses, and those are often in the form of sediment load rating curves of sediment discharge (tons per day) as a function of water discharge (cfs). Such a rating curve is illustrated in Figure 7.7.

Figure 7- 7: Example of Sediment Discharge Rating Curve



7.4.3.2 Sediment Concentration

Sediment discharge is a function of water discharge as expressed by

$$Q_s = 0.0027 C Q_w \qquad (7.7)$$

Where:

Q_s = sediment discharge in tons per day
Q_w = water discharge in cfs
C = concentration of sediment in mg/l
0.0027 = a unit conversion factor (Porterfield, 1972).

The sediment discharge rating curve should always be inspected by calculating and plotting C as a function of Q_w. The concentration of sediment will not always increase with increasing discharge. Generally the sediment discharge will be the greatest during the rising limb of the flood hydrograph as the watercourse is "flushed" of previous sediment deposits in the main channel and the overbank floodplain from lesser floods. Depending on the magnitude of such deposits and the shape of the flood hydrograph, the concentration of sediment discharge may decrease for water discharge above some level (see Figure 7.8). These phenomena can also be attributed to dilution of the sediment load for large floods. Inspection of the sediment concentration curve should always be performed. Qualitative inspection of the curve will provide some confidence that reasonable sediment discharges are being input to the analytic models and that reasonable results are being produced by the models. It is difficult to generalize these sediment concentration relations, however, concentrations in excess of 100,000 mg/l (ppm) of total load would be very high, but within reason under appropriate conditions. Concurrently, maximum concentrations of less than 10,000 mg/l for "typical" watercourses in Pinal County during major floods would probably be low.

It must be noted however, that many analytic techniques and sediment transport models are for only one component of the sediment load. For example, the sediment transport function may be limited to estimating the bed material load or to only the sand-sized fraction of the bed load. Often the bed material load is only a small fraction of the total sediment load, maybe only 10 percent. Therefore, two points must be carefully understood:

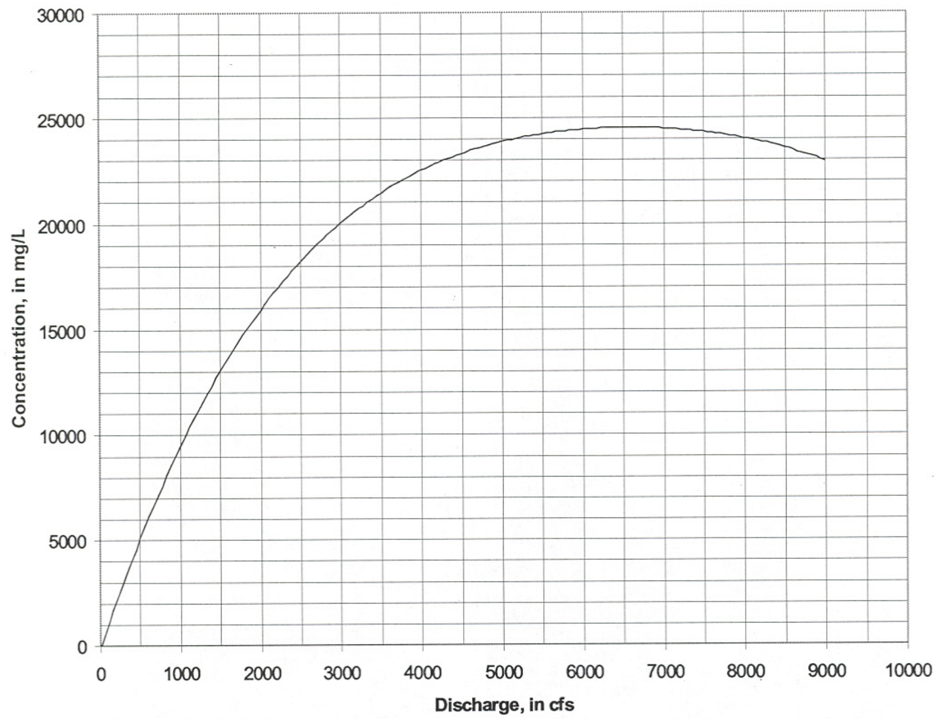
1. The limitations and applications of the sediment transport analysis or models used

2. A judgment based qualitative and quantitative estimation of the general sediment discharge relations.

Regional experience and judgment based on past experience is necessary.

Procedures for measurement of fluvial sediment are provided by Guy and Norman (1970). Methods for laboratory analyses of sediment samples are provided by Guy (1969).

Figure 7- 8: Example of Sediment Concentration Rating Curve



7.4.3.3 Sediment Discharge Characteristics

Bed material sediment discharge is limited by either the sediment transport capacity of the watercourse (transport control), or by the amount of sediment available to the watercourse (supply control). Transport control is dictated by hydraulic conditions in the watercourse. Supply control is dictated by conditions of sediment yield to the watercourse.

The concentration of sediment discharge can be used to classify flows (O'Brien, 1986), as illustrated in Table 7.5.

Table 7- 5: Water and Sediment Flow Classification

Type of Flow	Concentration Range, in mg/l
Water flood	0 - 410,000
Mud flood	410,000 – 650,000
Mudflow	650,000 – 730,000
Landslide	730,000 – 880,000

In general, floods in Pinal County will produce total sediment load concentrations well below 410,000 mg/l; however, steep hill slope processes could produce mud flood, mudflow and, conceivably, landslide conditions.

7.4.3.4 Sediment Bulking

High sediment concentrations can increase the total volume of the water and sediment discharge. This is referred to as bulking, and the total volume of the water-sediment mixture (V_m) is estimated by:

$$V_m = B_f V_w \tag{7.8}$$

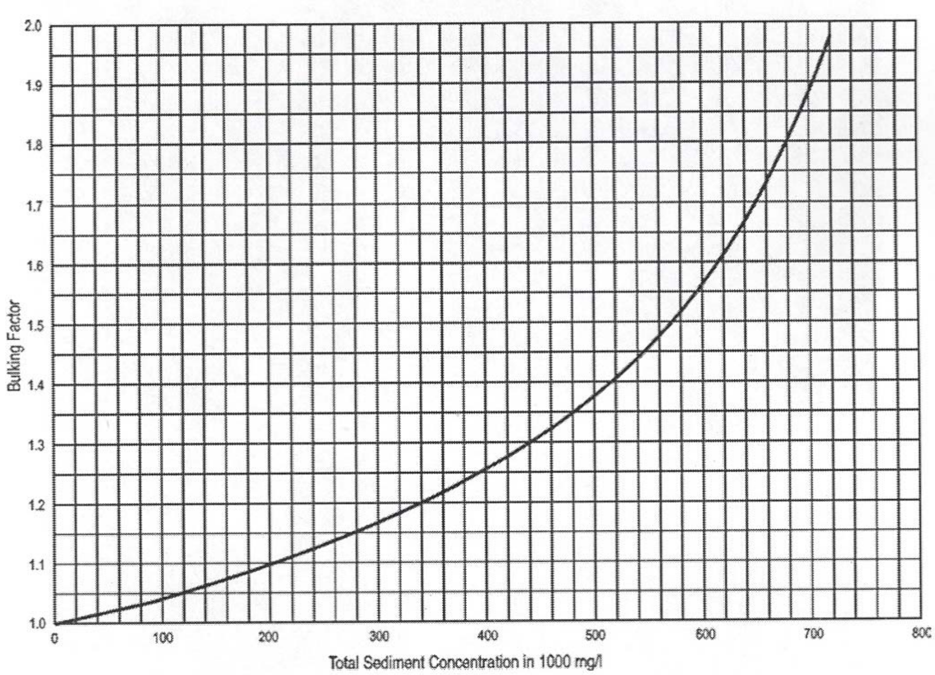
Where:

V_w = clearwater volume
B_f = bulking factor (Albuquerque Metropolitan Arroyo Flood Control Authority, 1994).

The relation between total sediment concentration and the bulking factor is given by Figure 7.9. For example, if the sediment load concentration is 200,000 mg/l, the total water-sediment volume discharge is increased by a factor of about 1.10. For high sediment concentration discharges, design capacities must accommodate the bulked volumetric discharge.

Figure 7- 9: RELATIONSHIP BETWEEN TOTAL SEDIMENT CONCENTRATION AND BULKING FACTOR

(Albuquerque Metropolitan Arroyo Flood Control Authority, 1994)



7.5 EROSION

7.5.1 Bank Erosion

Bank erosion and widening of watercourses occurs from two primary mechanisms; grain-by-grain erosion and bank failure. Commonly, grain-by-grain erosion and bank failure act together; fluvial erosion scours the toe of the bank, and failure follows. Removal of the failed bank material occurs through fluvial erosion and the process is repeated.

The bank erosion process can result from watercourse incision (degradation), flow around bends, flow deflection due to local deposition or obstructions, aggradation, or a combination of the above. For the case of an incising watercourse, exceeding the maximum stable bank height will lead to mass failure and bank line retreat. Flow around a bend can cause erosion at the toe of the bank and subsequent bank failure due to increased shear stress on the outside of the bend. Both local deposition and aggradation over a longer reach create midchannel bars that can deflect flow into the bank with essentially the same result as flow on the outside of a bend.

The specific failure mechanisms at a given location are related to the characteristics of the bank material. In general, bank material can be broadly classified as cohesive, noncohesive, and composite.

Noncohesive bank material tends to be removed grain by grain from the bank. The rate of particle removal and the rate of bank erosion are affected by factors such as particle size, bank slope, the direction and magnitude of the velocity adjacent to the bank, turbulent velocity fluctuations, the magnitude and fluctuations in the shear stress exerted on the banks, seepage forces, piping, and wave forces.

Cohesive material is more resistant to surface erosion and has low permeability, which may reduce the effects of seepage and subsurface flow on the stability of the banks. However, the lower permeability may increase pore pressure due to saturation resulting in bank collapse when water levels drop in the watercourse. When undercut and/or saturated, such banks are more likely to fail due to mass wasting processes. Composite or stratified banks consist of layers of materials of varying size, permeability, and cohesion. The layers of noncohesive material are subject to erosion, but may be partly protected by adjacent layers of cohesive material. This type of bank is vulnerable to erosion and sliding as a consequence of subsurface flows and interlayer piping.

Grain-by-grain erosion can be a significant process in areas of concentrated flow and high shear stress, that is, on the outside of bends. However, studies of bank erosion processes indicate that mass failure and subsequent fluvial transport of the failed material is the primary mechanism by which the lateral adjustments occur.

7.5.2 Lateral Migration

Estimating the rate and magnitude of lateral migration depends on the sediment balance in the reach being analyzed. If analysis indicates that the reach is degradational (that is, bed material transport capacity exceeds supply) either on an average annual basis or in response to a flood, lateral migration will be primarily the result of failure due to undercutting at the toe of the bank. When the bed material transport capacity and the supply are approximately equal, no net erosion or deposition of sediment is expected within the reach. For this case, lateral migration is the result of localized fluvial entrainment of the bank material as the watercourse seeks equilibrium. The preferred method for estimating the rate and magnitude of migration involves the use of historical data and empirically based migration rates. When historic data are not available or when conditions are significantly different than empirically based migration rates, lateral migration can be estimated using geomorphic techniques. The volume of material fluvially entrained from a failed bank (due to undercutting of the toe) within a given bend is approximately proportional to the transport capacity of the watercourse based on the sharpness of the bend.

For degradation reaches, bank failure occurs due to a combination of undercutting of the toe and/or exceeding the stable bank height. The process leading to lateral migration of the bank is, therefore, related to changes in bank height and volume of material moved to create a given lateral migration distance.

When the reach is strongly aggradational and the overbank area is relatively flat, the channel alignment and extent of flooding is essentially a random phenomenon (that is, alluvial fan flooding) and prediction of the erosion boundaries by analytical means is limited. For this case, however, the flood limits are often wider than the erosion limits.

For more information on channel stability, lateral migration and methods to estimate limits erosion and lateral migration see Hedman and Osterkamp (1982), Lagasse and Schall (1988), Leopold and Maddock (1953), Leopold and Wolman (1957), Leopold and others (1964), MacBroom (1981), Rosgen (1996), Schumm (1961, 1971 and 1977), Hjalmarson (1998), and Thorn (1998).

7.5.3 Scour

Scour, for the intent of this discussion, is the lowering of the bed elevation of a watercourse, either locally or over some defined reach length of watercourse, due to the hydraulics of flowing water. Scour is estimated as the sum of independent scour components that are due to factors along a defined reach of a watercourse plus scour at a specific location in a watercourse.

7.5.3.1 Purpose of Estimates

Scour estimates are often needed for the following drainage and flood related purposes:

- Estimating the response of a watercourse due to altered management in the watershed. For example, scour in a natural watercourse may need to be evaluated due to urbanization that would alter the natural flood magnitude-frequency relations.
- Estimating the response of a watercourse due to alterations of the hydraulic conditions in the watercourse. Examples in this regard include floodplain encroachment, flood control modifications such as bank protection, and instream mining of sand and gravel.
- Estimating depth of toe down for structural bank lining.
- Estimating depth of scour immediately at or downstream of hydraulic structures.
- Estimating potential scour depth for buried utility crossings of watercourses.

7.5.3.2 Applications and Limitations

The estimation of scour is an engineering application that requires both specific expertise and experience. Every application of scour technology is unique because of the wide variability of hydrologic, hydraulic and geologic/geomorphic factors. It is not possible to compile a comprehensive methodology in a drainage design manual that would be adequate to address all aspects of scour estimation. In addition, the knowledge of erosion and sedimentation is continually expanding because of the need to provide better technology in this field of engineering. Often, newer methodologies are presented in the engineering literature that should be considered and used, if appropriate. Therefore, the following are general guidelines for estimating scour along with currently used references that are considered applicable in Pinal County.

7.5.3.3 Types of Scour

Total Scour

Total scour, for a given application, should consider the following components of scour:

1. Long-term degradation of the bed of the watercourse.
2. General scour through a specific reach of the watercourse.
3. Local scour.
4. Scour induced due to a bend in the watercourse.
5. Scour associated with bedform movement through the watercourse.
6. Scour due to low flow incensement.

Total scour (Z_t) is the sum of each of these individual components (Z_i) of scour. Total scour can be expressed as:

$$Z_t = FS(Z_{long-term} + Z_{general} + Z_{local} + Z_{bend} + Z_{bedform} + Z_{low-flow}) \quad (7.9)$$

A multiplying factor (FS) is used depending upon the purposes of the total scour estimation. For example, an FS equal to 1.0 may be appropriate when estimating total scour due to altered conditions in a watershed. However, in that case it would be advisable to estimate maximum and minimums of each individual component of scour and to estimate the range of total scour that can be expected. An FS of 1.3 is often used for the design of toe down for bank protection. The use of higher FS, such as 1.5, may be justified where underestimation of scour would cause catastrophic failure that may result in loss of life or unacceptable economic consequences.

The following is a discussion of each component of scour that should normally be considered when estimating total scour.

Long-Term Degradation

Long-term degradation can be estimated by the following methods:

- A trend analysis of historic bed elevation data.
- Simulation by use of sediment transport modeling such as HEC-6 (USACE, 1991).
- Application of equilibrium slope analyses.

A trend analysis of historic bed elevation data is limited by the availability of adequate, long-term data for the watercourse. Therefore, such an analysis may be possible only for some of the major watercourses in Pinal County. In addition, factors such as instream gravel mining and channelization of the watercourse may complicate such historic analyses.

Simulation modeling may provide useful results; however, that method is dependent upon appropriate hydraulic data for the watercourse (hydraulic geometry and sediment characteristics). Furthermore, the results are highly sensitive to hydrologic input (flood magnitude-frequency relations, flow duration, shape of hydrograph, etc.). Simulation modeling may only be appropriate for regional studies of major watercourses, especially those for which structural flood control alternatives are being considered.

Equilibrium slope is a method that can often be applied to estimate long-term degradation without extensive data or modeling effort. The application of this method does require the identification of a downstream bed elevation control (pivot point) at which the bed elevation is not expected to change. Such a control can be bedrock, a reach of armored channel bed, or a constructed facility such as a diversion dam, roadway crossing, and so forth.

Long-term degradation using equilibrium slope analysis is estimated by

$$Z_{long-term} = L_w \Delta S \tag{7.10}$$

Where:

$Z_{long-term}$ = the bed elevation change, in feet, at a distance,
L_w, = upstream of the pivot point, in feet
ΔS = decrease in bed slope, in ft/ft from the existing slope.

Equilibrium slope analysis resulting in an increase in bed slope upstream from the pivot point would indicate an aggradational zone rather than long-term degradation.

Several methods are recommended by Pemberton and Lara (1984) for performing equilibrium slope analyses; the Schoklitsch bedload equation (Shulits, 1935), the Meyer-Peter, and Muller (1948) bed load equation, the Shields (1936) diagram, and Lane's (1952) relation for critical tractive force. The limitations and assumptions of each method should be carefully evaluated when making the selection of a preferred method. Often, more than one method can be used and the results compared. Corroborating results by two or more methods would increase reliance on those results. However, there often is considerable deviation in results by the various methods. In which case, independent data, regional experience and/or engineering judgment must be used in selecting the equilibrium slope.

General Scour

General scour is that component of total scour that would occur during the passage of a design flood. This type of scour involves the removal of material from the bed and banks across all or most of the width of a channel. The scour is caused by increased velocities and shear stresses dictated by the local area geometry (such as at constrictions) and water surface controls. For major watercourses, general scour would often be estimated by a sediment transport model study, such as the use of HEC-6 (USACE, 1991). General scour in minor watercourses can be estimated by the following equation (Zeller, 1981):

$$Z_{general} = Y_{\max}\left[\frac{0.0685 v^{0.8}}{Y_h^{\ 0.4} S_e^{\ 0.3}} - 1\right] \tag{7.11}$$

Where:

$Z_{general}$ is the general scour depth, in feet
Y_{max} is maximum depth of flow, in feet,
Y_h is the hydraulic depth, in feet,

v is the average velocity of flow, in ft/sec,
S_e is the energy slope (or bed slope if uniform flow is assumed), in ft/ft.

The reference by Zeller (1981) should be consulted prior to applying this equation. If Equation 7.11 yields negative results, a value of zero is to be used for general scour.

Local Scour

Local scour is that component of total scour that is caused by flow irregularities. If the transport rate of sediment away from the local region is greater than the transport rate into the region, a scour hole develops. As the depth of scour is increased, the strength of the vortex or vortices is reduced, the transport rate is reduced and equilibrium is reestablished and scouring ceases. Flow irregularities can occur in natural watercourses due to bends or restrictions along the banks. Flow irregularities also occur due to constructed facilities such as bank lining, bank protection works (such as groins), hydraulic structures across the watercourse (such as diversion dams or grade control structures), and structures in the watercourse (such as bridges or culverts). Bridge scour, including the local component of bridge scour, is discussed in Section 7.4.3.5

Generally, local scour depths are much larger than long-term degradation or general scour. But, if there are major changes in watercourse conditions, such as a water storage facility built upstream or downstream or severe straightening of the watercourse, long term bed elevation changes can be the larger element in the total scour.

Five methods for estimating local scour due to natural restrictions and bends, or bank lining are presented by Pemberton and Lara (1984). The USBR Method I is for wide, sand bed watercourses with d_{50} ranging from 0.5 to 0.7 mm, and slopes from 0.004 to 0.008 ft/ft. That method probably has limited application in Pinal County. USBR Method II is recommended for subcritical flow and includes consideration of watercourse curvature. The Lacey (1930) equation is for subcritical flow, includes consideration of watercourse curvature, and requires the use of bed material size and a “silt factor.” The Blench (1969) equation is a function of unit discharge (q in cfs/foot width), Blench’s “zero bed factor” and a factor for watercourse curvature. The Neill (1973) equation is based on flow depth and velocity and the estimation of “competent velocity.” These equations can be used to estimate the local scour due to bank lining or similar applications. The report by Pemberton and Lara (1984) or the individual references should be consulted prior to application of any method. Often, more than one method can be applied and the results compared. Engineering judgment and experience are needed when selecting the value for local scour.

Local scour downstream of a hydraulic structure can be estimated by empirical equations. Flow over the structure can be either submerged or free falling depending on tailwater conditions. For free falling conditions, three local scour equations are available. The Schoklitsch (1932) equation requires hydraulic parameters including the effective drop height and the bed material particle size. The Veronese (1937) equation requires hydraulic parameters including effective drop height, but is independent of bed material grain size and may overestimate local scour for some watercourses in Pinal County. The Zimmerman and Maniak (1967) equation is a function of the d85 bed material particle size, but is independent of many hydraulic parameters and does not consider the drop height. Therefore, that equation should only be used for relatively low (possibly not greater than half of the approach flow depth) drop heights. The Pemberton and Lara (1984) or the original references should be consulted when selecting or applying any of these equations.

For a submerged structure, the local scour depth can be estimated by the Simons, Li & Associates (1986) equation. The equation is a function of drop height and other hydraulic parameters, but is independent of bed material grain size. It may overestimate scour depth for coarse bed material watercourses. That reference should be consulted when using that equation.

Bend Scour

Bend scour may need to be estimated if not included as a component of local scour (see above). For sand bed watercourses, Zeller (1981) presents a bend scour equation. That reference should be consulted in its use and application.

Bedform Trough Depth

Bedforms develop in alluvial channels in response to the hydraulics of the flowing water and they are part of the mechanics of sediment transport. Bedforms are of various configurations and typically they consist of alternating “mounds” and “troughs,” and being mobile, they move longitudinally along the bed of the watercourse. A bedform trough is a component of total scour and should be accounted for under appropriate conditions. The component of scour that is associated with bedforms is equal to one-half of the bedform amplitude (vertical distance from top of mound to bottom of trough) as shown in the following equation.

$$Z_{bedform} = 0.5 d_h \tag{7.12}$$

Bedform trough depth should be estimated for dunes that occur during lower regime flow, and antidunes that occur during upper regime flow. Simons and Senturk (1992) provide dune height equations. Dune height is estimated by:

$$d_h = 0.066 Y_h^{1.21} \tag{7.13}$$

Where:

d_h = dune height, in feet, and
Y_h = hydraulic depth of flow, in feet.

Antidune height is estimated by:

$$d_h = 0.28\pi Y_h F_r^{\ 2} \tag{7.14}$$

Where:

d_h = antidune height, in feet,
Y_h = hydraulic depth of flow, in feet,
F_r = Froude Number.

Dunes form during lower regime flow, typically at F_r less than about 0.7, and antidunes form during the upper regime flow and may form during the transition from lower to upper regime flows. Therefore, antidunes can be expected for F_r greater than about 0.7. Antidune height will usually be greater than dune height. In the transition region, about 0.7 to 1.0 F_r, the larger of either dune or antidune height should be used.

Low-Flow Incisement

The normal irregularities in the bed of a watercourse (both natural and man-made) result in a low-flow channel. That channel is formed by the predominance of a low-flow condition or due too low-flows that persist after a flood. The magnitude of low-flow incisement may best be estimated by representative field assessment. In the absence of field data, or for planning and design purposes, low-flow incisement should be estimated as no less than 1 foot and possibly in excess of 2 feet. A lower value can be used for small and minor watercourses and a higher value should be used for regional watercourses.

7.5.3.4 Armoring

Armoring is the process in an alluvial watercourse wherein sediment transport removes bed material smaller than a certain size thus leaving a bed that is armored by the larger bed particle material. All alluvial channels experience the mechanics of armoring through the selective transport of finer bed material and leaving the coarser bed material. However, watercourses that continually receive the inflow of bed material load in excess of transport capacity or those watercourses for which the bed material does not contain adequate quantities of the larger, armoring-size bed material, will not experience armoring. Also, armoring is flood magnitude dependent; that is, an armoring layer can develop over time due to a sequence of flood events, but a flood event sufficiently larger than those that formed the armor layer can penetrate the armor layer resulting in additional scour depth.

Armoring can be a limiting agent to scour, and, in fact, the placement of riprap as a watercourse liner or around hydraulic structures is an “engineered” armoring.

Therefore, when considering scour, particularly long-term and general scour, the potential for armoring should be considered.

Several methods are available for evaluating the potential for armoring. The incipient motion method (see Section 7.3.1.2) is commonly used and easily applied. Other methods include use of the Meyer-Peter, Muller equation (Sheppard, 1960), the competent bottom velocity method (Mavis and Laushey, 1948), Lane's tractive force method (Lane, 1952), and Yang's incipient motion relation (Yang, 1973). The user should consult those references when making application of those methods to evaluate armor potential.

7.5.3.5 Bridge Scour

Total Scour at Bridges

Scour at bridges must consider all reasonable components of scour that can apply to detrimentally impact a bridge pier or abutment. The total scour (Z_t) at a bridge is typically expressed as:

$$Z_t = FS\,(Z_{\text{long-term}} + Z_{\text{local}} + Z_{\text{contraction}}) \tag{7.15}$$

Where:

FS is a factor of safety which is set at 1.0 for most conditions, but under certain conditions of hazard, including potential economic loss or uncertainty in analyses, could be set higher than 1.0.

The component of long-term scour ($Z_{\text{long-term}}$) can be estimated by procedures discussed in Section 7.4.3.3. The potential for armoring (Section 7.4.3.3) may be considered, but should be used cautiously to limit scour depth.

The procedure in Evaluating Scour at Bridges, HEC-18 (USDOT, 2001b) should be consulted when estimating scour at bridges. Usually the largest component of scour is from local scour at the pier or abutment. Certain scour equations include the angle of attack of the flow, and therefore, bend scour is not normally added because it can be accounted for in the local scour.

Contraction scour occurs when the flow area of the watercourse is reduced because of natural conditions or because of the bridge approaches encroaching into the watercourse. Two equations are provided in HEC-18 (USDOT, 2001b) for contraction scour. One is for live bed conditions; that is, when there is bed material transport from upstream of the bridge. For that condition, a modified version of Laursen's live-bed contraction scour equation (Laursen, 1960) is used. The second is for clear water conditions; that is, when there is little or no sediment transport from upstream of the bridge. For that condition, Laursen's clearwater contraction scour equation (Laursen, 1963) is used. The HEC-18 publication (USDOT, 2001b) should be consulted when estimating contraction scour.

Pier Scour

The commonly used pier scour equations are the Colorado State University equation (Richardson and others, 2001) and Froehlich (1988). Both of those equations are considered in the HEC-RAS program for bridge pier scour (USACE 2001 and 2001b); however, only the Colorado State University equation is recommended in HEC-18 (USDOT, 2001b). The Froehlich equation has been shown to compare well with observed data. Those references should be consulted when estimating pier scour.

Abutment Scour

The commonly used abutment scour equations are the HIRE equation (Richardson and others, 2001) and Froehlich (1989). Those equations are provided both in HEC-18 (USDOT, 2001b) and the HEC-RAS program (USACE 2001a and 2001b). Those references should be consulted when estimating abutment scour.

Watercourse Stability at Highways

The stability of the watercourse at and near highway structures should be considered if channel instability is suspected. Procedures to investigate watercourse stability are provided in HEC-20 (USDOT 2001c).

Bridge Scour Countermeasures

Procedures to provide bridge scour countermeasures are provided in USDOT (2001a).

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VOLUME 2
DESIGN METHODOLOGY AND PROCREDURES

Chapter 8: Hydraulic Structures

8.1 INTRODUCTION

The following sections describe varying types of structures that at used in drainage design. The sections are grouped by type of drainage structure and the procedure and application for structure type design are listed under the structure type.

