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Drainage Design Manual for Maricopa County, Arizona

Volume II
Hydraulics

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1 Introduction

1.1 Purpose

The objective of the *Drainage Design Manual for Maricopa County, Volume II, Hydraulics,* is to provide criteria and design guidance for storm drainage facilities in Maricopa County. There are two reasons to develop such a manual: 1) it provides a convenient source of technical information that is specifically tailored to the unique hydrologic, environmental, and social character of Maricopa County; and 2) it provides a consistent set of criteria that, when used by the local governing agencies and the land development community, will result in uniform drainage practices throughout the county. Use of Volume II of the *Drainage Design Manual for Maricopa County* will result in improved hydraulic performance of drainage facilities, uniformity in design practices across jurisdictional boundaries, and reduction of conflict between the regulatory agencies and the land development community. Although they have not yet been adopted by any regulatory agency, regulation requirements have been highlighted by using a ruled box, for example:

This is how a regulation requirement will be highlighted in this manual.

1.2 Background

This manual was produced by a team of consultants under contract to the Flood Control District of Maricopa County. Beginning in 1987, the manual was developed through a highly interactive process involving work groups for each major topic. The work groups were composed of the engineering consultant, the Flood Control District, representatives of the various communities in Maricopa County, and representatives of home builders and land developers. The work groups were charged with advising the consultant about applicability of technical criteria, special problem areas to be addressed, and resolving conflict over potential differences in drainage standards between communities.

1.3 Scope

The *Drainage Design Manual for Maricopa County, Volume II, Hydraulics,* is divided into nine chapters that address the major subject areas of hydraulic design. The authors' intention was to provide general design guidance for designs that are common to the Maricopa County environment. Complex designs requiring specific expertise are not included in this manual; however, where design exceeds the scope of this manual, the reader is referred to documentation appropriate for that design. The remainder of this manual is summarized briefly herein.

Hydrology

Chapter 2 provides an overview of the hydrology criteria for drainage structures; the flood hydrology that is recommended for use in Maricopa County is contained in Volume I of the *Drainage Design Manual for Maricopa County.* That procedure provides for the use of the Rational Method for small, uniform watersheds, and for use of the U.S. Army Corps of Engineers' HEC-1 Flood Hydrology Program for larger watersheds with diverse surface conditions. The procedure provides design rainfall criteria that has been developed specifically for Maricopa County, rainfall loss methods that are based on the best practical technology that is available for estimating surface retention losses and infiltration rates, and unit hydrograph procedures that have been selected and developed for the various land-uses in Maricopa County.

Street Drainage

Chapter 3 provides design guidelines for the drainage of streets using curbs and stormdrain inlets. An overall approach to storm runoff management includes using the street system to transport runoff to inlets and to transport runoff from storms that are greater than the storm sewer capacity. Design criteria, design procedures, and design aids are provided for streets and gutters, intersections, and roadside ditches. Catch basins are discussed in regard to alternatives and suggested applications, capacities, and design procedures.

Closed Conduits

Chapter 4 provides complete coverage of two major topics: storm sewers and culverts. A comprehensive treatment of storm sewers is provided including extensive use of design aids for catch basins, manholes, and various types of storm sewer junctions. All information for the design of culverts is provided. This includes the necessary design aids, guidance for treatment of culvert inlets and outlets, and scour protection at the culvert outlet. Extensive use of many detailed examples clearly illustrate the procedures to be used for most practical applications.

Open Channels

Chapter 5 is devoted to the analysis and treatment of both natural and artificial channels. The scope of this chapter covers the more commonly encountered open channel design applications by civil engineers who do not possess special design skills in open channel hydraulics. Applications involving rivers and large washes or channels, which are considered as non-rigid, require special design skills, and the design of these channels should not be attempted with the design techniques contained in this chapter. A simplified design procedure is presented that provides an appropriate level of analysis for most design problems that will be encountered for artificial channels. The simplified design procedure assumes a rigid channel, and the procedure is valid for both subcritical and supercritical flows. Channel linings of concrete, soil cement, riprap, wire enclosed rock (gabion), and grass are discussed in the manual. The analysis of natural channels is discussed in broader terms than is the treatment of artificial channels. Although the basic theory is the same for both channel types, more complex flow conditions (nonuniform and unsteady flow) and concepts of sediment transport often need to be incorporated in the analysis of natural channels.

Hydraulic Structures

The hydraulic structures that are described in Chapter 6 are used to control or alter the flow characteristics, such as velocity, depth, energy, and so forth, and to affect a change in the configuration of an open-channel, such as channel slope. The purpose of such structures is to achieve a safer, more stable, and improved maintainability of conveyance systems. Channel drop structures are a major topic of this chapter and guidance is provided for the design of baffle chute drops, vertical hard basin drops, vertical riprap basin drops, sloping concrete drops, and low flow check structures. Information is provided for the dissipation of energy at conduit outlet structures with emphasis on riprap protection for outlets with moderate flow conditions and concrete structures for more severe conditions. Guidance is provided for the design of channel transitions, supercritical flow chutes, and bends in supercritical flow. The manual provides instruction in the theory and use of the hydraulic jump as a means of energy dissipation. The design of various, appropriate hydraulic jump energy dissipators is included.

Bridges

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, Chapter 7 is intended to present an overview of the hydraulic analyses for bridge openings over open channels. There is also a general discussion of scour.

Detention and Retention

Detention and retention basins are man-made storage facilities that are intended to mitigate the effects of urbanization on storm drainage. They serve to reduce peak discharges and can also reduce the volume of storm runoff downstream of the basin under certain conditions. Chapter 8 presents the engineering methodologies and details associated with the planning, analysis, and design of detention and retention facilities. Since detention and retention basins often require a considerable commitment of land resources by the community or land developer, particular emphasis is placed on planning basins that are amenities, and, where possible, incorporate multiple-use concepts. National storm water quality standards are being promulgated and criteria for use of detention and retention basins that will not jeopardize the quality of surface water and ground water resources are presented. Safety concerns of such facilities are detailed along with the means to enhance safety. The theory and procedure for performing routing of an inflow flood through such facilities is provided along with a detailed example of the calculations.

Pump Stations

Stormwater pump stations are used where gravity discharge is infeasible, such as depressed highway intersections, or for the controlled release of outflow, such as from a detention or retention facility. The criteria for use of pump stations in Maricopa County are provided in Chapter 9, however, the intent is to provide only an overview of the conditions that should be considered in the design of stormwater pumping facilities. Reference to another readily available document for the rigorous design of stormwater pump stations is also provided.

2

Hydrology

2.1 Methodology

The determination of flood hydrology for designing stormwater facilities in Maricopa County is to be performed according to the procedures set forth in the *Drainage Design Manual for Maricopa County, Volume I, Hydrology* (hereinafter referred to as the *Hydrology Manual*). Deviations from the procedures in the *Hydrology Manual* require prior approval from the jurisdictional agency and/or the Flood Control District of Maricopa County before proceeding with the determination of design hydrology. However, it is not the intent of the *Hydrology Manual* to inhibit sound, innovative analysis, or the utilization of superior technology, or the development of improved techniques. Therefore, the investigation, development, and use of the best practical technology for flood hydrology is strongly encouraged in all situations.

The selection of the procedure to be used to define the design flood hydrology is dependent upon the intended application. For small urban watersheds (defined as less than 160 acres and having fairly uniform land use), the use of the Rational Method is acceptable. Use of this method will only produce peak discharges and it should not be used if runoff volume or a complete runoff hydrograph is needed, such as for routing through detention/retention facilities. For larger, more complex watersheds or drainage networks, a rainfall-runoff model should be developed. The *Hydrology Manual* provides guidance in the development of such a model and the estimation of the necessary input parameters to the model.

Although not necessarily required, the use of the U.S. Army Corps of Engineers' HEC-1 Flood Hydrology Program facilitates the use of the procedures that are contained in the *Hydrology Manual*, which was written to supplement the *HEC-1 Users Manual*.

All of the hydrology that is required for the design of stormwater drainage facilities that are normally encountered can be performed by using the HEC-1 program; this includes the routing of flood hydrographs through detention/retention structures. The design and performance of pump stations cannot normally be satisfactorily performed using the simplified procedures that are incorporated into the HEC-1 program. Although the inflow hydrograph to a pump station can be adequately developed with HEC-1, the performance and design of pump stations will often require the use of specialized programs.

2.2 Criteria

The *Hydrology Manual* can be used to develop design hydrology magnitudes for storms of frequencies up to and including the 100-year event. The design storm is of 6-hours duration and is to be used for the design of stormwater drainage facilities except for detention/retention facilities. According to the *Uniform Drainage Policies and Standards for Maricopa County, Arizona* (25 February 1987), all development shall make provisions to retain the peak flow and volume of runoff from rainfall events up to and including the 100-year, 2-hour duration storm falling within the boundaries of the proposed development. The criteria to be applied to the 2-hour storm is provided in the *Hydrology Manual.* Table 2.1 outlines the minimum hydrology design criteria for drainage features.

2.3 Master Drainage Planning

According to the *Uniform Drainage Policies and Standards for Maricopa County, Arizona,* master drainage planning shall be done in the earliest stages of the planning process. A master drainage plan incorporates the hydrological analysis for on-site and off-site runoff and outlines the recommended plan for drainage and the course of action for implementation.

Master drainage planning can be encountered on both basin-wide and local scales. When undertaking a basin-wide plan, the designer must comprehensively evaluate practical alternatives to find the most cost-effective solution for the general public. Modifications can result from land-use driven decisions that are more costly; however, these additional costs are considered "developer costs" by most agencies. When preparing master drainage plans for local development, the designer shall illustrate conformance with basin-wide master drainage plans where they exist, or shall demonstrate that the plan will not increase extraordinarily the cost of providing basin-wide drainage for the local agency or the District.

The master planning process begins with the conceptual layout of the drainage system, which includes both large and small drainage facilities. All drainageway entrance and exit points in the proposed development must remain in the original location and—as near as possible—in the original condition. In many areas about to be urbanized, the runoff has been so minimal that natural channels do not exist. However, surface depressions normally exist and will provide an excellent basis for the initial siting of open channels. This condition is also true for open channels that are to be used primarily for road or highway drainage.

Master plans illustrate selected alternatives, including the footprint of facilities or land uses, approximate sizes, and physical impact on the land. General requirements for structures and their overall size and impact are also determined during the master planning phase; however, detailed selection of structure types, sizing of riprap, structural design, and selection and detailing of peripheral elements (inlets, trash racks, fencing, etc.) are completed in later phases using the criteria outlined in this manual.

Table 2.1
Hydrology Design Criteria

Drainage Feature	Peak Frequencies		
	10 Year	50 Year	100 Year
Street with Curb and Gutter	Runoff contained within street curbs. For collector and arterial streets one 12-foot dry driving lane must be maintained in each direction.	N/A	Runoff to be contained below the finished floor of building. Q_{max} = 100 cfs V_{max} = 10 fps d_{max} = 8 inches above the centerline of the street.
Street without Curb and Gutter	Runoff contained within the roadside channels with the water surface elevation below the subgrade.	N/A	Same as Street with Curb and Gutter.
Street with Storm Drain System	Pipes or roadside channels are added if the 10-year runoff exceeds street capacity.	N/A	Storm drains are used if 100-year runoff inundates the building's first floor.
Cross Road Culvert for Collector and Arterial Streets	N/A	Runoff to be conveyed by culvert under road with no flow overtopping the road. V_{max} = 15 fps V_{min} = 2.5 fps	Runoff to be conveyed by culvert and by flow over the road with maximum 6-inch flow depth over the road.
FEMA Floodplain Channel(1)	N/A	N/A	100-year peak storm
Channel to Convey Offsite Flow Through Development	N/A	N/A	100-year peak storm
Finish Floor Elevation for Buildings within a FEMA Floodplain Area	N/A	N/A	Minimum finish floor elevation to be 1 foot above the floodplain water surface elevation.
Finish Floor not in a FEMA Floodplain	N/A	N/A	The finish floor will be free from inundation for the 100-year peak storm event.
Retention Basin	N/A	N/A	100-year 2-hour storm for determining on-site retention volume.

(1) Per ARS 48-3609.A, during the course of the Master Planning process, the 100-year runoff will be used to delineate a floodplain for major channels with discharges of more than 1,000 cfs and will be processed through the local government, ADWR, and FEMA.

Notes

3

Street Drainage

This chapter provides design guidelines for the drainage of streets using curbs and stormdrain catch basins.

3.1 Definition of Symbols

The following symbols will be used in equations throughout Chapter 3.

a	=	Gutter depression, inches
A	=	Clear area of opening, ft^2
A_g	=	Clear area of the grate, ft^2
C_o	=	Orifice coefficient = 0.67
C_w	=	Weir coefficient
d	=	Depth at curb measured from the normal cross slope, ft (i.e., $d = TS_x$)
d_i	=	Depth at lip of curb opening, ft
d_o	=	Effective depth at the center of the curb opening orifice, ft
E_o	=	Ratio of flow in the depressed section to total gutter flow
F_R	=	Reduction Factor (see Table 3.2)
F_s	=	Gutter capacity reduction factor (see Figure 3.4)
g	=	Gravity, 32.2 ft/s^2
h	=	Height of curb opening catch basin, curb opening orifice, or orifice throat width, ft
L	=	Length of curb opening, grate, or slot, ft
L_T	=	Curb opening length required to intercept 100 percent of the gutter flow, ft
n	=	Manning's roughness coefficient (0.016)
P	=	Perimeter of the grate, disregarding bars and side against the curb, ft
Q	=	Total gutter flow
Q_{cap}	=	Allowable flow rate per gutter, cfs

Q_t	=	Theoretical gutter carrying capacity
Q_w	=	Flow in width W, ft^3/s
S	=	Longitudinal street slope, ft/ft
S_w'	=	Cross slope of the gutter relative to the cross slope of the pavement
S_x	=	Pavement cross slope (a/12W), ft/ft
T	=	Allowable spread, ft
V	=	Velocity of flow in the gutter, ft/s
V_o	=	Gutter velocity where splash-over first occurs (see Figure 3.27, page 50)
W	=	Width of depression, depressed gutter or grate, or slot, ft
y	=	Depth of flow, ft
z	=	Reciprocal of the pavement cross-slope, ft/ft

3.2 Streets

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

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

An overall approach to storm runoff management includes using the street system to transport runoff to catch basins and to transport runoff from storms that are greater than the storm drain capacity.

Freeways and similar types of roadways are not addressed in this manual.

3.2.1 Design Criteria for Streets and Gutters

3.2.1.1 Design Frequency: Storm drainage within a street system serves two primary objectives:

1. Remove nuisance flows from pavement during frequent return period storms to maintain safe and efficient movement of traffic.

2. Protect adjacent properties from damage caused by large, infrequent storms.

The function of removing nuisance flows from pavement is based on providing storm drain catch basins at points where maximum depth or driving lane inundation criteria are reached.

Storm drain system design is based on the design storm. The design storm is the storm associated with the governing return period for longitudinal street flow from Table 3.1. In the upper reaches of a system the 10-year criteria will govern. Farther downstream in the system, the storm drain system design for the 10-year storm may not meet the criteria stated for the 100-year storm. Storm drains will then need to be upsized to meet the 100-year criteria. Both return periods need to be checked to determine which condition governs. The storm condition governing design at any point is the design storm.

3.2.1.2 Pavement Encroachment: The following sections present specific design requirements for storm drainage on urban type streets for the design storm. Typical street sections used in Maricopa County are presented in Figure 3.1. Determination of street carrying capacity for the design storm shall be based upon two considerations:

1. Pavement encroachment and depth for computed theoretical flow conditions.
2. An empirical reduction of the theoretical allowable rate of flow to account for practical field conditions.

The storm drain system should commence at or prior to the point where the maximum encroachment and/or depth is reached, and should be designed on the basis of the design storm. The final design must meet both the 10-year and 100-year criteria set forth in Table 3.1.

The preceding criteria is established for new construction. Changes to an existing system, or a retrofit situation may not be able to meet the 10-year criteria, however, any changes to a system should meet the 100-year storm criteria.

3.2.1.3 Theoretical Capacity: When the allowable pavement encroachment has been determined, the theoretical gutter carrying capacity shall be computed using the modified Manning's formula for flow in a shallow triangular channel, as shown in Figure 3.2 or as expressed in Equation 3.1:

$$Q_t = \left(\frac{0.56}{n}\right) S_x^{1.67}\, S^{0.5}\, T^{2.67} \tag{3.1}$$

Table 3.1
Design Storm Frequencies for Street Drainage[1]

Longitudinal Street Flow	Event
No curb overtopping. Maintain one dry 12-foot driving lane in each direction for collector and arterial streets.	10 year[2]
Flow to be calculated assuming contained between buildings with: 100 cfs maximum flow. 10 fps maximum velocity. Maximum 8 inches above centerline recommended.	100 year

(1) No new inverted crown streets.
(2) 2 year for City of Phoenix.

Note:
For conceptual purposes only. Consult municipality for their specific detail.

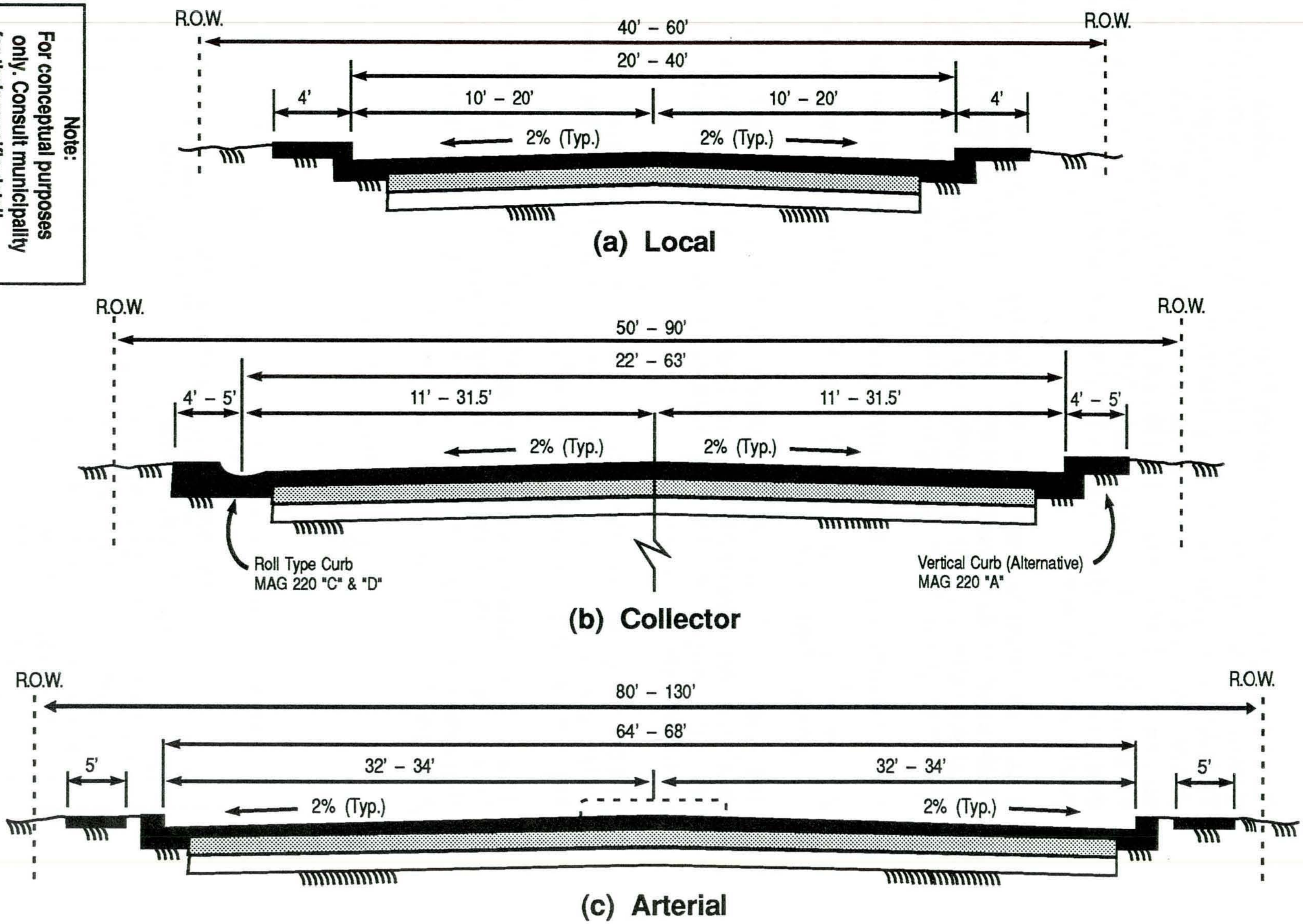


Figure 3.1
Typical Street Cross Section

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

Example:

Given:

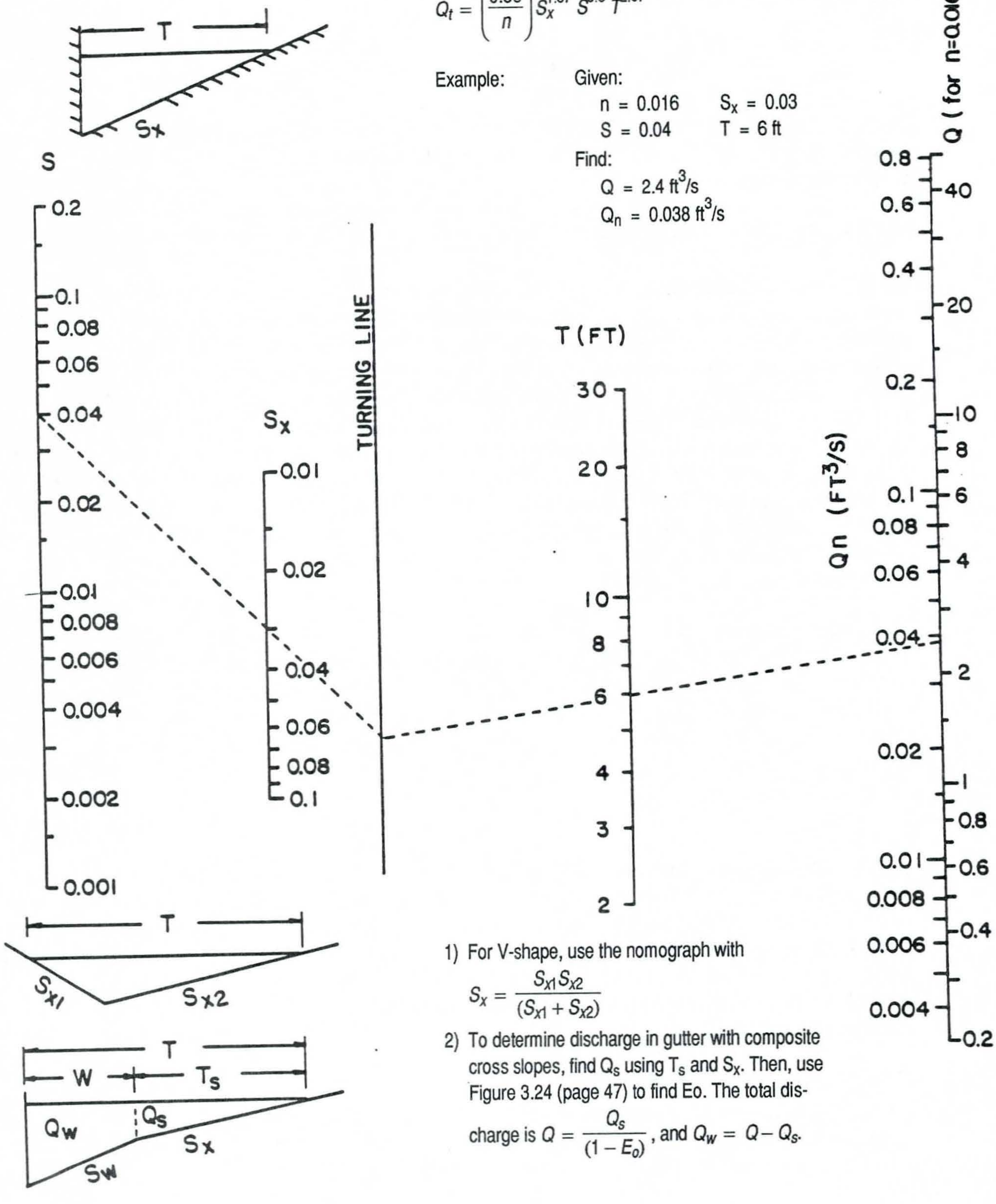
n = 0.016 $\quad S_x$ = 0.03

S = 0.04 $\quad$ T = 6 ft

Find:

Q = 2.4 ft^3/s

Q_n = 0.038 ft^3/s



1) For V-shape, use the nomograph with

$$S_x = \frac{S_{x1} S_{x2}}{(S_{x1} + S_{x2})}$$

2) To determine discharge in gutter with composite cross slopes, find Q_s using T_s and S_x. Then, use Figure 3.24 (page 47) to find Eo. The total discharge is $Q = \frac{Q_s}{(1 - E_o)}$, and $Q_w = Q - Q_s$.

Figure 3.2
Nomograph for Triangular Gutters
(source: FHWA, 1984, HEC-12)

Equation 3.1 can also be expressed as:

$$Q_t = 0.56\left(\frac{z}{n}\right)S^{1/2}y^{8/3}$$

Typical gutter configurations from MAG are shown in Figure 3.3. Figure 3.2 may be used for all gutter configurations. To simplify computations, graphs for particular street shapes may be plotted.

An n value of 0.016 shall be used for street flow unless special considerations exist.

3.2.1.4 Reduction Factors: The gutter capacity from Figure 3.2 is based on the theoretical capacity of a clean, unobstructed, continuous gutter section. In reality, parked car tires in the gutter and debris cause obstructions to flow. Driveways, alleys, and curb cuts cause discontinuities in the flow. When water flowing in a gutter encounters an obstruction or discontinuity, it is deflected out of the gutter into the street section. If the velocity is high enough, the flow diverted out of the gutter will flow across the street crown to the gutter on the opposite side. If an inlet is located just downstream of the obstruction the water will flow past the inlet without being intercepted. Gutter capacity reduction factors are established to limit velocities and reduce the lane encroachment caused by water deflected into the street and to allow adequate capacity for unanticipated inlet bypass flows caused by obstructions.

The actual flow rate allowable per gutter shall be calculated by multiplying the theoretical capacity by the corresponding factor, obtained from Figure 3.4. Discharge curves can be developed for standard streets by plotting the solution of the following equation (Q_{cap} vs. S) for a range of longitudinal slopes with the applicable gutter capacity reduction factor from Figure 3.4 applied to the theoretical discharge computed for each slope.

$$Q_{cap} = F_s \times Q_t \qquad \textbf{(3.2)}$$

3.2.1.5 Transition at Curb and Gutter End: Where curb and gutter sections end, care must be taken to transition the gutter flow into the receiving ditch or channel to prevent scour. When the flow encounters a widening in the channel cross section, it spreads at approximately 4:1 (longitudinal to transverse). A concrete or rock riprap apron should be provided to protect the receiving channel or ditch and spread the flow until the velocity is below the maximum allowable velocity for the channel material.

3.2.2 Design Criteria for Intersections

Figure 3.5 is a typical illustration of the variations in grade when local streets intersect. When local streets intersect arterial or collector streets, the grades of the arterial or collector street should be continued uninterrupted.

When collector and arterial streets intersect, the grade of the more major street should be maintained as much as possible. For drainage purposes, no form of valley gutter should be constructed across an arterial street.

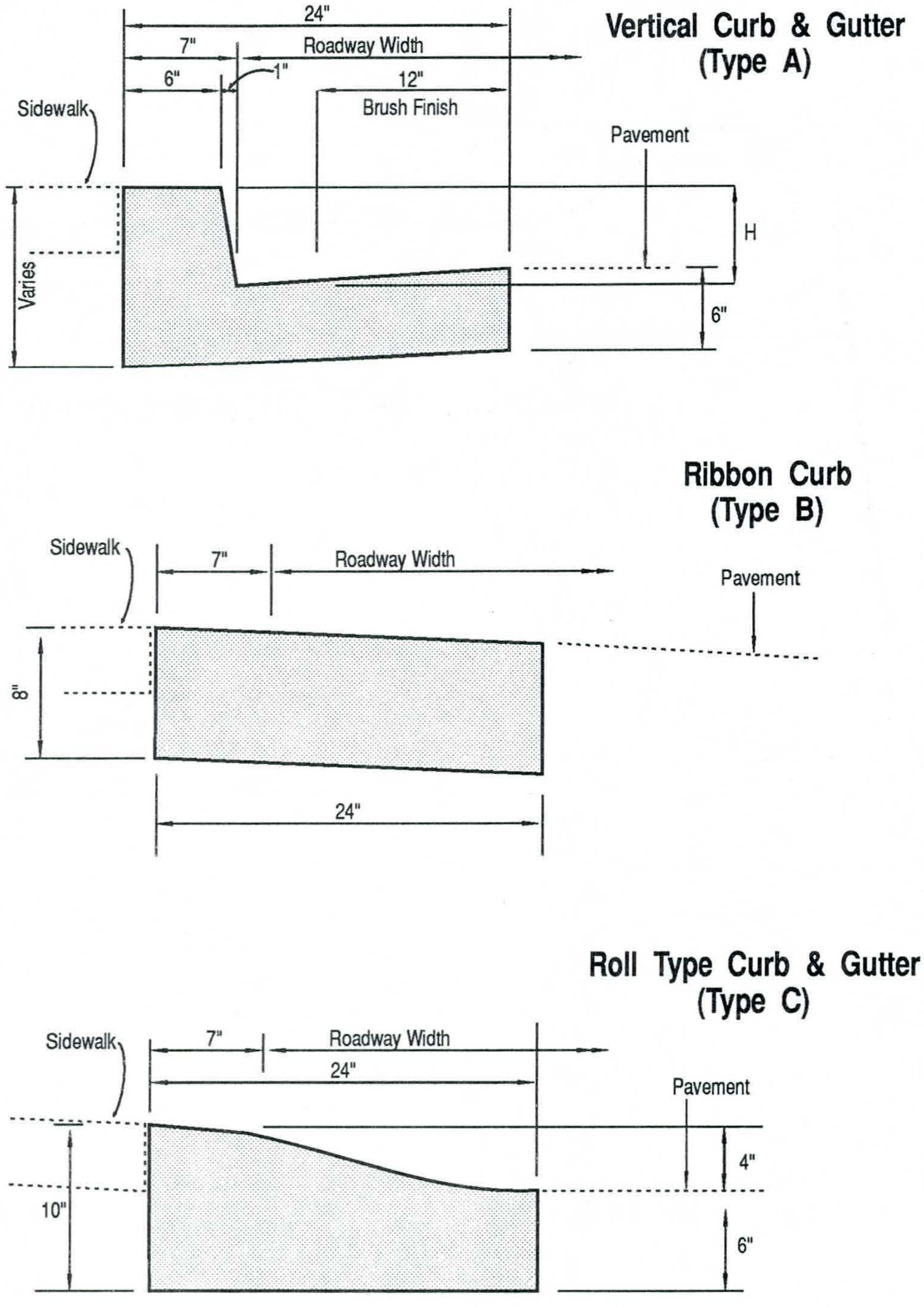


Figure 3.3
Detail for Curbs and Gutters
(Adapted from MAG Standards)

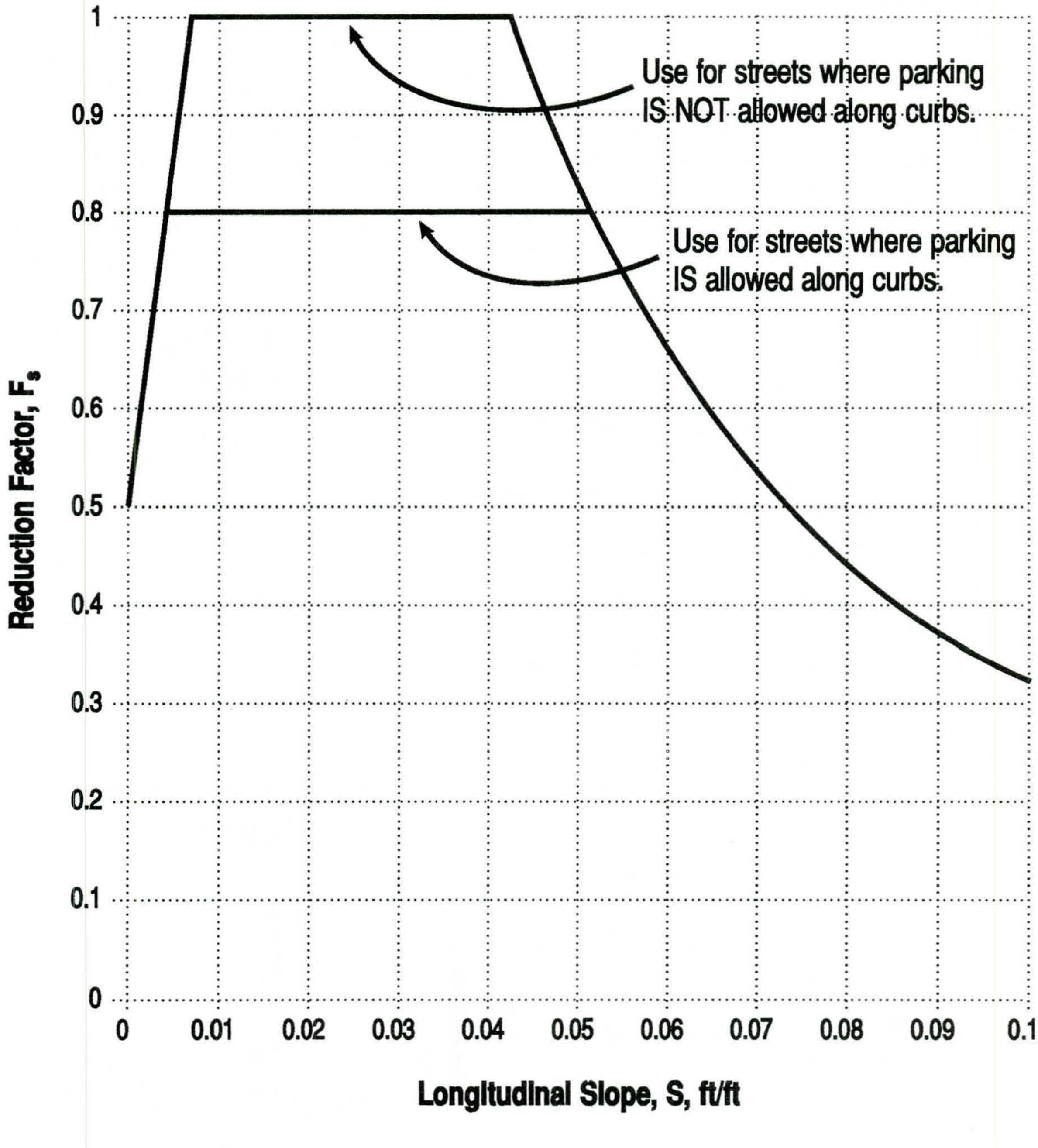
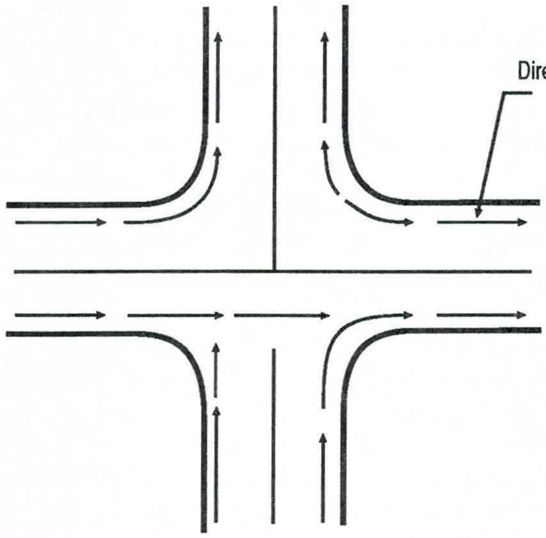
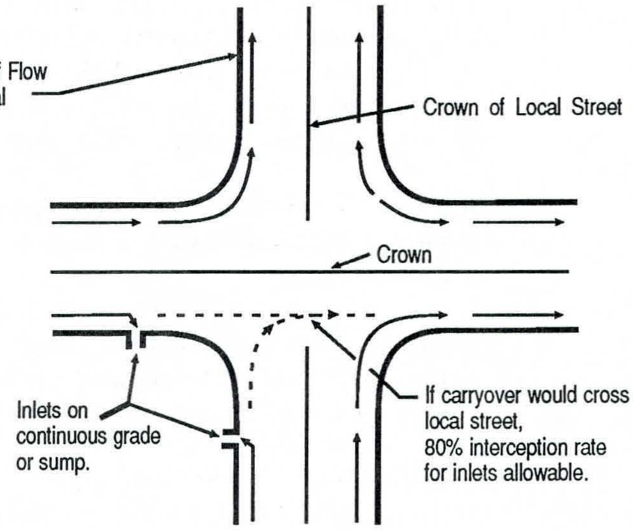


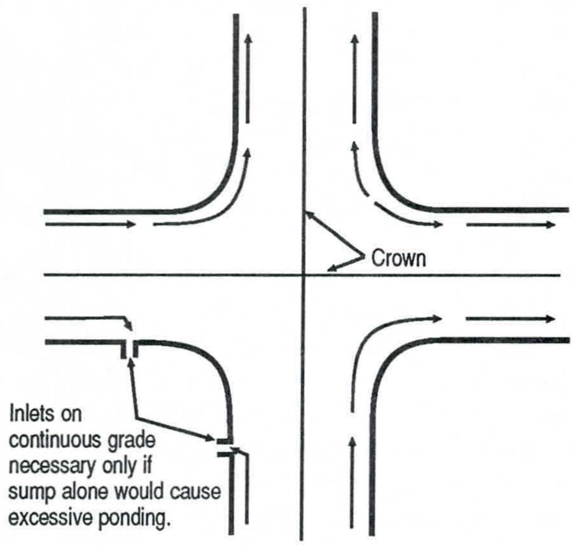
Figure 3.4
Gutter Capacity Reduction Factors



Local Street to Local Street

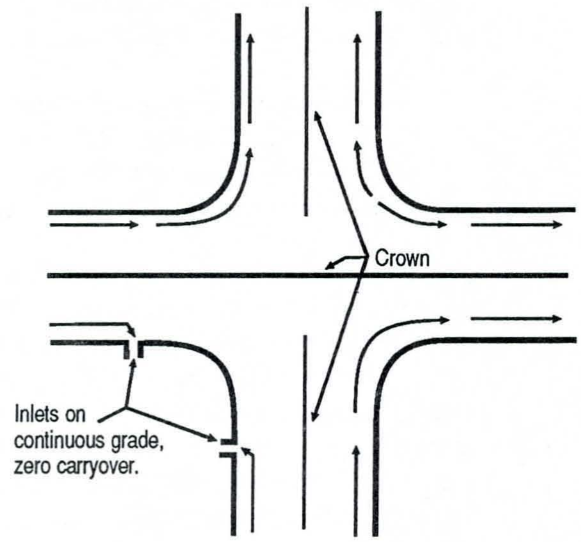


Local or Collector to Arterial



Arterial to Arterial

(Where Crowns are to be Maintained)



Arterial to Arterial

(One Continuous Crown)

These examples show the minimum required inlets. Additional inlets may be necessary based upon allowable carrying capacity of gutters.

Figure 3.5
Typical Street Intersection
Drainage to Storm Drain System

Conventional valley gutters may be used to transport runoff across local streets when 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 encroachment equivalent to that allowed on the street. Infrequently, with agency approval, valley gutters may be considered on collector streets.

3.2.2.1 Theoretical Capacity: The theoretical carrying capacity of each gutter approaching an intersection shall be calculated based upon the effective slope, as outlined herein.

Continuous Grade Across Intersection: When the gutter slope will be continued across an intersection—as when valley gutters are in place—use the slope of the gutter flow line crossing the street to calculate capacity.

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

Flow Interception by Inlet: When gutter flow will be intercepted by an inlet on continuous grade at the intersection, the effective gutter slope shall be utilized for calculations. Under this condition, the points for averaging shall be 0 feet, 25 feet, and 50 feet upstream from the inlet.

3.2.2.2 Allowable Capacity: The allowable carrying capacity for gutters approaching an intersection shall be calculated by applying the reduction factor found in Figure 3.4 to the theoretical capacity. The grade used to determine the reduction factor shall be the same effective grade used to calculate the theoretical capacity.

Special Considerations for Business Areas and Heavily-used Pedestrian Areas: In highly concentrated business areas where large volumes of pedestrian traffic are likely, consider using walk-over curbs (where the pavement grade is raised to match the curb elevation at the crosswalk) at intersections. If used, however, two catch basins would be required at nearly every corner as flow may not be allowed to continue around the corner. For the storm frequency being contemplated, the effect water may have on pedestrian walking area should be compatible with that on streets. Based upon vehicular traffic use in a business area all streets would probably be classified as collector or arterial, requiring a minimum of one water-free travel lane in each direction. The walk-over curbs should be available for limited pedestrian use.

Where concentration of pedestrians occurs, depth and area limitations may need modifications. As an example, streets adjacent to schools are arterials from a pedestrian standpoint and should be designed accordingly. Designing for pedestrian traffic is as important as designing for vehicular traffic. Ponding water and gutter flow wider than two feet is difficult for pedestrians to negotiate.

Where business buildings are constructed to property lines, the reduced clearance between buildings and heavy traffic must be considered. Splash from vehicles striking gutter flow may damage store fronts and make walking on sidewalks difficult.

Although not a necessity in many business areas, highly concentrated business areas should be designed to use reduced allowable pavement encroachment area and inundated areas, raised walk-over curbs at intersections, or additional catch basins to intercept flow before it reaches intersections. Generally, storm drains should be installed in these areas even if other criteria do not so indicate.

3.2.3 Design Criteria for Roadside Ditches

Roadside ditches are commonly used in rural areas to convey runoff from the highway pavement, and 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 5.

3.2.3.1 Permissible Use: Roadside ditches adjacent to public streets are discouraged in urban areas and require approval from the governing municipality. When they are allowed, adhere to the criteria outlined in this section.

3.2.3.2 Street Capacity for the Design Storm: Depth of flow in roadside ditches for the design storm shall be limited to preclude saturation of the adjacent roadway subgrade. Where curbs exist and roadside ditches are used in lieu of storm drains, catch basins or scuppers should be provided as needed to drain the pavement into the drainage ditch while meeting the criteria set forth in Table 3.1.

3.2.3.3 Geometry: Geometric considerations in the design of channel cross-sections should incorporate hydraulic requirements for the design discharge, safety, minimization of right-of-way acquisition, economy in construction and maintenance, and good appearance.

Channel side slopes should be as mild as practical. Slopes should be no steeper than 4:1 where terrain and right-of-way permit. The advantages of mild slopes are that the potential for erosion and slides is lessened, maintenance is eased, and the safety of errant vehicles is enhanced. Safety considerations are subject to the requirements of the local jurisdiction.

Trapezoidal channel bottoms should be a minimum of 4-feet wide for maintenance purposes. V-shaped channels may also be used when approved by the governing municipality.

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.

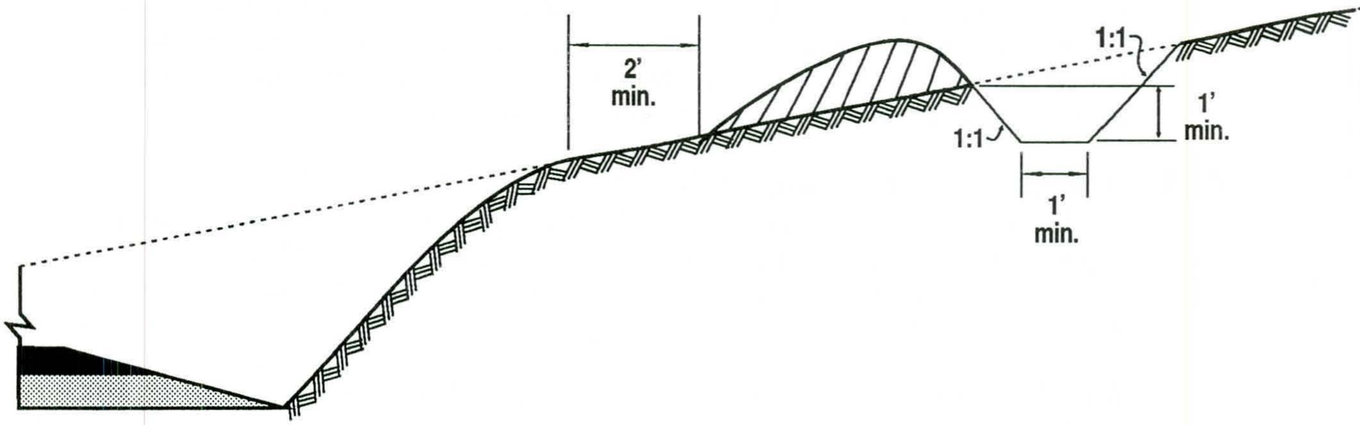


Figure 3.6
Crown Ditch
(Hillside Cuts Only)

3.2.3.4 **Rural Crown Ditch:** In mountainous terrain where large cuts are required, crown ditches constructed on top of the cut embankment will intercept runoff preventing it from eroding the face of the cut slope. A typical crown ditch is shown in Figure 3.6.

3.3 Catch Basins

Proper surface drainage of streets and highways may require intercepting excess flows with stormwater catch basins. A stormwater inlet or catch basin is an opening into a storm drain system to permit entrance of surface storm runoff. The most upstream catch basin in the system should be placed as far downstream as possible, because as soon as the runoff enters the pipe system, it is carried rapidly downstream which tends to reduce the time of concentration. The placing of catch basins is dictated by street encroachment and flow depth criteria (see Table 3.1).

3.3.1 Catch Basin Types

There are four categories of catch basins:

- curb opening catch basins
- grated catch basins
- combination catch basins
- slotted drain catch basins

Catch basins may be further classified as being on a continuous grade or in a sump. The *continuous grade* condition exists when the street grade is continuous past the catch basin and the water can flow past. The *sump condition* exists whenever 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 when the catch basin is located at an intersection.

Two or more catch basins placed close to each other may act as one hydraulic unit.

3.3.1.1 Curb Opening Catch Basins: Curb opening catch basins (Figure 3.7a) are best for use when a sump condition exists. Although a curb opening catch basin will not guarantee against plugging by debris, it is the most desirable type of catch basin.

A curb opening catch basin is a vertical opening in a curb through which the gutter flow passes. For safety reasons, the vertical opening should not be greater than 6 inches. The gutter may be undepressed or depressed in the area of the curb opening. The capacity of the curb opening is significantly increased by depressing the opening. Permissible curb opening catch basins are contained in the Maricopa Association of Governments (MAG) Standard Details. All details and inlet types must be approved by the governing municipality. A characteristic of the curb opening catch basin is its relative inefficiency on streets of steep grade. Therefore, curb opening catch basins that are 5 feet long or less are not recommended on continuous grades. Longer catch basins can be quite efficient, however, and should be considered for use on streets with slopes up to 1.0 percent.

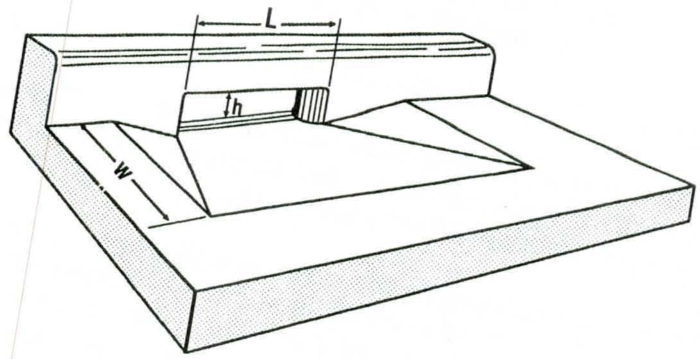
3.3.1.2 Grated Catch Basin: 'Grated' or 'gutter' catch basin refers to an opening in the gutter covered by one or more grates through which the water falls (see Figure 3.7b). As with other catch basins, grated catch basins may be either depressed or undepressed and are more efficient than curb opening catch basins when located on a continuous grade. Permissible grated catch basins are contained in the MAG Standard Details. Other details must be approved by the governing municipality.

When grated catch basins are used, the engineer should design them to optimize hydraulic efficiency, bicycle and pedestrian safety, structural adequacy, economy, and freedom from clogging (see *Combination Catch Basin*, below, for recommendations).

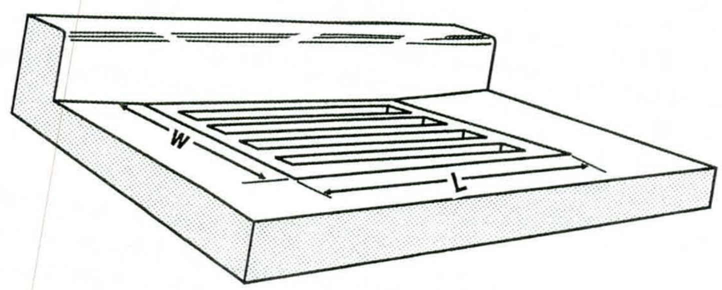
3.3.1.3 Combination Catch Basin: A combination catch basin is composed of a curb opening and a grated catch basin acting as a unit (see Figure 3.7c). Usually the gutter opening is placed adjacent to the curb opening. As with other catch basins, a combination catch basin may be either depressed or undepressed and located in a sump or on a continuous grade.

For the widest range of conditions, the combination catch basin is the most efficient type of stormwater catch basin for hydraulic interception capabilities and eliminating debris clogging. Permissible combination catch basins are contained in the MAG Standard Details. Other details must be approved by the governing municipality.

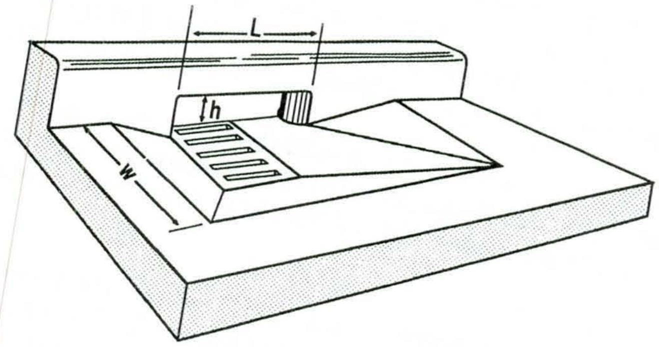
3.3.1.4 Slotted Drain Catch Basins: A slotted drain (Figure 3.7d) is a slot opening in the pavement which intercepts sheet flow and conveys it through a corrugated steel pipe. Slotted drains are most effective when street slopes are shallow.



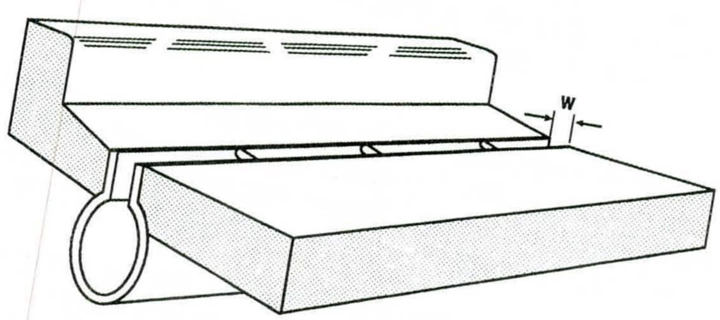
(a)
Curb Opening Catch Basin



(b)
Grated Catch Basin



(c)
Combination Catch Basin



(d)
Slotted Drain Catch Basin

Figure 3.7
Catch Basins

3.3.2 Catch Basin Applications

The following general recommendations are made for different types of stormwater catch basins.

3.3.2.1 Sump Conditions

True Sump: Depressed curb opening catch basins are recommended for true sump conditions. Each true sump should be reviewed to determine if the area affected by ponding is within acceptable limits. The ponding caused by the 100-year flow should also be checked to assure that the 100-year inundation criteria are met. True sumps should be designed with care, considering the flow path that will be taken by flows in excess of the design flow. In some cases a drainage easement may be necessary to prevent damage to adjacent property during storms in excess of the design storm or in case of total blockage of the inlet by debris. Placement of an inlet should not make conditions worse than they were before. This is an important consideration in retrofit situations.

Sumps Formed by Crown Slope of Cross-Street at Intersection: Curb opening catch basins are recommended for these conditions, although combination catch basins may be used successfully. Conditions need to be checked to prevent a small amount of ponding from causing excessive lane encroachment or storm runoff flowing over the crown of the cross street and continuing down the gutter.

3.3.2.2 Continuous Grade Conditions: Curb opening catch basins should be used on continuous grades, unless otherwise approved by the governing municipality. Inlet spacing should be limited to a maximum of 660 feet for collection of nuisance flows.

3.3.2.3 Shallow Sheet Flow Condition: Slotted drains may be used as permitted by the governing municipality.

3.3.2.4 Large Inlet Inflows: In areas where large inflows are admitted into the storm drain system, hazards may be introduced to vehicular and pedestrian traffic due to large depths and velocities. Special consideration should be given to design of large inlet facilities. When possible, large inlet facilities should be located off of the roadway and provided with trash racks and fencing to improve safety.

3.3.3 Allowable Catch Basin Capacities

The reduction factors outlined in Table 3.2 should be applied to the theoretical calculated capacity of catch basins based upon their type and function. The reduction factors compensate for effects which decrease the capacity of the catch basin such as debris plugging, pavement overlaying, and in variations of design.

The allowable capacity of a catch basin should be determined by applying the applicable factor from Table 3.2 to the theoretical capacity calculated in accordance with the appropriate design charts.

The percentage of theoretical capacity allowed may be even lower when the catch basin is likely to intercept large amounts of sediment or debris. For instance, the first catch basin to a pipe network draining a high debris-yielding area may actually

Table 3.2
Reduction Factors to Apply to Catch Basins, F_R

Condition	Inlet Type	Theoretical Reduction Factor, F_R
Sump	Curb Opening	0.80
Sump	Grated	0.50
Sump	Combination	0.65
Continuous Grade	Curb Opening	0.80
Continuous Grade	Longitudinal Bar Grate Longitudinal Bar Grate with recessed transverse bars	0.75 0.60
Continuous Grade	Combination(1)	Apply factors separately to grate and curb opening and combination
Shallow Sheet Flow(2)	Slotted Drains	0.80

(1) See Section 3.3.4.3, Combination Catch Basins.

(2) Slotted drains are most effective for shallow sheet flow conditions. With greater depths and flows, a different type of inlet should be used.

accept only 20 percent of the theoretical capacity allowed because of clogging. Sediment traps will not be designed into the catch basin. A sediment trap formed by lowering the floor of the catch basin below the elevation of the outlet pipe is unnecessary and undesirable since there is too much turbulence for effective trapping at design flow rates, and cleaning is costly. Catch basins should be self-scouring, even under low-flow conditions.

3.3.4 Design Procedures

Figures 3.9 to 3.19 (pages 32 to 42) are capacity curves for standard catch basins. When designing a *nonstandard* catch basin, use the equations and procdures outlined herein. The approval of the governing municipality should be obtained before designing a nonstandard catch basin. The procedures and equations in this section are adapted from the Federal Highway Administration Hydraulic Engineering Circular No. 12 (HEC-12), *Drainage of Highway Pavements* (FHWA, 1984). Refer to Section 3.1 for definitions of coefficients used in the following equations.

3.3.4.1 Curb Opening Catch Basins

On-Grade: The length of curb opening catch basin required for total interception of gutter flow on a pavement section with a straight cross slope is expressed as:

$$L_T = 0.6Q^{0.42} S^{0.3} \left(\frac{1}{nS_x}\right)^{0.6} \tag{3.3}$$

The efficiency of curb opening catch basins shorter than the length required for total interception is:

$$E = 1 - \left(1 - \frac{F_R L}{L_T}\right)^{1.8} \tag{3.4}$$

Figure 3.20 (page 43) is a nomograph for the solution of Equation 3.3; Figure 3.21 (page 44) provides a solution of Equation 3.4.

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, in Equation 3.5.

$$S_e = S_x + S_w' E_o \tag{3.5}$$

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 3.3 becomes:

$$L_T = 0.6Q^{0.42} S^{0.3} \left(\frac{1}{nS_e}\right)^{0.6} \tag{3.6}$$

Equation 3.4 is applicable with either straight cross slopes or compound cross slopes. Figures 3.20 and 3.21 are applicable to depressed curb opening catch basins using S_e rather than S_x.

Equation 3.5 uses the ratio E_o in the computation of the equivalent cross slope, S_e. Figure 3.22 (page 45) can be used to determine spread, and then Figure 3.23 (page 46) can be used to determine E_o.

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. At water depths between 1.0 and 1.4 times the opening height, flow is in a transition stage.

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 catch basin. Limited experiments and extrapolation of the results of tests on depressed catch basins indicate that the weir coefficient for curb opening catch basins without depression is approximately equal to that for a depressed curb opening inlet.

The equation for the interception capacity of a depressed curb opening catch basin operating as a weir is:

$$Q_i = C_w (L + 1.8W) d^{1.5} \tag{3.7}$$

where $C_w = 2.3$.

The weir equation is applicable for water depths at the curb less than or approximately equal to the height of the opening plus the depth of the depression. Thus, the limitation on the use of Equation 3.7 for a depressed curb opening catch basin is:

$$d \leq h + \frac{a}{12}$$

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 3.7 will yield conservative estimates of the interception capacity.

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

$$Q_i = C_w \, L d^{\,1.5} \qquad \textbf{(3.8)}$$

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

Curb opening catch basins operate as orifices at depths greater than approximately 1.4h. The interception capacity can be computed by Equation 3.9:

$$Q_i = C_o \, hL \, (2gd_o)^{0.5} \qquad \textbf{(3.9)}$$

Equation 3.9 is applicable 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 3.9 assumes a vertical orifice opening. As illustrated in Figure 3.8, other orifice throat locations can change the effective depth on the orifice and the dimension (d_i – h/2). 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.

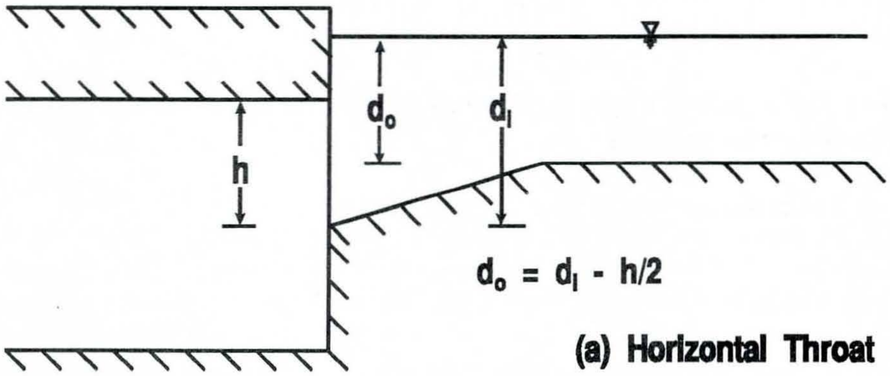
The orifice equation for curb opening catch basins with other than vertical faces (see Figure 3.8) is:

$$Q = C_o \, hL \, (2gd_o)^{0.5} \qquad \textbf{(3.10)}$$

Figure 3.24 (page 47) provides solutions for Equations 3.7 and 3.9 for depressed curb opening catch basins, and Figure 3.25 (page 48) provides solutions for Equations 3.8 and 3.9 for curb opening catch basins without depression. Figure 3.26 (page 49) is provided for use for curb openings with inclined or vertical orifice throats.

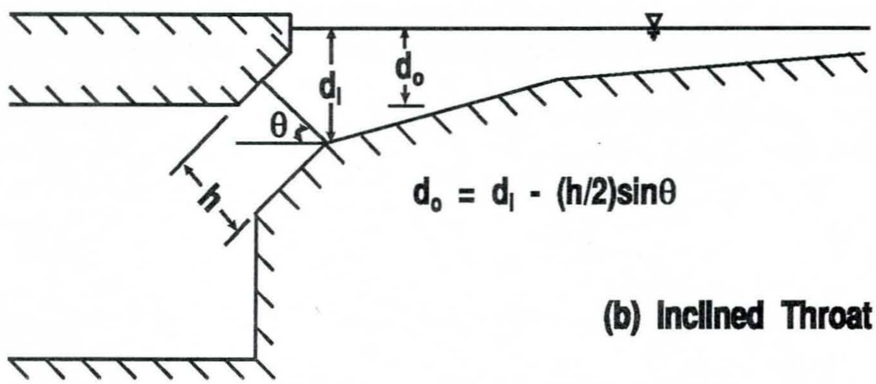
3.3.4.2 Grated Catch Basins

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.



$Q = 0.67hL\sqrt{2gd_o}$

L = Length of opening



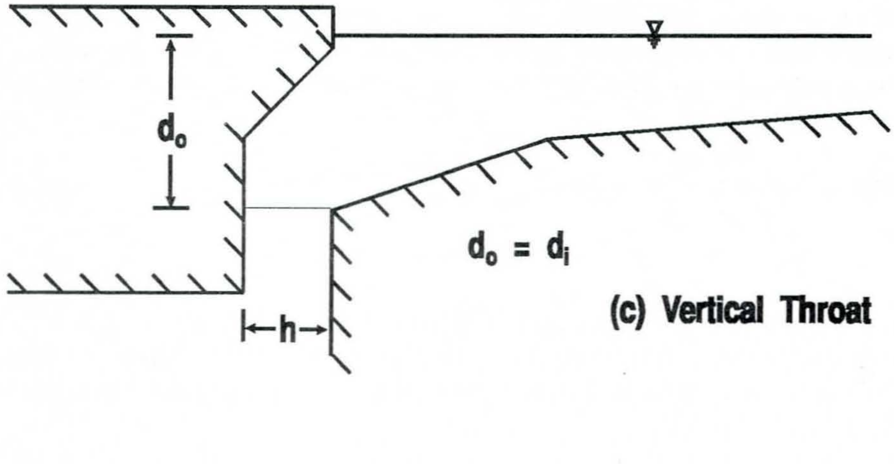


Figure 3.8

Curb Opening Catch Basins

(modified from: FHA, 1984, HEC-12)

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

$$E_o = \frac{Q_w}{Q} = 1 - \left(1 - \frac{W}{T}\right)^{2.67} \quad \textbf{(3.11)}$$

Figure 3.23 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 is:

$$\frac{Q_s}{Q} = 1 - \frac{Q_w}{Q} = 1 - E_o \quad \textbf{(3.12)}$$

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

$$R_f = 1 - 0.09\,(V - V_o) \quad \textbf{(3.13)}$$

This ratio is equivalent to frontal flow interception efficiency. Figure 3.27 (page 50) provides a solution of Equation 3.13 which takes into account grate length, bar configuration and gutter velocity at which splash-over occurs. The gutter velocity needed to use Figure 3.27 is total gutter flow divided by the area of flow.

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

$$R_s = \frac{1}{\left(1 + \dfrac{0.15V^{1.8}}{S_x L^{2.3}}\right)} \quad \textbf{(3.14)}$$

Figure 3.28 (page 51) provides a solution of Equation 3.14.

A deficiency in developing empirical equations and charts from experimental data is evident in Figure 3.28. 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.

The efficiency, E, of a grate is:

$$E = R_f E_o + R_s (1 - E_o) \quad \textbf{(3.15)}$$

The first term on the right side of Equation 3.15 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 of a grate catch basin on grade is equal to the efficiency of the grate multiplied by the total gutter flow:

$$Q_i = EQ = Q\,[R_f\,E_o + R_s\,(1 - E_o)] \tag{3.16}$$

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, i.e., (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:

$$Q_i = C_w\, Pd^{\,1.5} \tag{3.17}$$

where $C_W = 3.0$

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

$$Q_i = C_o\, A\,(2gd)^{0.5} \tag{3.18}$$

Use of Equation 3.18 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–1–1/8 grates, the clear opening is equal to the total area of the grate less the area occupied by longitudinal and lateral bars.

Figure 3.29 (page 52) is a plot of Equations 3.17 and 3.18 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 interception capacity less than that computed by either the weir or the orifice equation. This capacity can be approximated by drawing in a curve between the lines representing the perimeter and net area of the grate to be used.

3.3.4.3 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 alone. Capacity is computed by neglecting the curb opening. A combination catch basin is sometimes used with the curb opening or a part of the curb opening placed upstream of the grate. The curb opening in such an installation intercepts debris which might otherwise clog the grate and has been termed a "sweeper" by some. A combination 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 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 3.17 or Figure 3.29 can be used for weir flow in combination catch basins in sump locations. Assuming complete clogging of the grate, Equations 3.7, 3.8, and 3.9, or Figures 3.24, 3.25, and 3.26 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 Equations 3.18 and 3.10:

$$Q_i = 0.67\,A_g(2gd)^{0.5} + 0.67hL(2gd_o)^{0.5} \tag{3.19}$$

Trial and error solutions are necessary for depth at the curb for a given flow rate using Figures 3.29, 3.24, and 3.25 for orifice flow.

3.3.4.4 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.

Slotted drain catch basins are effective pavement drainage catch basins which have a variety of applications. They can be used on curbed or uncurbed sections and offer little interference to traffic operations.

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 3.3. Figure 3.20 is therefore applicable for both curb opening catch basins and slotted drain catch basins. Similarly, Equation 3.4 is also applicable to slotted drain catch basins and Figure 3.21 can be used to obtain the catch basin efficiency for the selected length of the catch basin.

Using Figures 3.20 and 3.21 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 Equation 3.20:

$$Q_i = 0.8LW(2gd)^{0.5} \qquad \textbf{(3.20)}$$

where: d = depth of water at slot, ft $d \geq 0.4$ ft

For a slot width of 1.75 inches or greater, Equation 3.20 becomes:

$$Q_i = 0.94Ld^{\,0.5} \qquad \textbf{(3.21)}$$

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 3.30 (page 53) provides solutions for weir flow, Equation 3.21, and a plot representing data at depths between weir and orifice flow.

3.3.5 Design Aids

The following inlet capacity curves have been developed using procedures set forth in HEC-12 and are to be used for selecting Standard MAG inlets for pavement drainage design in Maricopa County. The curves are developed for a street cross slope of two percent and do not include reduction factors to account for plugging. The reduction factors contained in Table 3.2 should be applied to the theoretical catch basin capacity from the curves to obtain the interception capacity used for design.

Additional design aids are included from HEC-12 for determining the interception capacity of inlets that do not have curves provided.