8.2 CONCEPTS

Hydraulic structures are used in stormdrainage works to control water flow characteristics such as velocity, direction and depth. Structures may also be used to control the elevation and slope of a channel bed, as well as the general configuration, stability and maintainability of the waterway. The use of hydraulic structures can increase the capital cost of drainage facilities while lowering operation and maintenance costs. The use of hydraulic structures should be limited by careful and thorough hydraulic engineering practices to locations and functions justified by prudent planning and design. On the other hand, use of hydraulic structures can reduce initial and future maintenance costs by changing the characteristics of the flow to fit the project needs, and by reducing the size and cost of related facilities. Hydraulic structures include channel drop structures, spillways, grade control structures, energy dissipators, bridges, transitions, chutes, bends and many other specific drainage works. Depending on the function to be served, the shape, size and other features of hydraulic structures can vary widely from project to project. Hydraulic design procedures (including model testing in some cases) that examine the structure and related drainage facilities are a key part of the final design for all structures. This chapter is oriented toward control structures for drainage channels, outlets for stormdrains and culverts, and spillways for nonjurisdictional dams. Design guidelines for spillways for jurisdictional dams or other specialized conveyance measures are beyond the scope of this manual. The design professional is referred to the references cited at the end of this chapter.

8.2.1 Channel Drop Structures

Drop structures are used to reduce the effective slope of a natural or artificial channel. Typically, a drop structure extends across the entire width of the channel and provides grade control for a full range of flows. Check structures are similar in concept, but their objective is to stabilize and control the channel bed or low flow zone. During a major flood, portions of the flow circumvent the structure, but erosion is maintained at an acceptable level. Overall stability is maintained by control of the low flow area, which would otherwise degrade downward. A series of check structures can be an economical interim grade control measure for natural channels in urbanizing areas or for artificial channels where funding is inadequate for construction of drop structures.

8.2.2 Conduit Outlet Structures

Energy dissipation structures are necessary at the outlets of culverts or stormdrains to reduce flow velocity and to provide a transition whereby the concentrated, high velocity flow exiting the conduit is changed to a wider, shallower and non-erosive flow. Outlet structures may be preformed rock riprap stilling basins or reinforced concrete structures such as impact basins.

Spillways

Spillways are conveyance features that permit outflow from stormwater basins. Engineering nomenclature divides these into principal spillways and emergency spillways. The principal spillway for a dam is that hydraulic structure that has been designed to pass the more frequent flow events while the hydraulic capacity of the emergency spillway is held in reserve for the rare flow events. Principal spillways are associated with water storage impoundments (i.e. those with a permanent pool) and stormwater detention basins (wet or dry). Emergency spillways, in one form or another, are provided at these facilities as well as stormwater retention basins. An emergency spillway is a flow conveyance feature designed to safely pass flows in excess of the facility design discharge in a manner that does not threaten the integrity of the principal spillway, facility embankment, or surrounding infrastructure. It also serves to pass flows normally conveyed by the principal spillway under circumstances when the principal spillway becomes plugged. This chapter presents the hydraulic equations used to determine hydraulic capacity for simple spillways.

8.2.3 Special Channel Structures

Bridges, spur dikes, channel transitions, bifurcations, constrictions and bends, and structures for lined channels and for long conduits are examples of hydraulic structures used for special applications. Access ramps, while not a hydraulic structure, are necessary components of a channel to facilitate maintenance.

8.2.3.1 Bridges and Related Structures

Bridges have the potential advantage of crossing a waterway without disturbing the flow. However, for overall economic and structural reasons, encroachments and piers in the waterway are a practical reality. A bridge structure can cause significant hydraulic effects, such as an increase in the water surface elevation, and channel scour. These conditions must be analyzed and measures must be designed for mitigation of negative impacts. Spur dikes, levees, drop or check structures, and pier and abutment protection are types of structures designed to control hydraulic effects at bridge crossings.

8.2.3.2 Channel Transitions

Channel transitions are typically used to moderately vary the cross sectional geometry to allow the waterway to fit within a more confined right-of-way. A channel transition can also be used to purposely accelerate the flow to be carried by a specialized high velocity conveyance structure.

An expansion structure is usually required at the downstream end of a constricted channel reach or structure to provide a safe, non-eroding transition to the unconstricted channel. Potential conditions for creation of a hydraulic jump must be examined and provisions made for control of a jump and associated turbulent flow conditions.

Bifurcations are structures that permit flow to be diverted within a channel. Similarly, side channel spillways also permit the diversion of flow. Finally, channel junctions pose interesting design considerations, especially under supercritical flow conditions.

8.2.3.3 Structures for Lined Channels and Long Conduits

Acceleration chutes can be used to maximize the use of limited downstream right-of-way, and to reduce downstream channel and pipe costs. However, chutes should only be used where good hydraulic and environmental design concepts permit the use of high velocity flow. In general, high velocity flow is not permitted in urban areas and applications in other areas will require careful scrutiny. Bends in lined channels and closed conduits require analysis to determine if supercritical flow occurs, or if special structural and other design considerations are needed.

8.2.4 Trashracks and Access Barriers

Trashracks serve two purposes when utilized in conjunction with stormdrains, culverts, and detention basin outlets. First, trashracks prevent entrapment of persons or animals inadvertently swept into flood waters. Secondly, these structures prevent debris from becoming lodged in the downstream conduit. The analysis and design considerations vary depending upon the flow characteristics.

Trashracks also serve as access barriers placed at the downstream end of stormdrains, culverts, and detention basin outlets to prevent the public from entering the conduit.

8.2.5 Access Ramps

Access ramps are required to facilitate maintenance for all channels. Access ramps for maintenance are recommended at all street crossings on both sides of the street.

8.2.6 Factors of Safety

Specific calculations to determine foundation stability and factors of safety against sliding, uplift, and overturning for a hydraulic structure are necessary in the design of safe structures. The factor of safety derived for a particular case depends, to a large degree, on the risk and consequence of failure. Therefore, the selected factor of safety must be appropriate for each structure being designed.

The factors of safety for sliding, uplift, and overturning all may be different for a particular structure. A general range of 1.5 to 2.0 for these factors is recommended for many types of structures subjected to a variety of loading conditions (see: Design Manual, Foundations and Earth Structures (U.S. Navy, 1982); Design of Small Dams (USBR, 1987); Design of Gravity Dams (USBR, 1976); and Drainage of Roadside Channels with Flexible Linings (USDOT, 1988)).

8.3 CHANNEL DROP STRUCTURES

The term drop structure is broadly defined. Included are structures built to restore previously damaged channels, those constructed during new urban development to prevent accelerated erosion caused by increased runoff, and applications in which other specialized hydraulic conditions are created in the flow channel.

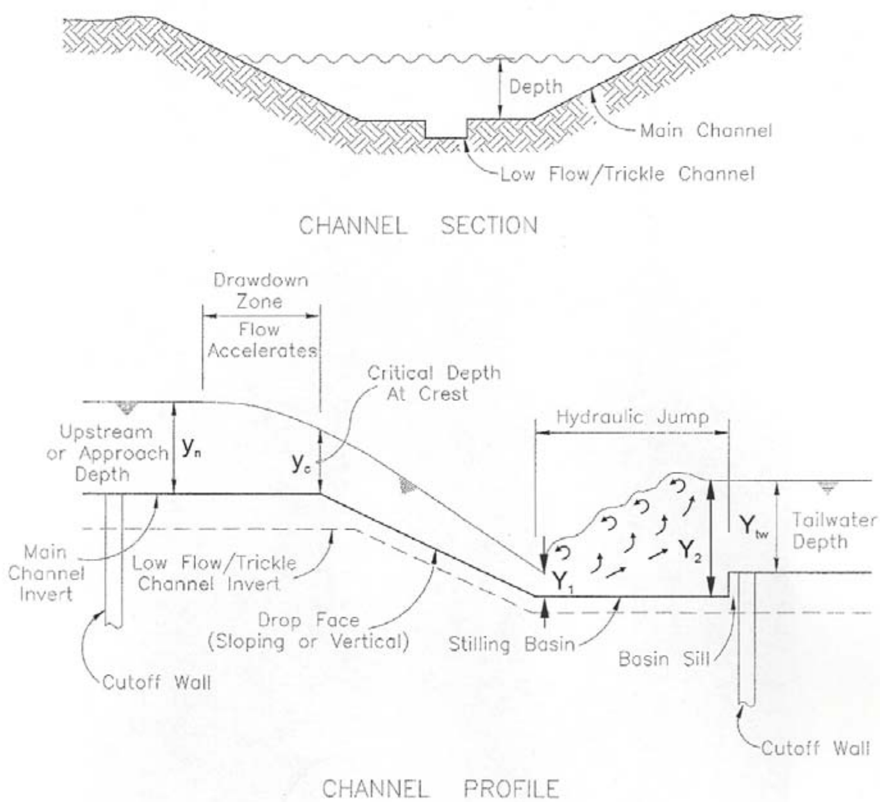
The focus of this design guideline is on drop structures with design flows up to 10,000 cfs. Flows less than 500 cfs are in the usual range for grade control structures.

8.3.1 Drop Structure Components

Figure 8.1 shows a typical channel drop structure with its various components. Once a particular structure type is selected for design, analyses are conducted to determine the optimal sizing or extent of the various components.

Figure 8- 1: Typical Drop Structure Components

(Adapted from McLaughlin Water Engineers, Ltd. 1986)



8.3.2 Design

8.3.2.1 Design Considerations

In addition to hydraulic performance (discussed in Section 8.3.1), a number of other considerations affect the selection of an appropriate drop structure for a particular application.

Soil and Foundation Condition

Geotechnical investigations should be completed to identify the characteristics of the onsite soils. Silty and sandy soils require detailed analyses for seepage control. Expansive soils require special design techniques to minimize differential movement.

Structure design for foundation, walls and slabs must consider soil and hydrostatic pressures, seepage and potential scour.

Construction Concerns

The selection of a drop and its foundation may also be tempered by construction difficulty, access, material availability, etc. Quality control through conscientious inspection is an important consideration.

Maintenance Concerns

Issues to be considered in the design include, ease of access to the crest and stilling basin areas, vandal resistance, eliminate trapped (ponded) water, sediment accumulation, and landscaped or grassed slopes that are easily maintained.

Sociological Considerations

These include public acceptability issues such as safety, visual appearance, mosquito breeding in ponded water, etc.

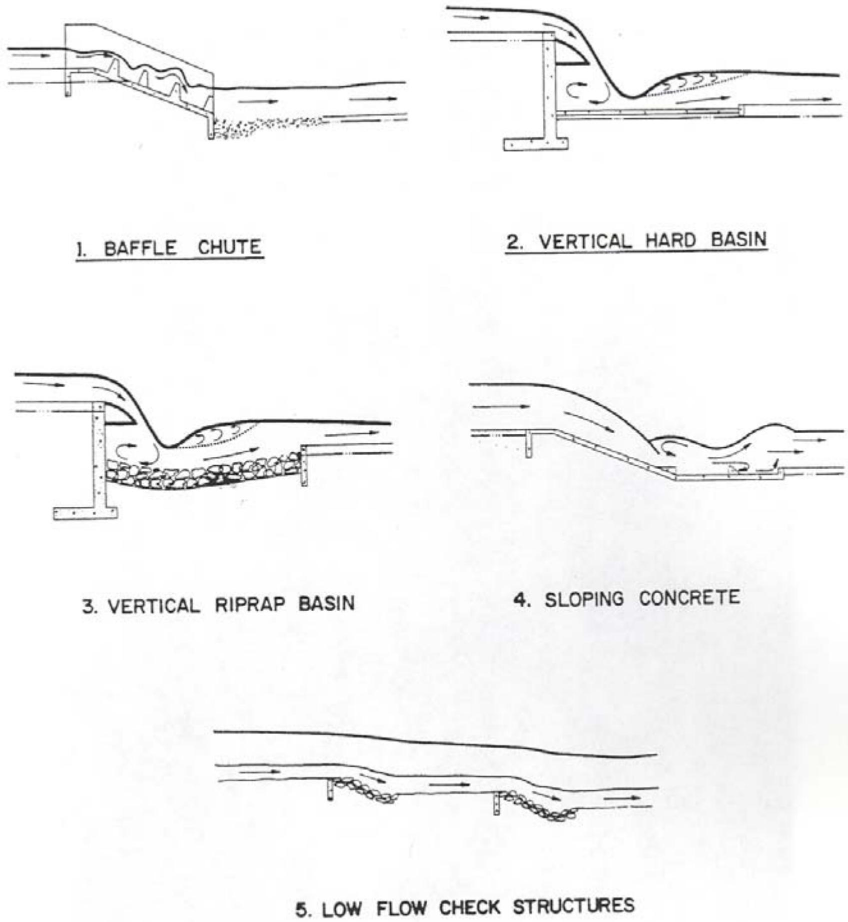
Drop Structure Types

Design guidance is presented in this section for the following drop structures:

- Baffle Chute Drops
- Vertical Hard Basin Drops
- Vertical Riprap Basin Drops
- Sloping Concrete Drops
- Grade Control Structures

Figure 8.2 shows schematic profiles of each type.

Figure 8- 2: Drop Structure Types
(McLaughlin Water Engineers, Ltd. 1986)



Due to a high failure rate and excessive maintenance costs, drop structures having loose riprap on a sloping face are not permitted.

All drop structures should be inspected on a regular basis during construction in regard to construction quality and integrity. In addition, drop structures must be monitored on a periodic basis after construction.

Additional bank and bottom protection may be needed if secondary erosional tendencies are revealed. Thus, it is advisable to establish construction contracts and budgets with this in mind.

Use of standardized design methods for the types of drops described herein can reduce the need for secondary design refinements. Section 8.3.4 presents considerations for the selection of the appropriate type of drop structure for particular application or site conditions.

8.3.2.2 Baffle Chute Drops

The USBR has developed design standards for a reinforced concrete chute with baffle blocks on the sloping face of the drop, which is commonly referred to as baffled apron or baffle chute drops. There are excellent references that should be used for the design of these structures: Hydraulic Design of Stilling Basins and Energy Dissipators (Peterka, 1984), and Design of Small Canal Structures (USBR, 1974). Another reference is Baffled Apron as Spillway Energy Dissipator (Rhone, 1977), which evaluates higher design discharges, and entrance modifications to reduce the backwater effect caused by the baffles.

The optimal performance occurs for a unit flow (q) at the chute width of 35 to 60 cfs/ft. Model testing has evaluated discharges up to 300 cfs/ft, and there have been structures built with up to 120 cfs/ft. The USBR states that the recommended design flow of 60 cfs/ft for baffle chute drops has been exceeded at several locations without causing significant problems.

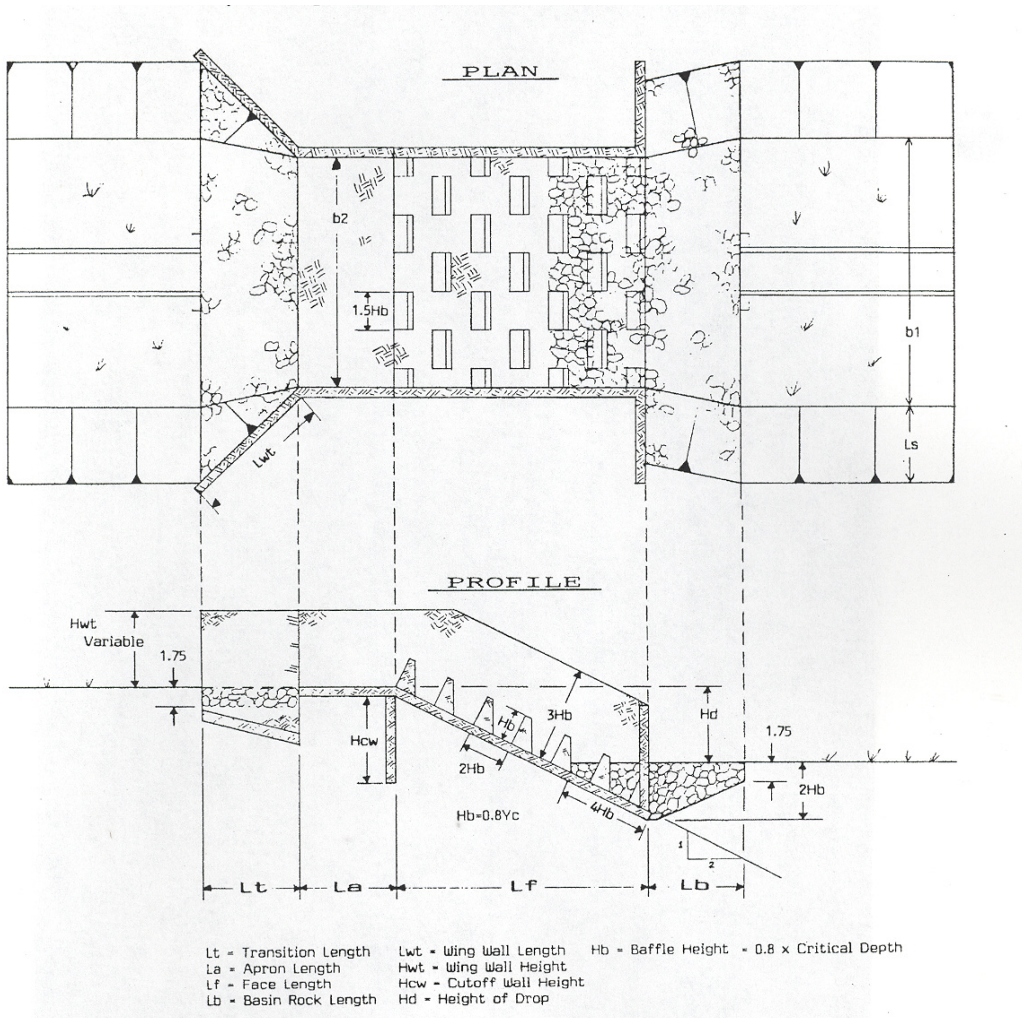
The hydraulic concept involves flow repeatedly encountering obstructions (baffle piers) that are of a nominal height equivalent to critical depth. The excess energy through the drop is dissipated by the momentum loss associated with the reorientation of flow. A minimum of four rows of baffle piers are recommended to achieve control of the flow and maximum dissipation of energy. Guidelines are given for sizing and spacing the blocks. Designing for proper approach velocities is critical to structure performance. One advantage of the baffle chute drop is that it does not require tailwater control.

Typical design consists of upstream transition walls, a rectangular approach chute, a sloping apron of 2:1, or flatter, slope with multiple rows of baffle piers (see Figure 8.3). The toe of the chute extends below grade and is backfilled with loose rock to prevent undermining the structure by eddy currents or minor degradation of the downstream channel. This rock will rearrange to establish a stable bed condition and produce additional stilling action. The structure is effective without tailwater; however, higher tailwater reduces scour at the toe. Grouted and concrete basins have also been used to prevent a standing pool from forming at the transitions to the downstream trickle and main channels. The structure also lends itself to a variety of soils and foundation conditions.

There are fixed costs associated with the upstream wing walls, crest approach section, downstream transition walls and a minimum length of sloping apron (for four baffle rows). Consequently, the baffle chute becomes more economical with increasing drop height.

This design is quite flexible in adaptation, once the hydraulic principles are understood. For example, the design has been modified for low drops by locating two rows of baffles on the slope and two rows on a horizontal extension of the chute. Another approach has been to use a flatter chute slope than the usual 2 horizontal to 1 vertical. There are examples where sloping abutments have been used. Other examples include the use of sloping abutments at the crest and chute sides. These drops can be extended at a later date if downstream bed degradation occurs beyond that initially anticipated.

Figure 8- 3: Baffle Chute Drop
(McLaughlin Water Engineers, Ltd., 1986)



The potential for debris flow must also be considered. Use caution when conditions include streams with heavy debris flow, because the baffles can become clogged between the interstices, resulting in overflow, low energy dissipation, and direct impingement of the erosive stream jet on the downstream channel.

The design performance has been documented for numerous baffled apron drops (USBR, 1974). The resulting design precautions generally relate to relatively minor problems, such as erosion protection in adjacent channels, spray above the chute walls, and debris problems. The basic design criteria and modification details are given in Figure 8.4 and Figure 8.5. Remaining structural design parameters must be determined for specific site conditions. The recommended design procedures are discussed on the following pages.

Figure 8- 4: Baffle Chute Design Criteria
(Adapted from: Peterka 1984)

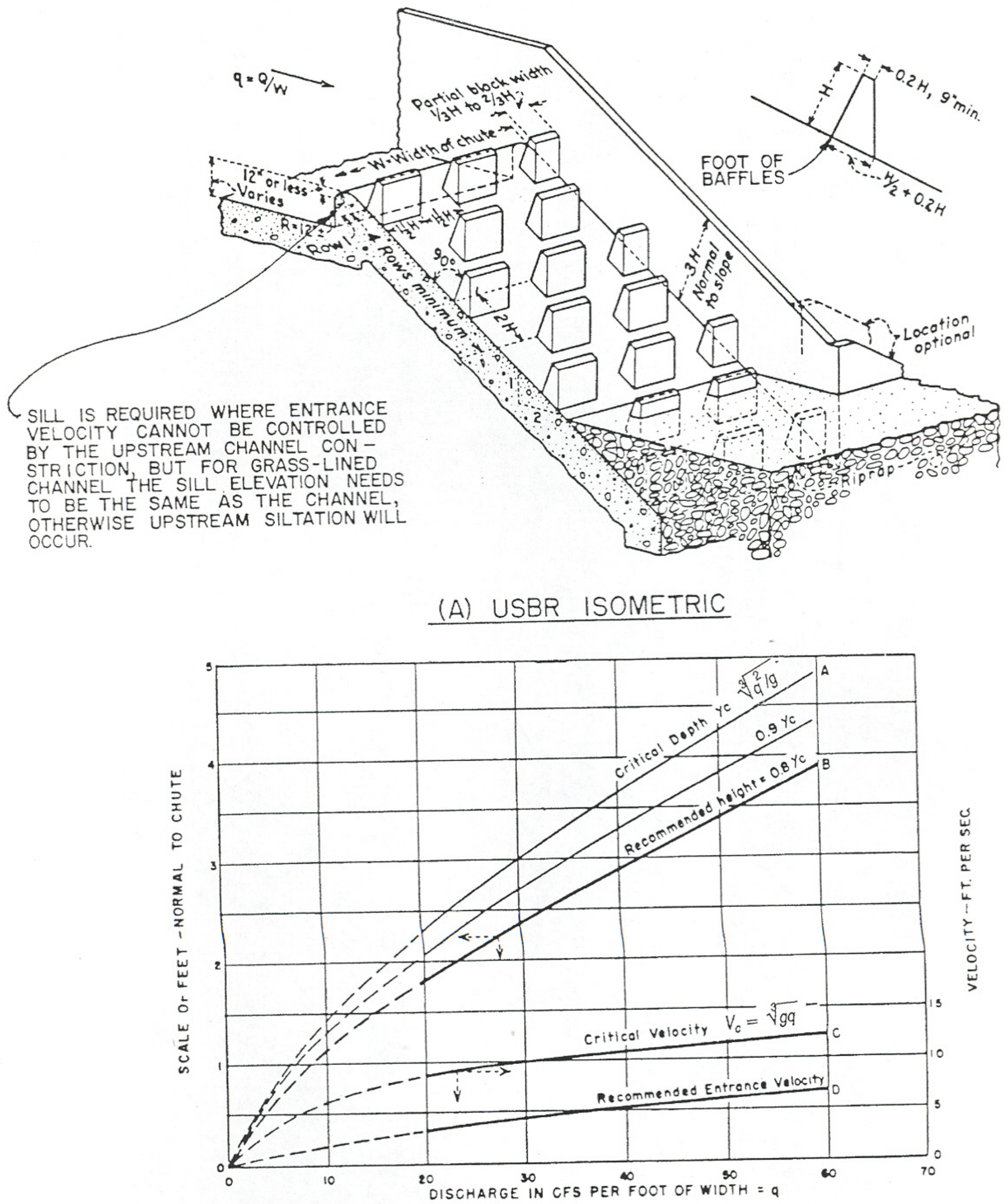
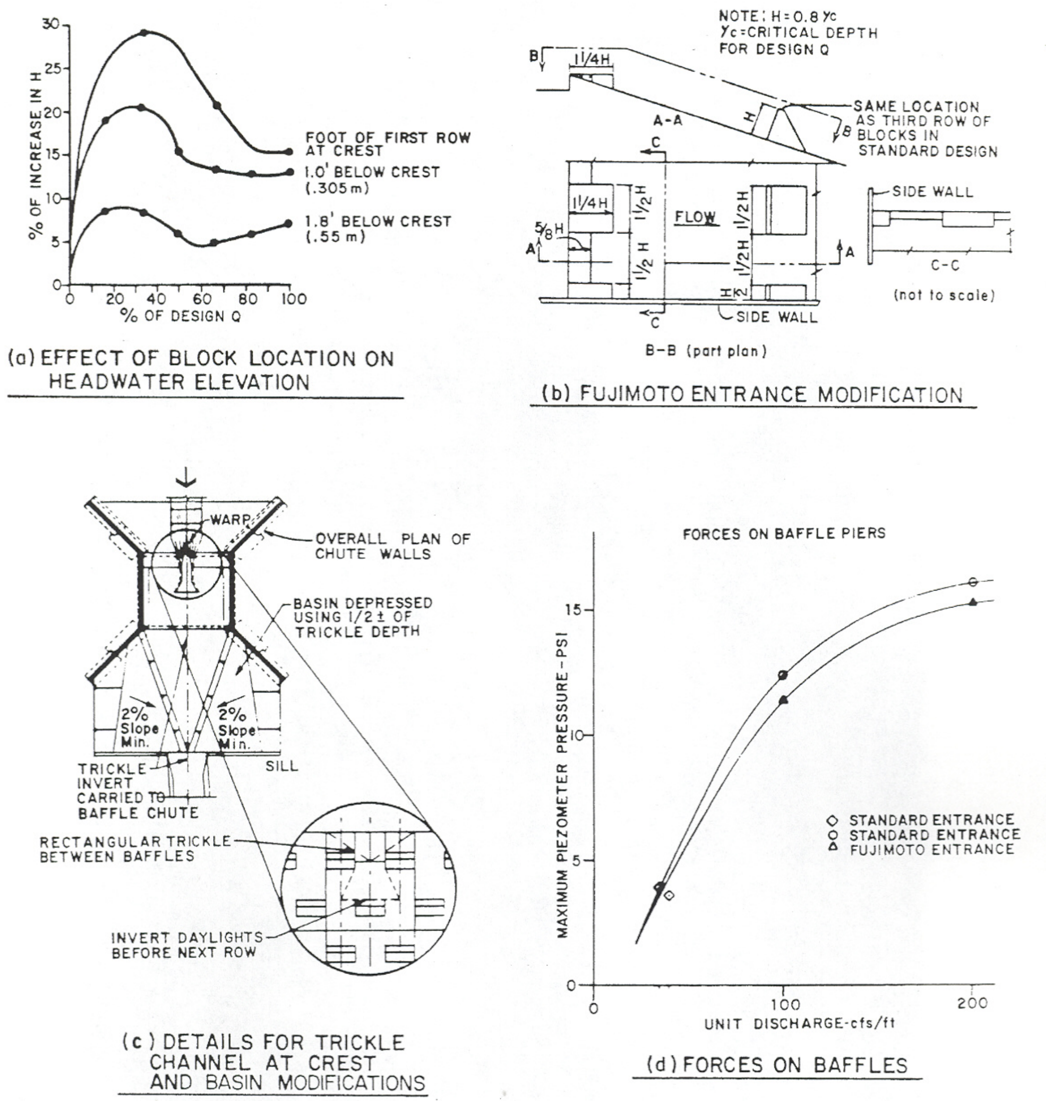


Figure 8- 5: Baffel Chute Crest Modifications and Forces
(Adapted from: Peterka 1984)



Hydraulic Design Procedure:

1. Determine the maximum inflow rate and the design discharge per unit width:

$$Q = Q / W \tag{8.1}$$

2. The chute width, W, may depend on the upstream or downstream channel width, the upstream hydraulic control, economy, or local site topography. Generally, a unit discharge between 35 to 60 cfs/ft is most economical.

3. An upstream channel transition section with vertical wing walls, constructed 45 degrees to the flow direction, causes flow approaching the rectangular chute section to constrict. It is also feasible to use walls constructed at 90 degrees to the flow direction. In either configuration, it is important to analyze the approach hydraulics and water surface profile. Often, the effective flow width at the critical cross section is narrower than the width of the chute opening due to flow separation at the corners of the abutment. To compensate for flow separation, it is recommended that the actual width constructed be 1 foot wider than the design analysis width if the constricted crest width is less than 90 percent of the upstream channel flow width. In any case, the design should carefully consider the approach hydraulics and contraction/separation effects. Depth and approach velocities should be evaluated through the transition to determine freeboard, scour, and sedimentation zones.

4. The entrance transition is followed by a rectangular flow alignment apron, typically 5 feet in length. The upstream approach channel velocity, V, should be as low as practical and less than critical velocity at the control section of the crest. Figure 8.4(b) gives the USBR recommended entrance (channel) velocity. In a typical grasslined channel, the entrance transition to the rectangular chute section will produce the desired upstream channel velocity reduction. The elevated chute crest above the channel elevation, as shown in Figure 8.4 (a), should only be used when approach velocities cannot be controlled by the transition. Special measures to prevent aggradation upstream would be necessary with the raised crest configuration.

Entrance Modification:

1. The trickle flow (or low flow) channel should be maintained through the apron, approach, and crest sections. It may be routed between the first row of baffle piers. The trickle channel should start again at the basin rock zone which should be slightly depressed and then graded up to transition to the downstream channel. Figure 8.5(c) illustrates one method of designing the low flow channel through the crest.

2. The conventional design shown in Figure 8.4(a) results in the top elevation of the baffles being higher than the crest, which causes a higher backwater surface effect upstream. Figure 8.4(b) may be used to estimate the extent of the effect and to determine corrective measures, such as increasing the upstream freeboard or widening the chute. Note that baffles projecting above the crest will tend to produce upstream sediment aggradation. Channel aggradation can be minimized by the low flow treatment suggested in the previous paragraph.

Another means of alleviating these problems is the Fujimoto entrance, developed by the USBR and illustrated in Figure 8.5(b). The upper rows of baffles are moved one row increment downstream. The important advantage of this entrance is that there is no backwater effect of the baffles. The serrated treatment of the modified crest begins disrupting the flow entering the chute without increasing the headwater. More importantly, this configuration provides a level crest control. The designer may either bring the invert of the upstream low flow channel into this crest elevation, widening the low flow channel as it approaches the crest, or the designer may have a lower trickle channel and bring it through the serrated crest similar to 1, above. These treatments will have to be observed until more application experience shows what may work best.

Structural Design Dimensions:

$$y_c = \left(\frac{q^2}{g}\right)^{0.33} \qquad (8.2)$$

1. Assume critical flow at the crest and determine critical depth for both peak flow and for 2/3 of peak flow. For unit discharge exceeding 60 cfs/ft, Figure 8.4(b) may be extrapolated:

2. The chute section (baffled apron) is concrete with baffles of height, H_b, equal to 0.8 times critical depth. The chute face slope is 2:1 for most cases, but may be reduced for low drops or where a flatter slope is desirable. For unit discharge applications greater than 60 cfs/ft, the baffle height may be based on 2/3 of the peak flow; however, the chute side walls should be designed for peak flow (see number 4).Baffle pier widths and spaces should equal, preferably, about 1.5 H_b but not less than H_b. Other baffle block dimensions are not critical hydraulically. The spacing between the rows of baffle blocks should be H_b times the slope. For example, a 2:1 slope makes the row spacing equal to 2Hb parallel to the chute floor. The baffle piers are usually constructed with the upstream face normal to the chute floor surface.

3. Four rows of baffle piers are required to establish full control of the flow, although fewer rows have operated successfully. At least one row of baffles is buried in riprap where the chute extends below the downstream channel grade. Riprap protection continues from the chute outlet to a distance of approximately $4H_b$, or as necessary to prevent eddy currents from undermining the walls. Additional rows of baffles may be buried below grade to allow for downstream channel degradation.

4. The baffle chute side wall height (measured normal to the floor slope) should be 2.4 times the critical depth based on peak discharge (or $3H_b$). The wall height will contain the main flow and most of the splash. The design of the area behind the wall should consider that some splash may occur, but extensive protection measures are not required.

5. Determine upstream transition and apron side wall height as required by backwater analysis. Lower basin wing walls are generally constructed normal to the chute side walls at the chute outlet to prevent eddy current erosion at the drop toe. These transition walls are of a height equal to the channel normal depth plus 1 foot, and length sufficient to inhibit eddy current erosion.

6. All concrete walls and footer dimensions are determined by conventional structural methods. Cutoff walls and underdrain requirements are determined by seepage analysis (see Section 8.3.3.6).

7. The most troublesome aspect of the design is the determination of the hydraulic impact forces on the baffles to allow the structural engineer to size adequate reinforcing steel. Figure 8.5(d) may be used as a guideline. The structural engineer should apply a conservative safety factor, as this curve is based on relatively sparse information.

Construction Considerations

There are numerous steps necessary in the construction of a baffle chute, but they are usually easily controlled by a contractor. For quality control and inspection, there are consistent, measurable, and repeatable standards to apply.

Potential areas of concern include foundation problems, riprap quality control and placement, and finish work with regard to architectural and landscape treatments. Formwork, form ties, and seal coatings can leave a poor appearance, if not handled properly. Poor concrete densification (usually using vibration) can result in surface defects (honeycombing) or more serious conditions, such as exposed rebar.

In summary, baffle chute drop structures are the most successful as far as hydraulic performance is concerned and are straightforward to construct. Steel, formwork, concrete placement and finish, and backfill require periodic inspection.

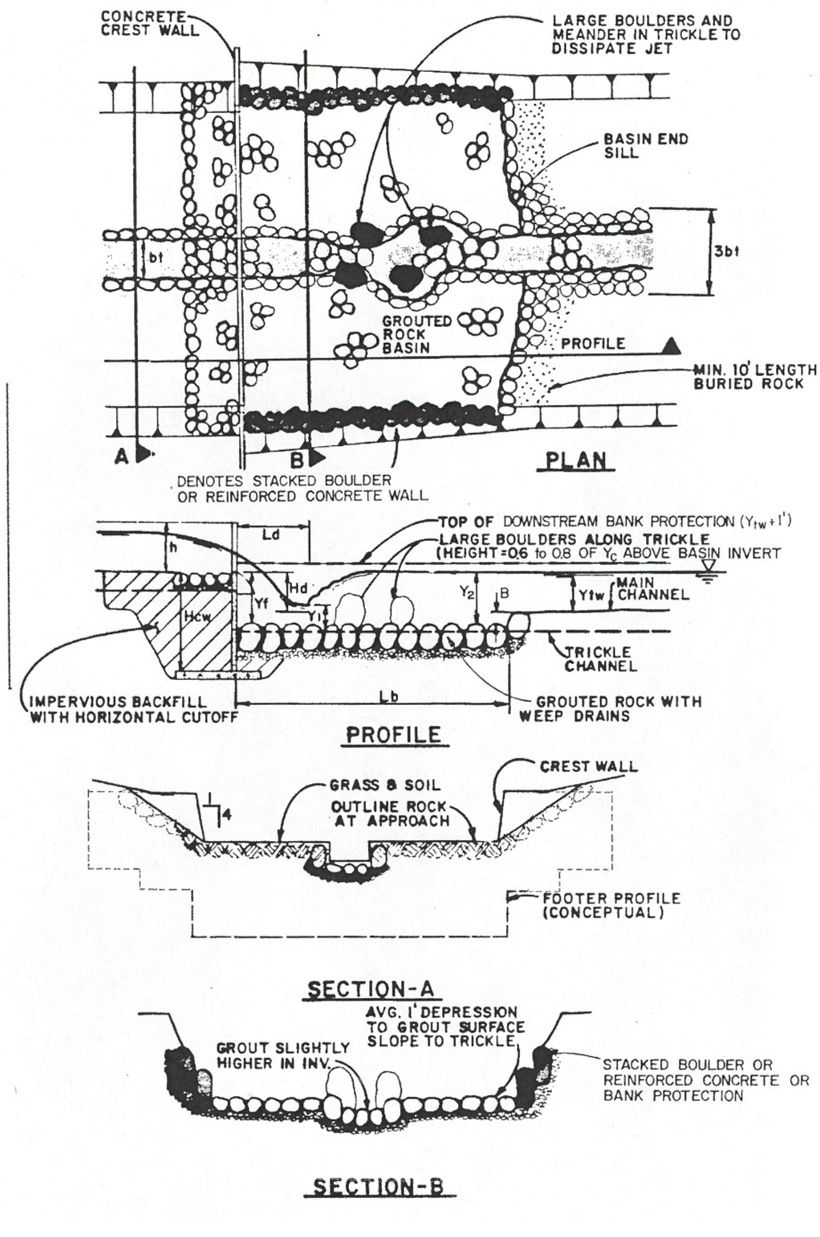
8.3.2.3 Vertical Hard Basin Drops

The vertical hard basin is a generalized category which can include a wide variety of structure design modifications and adaptations. A variety of components can be used for both the hard basin and the wall, various contraction effects can be implemented to reduce approach velocities, and different trickle channel options can be selected. The maximum vertical drop height from crest to basin for a vertical hard basin drop should be limited to 2.5 feet for safety considerations subject to the local jurisdiction's standards. Similarly, a 6-foot apron should be employed for each 2.5 feet of vertical drop. For drops greater than 2.5 feet, a stair step configuration is required.

The hydraulic phenomenon provided by this type of drop is a jet of water which overflows the crest wall into the basin below. The jet hits the hard basin and is redirected horizontally. With sufficient tailwater, a hydraulic jump is initiated. Otherwise, the flow continues horizontally in a supercritical mode until the specific force of the tailwater is sufficient to force the jump. Energy is dissipated in the turbulence through the hydraulic jump; therefore, the basin is sized to contain the supercritical flow and the erosive turbulent zone.

Generally, a rough basin is advantageous since increased roughness will result in a shorter, more economical basin. Figure 8.6 shows a vertical drop with a grouted boulder basin (concrete may also be used), and illustrates several important design considerations.

Figure 8- 6: Vertical Hard Basin Drop
(McLaughlin Water Engineers, Ltd., 1986)



General Hydraulic Design Procedure

1. The design approach uses the unit discharge in the main channel and the trickle channel to determine the separate water surface profiles and jump locations in these zones. The basin is sized to adequately contain the hydraulic jump and associated turbulent flows.

2. The rock lined approach length ends abruptly at a structural retaining crestwall which has a nearly rectangular cross section and trickle channel section.

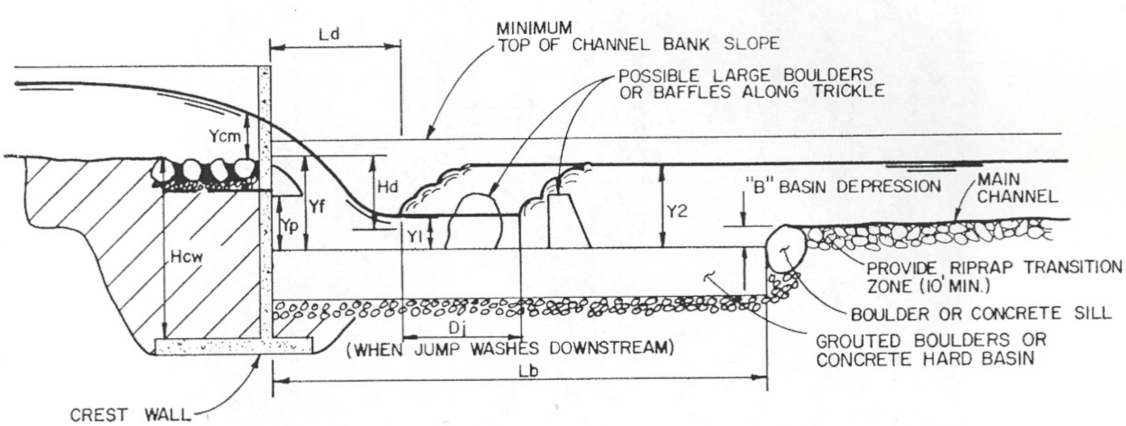
3. Crest wall and footer dimensions are determined by conventional structural methods. Underdrain requirements are determined from seepage analysis.

4. Open Channel Hydraulics (Chow, 1959), makes a brief presentation for the "Straight Drop Spillway," which applies here. Separate analysis would need to be undertaken for the trickle channel area and the main channel are. In the following equations add the subscript t for the trickle channel area, and the subscript m for the main channel area.

Refer to Figure 8.7 to identify the following parameters. L_b is the design basin length which includes L_d and the distance to the jump, D_j, which is measured from the downstream end of L_d. The jump length, L_j, is approximated as six times the sequent depth, Y_2. As a safety factor, to assure a sufficient length for L_b, 0.6 L_j is added in the design of L_b, such that:

$$L_b > L_d + D_j + 9.6\,Y_2 \tag{8.3}$$

Figure 8- 7: VERTICAL DROP HYDRAULIC SYSTEM
(McLaughlin Water Engineers, Ltd., 1986)



When a hydraulic jump occurs immediately where the nappe hits the basin floor, the following variables are defined:

$$\frac{L_d}{Y_f} = 4.3\,\mathrm{D_n}^{0.27} \tag{8.4}$$

Where:

$$D_n = \frac{q_c^{\ 2}}{gY_f^{\ 3}} \tag{8.5}$$

$$Y_p / Y_f = 1.0 D_n^{\ 0.22} \tag{8.6}$$

$$Y_1 / Y_f = 0.54 D_n^{\ 0.425} \tag{8.7}$$

$$Y_2 / Y_f = 1.66 D_n^{\ 0.27} \tag{8.8}$$

5. In the case where the tailwater does not provide a depth equivalent to or greater than Y_2, the jet will wash downstream as supercritical flow until its specific force is sufficiently reduced to allow the jump to occur. Determination of the distance to the hydraulic jump, D_j, requires a separate water surface profile analysis for the main and low flow zones. Any change in tailwater affects the stability of the jump in both locations.

6. Caution is advised regarding the higher unit flow condition in the low flow zone. Large boulders and meanders in the trickle zone of the basin are shown to help dissipate the jet, and rock is extended downstream along the low flow channel. This results in three possible basin length design conditions:

   a. At the main channel zone:

   $$L_{bm} = L_{dm} + D_{jm} + 0.60(6Y_2)_m \tag{8.9}$$

   b. At the trickle zone, standard design:

   $$L_{bt} = L_{dt} + D_{jt} + 0.60(6Y_2)_t \tag{8.10}$$

c. When large boulders or baffles are used to confine the jump to the impingement area of the low flow zone, the low flow basin length may be reduced:

$$L_{bt} = L_{dt} + 0.60(6Y_2)_t \quad (8.11)$$

7. The basin floor elevation is depressed at depth B, variable with drop height and practical for trickle flow drainage. Note that the basin depth adds to the effective tailwater depth. The basin is constructed of concrete or grouted rock. Either material must be evaluated for the hydraulic forces and seepage uplift.

8. There is a sill at the basin end to bring the invert elevation to that of the downstream channel and side walls extending from the crestwall to the sill. The sill is important in causing the hydraulic jump to form in the basin. Buried riprap should be used downstream of the sill to minimize any local scour caused by the lift over the sill.

9. Water surface profile analyses have proven that base widths of the rectangular crest which are less than that of the channel will result in high unit discharges and velocities, thereby requiring unreasonable extensions of both the basin length and upstream rock protection. Roughness in the basin area can reduce the basin length required to contain the hydraulic jump. This is the primary advantage of the use of grouted rock in the drop basin.

Construction Considerations

Foundation and seepage concerns are very critical with regard to the vertical wall, as poor control can result in sudden failure. The use of caissons or pile can mitigate this effect. Put in comparative terms with the baffle chute, seepage problems can result in displacement of the vertical wall with no warning, where the box-like structure of the baffle chute may evidence some movement or cracking, but not total failure, and thus allow time for repairs.

The quality control concerns and measures for reinforced concrete are described under baffle chutes. The foundation concerns for the wall are critical as described above. The subsoil conditions for the basin are also important so that the basin concrete or grouted riprap is stable against uplift pressures.

A grouted boulder stilling basin provides roughness, which is useful in shortening the basin length. As the name implies, the basin should be constructed of individual boulders placed on a prepared subgrade. Boulders should be a minimum dimension that exceeds the grout layer thickness, so that the contractor and the inspector can see and have grout placed directly to the subgrade and completely filling the voids. Graded riprap should not be used for grouting, as the smaller rock prevents the voids from being completely filled with grout. The result is a direct piping route for water beneath the grout, and a structural slab with insufficient mass. The completed combination of boulders and grout should have an overall weight sufficient to offset uplift forces. A minimum dimension of 18 inches is recommended for boulders, and 12 inches for the grout layer. By maintaining the finished surface of the grout below the top of the boulder, both appearance and roughness characteristics are enhanced. Seepage relief for the basin slab should be provided.

This type of structure has a moderate level of construction difficulty. The wall, once foundation conditions are addressed, is conventional construction. It is very possible for the construction of the seepage control and earthwork to go awry and problems to go undetected until the time of failure. The flat concrete or grouted rock placement is easier for the contractor than graded rock placement/quality control, but again poor placement and undetected subsoil, bedding or rock problems can result in failure. Thus, it is easier than many other types to construct, but susceptible to some hidden risks and problems.

8.3.2.4 Vertical Riprap Basin Drops

As shown in Figure 8.8, this structure is essentially a plunge pool drop that incorporates a reinforced concrete crest wall with a riprap lined dissipation pool below. A nearly rectangular crest section is recommended to reduce the width of the plunge pool. Maximum drop depth for a vertical riprap basin should be limited to 2.5 feet due to safety considerations and the practicality of obtaining the larger riprap needed for higher drops. This height limitation is subject to the standards evoked by the jurisdictional entity. Submergence by high tailwater can limit the dissipation efficiency.

The hydraulic design was developed through model testing by Smith and Strang in 1967 (Scour in Stone Beds) and design procedures were further developed by Stevens in 1981 (Hydraulic Design Criteria for Riprapped Chutes and Vertical Drop Structures).In this structure, flow passing over the vertical crest wall plunges into a riprap basin area. Energy is dissipated by turbulence in the plunge pool. Loose riprap is placed in the basin according to the initial design specifications. The rock is successively rearranged by inflows until a more stabilized basin plunge pool is formed. The depth of the scour hole, d_s, and the nominal rock size are inversely related.

Structural design for the vertical crest wall is complicated by the lack of downstream support, seepage, soil saturation and hydraulic loading on the upstream side. In sandy or erosive soils, it is common to use sheet pile for the crest wall construction, while caissons may be an acceptable foundation for certain other applications. A concrete retaining wall is frequently selected for ease of construction, seepage control and low maintenance.

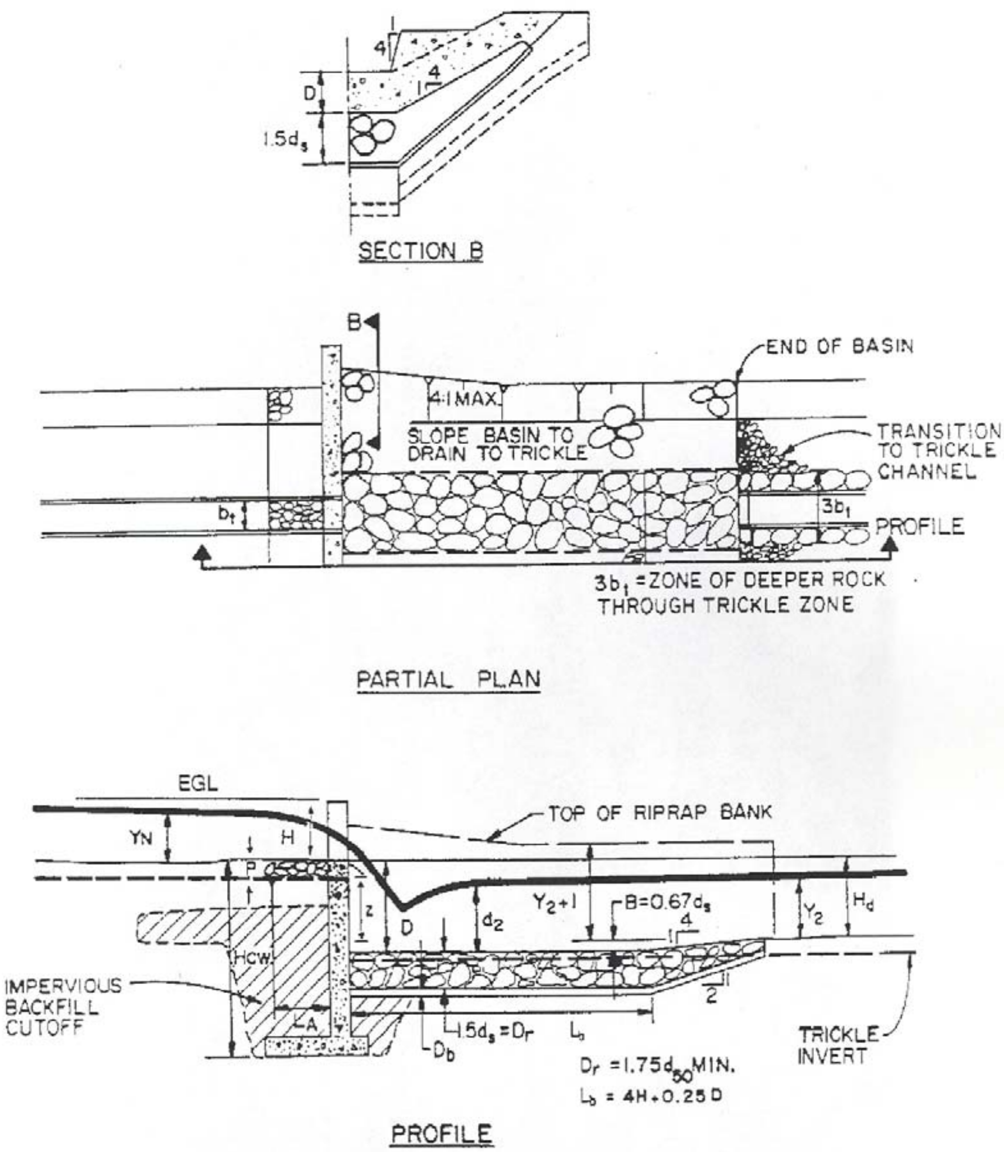
General Hydraulic Design Procedure

The hydraulic analysis of this type of drop is generally similar to that presented previously in this section for crest hydraulics. The design of the flexible plunge pool basin is described below.

The desired drop across the structure is the difference in the bed elevations of the approach channel at the weir and the downstream channel at the end of the structure. Let this difference be H_d. It follows from Figure 8.8 that:

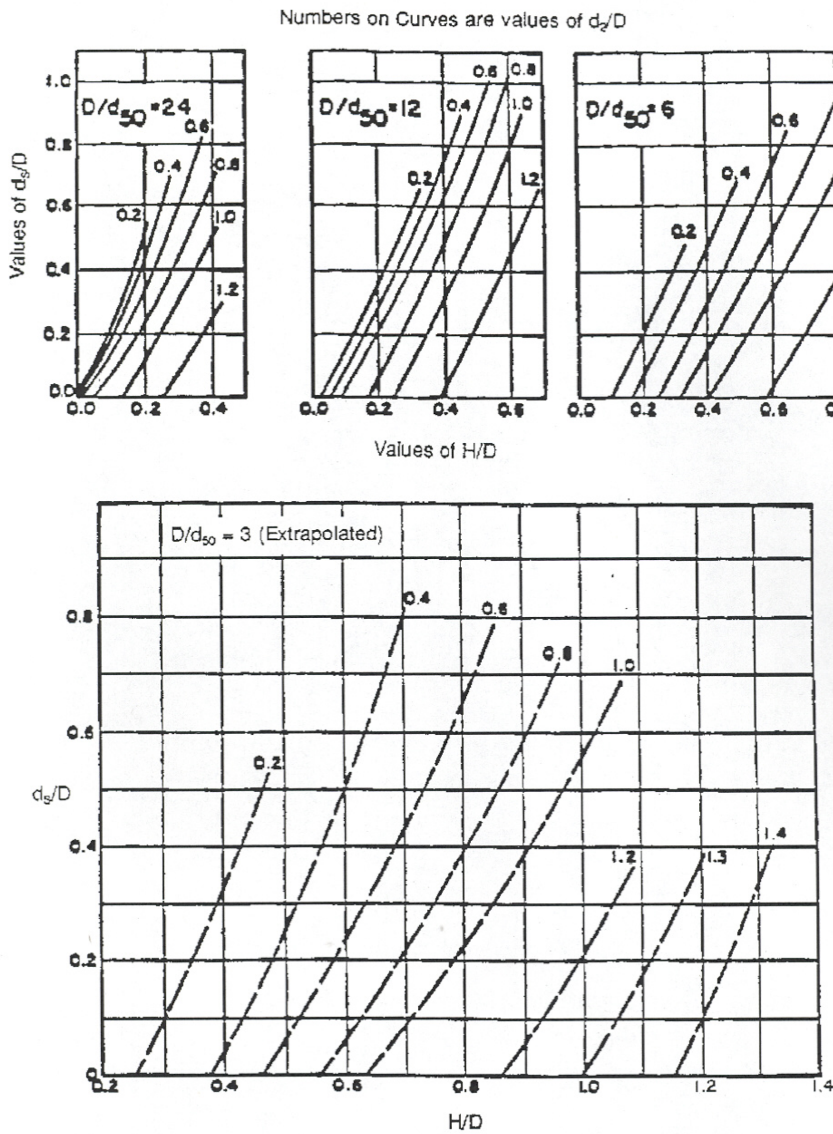
$$H_d = D - 0.67 d_s \qquad (8.12)$$

Figure 8- 8: Vertical Riprap Basin Drop
(Stevens, 1981)



The designer must find the combination of rock size and jet plunge height D that gives a depth of scour which balances Equation (8.16). The relation between rock size d_{50}, jet plunge height D, head on the weir, H, ($H = 1.5\ y_c$) and depth of scour d_s is given in Figure 8.9. As these values will be different in the main drop and the trickle, the design d_{50} and/or d_s will vary.

Figure 8- 9: Curves for Scour Depth at Vertical Drop
(Stevens, 1981)



To obtain an adequate cutoff, the depth of the vertical wall that forms the weir crest must extend below the bottom of the excavation for the riprap. Thus, it usually becomes uneconomical to design a scour depth d_s, any greater than 0.3 D. To meet this limitation in the field it is necessary to: increase the rock size d_{50}; decrease the jet plunge height D (by using more drops); decrease H (by using a wider structure); or, to use another type of drop structure.

The side slopes in the basin must be riprapped also as there are strong back currents in the basin. Granular filter material is required under this riprap. The side slopes in the basin should be the same slope as for the downstream channel.

Construction Considerations

Foundation and seepage concerns are critical with regard to the vertical wall in this type of drop. They are also generally more critical than with an equivalent vertical drop into a hard basin because the riprap basin may scour and reshape, leaving less supporting material on the downstream side. Thus, if seepage is worse than anticipated, backfill is poor, or if seepage control measures are not functioning, an immediate and severe structure stability problem can occur. The use of caissons or piles can mitigate this effect. Seepage problems can result in displacement of the vertical wall with no cracking as an advance warning. Seepage can also cause piping failure where the water will actually flow under the vertical wall. Problems can result from rock that does not meet specifications for durability, specific gravity or gradation. Quality control of rock installation can be difficult in regard to measuring performance and maintaining consistency. Undersized rock in the plunge pool basin can cause the basin to reshape differently than designed and result in stability problems for the wall, the basin, and the downstream channel.