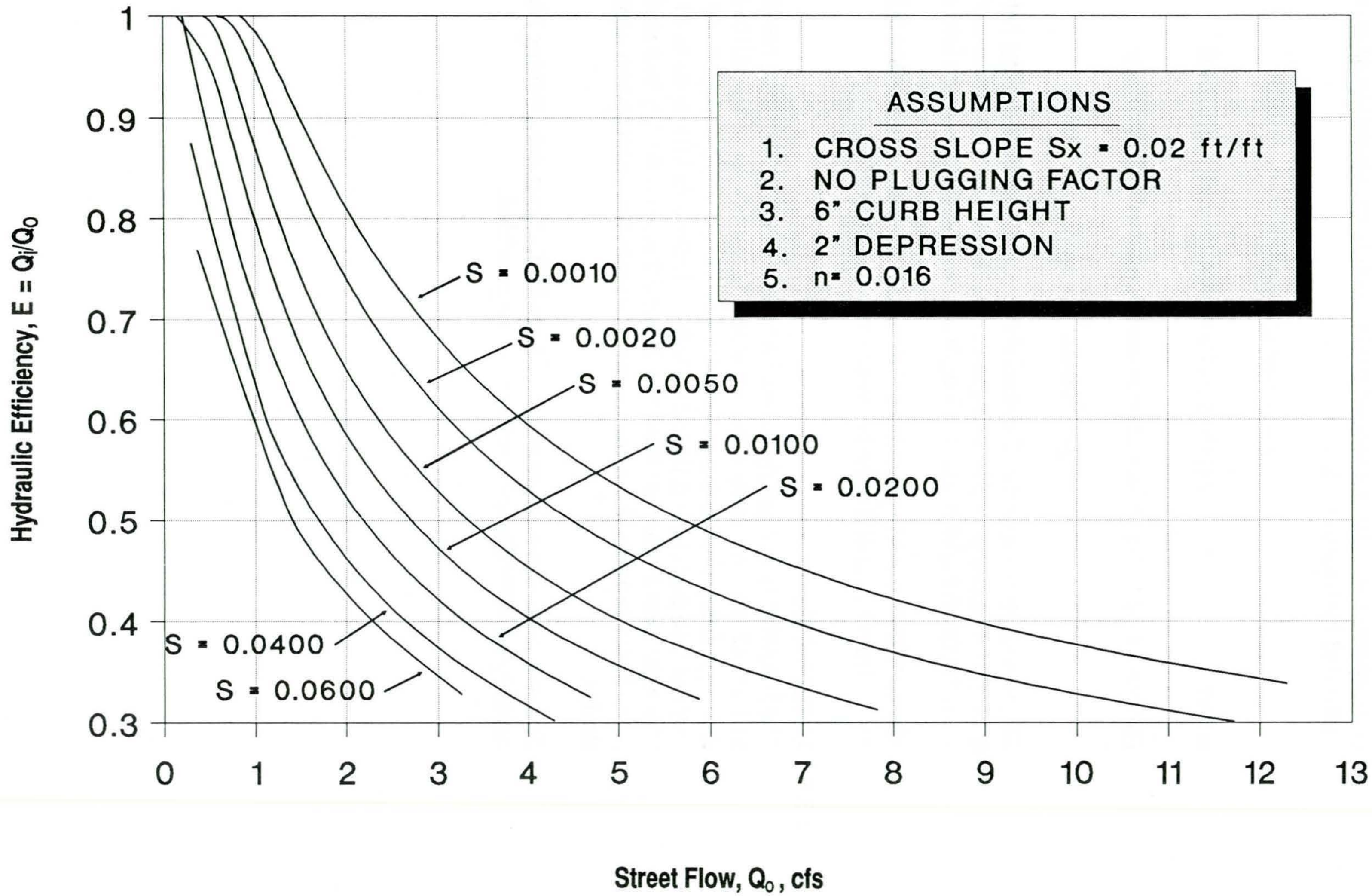


Figure 3.9
Curb Opening Inlet Capacity Curves
MAG Detail 530, 3'-6" Curb Opening Inlet

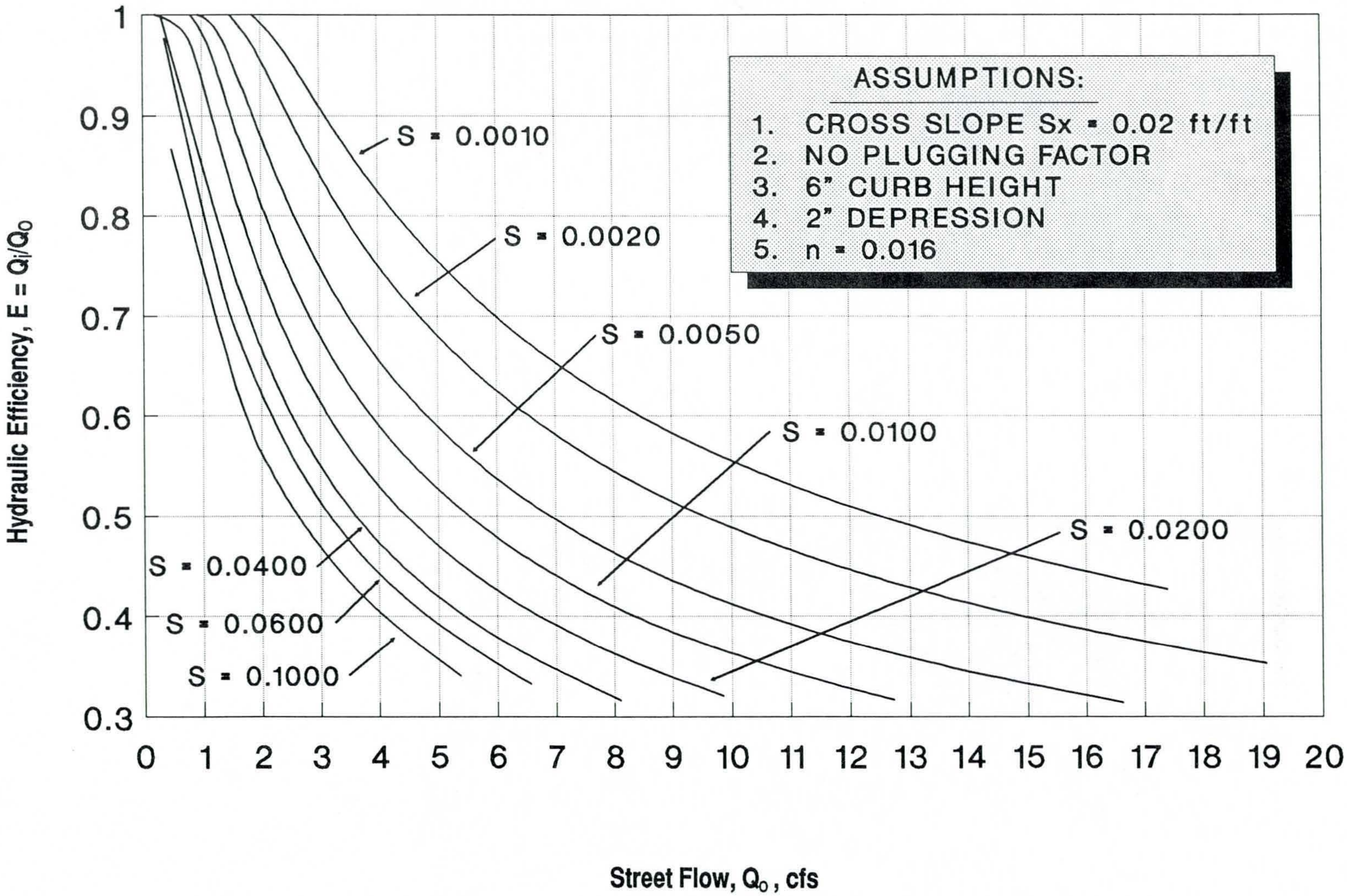


Figure 3.10
Curb Opening Inlet Capacity Curves
MAG Detail 531, 5'-6" Curb Opening Inlet

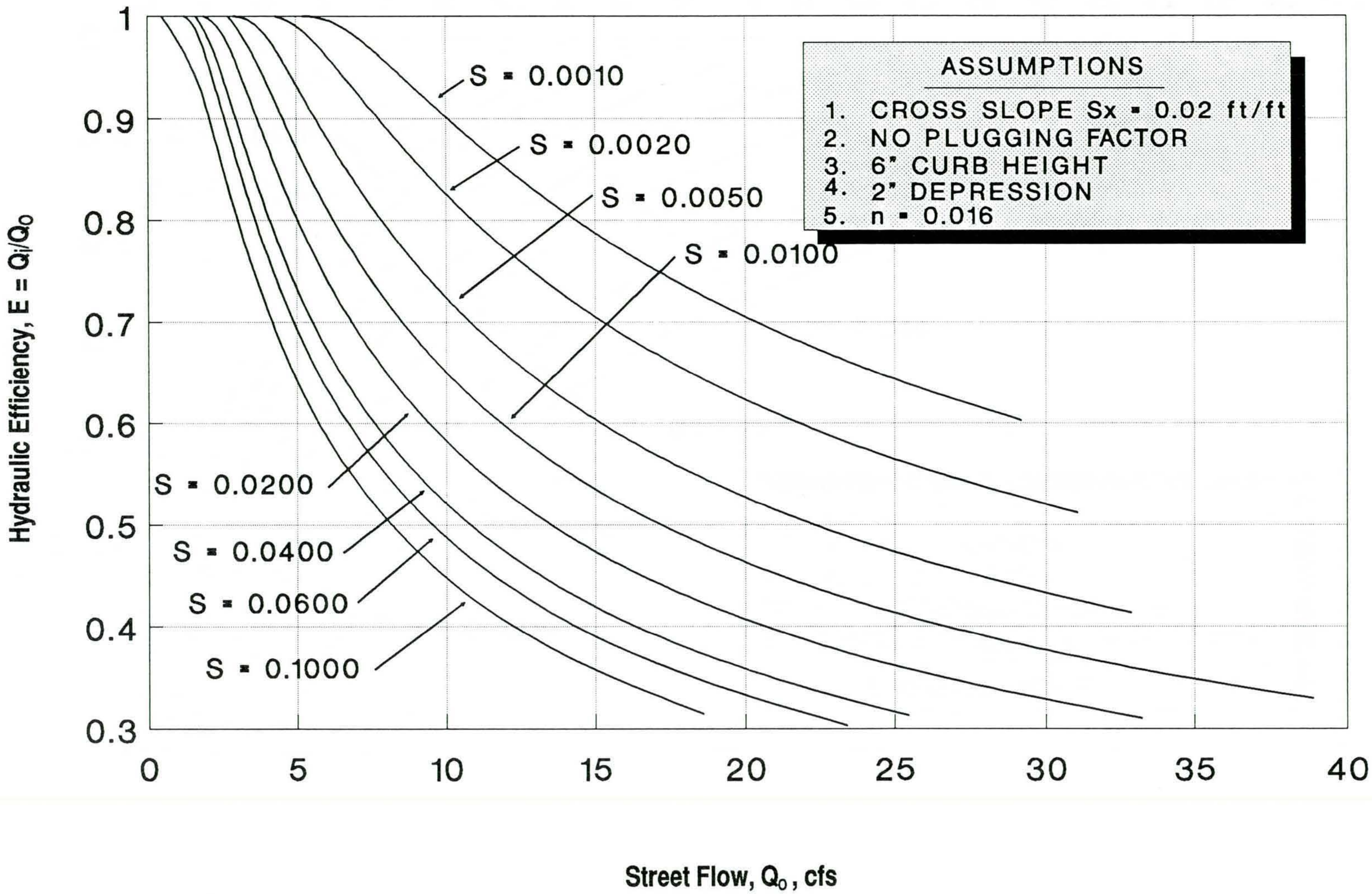


Figure 3.11
Curb Opening Inlet Capacity Curves
Double MAG Detail 531, 11' Curb Opening

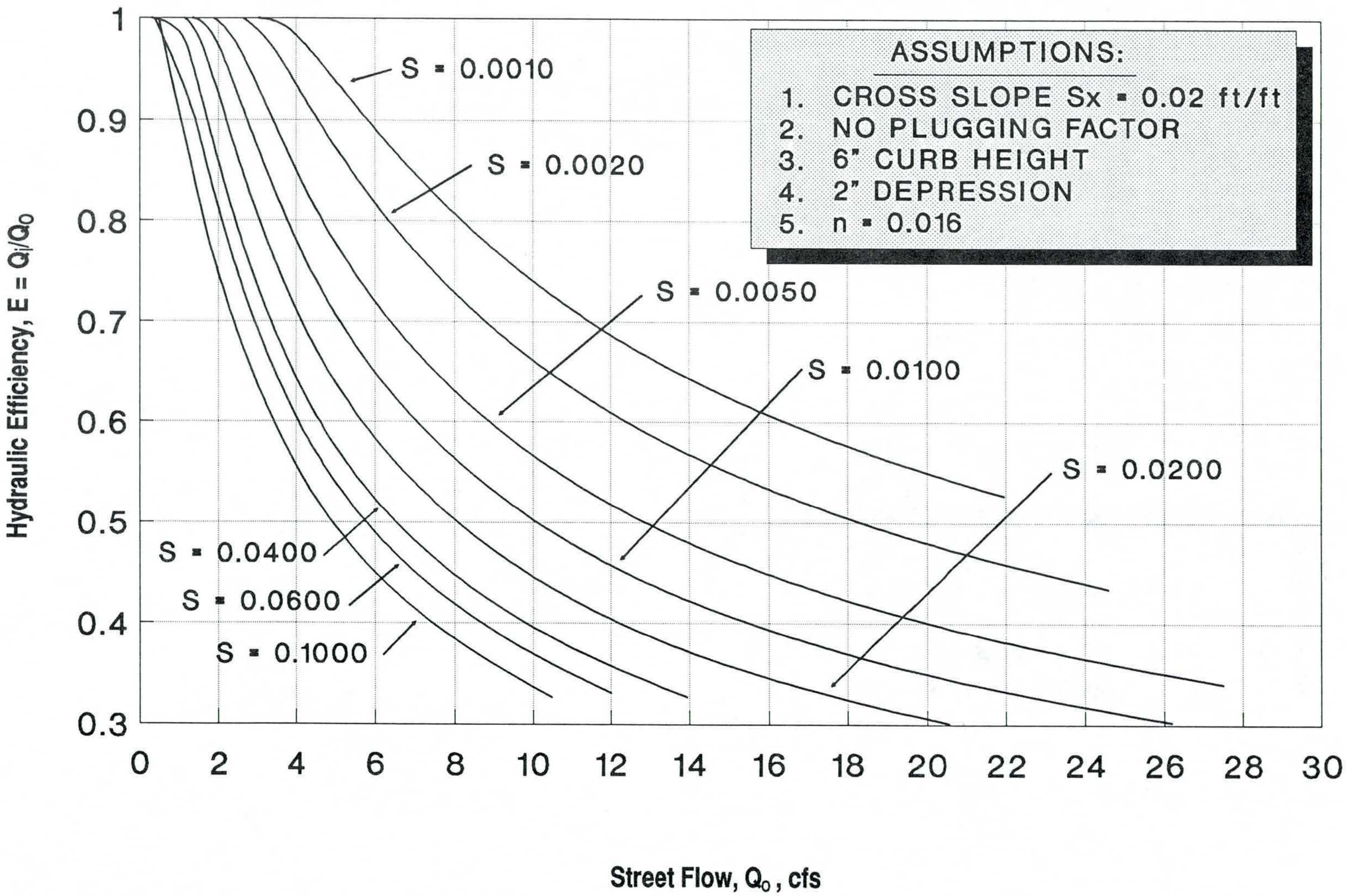


Figure 3.12
Curb Opening Inlet Capacity Curves
MAG Detail 532, 8' Curb Opening Inlet

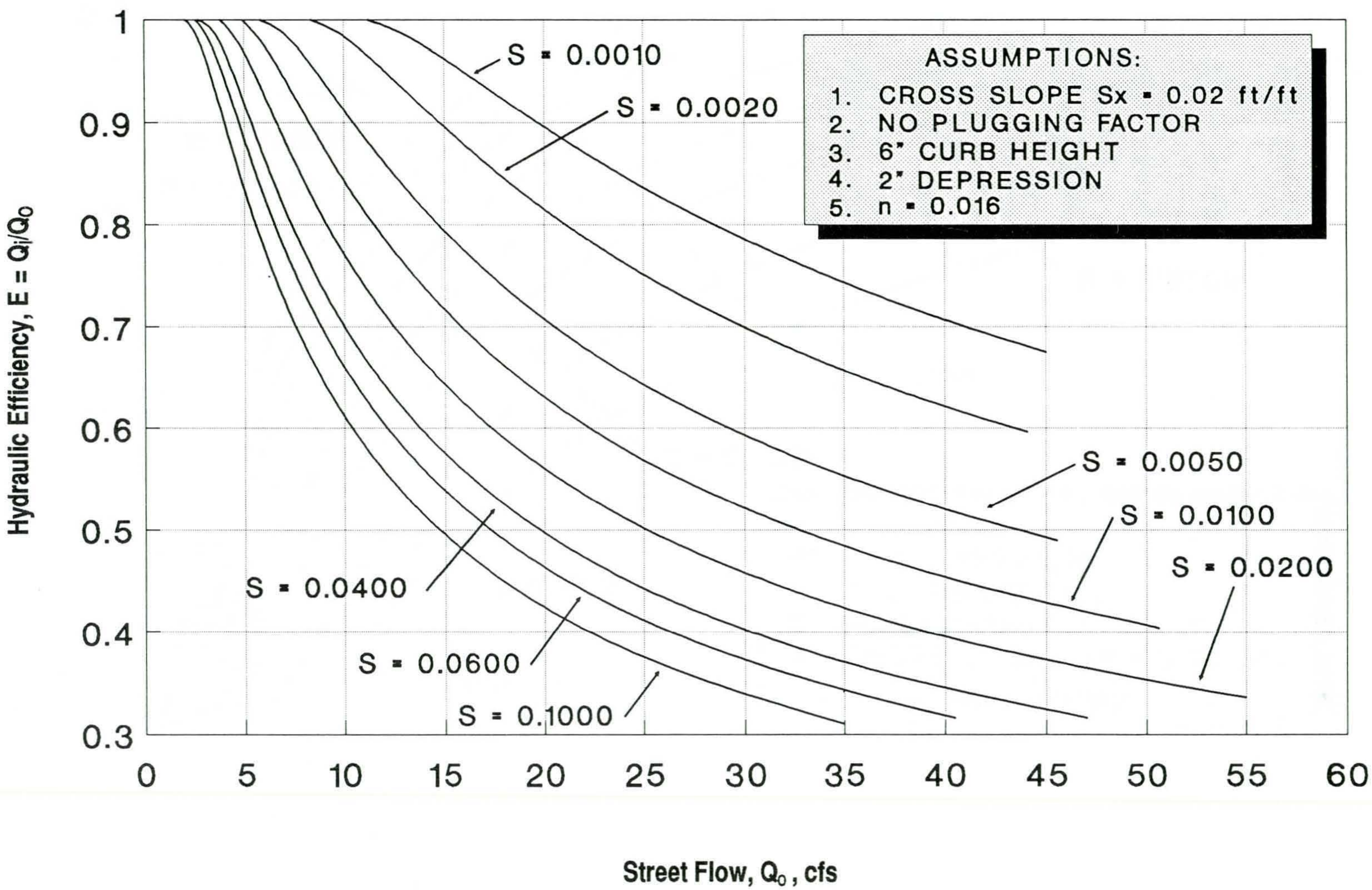


Figure 3.13
Curb Opening Inlet Capacity Curves
Double MAG Detail 532, 16' Curb Opening

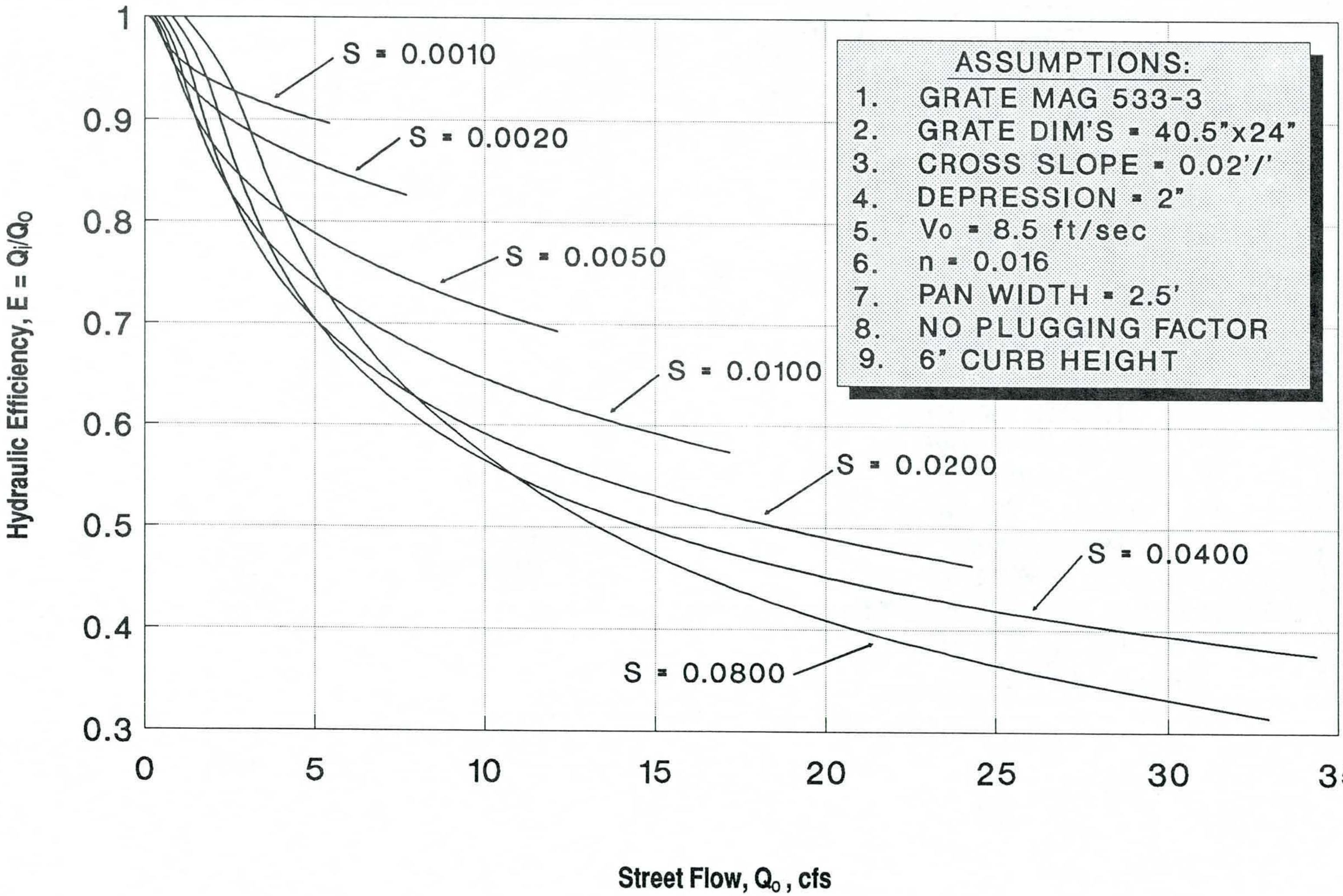


Figure 3.14
Combination Inlet Capacity Curves
MAG Detail 533-1, 3' Combination Inlet

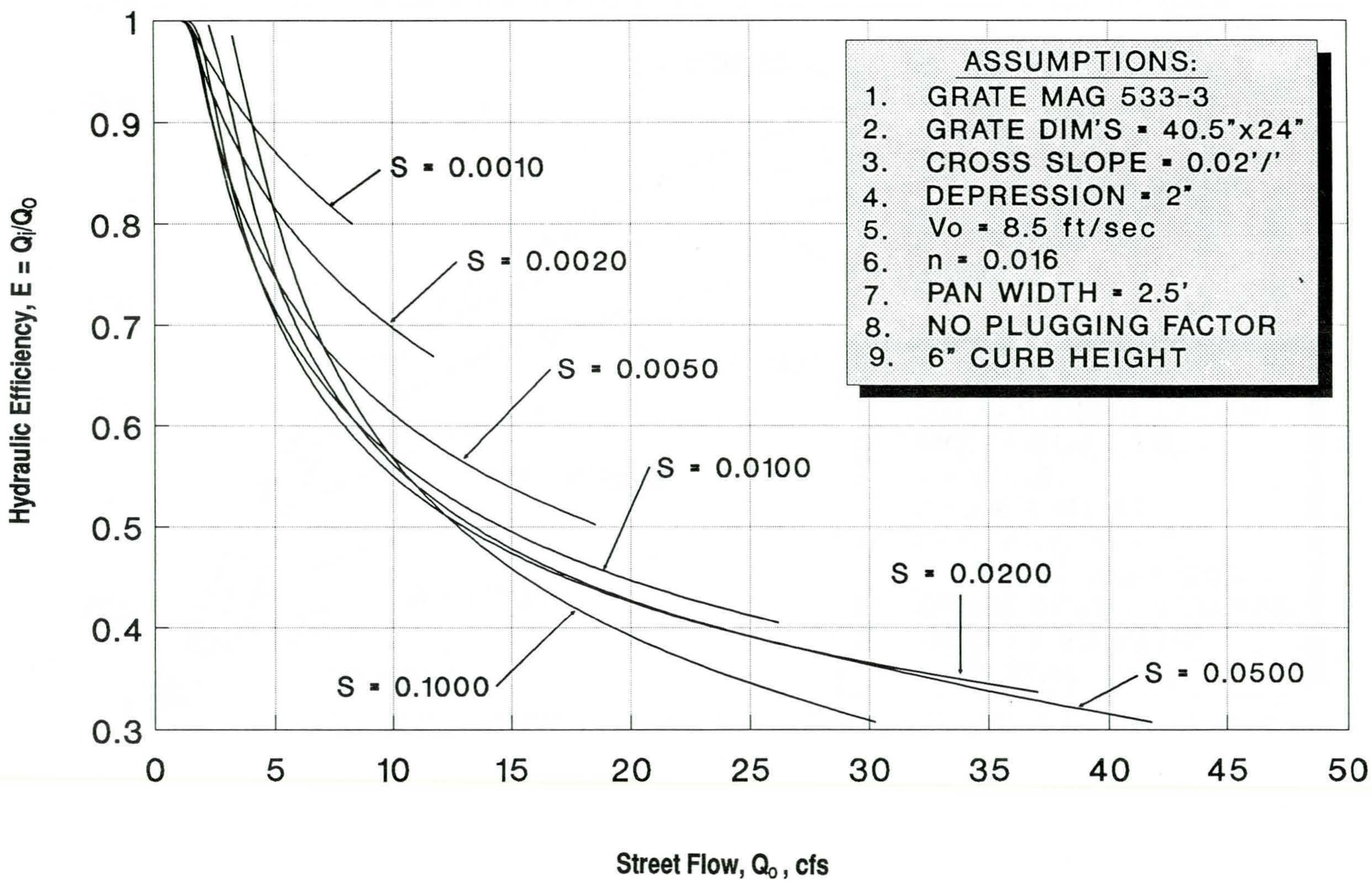


Figure 3.15
Combination Inlet Capacity Curves
MAG Detail 533-1, 6' Combination Inlet

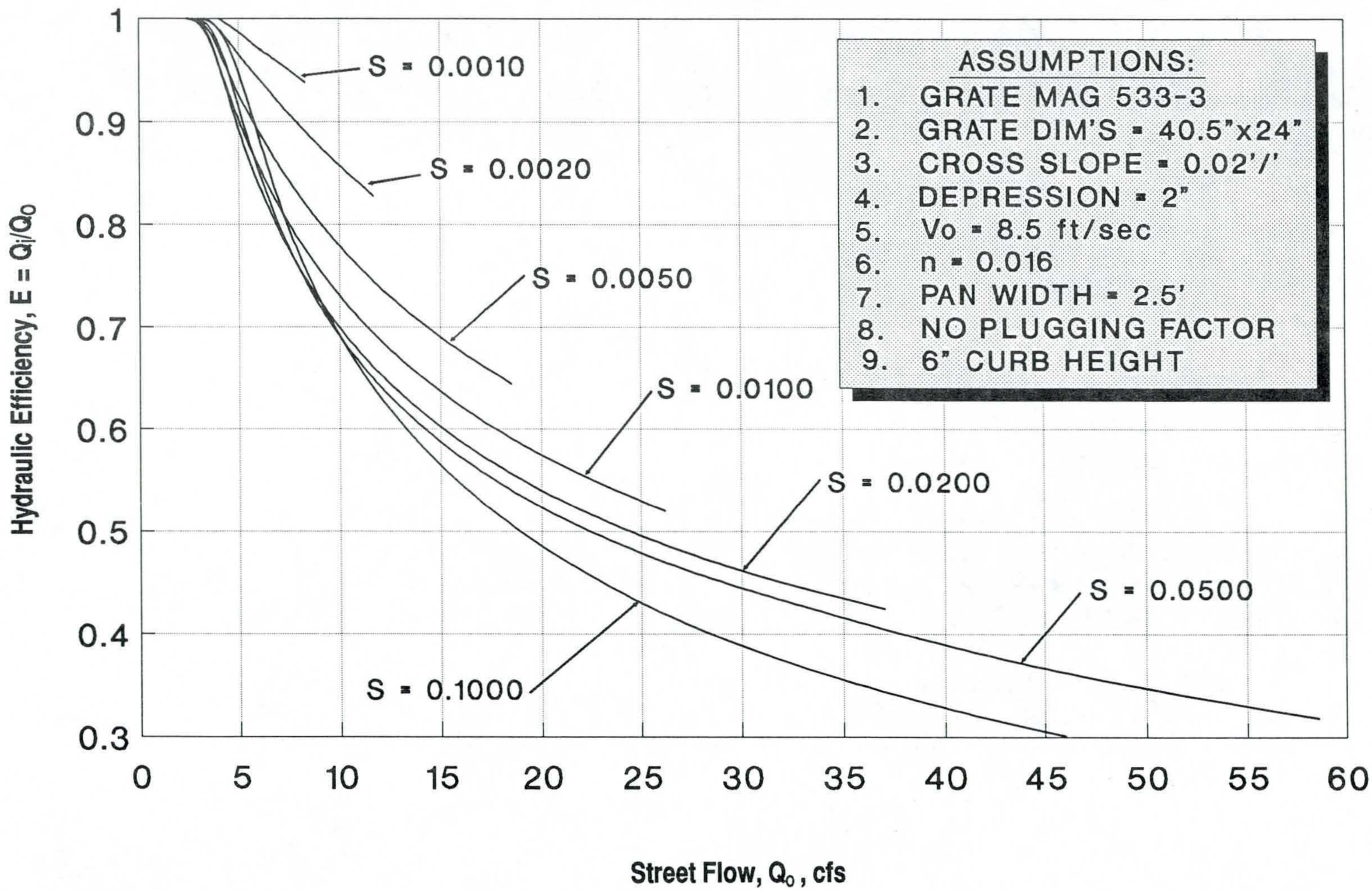


Figure 3.16
Combination Inlet Capacity Curves
MAG Detail 533-1, 10' Combination Inlet

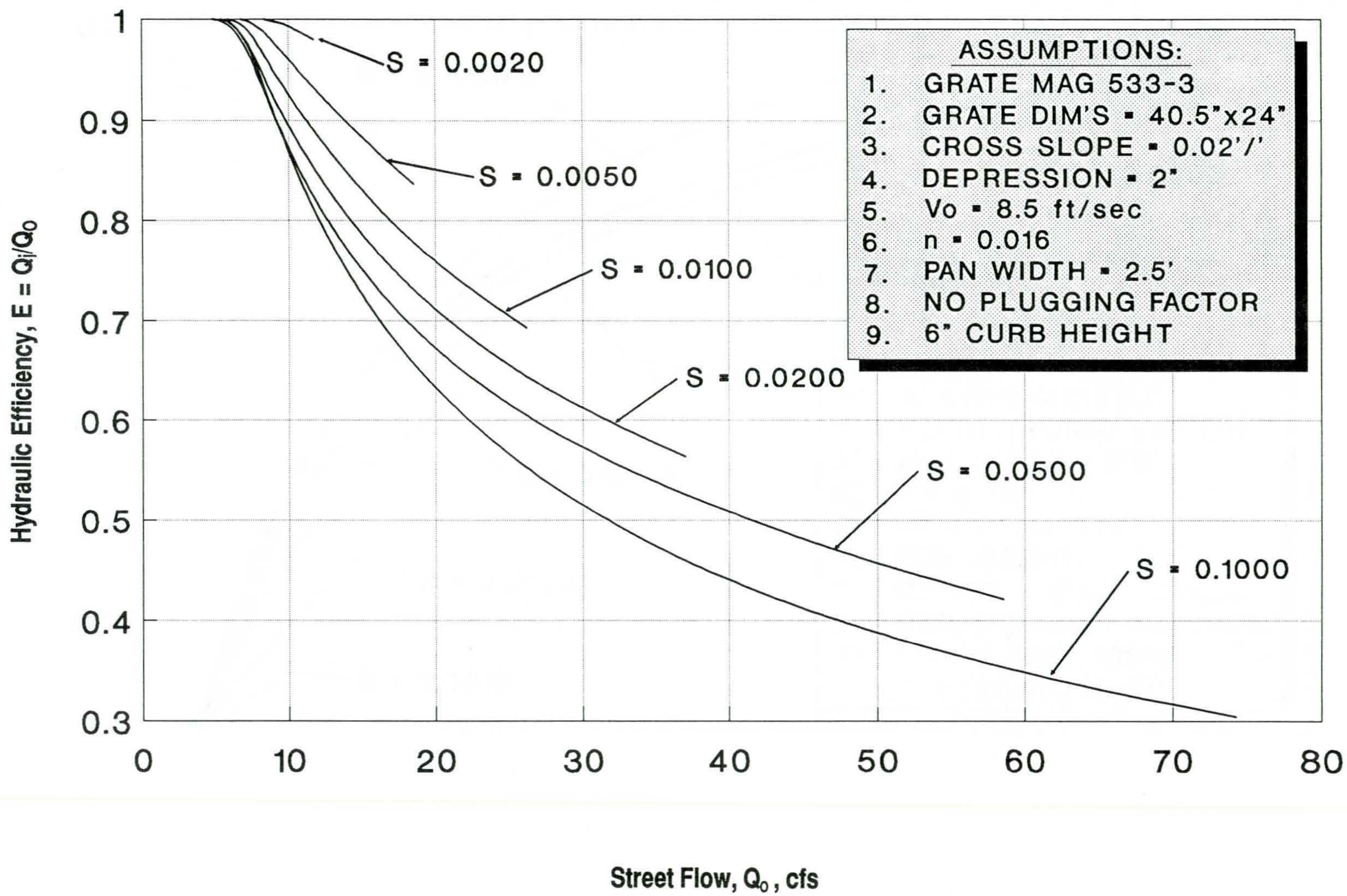


Figure 3.17
Combination Inlet Capacity Curves
MAG Detail 533-1, 17' Combination Inlet

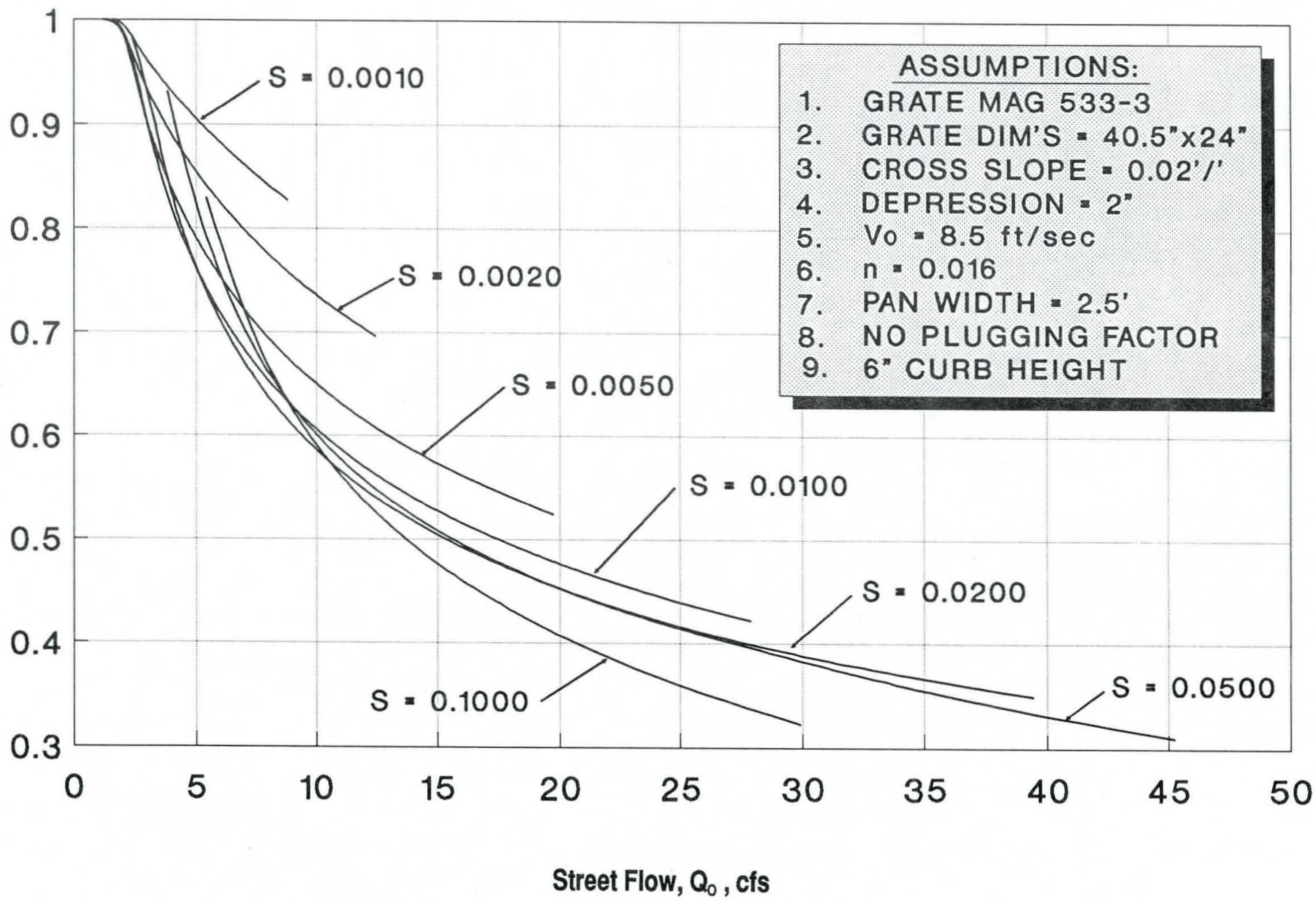


Figure 3.18
Combination Inlet Capacity Curves
MAG Detail 533-2, 7' Combination Inlet

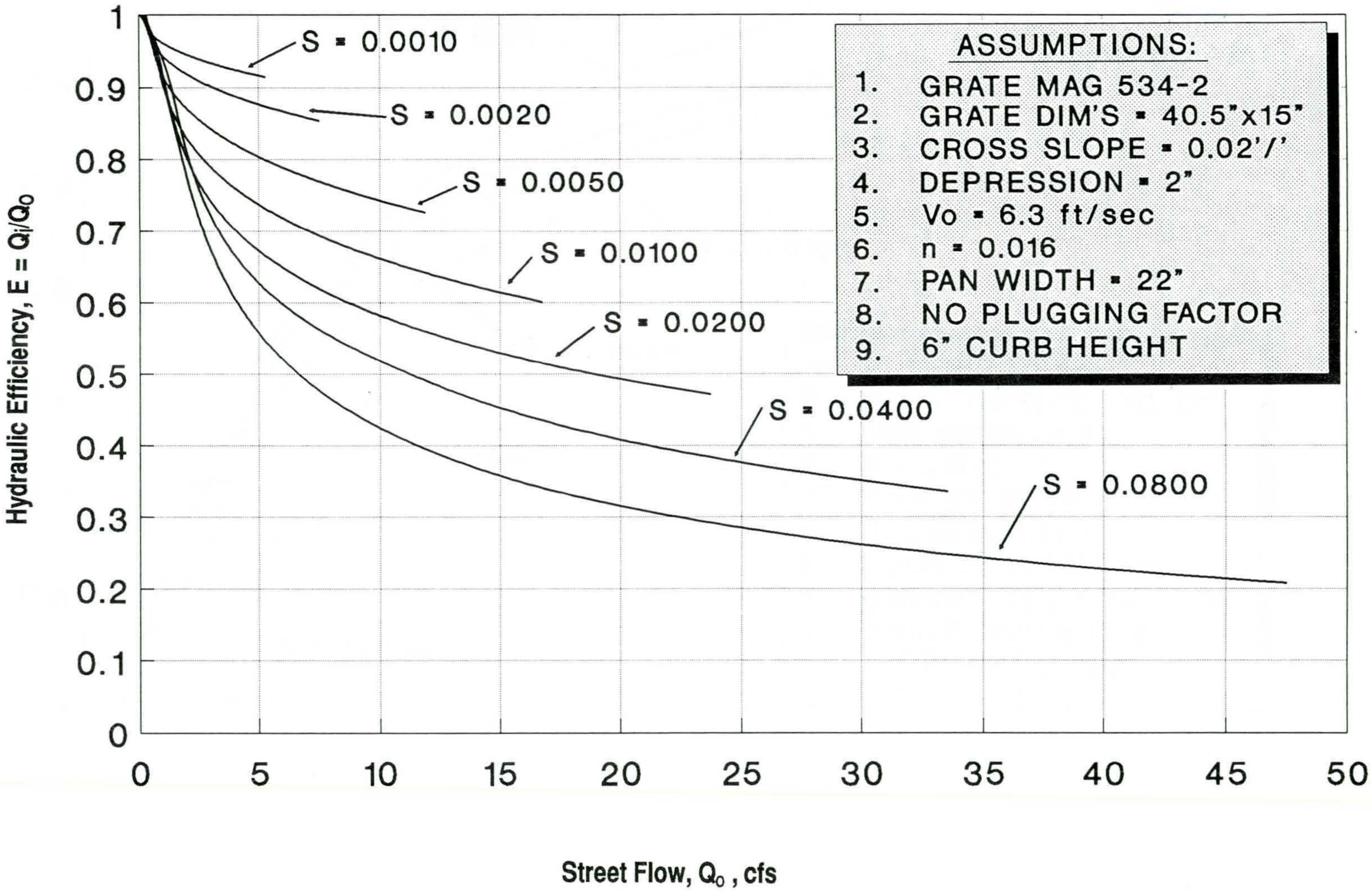


Figure 3.19
Grate Inlet Capacity Curves
MAG Detail 534, 3' Grate Inlet

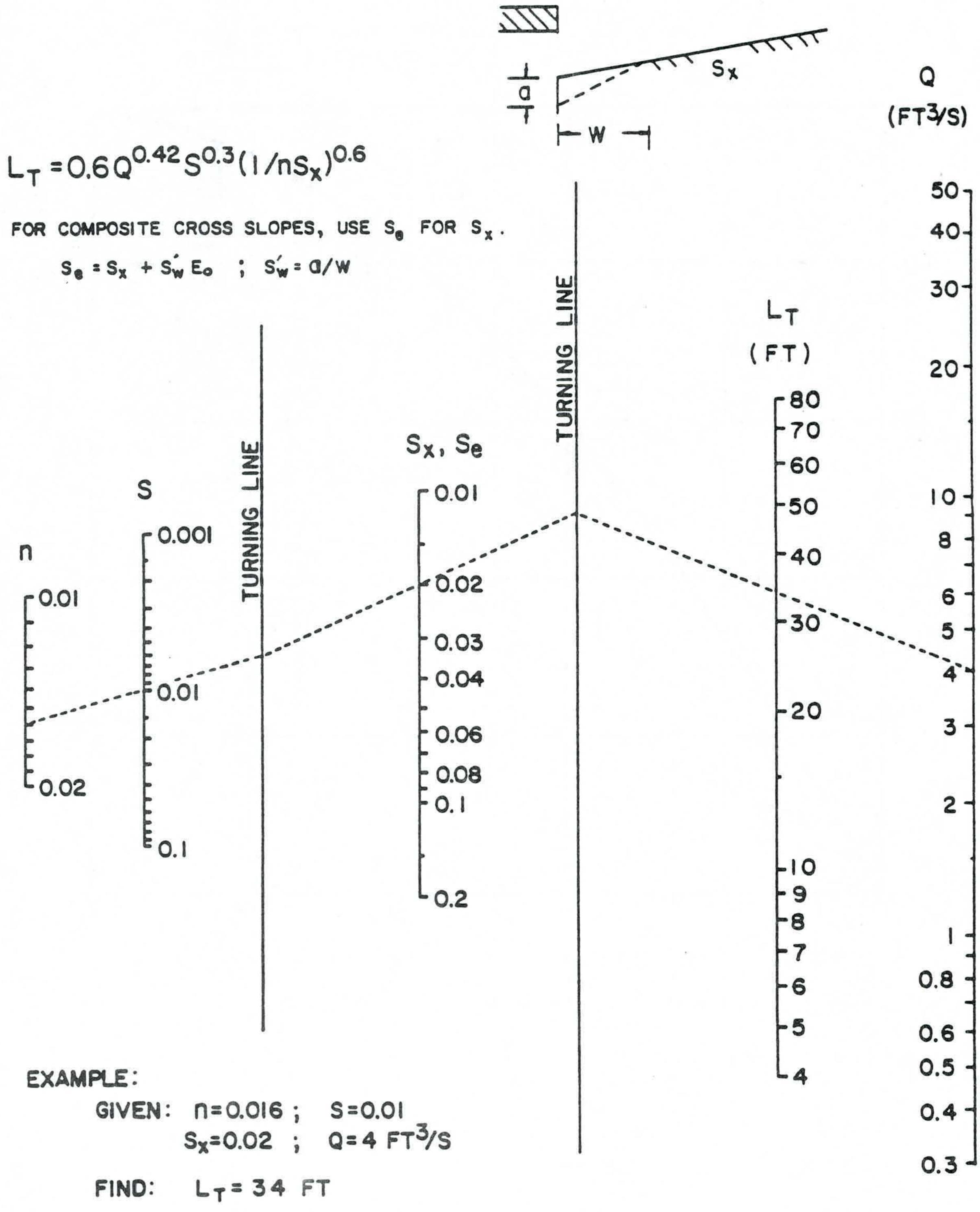


Figure 3.20
Curb Opening and Slotted Drain Inlet Length for Total Interception
(source: FHWA, 1984, HEC-12)

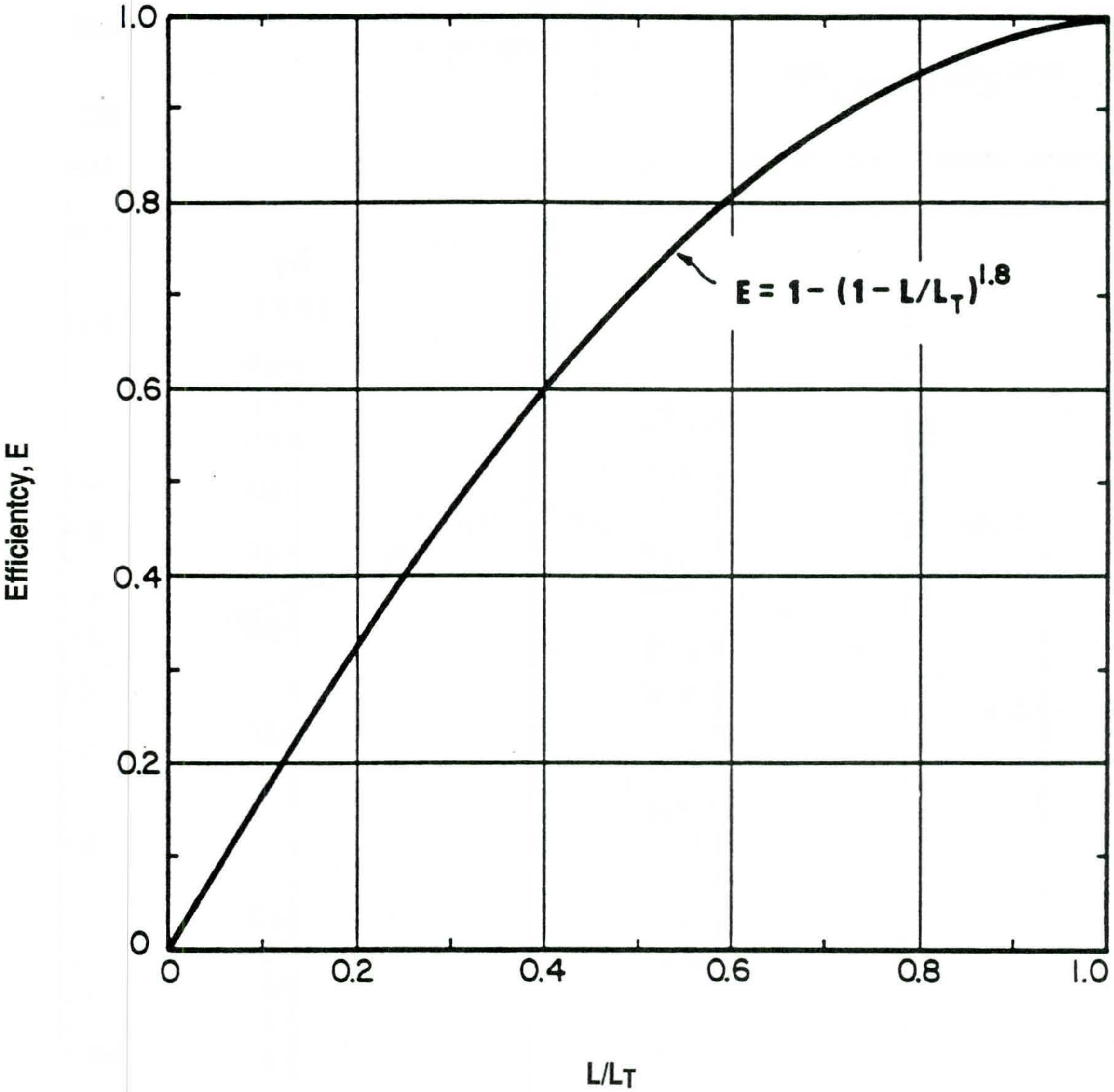


Figure 3.21
Curb Opening and Slotted Drain Inlet Interception Efficiency
(source: FHWA, 1984, HEC-12)

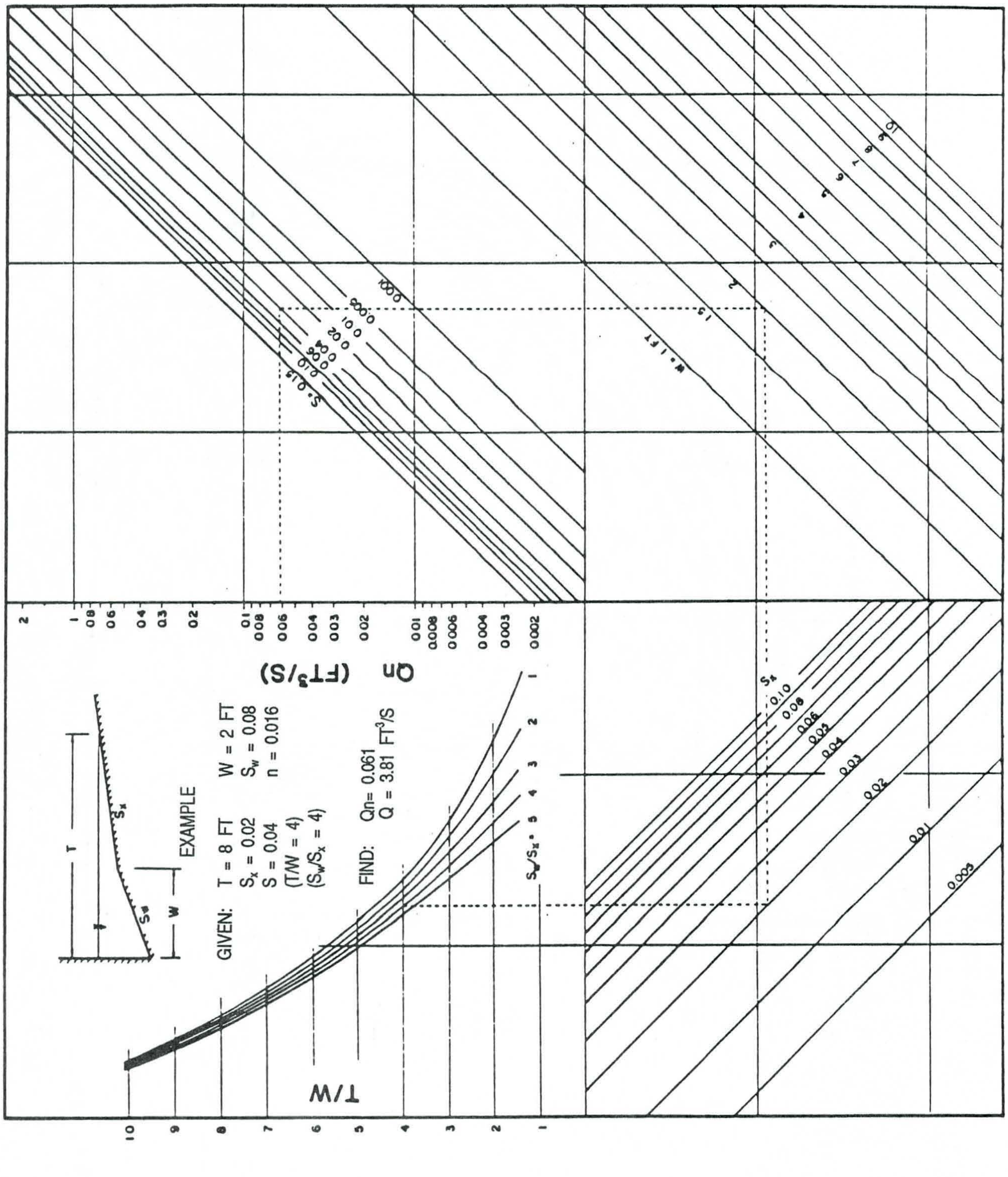


Figure 3.22
Flow in Composite Gutter Sections
(source: FHWA, 1984, HEC-12)

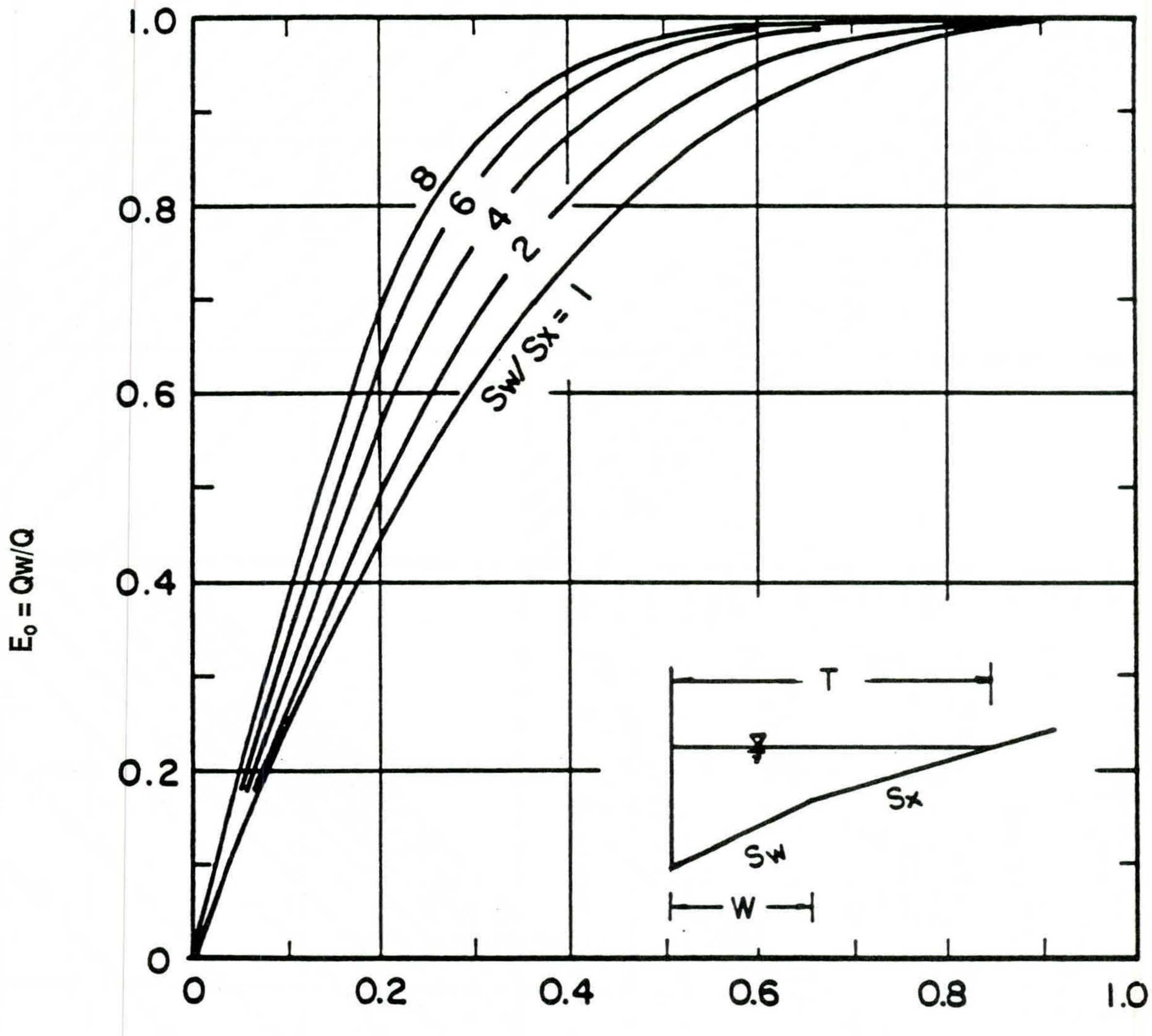


Figure 3.23
Ratio of Frontal Flow to Total Gutter Flow
(source: FHWA, 1984, HEC-12)

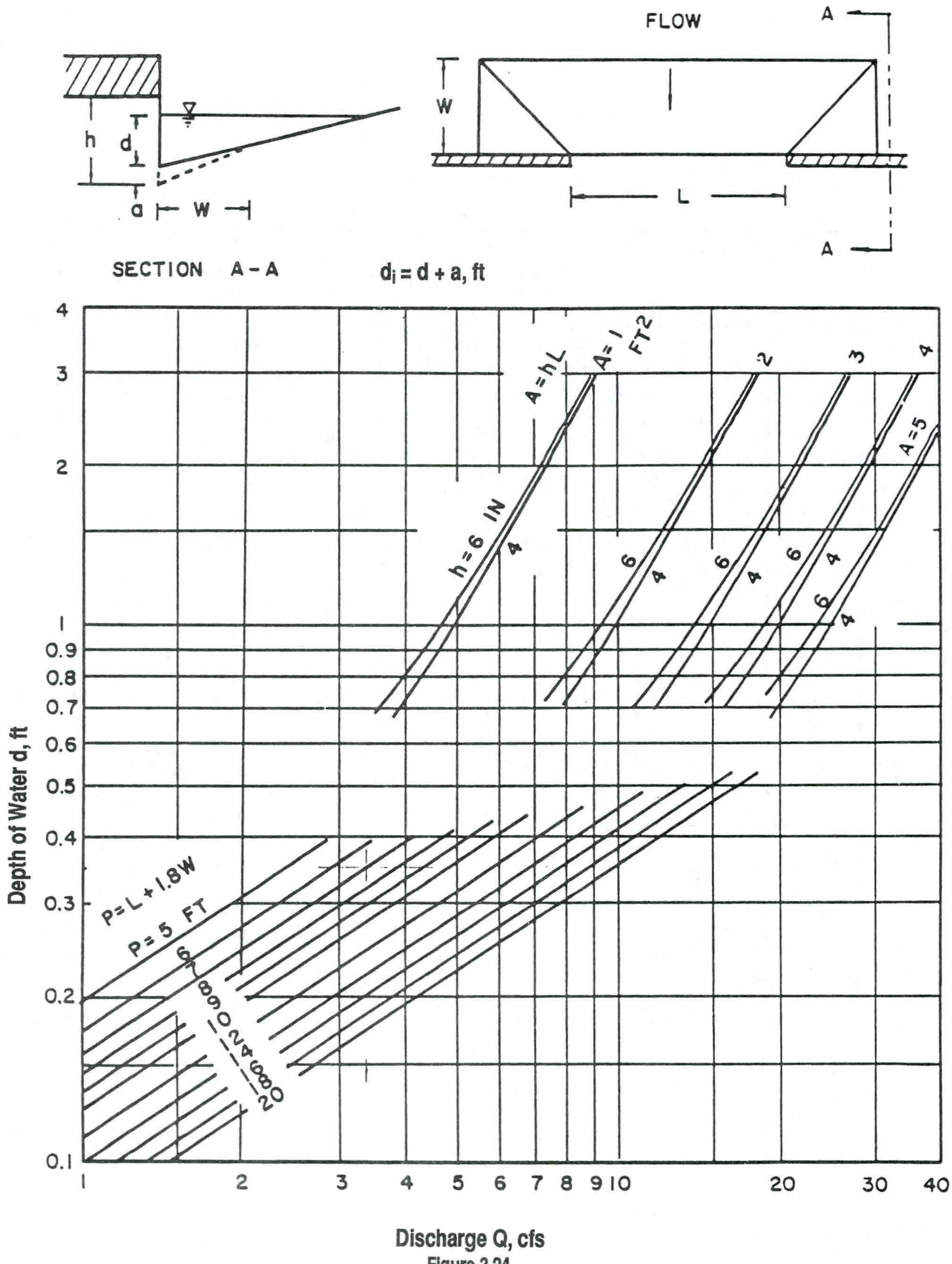


Figure 3.24

Depressed Curb Opening Inlet Capacity in Sump Locations

(source: FHWA, 1984, HEC-12)

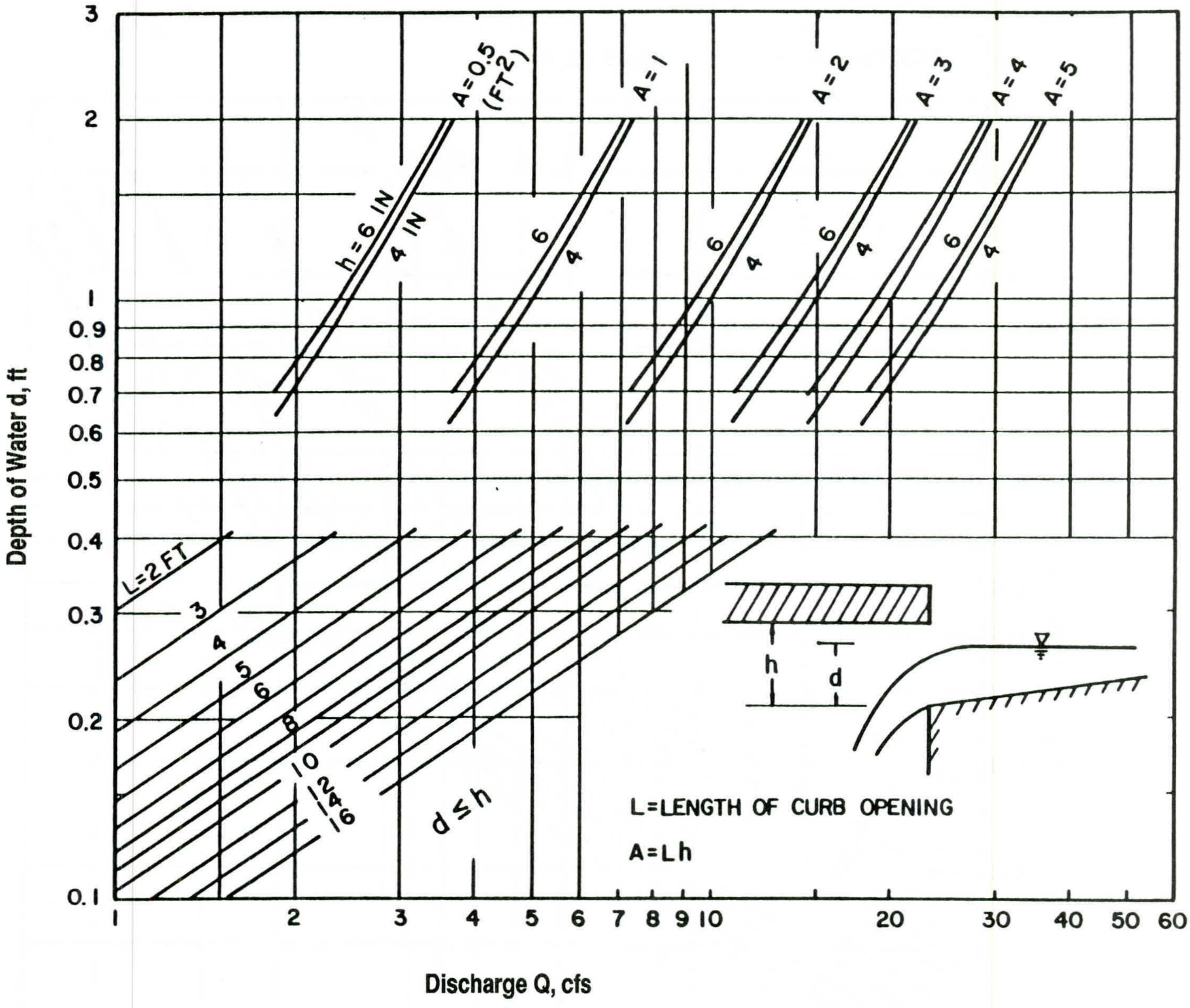


Figure 3.25
Curb Opening Inlet Capacity in Sump Locations
(source: FHWA, 1984, HEC-12)

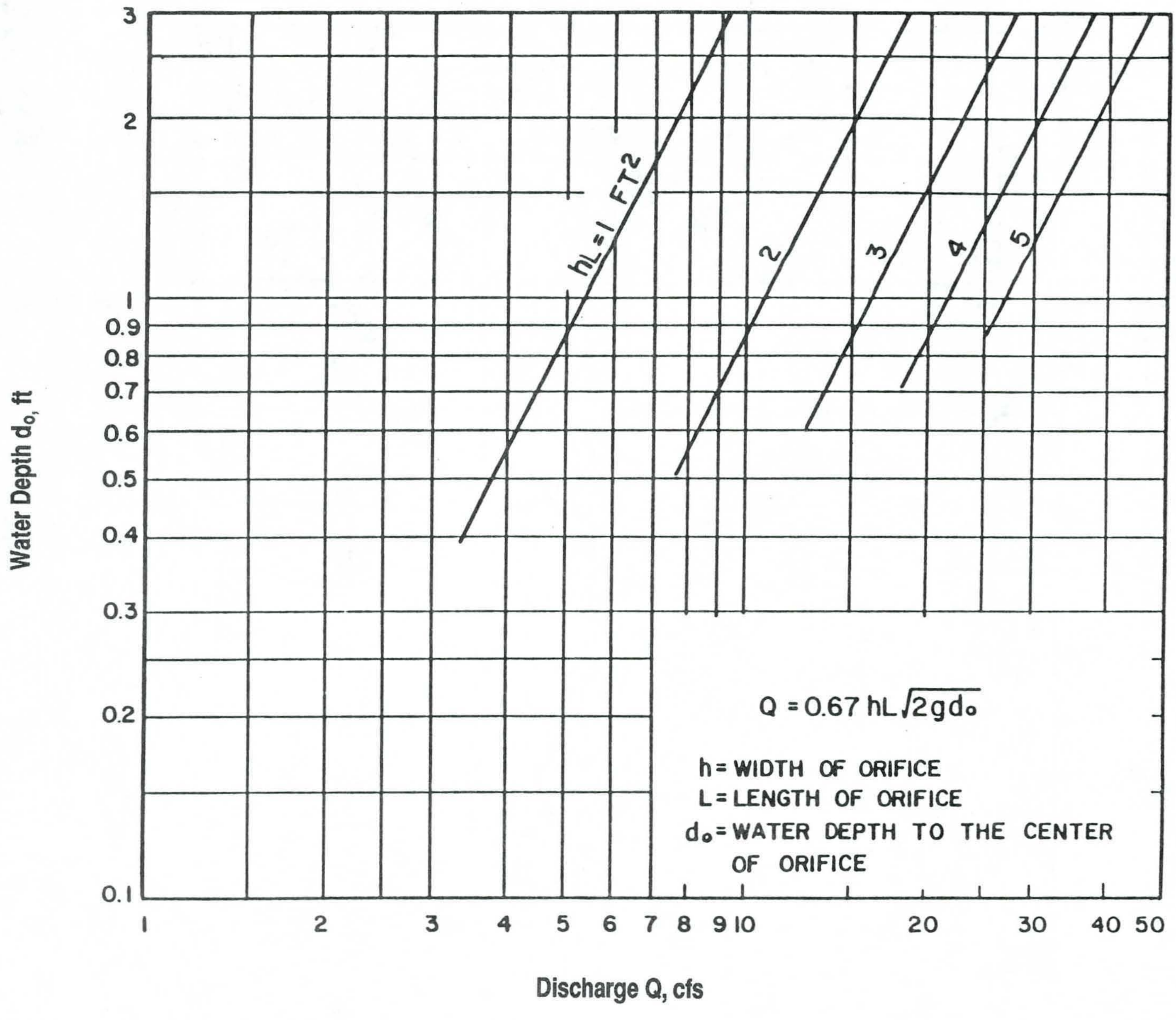


Figure 3.26
Curb Opening Inlet Orifice Capacity for Inclined and Vertical Orifice Throats
(source: FHWA, 1984, HEC-12)

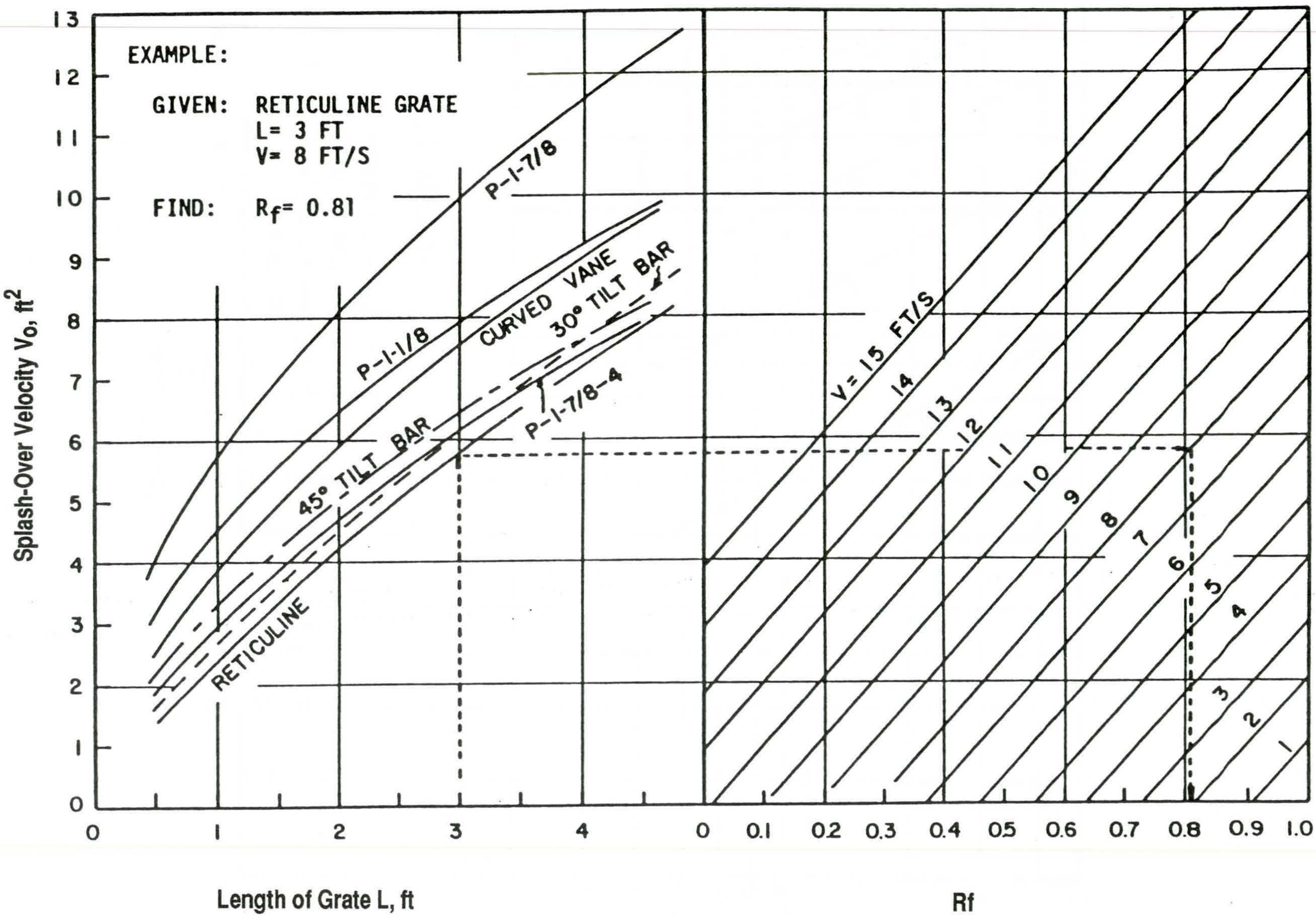


Figure 3.27
Grate Inlet Frontal Flow Interception Efficiency
(source: FHWA, 1984, HEC-12)