This type of structure has a moderate level of construction difficulty. The wall, once foundation conditions are addressed, is straightforward. It is very possible for the construction of the seepage control and earthwork to go awry and for problems to go undetected until the time of failure. The flat riprap placement is easier than sloping, but again poor placement and undetected subsoil, bedding, or rock problems can all contribute to failure.

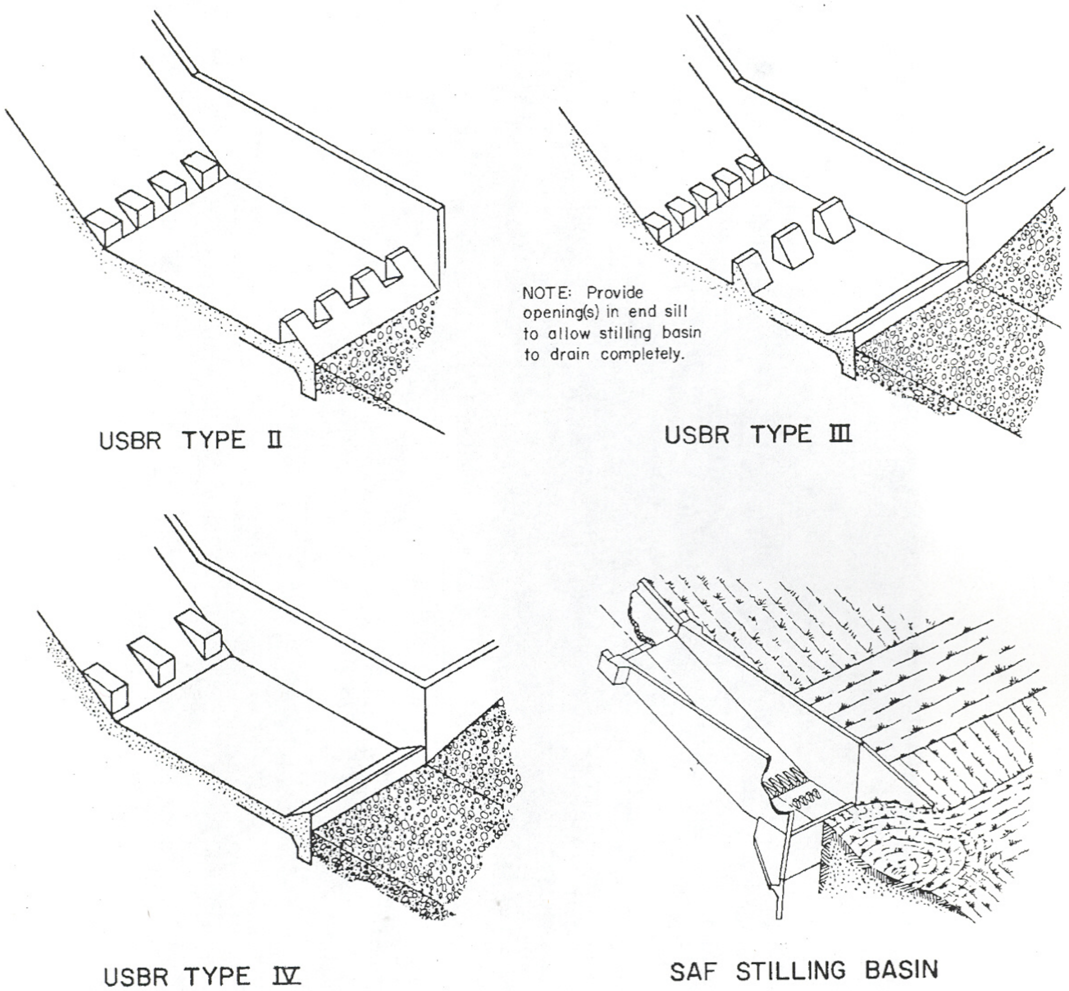
8.3.2.5 Sloping Concrete Drops

The hydraulic concept of these structures is to dissipate energy by formation of a conventional hydraulic jump, usually associated with a reverse current surface flow as the supercritical flow down the face converts to subcritical flow downstream.

Numerous concepts have been investigated. Among them are the Saint Anthony Falls (SAF) Stilling Basin, and the USBR Basins I, II, III, and IV (USDOT, 1983; and Peterka 1984). These drops and associated basins are suited for different kinds of situations.

The SAF and the USBR Basins (with the exception of Type I) all work at techniques to shorten the basin length. In the USBR Basin I, no special measures are provided. On the smooth concrete basin it can take considerable basin length to "burn off" enough energy to dissipate the supercritical flow of where a jump will begin, and then more length to allow for the turbulence of the jump. Basin I is relatively expensive because of its length. The other basins require a certain amount of tailwater, which requires depressing the basin, and the use of baffles or other shapes to allow shorter basins, related dissipation, and control of troublesome wave patterns. Figure 8.10 illustrates the various types of stilling basins for use with sloping concrete drops.

Figure 8- 10: Stilling Basins for Sloping Drop
(Adapted from: USDOT, FHWA, HEC-14, 1983)



General Hydraulic Design Procedure

Design procedures for USBR Basins II, III, and IV and the SAF Stilling Basin are presented in Hydraulic Design of Energy Dissipators for Culverts and Channels (USDOT, 1983) and Hydraulic Design of Stilling Basins and Energy Dissipators (Peterka, 1984). Once water surface profiles have been determined, including tailwater determination and supercritical water surfaces down the sloping face, seepage uplift forces must be evaluated. Net uplift forces vary as a function of location along the drop, cutoff measures, drain gallery locations and water surface profiles through the basin.

For a stable structure, net uplift force from seepage must be countered by net forces in the downward direction. For a smooth concrete chute, downward forces are the buoyant weight of the concrete structure and the weight of water (a function of the depth of flow). Significant pressure differentials can occur with a combination of high seepage forces and shallow supercritical flow. Seepage analyses should be conducted using Lane's weighted creep methodology (Section 8.3.3.6), and suitable countermeasures designed. Such measures include cutoff walls, weep drain galleries and concrete slab thickness design. A range of flood discharges should be evaluated, since differential pressure relationships can vary with flow depth and location of hydraulic jump.

Construction Considerations

There may be applications where sloping concrete drops are advantageous, but generally other drops such as baffle chutes or vertical drops are more appropriate for a wider range of applications. The design guidance provided by the literature is clear and relatively easy to use, but the implementation is often difficult or impractical. This basically has to do with providing basin depth without creating a maintenance problem and less flexibility in adapting to varying bed conditions.

The integrity of the cutoff is important as seepage and resultant uplift forces are key concerns. Uncontrolled underflow could easily lift a major concrete slab.

The stilling basin should be designed to drain completely, to eliminate nuisances related to ponded water, such as mosquito breeding and sediment/debris accumulation.

Considerations relating to general concrete construction are the same as discussed previously for baffle chute drops. Public acceptability is likely to be low in urban areas, as the sloping concrete face is inviting for bicyclists, roller skaters, and skateboard enthusiasts.

8.3.2.6 Other Types of Drop Structures

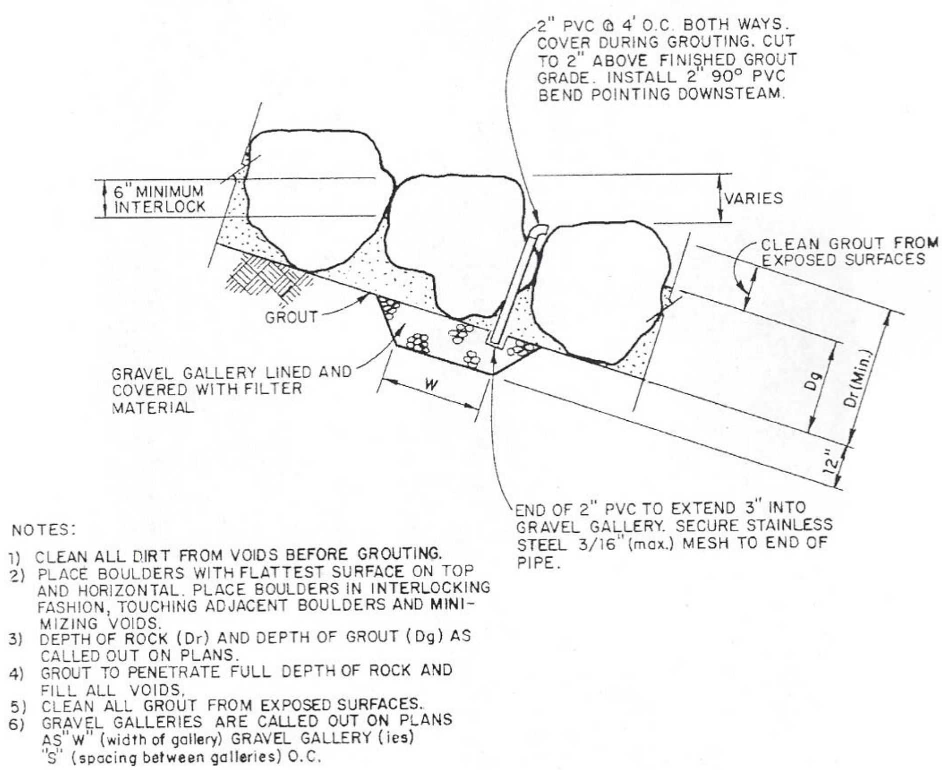
There are numerous other types of drop structures for specific applications in drainage design. The four types of structures presented above are appropriate for the majority of situations to be encountered in Pinal County. Some possible variations or modifications are presented below along with a few specialized types.

Sloping Drop Variations

The use of soil cement, roller compacted concrete, and grouted boulders are possible variations in sloping drop design. The primary concern with soil cement is its ability to resist the high abrasive action of turbulent flow associated with a drop structure. Adequate countermeasures would be required to demonstrate the suitability of soil cement prior to its approval for use on drop structures.

Addition of roughness elements on the face of a sloping concrete drop can provide increased energy dissipation. "Stepped" concrete has been successfully applied at spillways and drop structures. Roller compacted concrete is a methodology that can achieve the stairstep geometry on the face of a sloping drop. Reinforced concrete steps can be constructed by standard construction methods on small structures. Stepped drop structures have been found to be effective in dissipating the energy associated with low flows but fail to effectively dissipate energy of higher flows. Thus, stilling basin length for a stepped drop structure will be based upon the conventional length calculations for a sloping drop presented herein. Stepped drop structures will be no steeper than 2H:1V with a step height no greater than 2.5 feet and a step apron length of 6 feet. Construction of a drop with grouted boulders is another means of creating desirable roughness on the sloping face and in the stilling basin (see Figure 8.11).

Figure 8- 11: Boulder Placement
(McLaughlin Water Engineers, Ltd.,1986)



However, because the structure is comprised of a structural slab with two components (boulders and grout), great care must be taken to design the structure to withstand uplift and to specify boulder and grout material to assure full quality control in the field. Seepage analysis is required to determine a compatible combination of cutoff depth, location of the toe drain and/or other drains, and the thickness of rock and grout. Problems with rock specific gravity, durability and hardness are of concern. Gradation problems are largely eliminated because the boulders are specified to meet minimum physical dimensions and/or weights, which is much easier to observe and enforce in the field than with graded riprap.

The handling of the large boulders requires skilled work force and specialized equipment. Equipment similar to logging tongs, and specially modified buckets with hydraulically powered "thumbs" have been used in recent years and have greatly improved quality and placement rates. The careful placement of stacked boulders, so that the upstream rock is keyed in behind the downstream rock, and placed with a large flat surface horizontally, has been shown to be successful.

The greatest danger lies with a "sugar coated" grout job, where the grout does not penetrate the voids between the rock and the subgrade, leaving a direct piping route for water under the grout. This can easily occur when attempting to grout graded riprap, thus the need to use individual boulders that are larger in diameter than the grout layer so that the contractor and the inspector can see and have grout placed directly to the subgrade. The best balance appears to be boulders 33 to 50 percent greater in size than the grout thickness, but of an overall weight sufficient to offset uplift. Also, when holding grout to this level, the appearance will be much better.

The grout should have a minimum 4,000 psi compressive strength at 28 days, stone aggregate with a maximum dimension of one-half inch and a slump within a range of 4 to 7 inches. The water/cement ratio should not exceed 0.48. Addition of synthetic fiber reinforcement is also recommended to provide crack control, increased durability, and increased abrasion resistance.

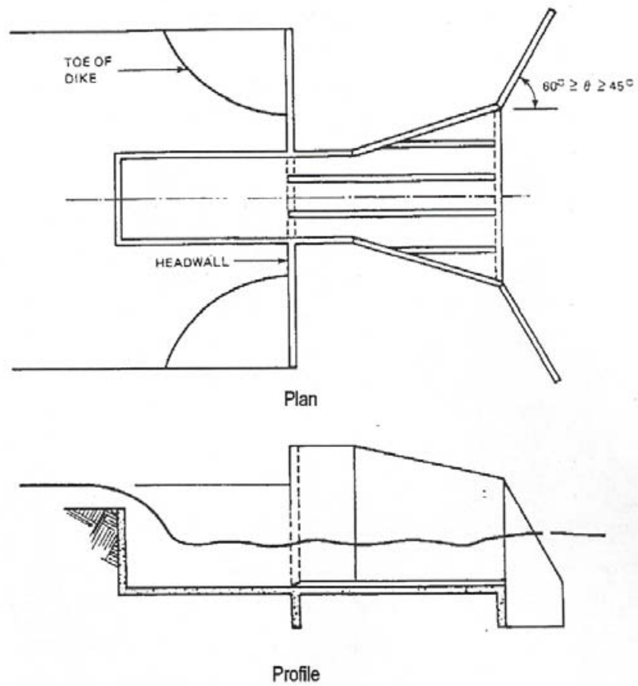
Other USBR Basins

Some other stilling basins developed by the USBR (Peterka, 1984) have limited application. For example, Basin I is basically a horizontal concrete apron downstream of a sloping or vertical drop. This type of basin is applicable only to a concrete lined channel, and, as the USBR states, has wave problems that are difficult to overcome. Maintenance of sufficient tailwater depth is important to cause a hydraulic jump within a practical zone close to the toe of the drop. Generally, other types of USBR basins are better alternatives to Basin I. USBR Basin V is a stilling basin with sloping apron, and provides dissipation as effective as that which occurs in the basin with a horizontal apron. Again, adequate tailwater is a must. This type of structure would have an application as a spillway into a pond with a permanent pool, so that minimum tailwater is essentially guaranteed.

Box Inlet Drop Structure

The box inlet drop structure may be described as a rectangular box open at the top and downstream end (Figure 8.12). Water is directed to the crest of the box inlet by earth dikes and headwalls. Flow enters over the upstream end and two sides and leaves the structure through the open downstream end. The long crest of the box inlet permits large flows to pass at relatively low heads. The width of the structure does not need to be greater than the downstream channel. It is applicable for drops from 2 feet to 12 feet. Designers of box inlet drop structures should review permissible drop heights allowed by governing jurisdictions as safety issues need to be considered.

Figure 8- 12: Box Inlet Drop Structure
(Adapted from: FHWA, HEC-14, 1983)



The outlet structure can be adjusted to fit a wide variety of field conditions. It is possible to lengthen the straight section and cover it to form a highway culvert. The sidewalls of the stilling basin section can be flared if desired, thus permitting use with narrow channels or wide floodplains. Flaring the sidewalls also makes it possible to adjust the outlet depth to match the natural channel.

Design guidelines are presented in Hydraulic Design of Energy Dissipators for Culvert and Channels (USDOT, 1983).

8.3.2.7 Grade Control Structures

Grade control structures can be effective in stabilizing natural channels and other unlined channels. These structures are designed to provide control points to maintain stable bed slopes. They do not stabilize channel side slopes. Set at grade across the channel/floodplain, these structures do not serve to change the velocity profile of the flow regime, but rather, serve as a barrier to headcutting. Here, headcutting is defined as the scouring of the channel bed proceeding from a downstream to upstream direction. Local soils, bed materials, and sediment gradation must be considered along with channel hydrology and hydraulics for the effective design of a grade control structure (See Chapter 7 for further discussion on sediment transport and estimating scour depth). The longevity of the structure is dependent upon the depth of toe down (among other things), which must exceed the depth of scour in order to stabilize the channel slope upstream of the structure. The potential for seepage cutoff must be assessed for hydrostatic pressure and the potential failure of the structure foundation due to “piping” of the underlying soils. If an issue, the appropriate engineered solutions should be employed in the design. These solutions include the use of geotextile filter fabrics to prevent soil loss and small diameter PVC pipes to relieve hydrostatic pressure. In any case, appropriate access to grade control structures is necessary to permit intermittent maintenance.

8.3.3 Hydraulic Analysis

A hydraulic analysis should be completed to ensure that an appropriate drop structure is designed and implemented.

8.3.3.1 Procedures

These design procedures are generalized. Use them to identify the most suitable approach, with the understanding that detailed analytical methods and design specifications may vary as a function of site conditions and hydraulic performance. A standard drop structure design approach would include at least the following steps:

1. Define the maximum design discharge (usually the 100-year) and other discharges appropriate for analysis (selected discharge(s) expected to occur on a more frequent basis, which may behave differently at the drop).

2. Select possible drop structure alternatives to be considered (Section 8.3.4).

3. Determine the required longitudinal channel slope and the total drop height required to produce the desired hydraulic conditions.

4. Establish the channel hydraulic parameters, reviewing drop structure and channel combinations that may be most effective.

5. Conduct hydraulic analyses for the structure. Where appropriate, apply separate hydraulic analyses to the main channel and the low flow zones of the drop to determine the extent of protection required, as well as the potential problems/solutions for each. (See discussion later in this section.)

6. Perform soils and seepage analyses to obtain foundation and structural design information. Combine seepage and hydraulic analysis data to determine forces on the structure. Evaluate uplifting, overturning, and sliding.

7. Evaluate alternative structures in terms of their estimated capital and maintenance costs, and identify comparable risks and problems for each alternative. Review alternatives with client and jurisdictional agency to select final plan. (This task is not specifically a part of the hydraulic analysis criteria, but is mentioned to illustrate other factors which are involved in the analysis of alternatives.)

8. Use specific design criteria to determine the drop structure dimensions, material requirements and construction methods necessary to complete the design for the selected structures.

8.3.3.2 Crest and Upstream Hydraulics

Usually, the starting point of drop analysis and design is the designation of the crest section (or review of existing configuration) at the top of the drop. As flow passes through critical depth near the crest, upstream hydraulics are separated from downstream. The critical flow state must be calculated and compared with the downstream tailwater effect which may submerge the crest and effectively control the hydraulics at the crest.

With control at the drop crest, upstream water surface profile computations are used to estimate the distance that protection should be maintained upstream, that is, the distance to where localized velocities are reduced to acceptable values. Backwater computations also yield the maximum upstream flow depth used to set wall abutment and bank heights. The water surface profile computations should include a transition/contraction head loss, which should typically range from 0.3 (modest transitions) to 0.5 (more abrupt transitions) times the change in velocity head. The reader should refer to standard hydraulic references for guidance (i.e., Chow 1959). For a given discharge, there is a balance between the crest base width, upstream and downstream flow velocities, the Froude Number in the drop basin, and the location of the jump. These parameters must be selected for each specific application.

Two basic configurations of crests are assumed. Baffle chutes, vertical hard basin and vertical riprap basin drops frequently have vertical or nearly vertical abutments with nearly rectangular cross sections. Sloping concrete drops generally have sloping abutments, forming a trapezoidal crest cross section. All drop types would typically have a low flow channel which is extended through the drop crest section at the channel invert.

Vertical or Near Vertical Abutments at Drop Crest - Figure 8.13 presents alternative drop crests at a vertical drop structure. In general, the objectives of upstream hydraulics and crest design are:

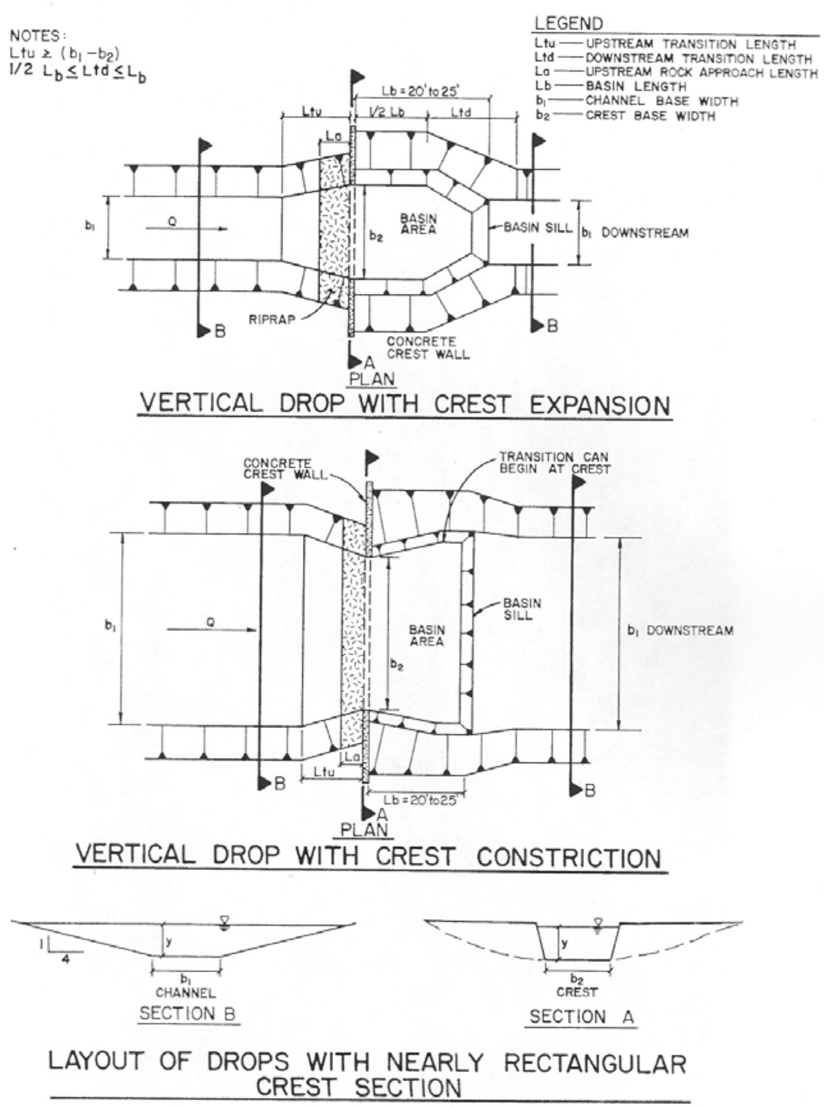
1. To maintain freeboard in the approach channel,
2. To optimize crest and basin dimensions to achieve the most costeffective structure, and

3. To prevent erosion in the transition zone, where flow accelerates approaching the crest.

A crest expansion may be necessary to maintain adequate freeboard in the upstream channel and reduce drawdown velocities just upstream of the crest. A crest constriction may be appropriate for wide channels to reduce the cost of the crest wall.

Figure 8- 13: Typical Vertical Drop Crest Configuration

(McLaughlin Water Engineers, Ltd. 1986)

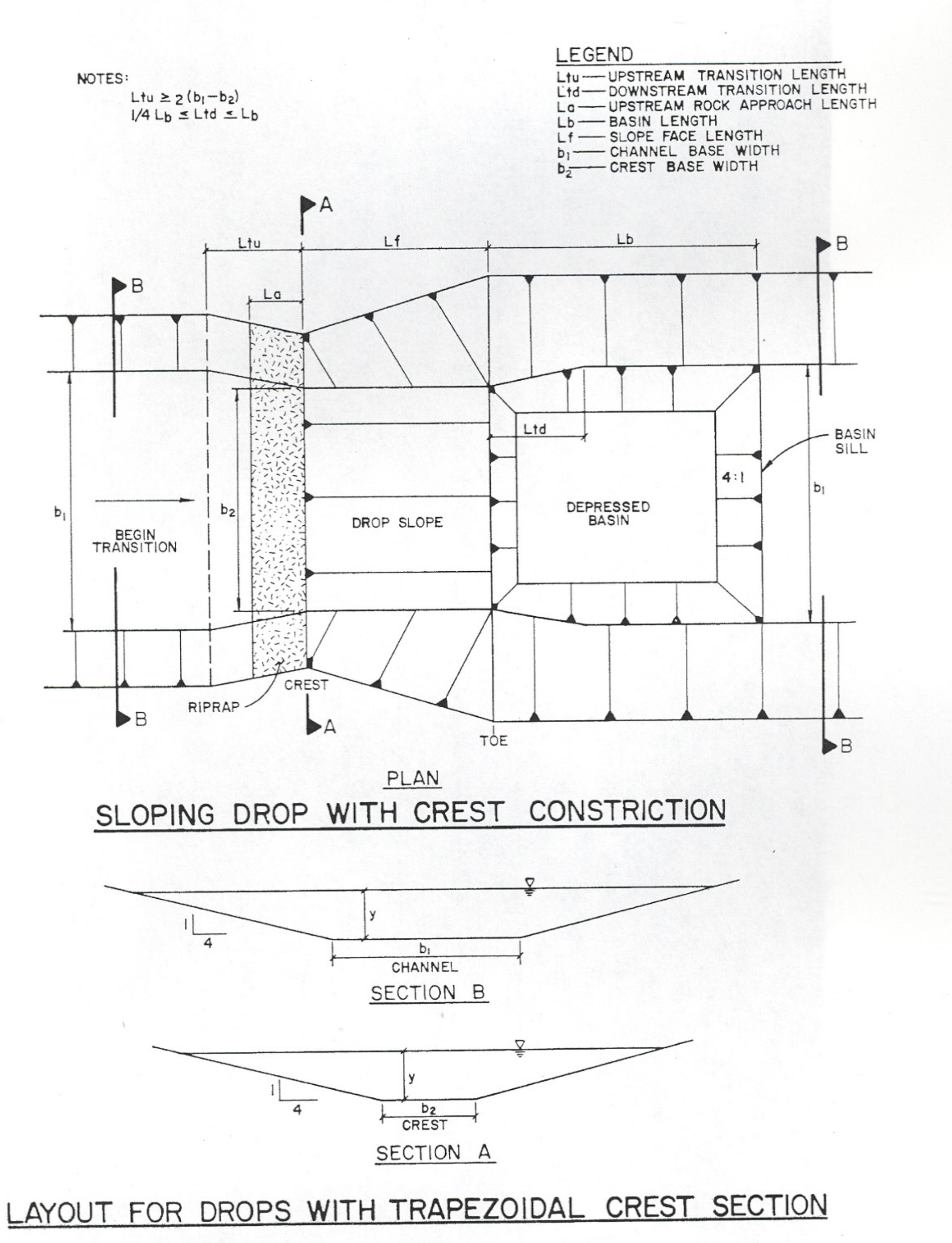


Sloping Abutments at Drop Crest

Figure 8.14 shows a schematic layout for the drop crest and upstream channel at a sloping drop structure. The design objectives discussed previously also apply here. Constricting the trapezoidal crest serves to economize the structure while maintaining upstream freeboard. The seepage cutoff wall is typically placed at or near the upstream end of the transition zone and the zone protected with concrete or grouted rock. This arrangement also provides better seepage control, as discussed later in this section.

Figure 8- 14: Typical Sloping Drop Crest Configuration

(McLaughlin Water Engineers, Ltd. 1986)



8.3.3.3 Water Surface Profile Analysis

Backwater computations should be completed for the channel reaches upstream and downstream of the proposed drop structure to establish approach flow conditions and tailwater conditions for the range of design flows.