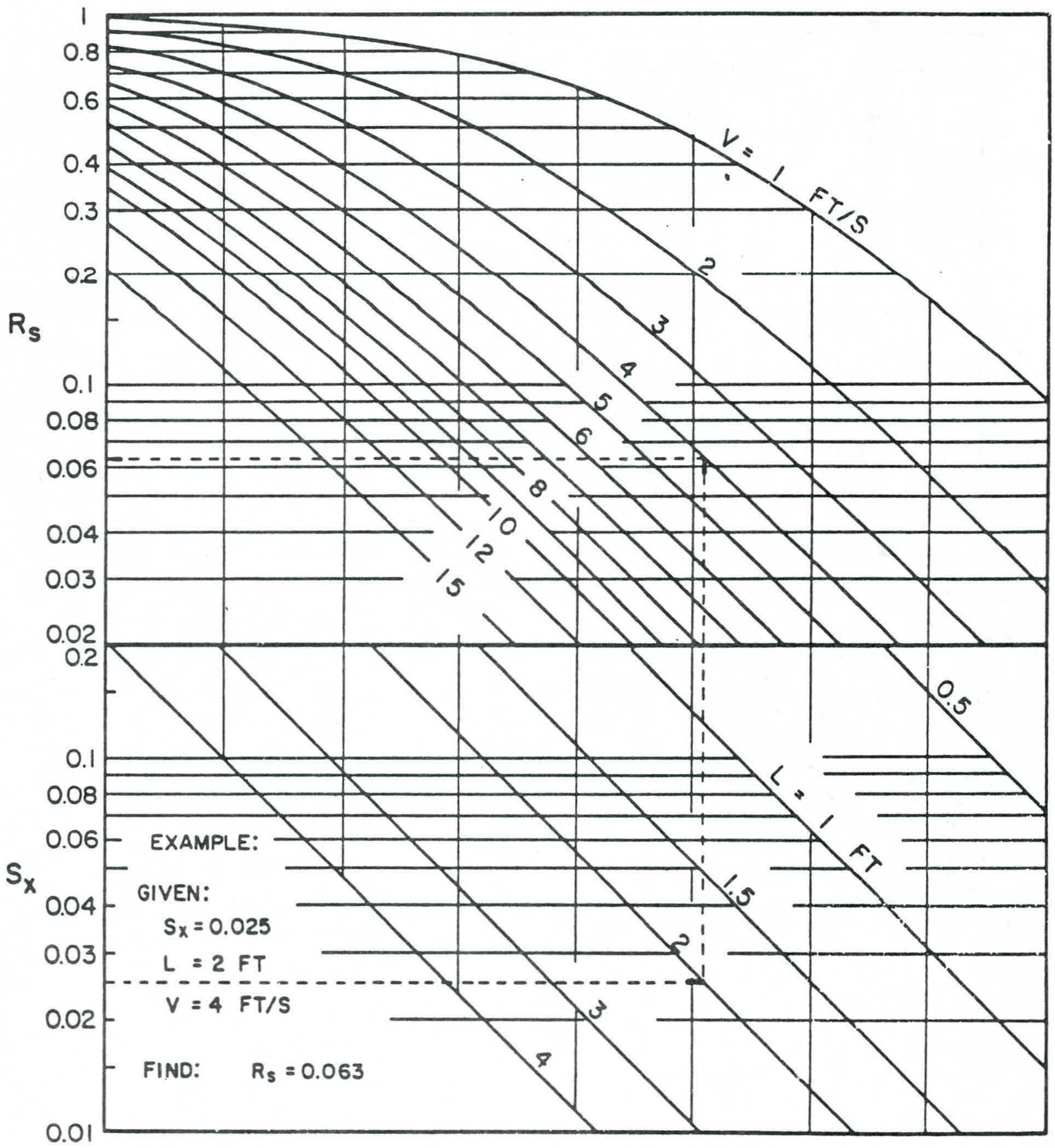


Figure 3.28
Grate Inlet Side Flow Interception Efficiency

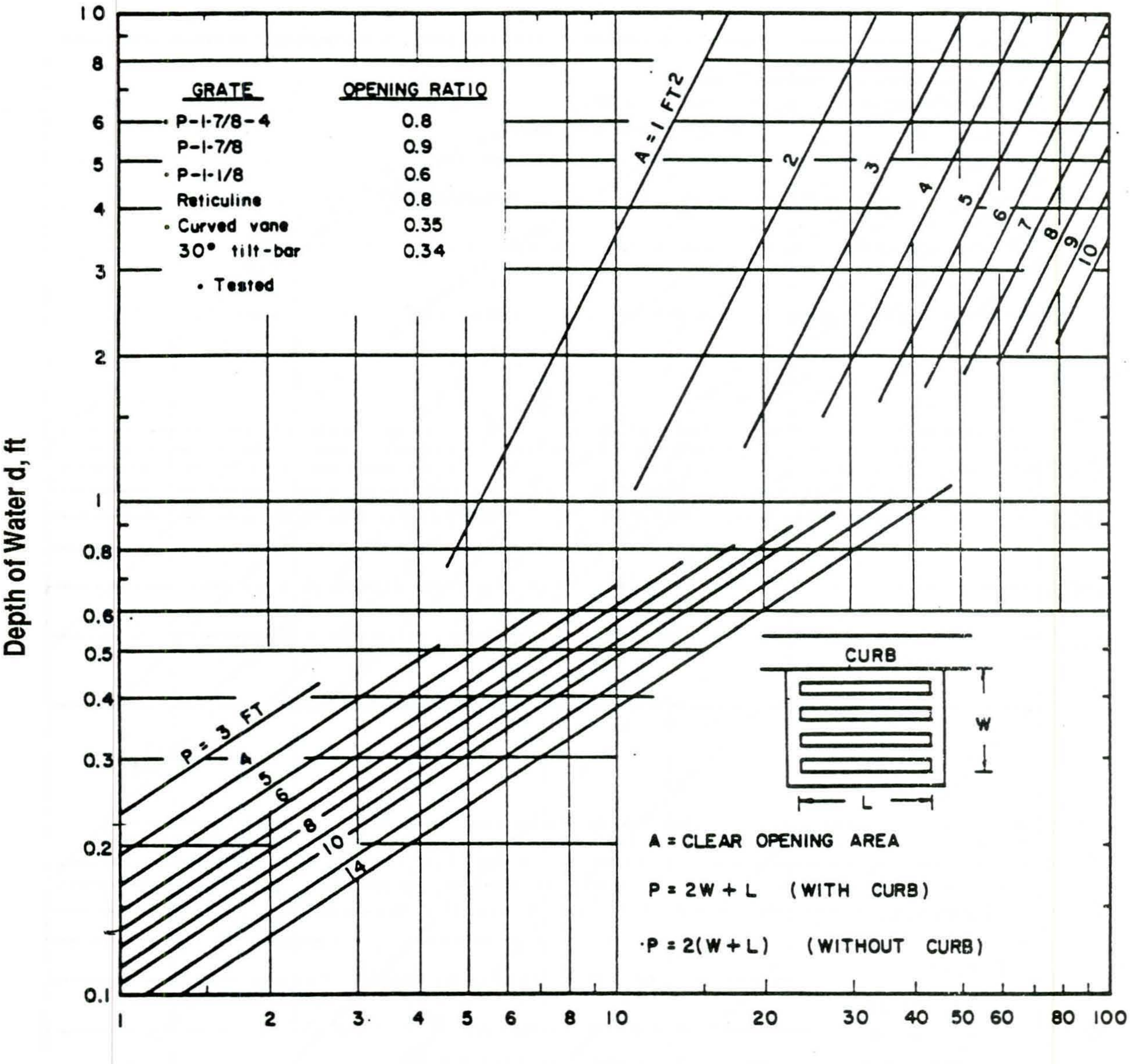


Figure 3.29
Grate Inlet Capacity in Sump Conditions

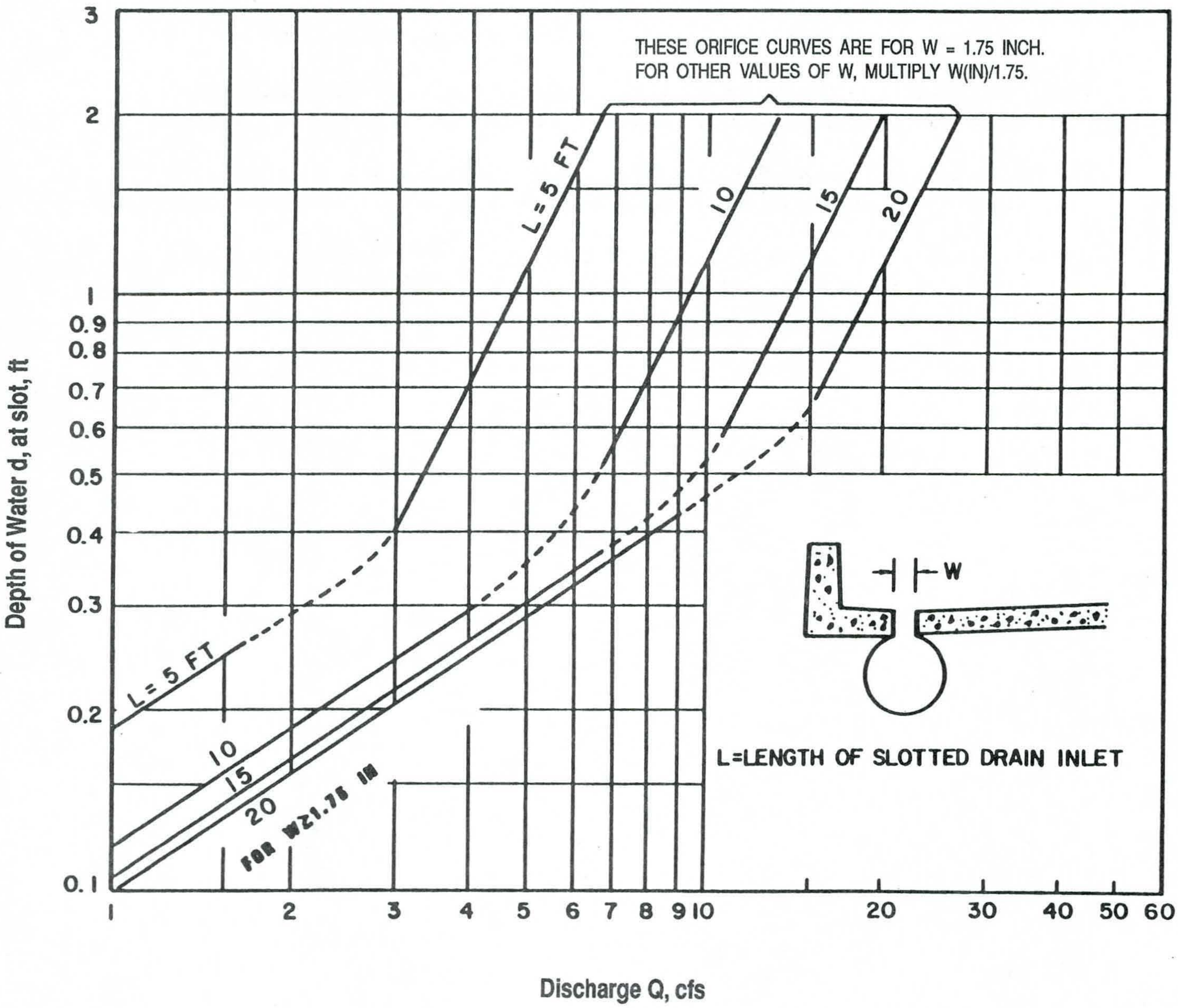


Figure 3.30
Slotted Drain Inlet Capacity in Sump Locations
(source: FHWA, 1984, HEC-12)

3.4 References

FHWA, see U.S. Department of Transportation Federal Highway Administration.

Wright-McLaughlin Engineers for the Urban Drainage and Flood Control District of Denver, Colorado, March 1969, *Urban Storm Drainage Criteria Manual, Volume 1 and 2.*

Wright-McLaughlin Engineers for the Urban Drainage and Flood Control District of Stillwater, Oklahoma, June 1979, *Stillwater Oklahoma, Criteria Manual and Handbook.*

U.S. Department of Commerce Bureau of Public Roads, Washington. May, 1965, Hydraulic Design Series No. 4, *Design of Roadside Drainage Channels.*

U.S. Department of Transportation Federal Highway Administration, March 1984, Hydraulic Engineering Circular No. 12, *Drainage of Highway Pavements.*

4

Closed Conduits

This chapter provides guidelines for the design of storm drains and inlets, culverts, syphons, and entrances and outlets for conduits.

4.1 Definition of Symbols

The following symbols are used in equations throughout Chapter 4.

θ_g	=	Angle of the grate with respect to the horizontal, degrees
θ_j	=	Angle of the lateral with respect to the outlet conduit, degrees
ρ	=	Fluid density, lb/ft^3
τ_c	=	Critical tractive shear stress, lb/ft^2
a	=	Angle of approach, degrees (see Figure 4.36, page 131)
A	=	Area of the barrel, ft^2
C_r	=	Overtopping discharge coefficient
D	=	Culvert or pipe diameter or depth, ft

Dimensionless Scour Geometry = h_s/y_e, W_s/y_e, L_s/y_e, or V_s/y_e
(h_s, W_s, L_s, and V_s are, respectively, depth, width, length and volume of scour, and y_e is the effective conduit flow depth)

F	=	Froude number of flow at the culvert outlet
F*	=	Factor used to account for the effects of plugging by debris. Figure 4.72 (page 174) can be used to determine F* for 50 percent plugging based on w/x.
g	=	Acceleration due to gravity, 32.2 ft/sec^2
H_b	=	Head loss through a bend of a culvert
H_g	=	Head loss through a trash rack
H_j	=	Head loss through the junction in the main conduit, ft
H_t	=	Head loss due to turning the flow, ft

H_{v1}	=	Velocity head in the upstream conduit, ft
H_{v2}	=	Velocity head in the downstream conduit, ft subscripts 1, 2, and 3 refer to the upstream pipe, the outlet pipe, and the lateral pipe, respectively
HW_r	=	Flow depth above the roadway, ft
K_g	=	Dimensionless bar shape factor, equal to: 2.42 - sharp-edged rectangular bars 1.83 - rectangular bars with semi-circular upstream face 1.79 - circular bars 1.67 - rectangular bars with semi-circular upstream and downstream faces
K_t	=	Submergence factor
L	=	Actual culvert length, ft
L_1	=	Adjusted culvert length, ft
L_s	=	Length of the roadway crest along the roadway, ft
L_{sb}	=	Length of scour basin, ft
n	=	Manning n value (from the outlet control chart)
n_1	=	Desired Manning value
Q	=	Rate of flow, cfs
Q_o	=	Overtopping discharge, cfs
t	=	Time, in minutes
t_o	=	Base time used in the experiments to derive coefficients (316 minutes unless specified otherwise).
V	=	Velocity, fps
V_o	=	Outlet mean velocity, fps
V^2/τ_c	=	Modified shear number, $ft^4/lb\text{-}s^2$
V_a	=	Approach channel velocity, fps
V_u	=	Spproach velocity, fps
w	=	Maximum cross-sectional width of the bars facing the flow, ft
x	=	Minimum clear spacing between bars, ft
y'	=	Change in hydraulic grade line through the junction, ft

4.2 Storm Drains

In this manual, storm drain system refers to the system of inlets, conduits, manholes, and other appurtenances which are designed to collect and convey storm runoff from the design storm to a point of discharge and into a major or regional drain outfall.

Storm drain system design is based on the design storm. The design storm is the storm associated with the governing return period from Table 3.1 (page 11). In the upper reaches of a system the 10-year criteria will govern. Farther downstream in the system, the storm drain system design for the 10-year storm may not meet the criteria stated for the 100-year storm. Storm drains will then need to be upsized to meet the 100-year criteria. Both return periods need to be checked to determine which condition governs. The storm condition governing design at any point is the design storm.

4.2.1 Interaction of Storm Drain Systems with other Systems

Storm drains will generally serve minor drains and the smaller major drains. Storm drains typically are not economical for large flows conveyed within the larger major drains and within regional drains. Therefore, the storm drain system will collect runoff to a point where storm drains become too large to be economical and will then discharge into a major or regional drain outfall consisting of a man-made channel, natural wash, or river.

4.2.2 Storm Drain Criteria

4.2.2.1 Maximum Velocity: Consideration should be given to possible pipe damage, the flow energy and how the system hydraulics will be affected. Large momentum forces are generated when high velocity flow is directed around bends and transitions. The conditions at the storm drain outlet should also be considered.

4.2.2.2 Minimum Velocity: Since conduits generally are designed on the basis of their capacity when flowing full, or nearly full, the provision of a velocity adequate for self-cleansing under these conditions does not necessarily ensure prevention of deposits at all conditions of flow. Research shows that full flow in a pipe with a Darcy-Weisbach friction factor of 0.025, at 2 fps, will barely move a coarse sand particle with a diameter of 1.8 mm.

As the friction factor increases, the scouring velocity decreases. Since the friction factor increases with decreasing depth of flow in a pipe, equal self-cleansing will occur in partially full pipes at somewhat less than the critical velocity when flowing full. Equal cleansing ability is computed based on the critical tractive force required to move a particle through the pipe. A minimum cleansing velocity of 5 fps should be maintained when possible, however a minimum of 2 fps must be maintained when it flows at one-half of the design discharge.

4.2.2.3 Hydraulic/Energy Grade Line: Often a closed conduit designed for open channel flow operates 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 storm drain 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.

Even though a conduit may be designed to carry stormwater as open- channel flow, losses at bends and junctions will frequently cause pressure flow to occur for some distance upstream of the "loss" area. Situations may occur in steep terrain where the flow often interchanges between open channel and pressure flows. 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. Planned management of stormwater runoff is also easier to achieve if the hydraulic grade line is kept higher than the crown of the conduit. The discharge through a circular pipe flowing full is constant for a given pipe diameter and hydraulic gradient. Once the flowing full discharge is reached in the pipe, no further runoff can be admitted to the pipe network. Designing for pressure flow allows for minimizing the capital expenditure required for a specified level of protection.

This phenomenon in the field would be evidenced by runoff passing directly over the catch basin to flow down the street (or overland) until it enters the system elsewhere. Another indication is water standing in sumps (detention ponding) until there is sufficient capacity in the storm drain to admit the ponded water. The designer should size the pipes so that the hydraulic grade line is at or very near 6 inches below the inlet elevation. In so doing, he has provided an "automatic safety valve" that will prevent additional runoff from entering the pipe network and causing unforeseen problems at other locations in the system.

Although not always feasible, the recommended procedure is to design storm drains to flow under pressure. Whether or not the final design is made with the pipe flowing partially or completely full, the hydraulic grade line should be computed and displayed on a profile drawing of the conduit. When the hydraulic grade line rises above ground level, stormwater can be found shooting out of catch basins or popping manhole covers, which can lead to needless damage and inconvenience to pedestrian and vehicular traffic.

4.2.2.4 Materials: All storm drain materials shall be approved by the governing municipality prior to use. Table 4.1 lists friction factors for commonly used pipe materials.

4.2.2.5 Sizing: The general procedures for establishing quantities of flow and horizontal layout are the same for closed conduits flowing either as open channels or as pressure conduits. Because of the nature of hydraulic elements in circular conduits, it may be reasonably assumed that open channel flow will occur only when the flow depth is less than 80 percent of the conduit diameter. Once criteria have been set, computations may be made to size the conduits and the various appurtenances.

The designer should initially size the pipes to carry excess runoff to meet the 10-year street encroachment criteria of Table 3.1. Once this runoff has been admitted to the pipe network, additional runoff can be carried by surcharge in the street to a level of encroachment allowed by Table 3.1 for the 100-year storm. If the encroachment criteria is exceeded for the 100-year storm, then the pipe sizes should be increased to carry the extra runoff. The system shall be designed so that the hydraulic grade line is at 6 inches below the inlet elevation.

4.2.2.6 Special Design Considerations: The following considerations are intended for use when junction losses are an important design consideration. They are not intended to be design requirements.

Table 4.1
Values of Effective Absolute Roughness and Friction Formula Coefficients for Closed Conduits

Conduit Material	Effective Absolute Roughness (Darcy-Weisbach) k, ft	Manning's n	Hazen-Williams C
Asbestos cement pipe	0.001 to 0.01	0.011 to 0.015	100 to 140
Brick	0.005 to 0.02	0.013 to 0.017	100
Cast iron pipe			
Uncoated (new)	0.00085	—	130
Asphalt dipped (new)	0.0004	—	115 to 135
Cement lined & seal coated	0.001 to 0.01	0.011 to 0.015	130 to 150
Concrete (monolithic)			
Smooth forms	0.001 to 0.005	0.012 to 0.014	140
Rough forms	0.005 to 0.02	0.015 to 0.017	120
Concrete pipe	0.001 to 0.01	0.011 to 0.015	100 to 140
Corrugated metal pipe (½ × 2⅔ inch corrugations)			
Plain	0.1 to 0.2	0.022 to 0.026	—
Paved invert	0.03 to 0.1	0.018 to 0.022	—
Spun asphalt lined	0.001 to 0.01	0.011 to 0.015	100 to 140
Plastic pipe (smooth)	0.01	0.011 to 0.015	140 to 150
Vitrified clay			
Pipes	0.001 to 0.01	0.011 to 0.015	100 to 140
Liner plates	0.005 to 0.01	0.013 to 0.017	100 to 140

Alignment of Pipe in Manholes: The following discussion applies to the location of pipes within a manhole to achieve maximum efficiency.

For a straight through-flow, pipes should be positioned vertically so that they are between the limits of inverts aligned or crowns aligned. An offset in the plan is allowable provided the projected area of the smaller pipe falls within that of the larger. Aligning the inverts of the pipes is probably the most efficient as the manhole bottom then supports the bottom of the jet issuing from the upstream pipe.

When two inflowing laterals intersect a manhole, the alignment should be quite different. If lateral pipes are aligned opposite one another so the jets may impinge upon each other, the magnitude of the losses are extremely high. A design figure for directly opposed laterals is included, although this arrangement should be avoided wherever possible.

If the installation of directly opposed inflow laterals is necessary, the installation of a deflector, as shown in Figure 4.1 will result in significantly reduced losses. The research conducted on this type deflector is limited to the ratios of $D_o/D_l = 1.25$. 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 catch basin flow is over 10 percent of Q_o.

Lateral infow 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. Reference to the design figures in Section 4.2.4 show that head losses are reduced as compared to directly opposed laterals, even with deflectors. Insufficient data has been collected to determine the effect of offsetting laterals vertically.

Shaping Inside of Manhole: Jets issuing from the upstream and lateral pipes must be considered when attempting to shape the inside of manholes.

The 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 head loss. Limited shaping of the invert to handle low flows is necessary from a practical point of view.

Figure 4.1 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 which were found inefficient are shown in Figure 4.2. The fact that several of these inefficient devices would appear to be improvements indicates that special shapings deviating from those in Figure 4.1 should be used with caution, possibly only after model tests.

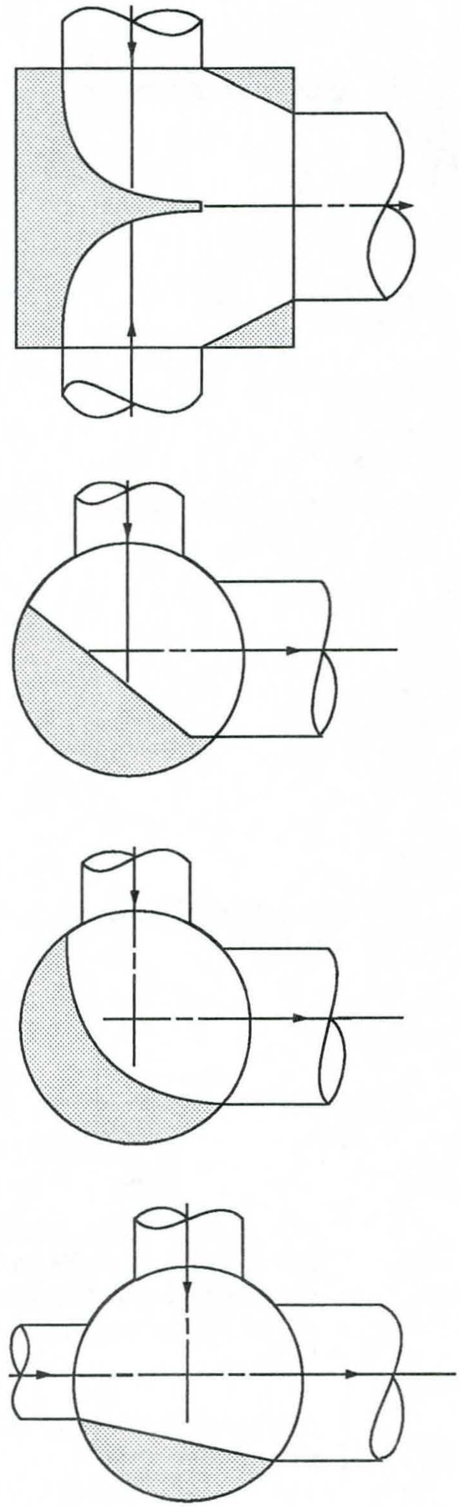
Entrances: Tests indicate that rounding entrances or the use of pipe socket entrances do not have the effect on reducing losses that might be expected. Once again, the effect of the jet from the upstream pipe must be considered. Specific reductions to the pressure change factors are indicated with each design figure.

4.2.2.7 Catch Basins: Certain specific design procedures are necessary for storm water catch basins on systems flowing full.

The design water surface should be at least 6 inches below the gutter grade at the catch basin to allow the catch basin to function properly. If there is any possibility of the hydraulic grade being above this level, adjustments should be made to lower the hydraulic grade line or the inlet should be eliminated in that location.

4.2.2.8 Outlets: Final hydraulic design of storm drains begins at the lowest point in the storm drain system. The beginning hydraulic grade line (water surface in open channel flow or hydraulic grade line in pressurized conduits) in the receiving facility must be determined coincident with the time of peak flow from the storm drain. If sophisticated hydrologic modeling techniques are utilized, the flow rate in the receiving facility (normally major drainage) may be known and the corresponding water surface elevation can be determined.

If the outlet is submerged or if the receiving water surface is higher than normal depth in the storm drain, the beginning hydraulic grade line is the hydraulic grade line in the receiving stream. With a submerged outlet, the design proceeds up the pipeline after inclusion of exit losses. For unsubmerged outlets, design can begin assuming normal depth at the first source of a point loss (lateral, manhole, or bend),



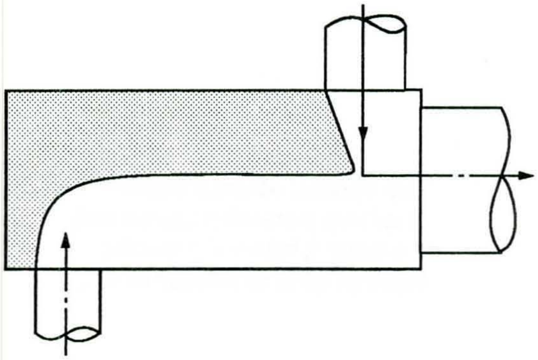
Directly opposed lateral with deflector (head losses are still excessive with this method, but are significantly less than when no deflector exists.)

Bend with straight deflector

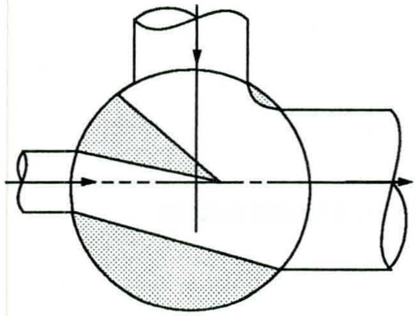
Bend with curved deflector

Inline upstream main and 90° lateral with deflector

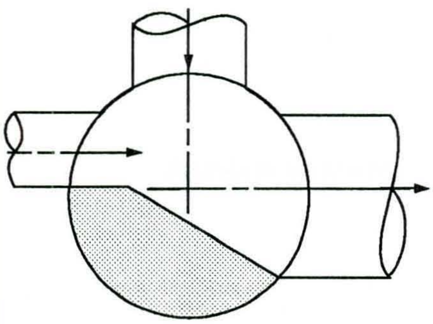
Figure 4.1
Efficient Manhole Shaping



Offset lateral with deflector



Inline upstream main and 90° lateral with divider



Inline upstream main and 90° lateral with deflector

> ***Note***
>
> **Although these modifications look like improvements, studies have proven these designs to be less efficient than the designs in Figure 4.1.**
>
> **Use caution when deviating from recommended designs.**

Figure 4.2
Inefficient Manhole Shaping

unless this first loss is hydraulically close to the outlet. In this case, backwater techniques will be necessary. For a conduit with an unsubmerged outlet and a greater hydraulic (and energy) grade line slope than pipe slope, the beginning water surface elevation is the critical depth in the storm drains.

Figure 4.3 illustrates some of the exit conditions and Figure 4.4 illustrates the various hydraulic relationships for closed conduits and for open channel flow.

4.2.3 Design Procedures

4.2.3.1 General Aspects of Storm Drain Design: Calculations to check the pressure of water surface elevations in the storm drain system begin with a known surface elevation at some downstream point. The rise of the hydraulic grade line along the pipe to the first upstream junction is added to this known elevation to obtain the outfall hydraulic grade line elevation at the upstream junction. The resistance loss—or friction loss—in the pipe is the hydraulic grade line rise through the pipe length and can be calculated by methods shown in *Design and Construction of Sanitary and Storm Sewers* (American Society of Civil Engineers 1969).

If the hydraulic grade line is above the pipe crown at the upstream junction, full flow calculations may proceed. If the hydraulic grade line is below the pipe crown at the upstream junction, then open channel flow calculations must be used at the junction.

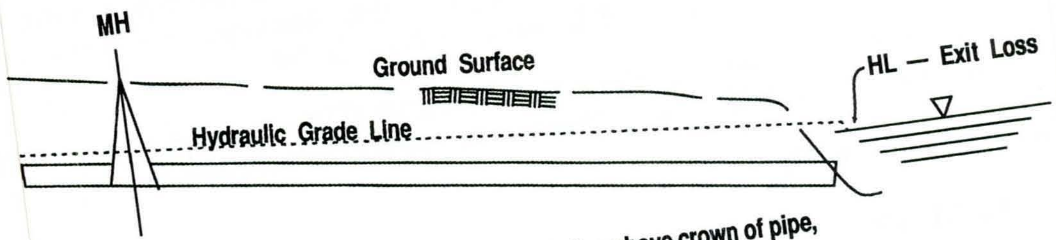
For an unsubmerged outlet, a flow depth must be determined at some control section to allow calculations to proceed upstream. As illustrated in Figure 4.3, the hydraulic grade line is then projected to the upstream junction. Full flow calculations may be utilized at the junction if the hydraulic grade is above the pipe crown.

For open channel flow, the assumption of straight hydraulic grade lines as shown in Figure 4.3 is not entirely correct, since backwater and drawdowns exist, but should be accurate enough for the size of pipes usually considered as storm drains.

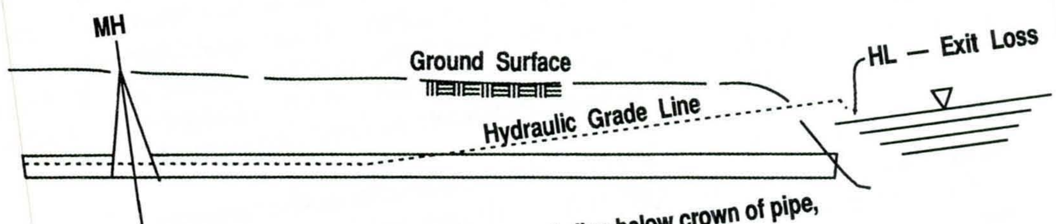
On steep storm drain grades, the upstream storm drain may enter the junction at an elevation somewhat higher than the crown of the downstream storm drain pipe. In this case, it may be assumed that the upstream flow acts as though it were catch basin flow since the jet is essentially broken up when it enters the junction. The designer may then use the relevant catch basin design figure to calculate the pressure change likely across the junction.

The pressure change for each upstream pipe at the junction is then added to the common outfall pipe hydraulic grade elevation at the center of the junctions. This accounts for all losses at the junction and gives a starting elevation for calculations to be made along each upstream pipe.

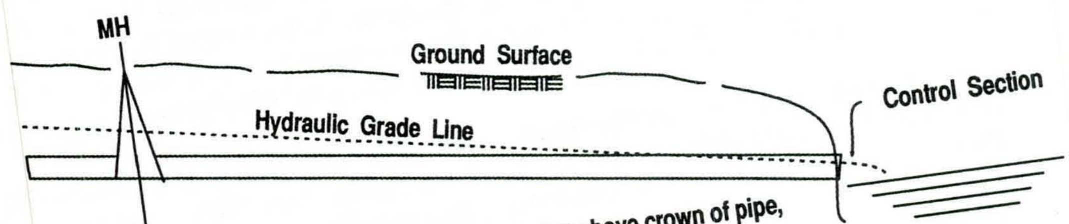
The water depth at each junction must be calculated to verify that the water level is above the crown of all pipes. Whenever the level is 80 percent or less of the pipe diameter, full flow methods are not applicable. When the pipe level exceeds the 80 percent level, pressurized pipe flow techniques are applicable.



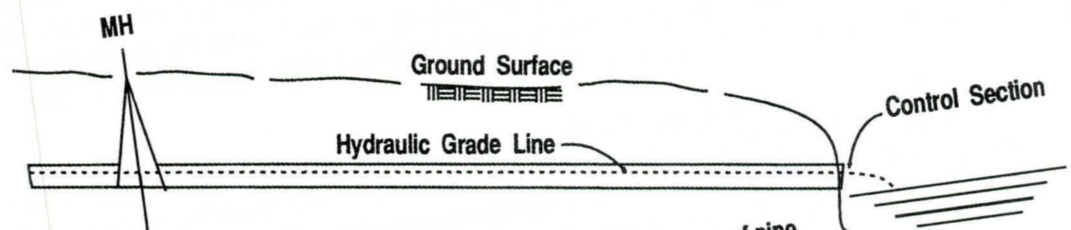
SUBMERGED OUTLET — Hydraulic grade line above crown of pipe, full flow design methods may be used at manhole.



SUBMERGED OUTLET — Hydraulic grade line below crown of pipe, open channel flow methods must be used at manhole.

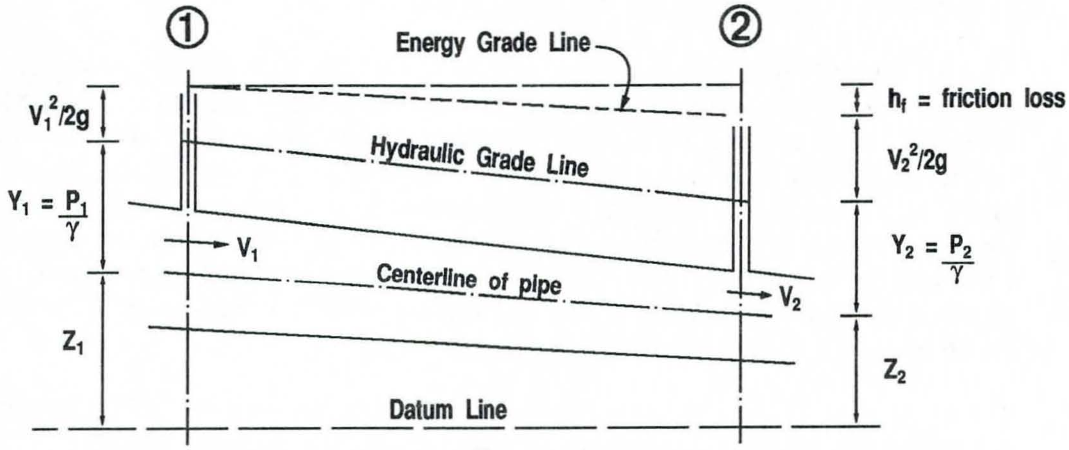


FREE DISCHARGE — Hydraulic grade line above crown of pipe, full flow design methods may be used at manhole.

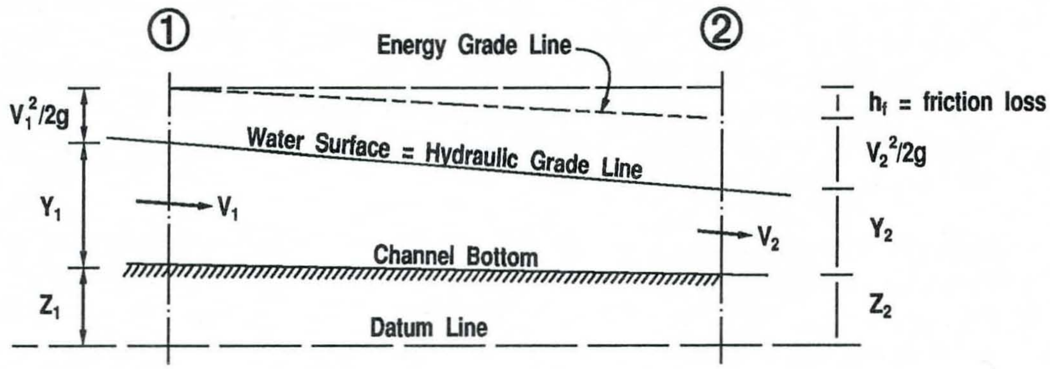


FREE DISCHARGE — Hydraulic grade line below crown of pipe, open channel flow methods must be used at manhole.

Figure 4.3
Determining Type of Flow



(a) Closed-Conduit Flow



(b) Open Channel Flow

Figure 4.4
Comparison Between Closed Conduit and Open Channel Flow

To expedite computations, the storm drain hydraulic grade line elevation determined at a junction should first be compared to the elevation of the top of the downstream pipe and the gutter. Because of usual losses at the junction, it is known that upstream hydraulic grade elevations and the water elevation in the catch basin are generally higher than the elevation of the downstream storm drain hydraulic grade line. Comparison to limiting conditions will indicate whether the design may or may not be applicable at the junction.

4.2.3.2 Design Procedure for Pressure Conduits: The methodology outlined in this chapter allows computations for required storm drain systems to be pursued with the degree of accuracy justified by the cost of subsequent construction. The figures in Section 4.2.4 offer one of the better design methodologies available, enabling a designer to include manhole losses in a progressive calculation of pressure elevations proceeding upstream along a storm drain system, determining the water surface elevation, hydraulic grade line, and total energy gradient. Emphasis must be placed on the fact that the figures are strictly applicable only when all pipes entering the manhole are flowing full.

The nomenclature used in the design figures that are suggested for design purposes is explained in Figure 4.5, where sketches of the junction types also appear.

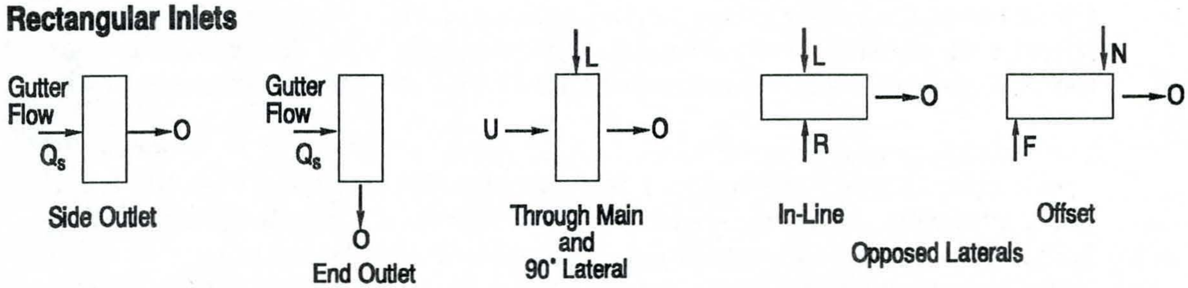
In this presentation of design methods, provision is made for reserving numbers to designate catch basins and junctions by using a system of letter subscripts for identifying pipes. The letter subscript is applied to the pipe diameter (D), its discharge (Q), and the resulting velocity of flow (V).

The letter subscript designates the function of that pipe at the particular junction under consideration. These letter subscripts are used consistently for each pipe of similar function at all junctions. Thus D_O designates the diameter of the outfall pipe at any junction, D_U that of the upstream pipe, D_L that of a lateral entering the left side of a junction (viewed looking downstream along the direction of the outfall pipe), and D_R a similar lateral at the right. Similarly, Q_O, Q_U, Q_L, and Q_R designate the rates of flow in the several pipes.

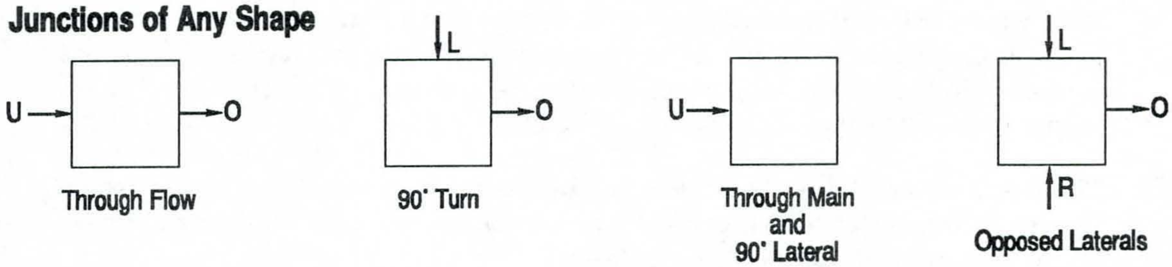
In the design applications, the outfall pipe is always used as the basic measurement. Pipe size ratios are stated as the ratio of upstream to outfall pipe diameter, e.g. D_U/D_O or D_L/D_O. Flow ratios are similarly stated, e.g. Q_U/Q_O. Pressure changes are stated in terms of outfall velocity head; that is, the pressure change coefficient K_U, equals the pressure change h_U in its ratio to $V_o2/2g$.

The line diagrams of Figure 4.5 illustrate the pipe positions and the function of each as supply or outfall for each type of inlet and junction involved. A detail plan is included to show junction and pipe diameter dimensions used. These dimensions may be in inches, feet, or any other unit of linear measurement since they are used in the design figures only as ratios of one to another. The figures included in Section 4.2.4 are based on tests of round pipe, and apply to pipes of circular cross section. However, the figures will apply accurately enough to pipes of any cross section.

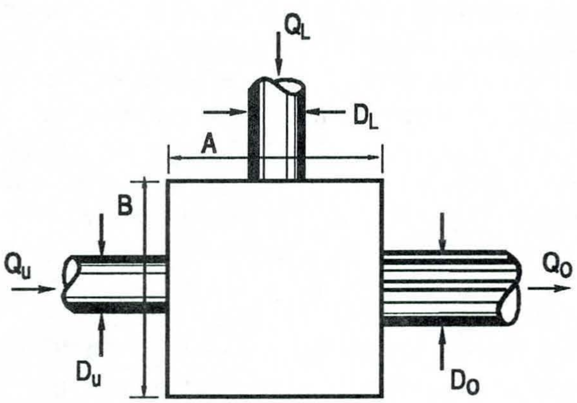
Rectangular Inlets



Junctions of Any Shape



Junction Dimensions



Nomenclature

Q	rate of flow
D	diameter of pipe
A	dimension of junction in direction of outfall pipe
B	dimension of junction at right angles to outfall pipe
d	depth of water in inlet
S	slope of pipe
S_f	friction slope
Q_s	flow into inlet through top grate
D_o Q_o	diameter and flow in outfall
D_U Q_U	diameter and flow in upstream main
D_L Q_L	diameter and flow in left lateral
D_R Q_R	diameter and flow in right lateral
D_N Q_N	diameter and flow in near lateral(1)
D_F Q_F	diameter and flow in far lateral(1)
D_{hv} Q_{hv}	diameter and flow in lateral w/higher-velocity flow(2)
D_{lv} Q_{lv}	diameter and flow in lateral w/lower-velocity flow(2)
	Pressure change coefficients for inlet water depth and an upstream pipe pressure relative to the outfall pipe pressure.
K_s	water depth with all flow through grate
K_U	upstream main pressure
K_R or K_L	lateral pipe pressure
K_N	near lateral pipe pressure(3)
K_F	far lateral pipe pressure(3)
$\overline{K_U}$, $\overline{K_L}$	pressure coefficient at QL = Qo
M_U, M_L	multipliers for $\overline{K_U}$ or $\overline{K_L}$ to obtain K_U or K_L

$$h_U = K_U \frac{V_O^2}{2g} \qquad h_L = K_L \frac{V_O^2}{2g}$$

(1) Of opposed laterals; designation with reference to position relative to outfall end of junction.
(2) For in-line opposed laterals only.
(3) Offset opposed laterals.

Pressure Line Elevation at Junction of a Lateral with a Through Main

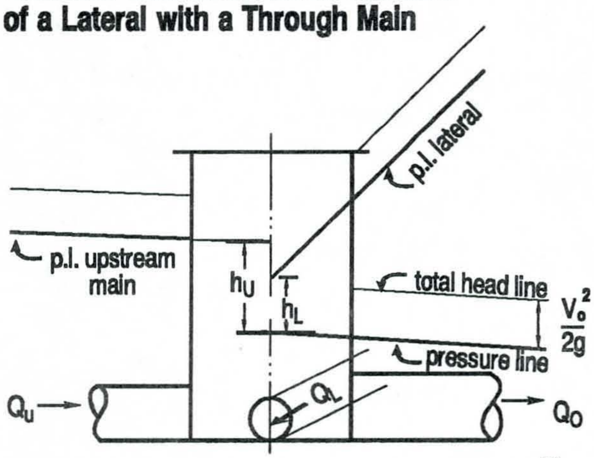


Figure 4.5
Manhole Junction Types & Nomenclature
(University of Missouri 1958)

One of the diagrams on Figure 4.5 shows a through main at a junction of a 90 degree lateral, with pressure lines and total head lines superimposed. It will be noted that the relative elevations of the various total head lines are not dealt with in this sketch.

The same diagram shows the hydraulic grade lines projected to a point above the "branch point," this being the location in plan of the intersection of the outfall pipe and lateral pipe centerlines. A similar point of reference, the center of the junction box, is used for the upstream in-line pipe and its hydraulic grade line where no lateral is present. The change of pressure at a junction is measured by the difference in elevation between the outfall hydraulic grade line and an upstream hydraulic grade line, along the vertical line through the branch point. The vertical dimensions h_U and h_L indicate the change of pressure for the upstream in-line and lateral pipes, respectively. The adjacent equations on Figure 4.5 state how each is calculated.

There will be situations where flow conditions are not represented in the design figures. In this instance, it is generally acceptable to extrapolate. It is rare to have required extrapolations that are significant.

In those rare instances where more than three pipelines enter the same manhole, it may be necessary to make simplifying assumptions in regard to the flow conditions and utilize the appropriate figure representing these assumptions. This approach is not considered a serious constraint as there are many similarities between the graphics for varying flow conditions. Table 4.2 lists the recommended graphs and assumptions.

At all junctions where a change of pressure occurs, a loss of total head must occur whether the pressure change is positive or negative. This basic fact may be used to check pressure results.

General Instructions for Use of Design Figures: Several operations are common to use of the design figures for various types of junctions. Instructions for performing these recurring procedures are consolidated in the following General Instructions. In the detailed instructions for use of the individual figures, references to these General Instructions are made by number (Gen. Instr. 1, etc.). The general instructions are as follows:

1. Determine and tabulate the elevation of the outfall pipe pressure line at the branch point or catch basin center (refer to Figure 4.5). This elevation is obtained by adding the pipe friction loss to the elevation of the pressure line at the preceding structure downstream.

2. Calculate the mean velocity head of the flow in the outfall pipe.

$$V_o^2 / 2g$$

3. Calculate the required flow rate and size ratios. Examples:

$$Q_U/Q_O, \quad Q_L/Q_U, \quad Q_G/Q_O, \quad \text{etc.}$$

$$D_U/D_O, \quad D_L/D_O, \quad B/D_O, \quad \text{etc.}$$

Table 4.2
Summary of Design Figure/Manhole Configuration Application

Case	Design Figure
Catch Basin With Inlet Flow Only	4.7
Flow Straight Through Any Manhole	4.8
Rectangular Manhole, Through Pipe and Inlet Flow	4.9
Rectangular Manhole With In-Line Upstream Main and 90° Lateral Pipe (with or without inlet flow)	4.10
Rectangular Manhole With In-Line Opposed Lateral Pipes Each at 90° to Outfall (with or without inlet flow)	4.11
Rectangular Manhole With Offset Opposed Lateral Pipes each at 90° to Outfall (with or without inlet flow)	4.12
Square Manhole at 90° Deflection	4.13
Round Manhole at 90° Deflection	4.13
Deflectors in Square or Round Manholes at 90° Deflection	4.13
Square Manhole on Through Pipeline at Junction of a 90° Lateral Pipe (large size laterals: $D_L/D_O > 0.6$)	4.13, 4. 14
Round Manhole on Through Pipeline at Junction of a 90° Lateral Pipe (large size lateral: $D_L/D_O > 0.6$)	4.13, 4.14
Deflectors in Square or Round Manholes on Through Pipelines at Junction of a 90° Lateral Pipe (large size laterals: $D_L/D_O > 0.6$)	4.13, 4.14
Square or Round Manhole on Through Pipeline at Junction of a 90° Lateral Pipe (smaller size laterals: $D_L/D_O < 0.6$) or laterals with no manhole.	4.15
Sewer Bends with or without Manhole	4.16
Square or Round Manhole on Through Pipeline with Two Laterals ($D_L/D_O \geq 0.5$) or ($Q_U/Q_O >$ 0.3), Consider Upstream Pipe as Grate Flows	4.10
Square or Round Manhole on Through Pipeline with Two Laterals (($D_L/D_O < 0.5$) or ($Q_U/Q_O <$ 0.3), Consider Upstream Pipe as Grate Flows	4.11, 4.12

4. Estimate the depth of water, d, in a manhole with flow into the manhole from a top inlet, either along or combining with flow from an upstream pipe.

   d = total depth of water, ft

   h = (outfall pressure line elevation minus catch basin bottom elevation) + (K) $V_O^2/2g$

   k = the pressure change coefficient for the catch basin water depth (This is estimated as detailed for each type of manhole. Such estimates are not necessary for manholes with in-line or offset opposed laterals.)

5. Use the coefficients K from the figures for manholes with square-edged entrance to the outfall pipe (entrance flush with box side, with square edges).

6. Use reduced coefficients K, where applicable, for a rounded entrance to the outfall pipe (rounded on 1/4 circle arc of approximate radius 1/8 D_O) or for

an entrance formed by the socket end of a standard tongue-and-groove concrete pipe.

Figure 4.7:	Insignificant effect, make no reduction.
Figure 4.8:	Read directly from the figure.
Figure 4.9:	Reduce K_U by 0.1 for usual proportions of inlet flow; by 0.2 for Q_G about 0.5Q.
Figure 4.10:	Reduce K_U and K_L in same manner as Figure 4.9.
Figure 4.11:	Insignificant effect, make no reduction.
Figure 4.12:	Insignificant effect, make no reduction.
Figure 4.13, 4.14, 4.15:	See specific instructions for each case.
Figure 4.16:	Make no reductions.

7. Calculate pressure change. To calculate the change of pressure at a manhole, working upstream from the outfall pipe to an upstream pipe, the design figure applying to the type of junction involved is selected. The pressure change coefficient for a specific upstream pipe is read from the figure for the particular flow rate and size ratios already calculated. The pressure change is calculated:

$$h = K \times V_O^2 / 2g \qquad \textbf{(4.1)}$$

The coefficient is a dimensionless number, and therefore, the change of pressure will be in feet.

8. Apply the pressure change. The pressure change, in feet, for each upstream pipe is added to the outfall pipe pressure line elevation at the branch point to obtain the elevation of each pressure line for further calculations upstream along the pipe. In some cases, the upstream pressure line at the branch point will be at a lower elevation than the downstream pressure line. Where this less common situation may occur with a particular type of junction, it is mentioned in the instructions for use of the specific figure.

9. Determine the elevation of the water surface. The elevation of the water surface in a manhole (with or without catch basin flow) receiving flow from a pipe or pipes will correspond to that of the upstream in-line pipe pressure line. At a junction with offset opposed laterals, the water surface will correspond to the elevation of the far lateral pipe pressure line. At a junction with in-line opposed laterals, the water surface will correspond to the elevation of the pressure line of the higher-velocity lateral pipe.

Verify that the water surface is above the crown elevation of all pipe connections to the structures that are being analyzed. Small pipes, such as laterals to catch basins, which carry a small portion of the total flow, may reasonably be constructed to affect a manhole in the same way as catch basin flow from the ground surface.

4.2.3.3 Design Procedure for Open Channel Flow: The HGL in an upstream pipe that is in line with the outlet will seek normal depth when the slope of the pipe is greater than the slope required for full flow. Should the slope of the pipe be less than that required for full flow, the HGL will be at an elevation greater than the crown of the pipe. Drawdown effects will be observed near the outfall from the pipe. In this case, the depth will pass through critical depth at or near the point of outfall. Backwater or drawdown calculations for large diameter pipes should be made along the length of the pipe to determine whether normal depth or pressure flow is attained before the next manhole.

For the size of pipes normally encountered in storm drain design, it is reasonable to assume a straight water surface. It is also assumed that the energy grade line is parallel to the pipe grade, and that any losses other than pipe friction may be accounted for by assuming point losses at each manhole.

The basic approach to design of open channel flow in storm drains should be to calculate the energy grade line along the system. Once the discharge has been determined and a pipe size and slope assumed for a given section, the d/D and v/V full ratios can be determined from a graph of Hydraulic Elements for Circular Conduits (Figure 4.6).

The next step is to calculate the energy grade line:

$$H = Z + d + (V^2/2g) \qquad \textbf{(4.2)}$$

At each manhole the energy grade line of all pipes should coincide, allowing for reasonable values of head loss to the junction.

The usual method of stating head losses at manholes is in terms of a constant K times the velocity head of the conduit in question,

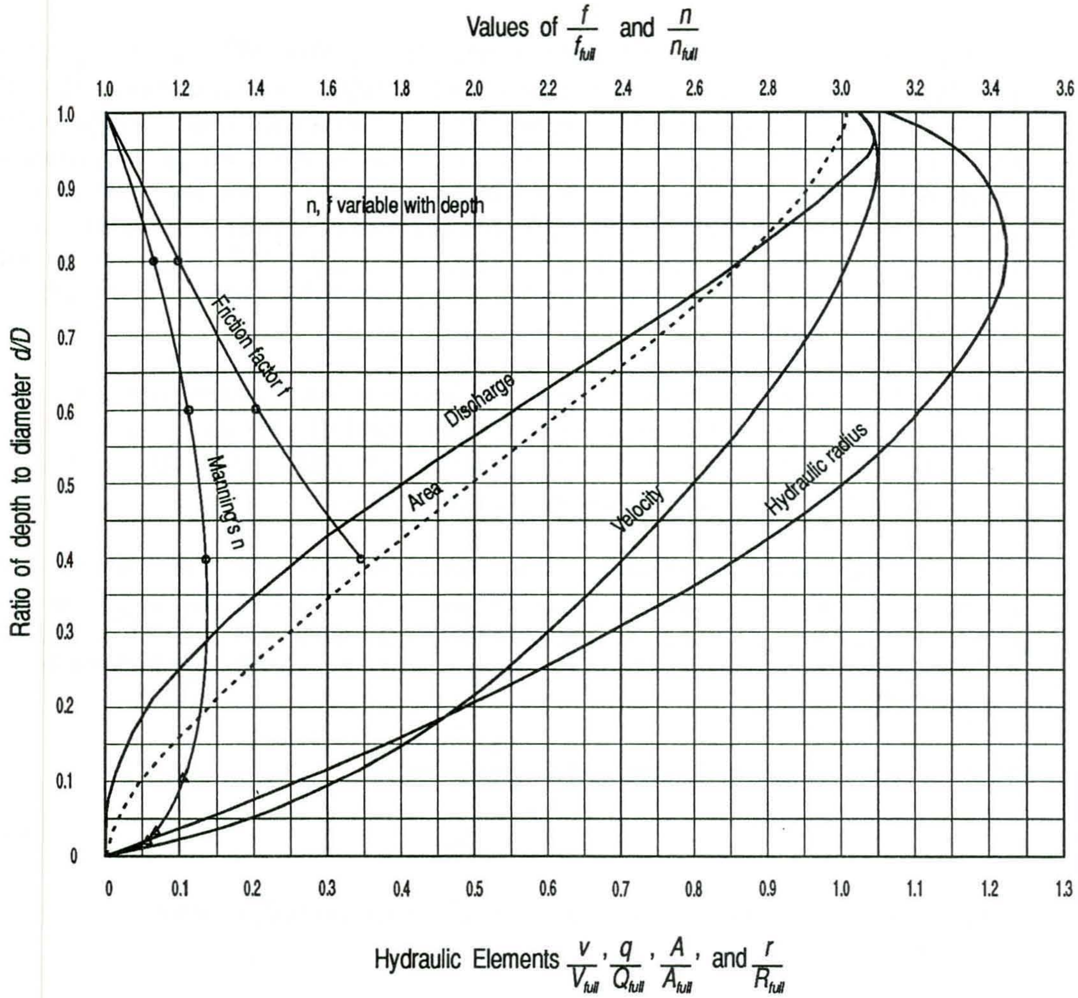
$$h_e = (K)\, V^2/2g \qquad \textbf{(4.3)}$$

A difficulty in design of systems is the determination of the value of K.

Simple Transitions in Pipe Size: Simple transitions in conduit size in a manhole with straight through flow may be analyzed by the following equation:

$$h_e = K\Delta\,(V^2/2g) \qquad \textbf{(4.4)}$$

The term $\Delta\,(V^2/2g)$ refers to the change in velocity head in the upstream and downstream conduits. The value of K varies from 0.1 for increasing velocity to 0.2 for decreasing velocity transitions if flow is subcritical. For supercritical flow, greater values of K are probable, but have not been determined.



v = Actual velocity of flow, fps

V_{full} = Velocity flowing full, fps

q = Actual quantity of flow, cfs

Q_{full} = Capacity flowing full, cfs

A = Area occupied by flow ft^2

A_{full} = Area of pipe, ft^2

r = Actual hydraulic radius, ft

R_{full} = Hydraulic radius of full pipe, ft

Figure 4.6
Hydraulic Elements of Circular Conduits

Bends: Reliable headloss coefficients through bends in open channel flow are almost entirely lacking. Reasonable assumptions may be made by utilizing existing information available on losses in bends and pressure conduits.

Junctions: Values for head loss coefficients at junctions on storm drains flowing as open channels are not readily available. Complicated methods for calculating head loss at certain types of junctions are available and are justified for certain situations.

Unless unusual conditions exist, the figures and procedures for pressure conduits should be used. Energy grade lines should be matched to insure continuity; that is, the upstream energy grade line elevation equals the downstream energy grade line elevation plus head loss.

Storm Water Catch Basins: As can be noted in Figure 4.7, the depth of water in a catch basin has a profound effect on the energy losses in a catch basin. The shallower the depth, the greater is the head loss. Normal culvert design aids are *not* applicable to this condition. The water falling into the catch basin causes significant turbulence and energy losses.

For this condition and for significant grate flow into any junction, the applicable curves for the pressure conduit analysis should be used.

4.2.4 Design Aids

The figures presented in this section are to be used for determining the pressure change at storm drain junctions. Instructions are included to aid in the application of these figures. Section 4.2.5 contains detailed storm drain design examples demonstrating the use of each figure.

4.2.4.1 Catch Basin with Inlet Flow Only: Pressure change coefficients are presented in Figure 4.7 for use in determining the elevation of the water surface in a catch basin with all inflow entering through an inlet. Separate curves are included for the outfall pipe connected at the box end (short dimension) and the box side (long dimension). The coefficient K_G depends on the pipe position and the depth of water in the inlet.

To use Figure 4.7:

1. Note whether outlet is at end or side.
2. Determine outfall pipe pressure line elevation (Gen. Instr. 1).
   [**Note**: See Section 4.2.3.2 for an explanation of the General Instructions.]
3. Calculate outfall velocity head (Gen. Instr. 2).
4. Estimate a value for water depth, d.
   » outfall pressure line elevation minus inlet bottom elevation plus h_G equals d, where

$$h_G = K_G V_O^2/2g$$

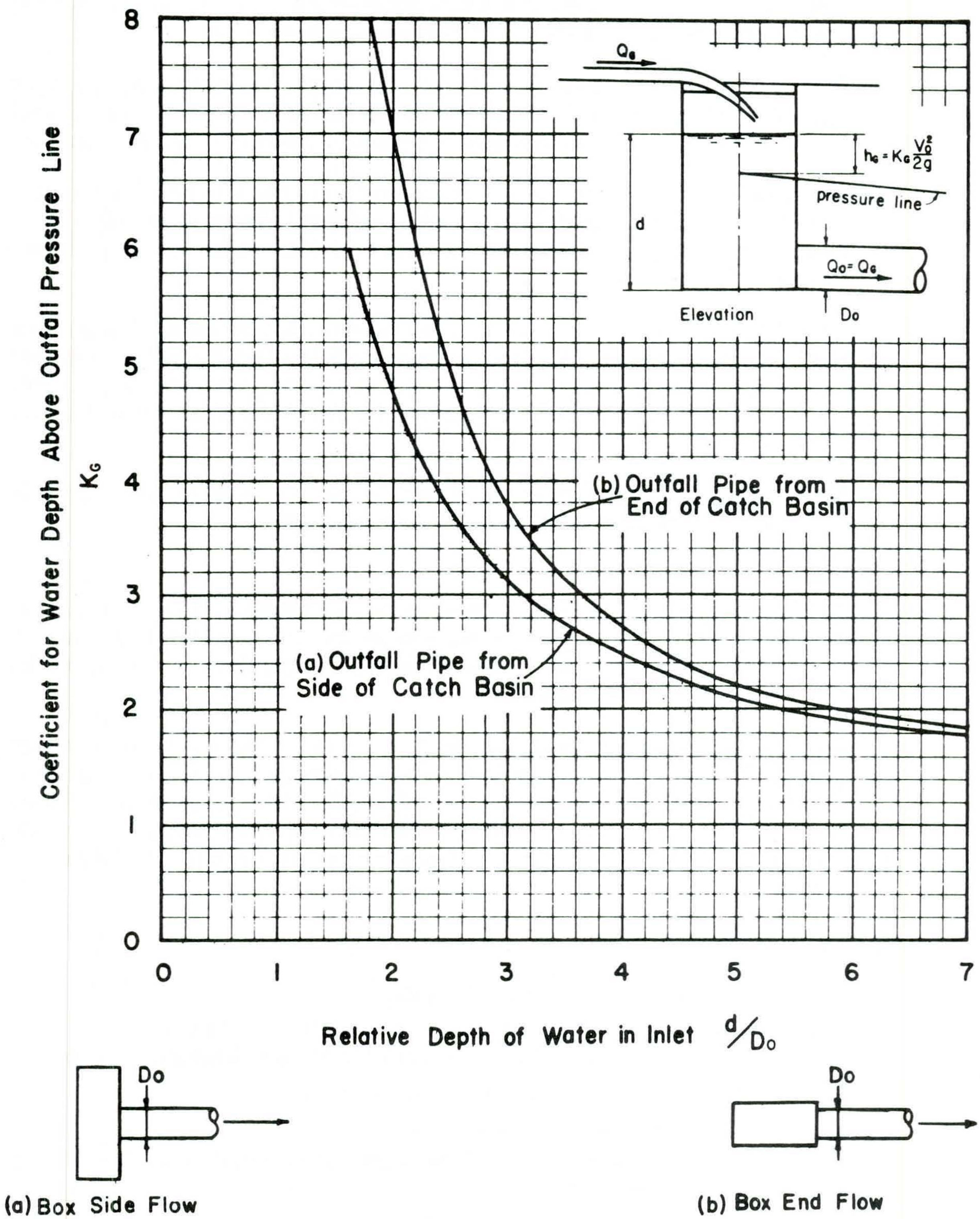


Figure 4.7
Catch Basin with Inlet Flow Only

» estimate K_G as follows:

7.0 for end outlet, 5.0 for side outlet—pressure line to bottom not over 2 pipe diameters.

4.0 for end outlet, 3.0 for side outlet—for higher pressure lines.

5. Calculate the estimated relative water depth of d/D_O.
6. Enter Figure 4.7 at this depth d/D_O and read K_G from the curve for the particular outfall pipe location.
7. Calculate h_G as indicated on the diagram on the figure and by Gen. Instr. 7.
8. Add h_G to the elevation of the outfall pressure line at the inlet center to obtain the water surface elevation in the inlet.
9. From this water surface elevation subtract the elevation of the inlet bottom to obtain a more precise value for the water depth, d.
10. Repeat the above procedure with the improved value of d from Step 9 if necessary. Such repetition may not be necessary if the estimated d/D_O of Step 5 was reasonably accurate.
11. Check to be sure that the inlet water elevation is at least 6 inches below the gutter elevation at the inlet so that inflow may be admitted.

4.2.4.2 Flow Straight through any Manhole: Pressure change coefficients are presented in Figure 4.8 for use in determining the elevation of the pressure line of an upstream in-line pipe relative to that of the outfall. The pipe centerlines must be parallel and not offset more than would permit the area of the smaller pipe to fall entirely within that of the larger if projected across the junction box along the pipe axis. The shape of the junction in plan is not significant in determining the pressure change. The effect of junction size and outfall pipe entrance conditions are included in Figure 4.8. Negative pressure changes occur with an upstream pipe smaller than the outfall pipe. That is, at the junction center, the upstream pressure line is below the outfall pressure line for this case. No flow other than that from the upstream in-line pipe may be involved where Figure 4.8 applies.

To use Figure 4.8:

1. Determine the outfall pipe pressure line elevation (Gen. Instr. 1).
2. Calculate the velocity head in the outfall (Gen. Instr. 2).
3. Calculate the size ratios D_UD_O and A/D_U (Gen. Instr. 3).
4. Note whether the outfall pipe entrance is to be square-edged or rounded smooth (note Gen. Instr. 6).
5. Enter Figure 4.8 at the pipe size ratio D_U/D_O and read K_U at the curve for the proper value of A/D_U for a square-edged entrance condition, or at the dashed curve for a rounded entrance.
6. Calculate h_U (positive or negative) as indicated on the diagrams on Figure 4.8 and by Gen. Instr. 7.