The next step is to determine the location of the hydraulic jump so that the stilling basin can be sized to adequately contain the zone of turbulence. The determination of the hydraulic jump's location is usually accomplished through the comparison of the unit specific force for the supercritical inflow and the downstream subcritical flow. For vertical drop structures, this requires analysis of the tailwater elevation to determine if it is sufficient to cause the jump to occur immediately, or if the jet will wash downstream until the specific force is sufficiently reduced to allow the jump to occur. For sloping drop structures, water surfaces must be determined for the supercritical profiles down the face of the drop. The location of the hydraulic jump can be determined by using Equation (8.13) to compute the unit specific force Fs, above and below the toe of the drop. The hydraulic jump, in either the trickle channel or the main drop, will begin to form where the unit specific force of the downstream tailwater is greater than the specific force of the supercritical flow below the drop.

$$F_s = \frac{q^2}{gy} + \frac{y^2}{2} \tag{8.13}$$

The depth y, for downstream specific force determination, is the tailwater surface elevation minus the ground elevation at the point of interest, which is typically the main basin elevation or the trickle channel invert (if the jump is to occur in the basin). The depth for the upstream specific force (supercritical flow) is the supercritical flow depth at the point in question.

For jumps in vertical riprap basins, the user has to rely on the criteria derived from laboratory studies. The shaping or reshaping of riprap influences the jump stability and location. Nevertheless, the basic specific force equation provides some guidance.

Ideally, for economic considerations, the jump should begin no further downstream than the drop toe. This is generally accomplished in the main drop zone by depressing the basin to a depth nearly as low as the downstream trickle channel elevation.

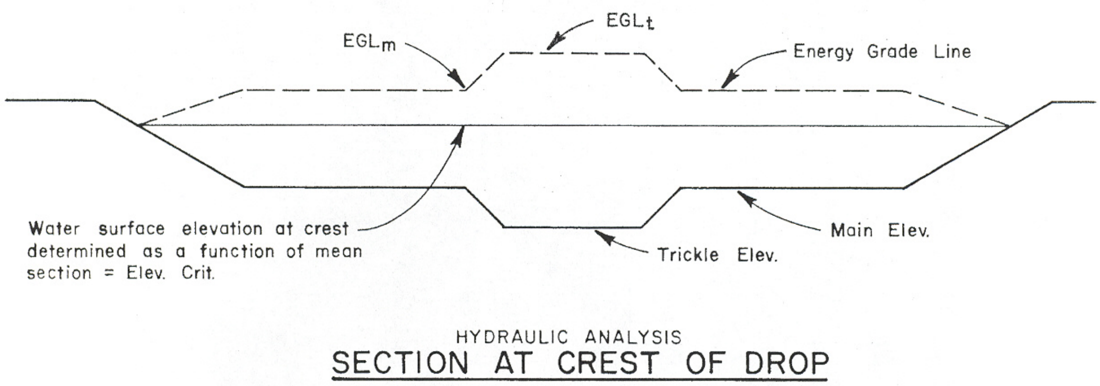
Analysis should be conducted for a range of flows, since flow characteristics at the drop can vary with discharge. For example, the 10-year flow may cascade down the face of a sloping drop and form a jump downstream of the toe, whereas the 100-year flow may totally submerge the drop.

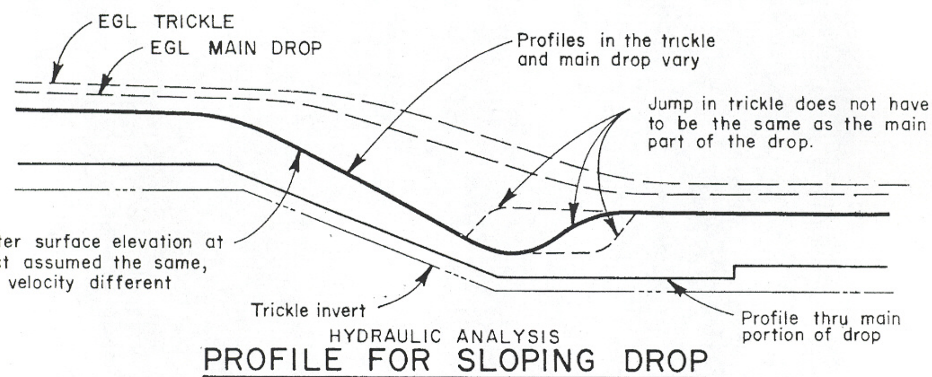
Where a major channel incorporates a low flow channel, separate analyses should be completed for the low flow zone and the major channel overbank zone.

This is because the deeper flow profile in the low flow channel zone has a higher energy grade line profile (Figure 8.15). Specific force analysis in this zone shows that the hydraulic jump will not occur in the same location as the rest of the flow over the drop, and in most cases the jump will occur further downstream. Separate analysis for this condition will determine if the stilling basin length is sufficient to contain the jump.

Figure 8- 15: Typical Section and Profile for Sloping Drop

(McLaughlin Water Engineers, Ltd. 1986)





8.3.3.4 Hydraulic Jump

With the exception of the baffle chute drop, all of the drop structures described in this chapter use the formation of a hydraulic jump to dissipate energy. A discussion of this hydraulic phenomenon is presented as follows.

A hydraulic jump occurs when flow changes rapidly from low stage supercritical flow to high stage subcritical flow. Hydraulic jumps can occur:

1. when the slope of a channel abruptly changes from steep to mild;

2. at sudden expansions or contractions in the channel section;
3. at locations where a barrier, such as a culvert or bridge, occurs in a channel of steep slope;
4. at the downstream side of dip crossings or culverts; and
5. where a channel of steep slope discharges into other channels.

Hydraulic jumps are useful in dissipating energy, and consequently they are often used at drainageway outlet structures and drop structures as an efficient way to minimize the erosive potential of floodwaters. However, because of the high turbulence associated with hydraulic jumps, they must be contained within a well-protected area. Complete computations must be made to determine the height, length, and other characteristics of the jump (including consideration of a range of flows) in order to adequately size the containment area.

The type of hydraulic jump that forms, and the amount of energy that it dissipates, is dependent upon the upstream Froude number (F_{r1}). The various types of hydraulic jumps that can occur are listed in Table 8.1.

Table 8- 1: Types of Hydraulic Jumps

Upstream Froude Number	Type of Jump	Energy Loss, %
1.0 < Fr1 < 1.7	Undular Jump	0 to 5
1.7 < Fr1 < 2.5	Weak Jump	5 to 18
2.5 < Fr1 < 4.5	Oscillating Jump	18 to 44
4.5 < Fr1 < 9.0	Steady Jump	44 to 70
9.0 < Fr1	Strong Jump	70 to 85

Jump Height

The depth of flow immediately downstream of a hydraulic jump is referred to as the sequent depth (Y_2). The sequent depth in rectangular channels whose upstream Froude number is > 1.7, can be computed by use of the following equation:

$$Y_2 = \frac{1}{2} Y_1 \left[\left(1 + 8 F_{r1}^{\ 2} \right)^{0.5} - 1 \right] \tag{8.14}$$

The solution for sequent depth in trapezoidal channels can be obtained from a trial-and-error solution of Equation (8.15), which is derived from momentum equations. It is also acceptable for design purposes to determine the sequent depth in trapezoidal channels from Equation (8.14). Equation (8.14) is much simpler to solve and produces only slightly greater values for sequent depth than does Equation (8.3). (*Note: check formula 8.15 because it appears to have an error in FCDMC manual*)

$$\frac{ZY_1^{\ 3}}{3} + \frac{ZY_1^{\ 2}}{2} + \frac{Q}{gA_1} = \frac{ZY_2^{\ 3}}{3} + \frac{bY_2^{\ 2}}{2} + \frac{Q}{gA_2} \tag{8.15}$$

Figure 8.16 and Figure 8.18 provide graphs of hydraulic jumps for a horizontal rectangular channel and a horizontal trapezoidal channel, respectively.

Undular Jump

An undular hydraulic jump is the type of jump which occurs where the upstream Froude number is between 1.0 and 1.7. This type of jump is characterized by a series of undular waves which form on the downstream side of the jump. Experiments have shown that the first wave of an undular jump is higher than the height given by Equation (8.15). Therefore, the height of this wave should be determined as follows:

$$\frac{Y_2 - Y_1}{Y_1} = {F_{r1}}^2 - 1 \qquad (8.16)$$

Jump Length

The length of a hydraulic jump is defined as the distance from the front face of the jump to a point immediately downstream of the roller. Jump length can be determined from Figure 8.17 and Figure 8.19.

Surface Profile

The surface profile of a hydraulic jump may be needed to design the extra bank protection, or training walls for containment of the jump. The surface profile can be determined from Figure 8.20.

Jump Location

In most cases a hydraulic jump will occur at the location in a channel where the initial and sequent depths and initial Froude number satisfy Equation (8.15). This location can be found by performing direct-step calculations in either direction toward the suspected jump location until the terms of the equation are satisfied. Specific force analysis can then be used by employing Equation (8.13) to establish where a jump will occur. The hydraulic jump will begin to form where the unit specific force of the downstream tailwater is greater than the unit force of the supercritical approach flow.

8.3.3.5 Design Charts and Figures

Figure 8.16 to Figure 8.20 and Table 8.2 have been included as additional aids to the user of this manual.

Figure 8- 16: Height for a Hydraulic Jump for a Horizontal Rectangular Channel

(USDOT, FHWA, HEC-14, 1983)

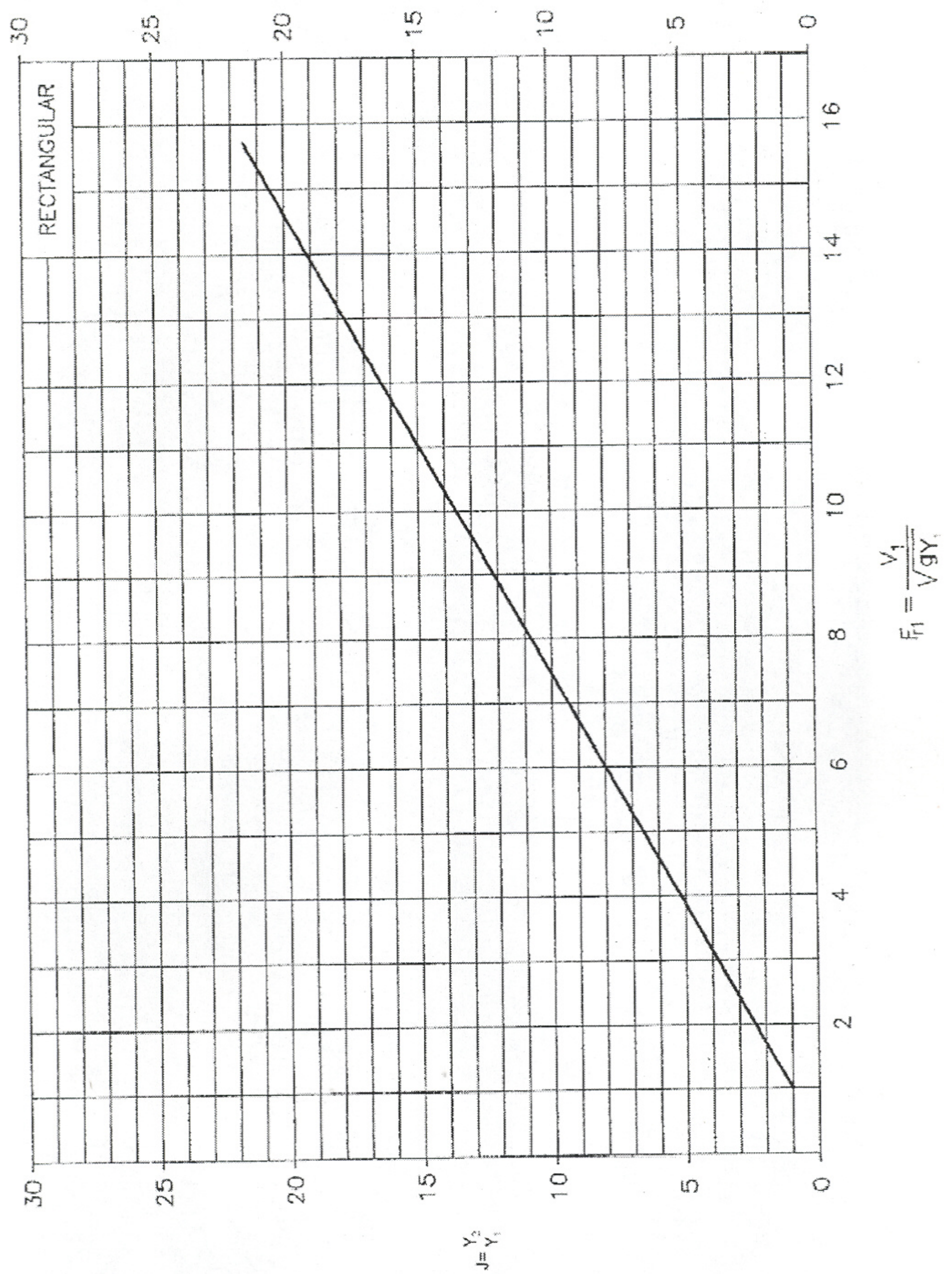


Figure 8- 17: Length of Hydraulic Jump for Rectangular Channels
(USDOT, FHWA, HEC-14, 1983)

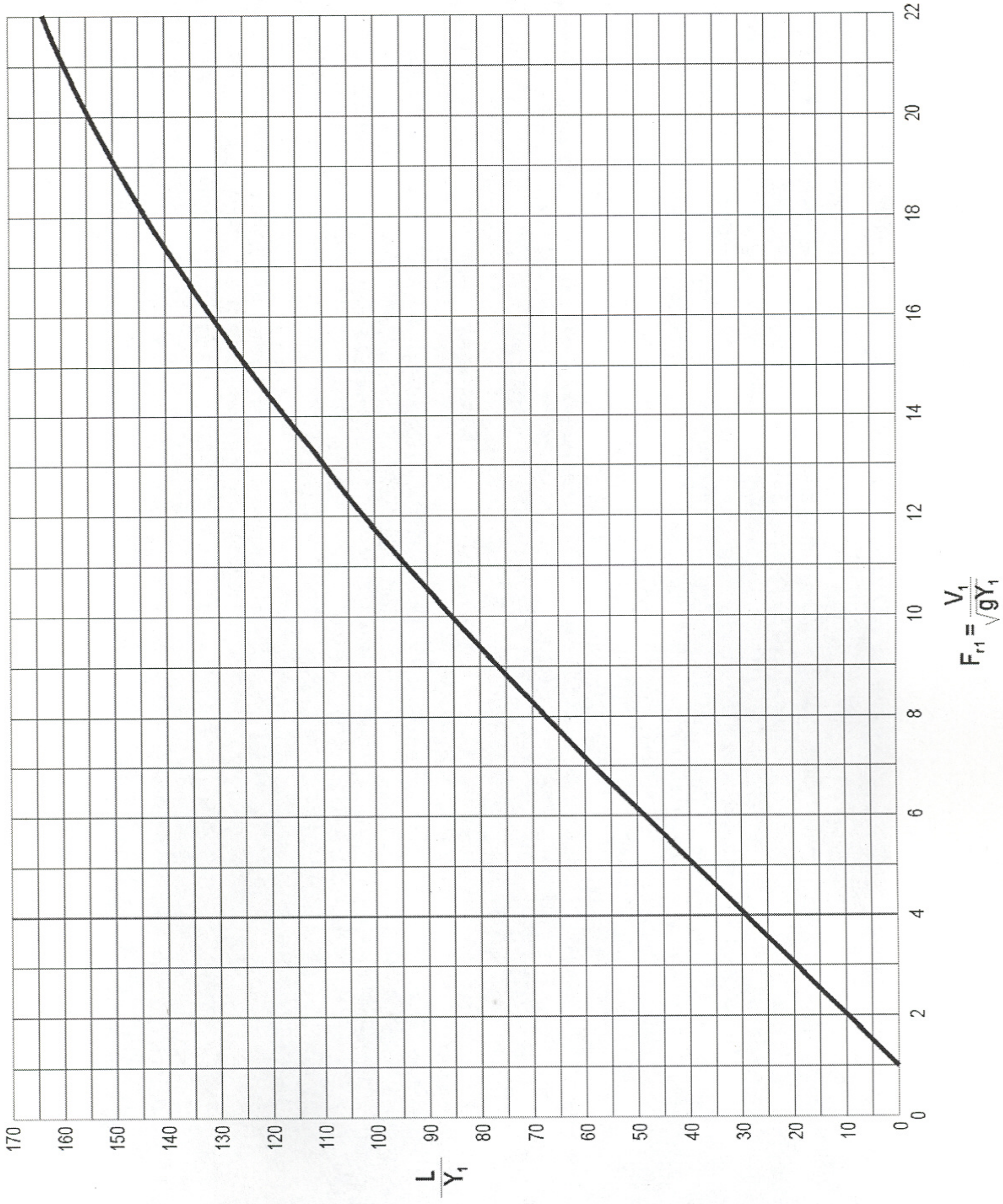


Figure 8- 18: Height of a Hydraulic Jump for a Horizontal Trapezoidal Channel (Using Hydraulic Depth)
(USDOT, FHWA, HEC-14, 1983)

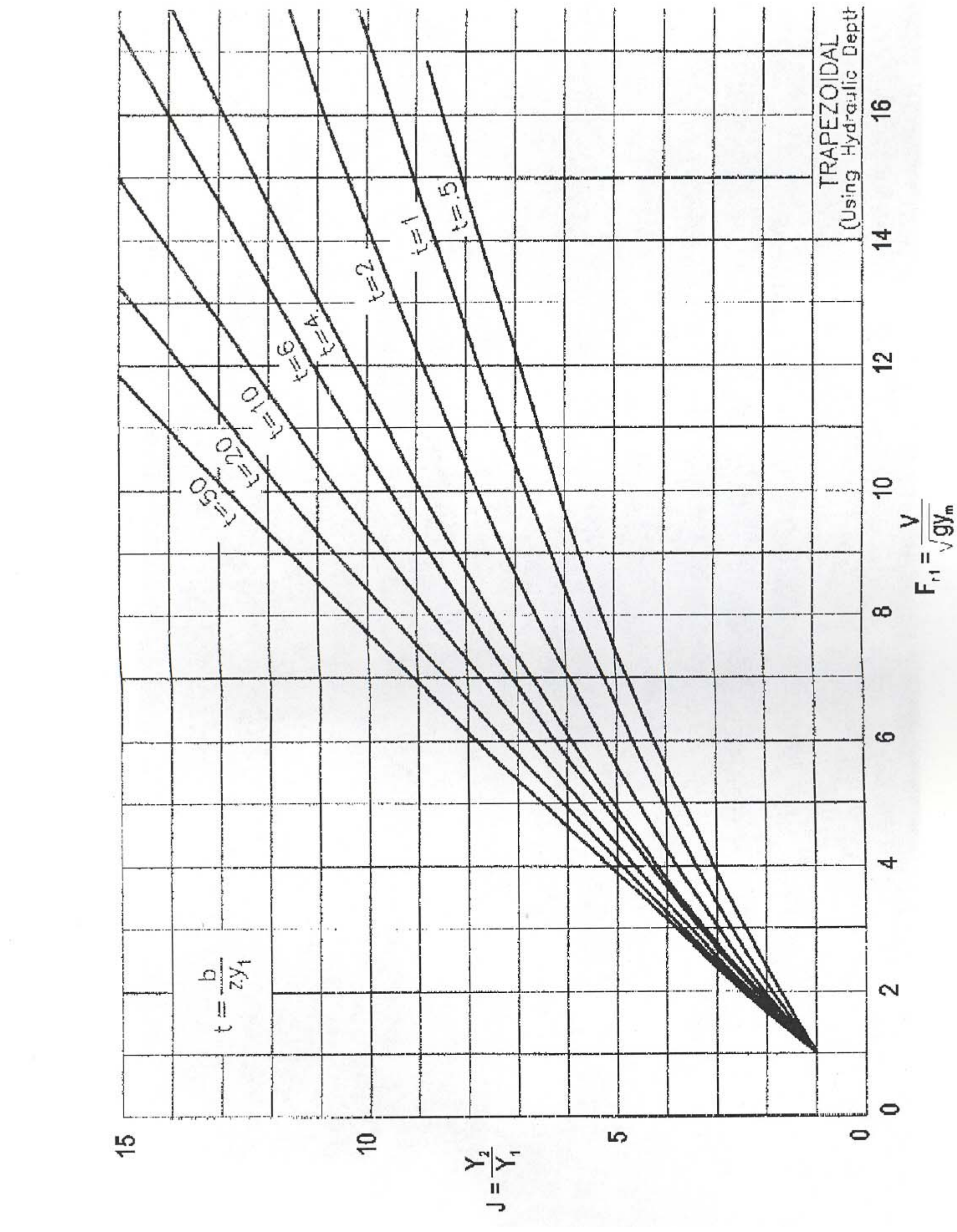


Figure 8- 19: Length of a Hydraulic Jump for non-Rectangular Channels

(USDOT, FHWA, HEC-14, 1983)

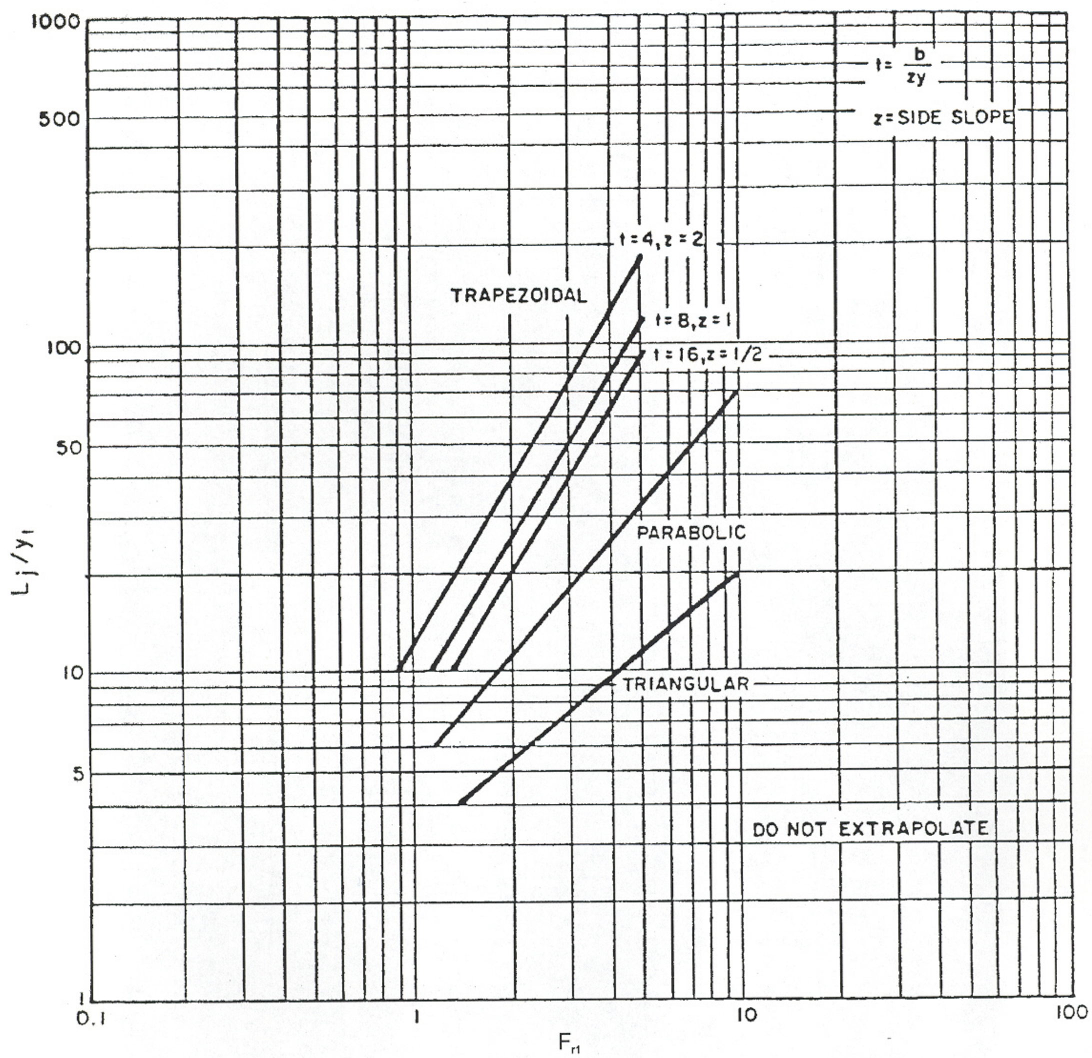


Figure 8- 20: Surface Profile of a Hydraulic Jump in a Horizontal Channel
(Chow, 1959)

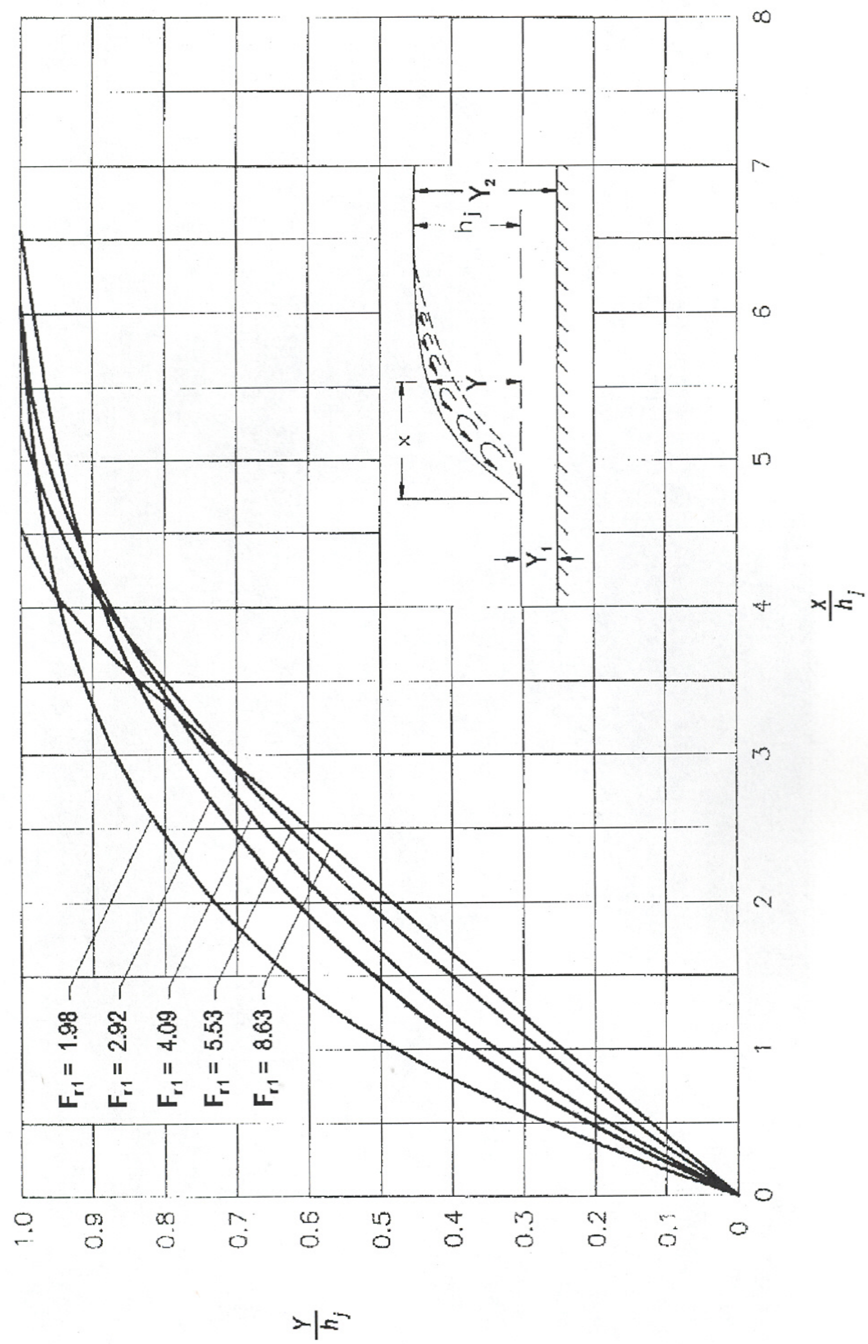


Table 8- 2: Uniform Flow in Circular Sections Flowing Partly Full (USDOT, FHWA, HEC-14, 1983)

d = depth of flow
D = diameter of pipe
A = area of flow
R = hydraulic radius
Q = discharge in cubic feet per second by Manning's formula
n = Manning's coefficient
S = slope of the channel bottom and of the water surface