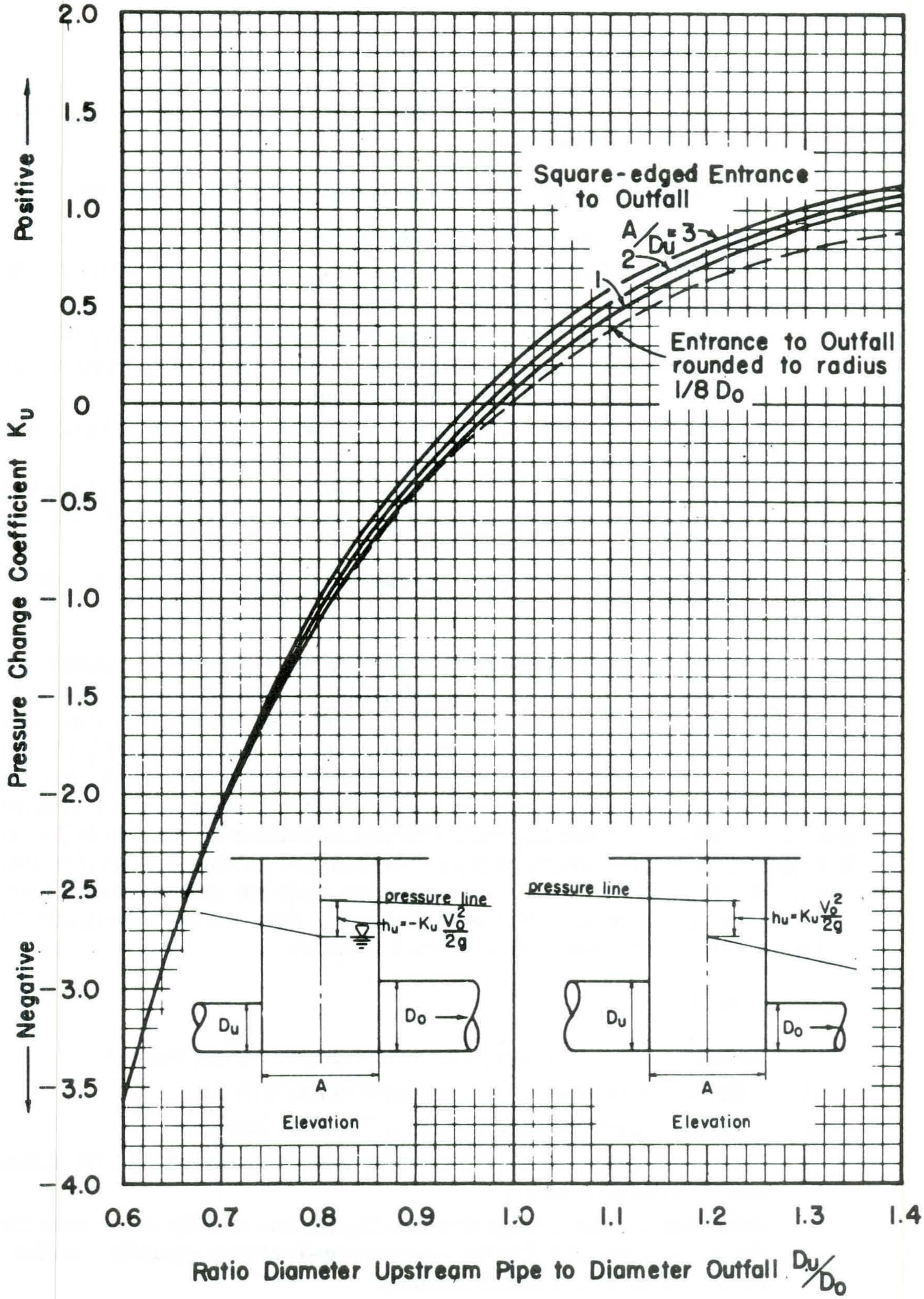


Figure 4.8
Flow Straight Through Any Manhole

7. Add a positive h_U to (or subtract a negative h_U from) the elevation of the outfall pressure at the junction center to obtain the elevation of the upstream pipe pressure line at the same location.
8. The water surface elevation in the junction corresponds to that of the upstream pipe, whether above or below the outfall pressure line.
9. Check to be sure the water surface elevation in the junction is at least 6 inches below the top of the junction box so that overflow may not occur.

Comments: For a square-edged entrance to the outfall pipe, values of A/D_U less than 1 do not appreciably reduce the values of K_U shown for $A/D_U = 1$. K_U increases for distances A/D_U greater than 3, but such values are not usual in storm drain construction. For rounded entrances, the curve shown will apply with sufficient accuracy for all values of A/D_U up to 3.

4.2.4.3 Rectangular Manhole—Through Pipeline—Lateral Pipeline: Coefficients for pressure changes are presented in Figure 4.9 for use in determining the common elevation of the upstream in-line pipe pressure line and the water surface in the manhole. The in-line pipes connect at the manhole sides (long dimension) and must meet the alignment requirement stated for Figure 4.9. As much as half the total flow may enter through a top inlet. The main graph of Figure 4.9 includes effects of various portions of grate flow for a relative water depth d/D_O of 2.5. Increments of K_U for other relative water depths are shown in the supplemental graphs; positive increments for d/D_O less than 2.5 and negative for greater depths.

To use Figure 4.9:

1. Determine the outfall pipe pressure line elevation (Gen. Instr. 1).
2. Calculate velocity head in the outfall (Gen. Instr. 2).
3. Calculate the ratios of D_U/D_O and Q_U/Q_O (Gen. Instr. 3). (The inlet flow ratio $Q_G/Q_O = 1 - Q_U/Q_O$).
4. Estimate a value for the water depth, d.
   » Follow Gen. Instr. 4.
   » Estimate $K = 3\, Q_G/Q_O$.
5. Calculate the corresponding relative water depth d/D_O.
6. If the estimated d/D_O is approximately 2.5, enter the lower graph on Figure 4.9 at the pipe size ratio D_U/D_O and read K_U at the curve of interpolated curve for Q_U/Q_O: then proceed as in Step 9.
7. If the estimated d/D_O is other than 2.5, follow Step 6, then enter the upper graph on Figure 4.9 at the given D_U/D_O and determine the increment of K_U required to account for the effects of the estimated relative water depth d/D_U.
8. Add K_U from Step 6 and the increment from Step 7 to determine the total value of K_U. Note that negative values of K_U may occur.
9. For a rounded outfall pipe entrance or one consisting of a pipe socket, reduce K_U according to Gen. Instr. 6.
10. Calculate h_U as indicated on the diagram on the Figure and by Gen. Instr. 7.

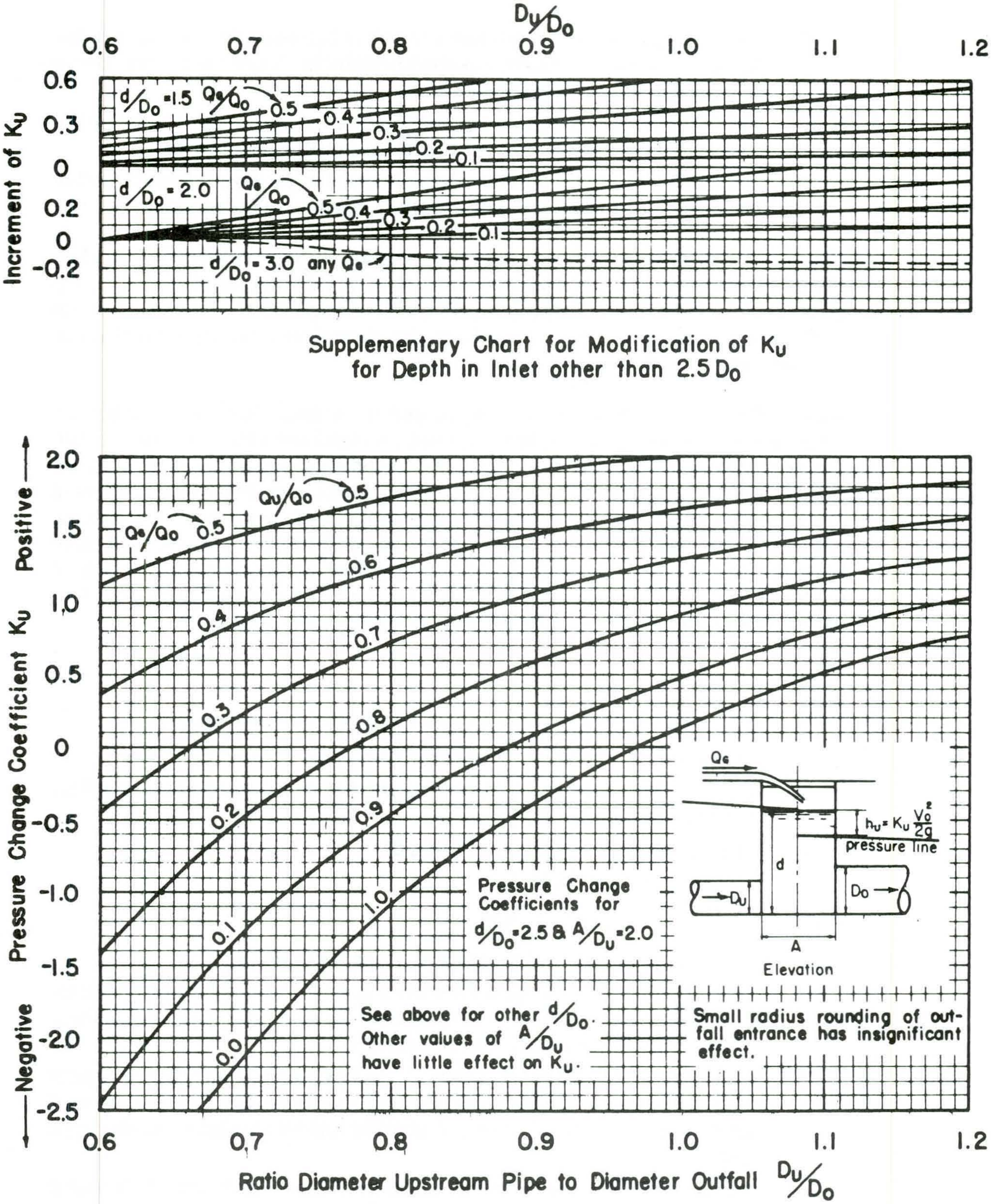


Figure 4.9
Rectangular Manhole with Through Pipeline and Inlet Flow

11. Add h_U to the elevation of the outfall pressure line at the inlet center to obtain a more precise value for the water depth, d.
12. Repeat the above procedure with the improved value of d from Step 11 if necessary. Such repetition may not be necessary if the original estimated d/D_O of Step 5 was reasonably accurate.
13. Check to be sure the water elevation is at least 6 inches below the gutter elevation at the inlet so that inflow may be admitted.

4.2.4.4 Rectangular Manhole—Upstream Main and 90 Degree Lateral Pipe—with or without Grate Flow: Pressure change coefficients are presented in Figure 4.10 for use in determining the common elevation of the two upstream pipe pressure lines and the water surface in the manhole. Flow into the combination inlet and junction box is supplied by an upstream main, in-line with the outfall and flowing through the short dimension of the manhole, and a 90 degree lateral pipe connected at one end of the box, supplemented by flow through a top inlet. The main graph of Figure 4.10 applies directly for no flow into the manhole through the inlet. Increments of K_U and K_L for inlet flow conditions are shown in the supplementary graphs of the upper portion of the figure.

To use Figure 4.10:

1. Determine the outfall pipe pressure line elevation (Gen. Instr. 1).
2. Calculate the velocity head in the outfall (Gen. Instr. 2).
3. Calculate the ratios D_U/D_O, Q_U/Q_O (Gen. Instr. 3).
4. If no inlet flow is involved, enter the lower graph on Figure 4.10 at the pipe size ratio D_U/D_O and read K_U (or K_L) at the curve or interpolated curve for Q_U/Q_O, then proceed as in Step 10.
5. With inlet flow, estimate a value for the water depth, d.
   » Follow Gen. Instr. 4
   » Estimate $K = 1.5$
6. Calculate the corresponding relative water depth d/D_O.
7. Enter the lower graph and obtain K_U (or K_L) as in Step 4, this value applying for $Q_G/Q_O = 0$.
8. Enter the appropriate upper graph on Figure 4.10 for the particular d/D_U nearest that estimated in Step 6 at the given D_U/D_O and determine the increment of K_U (or K_L) at the curve for Q_G/Q_O. This increment accounts for the effects of inlet flow and is always a positive value, even when K_U of Step 7 is negative.
9. Add K_U from Step 7 and the increment from Step 8 to obtain the total value of K_U. Note that in unusual cases the total value of K_U may be negative.
10. For a rounded outfall pipe entrance or one consisting of a pipe socket, reduce K_U and K_L according to Gen. Instr. 6.
11. Calculate h_U (also equal to h_L) as indicated by the diagram on the figure and by Gen. Instr. 7.
12. Add h_U to the elevation of the outfall pressure line at the branch point to obtain the elevation of the upstream in-line pipe pressure line at this point.

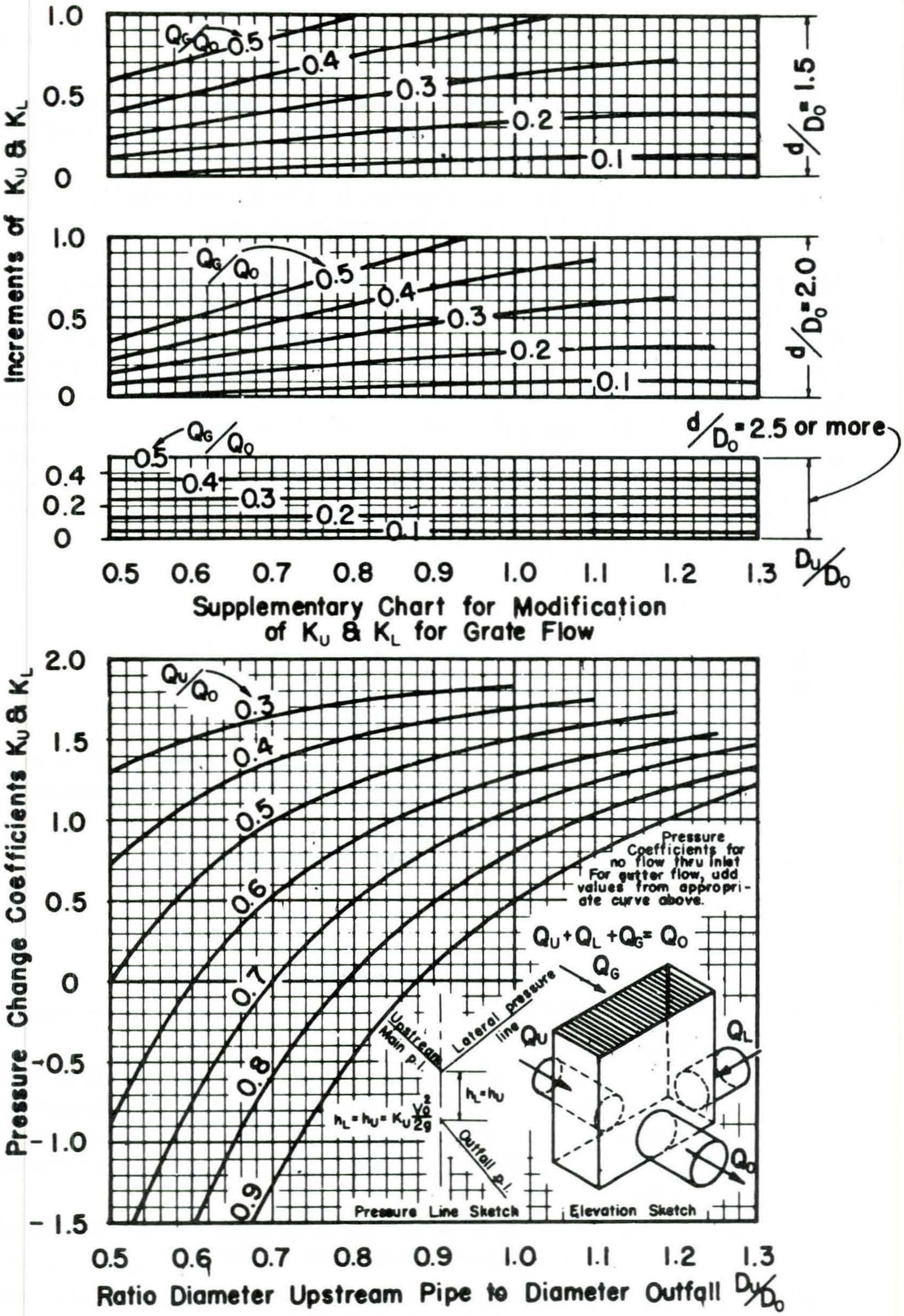


Figure 4.10
Rectangular Manhole with In-line Upstream Main and 90 Degree Lateral Pipe
(With or Without Inlet Flow)

The elevations of the upstream main pressure line, and lateral pipe pressure line and the water surface in the inlet will correspond.

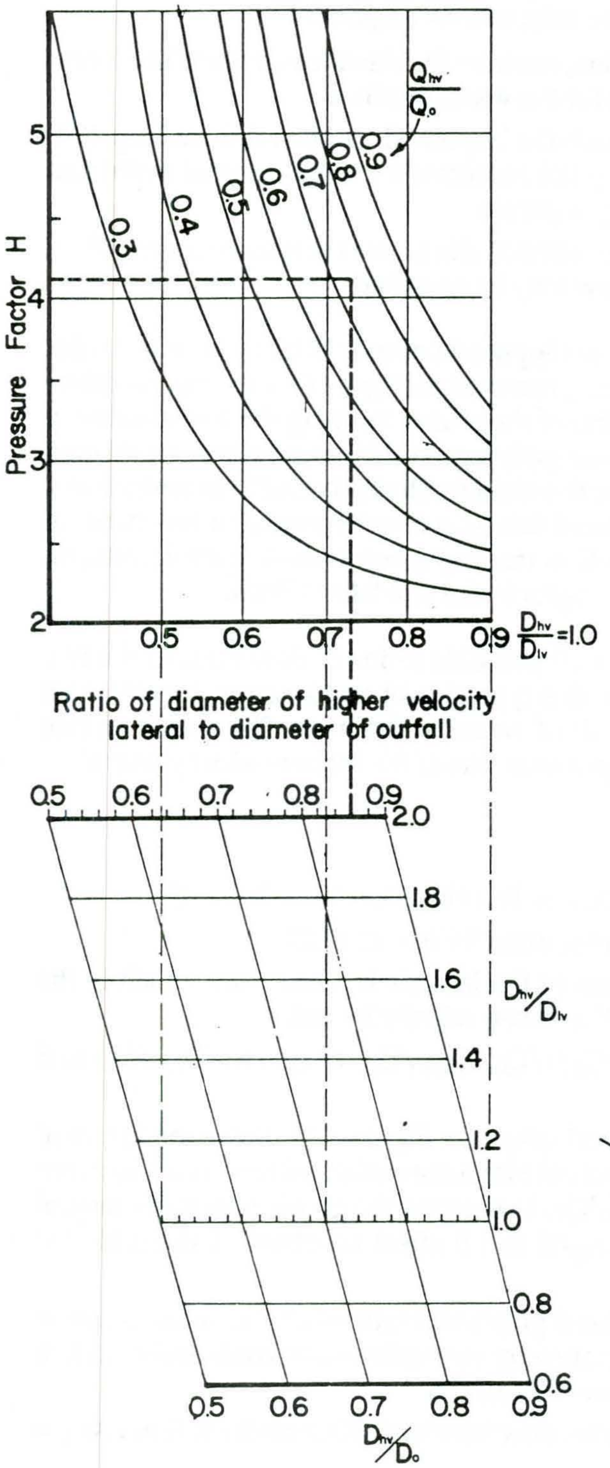
13. From this water surface elevation, subtract the elevation of the inlet bottom to obtain a more precise value for the water depth, d.
14. Repeat the above procedure with the improved value of d from Step 13 if necessary. Such repetition may not be necessary if the original estimated d/D_O of Step 6 was reasonably accurate.
15. Check to be sure the inlet water surface elevation is at least 6 inches below the top of the inlet so that inflow may be admitted.

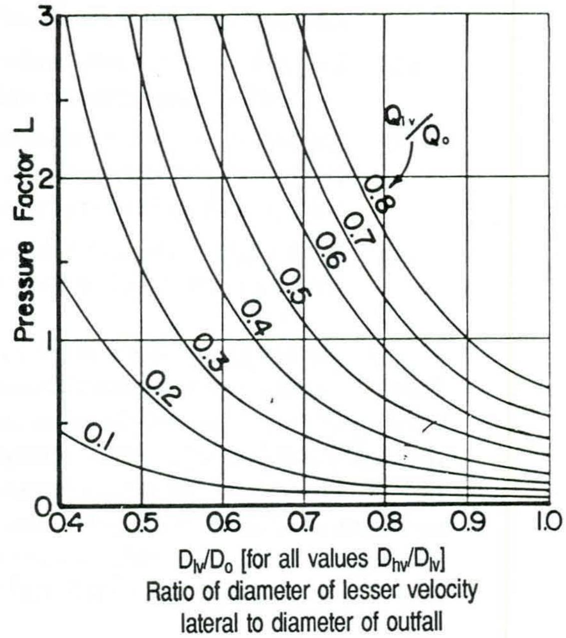
4.2.4.5 Rectangular Manhole—In-Line Opposed Laterals With or Without Inlet Flow: Pressure change coefficients are presented in Figure 4.11 for use in determining the elevation of the pressure line of the lateral carrying the lower-velocity flow of two in-line opposed lateral pipes supplying a combination junction and inlet box. The pressure change coefficient for the higher-velocity lateral is a constant and so is not read from the Figure. An inlet of this type may be used at a low point of street grade where lateral pipes supply flow from up-grade inlets in both directions, and the outfall pipe is located at right angles to the two lateral lines.

Figure 4.11 may be used for cases with all probable ratios of flow rates in the two laterals, with or without inlet flow. For this type of inlet and junction, the pressure changes are not modified by the depth of water in the inlet. The water surface elevation here will correspond to the pressure line of the higher-velocity lateral.

To use Figure 4.11:

1. Determine the outfall pipe pressure line elevation (Gen. Instr. 1).
2. Calculate the velocity head in the outfall (Gen. Instr. 2).
3. Calculate the velocities in each of the laterals to determine which is the higher-velocity and which is the lower-velocity lateral.
4. Calculate the ratios Q_G/Q_O, Q_{HV}/Q_O, Q_{LV}/Q_O, D_{HV}/D_O, D_{LV}/D_O and D_{HV}/D_{LV} (Gen. Instr. 3).
5. Determine H from the left-hand graph on Figure 4.11. Enter the figure at the pipe size ratio D_{HV}/D_O (note the relevant scale) and read H at the curve or interpolated curve for Q_{HV}/Q_O. In entering the graph, note that unequal size laterals (D_{HV}/D_{LV} not equal to 1.0 effect an offset of the scale for D_{HV}/D_O.).
6. Determine L from the right-hand graph on Figure 4.11. Enter the graph at the pipe size ratio D_{LV}/D_O (note only one scale is involved) and read L at the curve or interpolated curve of Q_{LV}/Q_O.
7. Calculate $K_{LV} = H - L$ with inlet flow involved. With no inlet flow, $K_{LV} = (H - L) - 0.2$.
8. $K_{HV} = 1.8$ with inlet flow involved. With no inlet flow, $K_{HV} = 1.6$.
9. Calculate $h_{LV} = K_{LV} (V_O^2/2g)$ and $h_{HV} = K_{HV} (V_O^2/2g)$.
10. Add h_{LV} to the elevation of the outfall pipe pressure line at the branch point to obtain the elevation of the lower- velocity lateral pressure line at this





To find K_R or K_L for the right or left lateral pipe with flow at a lesser velocity than the other lateral, read H for the higher velocity lateral D and Q, then read L for the lower velocity lateral D and Q; then: K_R (or K_L) = H − L

K_R or K_L for the lateral pipe with higher velocity flow is always 1.8

$$h_L = K_L \frac{V_o^2}{2g} \qquad h_R = K_R \frac{V_o^2}{2g}$$

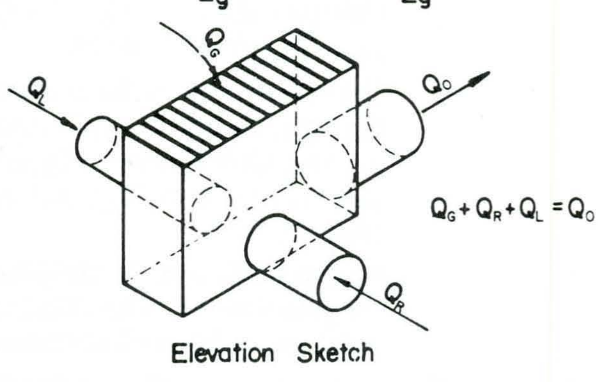


Figure 4.11
Rectangular Manhole with In-line Opposed Lateral Pipes Each at 90 Degrees to Outfall
(With or Without Grate Flow)
(University of Missouri 1958)

point, similarly, add h_{HV} to the outfall pipe pressure line elevation to obtain the elevation of the higher-velocity lateral pressure line at the branch point.

11. Determine the water surface elevation of the inlet, which is equal to the lower of the two lateral pressure line elevations that of the higher-velocity lateral).
12. Check to be sure the inlet water surface elevation is at least 6 inches below the top of the inlet so that inflow may be admitted.

4.2.4.6 Rectangular Manhole—Offset Opposed Laterals—With or Without Inlet Flow: Pressure change coefficients are presented in Figure 4.12 for use in determining the elevations of the pressure lines of each of the two horizontally offset opposed lateral pipes supplying a combination junction and inlet box. The inlet is used in the same situations as those to which Figure 4.12 applies, but the pressure rise of the lower-velocity lateral is restricted by locating the lateral pipes to enter opposite sides of the inlet box with their centerlines horizontally offset a distance not less than the sum of the two lateral pipe diameters. One lateral enters one side of the box near the outfall pipe end, and one, designated the far lateral, enters the opposite side near the other end.

Figure 4.12 is used for all probable ratios of flow rates in the two laterals, with or without inlet flow. For this type of junction the pressure changes are not modified by the depth of water in the manhole. The water surface elevation here will correspond to the pressure line of the far lateral.

To use Figure 4.12:

1. Determine the horizontal distance between the centerlines of the opposed flow laterals at the inlet; if it is more than the sum of the pipe diameters, Figure 4.12 will apply.
2. Determine the outfall pipe pressure line elevation at the branch points (Gen. Instr. 1). An average elevation applicable to both is sufficient.
3. Calculate the velocity head in the outfall (Gen. Instr. 2).
4. Calculate the ratios Q_F/Q_O, Q_N/Q_O, D_F/D_O, and D_N/D_O, observing the nomenclature of Figure 4.12 (Gen. Instr. 3).
5. Calculate the factors $Q_F/Q_O \times D_O/D_F$ and $Q_N/Q_O \times D_O/D_N$.
6. For the far lateral, enter the right-hand graph of Figure 4.12 at the abscissa value from Step 5 and read K_F at the curve or interpolated curve for D_F/D_O.
7. For the near lateral, obtain K_N from the left-hand graph by a similar procedure.
8. For a manhole with inlet flow, calculate h_F and h_N by multiplying the outfall velocity head by the corresponding coefficient K_F or K_N.
9. For a junction without inlet flow, calculate h_F and h_N by multiplying the outfall velocity head by the corresponding reduced coefficients (K_F - 0.2) or (K_N - 0.2).
10. Add h_F and h_N to the elevation of the downstream (outfall pipe) pressure line to obtain the elevations of the pressure lines of the two laterals at their branch points.

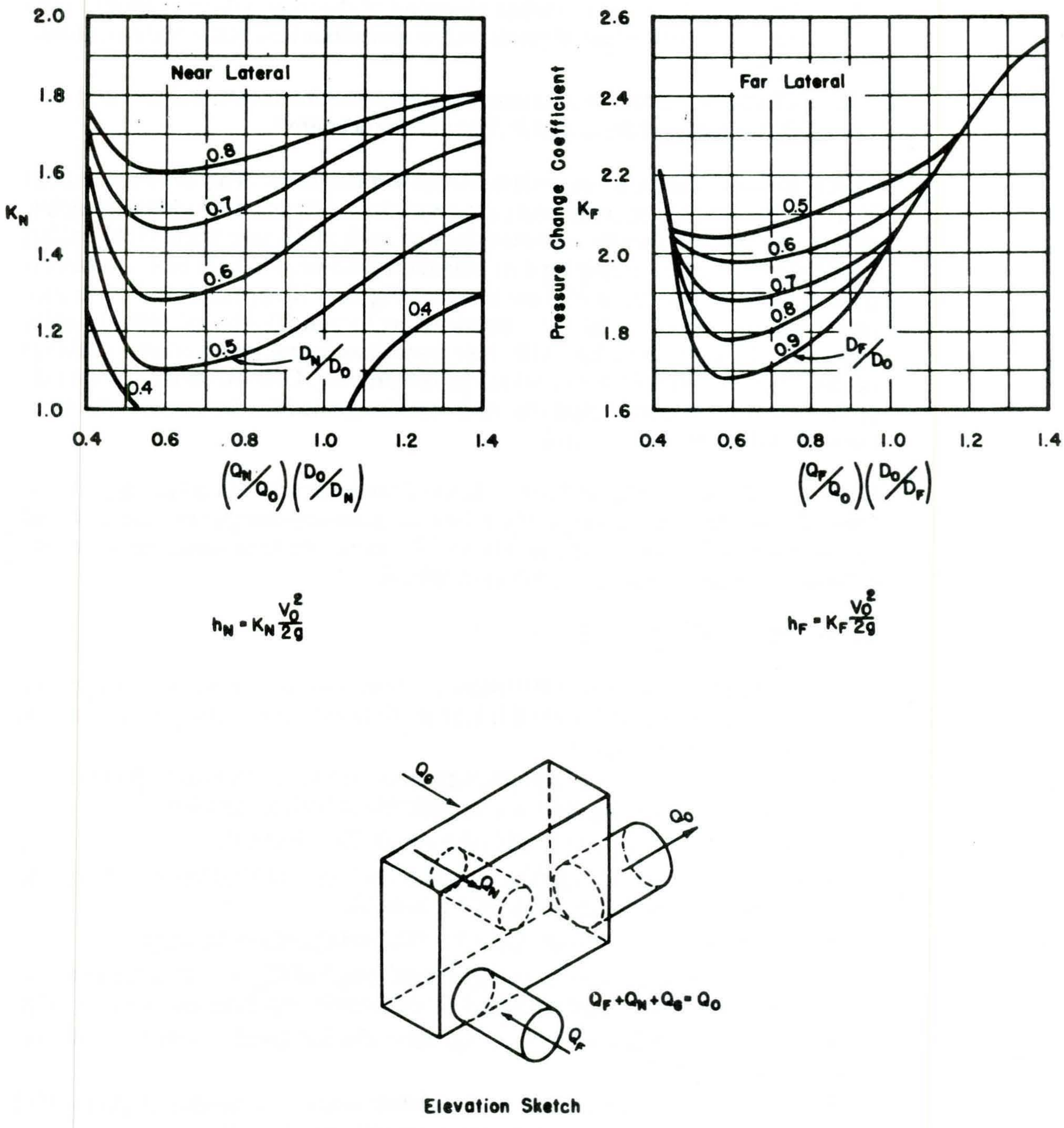


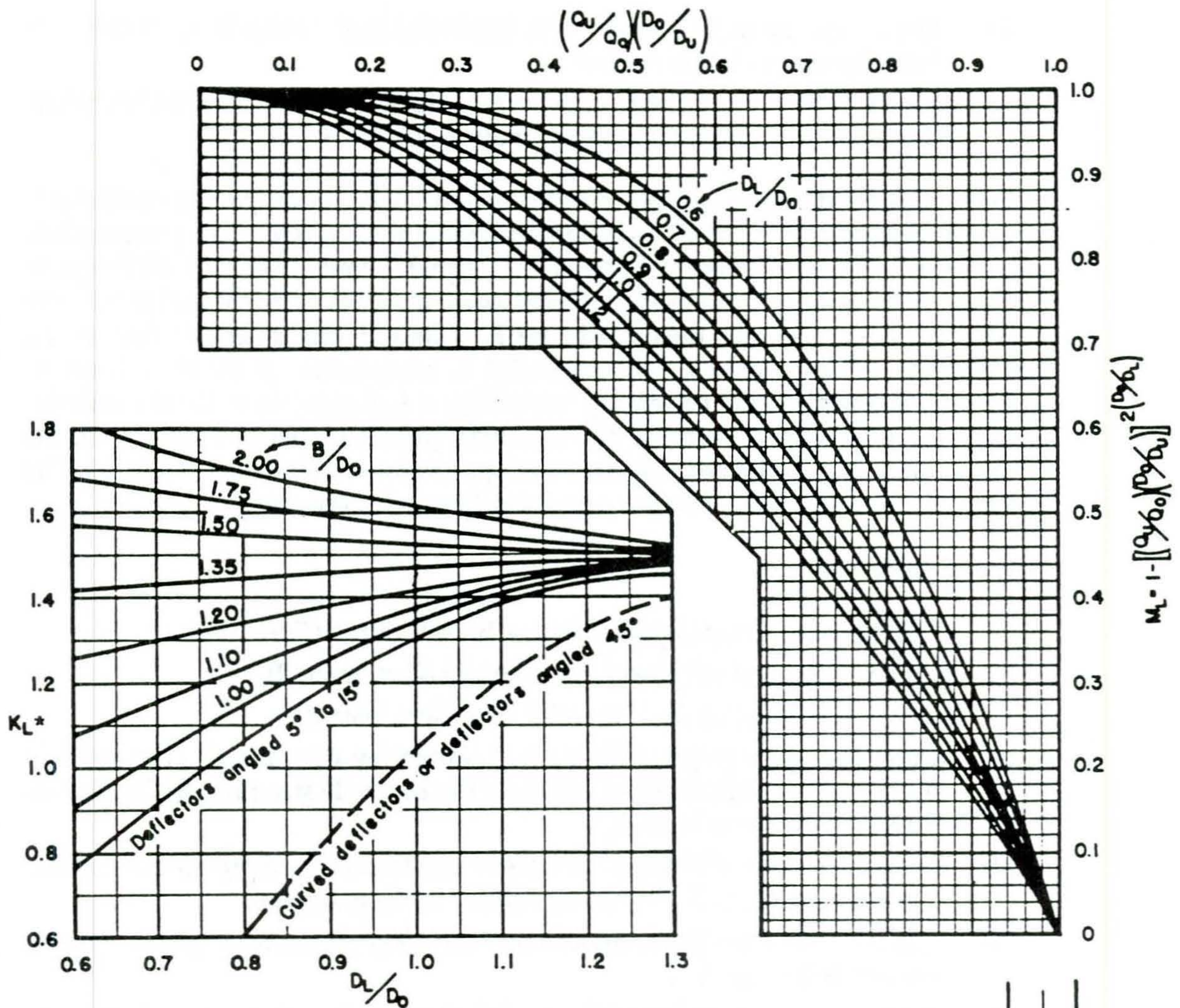
Figure 4.12
Rectangular Manhole with Offset Opposed Lateral Pipes—Each at 90 Degrees to Outfall
(With or Without Inlet Flow)
(Univeristy of Missouri 1958)

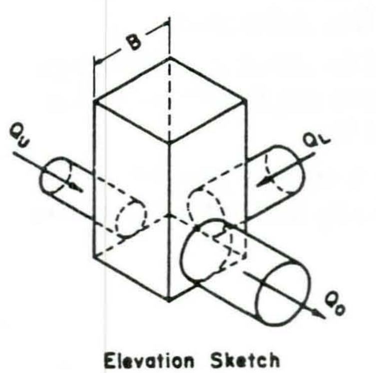
11. Determine the water surface elevation in the inlet, which is equal to the far lateral pressure line elevation.
12. Check to be sure the inlet water surface elevation is at least 6 inches below the top of the inlet so that inflow may be admitted.

4.2.4.7 Square Manhole—90 Degree Deflection: Pressure change coefficients are presented in Figure 4.13 for use in determining the elevation of the pressure line of an upstream pipe connected by means of a square manhole to an outfall pipe at a 90 degree angle. The manhole conditions covered by this figure do not involve an upstream pipe in-line with the outfall pipe. For this and other manhole figures, the lateral pipe is designated by the subscript L irrespective of its right-hand or left-hand position. The coefficients given in Figure 4.13 apply directly to manholes having a square-edged entrance to the outfall pipe. Coefficients for a rounded entrance are obtained by reduction of the Figure values as shown in Table 4.3. The design of manholes with deflector devices is discussed separately.

To use Figure 4.13:

1. Determine the outfall pipe pressure line elevation (Gen. Instr. 1).
2. Calculate the velocity head in the outfall (Gen. Instr. 2).
3. Calculate the ratios D_L/D_O and B/D_O (Gen. Instr. 3).
4. Enter the lower graph of Figure 4.13 at the pipe size ratio D_L/D_O and K_L^* at the curve or interpolated curve for the manhole size ratio B/D_O. For all flow from a lateral $K_L = K_L^*$.
5. For a rounded outfall pipe entrance or one formed by a pipe socket reduce the figure value of K_L^* by 0.3 as defined by Gen. Instr. 6.
6. Calculate the change of pressure $h_L = K_L \times V_O^2/2g$ (always positive for 90 degree deflections).
7. Add h_L to the elevation of the outfall pressure line at the branch point to obtain the elevation of the lateral pipe pressure line at this point.
8. The water surface elevation in the manhole will be above the lateral pipe pressure line. To determine the water-surface elevation, use Figure 4.14, as instructed in Steps 12 through 18 of Section 4.2.4.10.
9. Check to be sure the water surface elevation is above the pipe crowns to justify using these figures and that it is sufficiently below the top of the manhole to indicate safety from overflow.





To find K_L for the lateral pipe, first read $\bar{K}_L*$ from the lower graph. Next determine M_L. Then

$$K_L = K_L* \times M_L$$

Dashed curve for curved or 45° angle deflectors applies only to manholes without upstream in-line pipe.

Use this chart for round manholes also.

For rounded entrance to outfall pipe, reduce chart values of $\bar{K}_L*$ by 0.2 for combining flow.

For $Q_U/Q_O \times D_O/D_U > 1$ use

For $D_L/D_O < 0.6$ use

$$h_L = K_L \frac{V_O^2}{2g}$$

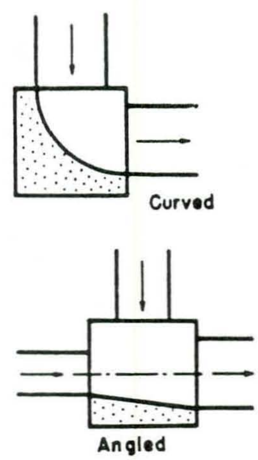


Figure 4.13
Manhole at 90 Degree Deflection or On Through Pipeline at Junction of 90 Degree Lateral Pipe (Lateral Coefficient)
(University of Missouri 1958)

Table 4.3
Reductions for K_L* for a Manhole with Rounded Entrance
Reductions For K_L

B/D_O \ D_L/D_O	0.6	0.8	1.0	1.2
1.75	0.4	0.3	0.2	0.0
1.33	0.3	0.2	0.1	0.0
1.10	0.2	0.1	0.0	0.0

4.2.4.8 Round Manhole—90 Degree Deflection: Pressure change coefficients may also be obtained from Figure 4.13 for use in determining the elevation of the pressure line of an upstream pipe connected by means of a round manhole to an outfall pipe at a 90 degree angle.

To use Figure 4.13 in this case:

1. Proceed as instructed in Steps 1 through 4 of Section 4.2.4.7 to obtain a base value of K_L* for the particular values of D_L/D_O and B/D_O.
2. To provide for the effects of the round manhole cross section, reduce K_L* in accordance with Table 4.3. The reduced values apply for a sharp-edged entrance to the outfall pipe.
3. With a well-rounded entrance to the outfall pipe from a round manhole, reduce K_L* of Step 1 by 0.3 with no further reduction for manhole cross section shape.
4. Follow Steps 6 through 9, Section 4.2.4.7.

4.2.4.9 Square or Round Manhole—90 Degree Deflection With Deflectors: Pressure change coefficients are presented in Figure 4.13 for determining the evaluation of the pressure line of an upstream pipe connected to an outfall pipe at a 90 degree angle by means of a square or round manhole modified by flow deflectors. Deflectors in a manhole effectively eliminate the effects related to the shape of the manhole. The basic types of deflector walls which may be constructed in square or round manholes to reduce the pressure loss are detailed and described in the main text.

The deflectors which are more easily constructed and are as effective as more complex types provide a vertical wall to guide the flow toward the outfall pipe diameter and must fill in that part of the manhole opposite the lateral pipe exit so that it is flush with the side of the outfall pipe. Three basic types of such deflector walls are possible and are included in the curves of Figure 4.13. These three are: 1) walls parallel to the outfall pipe centerline or 0 degree walls; 2) inclined walls, limited to an angle of about 15 degrees to the outfall centerline if an upstream in-line pipe is to be used; and 3) walls at 45 degrees to both the lateral and outfall pipes, or walls curved on a radius of about the manhole dimension extending from lateral to outfall, and therefore to be used only when no upstream in-line pipe is involved.

Rounding of the corner formed between the deflector wall and the manhole floor is not required, and may be detrimental in some cases.

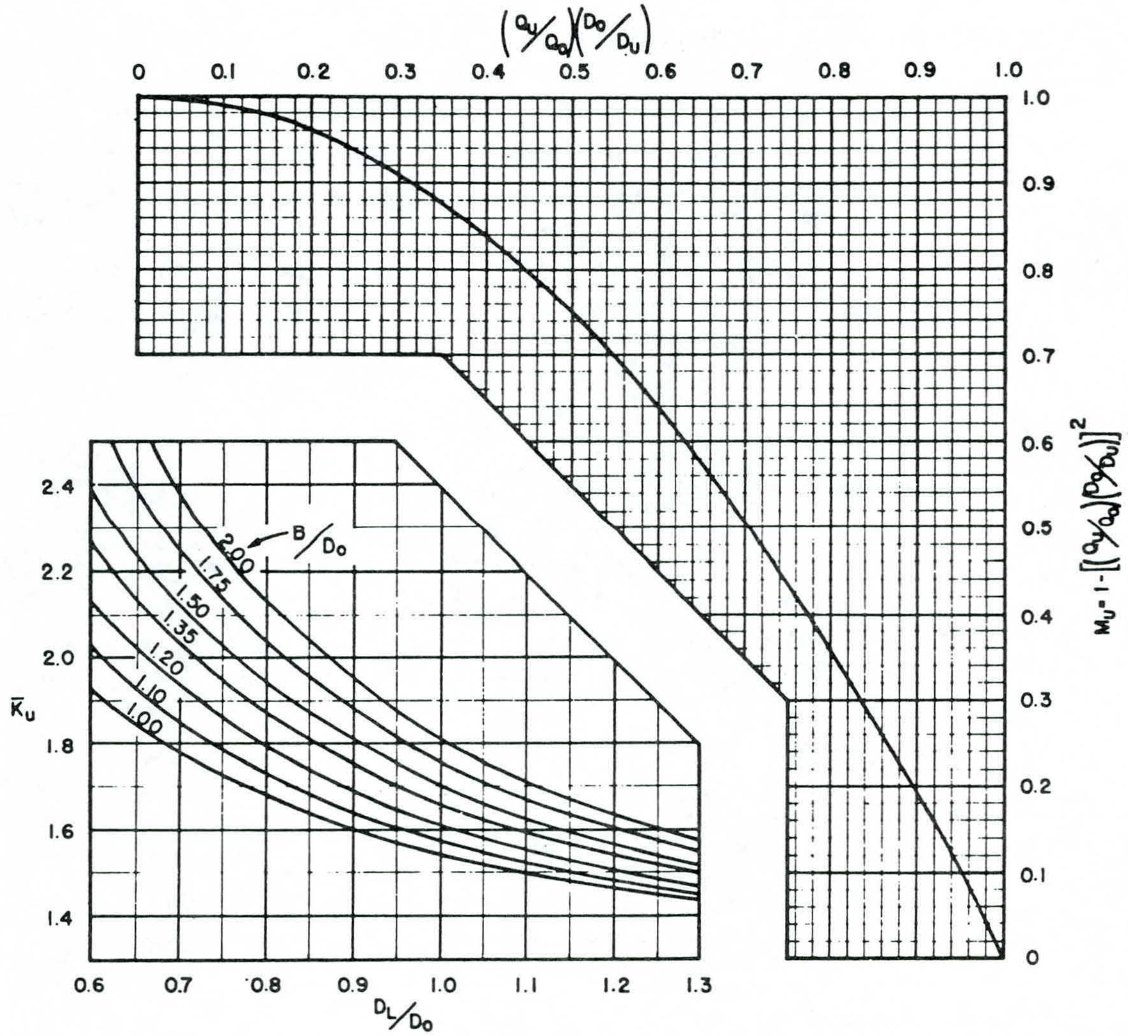
To use Figure 4.13 in this case:

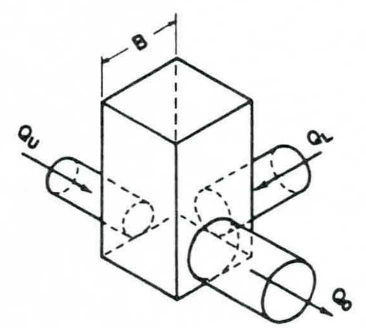
1. Determine the outfall pipe pressure line elevation (Gen. Instr. 1).
2. Calculate the velocity head in the outfall (Gen. Instr. 2).
3. Classify the type of deflector used:
   » Parallel wall - 0 degrees
   » Inclined wall - 5 to 15 degrees
   » 45 degree or curved wall
4. Calculate the ratios D_L/D_O and B/D_O. No distinction between square and round manholes is necessary.
5. If B/D_O is 1.5 or less, enter the lower graph of the figure at the ratio D_L/D_O and read K_L* at the curve for the appropriate deflector type. In the case of a parallel wall, use the curve for $B/D_O = 1.00$.
6. If B/D_O is more than 1.5 and less than 2.0, use the same dashed curve for 45 degree or curved deflectors; use the curve for $B/D_O = 1.10$ for 5 to 15 degree angle deflectors; and use the curve for $B/D_O = 1.20$ for 0 degree angle deflectors.
7. A rounded entrance to the outfall pipe or one formed by a pipe socket is less effective in reducing the pressure change with deflectors than when deflectors are not used. A reduction of K_L* by 0.1 may be justified.
8. Calculate the change of pressure:

$$h_L = K_L V_O^2/2g \quad (\text{for } Q_L = Q_O, K_L = K_L^*)$$

9. Add h_L to the elevation of the outfall pressure line at the branch point to obtain the elevation of the lateral pipe pressure line at this point.
10. The water-surface elevation in the manhole will be above the lateral pipe pressure line. To determine the water surface elevation, use Figure 4.14, as instructed in Steps 2 through 8 for deflectors in a manhole at the junction of a 90 degree lateral with a through main (Section 4.2.4.12).
11. Check to be sure the water surface elevation is above the pipe crowns to justify using these figures and that it is sufficiently below the top of the manhole to indicate safety from overflow.

4.2.4.10 Square Manhole—Upstream Pipe and Lateral: Pressure change coefficients for use in determining the elevation of the pressure line of the 90 degree lateral pipe are obtained from Figure 4.13 and the coefficients for the upstream in-line pipe are obtained from Figure 4.14. The diameter of the lateral pipe must be at least 0.6 of the diameter of the outfall pipe to permit use of these figures. Pressure changes at junctions of smaller laterals may be obtained through use of Figure 4.15. The coefficients given by the figures apply directly to a square-edged entrance to the outfall pipe. Coefficients for a rounded entrance are obtained by reduction of the figure values as stated below. The design of manholes with deflector devices is discussed separately.





Elevation Sketch

To find K_U for the upstream main, first read K_U^* from the lower graph. Next determine M_U. Then

$$K_U = K_U^* \times M_U$$

For manholes with deflectors at 0° to 15°, read K_U^* on curve for $B/D_O = 1.0$

Use this chart for round manholes also.

For rounded entrance to outfall pipe, reduce chart values of K_U^* by 0.2 for combining flow.

For deflectors refer to sketches on

For $Q_U/Q_O \times D_O/D_U > 1$ use

For $D_L/D_O < 0.6$ use

$$h_U = K_U \frac{V_O^2}{2g}$$

Figure 4.14
Manhole on Through Pipeline at Junction of a 90 Degree Lateral Pipe
(In-Line Pipe Coefficient)
(University of Missouri 1958)

To use Figures 4.13 and 4.14 in this case:

1. Determine the outfall pressure line elevation (Gen. Instr. 1).
2. Calculate the velocity head in the outfall (Gen. Instr. 2).
3. Calculate the ratios Q_U/Q_O, D_U/D_O, and D_L/D_O. If D_L/D_O is less than 0.6, use Figure 4.15 instead of Figures 4.13 and 4.14.
4. Calculate the ratio B/D_O and note if the outfall entrance is rounded.
5. Calculate the factor $(Q_U/Q_O) \times (D_U/D_O)$; if this is greater than 1.00, use Figure 4.15, instead of Figures 4.13 and 4.14.

For Lateral Pipe:

6. Enter the lower graph of Figure 4.13 at the ratio of D_L/D_O and read K_L* at the curve or interpolated curve for the ratio B/D_O.
7. For a rounded outfall pipe entrance or one formed by a pipe socket as defined by Gen. Instr. 6, reduce the figure values of K_L* by 0.2.
8. Determine the factor M_L by entering the upper graph of Figure 4.13 at the value of the factor $(Q_U/Q_O) \times (D_U/D_O)$ and at the curve or interpolated curve for D_L/D_O.
9. Calculate $K_L = M_L \times K_L^*$.
10. Calculate the lateral pipe pressure change

$$K_L \times V_O^2/2g$$

11. Add h_L to the elevation of the outfall pipe pressure line at the branch point to obtain the elevation of the lateral pipe pressure line at this point.

For Upstream In-Line Pipe:

12. Enter the lower graph of Figure 4.14 at the ratio of D_L/D_O and read K_U* at the curve or interpolated curve for B/D_O.
13. For a rounded entrance to the outfall pipe or one formed by a pipe socket, reduce K_U* by 0.2.
14. Determine the factor M_U from the upper graph of Figure 4.14.
15. Calculate $K_U = M_U \times K_U^*$.
16. Calculate the upstream in-line pipe pressure change

$$h_U = K_U \times V_O^2/2g$$

17. Add hU to the elevation of the outfall pipe pressure line at the branch point to obtain the elevation of the upstream in-line pipe pressure line at this point.

For Water Surface:

18. The water-surface elevation in the manhole will correspond to the upstream in-line pipe pressure line at the branch point.

19. Check to be sure that the water surface elevation is above the pipe crown to justify using these figures and that it is sufficiently below the top of the manhole to indicate safety from overflow.

4.2.4.11 Round Manhole—Upstream Pipe and Lateral: Pressure change coefficients may also be obtained from Figures 4.13 and 4.14 for use in determining the elevations of the pressure lines of the 90 degree lateral pipe and the upstream in-line pipe connected by a round manhole to an outfall pipe.

To use Figures 4.13 and 4.14 in this case:

1. Proceed as instructed in Steps 1 through 6, Section 4.2.4.10 to obtain a base value of K_L*.

For Lateral Pipes:

2. To provide for the effects of the round manhole cross sections, reduce K_L* as outlined in Table 4.3. The reduced values apply for a square-edged entrance to the outfall pipe.
3. With a well-rounded entrance to the outfall pipe from a round manhole, reduce K_L* obtained in step 2 by 0.1.
4. Determine the factor M_L from the upper graph of Figure 4.13 and proceed as instructed in Steps 8 through 11 of Section 4.2.4.10 to complete the determination of the elevation of the lateral pipe pressure line.

Upstream In-Line Pipe:

5. Proceed as instructed in Steps 12 through 17 of Section 4.2.4.10 to obtain the elevation of the upstream in-line pipe pressure line. Note that no reduction of K_U* is to be made for effects of the round manhole cross section.

For Water Surface:

6. Proceed as instructed in Steps 18 and 19 of Section 4.2.4.10.

4.2.4.12 Square or Round Manhole—Upstream Pipe and Lateral—Deflector: Pressure change coefficients are also presented in Figures 4.13 and 4.14 for use in determining the elevations of the pressure lines of the lateral and in-line pipes at a junction of this type, with either a square or a round manhole modified by flow deflectors. Deflectors in a manhole effectively eliminate the effects related to the shape of the manhole. Deflector types are described in the instructions for use of Figure 4.13 for a manhole with deflectors at a 90 degree deflection of a storm drain. The curved and 45 degree deflectors cannot be used in a manhole on a through pipeline because of the space required for through in-line flow.

To use Figures 4.13 and 4.14 in this instance:

1. Proceed as instructed in Steps 1 through 9 of Section 4.2.4.9, disregarding the references to 45 degree or curved walls. Using Figure 4.13 will give the

elevation of the lateral pipe pressure line at the branch point. As noted in the instructions for a manhole of this type without deflectors, Figure 4.15 must be used when $D_L/D_O < 0.6$ or $(Q_U/Q_O \times D_U/D_O) > 1.00$.

For Upstream In-Line Pipe:

2. Enter the lower graph of Figure 4.14 at the ratio of D_L/D_O and read K_U* for all manhole sizes and any deflector wall angle from 0 to 15 degrees at the curve for $B/D_O = 1.00$.
3. For a rounded entrance to the outfall pipe or one formed by a pipe socket, reduce K_U* by 0.1.
4. Determine the factor M_U from the upper graph of Figure 4.14.
5. Calculate $K_U = M_U \times K_U^*$.
6. Calculate the upstream in-line pipe pressure change

$$h_U = K_U \times V_O^2/2g$$

7. Add h_U to the elevation of the outfall pipe pressure line at the branch point to obtain the elevation of the upstream in-line pipe pressure line at this point.

For Water Surface:

8. The water-surface elevation in the manhole will correspond to the upstream in-line pipe pressure line at the branch point.
9. Check to be sure that the water-surface elevation is above the pipe crowns to justify using these figures and that it is sufficiently below the top of the manhole to indicate safety from overflow.

4.2.4.13 Square or Round Manhole—Upstream Pipe with Small Lateral or Lateral Connecting with no Manhole: Pressure change coefficients are presented in Figure 4.15 for use in determining the common elevation of the pressure lines of the lateral and in-line pipes at a junction of this type for cases of pipe sizes or flow divisions outside the range over which Figures 4.13 and 4.14 may be applied. Figures 4.13 and 4.14 are more reliable within their range and should be used if possible. Neither manhole shape nor size nor relative size of lateral pipe modify the coefficients of Figure 4.15. The figure may also be used for direct connection of a 90 degree lateral to a main without use of a manhole. The coefficients of the figure apply directly to a square-edged entrance to the outfall pipe. Coefficients for a rounded entrance are obtained by reduction of the figure values as stated below. Deflectors in the manhole are not effective in the ranges covered by Figure 4.15 and therefore need not be used.

To use Figure 4.15:

1. Determine the outfall pipe pressure line elevation (Gen. Instr. 1).
2. Calculate the velocity head in the outfall (Gen. Instr. 2).
3. Calculate the ratios D_L/D_O, D_U/D_O, and Q_U/Q_O. Note that use of Figures 4.13 and 4.14 is advisable if the size and flow factors are within their range.

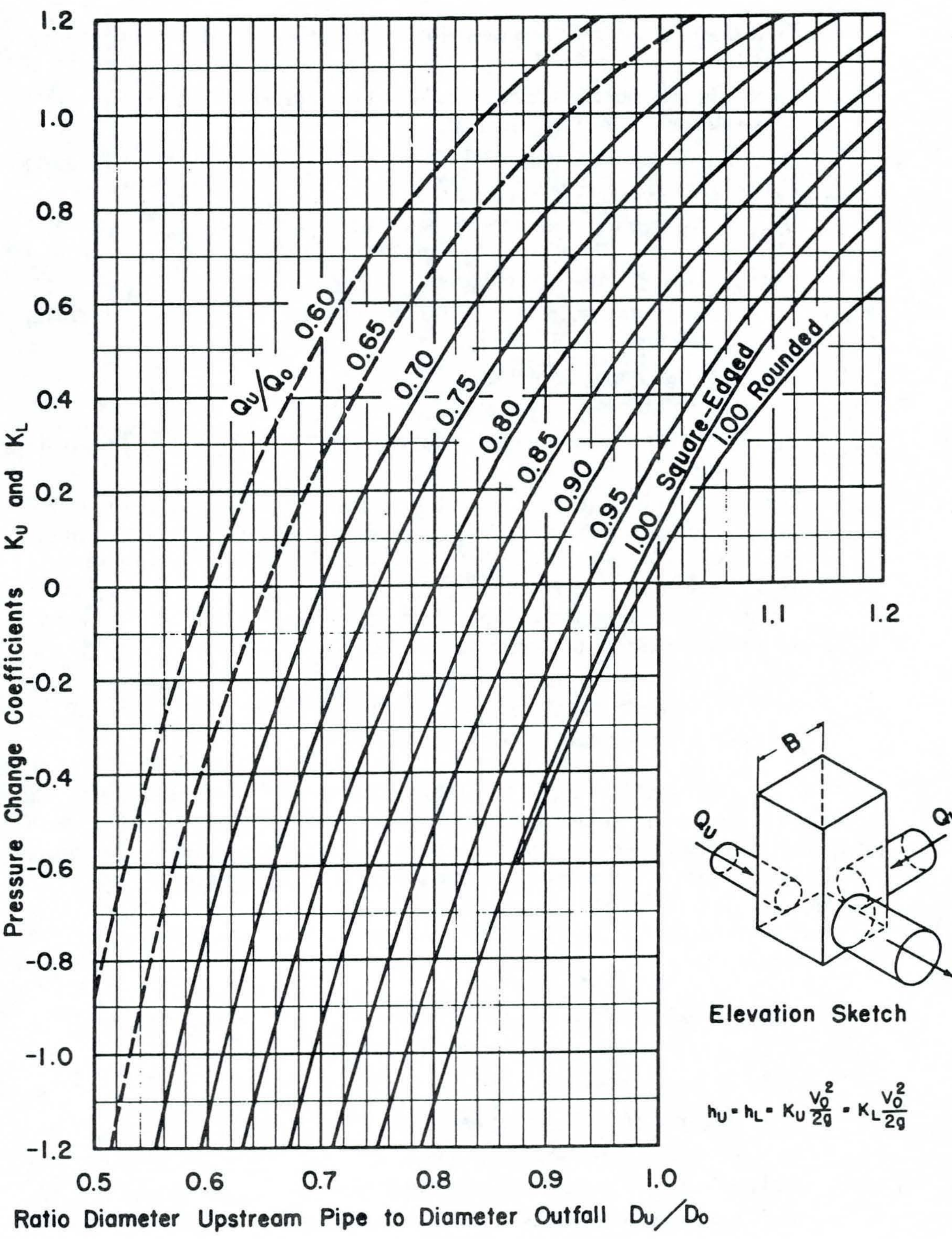


Figure 4.15
Manhole on Through Pipeline at Junction of a 90 Degree Lateral Pipe
(For Conditions Outside the Range of Figures 4.13 and 4.14)
(University of Missouri 1958)

Figure 4.15 should not be used for $Q_U/Q_O \leq 0.7$ if other solutions are possible.

4. Note whether the outfall entrance is to be rounded or formed by a pipe socket as defined by Gen. Instr. 6.
5. Enter Figure 4.15 at the ratio D_U/D_O and read K_U (also equal to K_L) at the curve or interpolate curve for Q_U/Q_O.
6. If $Q_U/Q_O \times D_U/D_O$ was found to be greater than 1.00 in an attempt to use Figures 4.13 and 4.14, K_U of Step 5 will be negative in sign, thus providing a check on proper use of the figures.
7. For rounded entrance from the manhole to the outfall pipe use the reduced values from Figure 4.15.
8. Calculate the change of pressure

$$h_U = h_L = K_U \times V_O^2/2g$$

   h_U and h_L are positive or negative depending on the sign of K_U as read from the figure.
9. Add a positive h_U to or subtract a negative h_U from the elevation of the outfall pipe pressure line at the branch point to obtain the elevation of the upstream in-line pipe pressure line at this point. The elevation of the lateral pipe pressure line at the branch point and the water surface elevation in the manhole will correspond to the upstream in-line pipe pressure line elevation found in Step 9 of Section 4.2.4.12.
10. Check to be sure that the water-surface elevation is above the pipe crowns to justify using these figures and that it is sufficiently below the top of the manhole to indicate safety from overflow.

4.2.4.14 Flow Straight Through a Deflection: Pressure change coefficients are presented in Figure 4.16 for use in determining the elevation of the pressure line of an upstream in-line pipe relative to that of the outfall. The cases to which the Figure may be applied are shown on the figure. No flow other than that from the upstream pipe may be involved where this figure is applied.

To use Figure 4.16:

1. Determine the outfall pipe pressure line elevation (Gen. Instr. 1).
2. Calculate the velocity head in the outfall (Gen. Instr. 2).
3. Determine the deflection angle, α.
4. Enter Figure 4.16 at the particular deflection angle to the proper curve and read the appropriate loss coefficient.
5. Calculate h_U (Gen. Instr. 7).
6. Add a positive h_V to the elevation of the outfall pressure line at the manhole center to obtain the elevation of the upstream pipe pressure line at the same location.
7. The water surface elevation in the manhole corresponds to that of the upstream pipe.
8. Check to be sure the water surface elevation in the junction is at least 6 inches below the top of the manhole so that overflow may not occur.

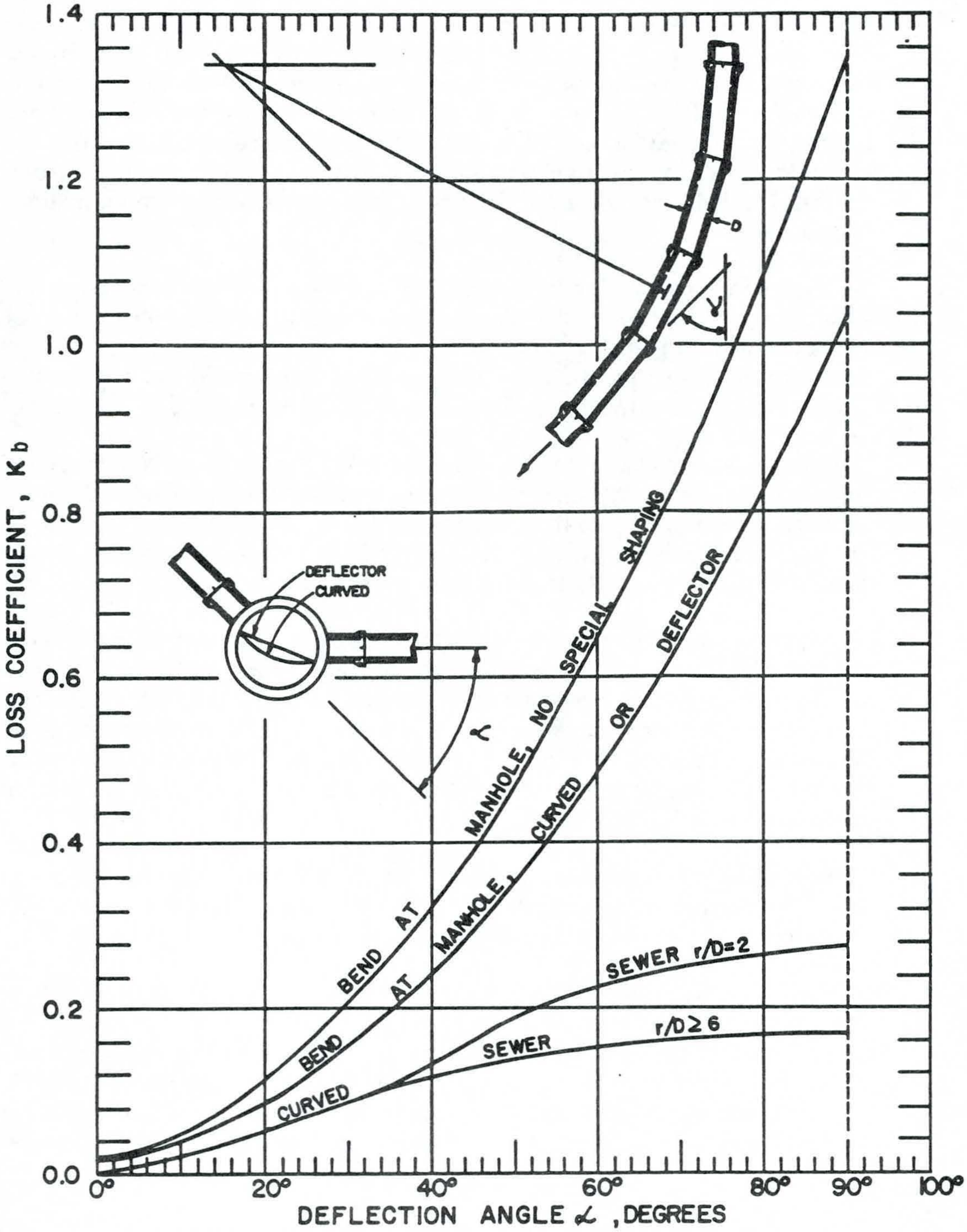


Figure 4.16
Sewer Bend Loss Coefficient
(University of Missouri 1958)

4.2.5 Design Example

The design example in this section analyzes a hypothetical pipe layout to demonstrate the method of application of the design figures and to provide an overview of the final hydraulic design procedure. Figure 4.17 illustrate how junction losses are computed for both pressure conduits and open channel flow. The design example was developed to illustrate at least one condition for each of the design figures detailed in Section 4.2.4. The design then proceeds upstream from junction to junction.

As shown in Figure 4.17, each inlet is numbered (e.g., I-4), and the design rate of flow into each is shown. The accumulated design rate of flow in each pipeline between inlets is given, together with the pipe diameter in inches and the length in feet from center to center of inlets. The pipe slope is not stated, but appears on the profiles at the end of the design discussion (Figure 4.33, page 126).

The manholes are also numbered (such as MH-1 for Manhole No. 1). The pipe arrangement at each manhole and inlet is evident from the plan, and serves to identify the design figure from Section 4.2.4 that will be used to determine the corresponding pressure change coefficients. These, in turn, are to be used to calculate, in feet, the pressure change for each upstream pipe.

The system is laid out during the preliminary design phase, with all inlets located, the rate of inflow to each determined* and the preliminary pipeline sizes selected, and a preliminary profile established. Proceeding from the outfall, the design moves to the next junction upstream by adding the friction loss in the pipeline to the hydraulic grade line at the outfall. The value obtained is the downstream hydraulic grade line for the junction, which needs to be checked to verify pressure conduit or open channel.

If the hydraulic grade line is less than 80 percent of depth in the downstream pipe and if the normal pipeline depth is less than 80 percent of the pipe vertical height, then the downstream water surface is set at normal depth.

A word of caution is needed to prevent the loss of significant design time: examine the conditions at each junction to try to determine whether the main line, the laterals, or a nearby inlet (usually with a high rate of inflow) is most likely to be more critical in regard to whether or not the preceding pipeline design may need to be revised. Keep in mind that the final hydraulic design procedure is iterative, and adjustments will probably be necessary to raise or lower the hydraulic grade lines for the design runoff event.

The design is carried upsteam from junction to junction with an explanation of the use of the applicable graph. Pipeline computations on each junction computation sheet are for the preceding or downstream pipeline. It is not recommended that junction computation sheets as elaborate as those included in this discussion be used. A simple hand sketch is usually sufficient.

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* **Note:** Due to the differing times of concentration, the rate of inflow for sizing of the storm drain pipeline may be different than the flow for sizing the inlets.

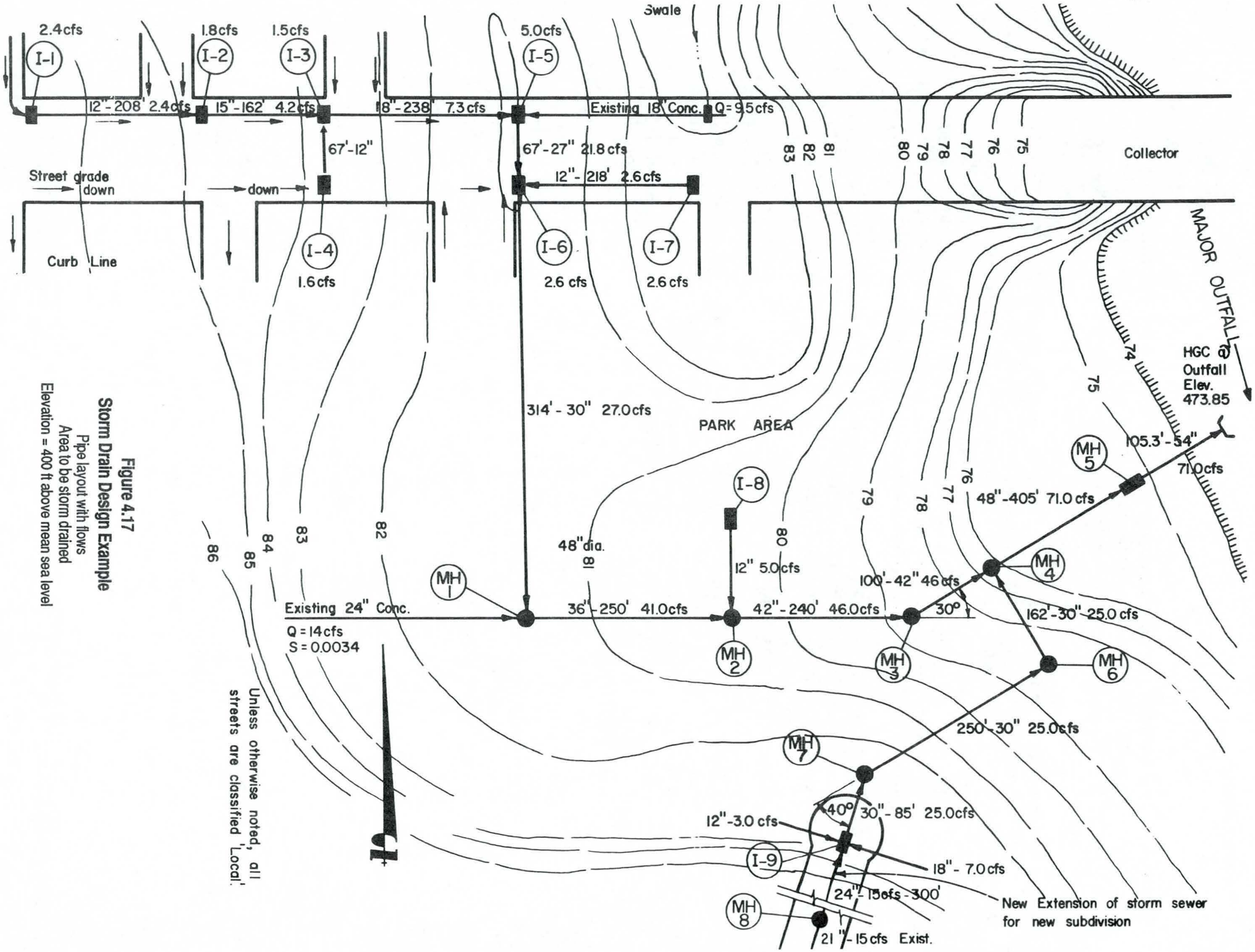


Figure 4.17
Storm Drain Design Example
Pipe layout with flows
Area to be storm drained
Elevation = 400 ft above mean sea level

In the design example, the accuracy of the computations is shown to 0.01 feet; however, in actual design, the needed accuracy is usually sufficient to 0.10 feet for hydraulic grade line computations. The pipeline inverts are to be designed to 0.01 feet.