$\frac{d}{D}$	$\frac{A}{D^2}$	$\frac{R}{D}$	$\frac{Qn}{D^{8/3}S^{1/2}}$	$\frac{Qn}{d^{8/3}S^{1/2}}$	$\frac{d}{D}$	$\frac{A}{D^2}$	$\frac{R}{D}$	$\frac{Qn}{D^{8/3}S^{1/2}}$	$\frac{Qn}{d^{8/3}S^{1/2}}$
0.01	0.0013	0.0066	0.00007	15.04	0.51	0.4027	0.2531	0.239	1.442
0.02	0.0037	0.0132	0.00031	10.57	0.52	0.4127	0.2562	0.247	1.415
0.03	0.0069	0.0197	0.00074	8.56	0.53	0.4227	0.2592	0.255	1.388
0.04	0.0105	0.0262	0.00138	7.38	0.54	0.4327	0.2621	0.263	1.362
0.05	0.0147	0.0325	0.00222	6.55	0.55	0.4426	0.2649	0.271	1.336
0.06	0.0192	0.0389	0.00328	5.95	0.56	0.4526	0.2676	0.279	1.311
0.07	0.0242	0.0451	0.00455	5.47	0.57	0.4625	0.2703	0.287	1.286
0.08	0.0294	0.0513	0.00604	5.09	0.58	0.4724	0.2728	0.295	1.262
0.09	0.0350	0.0575	0.00775	4.76	0.59	0.4822	0.2753	0.303	1.238
0.10	0.0409	0.0635	0.00967	4.49	0.60	0.4920	0.2776	0.311	1.215
0.11	0.0470	0.0695	0.01181	4.25	0.61	0.5018	0.2799	0.319	1.192
0.12	0.0534	0.0755	0.01417	4.04	0.62	0.5115	0.2821	0.327	1.170
0.13	0.0600	0.0813	0.01674	3.86	0.63	0.5212	0.2842	0.335	1.148
0.14	0.0668	0.0871	0.01952	3.69	0.64	0.5308	0.2862	0.343	1.126
0.15	0.0739	0.0929	0.0225	3.54	0.65	0.5404	0.2882	0.350	1.105
0.16	0.0811	0.0985	0.0257	3.41	0.66	0.5499	0.2900	0.358	1.084
0.17	0.0885	0.1042	0.0291	3.28	0.67	0.5594	0.2917	0.366	1.064
0.18	0.0961	0.1097	0.0327	3.17	0.68	0.5687	0.2933	0.373	1.044
0.19	0.1039	0.1152	0.0365	3.06	0.69	0.5780	0.2948	0.380	1.024
0.20	0.1118	0.1206	0.0406	2.96	0.70	0.5872	0.2962	0.388	1.004
0.21	0.1199	0.1259	0.0448	2.87	0.71	0.5964	0.2975	0.395	0.985
0.22	0.1281	0.1312	0.0492	2.79	0.72	0.6054	0.2987	0.402	0.965
0.23	0.1365	0.1364	0.0537	2.71	0.73	0.6143	0.2998	0.409	0.947
0.24	0.1449	0.1416	0.0585	2.63	0.74	0.6231	0.3008	0.416	0.928
0.25	0.1535	0.1466	0.0634	2.56	0.75	0.6319	0.3017	0.422	0.910
0.26	0.1623	0.1516	0.0686	2.49	0.76	0.6405	0.3024	0.429	0.891
0.27	0.1711	0.1566	0.0739	2.42	0.77	0.6489	0.3031	0.435	0.873
0.28	0.1800	0.1614	0.0793	2.36	0.78	0.6573	0.3036	0.441	0.856
0.29	0.1890	0.1662	0.0849	2.30	0.79	0.6655	0.3039	0.447	0.838
0.30	0.1982	0.1709	0.0907	2.25	0.80	0.6736	0.3042	0.453	0.821
0.31	0.2074	0.1756	0.0966	2.20	0.81	0.6815	0.3043	0.458	0.804
0.32	0.2167	0.1802	0.1027	2.14	0.82	0.6893	0.3043	0.463	0.787
0.33	0.2260	0.1847	0.1089	2.09	0.83	0.6969	0.3041	0.468	0.770
0.34	0.2355	0.1891	0.1153	2.05	0.84	0.7043	0.3038	0.473	0.753
0.35	0.2450	0.1935	0.1218	2.00	0.85	0.7115	0.3033	0.477	0.736
0.36	0.2546	0.1978	0.1284	1.958	0.86	0.7186	0.3026	0.481	0.720
0.37	0.2642	0.2020	0.1351	1.915	0.87	0.7254	0.3018	0.485	0.703
0.38	0.2739	0.2062	0.1420	1.875	0.88	0.7320	0.3007	0.488	0.687
0.39	0.2836	0.2102	0.1490	1.835	0.89	0.7384	0.2995	0.491	0.670
0.40	0.2934	0.2142	0.1561	1.797	0.90	0.7445	0.2980	0.494	0.654
0.41	0.3032	0.2182	0.1633	1.760	0.91	0.7504	0.2963	0.496	0.637
0.42	0.3130	0.2220	0.1705	1.724	0.92	0.7560	0.2944	0.497	0.621
0.43	0.3229	0.2258	0.1779	1.689	0.93	0.7612	0.2921	0.498	0.604
0.44	0.3328	0.2295	0.1854	1.655	0.94	0.7662	0.2895	0.498	0.588
0.45	0.3428	0.2331	0.1929	1.622	0.95	0.7707	0.2865	0.498	0.571
0.46	0.3527	0.2366	0.201	1.590	0.96	0.7749	0.2829	0.496	0.553
0.47	0.3627	0.2401	0.208	1.559	0.97	0.7785	0.2787	0.494	0.535
0.48	0.3727	0.2435	0.216	1.530	0.98	0.7817	0.2735	0.489	0.517
0.49	0.3827	0.2468	0.224	1.500	0.99	0.7841	0.2666	0.483	0.496
0.50	0.3927	0.2500	0.232	1.471	1.00	0.7854	0.2500	0.463	0.463

8.3.3.6 Seepage and Uplift Forces

The most common technique for seepage analysis is that proposed by E. W. Lane (1935), commonly referred to as "Lane's Weighted-Creep Method". The essential elements of this method are paraphrased as follows:

1. The weighted-creep distance of a cross section of a drop structure is the sum of the vertical creep distances (along contact surfaces steeper than 45 degrees), L_V, plus one-third of the horizontal creep distances (along contact surfaces less than 45 degrees), L_H.

2. The weighted-creep head ratio is defined as:

$$C_w = \frac{(L_H + 3L_V)}{3H_{df}} \tag{8.17}$$

3. Lane's recommended weighted-creep ratios are given for various foundation materials in Table 8.3.

4. Reverse filter drains, weep holes, and pipe drains are aids to provide security from seepage, and recommended safe weighted-creep head ratios may be reduced as much as 10 percent, if used.

5. Care must be exercised that cutoff walls extend laterally into each bank so that flow will not outflank them.

6. The upward pressure to be used in design may be estimated by assuming that the drop in pressure from headwater to tailwater along the contact line of the drop structure and cutoff wall is proportional to the weighted-creep distance.

Seepage is controlled by increasing the seepage length such that C_w is lowered to a conservative value. Soils tests must be taken during design and confirmed during construction. These tests are especially critical for reinforced concrete structures.

An example of this technique can be found in Design of Small Dams (USBR, 1987). An alternative approach is to use a flow net or computerized seepage analysis to estimate subsurface flows and uplift pressures under a structure. Seepage considerations should be included in the design of cutoff walls, wall footings, drains, filters, structural slabs, and grouted masses.

Locating a seepage cutoff wall upstream of the crest of a drop structure and using horizontal impervious blankets can be effective. It is also very important to control lateral seepage around the structure.

Table 8- 3: Lane's Weighted-Creep: Recommended Ratios

Material	CW Ratio
Very fine sand or silt	8.5
Fine sand	7.0
Medium sand	6.0
Coarse sand	5.0
Fine gravel	4.0
Medium gravel	3.5
Coarse gravel including cobbles	3.0
Boulders with some cobbles and gravel	2.5
Soft clay	3.0
Medium clay	2.0
Hard clay	1.8
Very hard clay or hardpan	1.6

8.3.4 Selection Considerations

There are four major considerations for the selection of the type of drop structure for a particular application:

1. surface flow hydraulic performance;
2. foundation and seepage control;
3. economic considerations; and
4. construction considerations.

Other factors which can affect selection are land uses, aesthetics, safety, maintenance, and anticipated downstream channel degradation.

8.3.4.1 Hydraulic Performance

The primary consideration for the selection of a drop structure should be functional hydraulic performance. The surface flow hydraulic system combines channel approach and crest hydraulics, sloping or vertical drop hydraulics and downstream tailwater conditions. Hydraulic analysis procedures are presented in Section 8.3.3.

8.3.4.2 Foundation and Seepage Control

Table 8.4 presents some typical foundation conditions and control systems typically used for various drop heights. Table 8.4 is presented only as a guide. The hydraulic engineer must calculate hydraulic loadings which can occur for a variety of conditions such as interim construction conditions, low flow, and flood flow. The soils/foundation engineer couples this information with the onsite soils information. Both work with a structural engineer to establish final loading diagrams, and selection and sizing of structural components. This section presents information relevant to hydraulics, but refer to geotechnical and structural books for related information.

Table 8- 4: General Seepage Cutoff Technique Suitability

Soil Conditions	Drop Height, feet			
	2	4	8	12
Sand and gravel over bedrock with sufficient depth of material to provide support - groundwater prevalent.	S*	S*	S/S_wB*	S/S_wB*
	CT_c	CT_c/ST	ST	ST
	CT_f	CT_f/CTI		
Sand and gravel with shallow depth to bedrock - groundwater prevalent.	CTc	CTc/ST	ST	ST
	CW	CW	CW	CW
	S**	S**	S**	SwB**
Sand and Gravel, great depths to bedrock - groundwater prevalent	S	S	S	S/SwB
	CTc	CTc/ST	ST	ST
Sand and gravel, no groundwater, or water table normally below requirement (for variation caused by depth to bedrock, see first case)	S	S	S	S/SwB
	CTf/CTI	CTI	CTI	CTI
	CW	CW		
Clay (and silt) - medium to hard	CTc	CTc	CTc	CTc
	CW	Reduce length for difficult backfill conditions		
	CTf/CTI	Only for local seepage zones/silts		
	ST	Expensive - for special problems		
Clay (and silt) - soft to medium with lenses of permeable material - groundwater present	S	S	S	S/SwB
	CTc	CTc	CTc/ST	ST
Clay (and silt) - soft to medium with lenses of permeable material-may be moist but not significant groundwater source	S	S	S	S/SwB
	CTc	CTc	CTc/ST	ST
	CTf	CTI	CTI	CTI
	CW	CW	CW	CW

*(Consider Scour in sheet pile support)
**(excavate onto bedrock and set into concrete)

Legend

S = Sheet pile
SwB = Sheet pile with bracing and extra measures
CTc = Cutoff Trench backfilled with concrete
ST = Slurry Trench; similar to CTc; but trench walls are supported with slurry and then later replaced with concrete or additives that effect cutoff
CW = Cutoff Wall; conventional wall, possibly with footer, backfilled; note that the effective seepage length should generally be decreased because of backfill
CTI = Cutoff Trench with synthetic liner and fill
CTf = Cutoff Trench with clay fill

8.3.4.3 Economic Considerations

Evaluation of alternative drop structure costs should include consideration of construction costs and maintenance costs. Construction costs include site work specific to the structure, seepage control, excavation, reinforced concrete, riprap, boulders, grout and backfill. Maintenance costs include rock replacement, debris removal, erosion repair, structural repairs, graffiti and silt removal. A standard method of cost comparison is present worth analysis by which estimated maintenance costs are converted to present worth amounts by applying an appropriate discount rate factor. The present worth maintenance cost is then added to the construction cost of each structure under consideration for comparison.

Other factors also affect the economics of alternative types of drop structures. In many cases, specific site requirements may dictate the direction of drop structure design. Depending on location, some construction materials, such as riprap or boulders, may not be readily available at reasonable cost. Analysis may include consideration of the cost of a single drop structure of height (H_d) versus the cost of two structures, each 1/2 H_d high.

8.3.4.4 Construction Considerations

The selection of a drop and its foundation may also be tempered by construction difficulty, location, access, and material availability/delivery. Table 8.5, shown on following pages, lists construction considerations for key drop structure materials. Additional discussion of construction concerns is included with the design guidelines for each drop type in the following section.

Table 8- 5: Quality Control Structures and Concerns of Drop Structure Components

Type	Quality Concerns	Quality Control Measures and Inspection
Concrete	The major concern is strength and ability to resist weathering. Aggregate strength and durability are important. Special architectural treatments include exposed aggregate, form liners and color additives	Preconstruction items include review of shop drawings for reinforcing steel, formwork patterns and ties, concrete design mix and related tests, color additives or coatings and architectural treatments such as form liners, handrails and fences. Any architectural test samples should be completed and approved, along with all coatings, weather protection or other items which could affect appearance.
Reinforcing Steel	Usually not a problem unless the wrong grade of steel is brought to job, or site conditions are conducive to corrosion problems. Epoxy coated reinforcement can be specified for critical conditions.	During construction there are numerous items which require checking, including: rebar placement, formwork, tie placement, weep holes and drains, form release coatings and form cleaning before concrete placement, form removal, concrete placement and testing, weather protection, sealants, tie hole treatment, concrete finish work, and earthwork, especially that related to seepage control.
Riprap and Rock	Hardness is of concern because the rock is subject to rough handling and impact forces. Durability concerns are: Oxidation, weathering (freeze thaw tests), and leaching or dissolving by water. Fracturing, which leads to odd or undesirable shapes, is to be avoided. Seams or other discontinuities can lead to breakup or undesirable shapes and damage during handling. Geologic type is important; sedimentary rocks are undesirable. Volcanic rock often has low density. Density of the rock requires specific gravity tests	A significant effort is needed in the area of rock quality control. Submittals should be required from suppliers to document quality. Rock should be durable, sound, and free of seams or fractures. The specific gravity should be a minimum of 2.40. Specifications should include requirements for orderly procedures and appropriate equipment, both for rock and grout placement. Gradation, durability and specific gravity tests of riprap at the quarry are needed, and should only be waived for small projects where the quarry can demonstrate recent tests. Handling that results in excessive breakage should result in changed methods and/or reexamination of rock quality. Subgrades should be dewatered and stabilized. Filters and bedding layers should be reviewed for compatibility to the on-site soil conditions. Rock handling and placement is critical. Riprap should be handled selectively so that the gradation is reestablished through any given vertical section. Areas where the thick-ness is comprised of all materials smaller than the d_{50}, or where excessive voids or radical surface variations occur should be reworked. Good placement techniques should result in a riprap layer with surface materials d_{50} size or greater, closely spaced with voids thoroughly chinked and locked between larger rock, top surfaces generally parallel to the plane of the overall riprap bank or surface, and no great departures in surface elevation from rock to rock. Graded riprap should not be used for grouting, as the smaller rock can prevent full penetration of the grout to the subgrade and can cause incomplete filling of the voids. Large rock or boulders should be placed with a gradall or multi-prong grapple device for ease of handling and to minimize disturbance of the subgrade. A minimum dimension should be specified for the rock to aid field inspection. On slopes, uphill boulders should be keyed in below the tops of downhill boulders for stability. A "stairstep" arrangement where the top surface of the rock is flat and horizontal is preferable for both aesthetic and hydraulic reasons.

Table 8- 5 (cont'd): Quality Control Structures and Concerns of Drop Structure Components

Grout	Cement content and type, aggregate and water content are important considerations for strength and durability. Synthetic fibers can be added to the concrete mix, to provide additional crack control and durability.	The key to success with grouting is to use rock that is no smaller in any dimension than the desired grout thickness (so that one can fully access and fill the voids), to pump and place the grout using a grout pumper with a nozzle that can penetrate to the subgrade, to vibrate using a "pencil vibrator" to assure complete filling of the voids, to have good control of the grout mix (too wet creates shrinkage cracks and stability problems on slope, too dry leads to poor penetration), and to place the grout to the desired thickness. A minimum grout thickness is needed to counteract uplift forces. However, placing too much is unattractive and reduces the roughness of the drop which is needed to prevent the jump from washing downstream. During grouting, it is important to protect the weep drains. With care, one can avoid getting grout on the top of the rock. Any spillage should be washed off immediately. A wood float leaves a smooth finish, and the "pencil vibrator", which is preferred, will generally leave a satisfactory appearance with some touch-up. Full time inspection is required during grouting, as is periodic inspection during the rock placement depending upon the performance of the contractor and the aesthetic appearance desired.
Sheetpile	Sheetpile comes in many configurations and, in particular, joint details. It requires geotechnical, structural and hydraulic expertise, as well as pile driving experience during construction.	Inspection is required to ensure that piling is driven to the design depth, or keyed into bedrock if required. Underground obstructions can create problems with driving. If piling becomes separated at the joints during installation, excessive subsurface flow can result.
Roller Compacted Concrete	Construction equipment limitations constrain drop structure dimensions.	The exposed horizontal portion of the step should be six feet at a minimum with the overall lift width at least nine feet. The designer should coordinate with prospective contractors during the design of the structure.
Soil Cement	Construction equipment limitations constrain drop structure dimensions.	The exposed horizontal portion of the step should be six feet at a minimum with the overall lift width at least nine feet. The designer should coordinate with prospective contractors during the design of the structure.
Synthetic Liners	Liners must be flexible and strong enough to allow adjustment to the actual subgrade, and to allow rock placement without significant damage to the liner material.	Subgrade must be well prepared to minimize voids and piping along the smooth surface of the liner. Certificates of conformance to the technical specifications should be provided by the manufacturer. Liners should be spliced only when necessary and placed in accordance with manufacturer's instructions.
Seepage Cut-off Soils	Important considerations are: classification and homogeneity of clay soils, placement and compaction techniques.	The subgrade should be inspected and sloped to achieve compaction of the cutoff soils and the adjacent subgrade. In order to use this type of drop structure, the subgrade soil needs to be a clay (CL), as classified by a qualified soils engineer.
Drains	Permeability and gradation of media, reverse filter characteristics and compatibility with in situ materials, pipe and other hydraulic components.	Gradation analysis of in situ materials and proposed filter media are advisable. Fabric materials should be used with caution to insure that plugging will not occur. Piping and valving components should comply with specifications and be double checked for suitability for the particular application. The toe drain and other drains should be placed and protected from contamination, particularly if grout or concrete is placed later.

Table 8- 5 (cont'd): Quality Control Structures and Concerns of Drop Structure Components

Cutoffs using Slurry Trench	The homogeneity and stability of the slurry cutoff is critical. The construction techniques to achieve a cutoff to the desired depth and width are also critical.	Practically, cutoffs using slurry trench techniques are more exotic applications and require intensive geotechnical engineering and custom specifications for individual applications. Measures can involve intensive soil testing, density testing of slurry mixtures, tests related to special chemicals and admixtures, and standard concrete and grout testing methods. Besides inspections related to all of the above, site environmental controls are required for slurry mixing and placement, and for disposal of materials displaced during the process.
Architectural and Landscape Items	Coatings are always subject to quality concerns, which are compounded by substrate conditions. Plantings are subject to a wide variety of quality and size.	Landscape and architectural treatments can make a big difference in appearance; take care to work with experienced professionals.

8.4 ENERGY DISSIPATION STRUCTURES

Concrete energy dissipation or stilling basin structures are required to prevent scour damages caused by high exit velocities and flow expansion turbulence at conduit outlets. Outlet structures can provide a high degree of energy dissipation and are generally effective even with relatively low tailwater control. Rock protection at conduit outlets is appropriate where moderate outlet conditions exist; however, there are many situations where rock basins are impractical even at low to moderate flow conditions. Concrete outlet structures can be designed easily and are suitable for a wide variety of site conditions. In some cases, they are more economical than large rock basins, particularly where long term costs are considered.

8.4.1 Riprap Protection at Outlets

8.4.1.1 Operating Characteristics

A stilling basin constructed of loose, graded riprap can be an effective and economical energy dissipation measure for a conduit outlet. Hydraulic Design of Energy Dissipators for Culverts and Channels (USDOT, 1983), contains a design procedure for riprap energy dissipators based on studies conducted at Colorado State University and sponsored by the Wyoming Highway Department. The following conclusions were drawn from an analysis of the experimental data and observed operating characteristics.

- The depth (h_s), length (L_s), and width (W_s) of the scour hole were related to the characteristic size of riprap (d_{50}), discharge (Q), brink depth (Y_o), and tailwater depth (TW).

- The dimensions of a scour hole in a basin constructed with angular rock were approximately the same as those of a scour hole in a basin constructed of rounded material when rock size and other variables were similar.

- When the ratio of tailwater depth to brink depth (TW/Y_o) was less than 0.75 and the ratio of scour depth to size of riprap (h_s/d_{50}) was greater than 2.0, the scour hole functioned very efficiently as an energy dissipator. As the concentrated flow at the culvert brink plunged into the hole, a jump formed against the downstream extremity of the scour hole, and the flow was generally well dispersed as it left the basin.

- The mound of material which formed on the bed downstream of the scour hole contributed to the dissipation of energy and reduced the size of the scour hole; i.e., if the mound from a stable scoured basin was removed and the basin was again subjected to design flow, the scour hole enlarged somewhat.

- For high tailwater basins (TW/Y_o> greater than 0.75) the high velocity core of water emerging from the culvert retained its jet-like character as it passed through the basin, and diffused in a manner very similar to that of a concentrated jet diffusing in a large body of water. As a result, the scour hole was much shallower and generally longer. Consequently, riprap may be required for the channel downstream of the rock-lined basin.

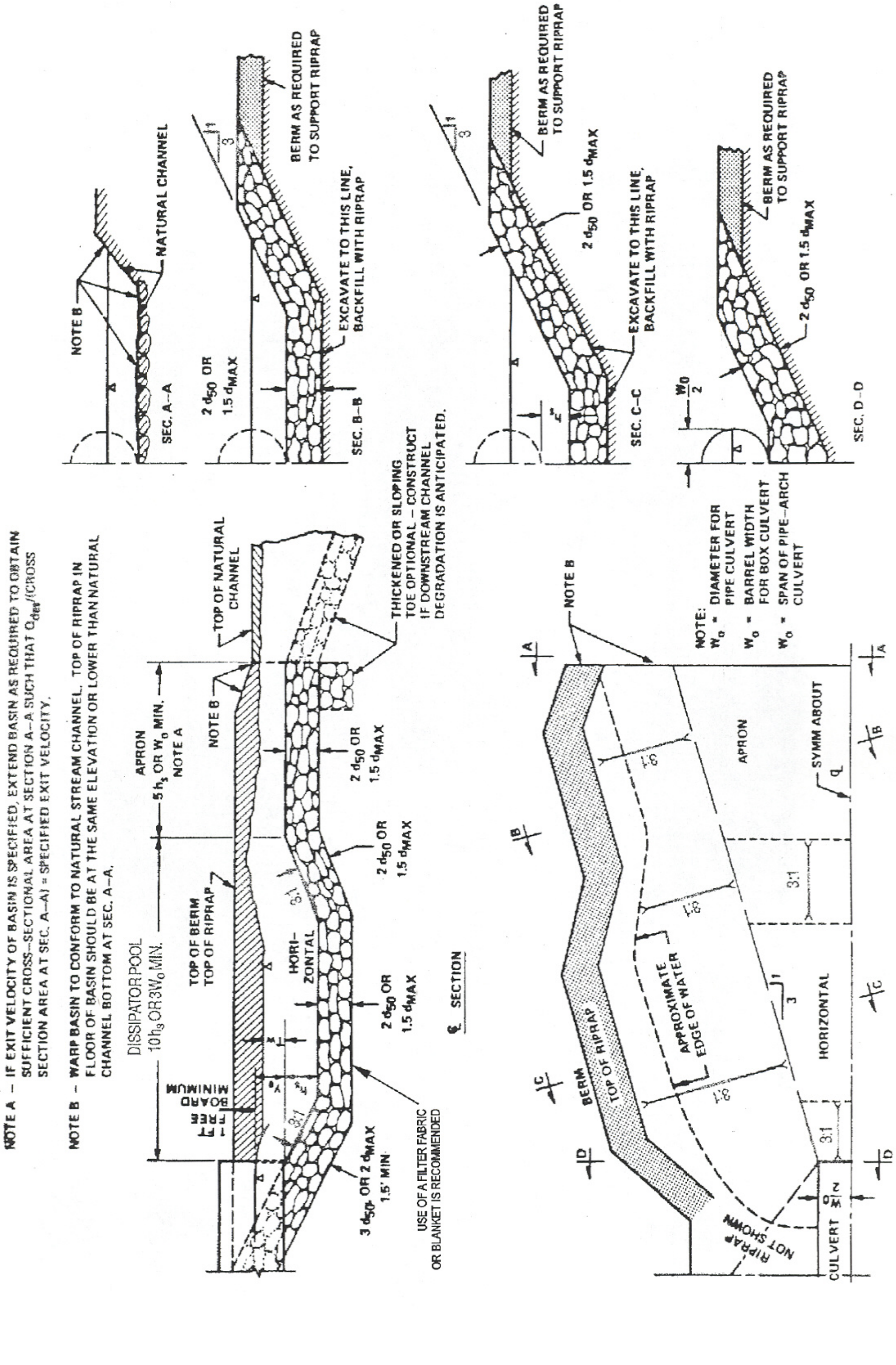
General details of the basin are shown on following page in Figure 8.21, and the principal features are:

- The basin is preshaped and lined with riprap of median size d_{50}.

- The surface of the riprapped floor of the energy dissipating pool is constructed at an elevation, h_s, below the culvert invert. Elevation h_s is the approximate depth of scour that would occur in a thick pad of riprap of size d_{50} constructed at the outfall of the culvert 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, L_s, is $10h_s$ or $3W_o$ whichever is larger. The overall length of the basin, L_b, is $15h_s$ or $4W_o$ whichever is larger.

Figure 8- 21: Detail od Riprapped Culvert Energy Basin
(USDOT, FHWA, HEC-14, 1983)



8.4.1.2 Hydraulic Design Procedure

1. Estimate the flow properties at the brink of the culvert. Establish the brink invert elevation such that $TW/Y_o < 0.75$ for design discharge.

2. For subcritical flow conditions (culvert set on mild or horizontal slope), use Figure 8.22 or Figure 8.23 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.

3. From site inspection and from field experience in the area, determine whether or not channel protection is required at the culvert outlet.