For the design example the pipeline is assumed to be reinforced concrete pipe without rubber gasket joints; therefore, the roughness factor "n" was assumed to be 0.013. In the design example, it was assumed that the inverts of the manholes and inverts were known, in many instances, due to utility conflicts or due to the desire to control the hydraulic grade line; the depth of hydraulic structures may be varied during final hydraulic alignment. A profile for each inlet connector pipe must be prepared where conflicting utilities may exist to allow for optimum hydraulic design.

The mainstem of the design example is shown in the profile at the end of the design example (see Figure 4.33). The profile includes the pipeline crown and invert, manholes and inlets, energy grade line, and hydraulic grade line.

4.2.5.1 Main Line Calculations

Outlet to MH-5: Preliminary surveys have shown that the tailwater elevation at the outlet is 473.85 feet (see Figure 4.17). The top of the 54-inch pipe is at an elevation of 473.89 feet, and the outlet is essentially submerged. The outfall exit loss of one velocity head should be added to the flowline elevation to establish the starting elevation of 474.13 feet. The friction loss in the pipe from the Outlet to MH-5 is added to this elevation to establish the downstream pressure line at MH-5.

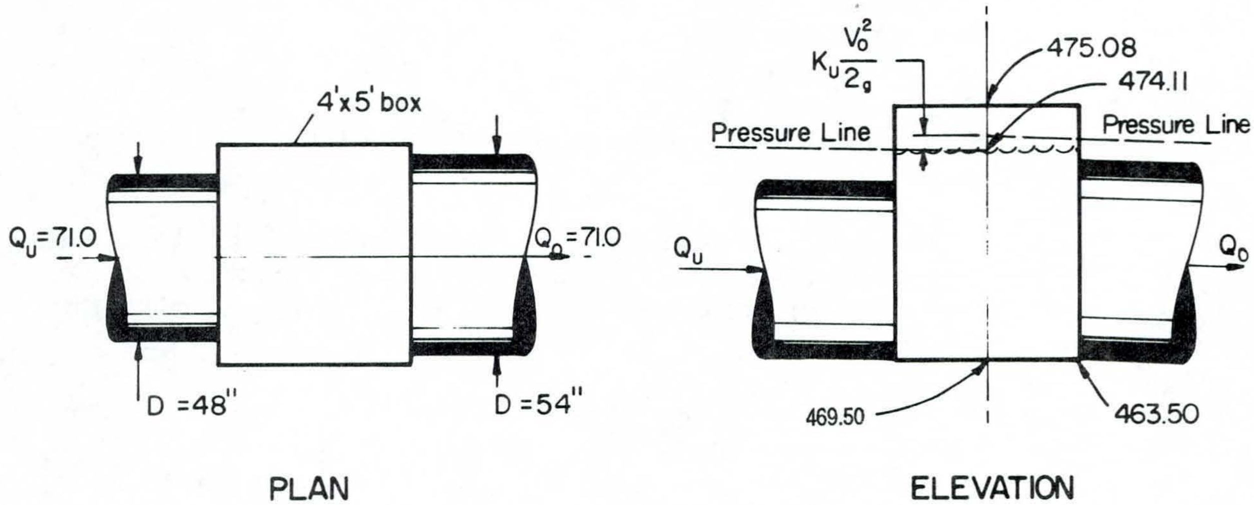
MH-5: Figure 4.18 shows the example storm drain design for MH-5 using the pressure change coefficients presented in Figure 4.8. The use of Figure 4.8 is restricted to cases where the pipe centerlines are parallel and not offset more than would permit the area of the smaller pipe to fall entirely within that of the larger one if projected across the junction box along the pipe axis. If grate flow enters the junction, use the coefficients presented in Figure 4.9.

Known quantities are:

» the gutter elevation,
» the inlet bottom elevation,
» the pipe flow rates and diameters,
» the inlet size, and
» the elevation of the downstream pressure line at the junction center.

From this information, calculate the velocity head of the outfall flow, the ratios D_U/D_O and A/D_U. Next, read K_U from Figure 4.8 and multiply it by the velocity head in the outfall to obtain h_U, the change of pressure. Subtract (or add) h_U to the outfall pressure line elevation to obtain the elevation of the upstream in-line pipe pressure line to which the water surface in the inlet corresponds. Check the clearance of the water below the gutter.

Figure 4.18
Storm Drain Design Example for Manhole No. 5
(Use Figure 4.8 for Calculations)



Item	MH-5	
Gutter Elevation	475.08	
Inlet Bottom Elevation	469.50	
Flow Rate Q_U	71.00	
D_U/D_O	0.89	
A/D_U	1.00	
Outfall Velocity Head $V^2/2g$	0.31	
Downstream Pressure Elevation	474.27	
Figure 4.13 square-edged entrance to outfall, K_U	–0.50	
Pressure Rise, $K_U \times V^2/2g$	–0.16	
Upstream Pressure Elevation	474.11	
Water Surface Elevation	474.11	
Distance Below Grate, ft	1.29	
Distance Above Invert, ft	4.61	Pressure Conduit
USHGL @ Outlet =	**474.13**	

Pipeline Data:

Downstream:		Upstream:
Q	= 71 cfs	Q = 71.0 cfs
Length	= 105.3 ft	D = 48 inches
D	= 54 inches	
s	= 0.0010 ft/ft	
V	= 4.46 fps	
$V^2/2g$	= 0.31 ft	
s_f	= 0.0013 ft/ft	
h_f	= 0.14 ft	

MH-4: Figure 4.19 depicts MH-4, which is typical of round manholes for which Figures 4.13 and 4.14 are used to determine pressure change coefficients.

Known quantities are:

» the elevations of the top and bottom of the manhole,
» the manhole diameter,
» the rates of flow in each pipe,
» the pipe diameters, and
» the elevation of the downstream (outfall pipe) pressure line at the branch point.

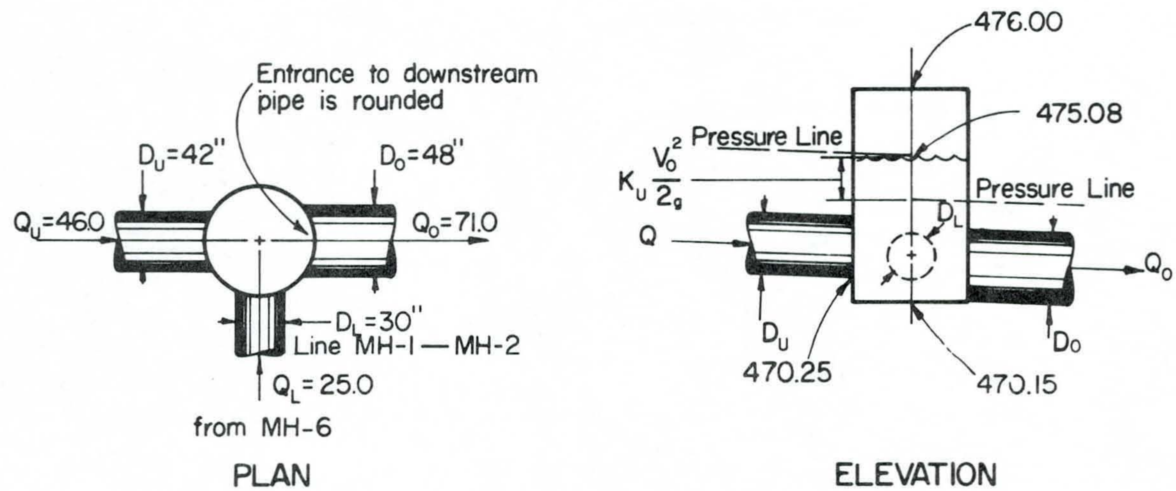
From this information, calculate the ratios D_L/D_O, D_U/D_O, Q_U/Q_O, and B/D_O, the figure factor $Q_U/Q_O \times D_O/D_U$ and the outfall velocity head. The values of D_L/D_O and $Q_U/Q_O \times D_O/D_U$ indicate that Figures 4.13 and 4.14 are applicable in this case.

Figure 4.13 (for square manholes) is used to obtain the pressure change coefficient K_L for the lateral pipe even though MH-4 is a round manhole. First, read K_L^* for a square manhole from the lower graph of Figure 4.13 and reduce it by 0.2 in accordance with the Table 4.3. The outfall pipe entrance is sharp-edged in this case, so no further reduction is made. The upper graph of the figure is used to obtain the multiplying factor M_L, then K_L is obtained by multiplying K_L^* by M_L. Next K_L is multiplied by the outfall velocity head to obtain h_L, the change in pressure (or pressure rise) at the manhole. Finally, h_L is added to the outfall pressure line elevation to obtain the elevation of the lateral pipe pressure line at the branch point.

Use Figure 4.14—for square or round manholes—to obtain the pressure change coefficients K_U for both the upstream in-line pipe and the water depth in the manhole. First, read K_U^* for all flow from the lateral from the lower graph of Figure 4.14 and use it without modification since, in this case, the outfall entrance is square-edged. Note that no reduction is to be made for the round manhole cross section. Next, read M_U from the upper graph of the figure and obtain K_U by multiplying K_U^* by M_U. Then multiply K_U by the velocity head in the outfall to obtain h_U, the change of pressure. Next, add h_U to the outfall pressure line elevation to obtain the elevation of the upstream in-line pipe pressure line at the branch point. The water surface elevation in the manhole is the same as the pressure line for the in-line pipe. Finally, check the clearance of the water surface below the top of the manhole to make certain it is ample.

Note that for a square-edged entrance to the outfall pipe, values of A/D_U less than 1 do not appreciably reduce the values of K_U shown for $A/D_U = 1$. For an enlargement of pipe size, as in this case, the pressure change across the junction is negative, even though there is a loss in total energy.

Figure 4.19
Storm Drain Design Example for Manhole No. 4
(Use Figures 4.13 and 4.14 for Calculations)



Item	MH-4
Top of Manhole Elevation	476.00
Bottom of Manhole Elevation	470.15
Lateral Flow Q_L cfs	25.00
Upstream In-Line Flow Q_U cfs	46.0
Outfall Flow Q_O cfs	71.0
Lateral Pipe Ratio D_L/D_O	0.63
In-Line Pipe Ratio D_U/D_O	0.88
Figure Factor $Q_U/Q_O \times D_O/D_U$	0.74
Manhole Diameter B inches	48
Manhole Size Ratio B/D_O	1.00
Outfall velocity head $V_O^2/2g$ ft	0.50
Downstream Pressure Elevation	475.08
Lateral Pressure Rise Coefficient (square-edged entrance)	
Figure 4.13 K_L* for square-edged manhole	0.93
K_L for round-edged manhole (less 0.2)	0.75
Figure 4.13 M_L	0.61
$K_L = K_L^* \times M_L$	0.57

* The use of rounded entrance from manhole to outlet pipe is usually not economically justified when $V_O^2/2g < 1.0$.

(continued)

Item	MH-4	
Lateral Pressure Rise, K_L x V_O 2/2g	0.28	
Lateral Upstream Pressure Elevation	475.36	
Upstream Pipe Pressure Rise Coefficient		
Figure 4.14 K_U* for square or round Manhole	1.86	
M_U	0.45	
$K_U = K_U^* \times M_U$	0.84	
In-Line Upstream Pressure Elevation	475.50	
Water Surface Elevation	475.50	
Clearance, Water Below Top ft	0.50	
Distance Above Invert, ft	5.35	Pressure Conduit
USHGL @ MH-5 =	**474.11**	

Pipeline Data:

Downstream:		Upstream:	Lateral:
Q	= 71.0 cfs	Q = 46.0 cfs	Q = 2 cfs
Length	= 405 ft		
D	= 48 inches		
s	= 0.0015 ft/ft		
V	= 5.65 fps		
$V^2/2g$	= 0.50		
s_f	= 0.0024 ft/ft		
h_f	= 0.97		

* This value will be used later in the computations to start the lateral pipe computations from MH-4 to MH-6.

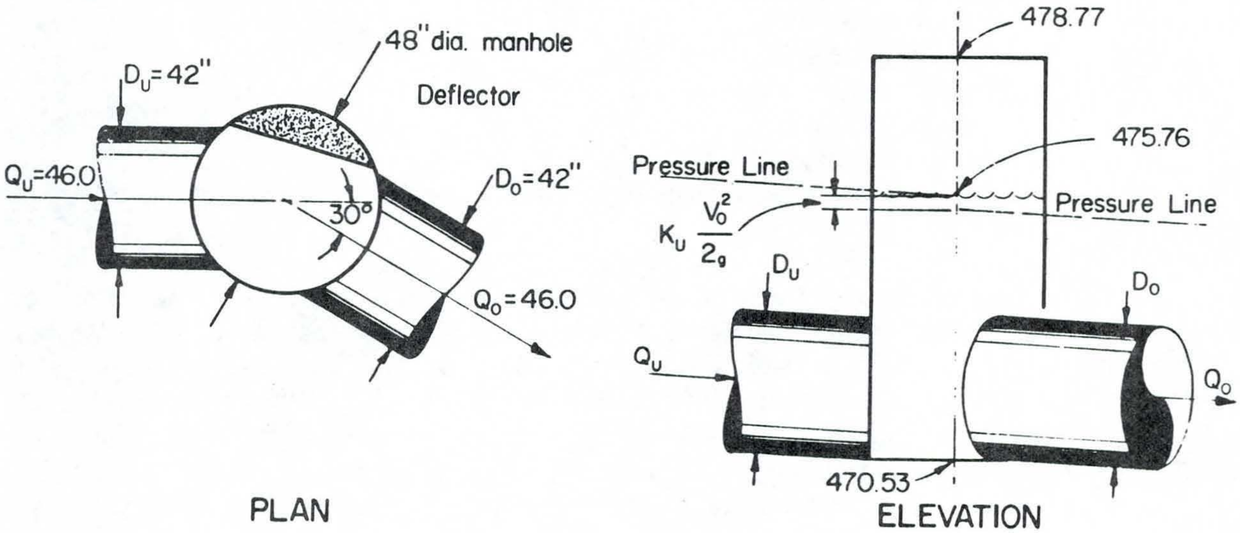
MH-3: MH-3 (Figure 4.20) is typical of junctions for which the pressure change coefficients in Figure 4.16 apply.

Known quantities are:

» the gutter elevations,
» the manhole bottom elevation,
» the flow rate,
» the pipe diameter,
» the deflection angle and characteristics, and
» the elevation of the downstream pressure line.

From this information, the velocity head of the outfall flow may be determined. Read the loss coefficient K from Figure 4.16 and multiply it by the outfall velocity head to obtain the rise of the water surface above the downstream pressure line elevation. This corresponds to the upstream pressure line elevation. Check the clearance of the water surface below the gutter.

Figure 4.20
Storm Drain Design Example for Manhole No. 3
(Use Figure 4.16 for Calculations)



Item	MH-3	
Gutter Elevation	478.77	
Manhole Bottom Elevation	470.53	
Upstream = Downstream Flow cfs	46.0	
Upstream Pipe Diameter, inches	42.0	
Downstream Pipe Diameter, inches	42.0	
Outfall Velocity Head $V_O^2/2g$ ft	0.36	
D_U/D_O	1.0	
(This implies that there is no contraction or expansion headloss)		
Deflection Angle	30°	
Downstream Pressure Elevation	475.71	
Figure 4.16, K (with Deflector)	0.15	
Upstream Pressure Rise = $K \times V_O^2/2g$	0.05	
Upstream Pressure Elevation, W.S.E.	475.76	
Clearance, Water Below Top, ft	3.01	
Distance Above Invert, ft	5.23	Pressure Conduit
USHGL @ MH-4 =	**475.50**	

Pipeline Data:

Downstream:		Upstream:
Q	= 46.0 cfs	Q = 46.0 cfs
Length	= 100.0 ft	D = 42 inches
D	= 42 inches	
s	= 0.0028 ft/ft	
V	= 4.78 fps	
$V^2/2g$	= 0.35 ft	
s_f	= 0.0021 ft/ft	
h_f	= 0.21	

MH-2: Figure 4.21 illustrates MH-2. Use Figure 4.15 to obtain the pressure change coefficients.

Known quantities are:
- » the gutter elevation,
- » the inlet bottom elevation,
- » the pipe inflow rates,
- » the outfall flow rate,
- » the pipe diameters, and
- » the elevation of the downstream (outfall pipe) pressure line at the inlet center.

From this information, calculate the velocity head of the outfall flow, the ratios D_L/D_O, D_U/D_O, and Q_U/Q_O. Figures 4.13 and 4.14 should not be used for $D_L/D_O < 0.6$ if other solutions are possible. Enter Figure 4.15 at the ratio D_U/D_O and read K_U (also equal K_L) at the curve or interpolated curve for Q_U/Q_O. When checking the applicability of Figures 4.13 and 4.14 D_O/D_U is greater than 1.00, K_U will be negative in sign. This provides a check on proper use of the figures. Neither manhole shape, nor size, nor relative size of the lateral pipe will modify the coefficients of Figure 4.15.

Next, multiply K_U by the outfall velocity head to obtain h_U, the change of pressure at the manhole. Finally, subtract (or add) h_U (or h_L) to the outfall pressure line elevation to obtain the elevation of the upstream in-line pipe pressure at the manhole center. The lateral pipe pressure line and water surface elevation will correspond to the upstream in-line pipe pressure line elevation. Check to make certain the water surface is above the pipe crowns to justify using these figures, and that it is sufficiently below the top of the manhole to indicate safety from overflow.

Deflectors in the manhole are not effective in the ranges covered by Figure 4.15, and, therefore, need not be used.

Inlet I-8 is similar to Inlets I-1 and I-7 and is not analyzed in this example.

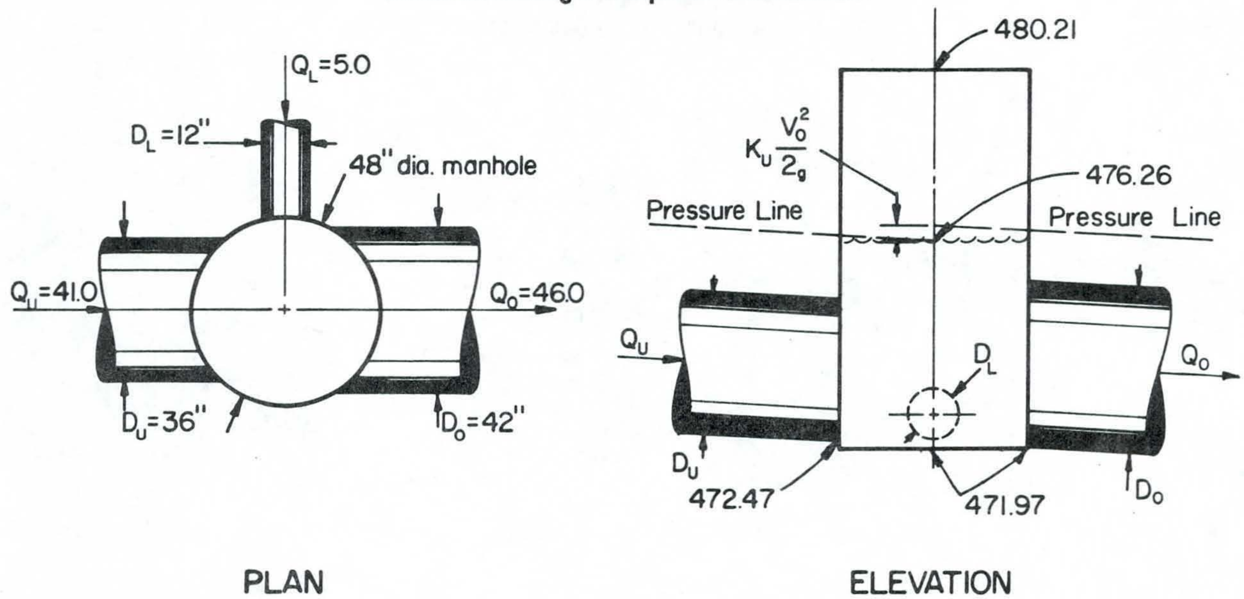
MH-1: MH-1 (Figure 4.22) has four laterals and has been included to illustrate the use of the design figures for a condition not specifically covered by them. The following parameters are needed to determine which figures to use (see Table 4.2):

$$Q_U/Q_O = 0.17 < 0.3, \qquad D_U/D_O = 0.50$$

The results indicate that Figures 4.11 and 4.12

Known quantities are:
- » the gutter elevation,
- » the elevation of the inlet bottom,
- » the lateral pipe and the grate inflow rates, and
- » their total—the outfall flow rate, the pipe diameters, and the elevation of the downstream (outfall pipe) pressure line.

Figure 4.21
Storm Drain Design Example for Manhole No. 2

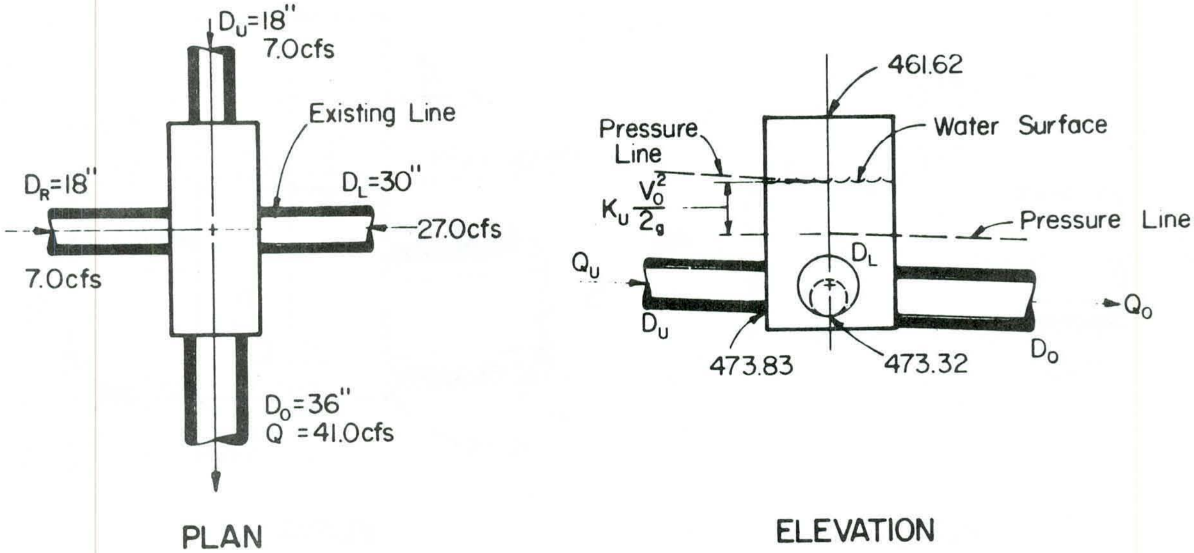


Item	MH-2
Top of Manhole Elevation	480.21
Bottom of Manhole Elevation	471.97
Lateral Flow Q_L cfs	5.00
Upstream In-Line Flow Q_U cfs	41.0
Outfall Flow Q_O cfs	46.0
Flow Ratio Q_U/Q_O	0 89 > 0.7
Lateral Pipe Ratio D_L/D_O	0 29 < 0.6
In-Line Pipe Ratio D_U/D_O,	0.86
Outfall Velocity head $V_O^2/2g$ ft,	0.35
Factor $Q_U/Q_O \times D_U/D_O$,	476.26
Downstream Pressure Elevation,	476.26
Assume Square-edged Entrance, Figure 4.15, Figure 4.16; K_U and K_L	-0.18
Upstream Pressure Rise - 0.18 × 0.35 ft	-0.06
Upstream Pressure Elevation and WSE	476.20
Clearance, Water Below Top, ft	3.85
Distance Above Invert ft	4.23
USHGL @ MH-3 =	**475.76 ft**

Pipeline Data:

Downstream:		Upstream:	Lateral:
Q	= 46.0 cfs	Q = 41.0 cfs	Q = 5.0 cfs
Length	= 240 ft	D_U = 36 inches	D_O = 12 inches
D	= 42 inches		
s	= 0.0060 ft/ft		
V	= 4.78 fps		
$V^2/2g$	= 0.35 ft		
s_f	= 0.0021 ft/ft		
h_f	= 0.50		

Figure 4.22
Storm Drain Design Example for Manhole No. 1
(Use Figure 4.11 for Calculations)



Item	MH-1	
Gutter Elevation	481.62	
Inlet Bottom Elevation	473.32	
Flow Ratios		
Q_G/Q_O	0.17	
Q_{HV}/Q_O	0.66	
Q_{LV}/Q_O	0.17	
Pipe Size Ratios		
D_{LV}/D_O	0.50	
D_{HV}/D_O	0.83	
D_{HV}/D_{LV}		
Velocity Head $V_O{}^2/2g$, ft	0.52	
Downstream Pressure Elevation	477.15	
Figure 4.11:		
Factor H for existing lateral	3.7	
Factor L for existing lateral	0.5	
K_L = for new lateral	1.8	
K_R = H-L, for lateral	3.2	
Pressure Rise existing lateral 3.2 × 0.52	1.66	
New 30-inch lateral 1.8 × 0.52	0.94	
Upstream Pressure Elevation		
Exist lateral & in-line	478.81	
New 30-inch	478.09	
Water Surface Elevation in Inlet	478.09	
Clearance, Gutter to Water, ft	3.53	
Distance Above Invert (ck 30-inch)	4.77	Pressure Conduit
USHGL @ MH-2 =	**476.2**	

(continued)

Pipeline Data for MH-1:

Downstream:		Left Lateral:	Right Lateral:
Q	= 41.0 cfs	Q = 27.0 cfs	Q = 7.0 cfs
Length	= 250 ft	D = 30 inches	D = 18 inches
D	= 36 inches	V = 5.5 fps	V = 3.96 fps
s	= 0.0060 ft/ft	High Velocity	Low Velocity
V	= 5.80 fps		
$V^2/2g$	= 0.52 ft		
s_f	= 0.0038 ft/ft		
h_f	= 0.95 ft		

From the lateral pipe flow rates and sizes, determine the velocity in each of the laterals and identify the two laterals as higher velocity and lower velocity. In this case, the existing line has the higher-velocity. From the known data and the above determination, calculate the ratios Q_G/Q_O, Q_{HV}/Q_O, Q_{LV}/Q_O, D_{HV}/D_O, D_{LV}/D_O, and D_{HV}/D_{LV}. Next, calculate the velocity head of the outfall. Then tabulate the elevation of the downstream pressure line for convenience in adding the pressure rise, which is now calculated using Figure 4.11. Read the pressure factors H and L from the figures, identified by the lateral to which the D and Q of the two graphs apply. The difference between H and L (3.7 - 0.5 = 3.2) is the pressure change coefficient $K_R = K_{LV}$ for existing lower velocity lateral, which is also to be applied to the upstream existing pipe in-line with the outlet pipe. The constant coefficient $K_L = K_{hv}$ is 1.8 because grate flow is involved. Each coefficient is multiplied by the velocity head of the outfall flow to obtain the pressure changes h_{LV} and h_{HV} for the laterals. The pressure change is always positive, that is, producing a rise in pressure upstream, for junctions of this type. Thus h_{LV}, the pressure rise, is added to the elevation of the outfall pipe (downstream) pressure line to obtain the elevation of the pressure line in the lower velocity lateral at the branch point. Similarly, h_{HV} is used to obtain the elevation of the pressure line of the higher-velocity flow in the existing line. The water surface elevation in the inlet corresponds to the latter pressure line. Finally, the clearance of the water surface below the gutter is checked.

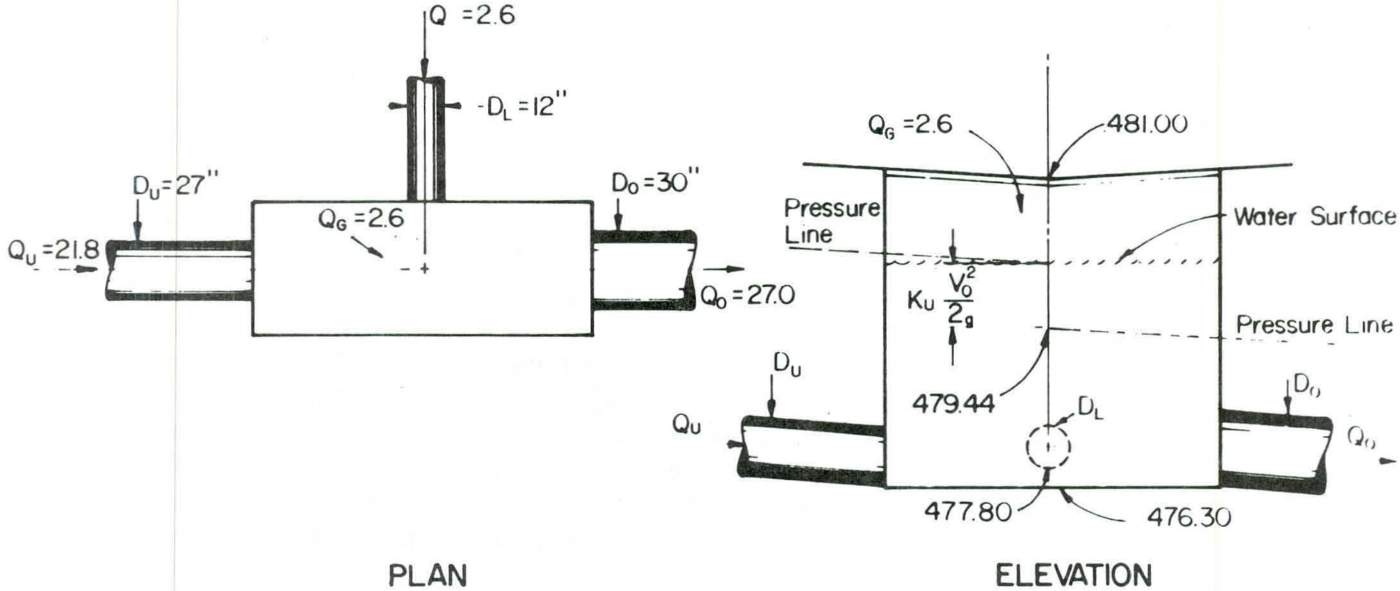
Inlet No. 6: I-6 (Figure 4.23) is an inlet to which Figure 4.10 applies. I-6 involves the basic through pipeline and lateral pipe arrangement, and also has flow through a top grate. It might appear that I-6 does not meet the requirements for Figure 4.10 because the in-line flow is through the length of the box rather than across its short dimension; however, this deviation from the more usual arrangement (see I-3) is not sufficient to effect a significant change in the hydraulic performance.

Inlet No. 5: I-5 (Figure 4.24) is a typical inlet to which the design methods of Figure 4.11 apply. The calculations of pressure changes at this inlet with in-line opposed laterals are presented in the left-hand column of the tabulation in Figure 4.24.

Known quantities are:

» the gutter elevation,
» the elevation of the inlet bottom,
» the lateral pipe and the grate inflow rates, and
» their total—the outfall flow rate, the pipe diameters, and the elevation of the downstream (outfall pipe) pressure line.

Figure 4.23
Storm Drain Design Example for Inlet No. 6
(Use Figure 4.10 for Calculations)



Item	Inlet 6
Gutter Elevation	481.00
Inlet Bottom Elevation	476.30
Grate Inflow, Q_G cfs	2.6
Upstream In-Line Flow, Q_U	21.8
Left Lateral Flow, Q_U	2.6
Outfall Flow, Q_O cfs	27.0
Outfall Pipe Diameter D_O, inches	30.0
Outfall Velocity Head $V_O^2/2g$, ft	0.47
Flow Ratios	
Q_U/Q_O	0.81
Q_G/Q_O	0.10
Pipe Size Ratio	
D_U/D_O	0.90
Downstream Pressure Elevation	479.44
Pressure Elevation Above Bottom, ft	3.14
Estimated d/D_O	1.4
Pressure Rise Coefficient for U.S. Main and Lateral	
Figure 4.10: $K_U = 0.45 + 0.10$	0.55
Pressure Rise 0.55 x $V_O^2/2g$, ft	0.26
Upstream Pressure Line Elevation for Main and Lateral and Water Surface Elevation	479.70
Check d/D_O	1.36
Clearance, Gutter to Water, ft	1.30
USHGL @ MH-1 =	**478.09**

(continued)

Pipeline Data for I-6:

Downstream:		Upstream:	Lateral:
Q	= 27.0 cfs	Q = 21.8 cfs	Q = 2.6 cfs
Length	= 314 ft	D = 27 inches	D = 12 inches
D	= 30 inches	V = 5.48 fps	V = 3.31 fps
s	= 0.0079 ft/ft		
V	= 5.5 fps		
$V^2/2g$	= 0.47 ft		
s_f	= 0.0043 ft/ft		
h_f	= 1.35 ft		

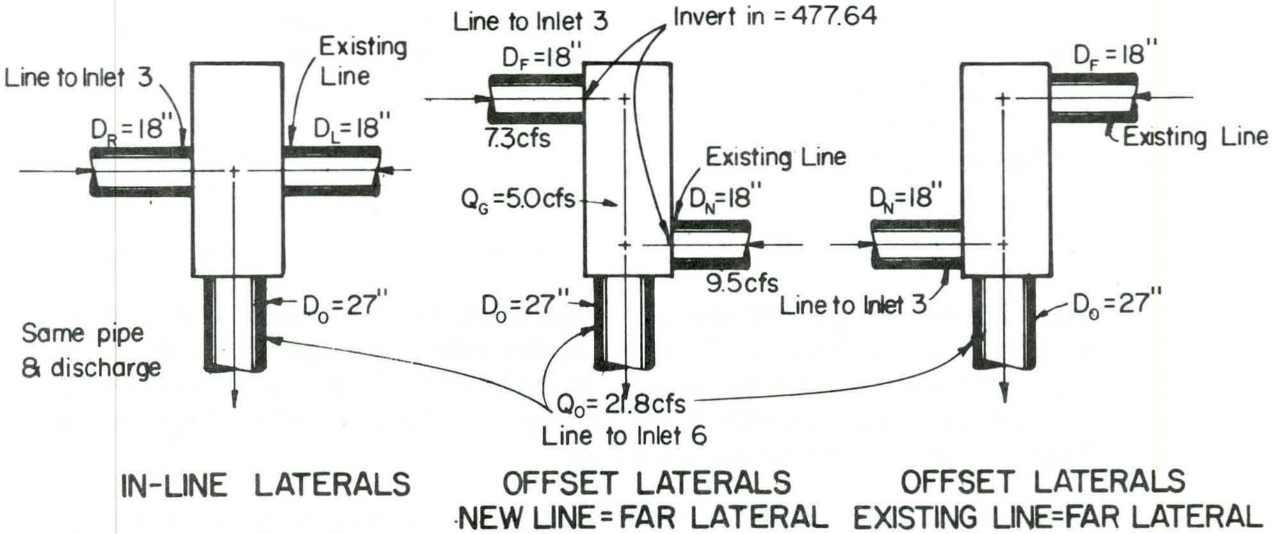
From the lateral pipe flow rates and sizes, determine the velocity in each of the laterals, and identify the two laterals as higher-velocity and lower-velocity. In this case, the existing line has the higher-velocity. From the known data and the above determination, the ratios Q_G/Q_O, Q_{HV}/Q_O, Q_{LV}/Q_O, D_{HV}/D_O, D_{LV}/D_O, and D_{HV}/D_{LV} (1.0 in this case) are calculated.

Next, calculate the velocity head of the outfall flow. Then, tabulate the elevation of the downstream pressure line for convenience in adding the pressure rise, which is now calculated using Figure 4.11. Read the pressure factors H and L from the figure, and identify, by the lateral, to which the D and Q of the two graphs apply. The difference between H and L (3.7 - 0.5 = 3.2) is the pressure change coefficient $K_R = K_{LV}$ for the new lateral to inlet 3, the lower-velocity lateral. The constant coefficient $K_L = K_{HV}$ is 1.8 because grate flow is involved. Each coefficient is multiplied by the velocity head of the outfall flow to obtain the pressure changes h_{LV} and h_{HV} for the laterals. The pressure change is always positive, that is, producing a rise in pressure upstream, for inlets of this type. Thus h_{LV}, the pressure rise, is added to the elevation of the outfall pipe (downstream) pressure line to obtain the elevation of the pressure line in the lower-velocity lateral at the branch point. Similarly, h_{HV} is used to obtain the elevation of the pressure line of the higher-velocity flow in the existing line. The water surface elevation in the inlet corresponds to the latter pressure line. Finally, check the clearance of the water surface below the gutter.

The alternate offset lateral arrangement, with the existing lines in the far position, is examined in the right-hand column of the tabulations. The position for use of Figure 4.12 is similar to that shown in the center column. In this case, the existing line is found to have the higher pressure line elevation. Although the pressure difference at I-5 is not large, it is significant in this case because the existing pipeline had the larger discharge rate and consequently the greater friction slope for its flow. Since the pressure line in this pipe is steeper in this example than in the new pipe to I-3, it is advisable to select the arrangement at the inlet which places the existing pipe in the near position; that is, the design shown by the center column in Figure 4.24.

It is worthwhile to note that the water surface elevation in I-5 is, for all intents and purposes, at the level of the gutter. This provides an "automatic valve" in the system, preventing extra runoff from entering the system and causing unforeseen problems at other locations in the pipe network.

Figure 4.24
Storm Drain Design Example for Inlet No. 5



Item	In-Line Laterals	New Line Far Lateral	Existing Line Far Lateral
Gutter Elevation	481.50	481.50	481.50
Inlet Bottom Elevation	476.50	476.50	476.50
Flow Ratios			
Q_G/Q_O	0.23	0.23	0.23
Q_F/Q_O		0.33	0.44
Q_N/Q_O		0.44	0.33
Q_{HV}/Q_O	0.44		
Q_{LV}/Q_O	0.33		
Pipe Size Ratios			
$D_{LV}/D_O = D_{HV}/D_O$	0.67		
$D_F/D_O = DN/D_O$		0.67	0.67
Factor $Q_F/Q_O \times D_O/D_F$		0.50	0.66
$Q_N/Q_O \times D_O/DN$		0.66	0.50
Velocity Head $V_O^2/2g$, ft		0.47	0.47
Downstream Pressure			
Elevation	480.04	480.04	480.04
Figure 4.11:			
Factor H for existing lateral	3.0		
Factor L for new lateral	0.6		
KR = H-L, new lateral	2.4		
K_L existing lateral	1.8		
Pressure Rise new lateral Inlet 3			
2.4 x 0.47	1.13		
Exist lateral 1.8 x 0.47	0.85		

(Continued)

Item	In-Line Laterals	3 - 5 New Lateral	Offset Existing Line Far Lateral	
Figure 4.12 K for 3-5		$K_F = 2.0$	$K_N = 1.4$	
K for existing		$K_N = 1.4$	$K_F = 1.9$	
Pressure Rise, new lateral 2.0 x 0.47		0.94		
Pressure Rise, existing 1.4 x 0.47	0.66			
Pressure Rise, new lateral 1.4 x 0.47			0.66	
Pressure Rise, existing, 1.9 x 0.47			0.89	
Upstream Pressure Elevation				
New Line to Inlet 3	481.17	480.98	480.70	
Existing lateral	480.89	480.70	480.93	
Water Surface Elevation in Inlet	480.89	480.98	480.93	
Clearance, gutter to water, ft	0.61	0.52	0.56	
Depth of Water in Inlet, ft	4.39	4.48	4.43	Pressure Conduit
USHGL @ Inlet I-6 =		**479.70**		
Pipeline Data for I-5:				
Downstream:	New Line to Inlet 3:	Existing Line:		
Q = 21.8 cfs	Q = 7.3 cfs	Q = 9.5 cfs		
Length = 67.0 ft	D = 18 inches	D = 18 inches		
D = 27 inches	V = 4.13 fps	V = 5.38 fps		
s = 0.0030 ft/ft				
V = 5.48 fps				
$V^2/2g$ = 0.47 ft				
s_f = 0.0050 ft/ft				
h_f = 0.34 ft				

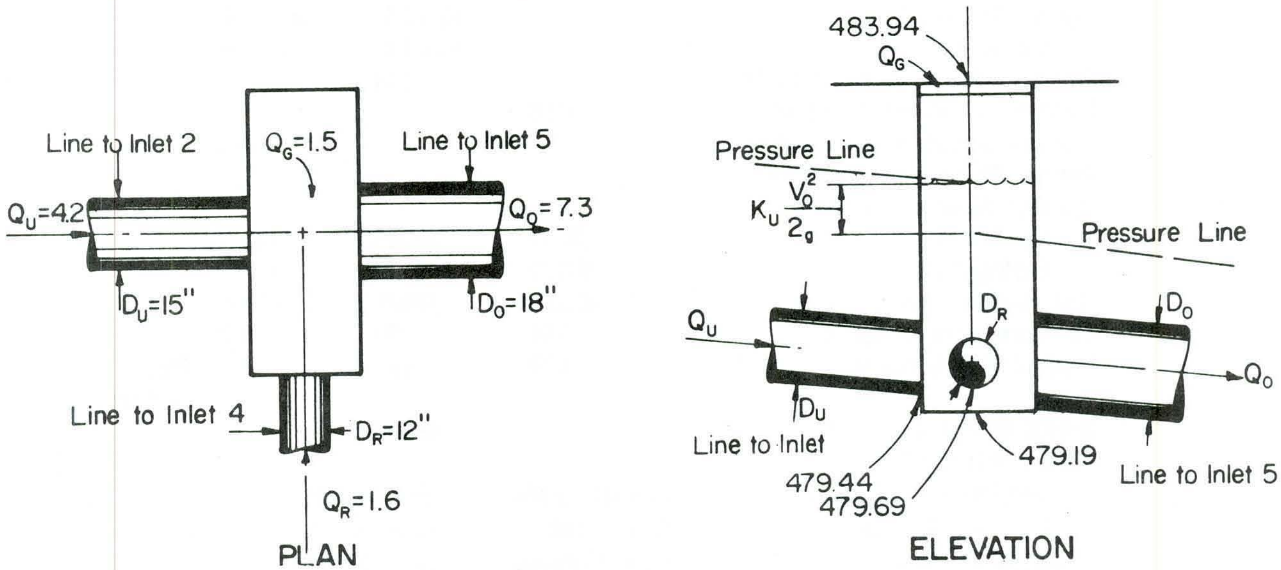
Inlet No. 3: Figure 4.25 illustrates I-3, using the pressure change coefficients from Figure 4.10.

Known quantities for I-3 are:

» the gutter elevation,
» the inlet bottom elevation,
» the pipe and grate inflow rates,
» the outfall flow rate,
» the pipe diameters, and
» the elevation of the downstream (outfall pipe) pressure line at the branch point.

From this information, calculate the velocity head of the outfall flow, the ratios D_U/D_O, Q_U/Q_O and Q_G/Q_O, and the distance from the downstream pressure line to the inlet bottom. Next estimate d/D_O including an allowance for h_U. Next obtain K_U from Figure 4.10 using a base value from the lower graph and adding an increment from the upper graph for $d/D_O = 2$. Multiply the total for K by the velocity head in the outfall to obtain h_U, the change of pressure. Add h_U to the outfall pressure line elevation to obtain the elevation of the upstream in-line pipe pressure line at the branch point. The pressure line of the lateral pipe and the water surface in the inlet will correspond to this upstream in-line pipe pressure elevation. Finally,

Figure 4.25
Storm Drain Design Example for Inlet No. 3
(Use Figure 4.10 for Calculations)



Item	I-3	
Gutter Elevation	483.94	
Inlet Bottom Elevation	479.19	
Grate Inflow, Q_G, cfs	1.5	
Upstream In-Line Flow, Q_R	4.2	
Right Lateral Flow, Q_R	1.6	
Outfall Flow, Q_O, cfs	7.3	
Outfall Pipe Diameter D_O, inches	18.0	
Outfall Velocity Head $V_O^2/2g$, ft	0.26	
Flow Ratios Q_U/Q_O	0.58	
Q_G/Q_O	0.21	
Pipe Size Ratio D_U/D_O	0.83	
Downstream Pressure Elevation	482.12	
Pressure Elevation Above Bottom	2.93	
Estimated d/D_O	2.1	
Pressure Rise Coefficient for U.S. Main and Lateral Figure 4.10 $K_U = 1.07 + 0.28$	1.35	
Pressure Rise $1.35 \times V_O^2/2g$, ft	0.35	
Upstream Pressure Line Elevation for Main and Lateral and Water Surface Elevation	482.47	
Check d/D_O	2.18	
Clearance, Gutter to Water, ft	1.47	
Depth of Water in Inlet, ft	3.28	Pressure Conduit
USHGL @ Inlet 5 =	**480.98**	

(continued)

Pipeline Data for I-3:

Downstream:		Upstream:	Lateral:
Q	= 7.3 cfs	Q = 4.2 cfs	Q = 1.6 cfs
Length	= 238 ft	D = 15 inches	D = 12 inches
D	= 18 inches	V = 3.42 fps	V = 2.04 fps
s	= 0.0065 ft/ft		
V	= 4.13 fps		
$V^2/2g$	= 0.26 ft		
s_f	= 0.0048 ft/ft		
h_f	= 1.14 ft		

recompute d/D_O to verify the initial estimate, and check the clearance of the water surface below the gutter.

An alternate arrangement of the lateral pipes at I-5 can be effected to produce an inlet with offset opposed laterals of the type to which Figure 4.12 will apply. Two different arrangements are possible, each placing one of the laterals in the far position. The pipe arrangement is shown in Figure 4.24, and the calculations of pressure changes are shown in the center and right-hand columns of the tabulation in the figure.

Known quantities are (with either placement of the laterals in the offset arrangement):

» the gutter and inlet bottom elevations,
» the flow rates,
» pipe diameters, and
» elevation of the downstream pressure line.

These quantities are all the same as for the in-line lateral arrangement. From this information and using the designation of the laterals as far and near in position, calculate the ratios Q_G/Q_O, Q_F/Q_O, Q_N/Q_Q, D_F/D_O, and D_N/D_O. Then multiply the flow ratio by the reciprocal of the pipe size ratio. Next, calculate the velocity head of the outfall flow and enter the downstream pressure elevation in the tabulations.

Figure 4.12 is used to determine the pressure change coefficients, working with each lateral arrangement separately to avoid confusion. Considering the lateral to I-3 as the far lateral (as shown in the center column of the tabulations), K_F for the new lateral is 2.0 and K_N for the existing line is 1.4. Each coefficient is multiplied by the outfall velocity head to obtain the pressure rises h_F and h_N for the corresponding laterals. Then each pressure rise is added to the elevation of the downstream (outfall pipe) pressure line to obtain the elevation of each lateral pipe pressure line at its branch point. The water surface elevation in the inlet will correspond to the far lateral pressure line (that is, the new lateral pipe in this case). Finally, the clearance of the water surface elevation below the gutter is checked. It will be noted that the pressure line of the existing line in the near position is at a lower elevation than that of the lateral to I-3 in the far position.

Inlet No. 2: I-2 (Figure 4.26) illustrates an inlet for which the pressure change coefficients in Figure 4.9 apply.

Known quantities are:

» the gutter elevation,
» the inlet bottom elevation,
» the pipe and grate inflow rates,
» the outfall flow rate,
» the pipe diameters, and
» the elevation of the downstream (outfall pipe) pressure line at the inlet center.

From this information, calculate the velocity head of the outfall flow, the ratios D_U/D_O and Q_U/Q_O, and the distance from the downstream pressure line to the inlet bottom. Next, estimate d/D_O including an allowance for h_U. Next, read K_U from Figure 4.9 (the lower graph in this case) and multiply it by the velocity head in the outfall to obtain h_U, the change of pressure. Then, add h_U to the outfall pressure line elevation to obtain the elevation of the upstream in-line pipe pressure line, to which the water surface in the inlet corresponds. Finally, recompute d/D_O to verify the initial estimate, and check the clearance of the water surface below the gutter.

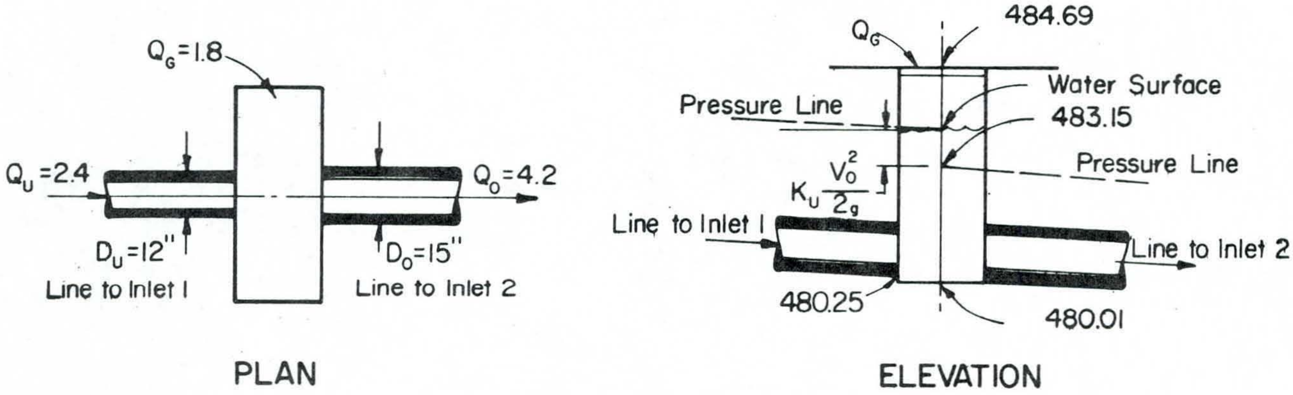
Inlet No. 1: I-1 (Figure 4.27) illustrates inlets to which Figure 4.7 applies for box-side outfall. The determination of the water surface elevation in the inlet proceeds in the same manner in either case.

Known quantities are:

» the gutter elevation,
» the inlet bottom elevation,
» the inflow rate,
» the outfall pipe diameter, and
» the elevation of the downstream (outfall pipe) pressure line.

From this information, calculate the velocity head of the outfall flow and the depth from the downstream pressure line to the inlet bottom. Then estimate d/D_O including an allowance for h_G. Then read K_G read from Figure 4.7 and multiply it by the velocity head in the outfall to obtain h_G, the rise of the water surface elevation above the pressure line. Finally, add h_G to the outfall pressure line elevation to obtain the water surface elevation; recompute d/D_O to determine the accuracy of the initial estimate; and check the clearance of the water surface below the gutter.

Figure 4.26
Storm Drain Design Example for Inlet No. 2
(Use Figure 4.78 for Calculations)

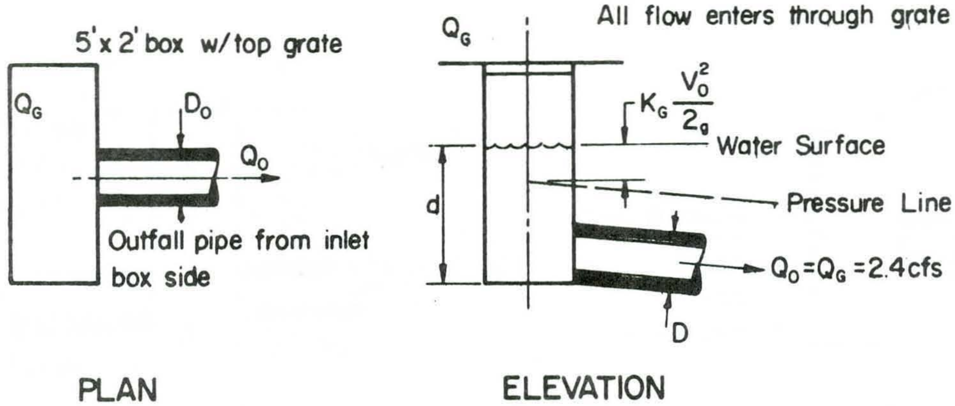


Item	I-2	
Gutter Elevation	484.69	
Inlet Bottom Elevation	480.01	
Q_G cfs	1.8	
Q_U cfs	2.4	
Q_O cfs	4.2	
D_O, inches	15.0	
Outfall Velocity Head $V_O^2/2g$, ft	0.18	
Downstream Pressure Elevation	483.15	
Pipe Size Ratio D_U/D_O	0.80	
Flow Ratios Q_U/Q_O	0.57	
Pressure Elevation Above Bottom, ft	3.14	
Estimated d/D_O	2.7	
Figure 4.9: $K_U = K_G$	1.40	
Pressure Rise $K_G \times V_O^2/2g$, ft	0.25	
Upstream Pressure Line and Water Surface Elevation	483.40	
Check d/D_O	2.72	
Clearance, Gutter to Water, ft	1.29	
Depth of Water in Inlet, ft	3.39	Pressure Conduit
USHGL @ Inlet 3 =	**482.47**	

Pipeline Data for I-2:

Downstream:		Upstream:
Q	= 4.2 cfs	Q = 2.4 cfs
Length	= 162 ft	D = 12 inches
D	= 15 inches	V = 3.06 fps
s	= 0.0035 ft/ft	
V	= 3.42 fps	
$V^2/2g$	= 0.18 ft	
s_f	= 0.0042 ft/ft	
h_f	= 0.68 ft	

Figure 4.27
Storm Drain Design Example for Inlet No. 1
(Use Figure 4.7 for Calculations)



Item	I-1	
Gutter Elevation	486.28	
Inlet Bottom Elevation	482.00	
$Q_G = Q_O$, cfs	2.4	
D_O, inches	12.0	
Outfall Velocity Head $V_O^2/2g$, ft	0.14	
Downstream Pressure Elevation	484.34	
Pressure Elevation Above Bottom, ft	2.34	
Estimated d/D_O	2.8	
Water Depth Over Pressure Line		
Figure 4.7: K_G	3.3	
Rise, $K_G \times V_O^2/2g$, ft	0.46	
Water Surface Elevation	484.80	
Check d/D_O	2.8	
Clearance, Gutter to Water, ft	1.48	
Depth of Water in Inlet, ft	2.80	Pressure Conduit
USHGL @ Inlet 2 =	**483.40**	
Pipeline Data for I-1:		
Q = 2.4 cfs		
Length = 208 ft		
D = 12 inches		
s = 0.0084 ft/ft		
V = 3.06 fps		
$V^2/2g$ = 0.18 ft		
s_f = 0.0045 ft/ft		
h_f = 0 .94 ft		

4.2.5.2 Lateral Calculations: Inlet No. 1 completes the main line computations. It is now necessary to compute the hydraulic grade line in the laterals. Some inlets which are repetitive in procedure have not been included in the example computations.

MH-6: MH-6 is a typical round manhole using data from Figure 4.13. Calculations for determining the pressure change coefficients for MH-6 are presented in Figure 4.28.

Known quantities are:

» the elevations of the top and bottom (flowline) of the manhole,
» the manhole diameter,
» the rate of flow,
» the two pipe diameters (equal in this case), and
» the elevation of the downstream (outfall pipe) pressure line at the branch point.

From this information, calculate the ratios D_L/D_O and B/D_O, and the velocity head of the outfall flow. Figure 4.13 (for square manholes) is used to obtain the pressure change coefficient K_L even though MH-6 is round (where $Q_L = Q_O$, $K_L = K_L^*$). First, read K_L for a square manhole from Figure 4.13 at $D_L/D_O = 1.00$ and $B/D_O = 1.33$, reducing this value by 0.1 for the round manhole in accordance with Table 4.3. The outfall pipe entrance is sharp-edged in this case, so no further reduction is made. Next, multiply K_L^* by the outfall velocity head to obtain h_L, the change of pressure (or pressure rise) at the manhole. Finally, add h_L to the outfall pressure line elevation to obtain the elevation of the lateral pipe pressure line at the branch point, or manhole center.

Use Figure 4.14 to obtain the water-surface elevation in the manhole. For MH-6, the coefficient K_U^* (an upstream in-line pipe with no flow, $Q_L/Q_O = 1.00$) is found to be 1.73 from Figure 4.13. This coefficient will define the depth of water above the downstream pressure line for no in-line flow, whether or not the in-line pipe is actually present. Then, multiply K_U^* by the outfall velocity head to obtain h_U, the depth of water over the outfall pressure line. Next, add h_U to the elevation of the outfall pressure line at the branch point to obtain the elevation of the water surface in the manhole. Finally, check the clearance of the water surface below the manhole top.

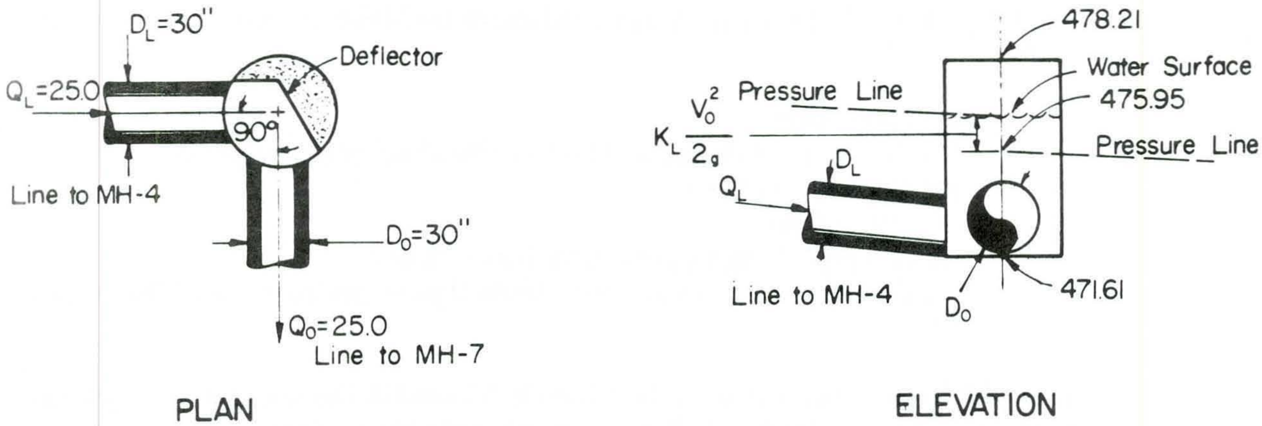
MH-7: Use Figure 4.16 to determine the pressure change coefficients for MH-7 (Figure 4.29).

Known quantities are:

» the gutter elevations,
» the manhole bottom elevation,
» the flow rate,
» the pipe diameter,
» the deflection angle and characteristics, and
» the elevation of the downstream pressure line.

From this information, determine the velocity head of the outfall flow. Read the loss coefficient K from Figure 4.16 and multiply it by the outfall velocity head to obtain

Figure 4.28
Storm Drain Design Example for Manhole No. 6
(Use Figure 4.16 for Calculations)

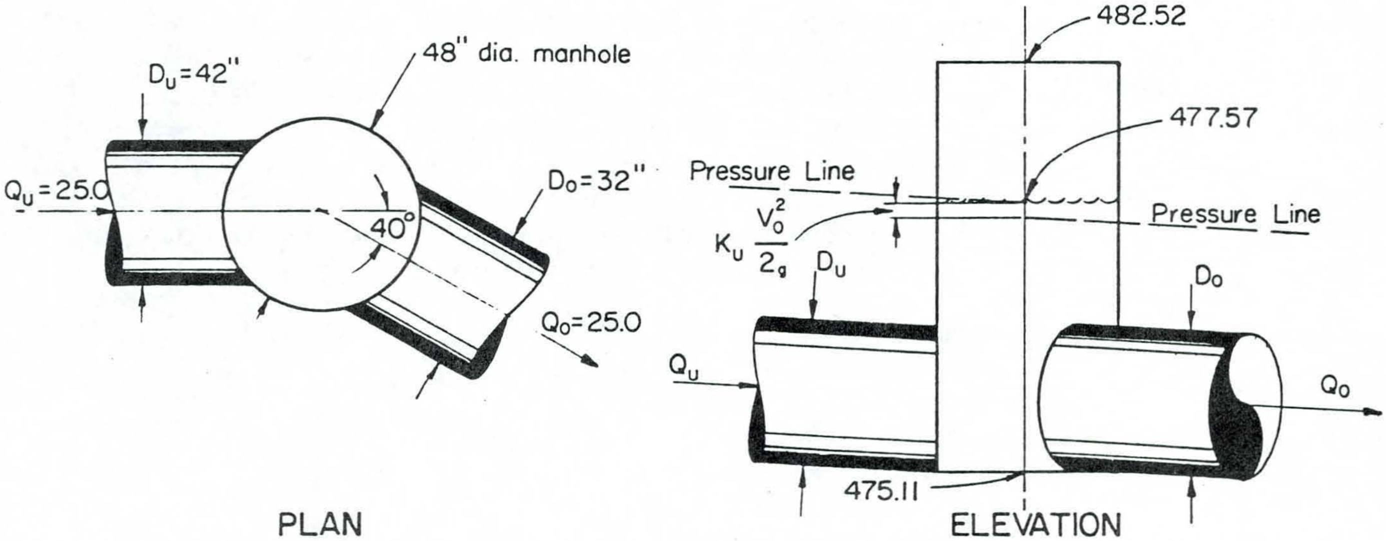


Item	**MH-6**	
Top of Manhole Elevation	478.21	
Bottom Manhole Elevation	471.61	
Lateral Flow Q_L, cfs	25.0	
Outfall Flow Q_O, cfs	25.0	
Outfall Pipe Diameter D_O, inches	30.0	
Pipe Size Ratio D_L/D_O,	1.00	
Manhole Diameter B, inches	48.0	
Manhole Size Ratio B/D_O	1.60	
Outfall Velocity Head $V_O^2/2g$, ft	0.40	
Downstream Pressure Elevation	475.96	
Pressure Rise Coefficient (square-edged entrance),		
Figure 4.13 K_L for square manhole	1.55	
K_L for round (less 0.1)	1.45	
Pressure Rise 1.4 x 0.40 ft	0.58	
Upstream Pressure Line Elevation	476.54	
Water Surface (Figure 4.14 K_U*)	1.73	
Water Depth Over Outfall Pressure - 1.73 x 0.40 ft	0.69	
Water Surface Elevation	476.65	
Clearance, Water Below Top, ft	1.56	
Depth of Water in Manhole, ft	5.04	Pressure Conduit
USHGL @ MH-4 =	**475.36**	

Pipeline Data for MH-6:

Downstream:			Upstream:
Q	=	25 cfs	Q = 25 cfs
Length	=	162 ft	D = 30 inches
D	=	30 inches	
s	=	0.0084 ft/ft	
V	=	5.09 fps	
$V^2/2g$	=	0.40 ft	
s_f	=	0.0037 ft/ft	
h_f	=	0.60 ft	

Figure 4.29
Storm Drain Design Example for Manhole No. 7
(Use Figure 4.16 for Calculations)



Item	MH-7	
Gutter Elevation	482.52	
Manhole Bottom Elevation	475.11	
Upstream = Downstream Flow, cfs	25.0	
Upstream Pipe Diameter, inches	30.0	
Downstream Pipe Diameter, inches	30.0	
Outfall Velocity Head $V_O^2/2g$, ft	0.40	
D_U/D_O	1.0	
(This implies that there is no contraction or expansion headloss)		
Deflection Angle	40°	
Downstream Pressure Elevation	477.47	
Figure 4.17, K	0.25	
Upstream Pressure Rise = K x $V_O^2/2g$	0.10	
Upstream Pressure Elevation, Water Surface Elevation	477.57	
Clearance, Water Below Top, ft	4.96	
Depth of Water in Manhole	2.46	Assuming flowing full
USHGL @ MH-6 =	**476.54**	

Pipeline Data for MH-7:

Q	=	25.0 cfs
Length	=	250.0 ft
D	=	30 inches
s	=	0.0140 ft/ft
V	=	5.09 fps
$V^2/2g$	=	0.40 ft
S_f	=	0.0037 ft/ft
h_f	=	0.93

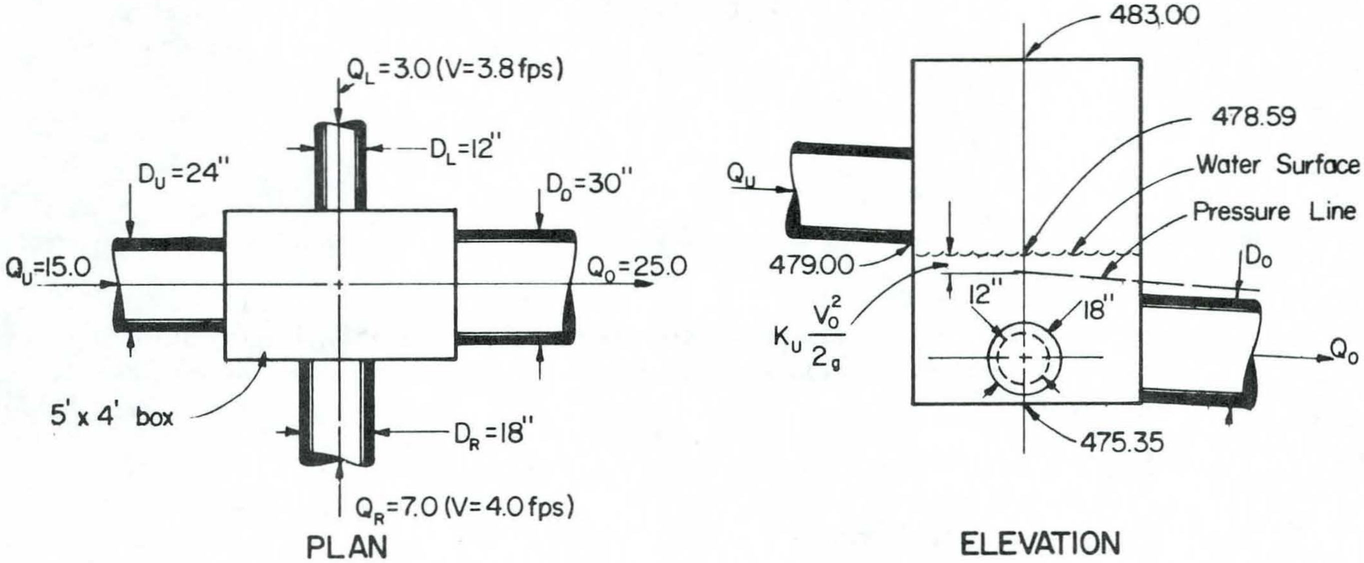
the rise of the water surface above the downstream pressure line elevation. This corresponds to the upstream pressure line elevation. Check the clearance of the water surface below the gutter.

Inlet No. 9: Figure 4.30 is an example of the type of inlet that uses the pressure change coefficients from Figure 4.11. The ground above I-9 rises very sharply and for the sake of economy, the upstream in-line pipe is raised 3.66 feet above the inlet bottom. As will be seen in the worked example, the water surface elevation in the inlet is below the invert of the upstream pipe. This implies that flow will be open-channel in the upstream pipe, at least in its lower section. The flow from the upstream pipe will be treated as grate flow to the inlet. Check upstream junctions to identify the type of flow at each junction and design accordingly.

Known quantities are:
» the gutter elevation,
» the elevation of the inlet bottom,
» the lateral pipe inflow rates,
» the outfall flow rates,
» the pipe diameters, and
» the elevation of the downstream pressure line.

From the lateral pipe flow rates and sizes, the velocity in each of the laterals is determined, and the two laterals are identified as higher-velocity and lower-velocity. In this case, the right lateral, looking downstream, is the higher velocity lateral. From the given data and the above determination, calculate the ratios Q_G/Q_O, Q_{HV}/Q_O, Q_{LV}/Q_O, D_{HV}/D_O, D_{LV}/D_O, and D_{HV}/D_{LV} (=1.50). Next, determine the velocity head in the outfall pipe. Read the pressure factors H and L from Figure 4.11, and identify, by the lateral, to which the D and Q of the two graphs apply. The difference between H and L (3.1 - 0.6 = 2.5) is the pressure change coefficient $K_L = K_{LV}$, the lower velocity lateral. The constant coefficient $K_R = K_{HV}$ is 1.8 because the upstream flow is treated as grate flow entering an inlet. Each coefficient is multiplied by the velocity head of the outfall flow to obtain the pressure changes, h_{LV} and h_{HV}, for the laterals. The pressure change is always positive and so produces a rise in pressure upstream. The pressure rise, h_{LV}, is used to obtain the pressure line elevation on the higher velocity lateral. The water surface elevation corresponds to the latter pressure line. Check the clearance of the water surface below the gutter.

Figure 4.30
Storm Drain Design Example for Inlet No. 9
(Use Figure 4.11 for Calculations)



Item	I - 9	
Gutter Elevation	483.00	
Inlet Bottom Elevation	475.35	
Flow Ratios		
Q_U/Q_O	0.60	
Q_{hv}/Q_O	0.28	
Q_{lv}/Q_O	0.12	
Pipe Size Ratio		
D_{lv}/D_O	0.40	
D_{hv}/D_O	0.60	
D_{hv}/D_{lv}	1.50	
Velocity Head $V_O^2/2g$ ft	0.40	
Downstream Pressure Elevation	477.88	
Figure 4.11		
Factor H	3.1	
Factor L	0.6	
K_L = H-L	2.5	
K_R =	1.8	(high velocity lateral)
Pressure Rise Left Lateral = 2.5 x 0.40 ft	1.0	
Pressure Rise Right Lateral = 1.8 x 0.40 ft	0.72	
Upstream Pressure Elevation Left Lateral	478.88	
Upstream Pressure Elevation Right Lateral	478.60	
Water Surface Elevation	478.60	
Clearance, Gutter to Water, ft	4.41	
Depth of Water in Inlet, ft	3.24	Pressure Conduit Downstream & Laterals
USHGL @ MH - 7 =	**477.57**	

(continued)

Pipeline Data for I-9:

Downstream:	Upstream:	L. Lateral:	R. Lateral:
Q = 25.0 cfs	Q = 15.0 cfs	Q = 3.0 cfs	Q = 7.0 cfs
Length = 85 ft	D = 24 inches	D = 12 inches	D = 18 inches
D = 30 inches		V = 3.82 fps	V = 3.96 fps
s = 0.0028 ft/ft			
V = 5.09 fps			
$V^2/2g$ = 0.40 ft			
s_f = 0.0037 ft/ft			
h_f = 0.0031 ft			

MH-8: Although no figure is provided for MH-8, this discussion has been included to illustrate junction energy loss computations with open channel flow. The following data apply:

USHGL @ Inlet I-9 must be computed:

Q = 15.0 cfs
D = 24 inches Downstream
D = 21 inches Upstream
Length to MH-8 = 300 feet
s = 0.012 ^{1}/ft Downstream
s = 0.014 ^{1}/ft Upstream
No Deflection in Manhole

Computations:

Downstream Pipe:
Q = 25.0 cfs
Q/Q_f = 0.60
d/D = 0.63 (Figure 4.9)
d = 1.26 ft = Normal Depth
V_f = 7.97 fps
V/V_f = 0.90 (Figure 4.6)
V = 7.16 fps
$V^2/2g$ = 0.80 ft
Downstream Invert Elevation = 482.60
Downstream Energy Grade Line Elevation = 484.66

Upstream Pipe:
Q_f = 19 cfs
Q/Q_f = 0.79
d/D = 0.75 (Figure 4.6)
d = 1.31 ft = Normal Depth
V_f = 6.24 fps
V/V_f = 0.98 (Figure 4.6)
V = 6.11 fps
$V^2/2g$ = 0.58 ft

Head Losses for Expansion = 0.2 (Δ hv) = 0.2 (0.80 - 0.58) = 0.04 ft

Downstream Energy Grade Line Elevation	484.66
+ Loss for Expansion	+ 0.04
Upstream Energy Grade Line Elevation	484.70
– Upstream Velocity Head	0.58
– Upstream Depth of Flow	1.31
Upstream Invert Elevation	482.81

Inlets 4 and 7: I-7 (Figure 4.31) uses Figure 4.7 for box-side outfall. I-4 (Figure 4.32) also uses Figure 4.7, but for box-end outfall. The determination of the water surface elevation in the inlet proceeds in the same manner in either case.

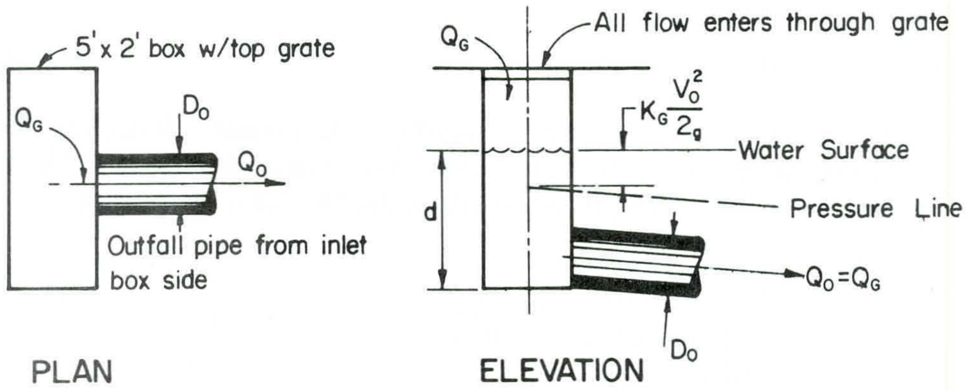
Known quantities are:

» the gutter elevation,
» the inlet bottom elevation,
» the inflow rate,
» the outfall pipe diameter, and
» the elevation of the downstream (outfall pipe) pressure line.

From this information, calculate the velocity head of the outfall flow and the depth from the downstream pressure line to the inlet bottom. Then estimate d/D_O including an allowance of h_G. Then read K_G from Figure 4.7 and multiply it by the velocity head in the outfall to obtain h_G, the rise of the water surface elevation above the pressure line. Finally, add h_G to the outfall pressure line elevation to obtain the water surface elevation; recompute d/D_O to verify the initial estimate; and check the clearance of the water surface below the gutter.

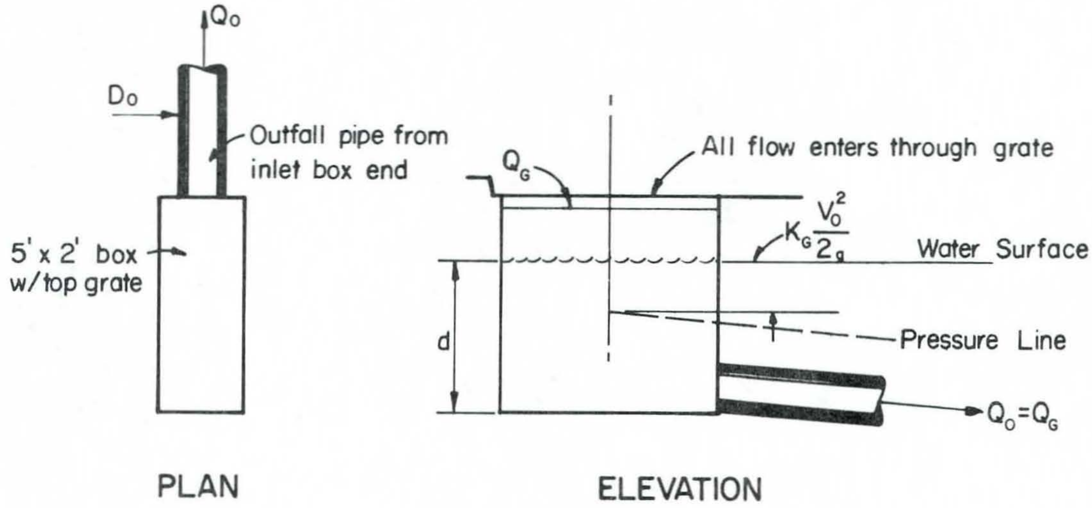
Figure 4.33 provides a profile for the design example discussed in this section.

Figure 4.31
Storm Drain Design Example for Inlet No. 7
(Use Figure 4.7 for Calculations)



Item	I - 7
Gutter Elevation	483.77
Inlet Bottom Elevation	479.00
$Q_G = Q_O$, cfs	2.6
D_O	12.0
Outfall Velocity Head $V_O^2/2g$ ft	0.17
Downstream Pressure Elevation	480.86
Pressure Elevation Above Bottom, ft	1.86
Estimated d/D_O	2.5
Water Depth Over Pressure Line	
Figure 4.7, K_G	3.7
Rise, $K_G \times V_O^2/2g$	0.63
Water Surface Elevation	481.49
Check d/D_O	2.49
Clearance, Gutter to Water, ft	2.28
Depth of Water in Inlet, ft	2.49
USHGL @ Inlet 6 =	**479.70**
Pipeline Data for I-7:	
Q = 2.6 cfs	
Length = 218 ft	
D = 12 inches	
s = 0.0055 ft/ft	
V = 3.31 fps	
$V^2/2g$ = 0.17 ft	
s_f = 0.0053 ft/ft	
h_f = 1.16	

Figure 4.32
Storm Drain Design Example for Inlet No. 4
(Use Figure 4.7 for Calculations)



Item	I - 4
Gutter Elevation	483.94
Inlet Bottom Elevation	480.50
$Q_G = Q_O$, cfs	1.6
D_O inches	12.0
Outfall Velocity Head $V_O^2/2g$ ft	0.06
Downstream Pressure Elevation, ft	482.60
Pressure Elevation Above Bottom ft	2.10
Estimated d/D_O	2.5
Water Depth Over Pressure Line	
Figure 4.7 K_G	5.0
Rise, $K_G \times V_O^2/2g$, ft	0.30
Water Surface Elevation	482.90
Check d/D_O	22.40
Clearance, Gutter to Water, ft	1.04
Depth of Water in Inlet, ft	2.40
USHGL @ Inlet 3 =	**482.47**

Pipeline Data for I-4:

Q	=	1.6 cfs
Length	=	67.0 ft
D	=	12 inches
s	=	0.0121 ft/ft
V	=	2.04 fps
$V^2/2g$	=	0.06 ft
s_f	=	0.0020 ft/ft
h_f	=	0.13

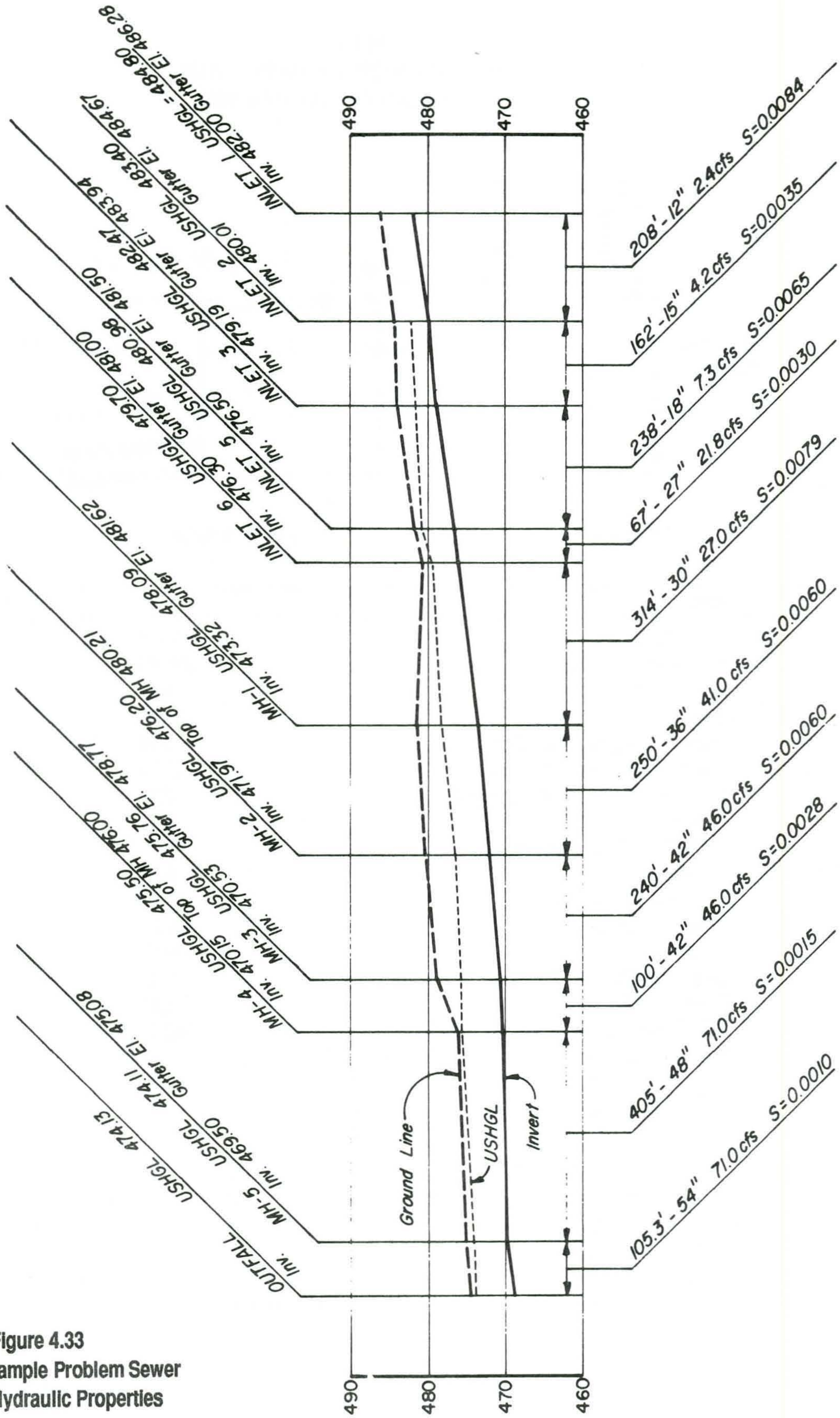


Figure 4.33
Profile of Example Problem Sewer Showing Hydraulic Properties

4.3 Culverts

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*. Culvert designers use this reference liberally as it is the result of years of research and experience in culvert design and at this time (1988) represents the state of the art.

4.3.1 Interaction of Culverts with Other Systems:

Culverts are primarily used for conveying runoff through a roadway embankment. They are normally aligned with a natural wash or drainage channel, which are often outfalls for storm drainage systems. Culverts are typically associated with drains on such a scale where bridges are not feasible. Regional drains are generally of a magnitude that justifies the use of bridges.

4.3.2 Culvert Criteria

4.3.2.1 Sizing

Crossroad culverts should be sized for the 50-year peak discharge. Other design frequencies shall be used only with the approval of the governing municipality.

4.3.2.2 Minimum Velocity: A minimum velocity of 2.5 feet per second at design capacity is recommended to assure a self-cleaning condition during partial depth flow.

4.3.2.3 Maximum Velocity: As a practical limit, outlet velocities should be kept below 15 feet per second unless special conditions exist. The maximum velocity should be consistent with channel stability requirements at the culvert outlet. As outlet velocities increase, the need for channel stabilization at the culvert outlet increases. If culvert outlet velocities exceed permissible velocities for the outlet channel lining material, suitable outlet protection must be provided. Outlet velocities may exceed permissible downstream channel velocities by up to 10 percent without providing outlet protection if the culvert tailwater depth is greater than the culvert critical depth of flow under design flow conditions. Tables 5.1 and 5.2 (pages 212 and 213) outline the permissible velocities for several channel lining materials.

4.3.2.4 Materials: The selection of a culvert material may depend upon structural strength, hydraulic roughness, durability, and corrosion and abrasion resistance. The three most common culvert materials are concrete (reinforced and nonreinforced), corrugated aluminum, and corrugated steel. Culverts may also be lined with other materials to inhibit corrosion and abrasion. Linings are not recommended to reduce hydraulic resistance because culvert linings have a short lifespan and are seldom reapplied as part of normal culvert maintenance. When linings are applied, the culvert sizing should neglect the reduced roughness from the lining material.

4.3.2.5 Minimum Cover: Minimum cover of fill over culverts must be provided to maintain the structural integrity of the pipe under anticipated loading conditions. Culvert manufacturers provide minimum cover requirements for prefabricated pipe. A rule of thumb for estimating minimum cover requirements is to provide one-eighth of the barrel diameter or span, with a minimum of 1 foot. The top of culverts should not extend into the roadway subgrade. Minimum cover should be measured from the top of subgrade.

4.3.2.6 Depth For Road Crossing

Culverts for collector and arterial streets are to be designed to convey the 50-year peak discharge with no flow crossing over the roadway. Additionally, the flow depth over the roadway shall be limited to 0.5 feet for the 100-year peak discharge.

Regardless of the size of the culvert, street crossings are to be designed to convey the 100-year storm runoff under and/or over the road to an area downstream of the crossing to which the flow would have gone in the absence of the street crossing. Flows up to or including 100-year frequencies should not cause increased flooding of farm land, developable lands, or buildings, unless a drainage easement is acquired for those areas. The ponded headwater elevation should be delineated on a contour map or using other surveying methods, as required.

In general, dip sections are not allowed, however, for flows crossing broad shallow washes where the construction of a culvert is not practical or desirable, the road may be dipped to allow the entire flow to cross the road. Use of dip sections must be approved by the governing municipality. The pavement through the dip section should have a one way slope parallel to flow and curbing and medians must not be raised. Upstream and downstream cutoff walls and aprons should be provided to minimize headcutting and erosion.

4.3.2.7 Special Design Considerations

Scour and Sedimentation: Possible aggradation or degradation at culvert crossings must be examined in the design of culverts. An adequate system of culvert design involves passing drainage water and sediment from the upstream regime condition of the channel crossing without upsetting the delicate balance between hydraulics and sediment transport flow.

The effects of scour and deposition should be minimized by protecting the outlet from scour with suitable outlet protection measures and reducing sedimentation problems by avoiding inlets depressed below the natural channel flowline and multi-barrel installations that reduce the channel velocity for low flows. Culverts which are located on and aligned with the natural channel are less susceptible to sedimentation problems.

Skewed Channels: A good culvert design is one that limits the hydraulic and environmental stress placed on the existing natural water course. This stress can be minimized by designing a culvert which closely conforms to the natural stream in alignment, grade, and width.

Often the culvert barrel must be skewed with respect to the roadway centerline to accomplish these goals. Alterations to the normal inlet alignment are also quite common.

The alignment of a culvert barrel with respect to a line normal to the roadway centerline is referred to as the barrel skew angle. A culvert aligned normal to the roadway centerline has a zero barrel skew angle. Directions (right or left) must accompany the barrel skew angle (Figure 4.34).

The barrel skew is established from the stream location and the proposed or existing roadway plan. The advantages of using a natural stream alignment include a reduction of entrance losses, equal depths of scour at the footings, less sedimentation in multibarrel culverts, and less excavation. The disadvantage of this design procedure is that the inlet may be skewed with respect to the culvert barrel and the culvert will be longer, sometimes resulting in increased initial costs.

The angle from the culvert face to a line normal to the culvert barrel is referred to as the inlet skew angle (Figure 4.35). The structural integrity of circular sections is compromised when the inlet is skewed due to loss of a portion of the full circular section where a portion of 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.

Culverts which have a barrel skew angle often have an inlet skew angle as well. This is because headwalls are generally constructed parallel to a roadway centerline to avoid warping of the embankment fill.

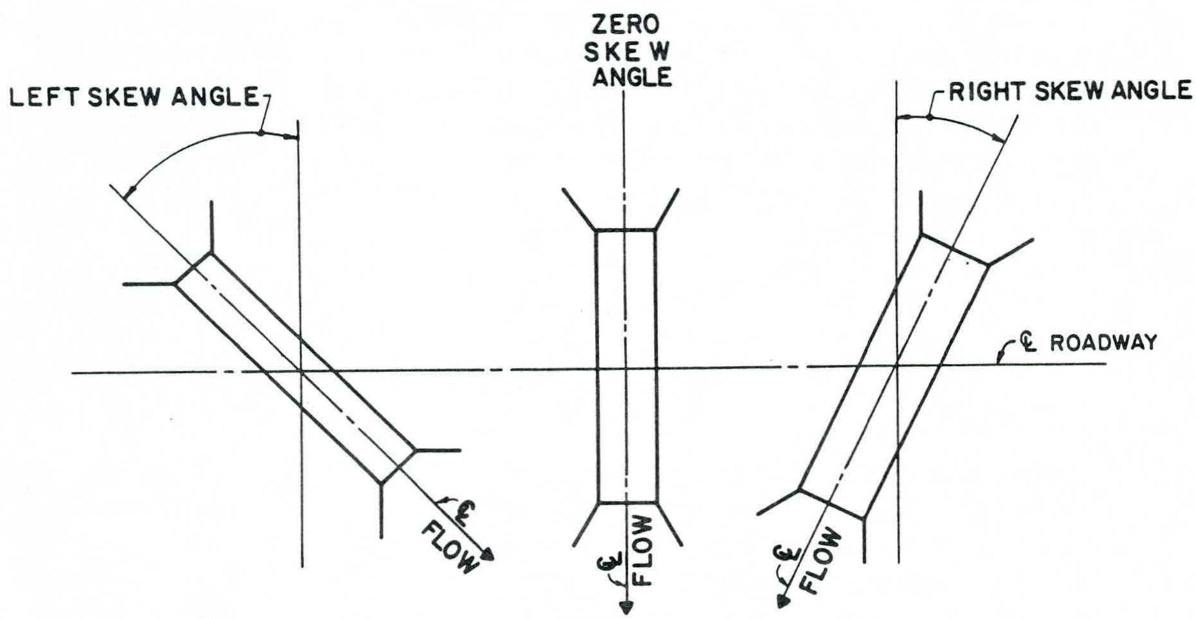


Figure 4.34
Barrel Skew Angle

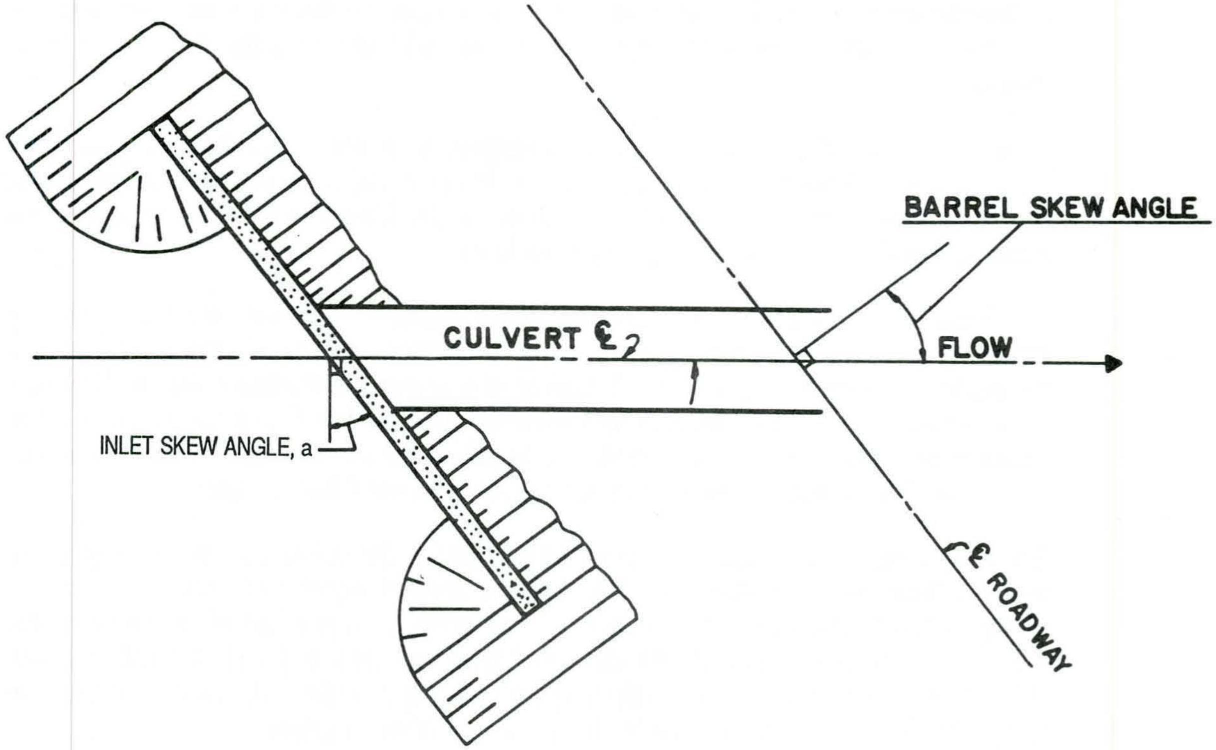


Figure 4.35
Inlet Skew Angle

In cases where the culvert barrel cannot be aligned with the channel flowline, such as when runoff is directed parallel to the roadway embankment to a suitable crossing location, the flow enters the culvert barrel at an angle. The approach angle should be limited to a maximum of 90 degrees. 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:

$$H_t = (V_a{}^2/2g)\sin a \tag{4.5}$$

Typical headwall/wingwall configurations for skewed channels are shown in Figure 4.36.

Bends: A straight culvert alignment is desirable to avoid clogging, increased construction costs, and reduced hydraulic efficiency. However, site conditions may dictate a change of alignment, either in plan or in profile. When considering a nonlinear culvert alignment, particular attention should be given to erosion, sedimentation, and debris control. Vertical bends are permitted when they transition from a flatter to a steeper slope, but should not transition from steeper to flatter slopes because of the potential for sediment deposition in the flatter reach.

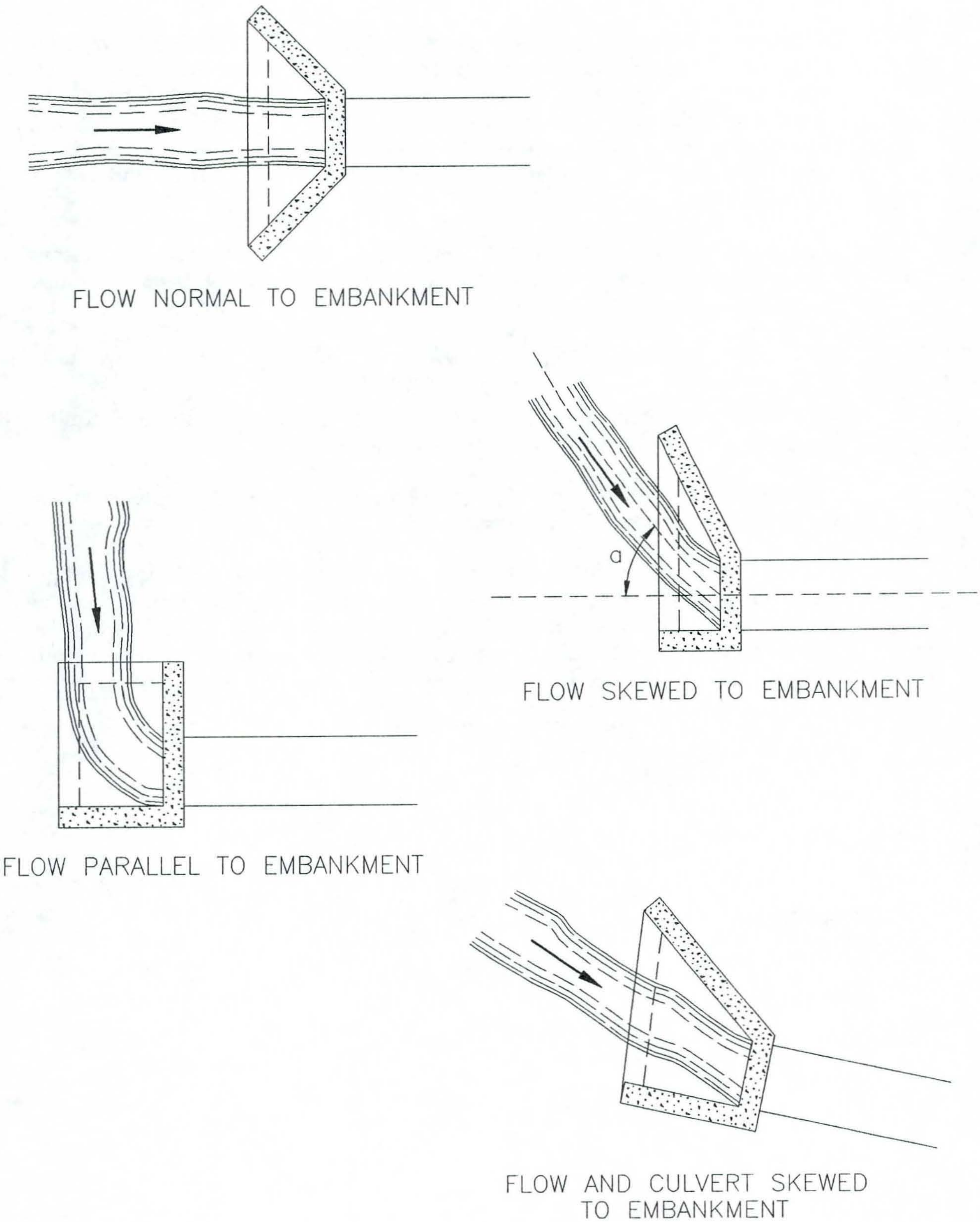


Figure 4.36
Typical Headwall/Wingwall Configurations

In designing a nonlinear culvert, the energy losses due to the bends must be considered. If the culvert operates in inlet control, no increase in headwater occurs unless the bend losses cause the culvert to flow under outlet control. If the culvert operates in outlet control, an increase in energy losses and headwater will result due to the bend losses. To minimize these losses, the culvert should be curved or have bends not exceeding 15 degrees at intervals of not less than 50 feet. Under these conditions, bend losses can be ignored.

If these conditions 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:

$$H_b = K_b V^2/2g \tag{4.6}$$

H_b is added to the other outlet losses. Figure 4.16 (page 95) can be used to determine loss coefficients (K_b) for bend losses in conduits flowing full.

The broken back culvert, shown in Figure 4.37, 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,

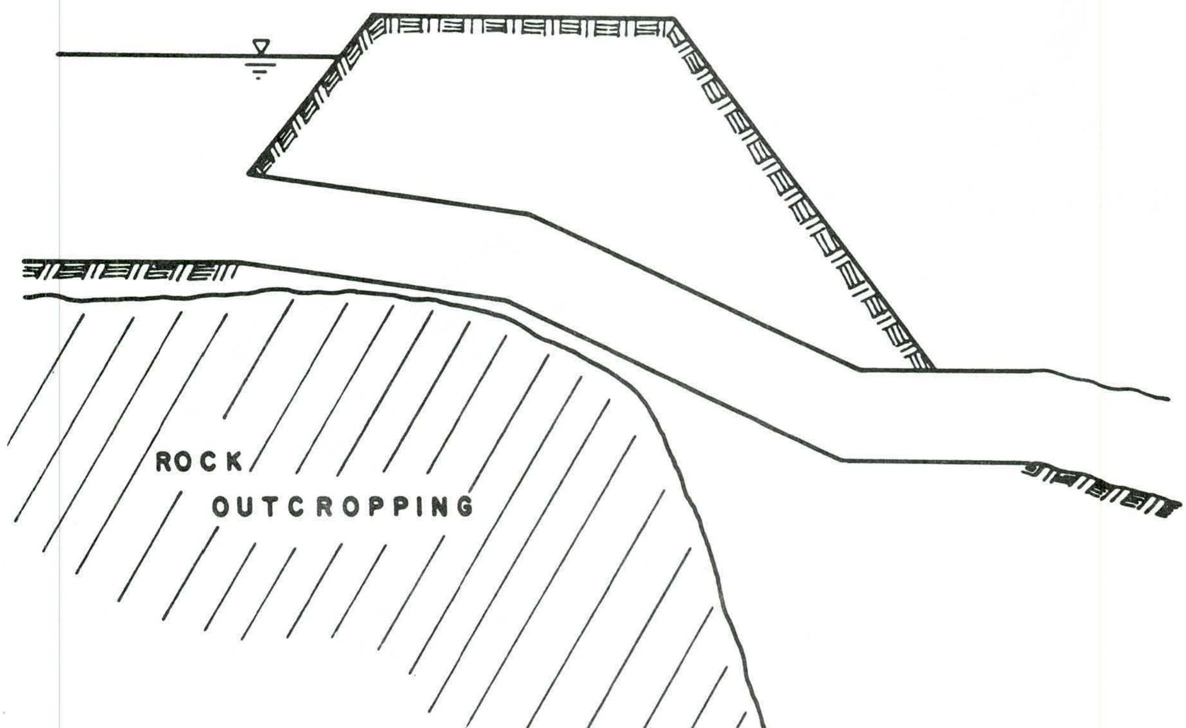


Figure 4.37
"Broken Back" Culvert

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 and that possible control section can be ignored except for the bend losses in outlet control.

Flotation and Anchorage: Flotation is the term used to describe the failure of a culvert due to the uplift forces caused by buoyancy. The buoyant force is produced from a combination of high head on the outside of the inlet and the large region of low pressure on the inside of the inlet caused by flow separation. As a result, a large bending moment is exerted on the end of the culvert. This problem has been noted in the case of culverts under high head, with shallow cover, on steep slopes, and with projecting inlets. The phenomenon can also be caused by debris blocking the culvert end or by damage to the inlet. The resulting uplift may cause the inlet ends of the barrel to rise and bend. Occasionally, the uplift force is great enough to dislodge the embankment. Generally, flexible barrel materials are more vulnerable to failure of this type because of their light weight and lack of resistance to longitudinal bending. Large, projecting, or mitered corrugated metal culverts are the most susceptible.

A number of precautions can be taken by the designer to guard against flotation. Steep slopes (1 to 1 or steeper) of adequate height, which are protected against erosion by slope paving or head walls, help inlet and outlet stability. When embankment fill heights are less than 1.5 times the pipe diameter or fill slopes are flatter then 1 to 1, flexible pipe installations should be provided with concrete headwalls for dead load, and rigid pipe installations susceptible to separation at the joints should be protected with tie bars. Limiting headwater buildup also helps prevent flotation. It is desirable to limit design headwater depths to 1.5 times the culvert height.

Safety: The issue of safety includes the following principals:

1. Stormwater naturally accumulates, frequently in amounts that present hazards to property, traffic, and life and health.
2. Because of the accumulation of stormwater, certain levels of hazards cannot be eliminated.
3. Stormwater frequently carries substantial amounts of debris that can threaten the hydraulic capacity of drainage facilities.
4. Devices placed on drainage facilities to restrict access by pedestrians will also restrict hydraulic capacity.
5. Multipurpose uses of many conduits are desirable, may provide safer day to day conditions, and require relatively easy pedestrian access to drainage works in order to be effectively used (trash racks would preclude this use).

During design, culvert entrances may require additional consideration for safety and for debris transported by stormwaters. Frequently, trash collection devices are also used as safety devices. The need for trash collection or safety devices should be determined during planning and before the design of drainage facilities. It is rare that cost-effective trash collectors can be retroactively added without a reduction of intended system design capacity. In any case, it is not a good policy for a failure of protection devices for humans to result in more property damage or greater hazards to traffic than would have happened if the protection devices were not used.

Safety devices can be divided into two types (U.S. Bureau of Reclamation):

Category I - Devices that Limit or Deter Access

» Fencing

» Guard rails

» Warning signs

» Pipe safety barriers

Category II - Devices that Permit Escape

» Safety nets

» Safety cables

» Safety racks

» Safety ladders

An important distinction between these two categories is that Category II, Safety Devices, may also impede the flow of stormwater into or through drainage facilities. There are three categories of safety to consider:

1. Life and Health
2. Traffic
3. Property

Primary safety issues are life and health safety; however, protection of traffic and property are also concerns. Life and health hazards are classified according to Table 4.4 (Source: U.S. Bureau of Reclamation).

From Table 4.4, all of the Phoenix Metropolitan Area would be categorized by classes A, B, or C. Considering growth potential, there are probably few areas of the

Table 4.4
Classification of Hazard Exposures

Class A	Waterways adjacent to schools and recreational areas, such as playgrounds, subject to frequent visits by children.
Class B	Waterways nearby or adjacent to urban areas or highways and subject to frequent visits by the public.
Class C	Waterways nearby or adjacent to farms or highways which could be subject to visits by children seeking recreation.
Class D	Waterways far removed from any dwelling, subject to infrequent visits by operating personnel and an occasional sportsman.
Class E	Waterways that would be a hazard to domestic animals.
Class F	Waterways that would be an extreme hazard to big game animals.

county that a classification of less than class B should be considered. Safety for drainage facilities should be considered for both dry weather and runoff conditions. Dry weather hazards include traffic and personal safety. Examples of traffic hazards include: improper placement of guard rails on structures; unprotected drops at structures located near roads; and grading which promotes vehicle rollovers.

During large storm events, people will sometimes walk or play in water that can carry them to drainage structures which are dangerous during flood conditions. Or, worse, purposely boat or float in drainage facilities during high runoff levels with the same results. It is not possible to develop drainage facilities that are totally safe, that will preclude people from doing unintelligent acts, and that will also be hydraulically efficient. These objectives are, for the most part, mutually exclusive. However, reasonable levels of protection can be provided to people exercising reasonable judgement even when the structure is performing its primary function, i.e., efficiently passing storm water.

The basic concept of this proposed approach to safety is to apply more restrictive measures as hazards increase. The primary purpose for constructing drainage facilities is the efficient conveyance of stormwater to minimize property damage and to permit traffic flow across and parallel to drainages; therefore, safety in this context will refer to protection from life and health hazards.

When any of the following conditions are met, trash racks will be required on the entrances to all conduits in areas of Class A or B hazard as determined from Table 4.4:

» Side slopes in channels steeper than 4(H):1(V) for concrete, grass and earth linings, and 3(H):1(V) for riprap linings.

» Conduits smaller than 7 feet in diameter, longer than 100 feet in length, and without 12-inches of freeboard at the design flow rate.

» Conduits with energy dissipators at the end.

» Conduits exiting multi-use detention facilities.

» Conduits with sufficient bend that the opposite end cannot be clearly seen.

A plugging factor of 50 percent will be used on all trash racks, and in areas that are considered a Class A hazard, the velocity through the rack shall be less than or equal to 2.0 fps (after plugging factor applied). For trash racks with velocities less than 3.0 fps after the plugging factor is applied, the losses caused by the trash rack can be ignored in computations. For greater velocities the loss will be computed, and added to the computed water surface, using the formula in Section 4.2.3.

Conditions that will cause racks to be used on outlets include:

1. Storm sewers, and
2. Inlets of pipes smaller than 7 feet in diameter that flow into recreation areas that are not designed for pedestrian use.

Flap gates can be considered for substitution for trash racks on conduit outlets when it can be shown that sedimentation will not prevent the flap gate from opening or that the design of the outlet structure will reduce downstream sedimentation that would prevent the flap gate from opening.

In instances where open channels connect conduits that meet the geometric and hazard requirements previously listed, Category I safety devices are required to restrict access to the general public along the entire reach of that channel. An example is a concrete lined channel with 1:1 side slopes in a Class B hazard, where the channel connects to culverts and the lower culvert has an energy dissipator at its outlet.

It should be noted that the current MAG standards cannot be used in most Class A hazard conditions, because the rack slope is too steep to provide sufficient open area to reduce the velocity below 2 fps. These racks can be used in Class B hazard areas and may require that losses be calculated.

Some additional conditions to consider are:

» New development must meet predetermined standards that control flooding. Design for safety should not compromise those standards.

» Drainage works in existing areas will often not meet the standards for flooding that is required of new development; however, where possible, the generally agreed level of protection against flooding should be attained without compromise for life and health safety.

4.3.2.8 Inlets: Culvert inlets are used to transition the flow from a ponded 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 4.3.4 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 natural 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* (FHWA 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 beveled-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 K_e is reduced from 0.5 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 4.38.

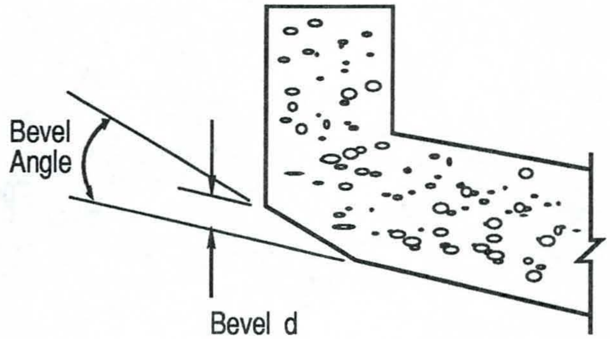


Figure 4.38
Inlet Bevel Detail

Side-tapered inlets have an enlarged face area accomplished by tapering sidewalls as shown on Figure 4.39. 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 inlet under throat control are: reduced flow contraction at the throat and increased head at the throat control section.

Slope-tapered inlets provide additional head at the throat section as shown on Figure 4.40. 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 insure flow control at the throat. More of the potential capacity of the culvert can then be insured.

The inlet control nomographs contained in Section 4.3.4 do not apply to the condition when drop inlets are used with or without grates. The turbulence caused by the flow dripping into the inlet box causes additional losses that are not accounted for in the inlet nomographs. When drop inlets are used, the headwater depth should be determined using Figure 4.7 (page 74) for catch basins with inlet flow only. The use of drop inlets is discouraged in culvert applications because of the danger of plugging from sediment and plugging of grated inlets from debris.

Prefabricated steel inlet end sections (Figure 4.41) 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.

Because of the potential for inlet uplift failure and inlet damage from other sources, concrete headwalls are required on all culvert installations unless it can be shown that these dangers do not exist. In those cases, metal end sections such as those shown in Figure 4.41 may be used.

4.3.2.9 Outlets: The receiving channel at culvert outlets must be protected from the high culvert outlet velocities caused by the flow constriction that is inherent in culvert operation. If the culvert outlet velocity is greater than the allowable velocity for the receiving channel lining material, protective measures must be provided.

Projecting culvert outlets are not permitted. Some means for flow transition must be provided. The minimum requirement is to provide a preformed metal or concrete end section or a headwall with or without a wingwall configuration with a cutoff wall provided at the end of the apron. Standard outlets for closed conduits are presented in Section 4.5. Energy dissipation structures are presented in Chapter 6.

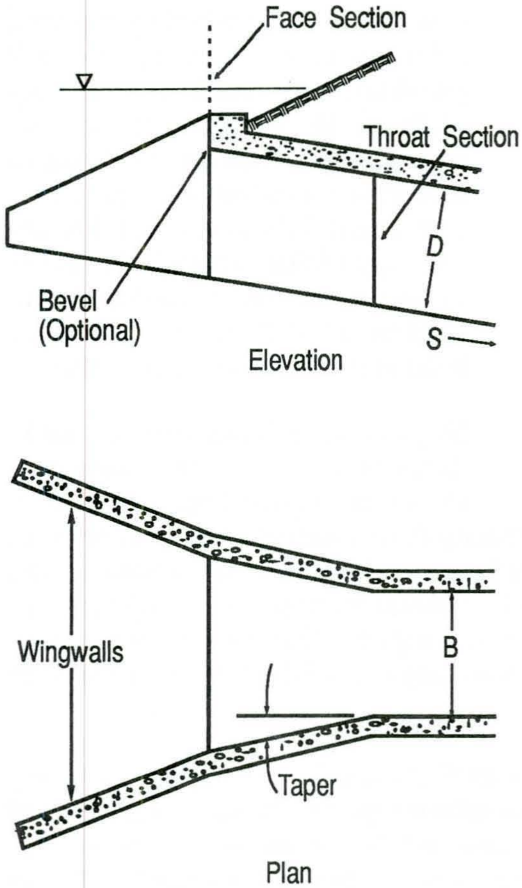


Figure 4.39
Side Tapered Inlet

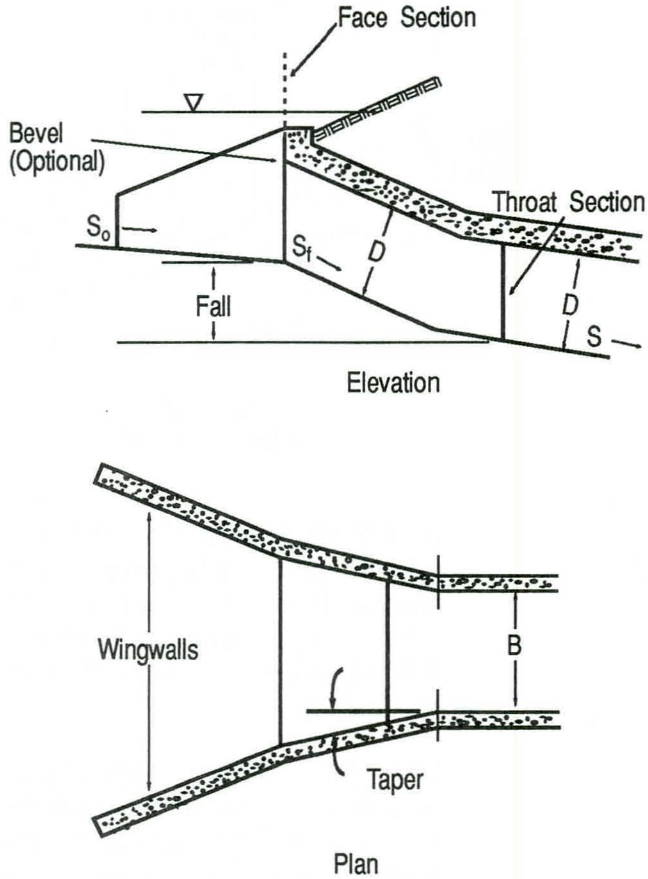


Figure 4.40
Slope Tapered Inlet

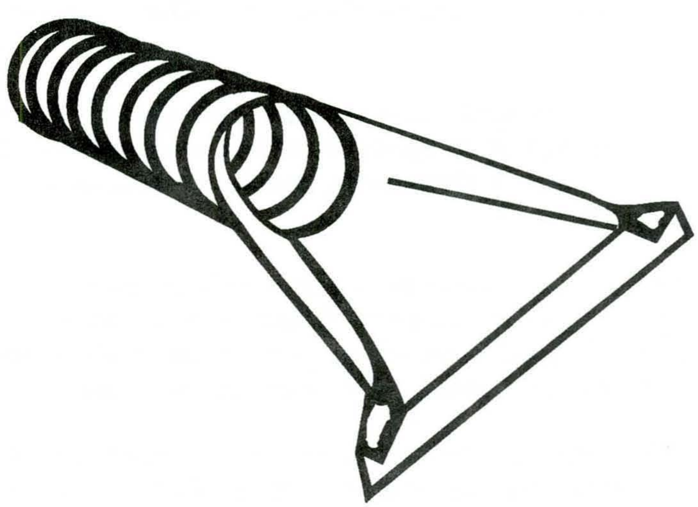


Figure 4.41
Prefabricated Culvert End Section

4.3.3 Design Procedures

4.3.3.1 Common Culvert Installations: The culvert design method provides a convenient and organized procedure for designing culverts, considering inlet and outlet control. While it is possible to follow the design method without an understanding of culvert hydraulics, this is not recommended.

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 4.42). 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.

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 of Section 4.3.4 are used in the design process. For the following discussion, refer to the schematic inlet control nomograph shown in Figure 4.43.

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 (HW_i). If trash racks are used, add trash rack losses to HW_i.

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 4.3.4 are used in the design process. For illustration, refer to the schematic critical depth chart and outlet control nomograph shown in Figures 4.44 and 4.45, respectively.

1. Determine the tailwater depth above the outlet invert (TW) at the design flow rate. This is obtained from backwater or normal depth calculations, 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. 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.

Figure 4.42
Culvert Design Form

PROJECT: ______________________________
____

STATION: ______________________
SHEET ______ OF ______

CULVERT DESIGN FORM
DESIGNER / DATE: __________ / __________
REVIEWER / DATE: __________ / __________

HYDROLOGICAL DATA

SEE ADD'L. SHTS.

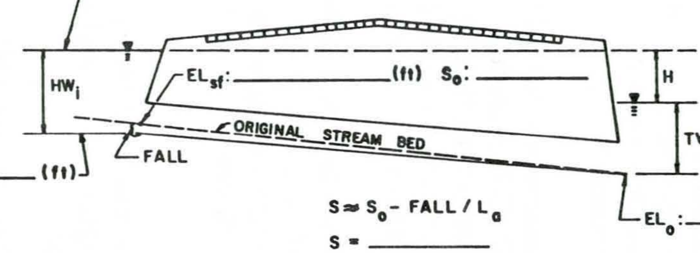
- ☐ METHOD: ______________________
- ☐ DRAINAGE AREA: __________ ☐ STREAM SLOPE: __________
- ☐ CHANNEL SHAPE: ______________________
- ☐ ROUTING: __________ ☐ OTHER: __________

DESIGN FLOWS/TAILWATER

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

ROADWAY ELEVATION: __________ (ft)

EL_{hd}: __________ (ft)



EL_i __________ (ft)

$S \approx S_o - FALL / L_a$

S = __________

L_a = __________

EL_o: __________ (ft)

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_f/D FROM DESIGN CHARTS

(3) $FALL = HW_i - (EL_{hd} - EL_{sf})$; FALL IS ZERO FOR CULVERTS ON GRADE

(4) $EL_{hi} = HW_i + EL_i$ (INVERT OF INLET CONTROL SECTION)

(5) TW BASED ON DOWN STREAM CONTROL OR FLOW DEPTH IN CHANNEL.

(6) $h_o = TW$ or $(d_c + D/2)$ (WHICHEVER IS GREATER)

(7) $H = \left[1 + k_e + (29 n^2 L) / R^{1.33}\right] V^2/2g$

(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: ______________________

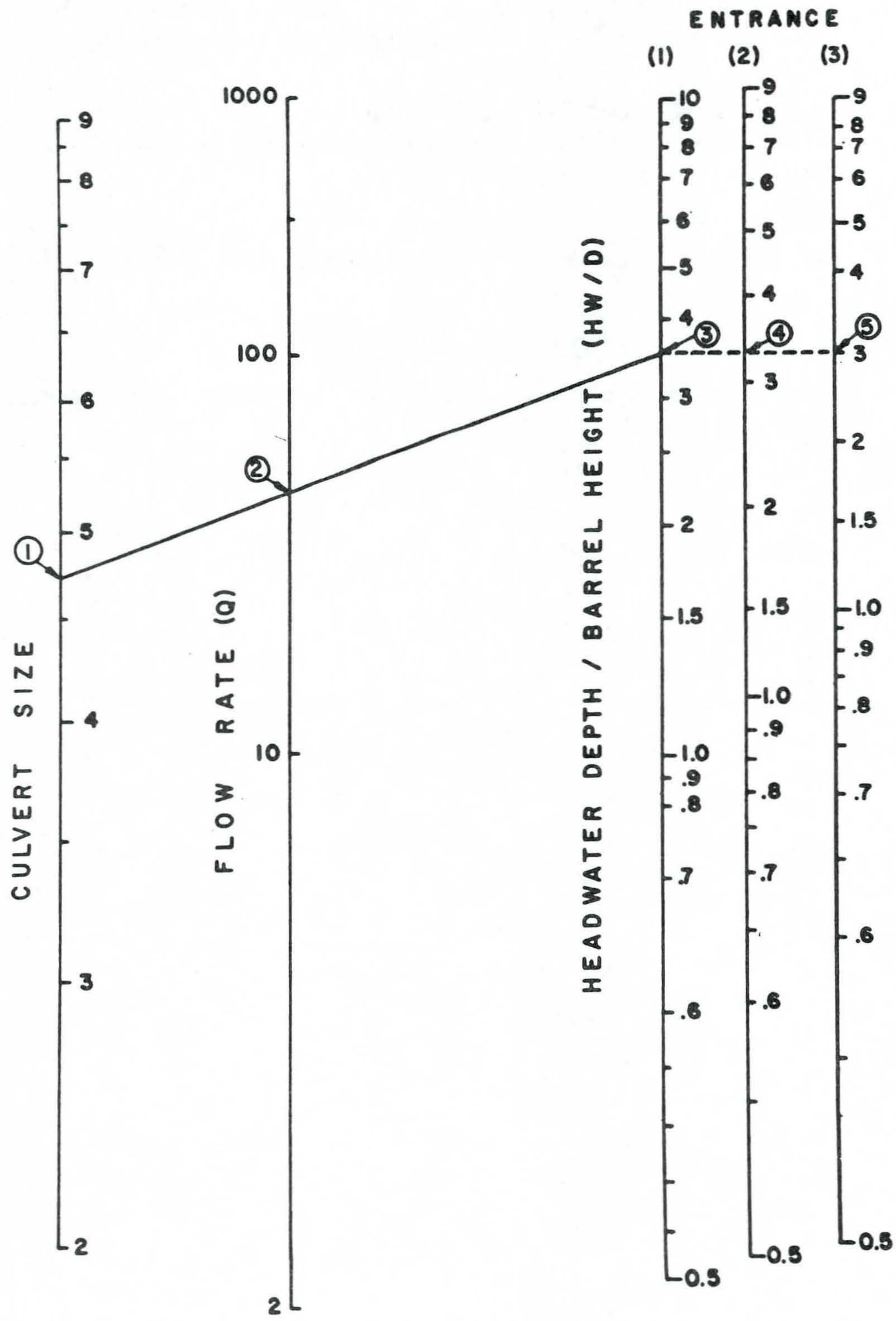


Figure 4.43
Inlet Control Nomograph (Schematic)

2. Enter the appropriate critical depth chart (Figure 4.44) 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.

   (**Note:** The d_c curves are truncated for convenience when they converge. If an accurate d_c is required for $d_c > 0.9D$, consult the *Handbook of Hydraulics* by King and Brater, 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).

$$h_o = TW \text{ or } (d_c + D)/2, \text{ whichever is larger.} \quad \textbf{(4.7)}$$

5. From Table 4.6 (page 153) obtain the appropriate entrance loss coefficient, k_e, for the culvert inlet configuration.

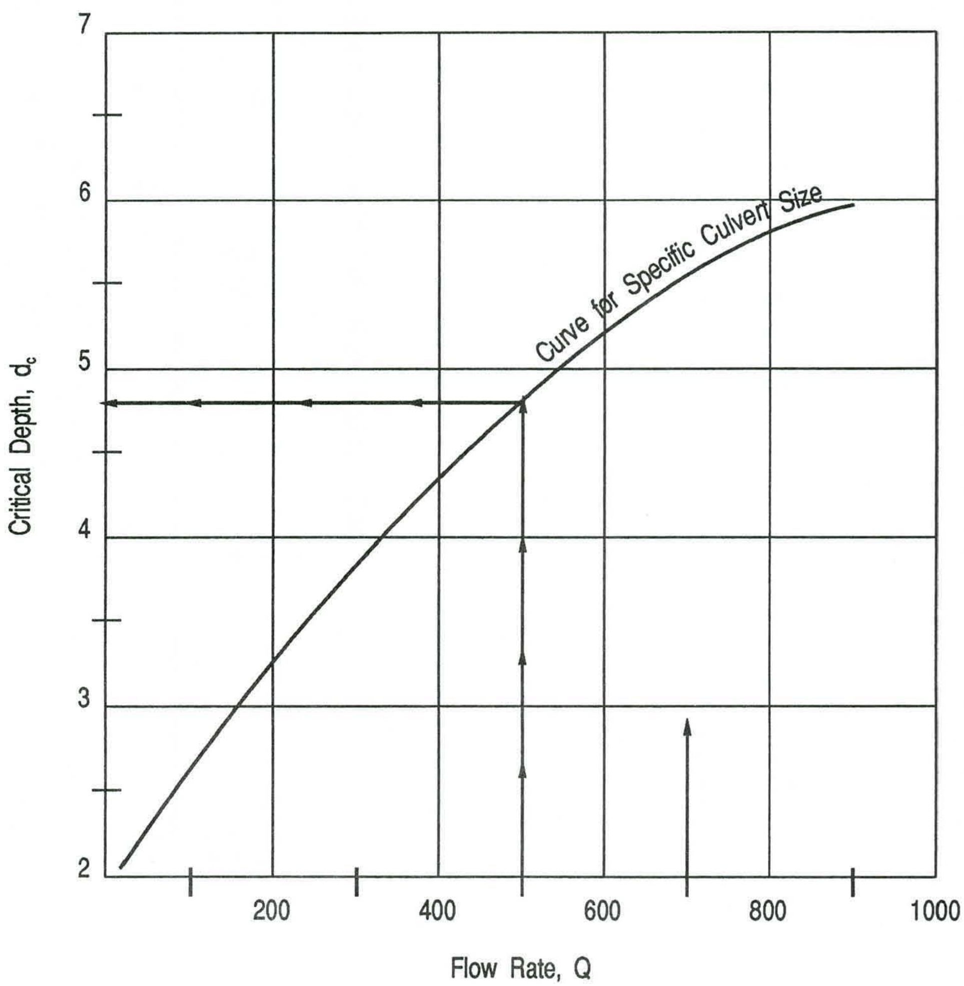


Figure 4.44
Critical Depth Chart (Schematic)

6. Determine the losses through the culvert barrel, H, using the outlet control nomograph (Figure 4.45) or appropriate equations if outside the range of the nomograph.

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

   $$L_1 = L\,[n_1/n]^2 \tag{4.8}$$

   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 Head Loss (H) scale. Read H, the energy loss through the culvert, including entrance, friction, and outlet losses.

7. Calculate the required outlet control headwater elevation.

   $$EL_{ho} = EL_o + H + h_o \tag{4.9}$$

   where EL_o is the invert elevation at the outlet.

8. If the outlet control headwater elevation exceeds the design headwater elevation, 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.

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 4.46). Normal depth for circular and rectangular culverts can be found using Figure 4.52 (page 154).

2. If the controlling headwater is in outlet control, determine the area of flow at the outlet based on the barrel geometry (see Figure 4.47) 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

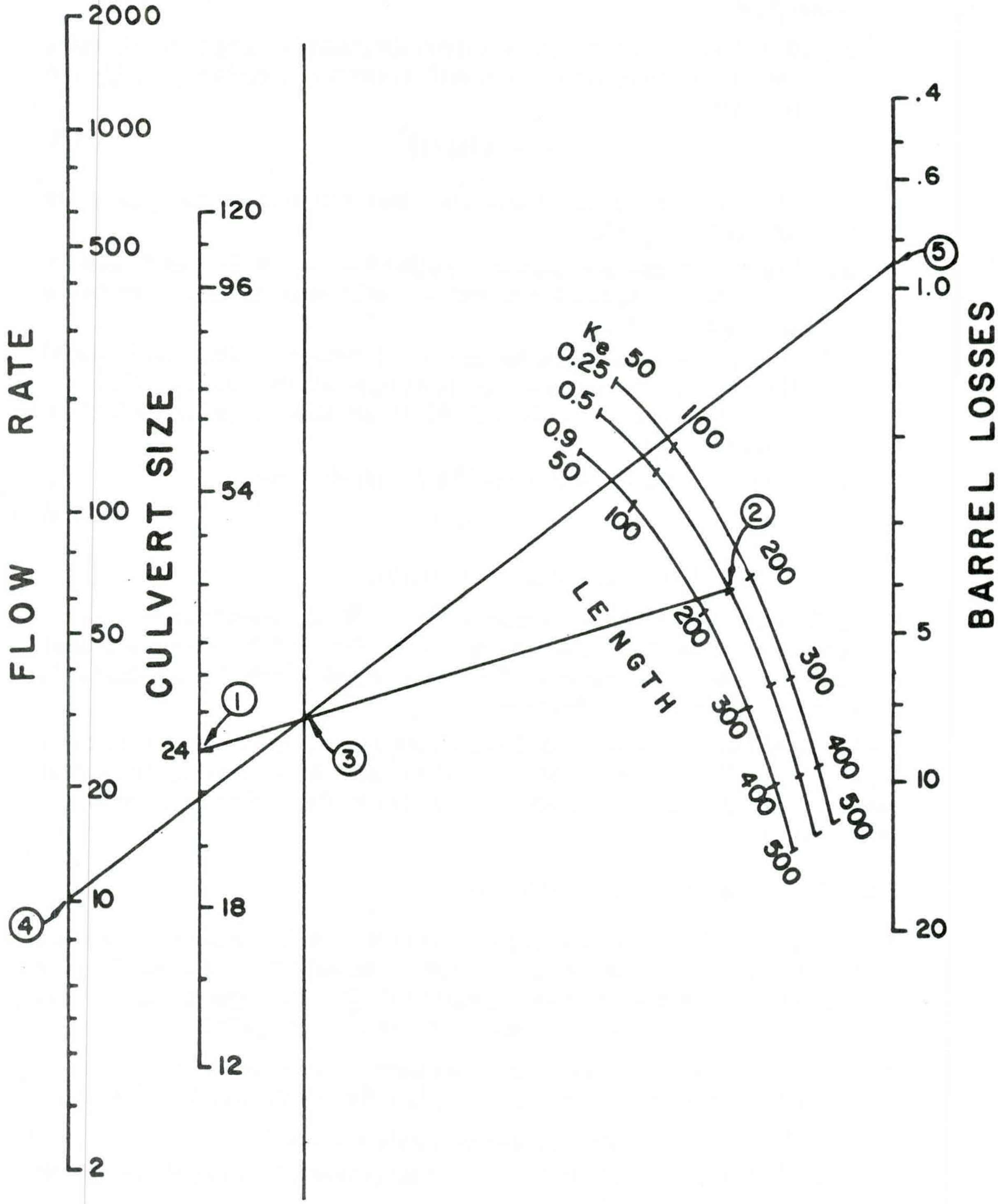
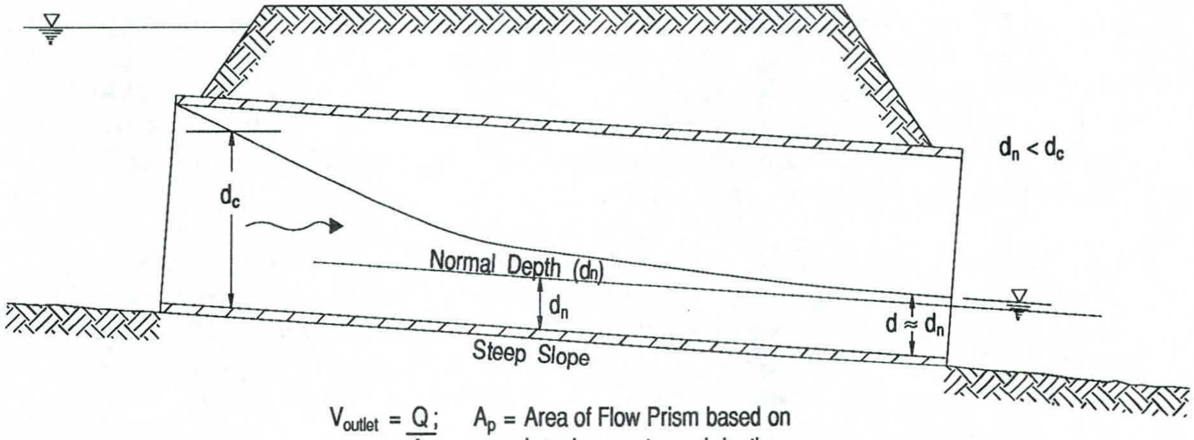
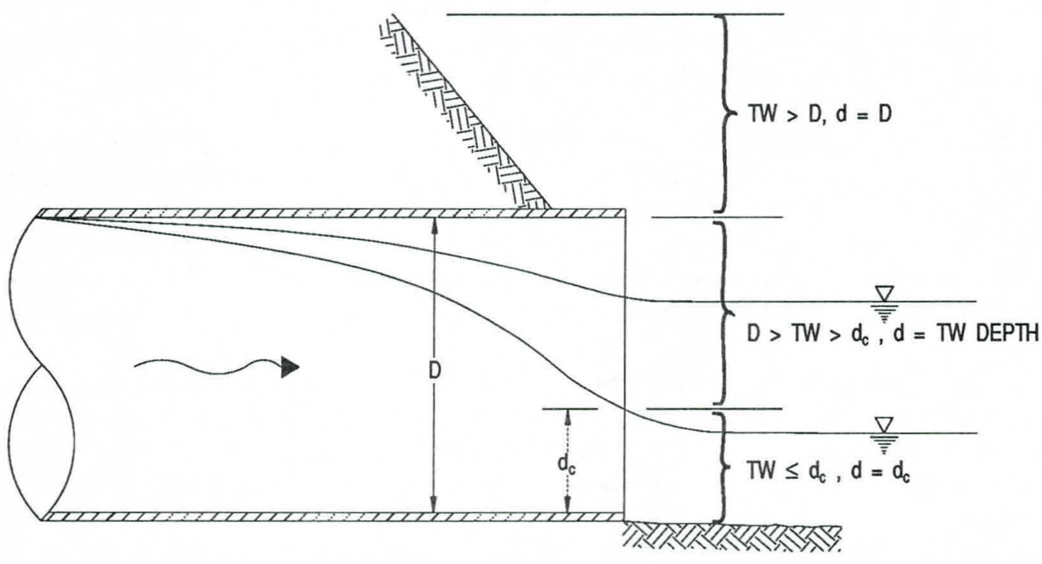


Figure 4.45
Outlet Control Nomograph (Schematic)



$V_{outlet} = \frac{Q}{A_p}$; A_p = Area of Flow Prism based on barrel geometry and depth equal to normal depth

Figure 4.46
Outlet Velocity — Inlet Control



$V_{outlet} = \frac{Q}{A_p}$; A_p = Area of Flow Prism based on Barrel Geometry and d

Figure 4.47
Outlet Velocity — Outlet Control

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 $(d_c + D)/2$. 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 0.75D, the approximate method should not be used.

If the selected culvert will not fit the site, return to the culvert design process and select another culvert. If neither tapered inlets nor flow routing are to be applied, document the design. Culvert design shall include a performance curve which displays culvert behavior over a range of discharges. Development of performance curves is presented in Section 4.3.3.4 and example problem number 4 in Section 4.3.5.4 (page 181) contains a performance curve calculation.

4.3.3.2 Uncommon Culvert Requirements

Storage Routing: A significant storage capacity behind a highway embankment attenuates a flood hydrograph. Because of the reduction of the peak discharge associated with this attenuation, the required capacity of the culvert, and its size, may be reduced considerably in some cases. The reduced size may well justify some increase in the hydrologic design effort.

All reservoir routing procedures require three basic data inputs: 1) an inflow hydrograph; 2) an elevation versus storage relationship; and 3) an elevation versus discharge relationship. A complete inflow hydrograph, not just the peak discharge, must be generated. Elevation, often denoted as stage, is the parameter which relates storage to discharge providing the key to the storage routing solution.

Elevation versus storage data can be obtained from a topographic map of the culvert site. The area enveloped by each contour line is planimetered and recorded. The average area between each set of contour lines is obtained and multiplied by the contour interval to find the incremental volume. These incremental volumes are added together to find the accumulated volume at each elevation. These data can then be plotted, as stage versus storage.

Elevation versus discharge data can be computed from culvert data and the roadway geometry as described elsewhere in this section. 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.

Despite the consideration of storage routing, the selection of an appropriate culvert size for a given set of hydrologic and site conditions is the design objective. However, in order to perform the storage routing calculations, a culvert must first be selected. Storage routing calculations will then be required to verify the selected size. *Hydraulic Design of Highway Culverts* (FHWA 1985), contains a procedure to aid in selecting an initial culvert size based on an estimated peak discharge achieved from storage routing.

The storage-indication method of flood routing is used to establish the outflow hydrograph and attenuated peak discharge resulting from the embankment storage. Section 8.7 describes the application of the storage-indication method and contains a flood routing example.

Culverts With Drop Inlets: When culverts have drop inlets, normal culvert design nomographs are not applicable. The water falling into the catch basin causes significant turbulence and energy losses. For this condition, Figure 4.7 (page 74) for storm drain inlets should be used.

Detention Basin Outlets: 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 storm drain system, procedures from Section 4.2 should be used to establish the hydraulic grade line for that storm drain to check for outlet control.

4.3.3.3 Head Loss For Trash Racks: For trash racks with approach velocities less than 3 fps, it is not necessary to include a loss for the trash rack; however, for velocities greater than 3 fps, such computations are required.

Trash rack losses are a function of velocity through the rack, bar thickness, bar spacing, and orientation of the flow entering the rack, the latter condition being an important factor. Trash racks 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 losses through the rack can be computed with the formula:

$$H_g = F^* K_g [w/x]^{4/3} [V_u^2/2g] \sin \theta_g \tag{4.10}$$

The expected loss from a trash rack is greatly affected by the approach angle. The loss computed by Equation 4.10 should be multiplied by the appropriate value from Table 4.5, when the approach channel and culvert are at an angle to each other.

4.3.3.4 Performance Curves: Performance curves are representations of flow rate versus headwater depth or elevation 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 4.3.4. The headwaters corresponding to the series of flow rates are determined and then plotted. The transition zone is inherent in the nomographs.

Table 4.5
Loss Factors for Approach Angle Skewed to Entrance

Approach Angle, degrees	Loss Factor
0	0.4
20	1.7
40	3.0
60	6.0

Outlet Control: The outlet control performance curves are developed using the outlet control nomographs of Section 4.3.4. 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.

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. An example of development of a performance curve is contained in Section 4.3.5.

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.

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-, and 50-year flow rates be used for developing the performance curve below the headwater depth where roadway overtopping begins.
2. Combine the inlet and outlet control performance curves to define a single performance curve for the culvert.
3. When the culvert headwater elevations exceed the roadway crest elevation, overtopping will begin. Calculate the equivalent upstream water surface depth above the roadway (crest of weir) for each selected flow rate. Use these water surface depths and Equation 4.11 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 elevation 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 flatten severely. The headwater will rise very slowly from that point on. Figure 4.48 depicts an overall culvert performance curve with roadway overtopping. Example problem number 4 in Section 4.3.5 illustrates the development of an overall culvert performance curve. The 100-year discharge should be identified on the performance curve and the corresponding depth of flow over the roadway.