4. If the channel protection is required, compute the Froude number for brink conditions ($y_e= (A/2)^{½}$ for nonrectangular culverts). 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 8.24, and check to see that $2 < h_s/d_{50} < 4$. Recycle computations if h_s/d_{50} falls out of this range.

5. Size basin as shown in Figure 8.21.

6. Design procedures where allowable dissipator exit velocity is specified:

    - Determine the average normal flow depth in the natural channel for the design discharge.

    - Extend the length of the energy basin (if necessary) so that the width of the energy basin (at Section A-A, Figure 8.21), times the average normal flow depth in the natural channel is approximately equal to the design discharge divided by the specified exit velocity.

7. In the exit region of the basin, the walls and apron of the basin should be warped (or transitioned) so that 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.

8. If high tailwater is a possibility and erosion protection is necessary for the downstream channel, it is recommended to 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 8.25. Shape the downstream channel and size riprap using guidelines presented in Chapter 5 and the stream velocities obtained above.

Figure 8- 22: Dimensionless Rating Curves for the Outlets of Rectangular Culverts on Horizontal and Mild Slopes
(Simons, et al, 1970)

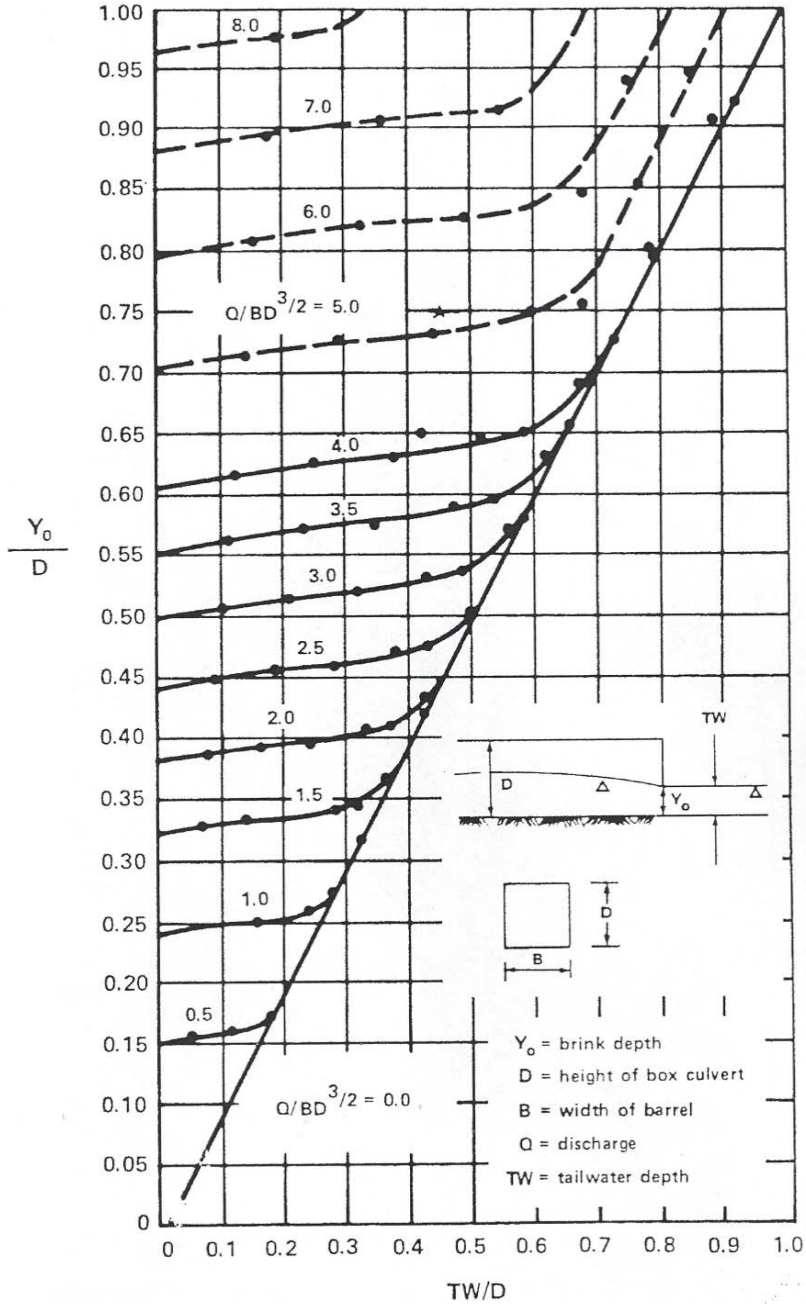


Figure 8- 23: Dimensionless Rating Curves for the Outlets of Circular Culverts on Horizontal and Mild Slopes
(Simons, et al, 1970)

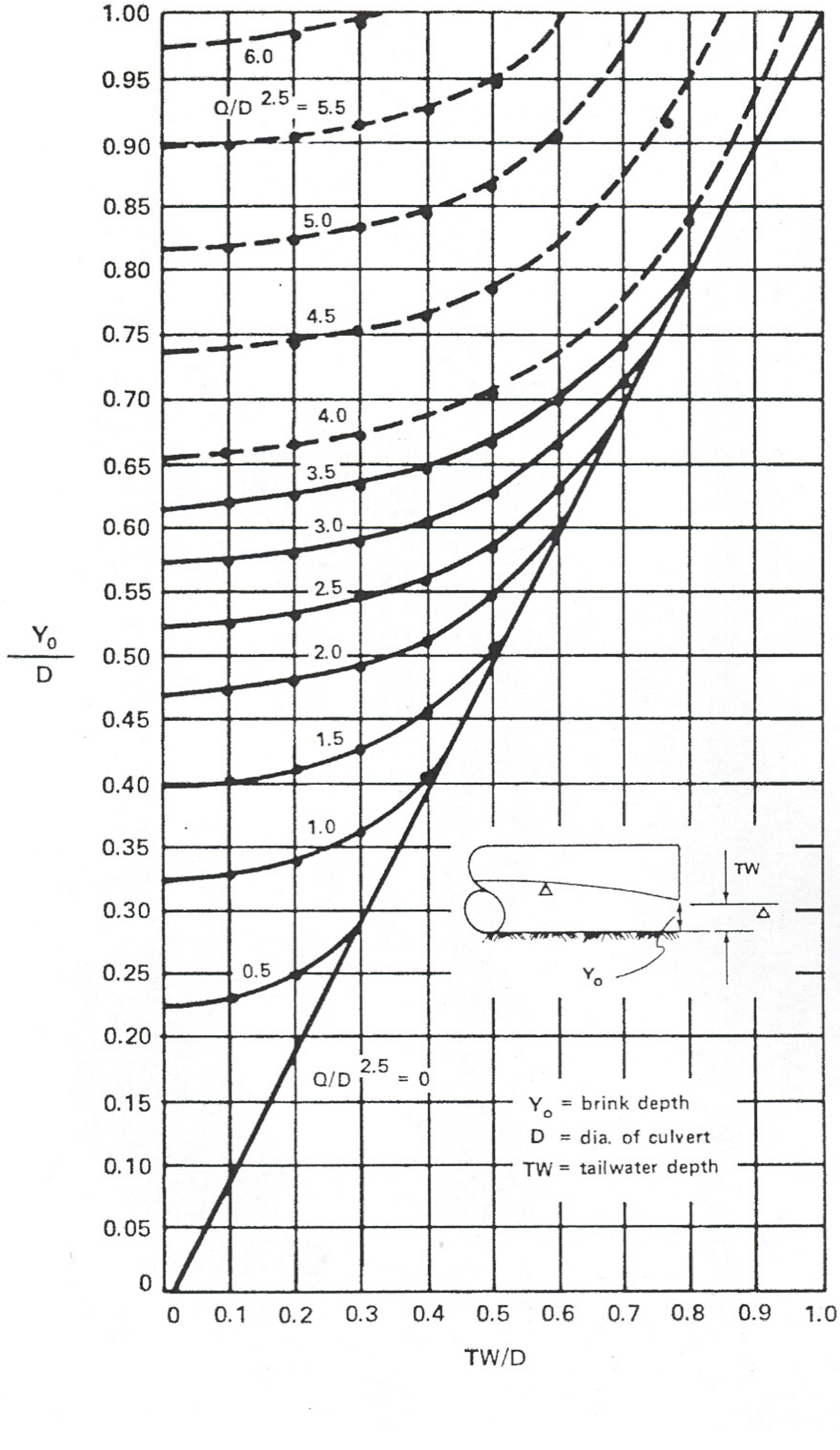


Figure 8- 24: Relative Depth of Scour Hole Versus Froude Number at Brink of Culvert with Relative Size of Riprap as a Third Variable
(USDOT, FHWA, HEC-14, 1983)

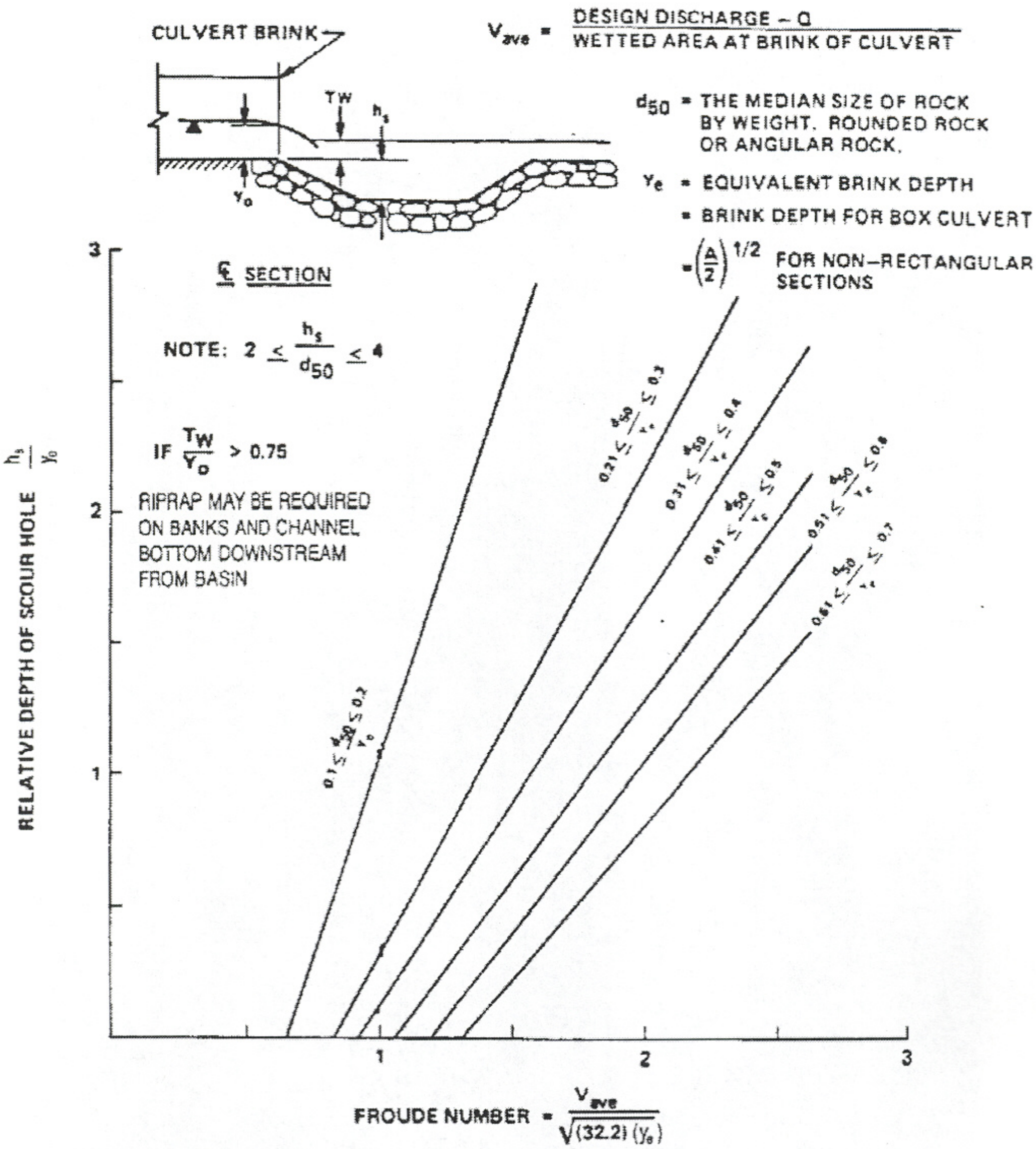
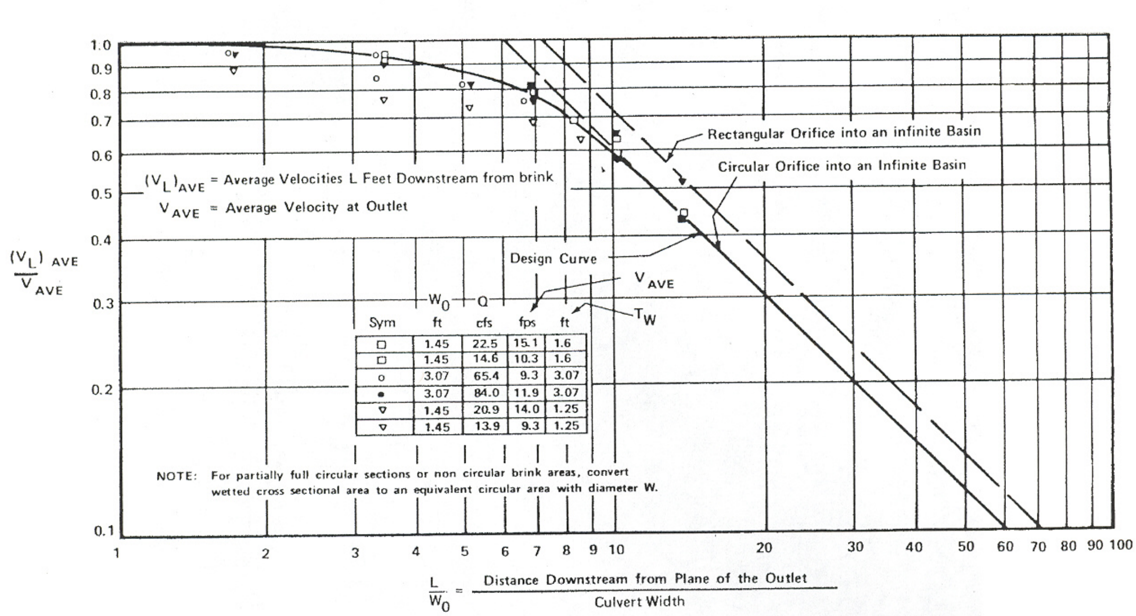


Figure 8- 25: Distribution of Centerline Velocity for Flow from Submerged Outlets

(Use for predicting channel velocities downstream from culvert outlets where high tailwater prevails)
(Simons, et al, 1970; and USDOT, FHWA, HEC-14, 1983)



Sym	W0 ft	Q cfs	VAVE fps	TW ft
□	1.45	22.5	15.1	1.6
□	1.45	14.6	10.3	1.6
o	3.07	65.4	9.3	3.07
•	3.07	84.0	11.9	3.07
▽	1.45	20.9	14.0	1.25
▽	1.45	13.9	9.3	1.25

Additional information regarding design of riprap basins for conduit outlets may be found in Hydraulic Design of Energy Dissipators for Culverts and Channels (USDOT, 1983).

8.4.2 Concrete Protection at Outlets

This section provides hydraulic concepts and design criteria for an impact stilling basin and adaptation of a baffled apron to conduit outlets. Initial design selection should include at least the following aspects concerning conduit outlet structures.

1. High energy dissipation efficiency is required - hydraulic conditions exceed the limits for alternate designs (such as riprap outlet protection).

2. Low tailwater control is anticipated. For example, at outfalls to detention/retention facilities which are empty or have low water levels.

3. Use of concrete is more economical due to structure size or local availability of materials.

4. Site conditions direct the use of an outlet structure such as public use areas where plunge pools and standing water are unacceptable or locations with severe space limitations.

8.4.2.1 Impact Stilling Basin

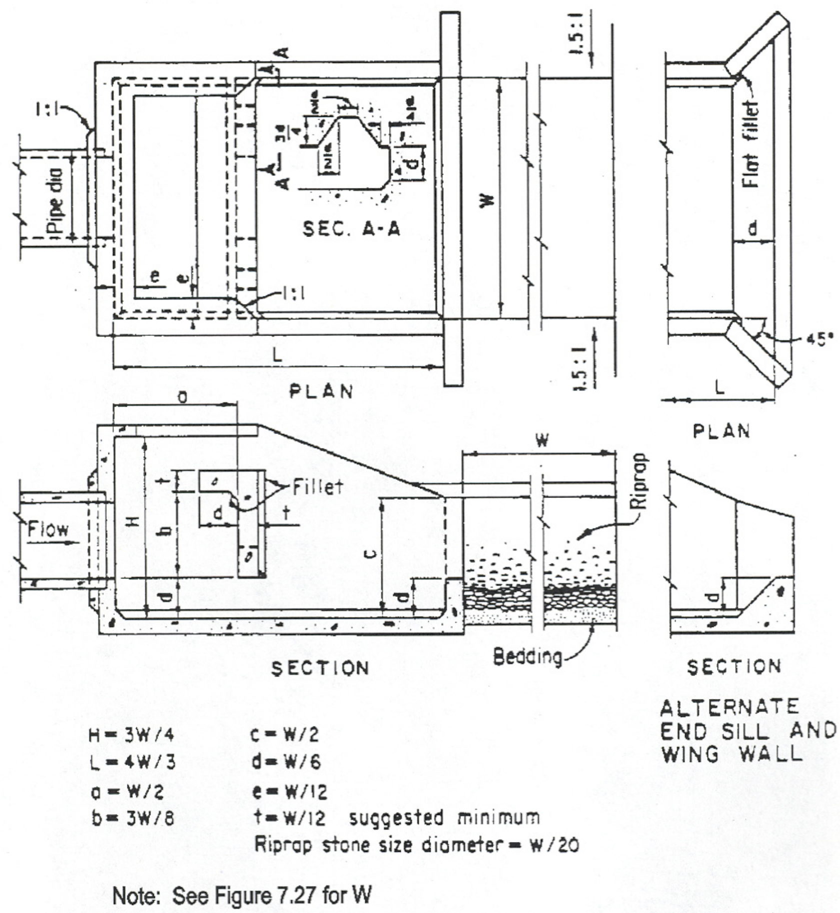
Design standards are based on the USBR Type VI Basin, commonly referred to as an impact dissipator or conduit outlet stilling basin. The Type VI Basin is a relatively small structure which produces highly efficient energy dissipation characteristics without tailwater control. The original hydraulic design reference is Hydraulic Design of Stilling Basins for Pipe or Channel Outlets (Peterka, 1984). Additional structural details are provided in Design of Small Canal Structures (USBR, 1974).The structure is designed to operate continuously at the design flow rate. Maximum entrance conditions are up to 50 feet per second velocity and Froude number less than 9.0. Conditions exceeding this criterion would be extremely rare in typical urban drainage applications. As a result, the use of this outlet basin is limited only by structural and economic considerations.

Energy dissipation is accomplished through momentum transfer as flow entering the basin impacts a large overhanging baffle. At high flow, further dissipation is produced as water builds up behind the baffle to form a highly turbulent backwater zone. Flow is then redirected under the baffle to the open basin and out to the receiving channel. A sill at the basin end reduces exit velocities by breaking up the flow across the basin floor and improves the stilling action at low to moderate flow rates. A notch is recommended in end sills to provide for low flow drainage.

The generalized design configuration (Figure 8.26) consists of an open concrete box attached directly to the conduit outlet. The side walls are high enough to contain most of the splashing during high flows and slope down to form a transition to the receiving channel. The inlet pipe is vertically aligned with an overhanging L-shaped baffle such that the pipe invert is not lower than the bottom of the baffle. The end sill height is equal to the height under the baffle to produce tailwater in the basin. The alternate and transition (at 45 degrees) is recommended for grass lined channels to reduce the overall scour potential just downstream of the sill.

The standard USBR design has been modified for urban applications to allow drainage of the basin bottom during dry periods. The impact basin can also be adapted to multiple pipe installations. These modifications are discussed following the basic criteria. It should be noted that modifications to the design may affect the hydraulic performance of the structure. Model testing is advised for significant changes to the design.

Figure 8- 26: General Design of the USBR Type VI Impact Stilling Basin
(Adapted from: Peterka 1984)



Hydraulic Design Procedure for Stilling Basins:

1. Determine the design pipe flow rate Q and the effective flow area A at the outlet. For partial flow conditions, refer to the partial flow diagram in Section 8.3. Using the relationship $Q = AV$, determine the flow velocity V at the pipe outlet. Assume depth $D = A^{0.5}$ and compute the Froude number.

2. The entrance pipe should be oriented horizontal at least one pipe diameter equivalent length upstream from the outlet. For pipe slopes greater than 15 degrees, the horizontal length should be a minimum of two pipe diameters.

3. Do not use this type of outlet energy dissipator when exit velocities exceed 50 feet per second or Froude numbers exceed 9.0. These conditions would be extreme and must be considered as special cases. Performance is achieved with a tailwater depth equal to half full flow level in the pipe outlet.

4. Determine the basin width (W) by entering the appropriate Froude number and effective flow depth on Figure 8.27. The remaining dimensions are proportional to the basin width according to the legend in Figure 8.26. Note that the baffle thickness, t, is a suggested minimum. It is not a hydraulic parameter and is not a substitute for structural analysis. The basin width should not be increased since the basin is inherently oversized for less than design flows. Larger basins become less effective as the inflow can pass under the baffle.

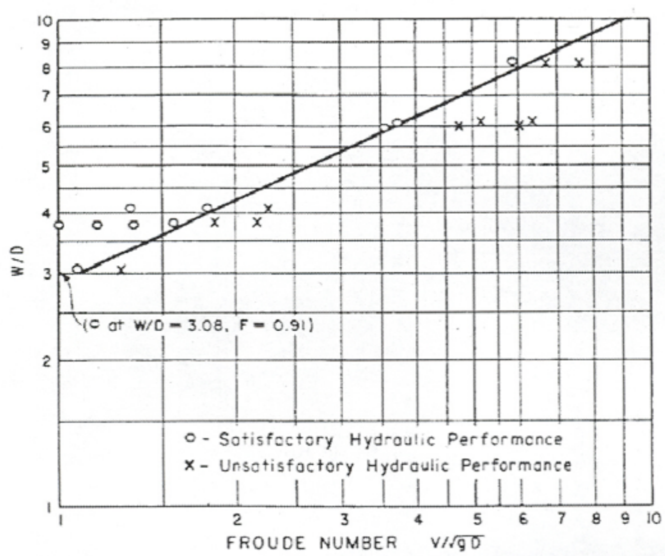
5. Structure wall thickness, steel reinforcement, and anchor walls (underneath the floor) should be designed using accepted structural engineering methods. Hydraulic forces on the overhanging baffle may be approximated by determination of the jet momentum force:

$$F_m = \rho VQ = 1.94VQ \tag{8.18}$$

6. Riprap with a minimum d_{50} of 18 inches should be provided in the receiving channel from the end sill to a minimum distance equal to the basin width. The depth of rock should be equal to the sill height or at least 2.5 feet. Rock may be buried below finished grades and the area vegetated as desired to match the site.

7. The alternate end sill and wingwall shown in Figure 8.26 is recommended for all grass lined channels to reduce the scour potential below the sill wall.

Figure 8- 27: Design Width of the USBR Type VI Basin
(Adapted from: Peterka, 1984)

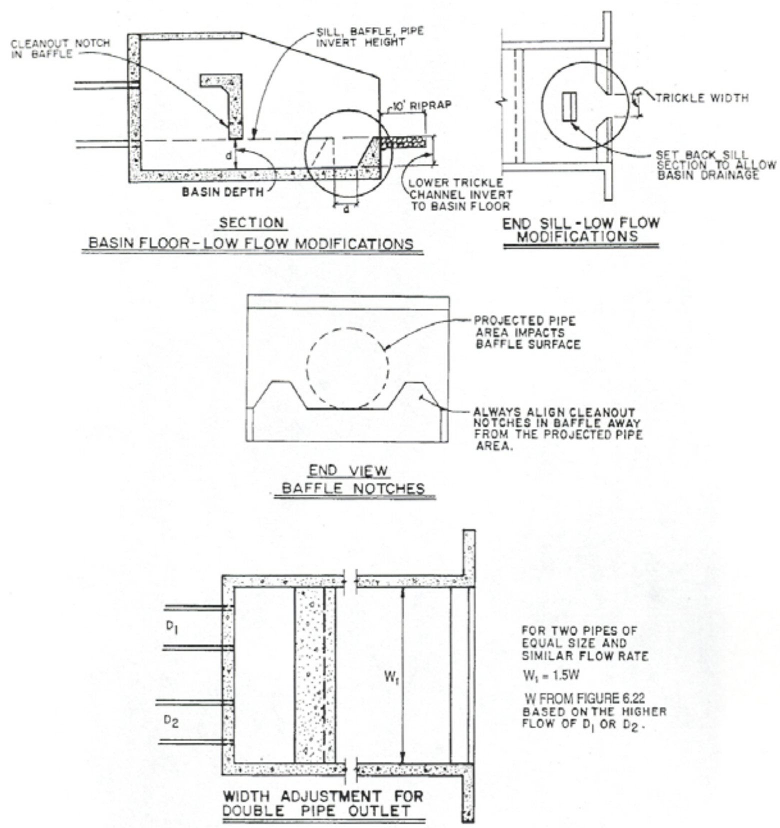


"W" Is the inside width of the basin
"D" Represents the depth of flow entering the basin and is the square root of the area at the conduit outlet
"V" Is the velocity of the incoming flow
The tailwater depth is uncontrolled

Low Flow Modifications

The standard design will retain a standing pool of water in the basin bottom which is generally undesirable from a safety and maintenance standpoint. This situation should be alleviated where practical by matching the receiving channel low flow depth to the basin depth, see Figure 8.28.

Figure 8- 28: Modifiactions to Impact Stilling Basin
(To allow basin drainage for urban applications)
(McLaughlin Water Engineers, Ltd., 1986)



A low flow gap is extended through the basin end sill wall. The gap in the sill should be as narrow as possible to minimize effects on the sill hydraulics. This implies that a narrow and deeper (1.5- to 2-foot) low flow channel will work better than a wider gap section. The low flow width should not exceed 60 percent of the pipe diameter to prevent the jet from short-circuiting through the cleanout notches.

Low flow modifications have not been fully tested to date. Caution is advised to avoid compromising the overall hydraulic performance of the structure. Other ideas are possible, including locating the low flow gap at one side (off center) to prevent a high velocity jet from flowing from the pipe straight down the low flow channel.

The optimal configuration results in continuous drainage of the basin area and helps to reduce the amount of sediment entrapment.

1. For large basins where the sill height is greater than 2.0 feet, the depth dimension, d, (in Figure 8.26) may be reduced to avoid a secondary drop from the sill to the main channel. The low flow invert thereby matches the floor invert at the basin end and the main channel elevation is equal to the sill. Dimension d should not be reduced by more than one-third and not less than 2 feet. This implies that a deeper low flow channel (1.5 to 2.0 feet) will be advantageous for these installations. Note that dimension d is also reduced at the minimum pipe invert height and at the bottom of the baffle wall.