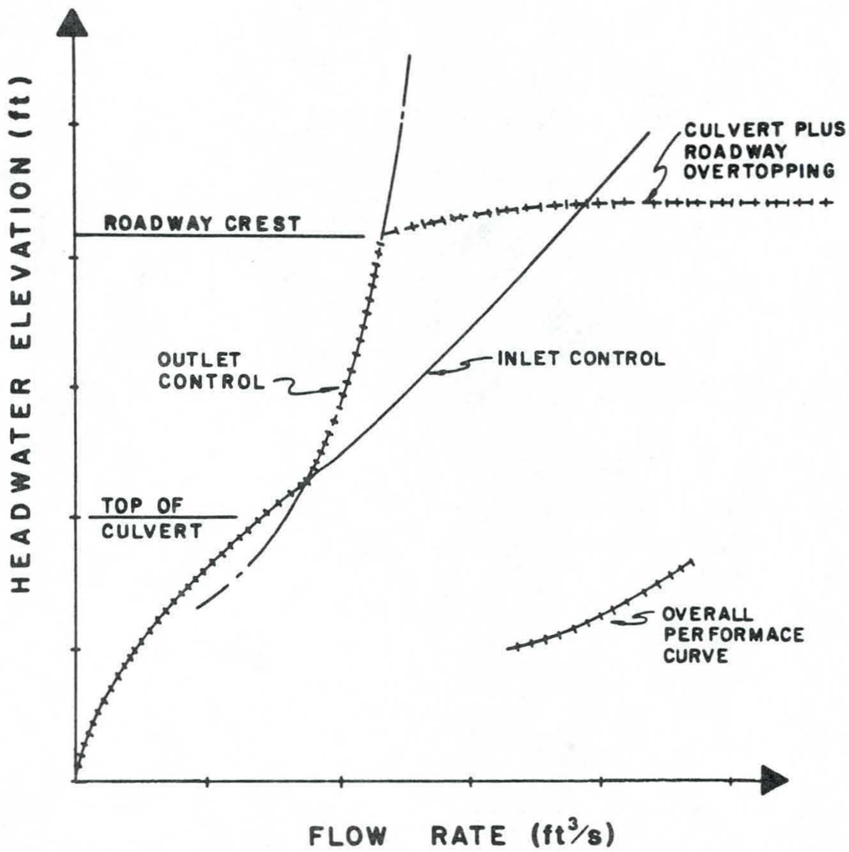


Figure 4.48
Culvert Performance Curve with Roadway Overtopping

4.3.3.5 Roadway Overtopping: 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 across the roadway. The flow across the roadway is calculated from the broad crested weir equation:

$$Q_o = K_t C_r L_s (HW_r)^{1.5} \qquad \textbf{(4.11)}$$

The charts in Figure 4.49 indicate how to evaluate the correction factors K_t and C_r.

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 4.50).

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

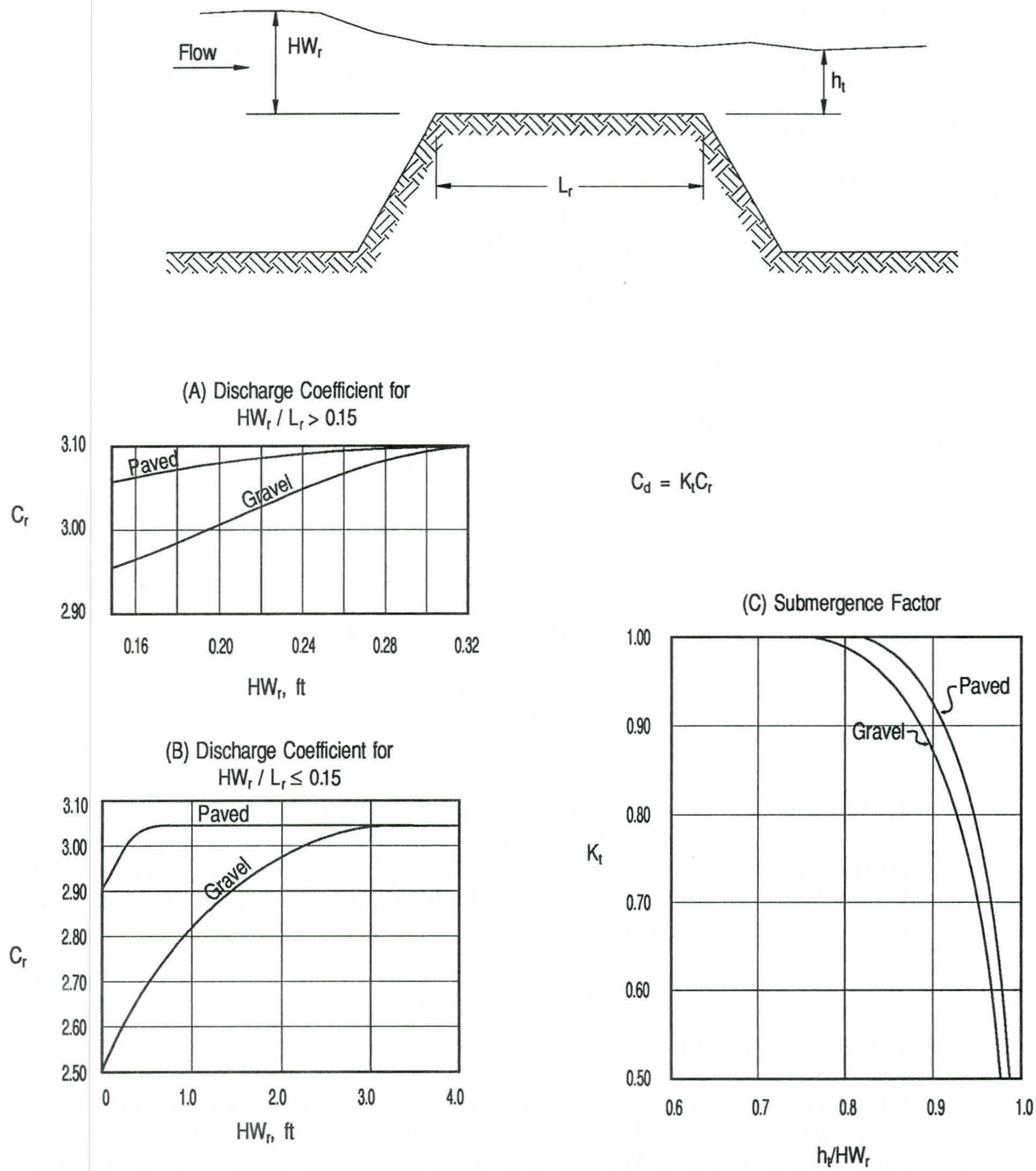


Figure 4.49
Discharge Coefficient and Submergence Factor for Roadway Overtopping

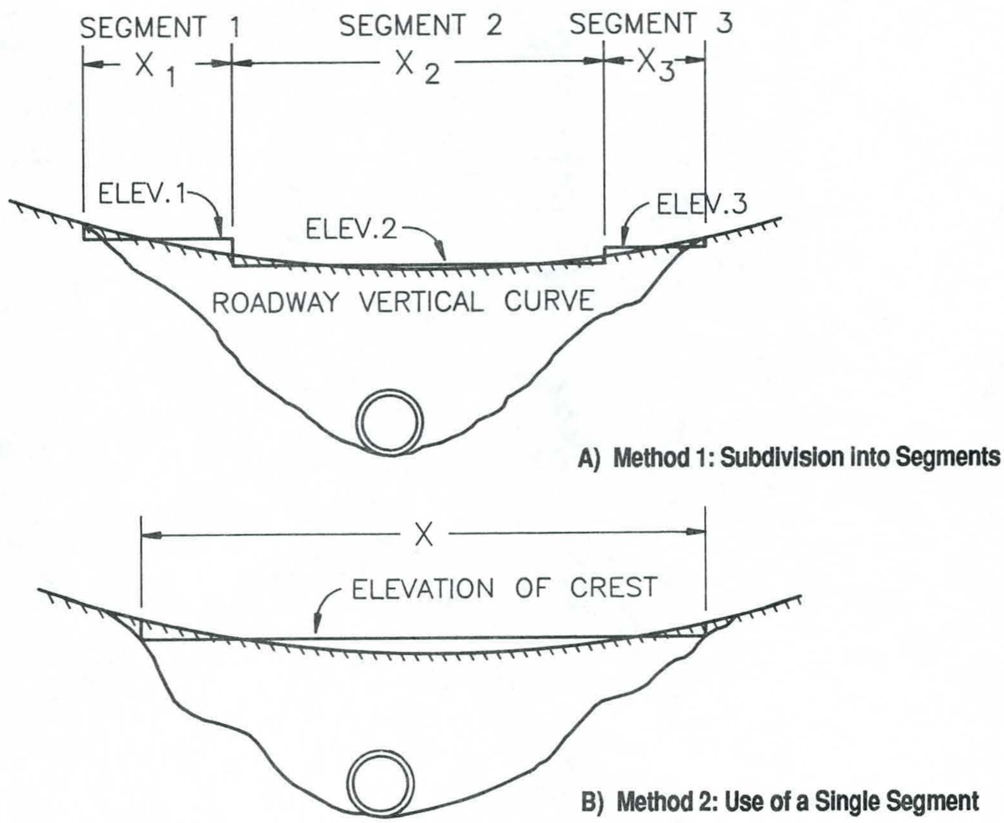


Figure 4.50
Weir Crest Length Determinations for Roadway Overtopping

flow and road overflow. The headwater depth for a specific discharge, such as the 100 year discharge can then be read from the curve.

Design example 4 in Section 4.3.5 illustrates this procedure.

4.3.3.6 Junctions: Flow from two or more separate culverts or storm sewers 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 4.51). A drainage pipe collecting runoff from the overlying roadway surface and discharging into a culvert barrel is an example of a storm sewer/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.

$$H_j = y' + H_{v1} - H_{v2} \qquad \textbf{(4.12)}$$

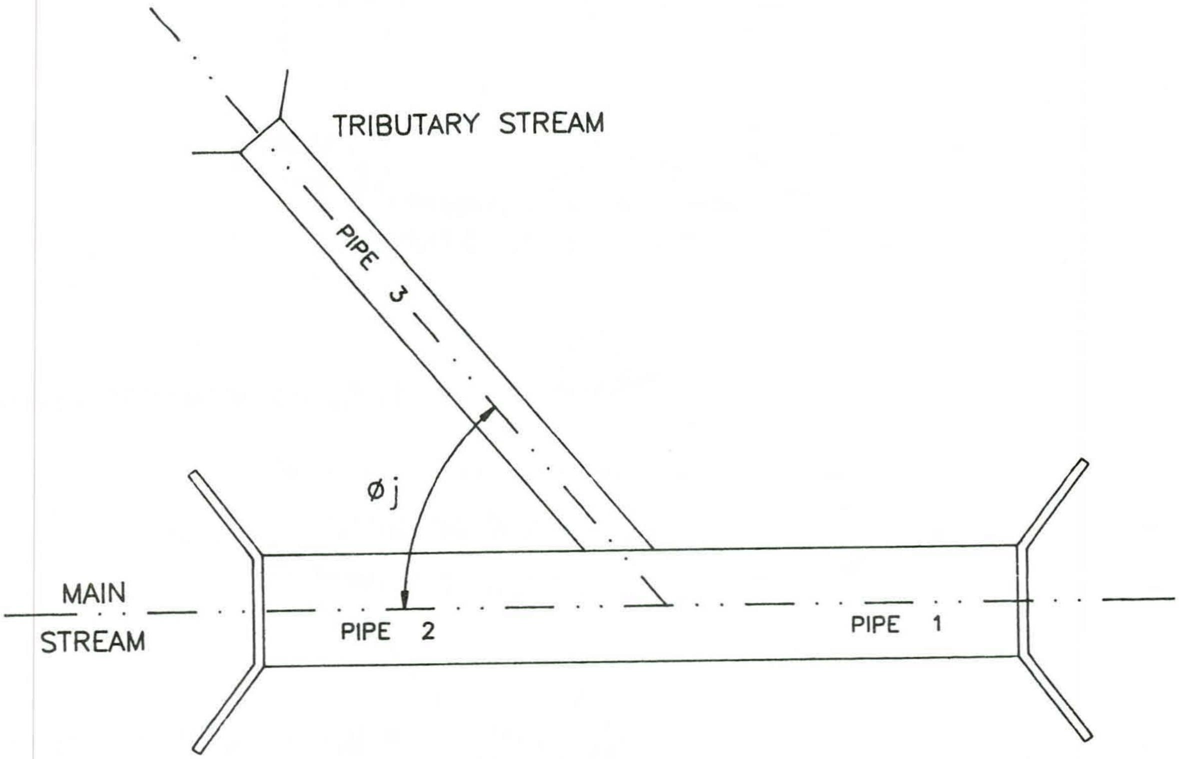


Figure 4.51
Culvert Junction

The formula for y′ is based on momentum considerations and is as follows:

$$y' = \frac{(Q_2V_2 - Q_1V_1 - Q_3V_3 \cos \phi_j)}{(0.5\,(A_1 + A_2)\,g)} \qquad \textbf{(4.13)}$$

4.3.4 Design Aids

4.3.4.1 Culvert Design Form: The Culvert Design Form(Figure 4.42, page 140) 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 of the form are also included. At the top right is a small sketch of the 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 4.6 and Figures 4.52 through 4.72 should facilitate completion of the Culvert Design Form.

Table 4.6
Entrance Loss Coefficients
Outlet Control, Full or Partly Full Entrance Head Loss
$$H_e = k_e \,(V^2/2g)$$

Type of Structure and Design of Entrance	Coefficient k_e
Pipe, Concrete	
Projecting from fill, socket end (grove-end)	0.2
Projecting from fill, square cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove-end)	0.2
Square-edge	0.5
Rounded (radius = 1/12D)	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 edges on 3 sides	0.2
Wingwalls at 30° to 75° to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius to 1/12 barrel dimension, or beveled top edge	0.2
Wingwall 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

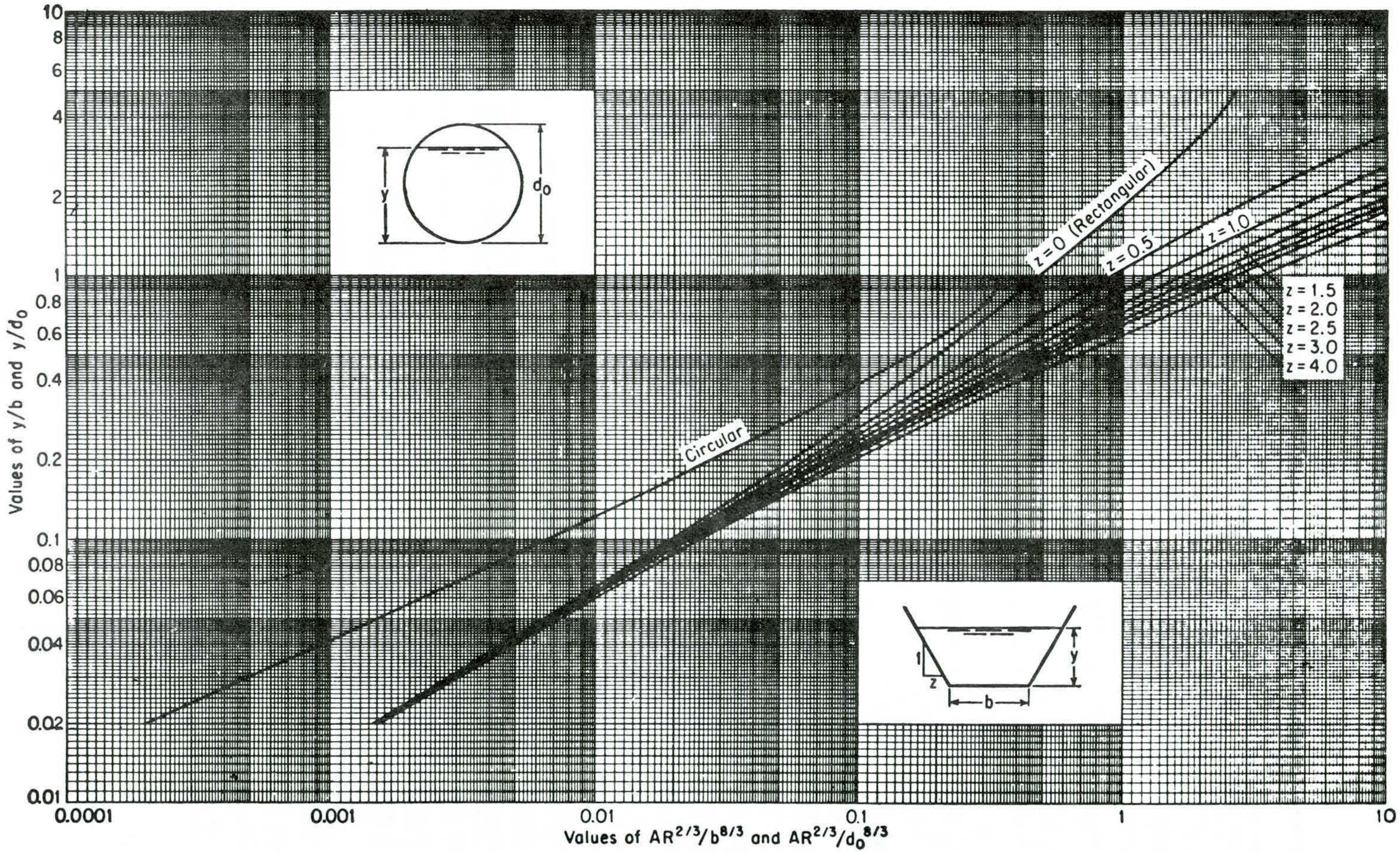


Figure 4.52
Curves for Determining the Normal Depth

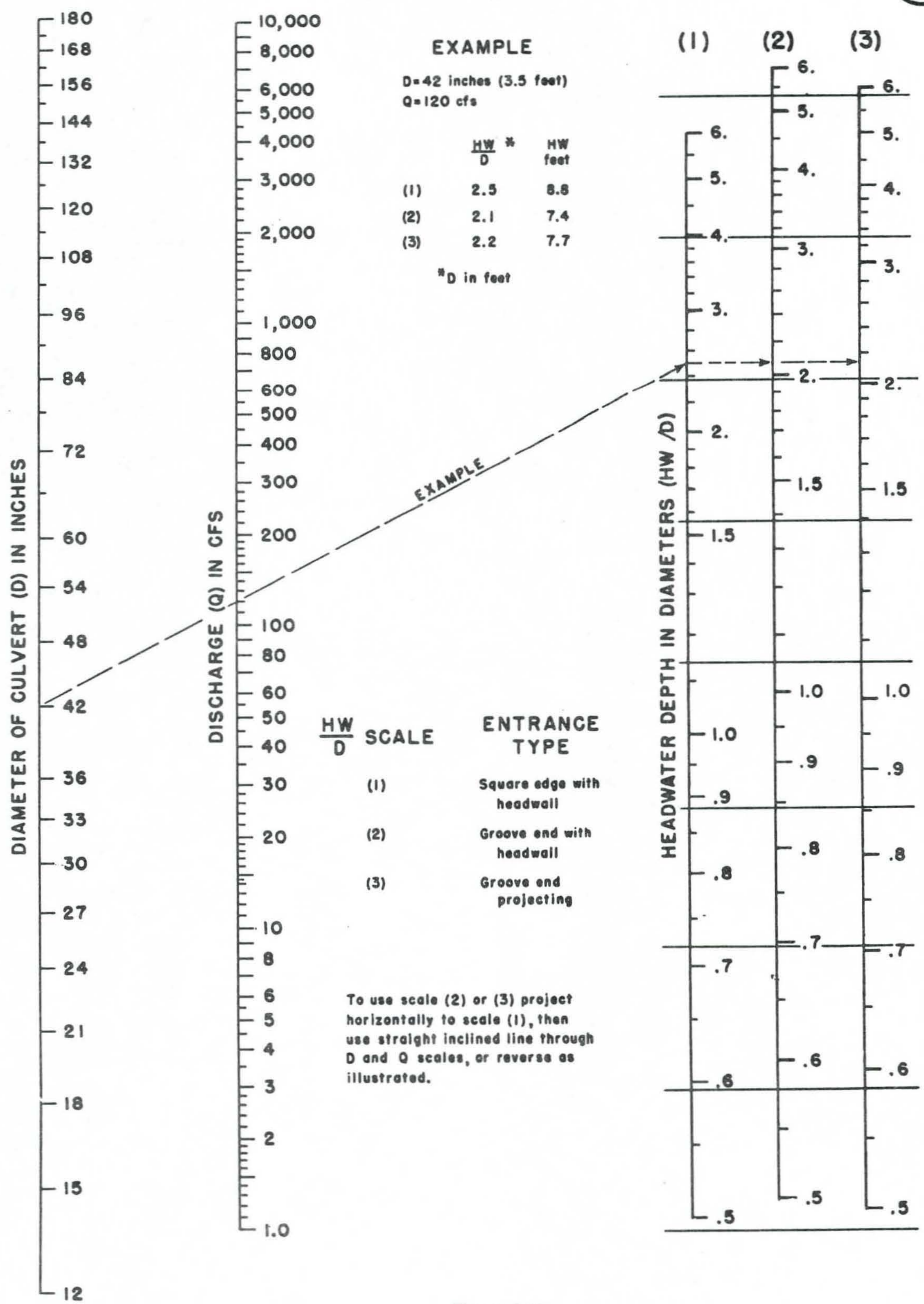


Figure 4.53
Headwater Depth for Concrete Pipe Culverts with Inlet Control

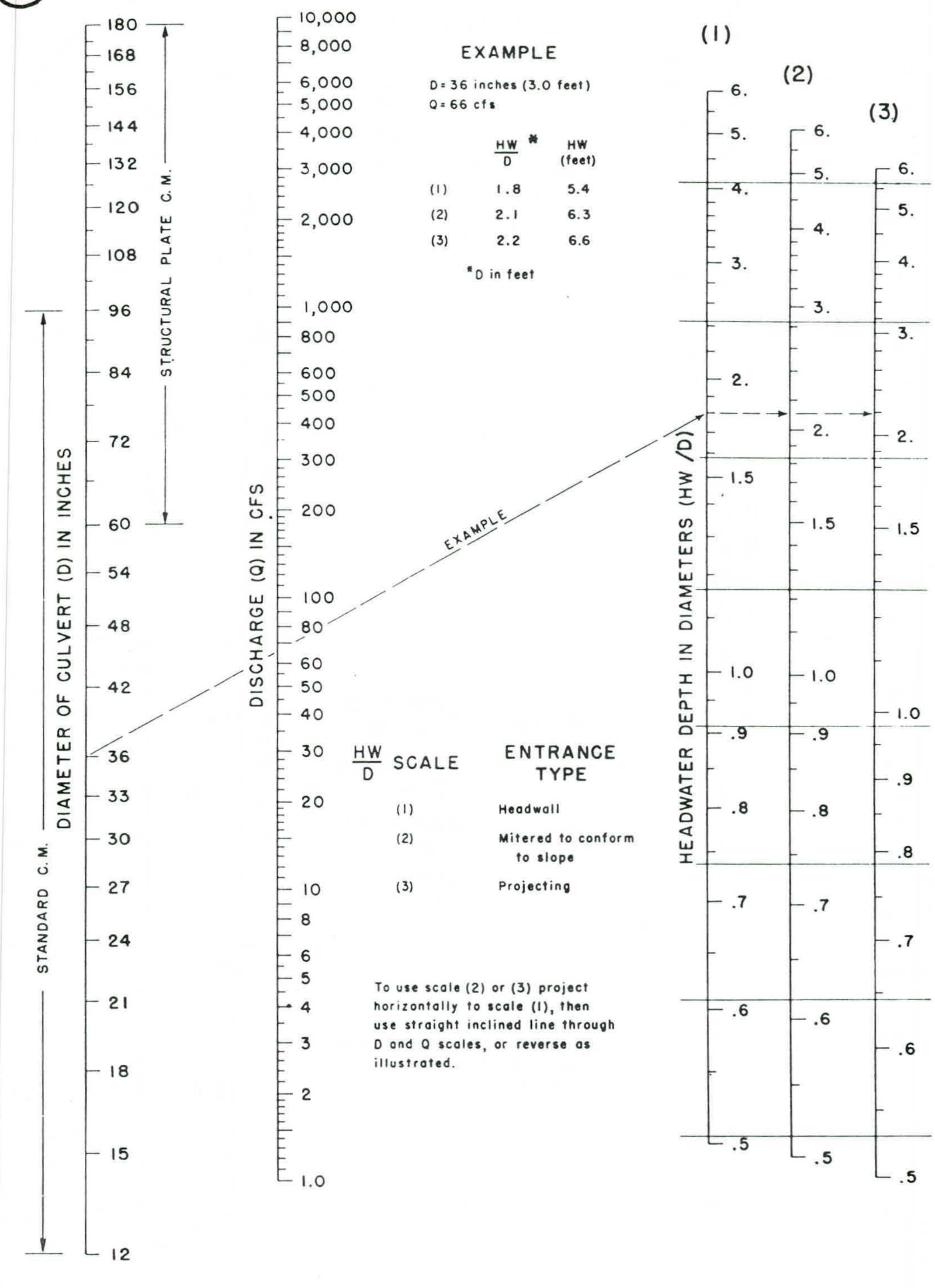


Figure 4.54
Headwater Depth for C.M. Pipe Culverts with Inlet Control

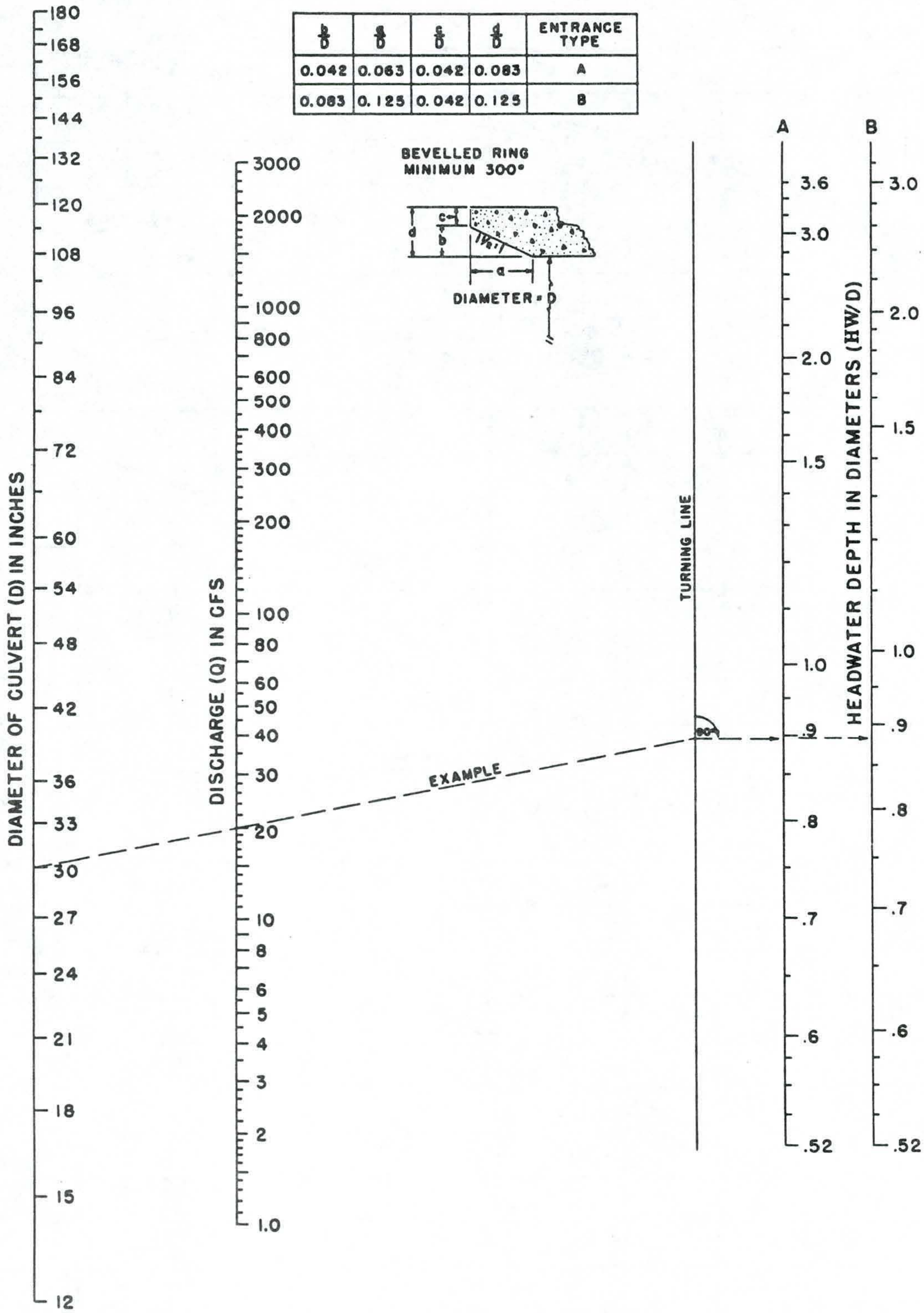


Figure 4.55
Headwater Depth for Circular Pipe Culverts with Beveled Ring Inlet Control

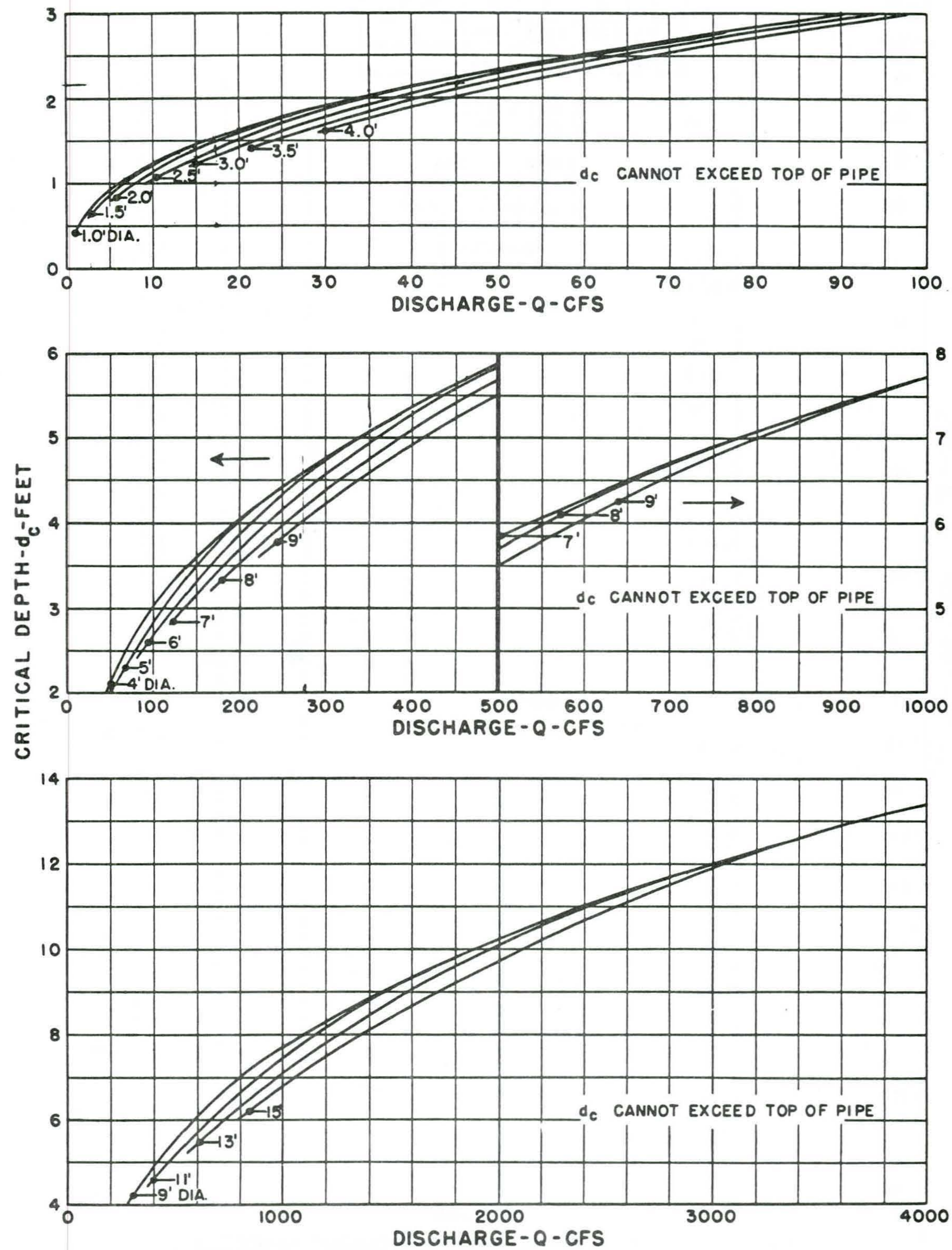


Figure 4.56
Critical Depth for Circular Pipe

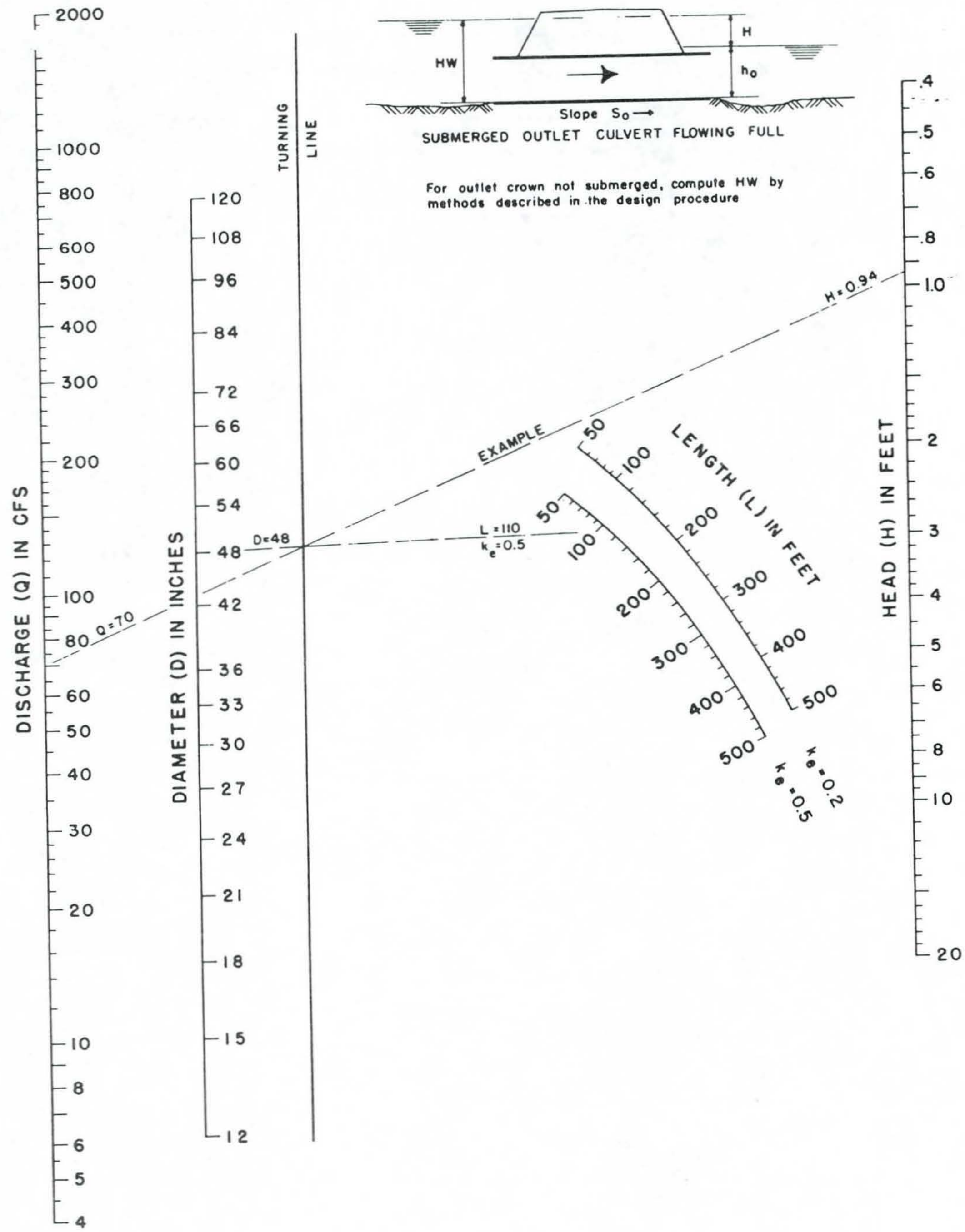


Figure 4.57
Head for Concrete Pipe Culverts Flowing Full

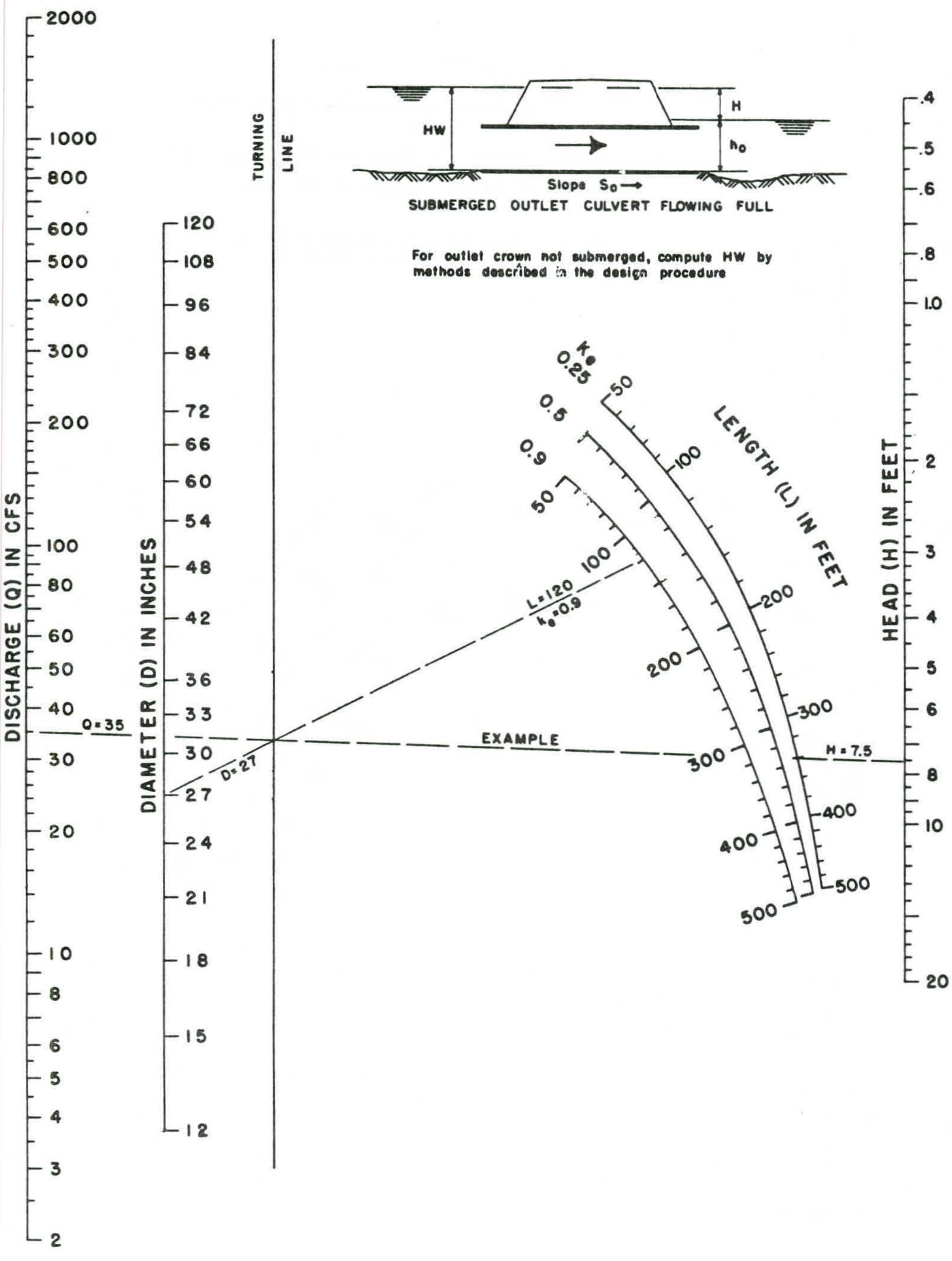


Figure 4.58
Head for Standard C.M. Pipe Culverts Flowing Full
n = 0.024

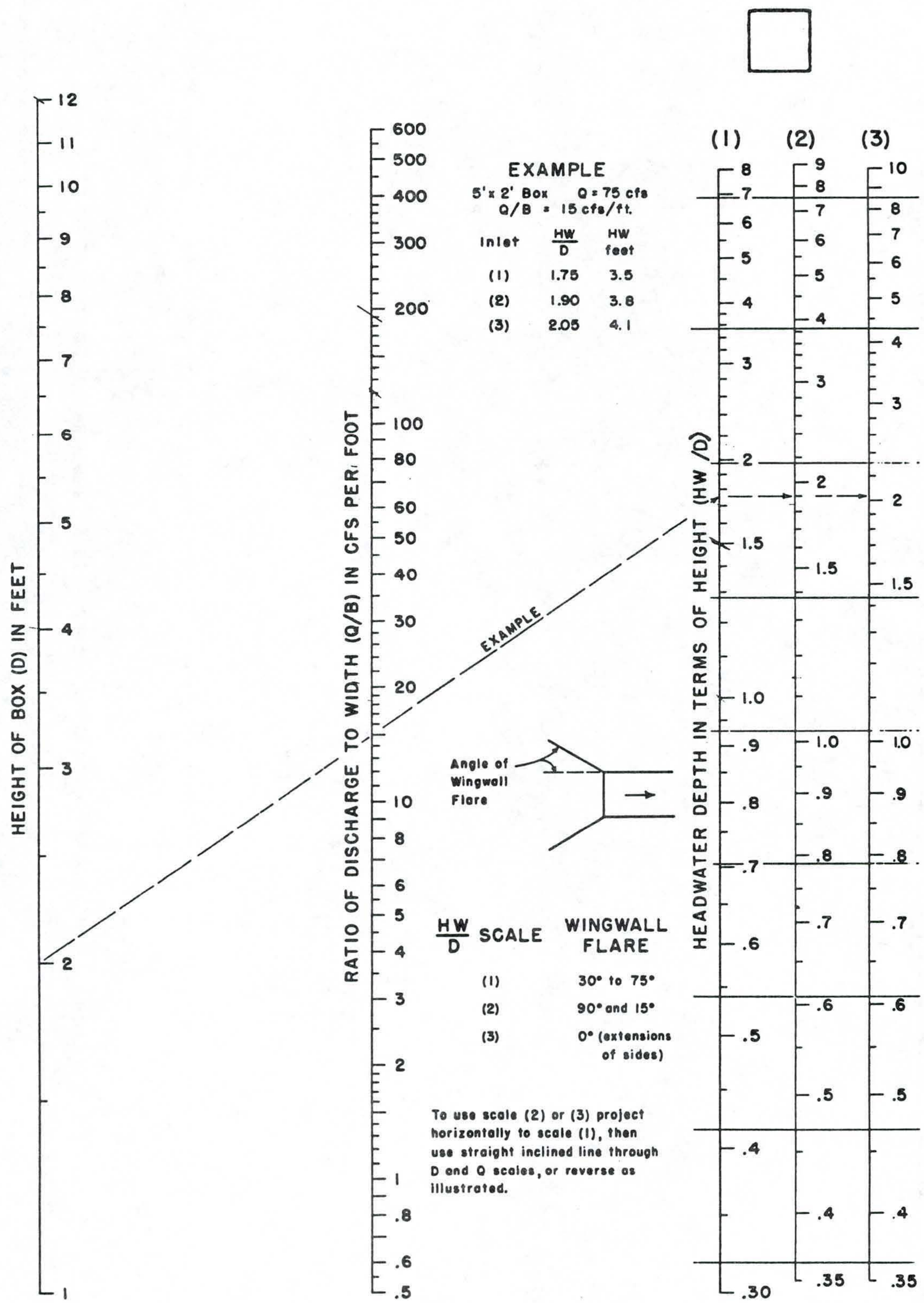


Figure 4.59
Headwater Depth for Box Culverts with Inlet Control

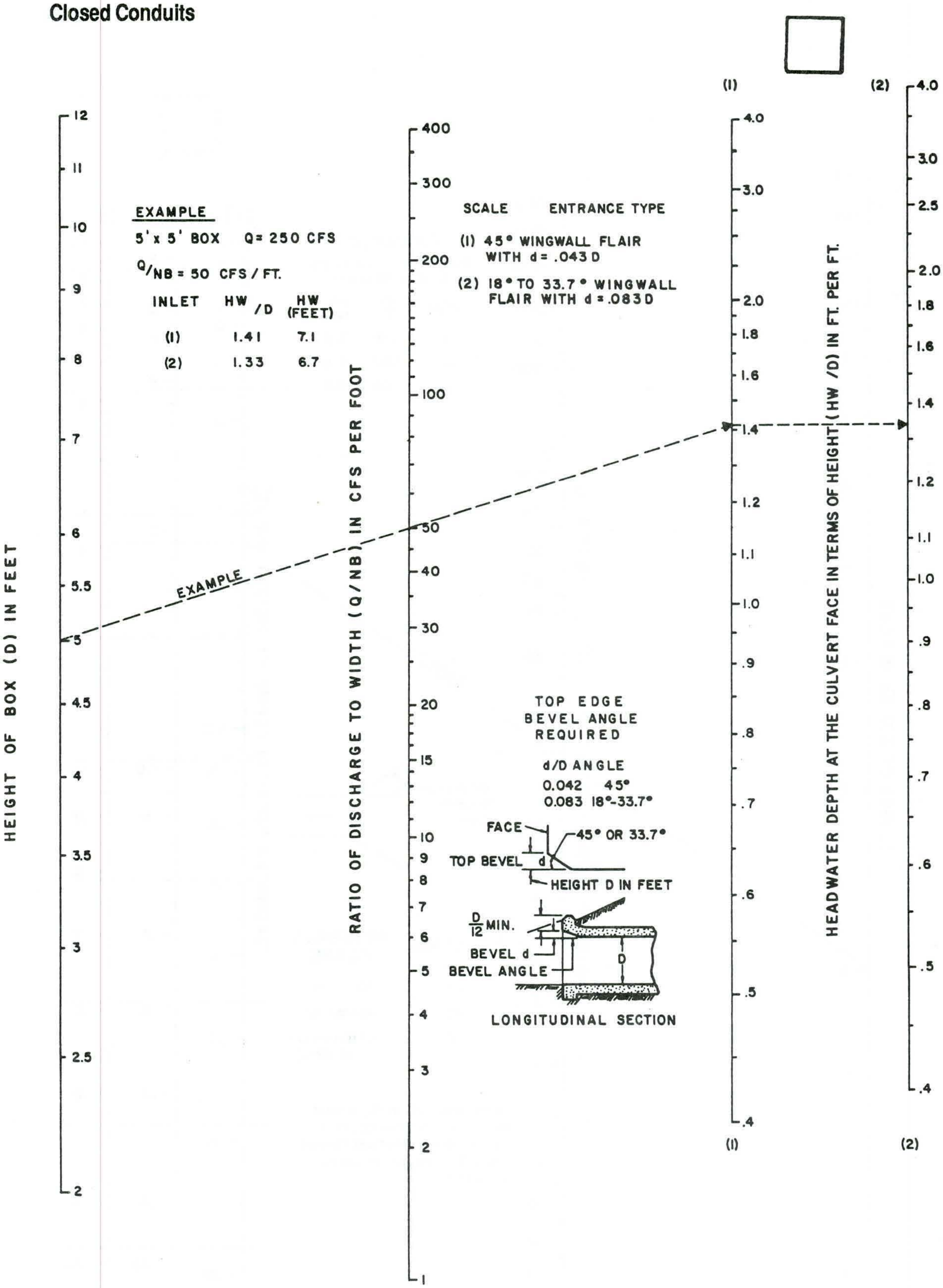


Figure 4.60
Headwater Depth for a Rectangular Box Culvert with Inlet Control, Flared Wingwalls (18 to 33.7 Degrees and 45 Degrees), and Beveled Edge at the Top of the Inlet

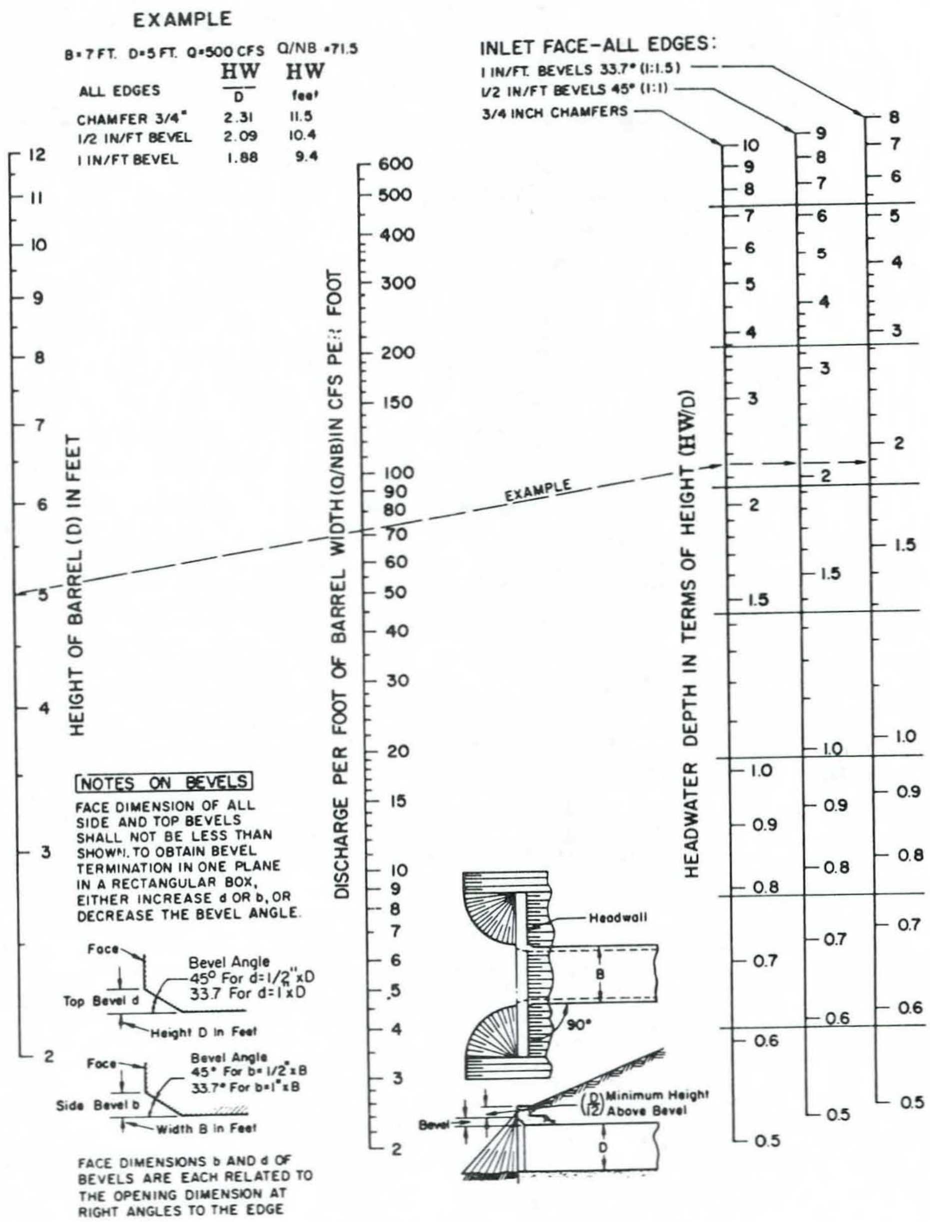


Figure 4.61
Headwater Depth for Inlet Control Rectangular Box Culverts
90° Headwall—Chamfered or Beveled Inlet Edges

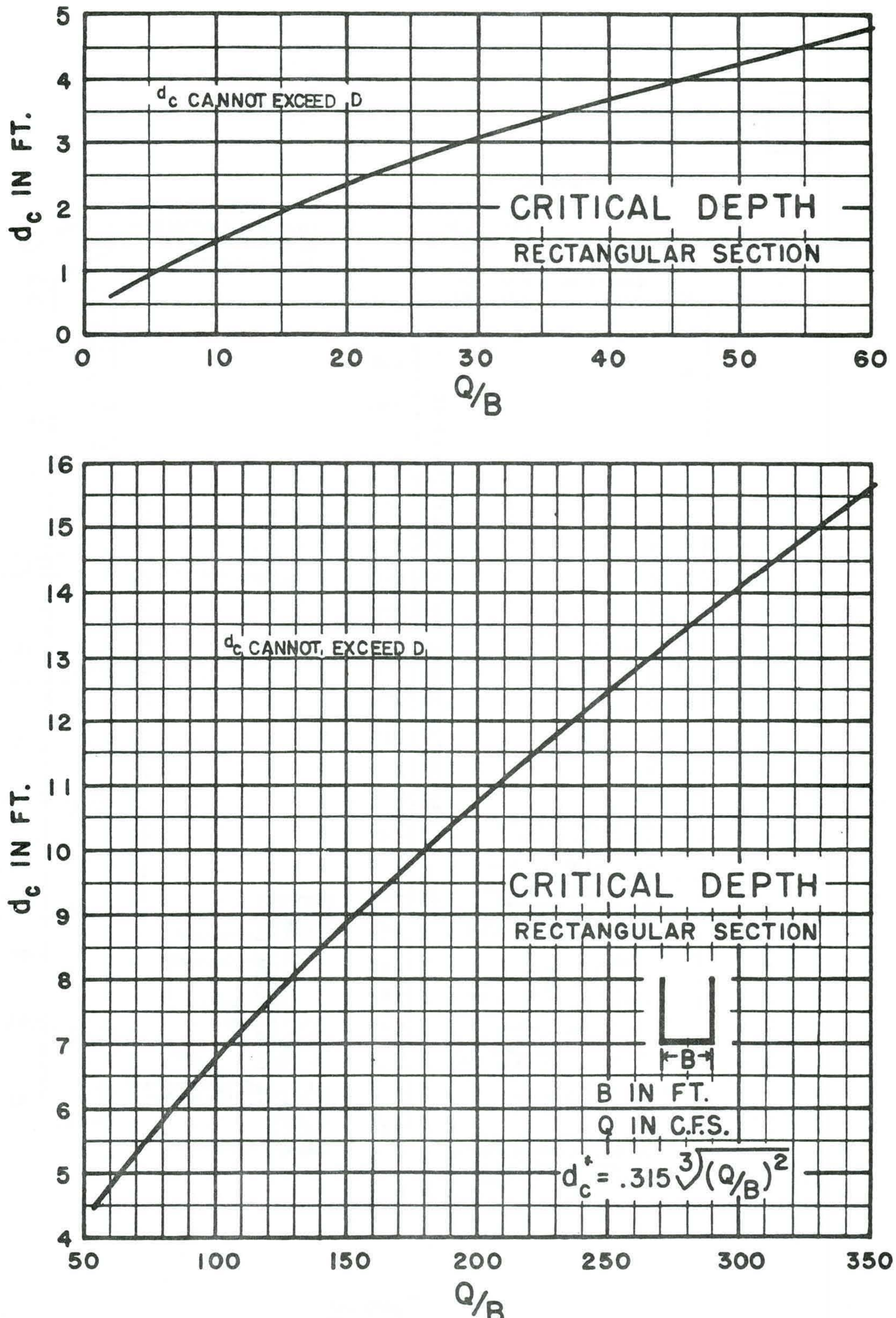


Figure 4.62
Critical Depth Rectangular Section

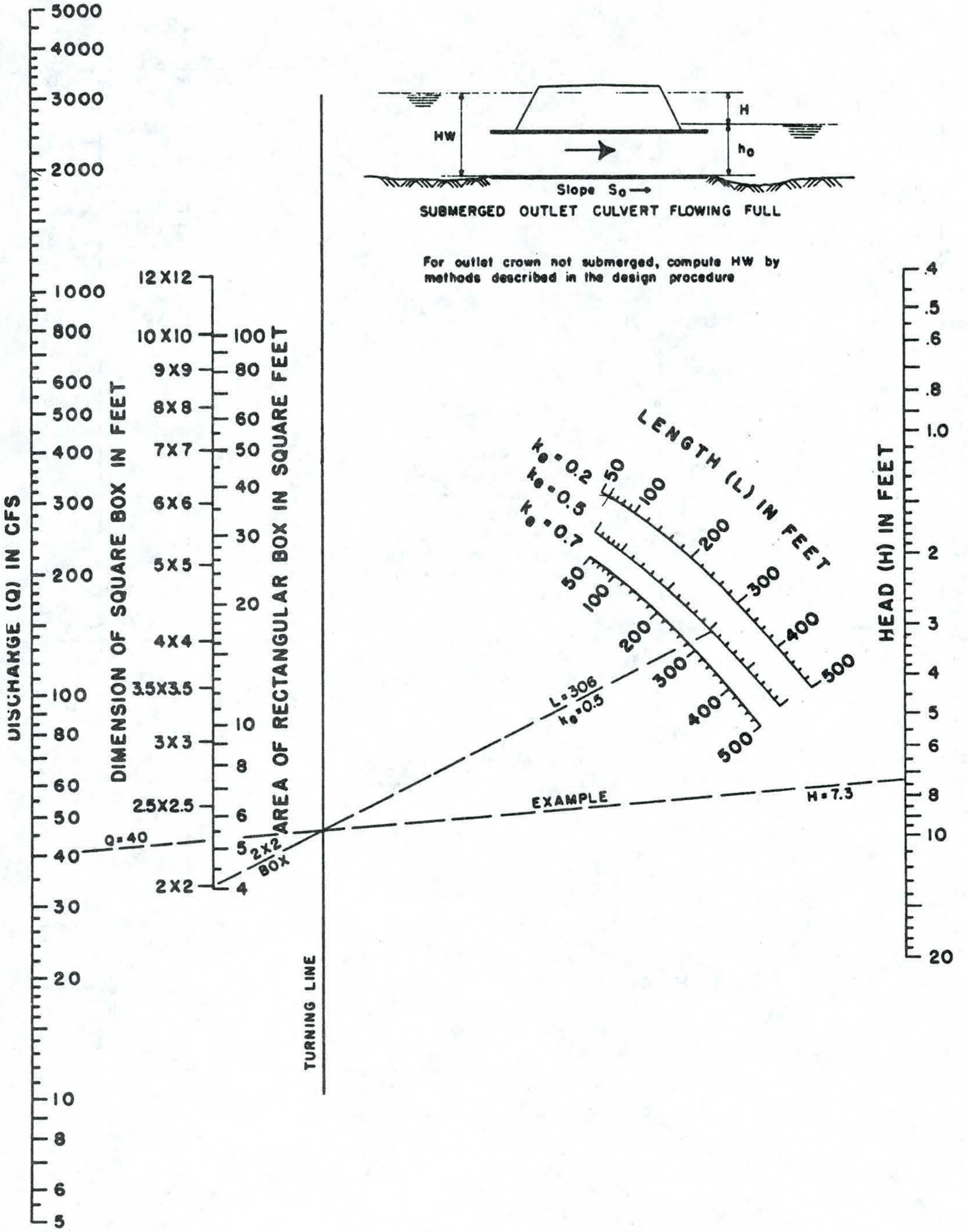


Figure 4.63
Head for Concrete Box Culverts Flowing Full
n = 0.012

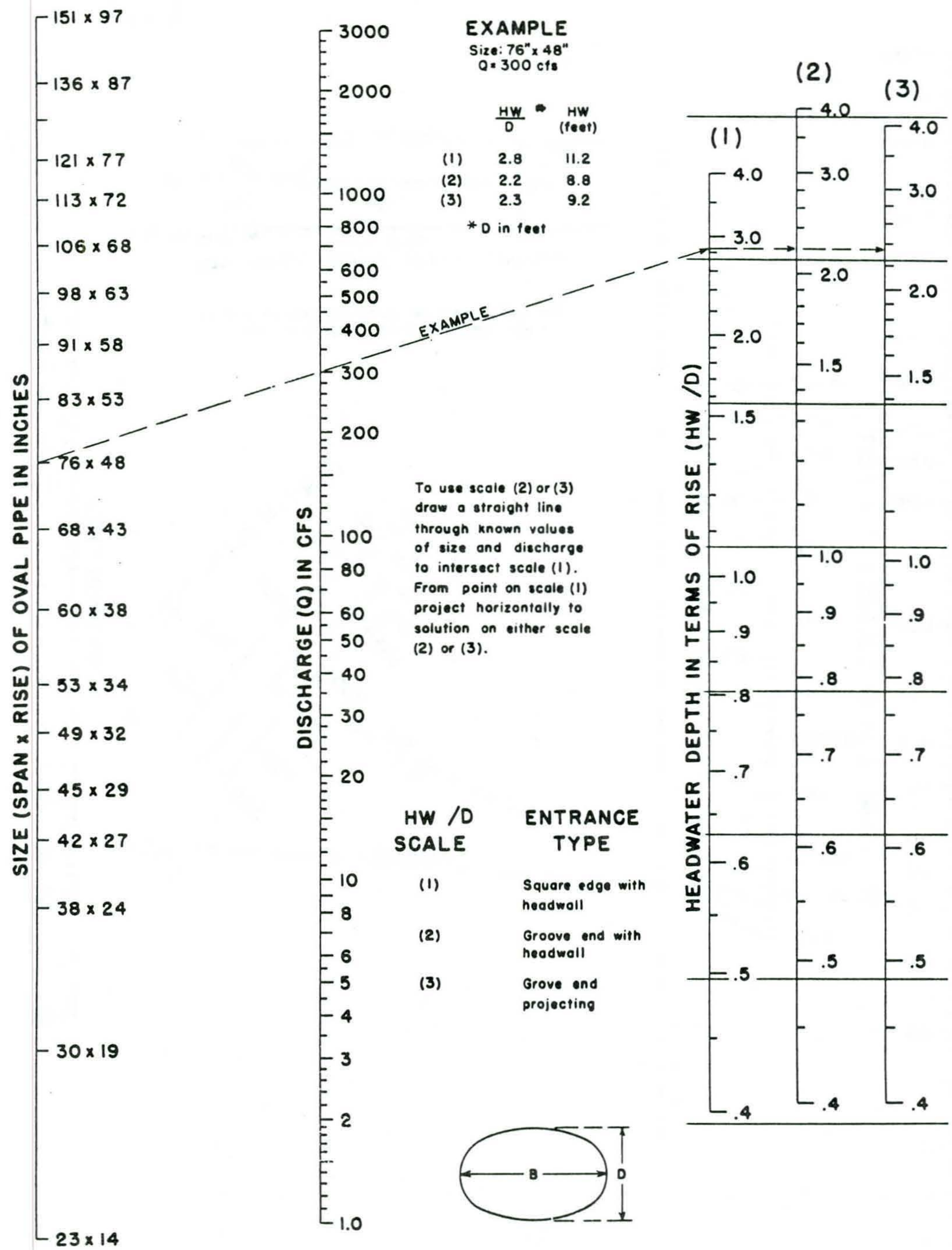


Figure 4.64
Headwater Depth for Oval Concrete Pipe Culverts
Long Axis Horizontal with Inlet Control

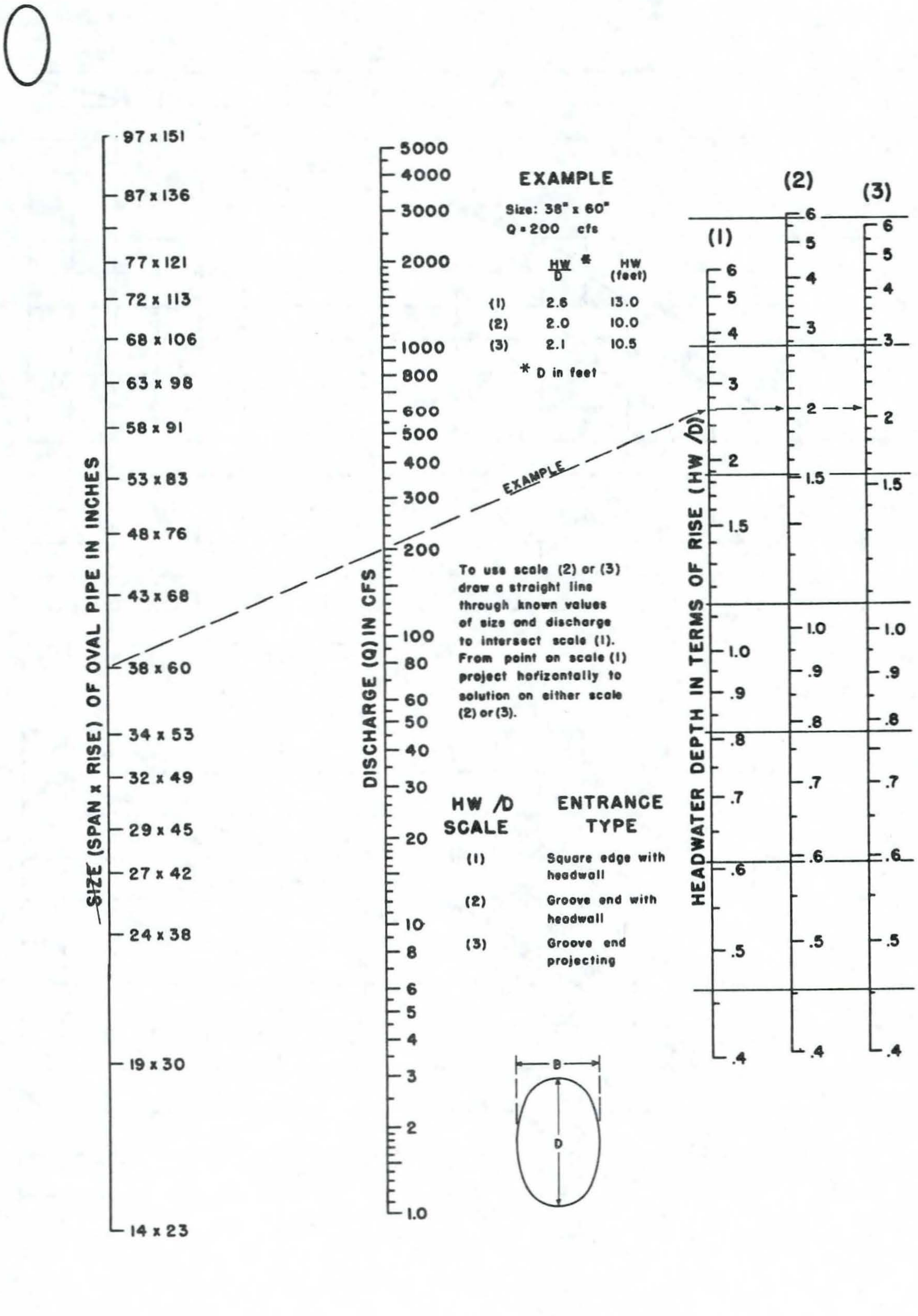


Figure 4.65
Headwater Depth for Oval Concrete Pipe Culverts
Long Axis Vertical with Inlet Control