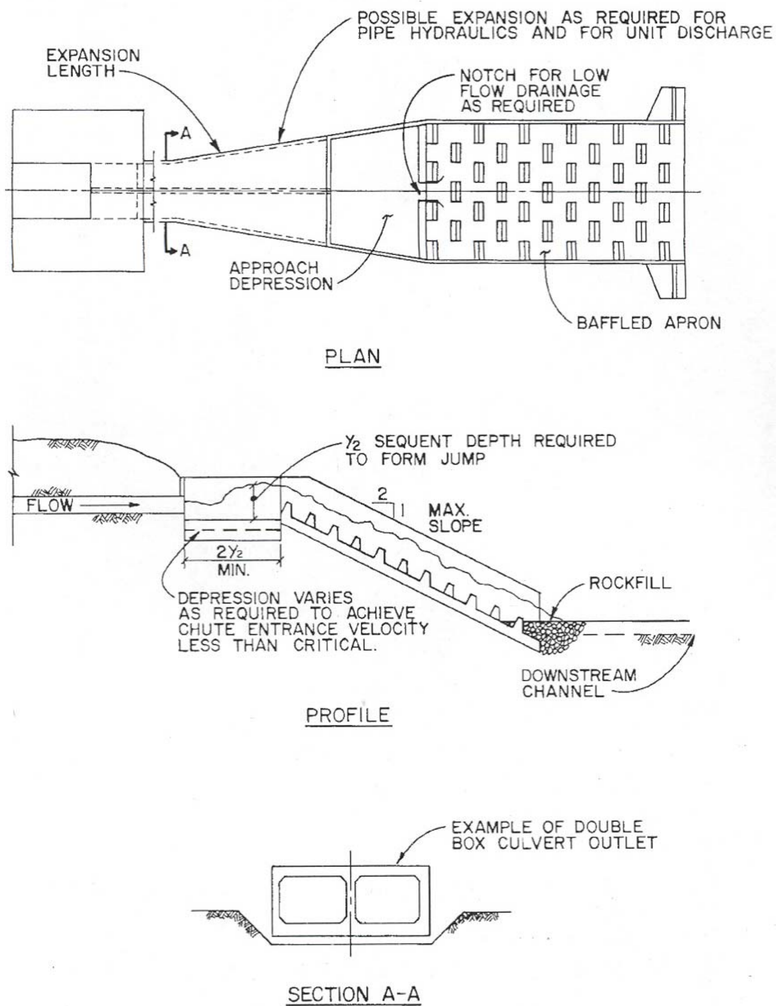
2. A sill section should be constructed directly in front of the low flow notch to break up bottom flow velocities. The length of this sill section should overlap the width of the low flow by about 1 foot. The general layout for the low flow modifications is shown in Figure 8.28.

8.4.2.2 Baffle Chute Energy Dissipator

The baffle chute developed by Peterka (1984) has also been adapted to use at pipe outlets. This structure is particularly well suited to situations with very large conduit outfalls and at outfalls to channels in which some future degradation is anticipated. As mentioned previously, the apron can be extended at a later time to account for channel subsidence. Generally, this type of structure is only cost effective if a grade drop is necessary below the outfall elevation and a hydraulic backwater can be tolerated in the culvert design.

Figure 8.29, on the following page, illustrates a general configuration for baffled outlet for a double box culvert outlet. In this case, an expansion zone occurs just upstream of the approach depression. The depression depth is designed as required to achieve the flow velocity at the chute entrance as described in Section 8.3.2.2. The remaining hydraulic design is the same as for a standard baffle chute. The same crest modifications are applicable to allow drainage of the approach depression, to reduce the upstream backwater effects of the baffles, and to reduce the problems of debris accumulation at the upstream row of baffles.

Figure 8- 29: Baffle Chute at Conduit Outlet
(Adapted from: Peterka, 1984)



An effective means of controlling velocities within the culvert is the use of reinforced concrete pipe (RCP) velocity control rings. The culvert velocity reduction by internal energy dissipators (velocity control rings or roughness elements) force the hydraulic jump to occur within the culvert, thus eliminating costly outlet structures. The design procedures can be found in Concrete Pipe Handbook (ACPA, 1988) and HEC-14 (USDOT, 1983).

8.4.2.3 Multiple Conduit Installations

Where more than one conduit of different sizes has outlets in close proximity, a composite structure can be constructed to take advantage of common walls. This can be somewhat awkward since each basin "cell" must be designed as an individual basin with different dimensions. Where two conduits of the same size have close outlets, the structures may be combined into a single basin as shown in Figure 8.28.The total width of a combined dual inlet basin can be reduced to three-fourths of the total width for separate basins. For example, if the design width for each pipe is W, the combined basin width would be 1.5W.The effect of mixing and turbulence of the combined flows in the basin has not been model tested to date. It is suggested that no wall be constructed to separate flow behind the baffle, thereby allowing greater turbulence in the combined basin.

Remaining structure dimensions are based on the design width of a separate basin W. If the two pipes have different flows, the combined structure should be based on the higher Froude number flows.

8.5 SPILLWAYS

8.5.1 Hydraulic Analysis

Spillways can take a variety of forms. Some of those, such as morning glory and fuseplug, are beyond the scope of this manual. In application of the more complex spillways, the appropriate hydraulic analyses must be performed by an experienced hydraulic engineer with due consideration of all aspects of the flow hydraulics. The most common spillways for use in typical drainage structures are of the weir or orifice type. In those cases, the weir equation (Equation (8.19)) and the orifice equation (Equation (8.20)) are the commonly used analytic methods.

$$Q = CLH^{3/2} \tag{8.19}$$

$$Q = C_o A(2gH)^{1/2} \tag{8.20}$$

8.5.2 Design

8.5.2.1 Weir Type Spillways

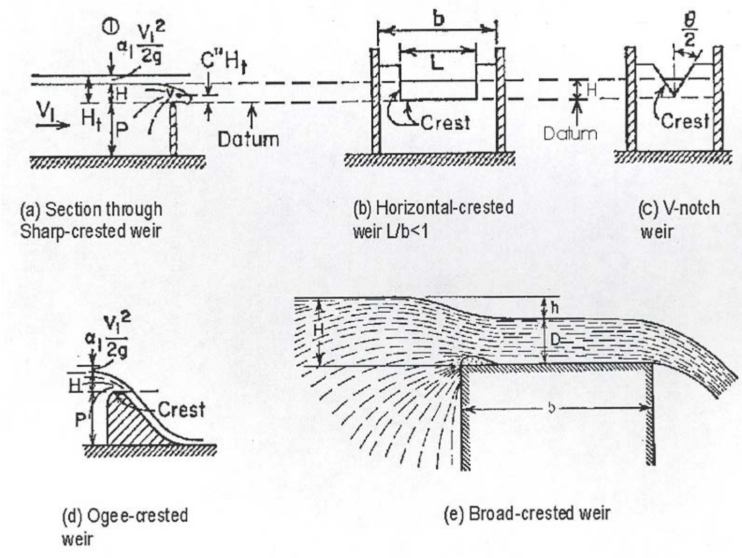
Weir-type spillways can be generally classified as sharp crested, broad crested or compound curve (ogee) shaped (Figure 8.30). The primary difference between sharp crested and broad crested weirs is the thickness of the weir (in profile) relative to the depth of water passing the crest. Where the crest thickness is greater than 6/10 the depth of flow over the weir, the weir can be considered to be broad crested (Simon, 1981). In all cases, the weir equation is generally used to assess spillway performance and to establish a spillway capacity rating curve. However, the selection of the weir coefficient, C, is a function of numerous factors including the total head on the weir, the vertical height of the weir, inclined faces of the weir (both upstream and/or downstream), submergence conditions, and breadth of broad crested weirs. Care must be taken in selecting the value of C and in applying appropriate correction factors to C depending upon the structure configuration and flow conditions.

For sharp crested weirs, the weir coefficient can range from about 3.2 to an excess of 5.0. The Rehbock equation (Equation (8.21)) (Chow, 1959, pg. 362) is often used to estimate C;

$$C = 3.27 + 0.40\frac{H}{h_w} \tag{8.21}$$

Where H is the measured head and h_w is the height of the weir.

Figure 8- 30: Weir Spillway Configurations
(Adapted from: Brater and King, 1976)



That equation is valid for H/h_w up to 5 but can be extended to H/h_w = 10 with fair approximation. Values of C in excess of 5.0 should not be used without careful deliberation of all factors including consequence of overestimated capacity. Typical C values are in the lower end of the aforementioned range. It is important to note that this discussion assumes that the nappe of water over the sharp crested weir is fully aerated. Insufficient aeration will result in undesirable performance, including pressure differential on the structure, unsteady and pulsing discharge over the weir, and increase in spillway discharge. Brater and King (1976) provides useful tables in selecting appropriate values for C.

Broad crested weirs have widely varying physical conditions which significantly affects the value of the weir coefficient. The normal range of C is from about 2.4 to about 3.5, however, use of values in excess of 3.1 must be carefully analyzed and are generally not recommended. A discharge coefficient of 3.0 is typical for flow over roadway embankments without backwater (Bureau of Public Roads, 1978). The head, H, is measured at least 2.5H upstream of the weir for broad crested weirs.

Ogee shaped spillways can offer the best hydraulic performance; however, the cost of such spillways is usually greater than other comparable weir types. Ogee spillways must be designed and analyzed by appropriate methods, such as those enumerated in the Design of Small Dams(USBR, 1987).It is important to note that downstream water surface elevation (tailwater) must be analyzed by appropriate methods (see the Open Channel Chapter) to assess potential for submergence of any weir.

8.5.2.2 Conduit Type Spillways

Impoundments that incorporate pipe or conduit in the principal outlet can be assessed as a culvert as detailed in Chapter 5. Under inlet control, the orifice equation provides a relation between ponded depth and outlet discharge. The orifice equation is useful in preparing rating curves for detention basins where one or more openings are incorporated into the riser of the primary outlet structure.

Principal spillway conduits other than those that can be analyzed by culvert hydraulics (see Chapter 5) can usually be analyzed under conditions of inlet and outlet control by procedures contained in hydraulic references such as Design of Small Dams (USBR, 1987) or Brater and King (1976). It is important to note that such structures must be analyzed for both inlet and outlet control with appropriate consideration of tailwater conditions that may exist at the outlet of structure.

8.5.2.3 Compound Rating Curves

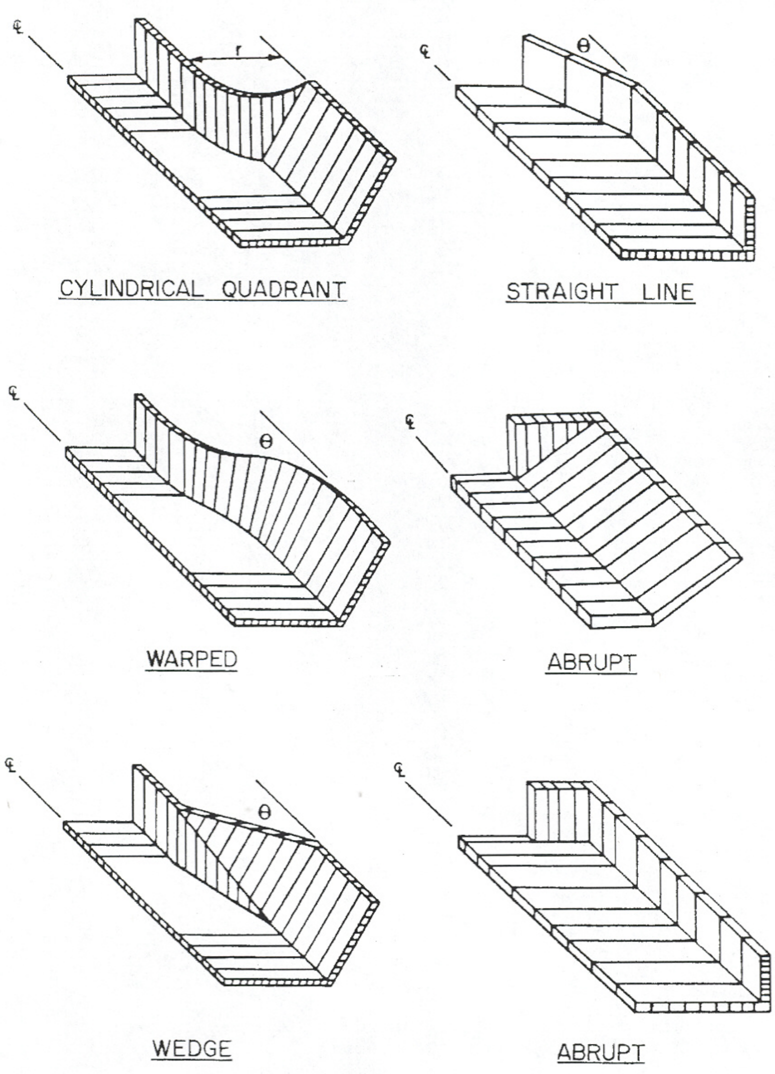
When an impoundment incorporates more than one spillway, a compound rating curve is developed for use in storage routing. Coupled with stage-storage data, an inflow hydrograph can be routed through a basin, thereby estimating ponded water surface elevation and outflow discharge. The principal and emergency spillways are individually assessed for discharge over a range of impoundment water levels, starting at the lowest anticipated level to above the height of the dam. The discharge from each spillway at each elevation is totaled to develop the compound rating curve. For stormwater detention facilities, it is usual to prepare compound rating curves for the principal spillway as these structures may have low, middle, and high level inlets to meter outflow from the basin. The controlling hydraulic conditions must be considered when developing a rating curve for an outlet structure. For example, consider a principal spillway represented by a pipe culvert with a grated drop inlet. The weir equation is used to develop a discharge rating based upon the length and width of the drop inlet. The orifice equation is used to develop a discharge rating based upon the grate opening. For these two ratings, the lesser discharge for a given elevation is the governing discharge for the outlet rating curve. In this example, the outlet pipe capacity would also be assessed to verify that it does not control outlet hydraulics.

8.6 SPECIAL CHANNEL STRUCTURES

8.6.1 Channel Transitions

A flow transition is a change of the open channel flow cross section designed to be accomplished in a short distance with a minimum amount of flow disturbance. Types of transitions are illustrated in Figure 8.31. Of these, the abrupt (headwall) and the straight line (wingwall) are the most common.

Figure 8- 31: Channel Transition Types
(Adapted from: USDOT, FHWA, HEC-14, 1983)



8.6.1.1 Contractions

Specially designed open channel flow transitions (contractions) are normally not required for highway culverts. A culvert is normally designed to operate with an upstream headwater pool which dissipates the channel approach velocity and, therefore, negates the need for an approach flow transition. The side and slope tapered inlets for culverts are also designed primarily as submerged transitions and are discussed in Chapter 5.Special inlet transitions are useful when the conservation of flow energy is essential because of allowable headwater consideration, such as an irrigation structure in subcritical flow, or where it is desirable to maintain a small cross section with supercritical flow in a steep channel. Furthermore, special transitions should be considered at locations where channel geometry changes, bridges, chutes, and other structures.

8.6.1.2 Expansions

Outlet transitions (expansions), changes in Q, R/W, channel geometry, bridges, chutes and other structures must be considered in the design of all culverts, channel, protection, and energy dissipators. Design considerations for subcritical channel transitions are presented in Hydraulic Design of Energy Dissipators for Culverts and Channels (USDOT, 1983)

8.6.1.3 Bifurcation Structures

It may occasionally be necessary to divert part of the flow in a channel. For example, the designer may need to divert a portion of the flow to a stormwater basin or, the downstream right-of-way may be too narrow to accommodate the full flow and a portion of the flow may have to be diverted to another outfall point. In these instances the designer will have to provide a “splitter” or bifurcation structure to apportion the flow in the appropriate direction.

In order for the structure to work as designed, the water surface elevation must be the same in all three channels at the proposed structure. This is accomplished by determining the water surface elevation in the upstream channel at the proposed structure. Then, the exact location of the splitter wall to divert the desired amount of water is calculated. Last, the geometry of both downstream channels must be adjusted to produce water surface elevations at the structure that match the water surface elevation in the upstream channel.

If the flow in the channel at the structure site is supercritical, the process is reversed and the water surface profiles are calculated in the downstream direction. However, considerable caution should be exercised in attempting to split supercritical flows. Readers are strongly encouraged to consult appropriate references listed at the end of this chapter or seek the advice of an experienced professional.

Once the water surface at the structure site has been established, the amount of flow in each area of the upstream channel can be calculated and the precise horizontal location of the splitter wall established. The initial angle of departure of the diverted channel should not exceed 12 degrees. This will minimize the formation of standing waves and turbulence that could encroach on the channel freeboard or otherwise reduce the capacity of the channel.

8.6.1.4 Side Channel Spillways

Side channel spillways offer a unique design consideration since the channel energy grade varies parallel to the spillway. Thus, weir equations are not always applicable. Pinal County reviews side channel spillways on a case by case basis. The hydraulic analysis of side channel spillways should be pursued with consultation with County staff.

8.6.1.5 Channel Junctions

Special design considerations are needed for channel junctions as follows:

- The design water-surface elevations immediately upstream of the confluence should be equal.

- The angle of junction intersection should be less than 12 degrees (zero is preferred). The centerline radius of any channel can not be less than 3 times the top-width at the water surface.

- The design depth of the main channel below the junction should be the same (or virtually so) as the main channel upstream of the confluence.

- For supercritical flow regime a momentum analysis as outlined in the Corps of Engineers document EM 1110-2-1601 (USACE, 1991) must be undertaken. On a case by case basis, model testing will be required.

- Channels designed with Froude numbers between 0.9 and 1.13 will not be allowed.

8.6.2 Supercritical Flow Structures

8.6.2.1 Acceleration Chutes

Acceleration chutes, whether leading into box culverts, pipes, or high velocity open channels, are often used to reduce downstream cross sections, hence, reducing costs. Chute spillways may be used in connection with both off-stream and on-stream stormwater storage reservoirs for a control structure and/or a spillway.

Acceleration chutes are potentially hazardous if inadequately planned and designed (see USBR, 1974; Peterka, 1984; and SCS, 1976). High velocity flow can wash out channels and structures downstream in short order, resulting in property damage and uncontrolled flow. The references cited previously, address acceleration chutes in greater detail than can be discussed in this manual. Refer to these publications for a detailed analysis. Chutes have four component parts:

1. Inlet
2. Vertical Curve Section
3. Concrete, Steeply Sloped Channel
4. Outlet

Several types of inlets can be incorporated depending on the physical conditions and the type of control desired, particularly when using chute spillways for off-stream stormwater storage facilities. The types of inlets to be considered are:

- Straight Inlet
- Box Inlet
- Side-Channel Inlet
- Culvert Inlet
- Drop Inlet

Normally, the flow must remain at supercritical through the length of the chute and into the channel or conduit downstream. Care must be exercised in the design to insure against an unwanted hydraulic jump in the downstream channel or conduit. The analysis must include computation of the energy gradient through the chute and in the downstream channel or conduit.

8.6.2.2 Bends

Structures are generally unnecessary in subcritical flow channels unless the bend is of small radius. Structures for supercritical flows are complex and require careful hydraulic design to control the flow.

Bends are normally not used in supercritical flow channels because of the costs involved and the hazards introduced. It is possible to utilize banking, easement curves, and diagonal sills (Knapp, 1951). Sometimes outside bank rollover structures might even be considered. All of these, however, are generally out of place in urban drainage works. Additional design guidelines for open channel bends may be found in Hydraulic Design of Flood Control Channels (USACE, 1991). When a bend is necessary, and it is not practical to first take the flow into subcritical flow, the designer will generally conclude that the channel should be placed in the closed conduit for the entire reach of the bend, and downstream far enough to eliminate the main oscillations. A model test is usually required on such structures. Furthermore, the forces exerted on the structure are large and must be analyzed. The forces involved with hydraulic structures are large, and their analyses are often complex. The forces created can cause substantial damage if provisions are not made for their control. In bends, forces are usually larger than what is intuitively assumed. The momentum equation permits solution for the force acting upon the flow boundary at a bend.

$$F_b = M\Delta V \tag{8.22}$$

where *ΔV* represents the change in direction and/or magnitude of the velocity through the section bend. The force due to pressure on the bend should also be calculated when conduits flow under pressure.

$$\Delta P = \frac{P}{2}\left(\Delta V^2\right) \tag{8.23}$$

where *ΔP* represents the pressure change caused by the difference in the squares of the velocities through the bend. The total exerted force on the bend by the water, the total of momentum and pressure forces, must be counteracted by external forces. Allowable soil bearing should be determined using soil tests if necessary. Forces which cannot be handled by conduit bearing on the soil must be compensated for by additional thrust blocks or other structures.

8.6.3 Groins and Guide Dikes

There are several flow control structures that are similar in configuration and serve to reduce erosion and scour. Some of these also serve to train flow away from critical areas. Because of the similarity in form or function, the terminology used in practice tends to be overlapping in that the term used by one entity or organization conflicts in meaning with the same term used by another. In this section, two hydraulic structures will be discussed. Groins are used to train flow and reduce erosion in channels. Guide dikes serve a similar purpose, but are typically found in a natural floodplain setting.

8.6.3.1 Groins

Structures located along and protruding from the banks of a channel for purposes of training flow away from the bank, reducing velocities, or reducing erosion are termed groins here. Other terms used for structures meeting this definition are spurs, hardpoints, and dikes. In a natural setting, these structures are often deployed at the outside of bends in a channel to reduce bank erosion and redirect higher velocities towards the center of the channel, where higher velocities are better tolerated due to armoring. In the absence of armoring, these structures merely relocate the area subject to continued erosion (see Chapter 7 for further discussions on sedimentation). Hydraulically, groins create greater depths of flow upstream of the structure in subcritical flow conditions and flatten the energy grade line. Acting like a constriction, the energy grade line is steeper at the structure while backwater eddies are created immediately downstream of the structure unless they are drowned out by overtopping flow. Engineers often use these structures to train low to moderate flows without overtopping. Higher flood flows usually overtop the structure. Under certain circumstances, groins deployed on both sides of an engineered channel can be used to flatten the energy grade line, thereby allowing a steeper channel slope. Under all applications, the appropriate hydraulic analysis should be employed to evaluate velocities under the range of conditions expected or required to meet regulatory requirements. Erosion protection is often required at the groin and downstream of the groin.

Groins may be made of many different materials including riprap, gabions, piling (wood or steel), and rock and earth filled cribs. The designer should verify with the entity responsible for maintenance acceptable materials for the application at hand.

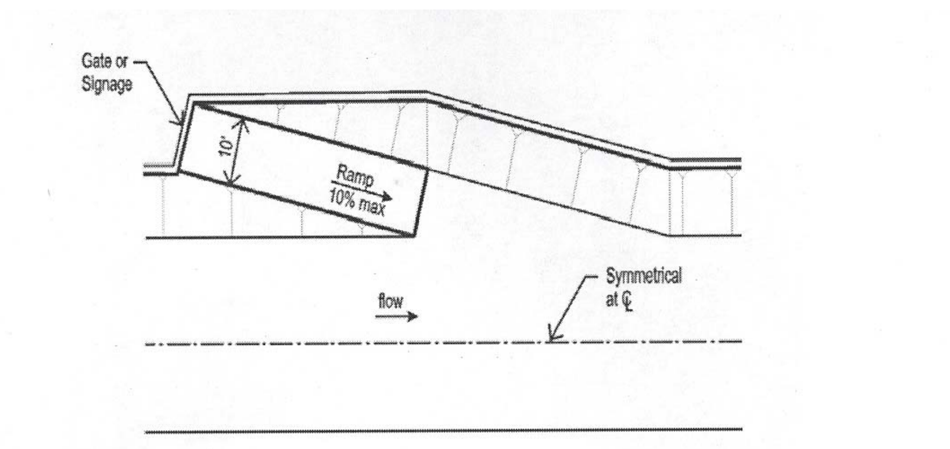
8.6.3.2 Guide Dikes

These structures are deployed upstream of bridge abutments and serve to transition flow into the bridge from the floodplain. Also called guide banks, these structures have been found to minimize scour of the abutments and piers. Here, the scour is relocated to the head of the guide dike, thereby offering hydraulic efficiency and scour protection to the bridge structure. Design procedures for guide banks are enumerated in Bridge Scour and Stream Instability Countermeasures (USDOT, 2001).

8.6.4 Access Ramps

Vehicular access to drainage and flood control channels must be provided at periodic intervals to permit the efficient removal of sediment and accumulated debris and to facilitate structural maintenance. Access is typically provided by 10-foot wide ramps constructed in the channel sideslopes. Figure 8.32 illustrates a typical ramp design and a typical flared sideslope design.

Figure 8- 32: Transverse Slope Right Angle Channel Access



The City of Albuquerque, New Mexico investigated the hydraulic effects of vehicle ramps and flared side slopes in channels (Heggen, 1991). Although the study is too long to be included in this manual, the final recommendations are consistent with other recommendations in this manual and can be summarized as follows:

1. For subcritical flow, the hydraulic consequences of occasional access structures are minor. For supercritical flow, the hydraulic consequences of channel cross sectional changes can be major. Hydraulic jumps or oblique waves can jeopardize the entire channel.

2. Ramps should be directed downstream.

3. The Froude number approaching downstream ramps should not exceed 2.2 for a one-sided configuration.

4. Flared sideslopes should be as steep as vehicle access allows.

5. The Froude number approaching 3:1 flared sideslopes should not exceed 3.5 for a one-sided configuration.

6. The Froude number approaching a 6:1 flared sideslopes should not exceed 2.2 for a one-sided configuration.

7. Structures should be symmetrical.

8. Upstream and downstream channel slopes are not a significant factor in performance.

9. Ramps perform somewhat better than 6:1 flared sideslopes, but not as well as 3:1 flared sideslopes.

As a general rule, access structures should be provided at the upstream and downstream side of every culvert and street crossing. Access over or around drop structures also needs to be considered.

8.6.5 Trashracks and Access Barriers

The necessity for trashracks depends on the size of the conduit, the nature of the trash and debris, public safety and other factors. These factors will determine the type of trashracks and the size of the openings. A smaller conduit will require closely spaced trash bars and a larger conduit requires more widely spaced trash bars. If there is no danger of clogging or damage from small trash, a trashrack may consist simply of struts and beams placed to exclude only the larger trees and such floating debris. For trashracks with approach velocities less than 3 ft/sec, it is not necessary to include a head loss for the trashrack; however, for velocities greater than 3 ft/sec, such computations are required. Trashracks can promote debris buildup and the subsequent reduction of hydraulic performance. Thorough analysis of this potential should be undertaken prior to their use. Depending on the anticipated volume and size of the debris an open area between the bars of 1.5 to 3.0 times the area of the culvert entrance should be provided. Trashrack losses are a function of velocity, bar thickness, bar spacing, rack angle, and orientation of the flow entering the rack, the latter condition being an important factor. Trashracks with bars oriented horizontally are not permitted, and horizontal bars used to support vertically oriented bars should be as small as practical and kept to the minimum required to meet structural requirements.

The expected head loss from a trashrack in a channel is greatly affected by the approach angle. The head loss computed by Equation (8.24) should be multiplied by the appropriate value from Table 8.6, when the approach channel and trashrack are at an angle to each other. Equation (8.24) applies to access barriers placed on conduit outlets and should be used when approach velocities are greater than 3 ft/sec. The approach angle loss factor does not apply when the outlet works trashrack is within a detention basin, reservoir, dam or other ponded area.

$$H_g = 1.5\frac{\left(V_g^{\,2} - V_a^{\,2}\right)}{2g} \tag{8.24}$$

Table 8- 6: Loss Factors for Approach Angle Skewed to Trashrack

Approach Angle, degrees	Loss Factor
0	1.0
20	1.7
40	3.0
60	6.0

For trashracks in detention basins, reservoirs, dams or areas where the flow into the outlet conduit is ponded, the headloss shall be computed by Equation (8.25):

$$H_g = K_t\frac{V_n^{\,2}}{2g} \tag{8.25}$$

Where: K_t is given by Equation (8.26):

$$K_t = 1.45 - 0.45\frac{a_n}{a_g} - \left(\frac{a_n}{a_g}\right)^2 \tag{8.26}$$

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