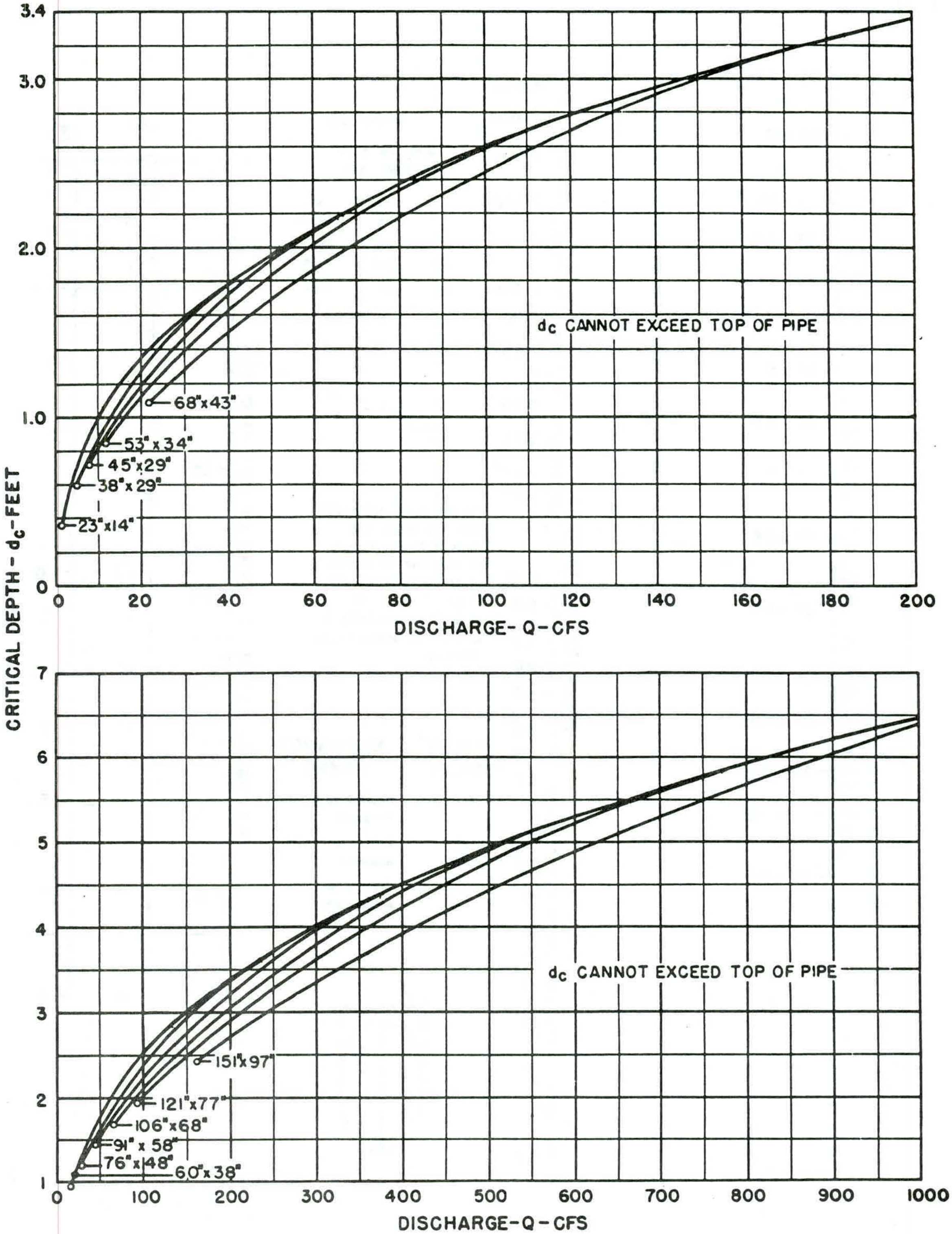


Figure 4.66
Critical Depth for an Oval Concrete Pipe—Long Axis Horizontal

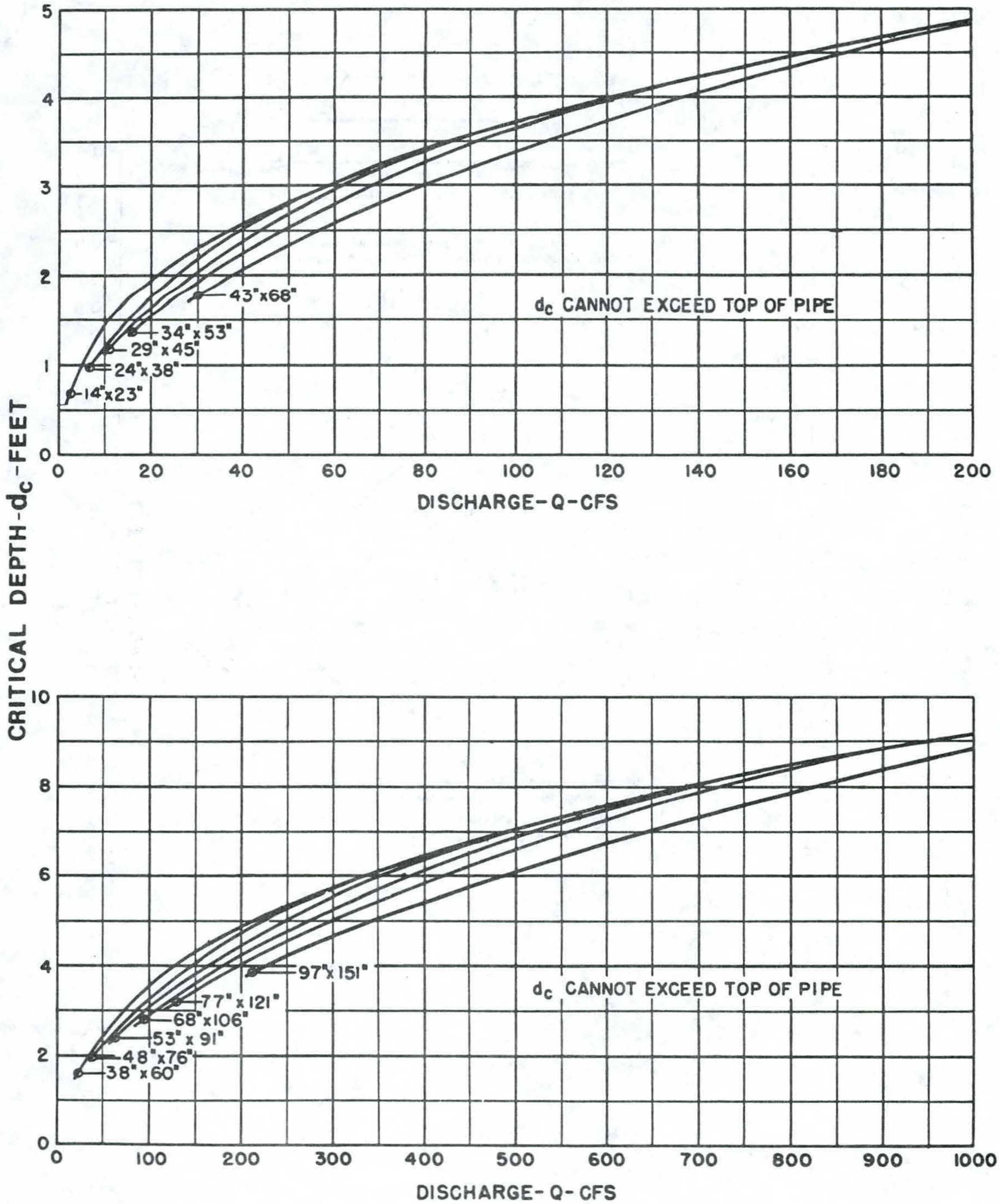


Figure 4.67
Critical Depth for an Oval Concrete Pipe—Long Axis Vertical

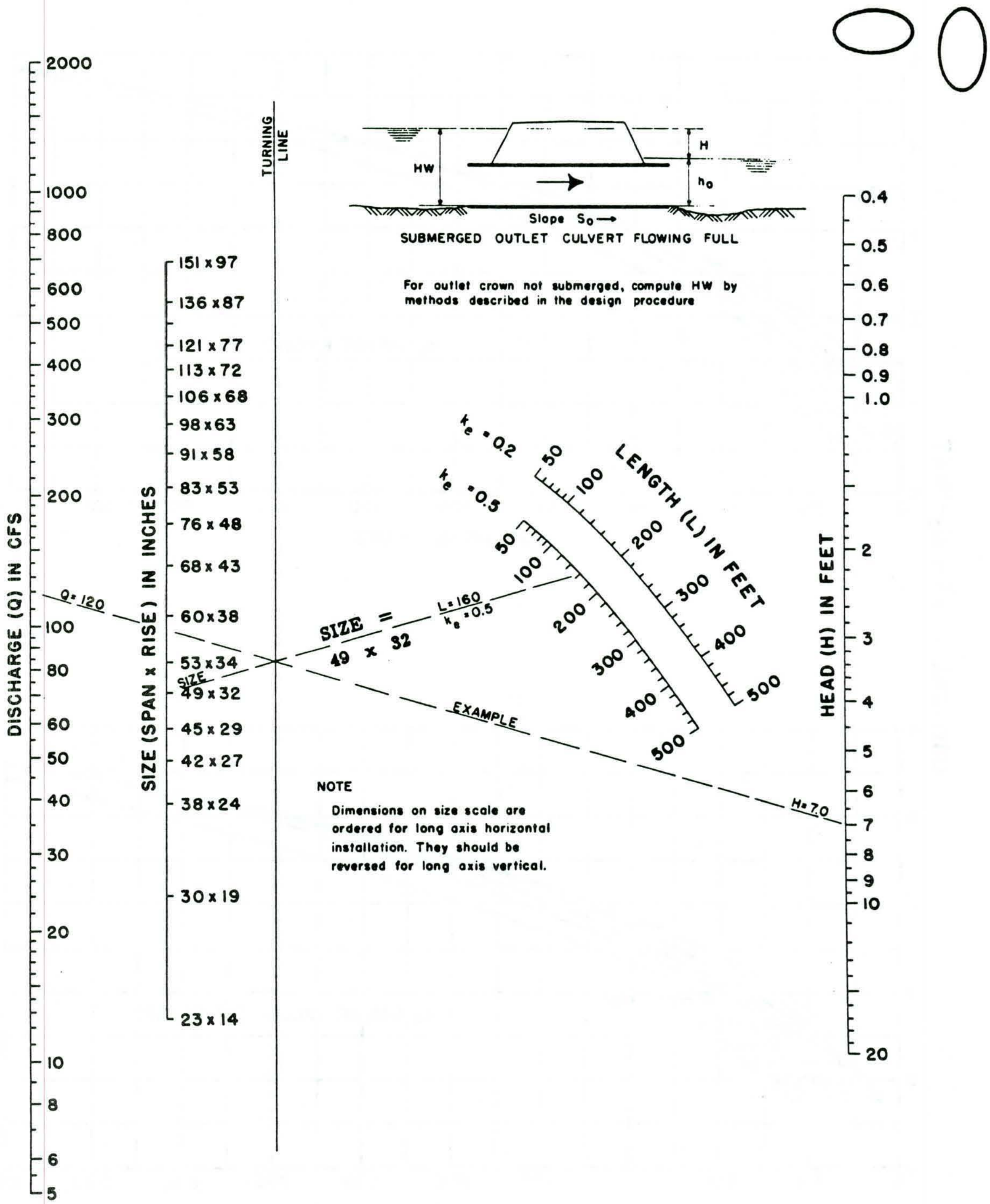


Figure 4.68
Head for Oval Concrete Pipe Culverts—Long Axis Horizontal or Vertical—Flowing Full
n = 0.012

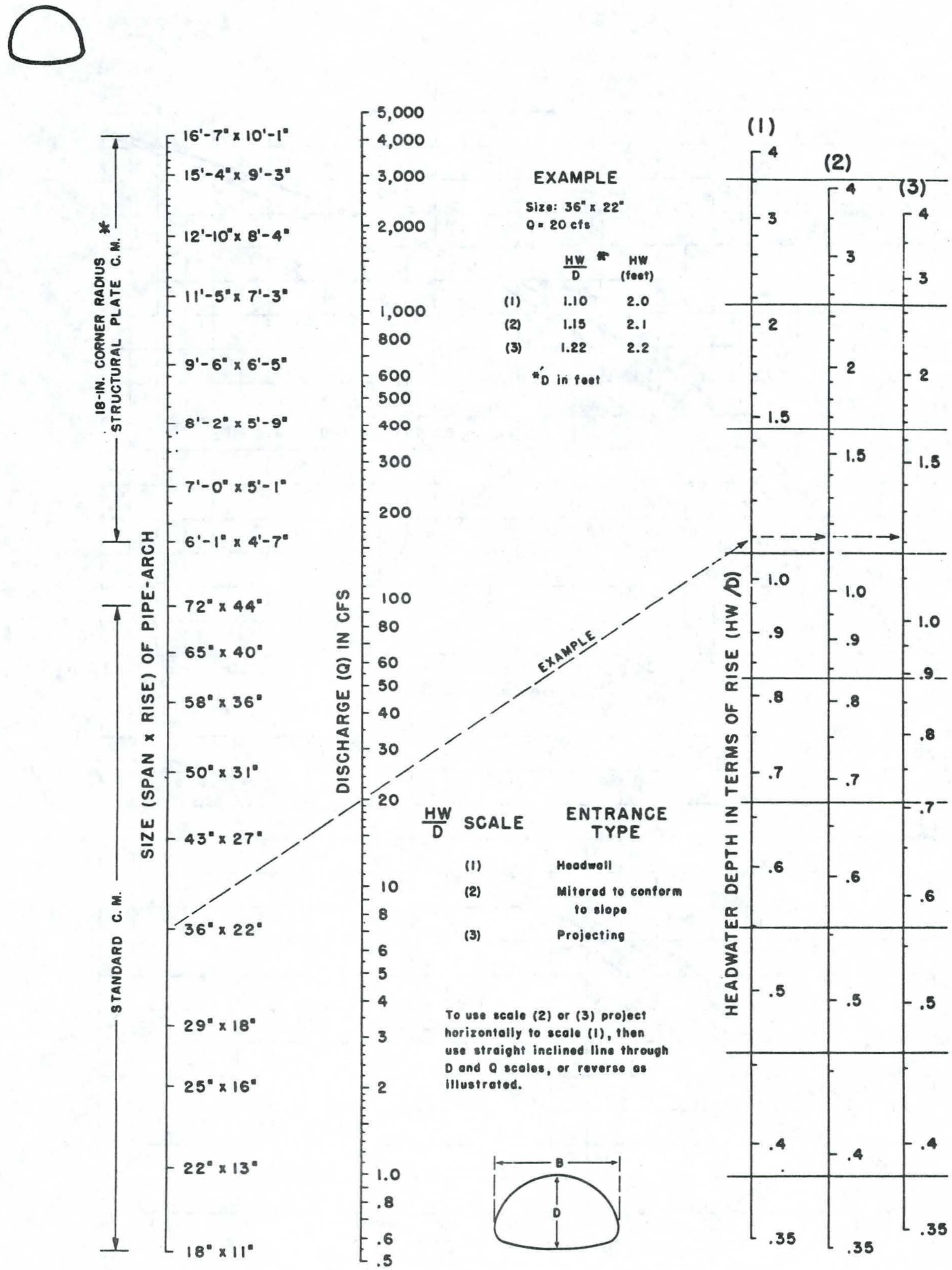


Figure 4.69
Headwater Depth for C.M. Pipe-Arch Culvert with Inlet Control

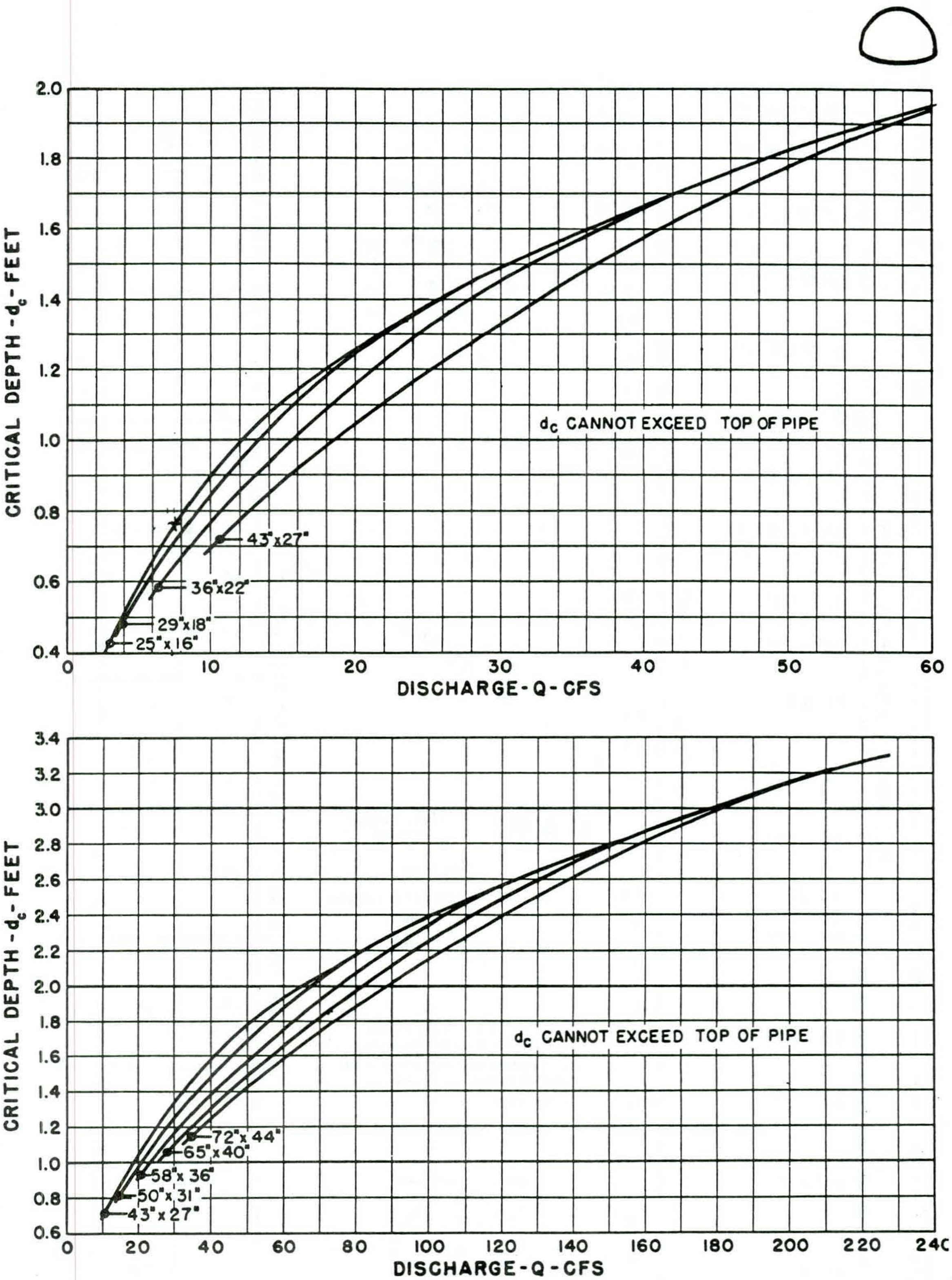


Figure 4.70
Critical Depth for Standard C.M. Pipe-Arch

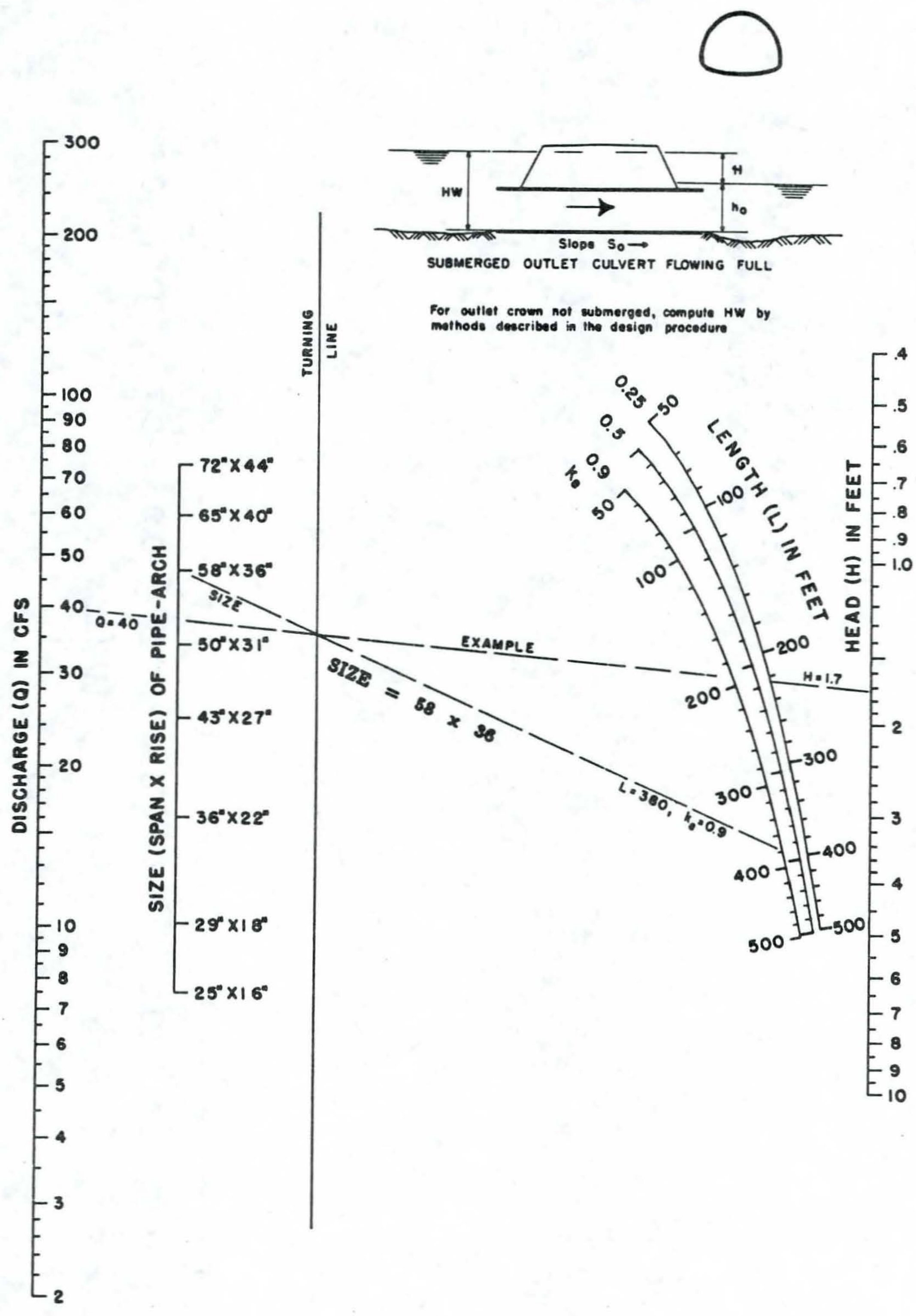


Figure 4.71
Head for Standard C.M. Pipe-Arch Culverts Flowing Full
n = 0.024

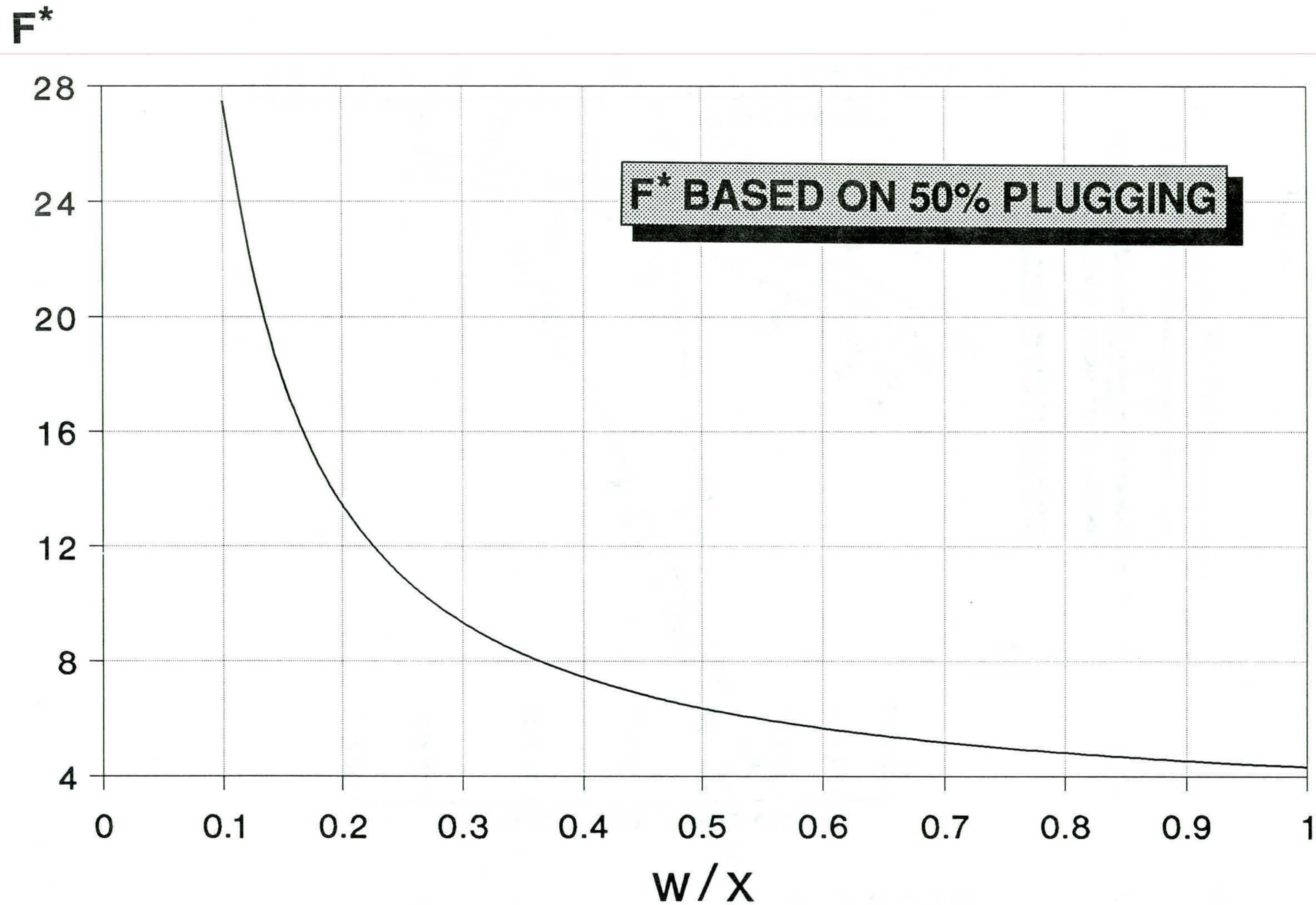


Figure 4.72
Factor for Trash Rack Plugging (50 Percent)

4.3.5 Design Examples

The following example problems illustrate the use of the design methods and charts for selected culvert configurations and hydraulic conditions. The problems cover the following situations:

» Problem No. 1: Circular pipe culvert, standard 2-2/3 by 1/2 inch (6.8 by 1.3 cm) CMP with beveled edge or reinforced concrete pipe with groove end. No FALL.

» Problem No. 2: Reinforced cast-in-place concrete box culvert with square edges and with bevels. No FALL.

» Problem No. 3: Elliptical pipe culvert with groove end and a FALL.

» Problem No. 4: Roadway overtopping calculations and performance curve development.

4.3.5.1 Example Problem No. 1: A culvert at a new roadway crossing must be designed to pass the 25-year flood. Hydrologic analysis indicates a peak flow rate of 200 cfs. Use the following site information:

» Elevation of stream bed at Culvert Face: 100 ft

» Natural Stream Bed Slope: 1 percent = 0.01 ft/ft

» Tailwater for 25-Year Flood: 3.5 ft

» Approximate Culvert Length: 200 ft

» Shoulder Elevation: 110 ft

Design a circular pipe culvert for this site. Consider the use of a corrugated metal pipe with standard 2-2/3 by 1/2 inch corrugations and beveled edges and concrete pipe with a groove end. Base the design headwater on the shoulder elevation with a 2-foot freeboard (elevation 108.0 ft). Set the inlet invert at the natural streambed elevation (no FALL).

Figure 4.73 represents a completed Culvert Design Form for this example.

Note: Figures 4.53, 4.55, 4.56, 4.57, and 4.58, and were used in this example.

PROJECT: EXAMPLE PROBLEM No. 1 CHAPTER III, HDS No. 5	STATION: 1+00 SHEET 1 OF 1	CULVERT DESIGN FORM DESIGNER / DATE: WJJ / 7/18 REVIEWER / DATE: JMN / 7/19

HYDROLOGICAL DATA

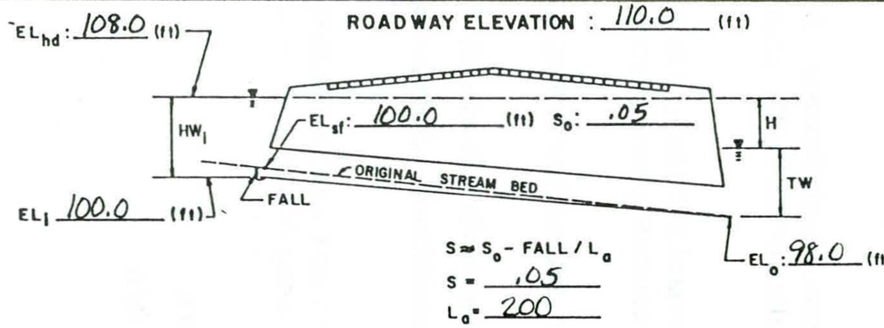
SEE ADD'L SHTS.

- ☐ METHOD: RATIONAL
- ☐ DRAINAGE AREA: 125 AC ☐ STREAM SLOPE: 1.0%
- ☐ CHANNEL SHAPE: TRAPEZOIDAL
- ☐ ROUTING: N/A ☐ OTHER:

DESIGN FLOWS/TAILWATER

R.I. (YEARS)	FLOW (cfs)	TW (ft)
25	200	3.5

ROADWAY ELEVATION: 110.0 (ft)



CULVERT DESCRIPTION: MATERIAL - SHAPE - SIZE - ENTRANCE	TOTAL FLOW Q (cfs)	FLOW PER BARREL Q/N (1)	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
CMP - CIRC. - 72 IN. - BEVEL 45° IN HEADWALL	200	200	0.96	5.8	–	105.8	3.5	3.8	4.9	4.9	0.2	2.6	105.5	105.8	8.6	TRY 60" C.M.P.
C.M.P. " - 60 IN. - " 45°			1.43	7.15	–	107.2		4.1	4.6	4.6		6.3	108.9	108.9	12.0	TRY 60" CONC.
CONC. - " - 60 IN. - GROOVE END			1.36	6.8	–	106.8			4.6	4.6		2.9	105.5	106.8	16.0	TRY 54" CONC.
" - " - 54 IN. - "			1.77	7.97	–	108.0			4.3	4.3		4.7	107.0	108.0	13.5	OK

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

(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:

HIGH OUTLET VELOCITY - OUTLET PROTECTION OR LARGER CONDUIT MAY BE NECESSARY

CULVERT BARREL SELECTED:

SIZE: 54 IN.
SHAPE: CIRCULAR
MATERIAL: CONC. n .012
ENTRANCE: GROOVE END

Figure 4.73
Example Problem No. 1
Culvert Design Sheet

4.3.5.2 Example Problem No. 2: A new culvert at a roadway crossing is required to pass a 50-year flow rate of 300 cfs. Use the following site conditions:

» EL_{hd}: 110 ft based on adjacent structures

» Shoulder Elevation: 113.5 ft

» Elevation of Stream Bed at Culvert Face (EL_{sf}): 100 ft

» Natural Stream Slope: 2 percent

» Tailwater Depth: 4.0 ft

» Approximate Culvert Length: 250 ft

Design a reinforced concrete box culvert for this installation. Try both square edges and 45 degree beveled edges in a headwall. Do not depress the inlet (no FALL).

Figure 4.74 represents a completed Culvert Design form for Problem No. 2.

Note: Figures 4.59, 4.61, 4.62, and 4.63 are used in this solution.

PROJECT: EXAMPLE PROBLEM No. 2 CHAPTER 3, HDS No. 5	STATION: 1+00 SHEET 1 OF 1	CULVERT DESIGN FORM DESIGNER/DATE: WJT / 7/18 REVIEWER/DATE: JMN / 7/19

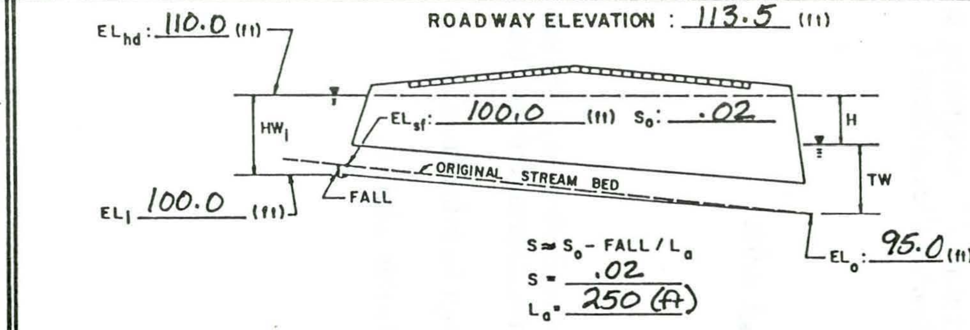
HYDROLOGICAL DATA

SEE ADD'L SHTS.

- ☐ METHOD: SCS
- ☐ DRAINAGE AREA: 200 AC ☐ STREAM SLOPE: 2.0%
- ☐ CHANNEL SHAPE: TRAPEZOIDAL
- ☐ ROUTING: N/A ☐ OTHER:

DESIGN FLOWS/TAILWATER

R.I. (YEARS)	FLOW (cfs)	TW (ft)
50	300	4.0



CULVERT DESCRIPTION: MATERIAL - SHAPE - SIZE - ENTRANCE	TOTAL FLOW Q (cfs)	FLOW PER BARREL Q/N (1)	INLET CONTROL HW_i/D (2)	INLET CONTROL HW_i	INLET CONTROL FALL (3)	INLET CONTROL EL_{hi} (4)	OUTLET CONTROL TW (5)	OUTLET CONTROL d_c	OUTLET CONTROL $\frac{d_c+D}{2}$	OUTLET CONTROL h_o (6)	OUTLET CONTROL k_e	OUTLET CONTROL H (7)	OUTLET CONTROL EL_{ho} (8)	CONTROL HEADWATER ELEVATION	OUTLET VELOCITY	COMMENTS
CONCRETE - BOX - 6'x5' - SQ. EDGE	300	50	1.57	7.9	—	107.9	4.0	4.2	4.6	4.6	0.5	3.55	103.2	107.9	21.7	OK TRY SM. BOX
" " - 5'x5' - "	300	60	1.91	9.6	—	109.6	4.0	4.3	4.9	4.9	0.5	5.2	105.1	109.6	20.8	CHECK BEVELS
" " - 5'x5' - 45° BEVEL	300	60	1.71	8.55	—	108.6	4.0	4.3	4.9	4.9	0.2	4.6	104.5	108.6	20.8	OK

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:

5'x5' BOX WILL WORK WITH OR WITHOUT BEVELS. BEVELS PROVIDE ADDITIONAL FLOW CAPACITY.

CULVERT BARREL SELECTED:

SIZE: 5 ft. x 5 ft.
SHAPE: SQUARE
MATERIAL: CONC. n .012
ENTRANCE: 45° BEVEL - 90° HEADWALL

Figure 4.74
Example Problem No. 2
Culvert Design Form

4.3.5.3 Example Problem No. 3: Design a culvert to pass a 25-year flow of 180 cfs. Minimum depth of cover for this culvert is 2 feet.

» EL_{hd}: 105 ft based on adjacent structures

» Shoulder Elevation: 105.5 ft

» Elevation of Stream Bed at Culvert Face (EL_{sf}): 100 ft.

» Original Stream Slope: 5 percent

» Tailwater Depth: 4 ft

» Approximate Culvert Length: 150 ft

Due to the low available cover over the conduit, use a horizontal elliptical concrete pipe. Use of a small depression (FALL) of about 1 ft at the inlet is acceptable.

Refer to Figure 4.75 for a completed Culvert Design Form for this problem.

Note: Figures 4.64, 4.66, and 4.68 are used in this solution.

PROJECT: EXAMPLE PROBLEM No. 3 CHAPTER III, HDS No. 5

STATION: 2+00

SHEET 1 OF 1

CULVERT DESIGN FORM

DESIGNER / DATE: WJJ / 7/18

REVIEWER / DATE: JMN / 7/19

HYDROLOGICAL DATA

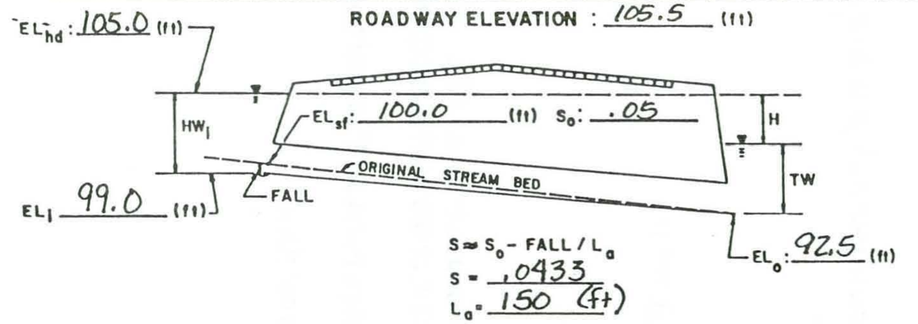
SEE ADD'L SHTS.

- ☐ METHOD: RATIONAL
- ☐ DRAINAGE AREA: 110 AC ☐ STREAM SLOPE: 5.0%
- ☐ CHANNEL SHAPE: Semi-Circular
- ☑ ROUTING: 40 CFS RED. FROM PEAK ☐ OTHER: DEPTH OF COVER 2' MIN.

DESIGN FLOWS/TAILWATER

R.I. (YEARS)	FLOW (cfs)	TW (ft)
25	180*	4.0

* APPROX. ROUTED FLOW RATE



CULVERT DESCRIPTION: MATERIAL - SHAPE - SIZE - ENTRANCE	TOTAL FLOW Q (cfs)	FLOW PER BARREL Q/N (1)	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
CONC. - HORIZ ELLIPSE - 68"x43" - SQ. EDGE	180	–	1.98	7.1	–	107.1	4.0	3.2	3.4	4.0	0.5	3.7	100.2	107.1	–	TRY GROOVE END W/HEADWALL
" - " - " x " - GROOVE END	180	–	1.63	5.8	–	105.8	4.0	3.2	3.4	4.0	0.2	3.2	99.7	105.8	–	EL_{hi} TOO HIGH USE FALL
" - " - " x " - "	180	–	1.63	5.8	1.0	104.8	4.0	3.2	3.4	4.0	0.2	3.2	99.7	104.8	14.9	OK- DEPTH OF
																COVER OK ALSO

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:

HIGH OUTLET VELOCITY – CHECK STREAM BED STABILITY

CULVERT BARREL SELECTED:

SIZE: 68" x 43"

SHAPE: HORIZONTAL ELLIPSE

MATERIAL: CONC. n .012

ENTRANCE: GROOVE END-1'FALL

Figure 4.75
Example Problem No. 3
Culvert Design Form

4.3.5.4 Example Problem No. 4 Develop a performance curve for the installation in Figure 4.76, below, including roadway overtopping up to 0.5 feet above the roadway. Use the following dimensions:

Tailwater Channel:	
Flow, cfs	**TW, ft**
50	101.8
100	102.6
150	103.1
200	103.5
250	103.8
300	104.2
350	104.4

Figure 4.77 represents a completed Culvert Design Form for this problem. Figure 4.78 provides the performance curve and roadway overtopping computations.

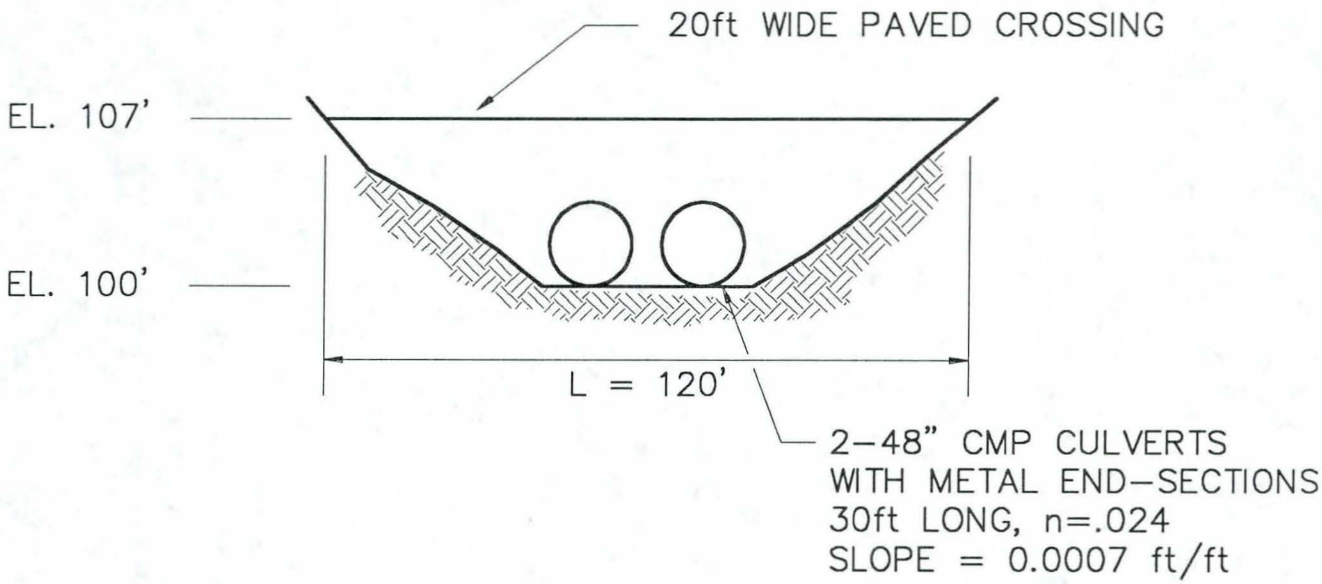


Figure 4.76
Example Problem No. 4
Roadway Overtopping and Performance Curve Development

PROJECT : Example Roadway Overtopping – Performance Curve	STATION : ______ SHEET 1 OF ______	CULVERT DESIGN FORM DESIGNER / DATE : RJF / 1-13-?? REVIEWER / DATE : ______ / ______

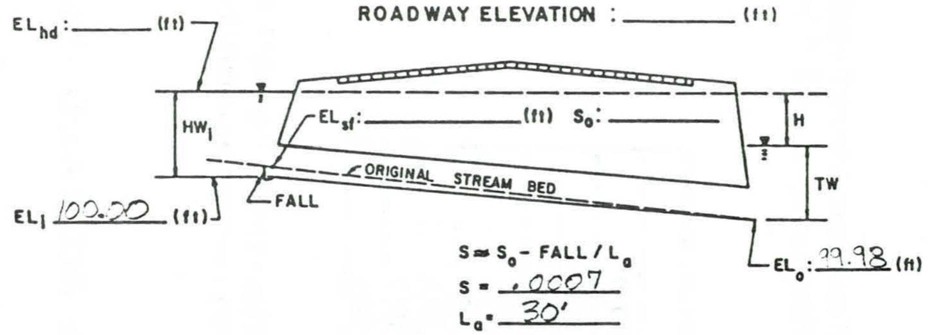
HYDROLOGICAL DATA

SEE ADD'L. SHTS.

- ☐ METHOD: ______
- ☐ DRAINAGE AREA: ______ ☐ STREAM SLOPE: .0008
- ☐ CHANNEL SHAPE: Trapezoid
- ☐ ROUTING: ______ : ☐ OTHER: ______

DESIGN FLOWS/TAILWATER

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



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
CMP – Circular – 48" – end sect	50	25	.52	2.08	—	102.08	1.8	1.5	2.75	2.75	0.5	0.22	103.0			
	100	50	.78	3.12	—	103.12	2.6	2.1	3.05	3.05	↓	0.60	103.6			
	150	75	1.03	4.12	—	104.12	3.1	2.6	3.30	3.30	↓	1.35	104.6			
	200	100	1.30	5.20	—	105.20	3.5	3.0	3.50	3.50	↓	2.40	105.9			
	250	125	1.63	6.52	—	106.52	3.8	3.4	3.70	3.80	↓	3.75	107.5			

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

(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:

Used scale (1) of Chart 2 for inlet control comps.
used chart 6 for outlet control head.

CULVERT BARREL SELECTED:

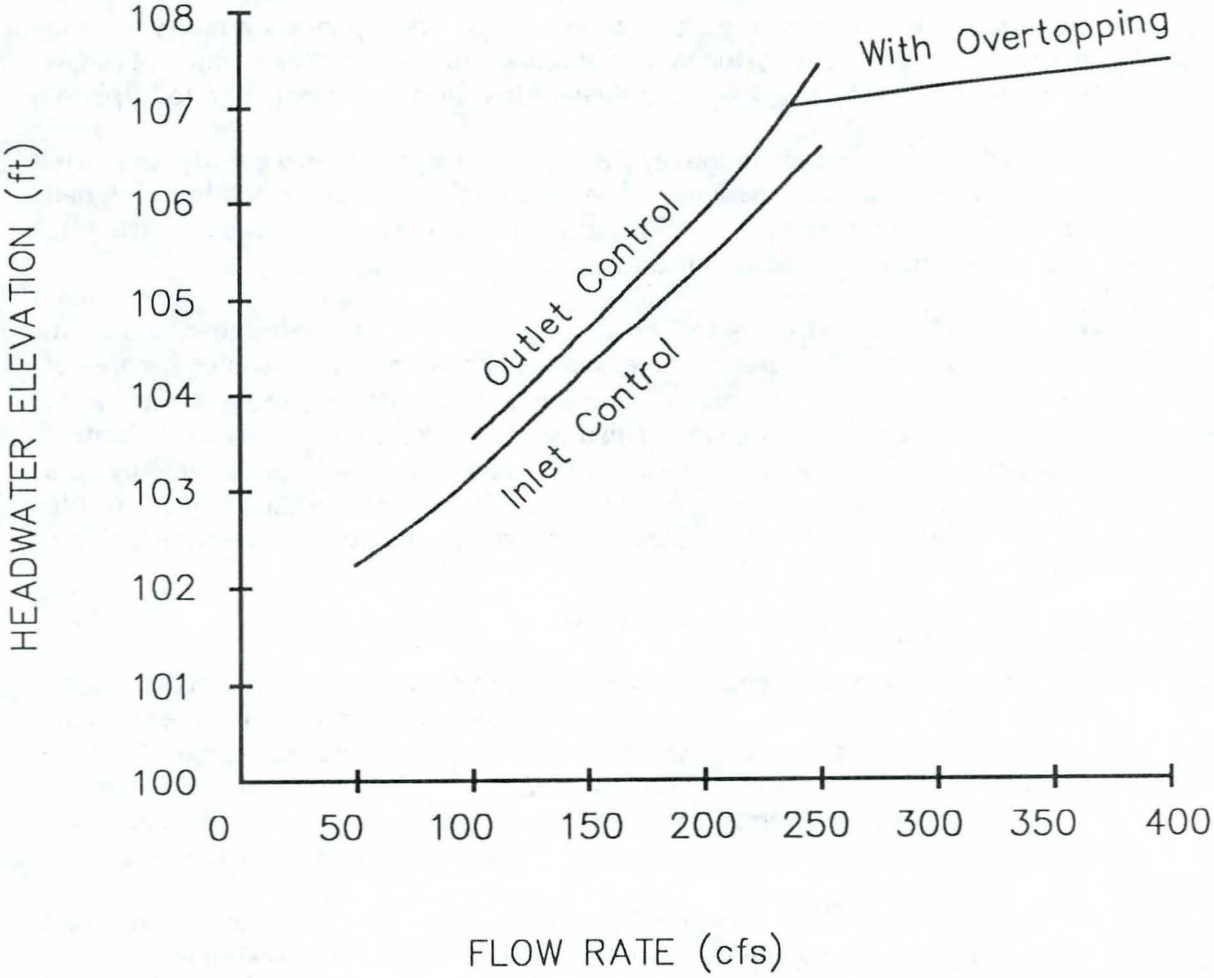
SIZE: ______

SHAPE: ______

MATERIAL: ______ n ______

ENTRANCE: ______

Figure 4.77
Example Problem No. 4
Culvert Design Form



$$Q_o = K_t C_r L_s (H W_r)^{1.5}$$

HW_r	C_r	K_t	L_s	Q_o	Q_{pipe}		Q_{total}
0.25	2.98	1	120	44.7cfs	+244	=	289
0.50	3.02	1	120	128.1cfs	+250	=	378

Figure 4.78
Performance Curve and Roadway Overtopping Computations
(Example Problem No. 4)

4.4 Inverted Siphons

4.4.1 General

Because of the resulting physical conditions, inverted siphons are rarely used in urban drainage; however, due to the flat topography and a large number of canals in Maricopa County, 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 permitted by the jurisdictional agency.

4.4.2 Design Criteria

All pipe should be designed for water-tight joints. Velocity in the conduit should be a minimum of 5.0 fps. The minimum cover over the conduit should exceed 3.0 feet. Inlet and outlet structures are required, and the facility shall meet the requirements for safety described in Section 4.3.2.7. 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.

4.4.3 Design Procedure

A design procedure and design examples are 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 Maricopa County.

4.5 Entrances and Outlets for Conduits

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 Section 3.3 and are not addressed here.

4.5.1 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 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. A drop structure may be located upstream or downstream from the closed conduit and should be incorporated into the design. 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, detention or retention basin drains into a storm drain system, culvert type inlets are frequently used. The storm drain hydraulic grade line must be considered when estimating the inlet capacity for culvert type inlets. The storm drain 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.

4.5.2 Special Criteria for Closed Conduits

4.5.2.1 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 should be protected by paving, grouted riprap, or other means of permanent stabilization.

4.5.2.2 Entrance Structures and Transitions: Criteria for culvert entrances are contained in Section 4.3.2. The same criteria apply to culvert type entrances for storm drains. 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 Section 4.3.

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* (FHWA 1983).

4.5.2.3 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 to ensure 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. Due to the nature of 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.

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 shall require 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. Design procedures for riprap aprons are in Section 4.5.3.2.

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 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. Outlet discharge must be transitioned to limit the velocity to the allowable. Allowable velocities for several channel lining materials are shown in Tables 5.1 and 5.2 (pages 212 and 213).

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 must be provided with a riprap, concrete, or other suitable apron to transition the velocity 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 6.

Cutoff Walls: A cutoff wall placed at the culvert outlet in a natural stream provides adequate protection downstream when the scour will not be excessive, or where the development of a scour hole will not undermine nearby structures so that it is practical to allow localized scour.

The following criteria applicable to cutoff walls is based on the computed scour hole geometry. The procedure for determining the scour hole geometry is presented in Section 4.5.3.1.

1. The depth of the cutoff wall shall be equal to the maximum depth of scour.
2. The width of the cutoff wall shall be a minimum of one-third the maximum scour width.
3. The depth of the cutoff wall should not normally exceed six feet. Where a deeper wall is necessary to meet the above criteria, either another form of protection should be employed or an analysis will be required to substantiate the walls structural stability.

4.5.2.4 Safety: Inlets and outlets to closed conduits may present dangers to the public when access is not controlled. Refer to Section 4.3.2.7 for the safety requirements related to conduit inlets and outlets.

4.5.3 Design Procedures

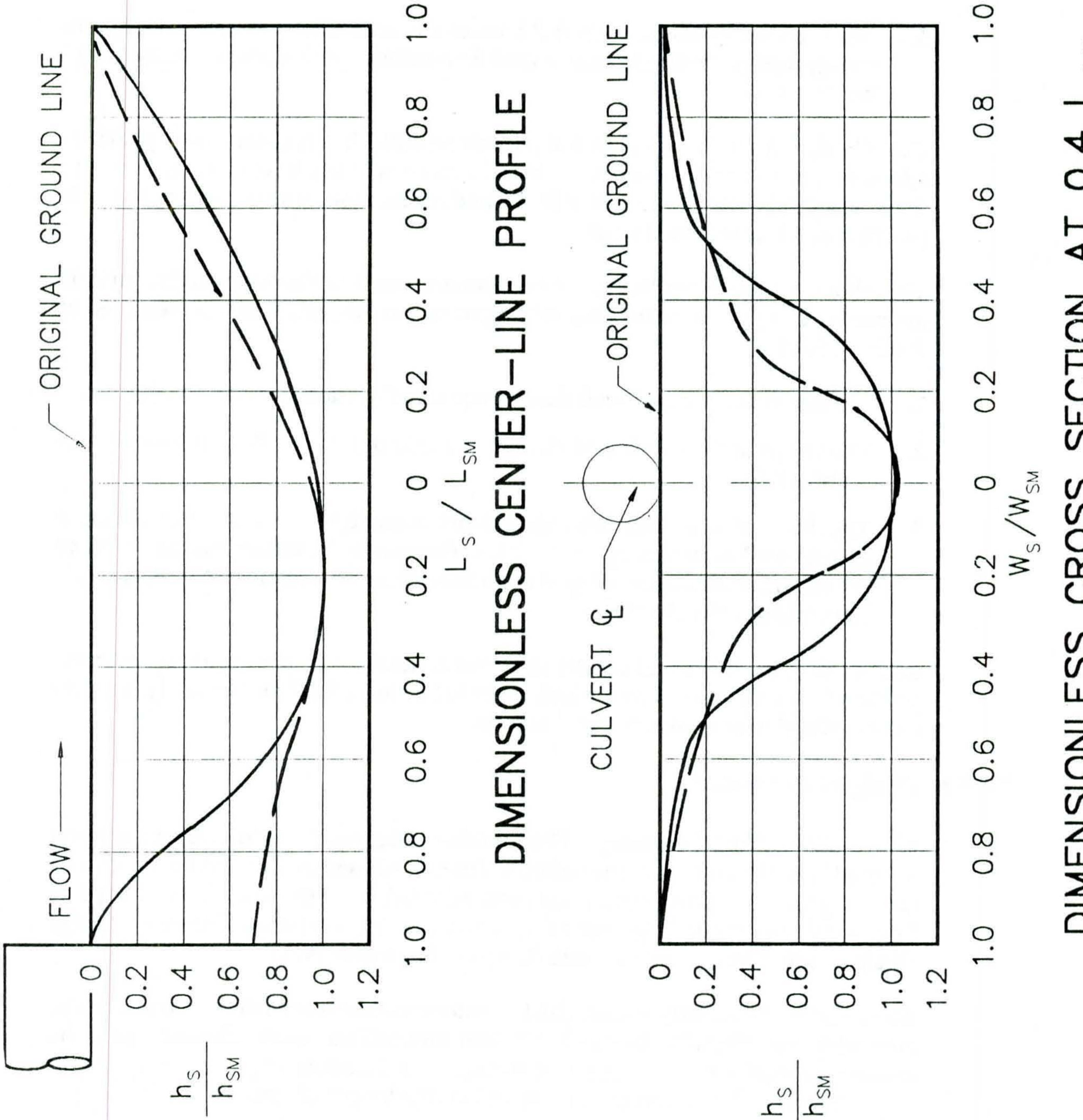
4.5.3.1 Scour Hole Geometry: The objective of this section is to present a method for predicting local scour at the outlet of structures based on soil and flow data and culvert geometry. This section has been adapted from the U.S. Army Corps of Engineers' Hydraulic Engineering Circular No. 14, *Hydraulic Design of Energy Dissipators for Culverts and Channels* (Corps of Engineers 1975).

The scour hole geometry varies with tailwater conditions with the maximum scour geometry occurring at tailwater depths less than half the culvert diameter with the maximum depth of scour (h_s) occurring at a location approximately 0.4 L_s downstream of the culvert outlet, where L_s is the length of scour.

Empirical equations defining the relationship between the culvert discharge intensity, time, and the length, width, depth, and volume of scour hole are presented for the maximum or extreme scour case. The dimensionless scour hole geometry is shown in Figure 4.79.

Cohesionless Material: The general expression for determining scour geometry in a cohesionless soil for a circular pipe flowing full is:

$$\text{Dimensionless Scour Geometry} = \alpha \left(Q/g^{1/2}D^{5/2}\right)^{\beta} \left(t/t_o\right)^{\theta} \qquad \textbf{(4.14)}$$



LEGEND

h_S = Depth of scour
h_{SM} = Maximum depth of scour
W_S = Width of scour
W_{SM} = Maximum width of scour

L_S = Length of scour
L_{SM} = Maximum length of scour
———— Maximum Tailwater
— — — Minimum Tailwater

Figure 4.79
Dimensionless Scour Hole Geometry

For noncircular or partly full culverts, the diameter D can be replaced by an equivalent depth, y_e:

$$y_e = (A/2)^{1/2} \quad \textbf{(4.15)}$$

A is the cross sectional area of flow. Modifying Equation 4.14 to include the equivalent depth results in the general expression:

$$\text{Dimensionless Scour Geometry} = \alpha_e \left(Q/g^{1/2} y_e^{5/2}\right)^{\beta} (t/t_o)^{\theta} \quad \textbf{(4.16)}$$

where:

α_e = $0.63^{2.5\beta-1}$ for hs, W_s, and L_s

α_e = $\alpha\, 0.63^{2.5\beta-3}$ for Vs

The values of the coefficients α, α_e, β, and θ in Equations 4.14 and 4.15 are given in Table 4.7.

Gradation: The cohesionless bed materials presented in Table 4.7 are categorized as either uniform (U) or graded (G). The grain size distribution is determined by performing a sieve analysis (ASTM DA22-63). The standard deviation (σ) is computed as:

$$\sigma = (d_{84}/d_{16})^{1/2} \quad \textbf{(4.17)}$$

where the values of d_{84} and d_{16} are extracted from the grain size distribution. If $\sigma \leq 1.5$, the material is considered to be uniform; if $\sigma > 1.5$, the material is classified as graded.

Cohesive Soils: If the cohesive soil is a sandy clay similar to the one tested at Colorado State University by Abt, et al, Equations 4.14 or 4.15 and the appropriate coefficients in Table 4.7 can be used to estimate the scour hole dimensions. The sandy clay tested had 58 percent sand, 27 percent clay, 15 percent silt, and 1 percent organic matter; had a mean grain size of 0.15 mm, and had a plasticity index (PI) of 15.

Since Equations 4.14 and 4.16 do not include soil characteristics, they can only be used for soils similar to the ones tested. Shear number expressions, that related scour to the critical shear stress of the soil, were derived to have a wider range of applicability for cohesive soils besides the one specific sandy clay that was tested. The shear number expressions for circular culverts are:

$$h_s/D,\ W_s/D,\ L_s/D, \text{ or } V_s/D = \alpha \left(\rho V^2/\tau_c\right)^{\beta} (t/t_o)^{\theta} \quad \textbf{(4.18)}$$

and for other shaped culverts:

$$h_s/y_e,\ W_s/y_e,\ L_s/y_e, \text{ or } V_s/y_e = \alpha_e \left(\rho V^2/\tau_c\right)^{\beta} (t/t_o)^{\theta} \quad \textbf{(4.19)}$$

where:

α_e = $\alpha/(0.63)$ for h_s, W_s, and L_s

α_e = $\alpha/(0.63)^3$ for V_s

The values of the coefficients α, β, θ, and α_e in Equations 4.18 and 4.19 are presented in Table 4.7. The critical tractive shear stress is defined as:

$$\tau_c = 0.001\,(S_v + 180)\tan(30 + 1.73\,\mathrm{PI}) \qquad \textbf{(4.20)}$$

where S_v is the saturated shear strength in pounds per square inch and PI is the Plasticity Index from the Atterberg limits.

It is recommended that Equations 4.18 and 4.19 be limited to sandy clay soils with a plasticity index of 5 to 16.

Time of Scour: The time of scour is estimated based upon a knowledge of peak flow duration. Lacking this knowledge, it is recommended that a time of 30 minutes be used in Equations 4.14, 4.16, 4.18, and 4.19. The tests indicate that approximately two-thirds to three-fourths of the maximum scour occurs in the first 30 minutes of the flow duration.

It should be noted that the exponents for the time parameter in Table 4.7 reflect the relatively flat part of the scour-time relationship and are not applicable for the first 30 minutes of the scour process.

Headwalls: Installation of headwalls flush with the culvert outlet moves the scour hole downstream. However, the magnitude of the scour geometries remain essentially the same as for the case without the headwall. The headwall should extend to a depth equal to the maximum depth of scour.

Summary: The prediction equations presented in this section are intended to serve along with field reconnaissance as guidance for determining the need for energy dissipators at culvert outlets. Remember that the equations assume that grade control exists whether it be manmade or natural, and do not include long-term channel degradation of the downstream channel. The equations are based on tests which were conducted to determine maximum scour for the given condition and therefore represent what might be termed worst case scour geometries. The procedure presented is from *Hydraulic Design of Energy Dissipators for Culverts and Channels* (FHWA 1983).

Design Procedure:

1. Perform a hydrologic analysis of the drainage in which the culvert is located or is to be placed. Estimate the magnitude and duration of the peak discharge. Express the discharge in cfs and the duration in minutes.

   The discharge intensity is

   D.I. $= Q/(g^{1/2} D^{5/2})$ for circular culverts flowing full

   D.I. $= Q/(g^{1/2} y_e^{5/2})$ for other shapes

   where $y_e = (A/2)^{1/2}$.

Table 4.7
Experimental Coefficients for Culvert Outlet Scour

Material	Nominal Grain Size d_{50}, mm	Scour Equation (below)	Depth, h_s				Width, W_s				Length, L_s				Volume, V_s			
			α	β	θ	α_e	α	β	θ	α_e	α	β	θ	α_e	α	β	θ	α_e
Uniform Sand	0.20	V-1 or V-2	2.72	0.375	0.10	2.79	11.73	0.92	0.15	6.44	16.82	0.71	0.125	11.75	203.36	2.0	0.375	80.71
Uniform Sand	2.0	V-1 or V-2	1.86	0.45	0.09	1.76	8.44	0.57	0.06	6.94	18.28	0.51	0.17	16.10	101.48	1.41	0.34	79.62
Graded Sand	2.0	V-1 or V-2	1.22	0.85	0.07	0.75	7.25	0.76	0.06	4.78	12.77	0.41	0.04	12.62	36.17	2.09	0.19	12.94
Uniform Gravel	8.0	V-1 or V-2	1.78	0.45	0.04	1.68	9.13	0.62	0.08	7.08	14.36	0.95	0.12	7.61	65.91	1.86	0.19	12.15
Graded Gravel	8.0	V-1 or V-2	1.49	0.50	0.03	1.33	8.76	0.89	0.10	4.97	13.09	0.62	0.07	10.15	42.31	2.28	0.17	32.82
Cohesive Sandy Clay 60% Sand PI 15	0.15	V-1 or V-2	1.86	0.57	0.10	1.53	8.63	0.35	0.07	9.14	15.30	0.43	0.09	14.78	79.73	1.42	0.23	61.84
Clay PI 5-16	Various	V-3 or V-4	0.86	0.18	0.10	1.37	3.55	0.17	0.07	5.63	2.82	0.33	0.09	4.48	0.62	0.93	0.23	2.48

Equations

V-1. **For Circular Culverts.** Cohesionless material or the 0.15mm cohesive sandy clay:

$$\left[\frac{h_s}{D}, \frac{W_s}{D}, \frac{L_s}{D}, \text{or } \frac{V_s}{D^3}\right] = \alpha\left(\frac{Q}{\sqrt{g}\,D^{5/2}}\right)^{\beta}\left(\frac{t}{t_o}\right)^{\theta}$$

where t_o = 316 min.

V-2. **For Other Culvert Shapes.** Same material as above:

$$\left[\frac{h_s}{y_e}, \frac{W_s}{y_e}, \frac{L_s}{y_e}, \text{or } \frac{V_s}{y_e^3}\right] = \alpha_e\left(\frac{Q}{\sqrt{g}\,y_e^{5/2}}\right)^{\beta}\left(\frac{t}{t_o}\right)^{\theta}$$

where t_o = 316 min.

V-3. **For Circular Culverts.** Cohesive sandy clay with PI = 5-16:

$$\left[\frac{h_s}{D}, \frac{W_s}{D}, \frac{L_s}{D}, \text{or } \frac{V_s}{D^3}\right] = \alpha\left(\frac{\rho V^2}{\tau_c}\right)^{\beta}\left(\frac{t}{t_o}\right)^{\theta}$$

where t_o = 316 min.

V-4. **For Other Culvert Shapes.** Cohesive sandy clay with PI = 5-16:

$$\left[\frac{h_s}{y_e}, \frac{W_s}{y_e}, \frac{L_s}{y_e}, \text{or } \frac{V_s}{y_e^3}\right] = \alpha_e\left(\frac{\rho V^2}{\tau_c}\right)^{\beta}\left(\frac{t}{t_o}\right)^{\theta}$$

where t_o = 316 min.

For Cohesionless Materials, or 0.15mm Sandy Clay:

2. Compute the discharge intensity when the culvert is flowing at the peak discharge.

3. Determine scour coefficients from Table 4.7.

4. Compute the scour hole dimensions from

$$h_s/D, W_s/D, L_s/D, \text{ or } V_s/D^3 = \alpha(Q/g^{1/2} D^{5/2})^{\beta} (t/316)^{\theta}$$

$$h_s/y_e, W_s/y_e, L_s/y_e, \text{ or } V_s/y_e3 = \alpha_e (Q/g^{1/2} y_e^{5/2})^{\beta} (t/316)^{\theta}$$

For Other Cohesive Materials with PI From 5 to 16:

2. a. Compute the culvert outlet velocity in ft/sec.

   b. Obtain a soil sample at the proposed culvert location.

   c. Perform Atterberg limits tests and determine the plasticity index, PI (ASTM D423-36).

   d. Saturate a sample and perform an unconfined compressive test (ASTM D211-66-76) to determine the saturated shear stress, S_v, lb/in^2.

   e. Compute the critical tractive shear strength, τ_c, from Equation 4.20.

   f. Compute the modified shear number $\rho V^2/\tau_c$

3. Determine scour coefficients from Table 4.7.

4. Compute the desired scour hole dimensions for circular culverts from:

$$h_s/D, W_s/D, L_s/D, \text{ or } V_s/D^3 = \alpha(V^2/\tau_c)^{\beta} (t/316)^{\theta}$$

or, for noncircular culverts:

$$h_s/y_e, W_s/y_e, L_s/y_e, \text{ or } V_s/y_e3 = \alpha_e (V^2/\tau_c)^{\beta} (t/316)^{\theta}$$

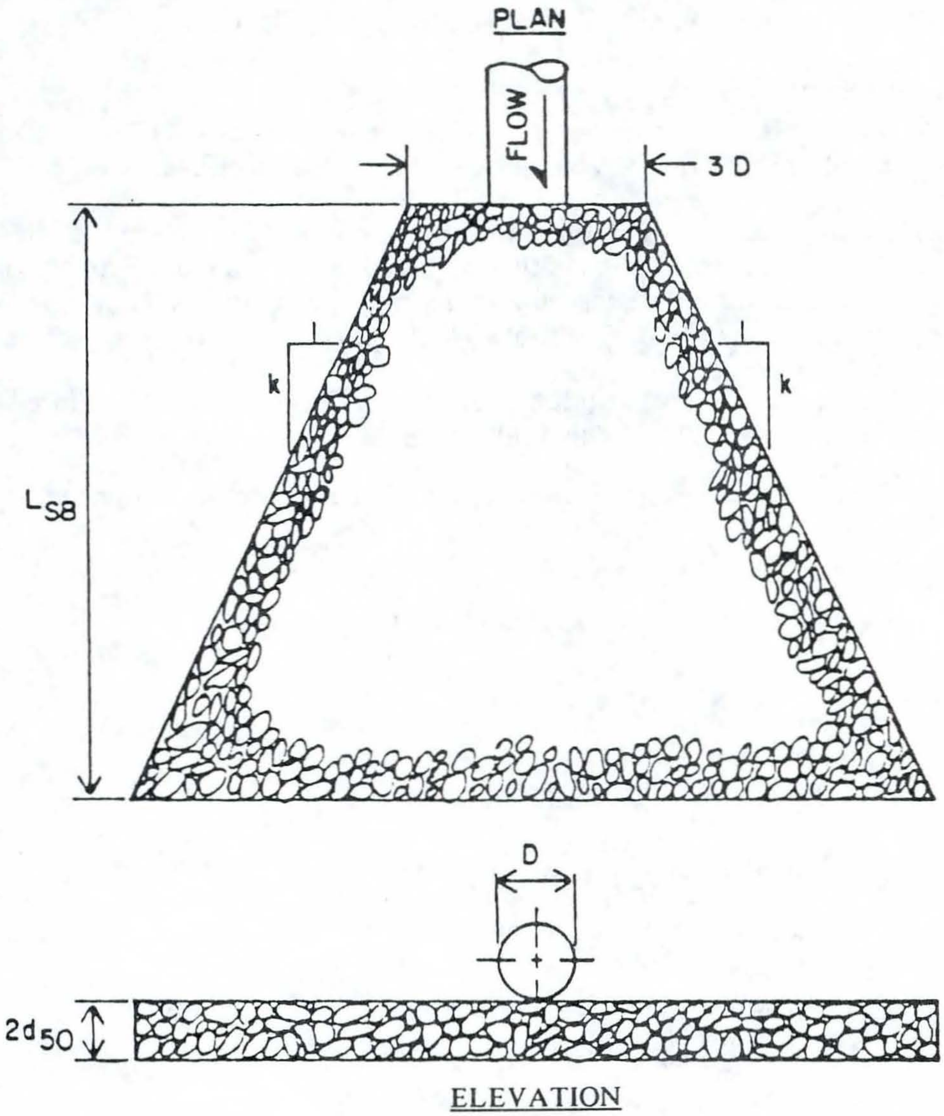
4.5.3.2 Riprap Apron: Riprap aprons placed downstream of culverts provide protection against scour immediately around the culvert as well as providing for the uniform spreading of the flow and decreasing the flow velocity, thus mitigating downstream damages.

These riprap aprons may be designed as simple horizontal aprons as shown in Figure 4.80, with tapered sides of 2:1 for minimum tailwater and 5:1 for maximum tailwater. Minimum tailwater is assumed when the tailwater depth is less than half the culvert height. Maximum tailwater is assumed when the tailwater depth is greater than half the culvert height. The total length of the basin for minimum tailwater should be:

$$L_{sb} = D\,(8 + 17 \log F) \tag{4.21}$$

and for maximum tailwater:

$$L_{sb} = D\,(8 + 55 \log F) \tag{4.22}$$



k	=	2 for minimum tailwater
k	=	5 for maximum tailwater
L_{SB}	=	Length of scour basin, ft
D	=	Culvert diameter, ft.
d_{50}	=	Riprap median diameter

Figure 4.80
Recommended Configuration of Riprap Blanket

For Froude number less than or equal to one, an apron length of approximately eight times the culvert diameter should be used. If the length of protection becomes excessive another form of protection should be used as the outlet velocity exceeds the design criteria for rock aprons. The correct size of riprap for this type of horizontal apron is determined from Figure 5.9 (page 243).

4.5.4 Design Examples

The following design examples have been taken from HEC-14 (Corps, 1975). For more details, refer to that or later editions of the Corps document.

4.5.4.1 Scour Hole Computation Cohesionless Material: Determine the scour geometry—maximum depth, width, length and volume of scour—for a proposed circular 30-inch C.M.P. discharging an estimated 50 cfs when flowing full. The downstream channel is composed of a graded gravel material.

1. The duration of the peak discharge of 50 cfs is not known. Therefore, a peak flow duration of 30 minutes will be estimated.

2. The circular, 30-inch C.M.P. at 50 cfs will have a discharge intensity of

$$D.I. = 50/g^{1/2}(30/12)^{5/2} = 50/(5.67)(2.5)^{5/2} = 0.89$$

3. The coefficients of scour obtained from Table 4.7 are:

	α	β	θ
Depth of Scour	1.49	0.50	0.03
Width of Scour	8.76	0.89	0.10
Length of Scour	13.09	0.62	0.07
Volume of Scour	42.31	2.28	0.17

4. Scour hole dimensions:

Depth: $h_s/D = \alpha(Q/g^{1/2}D^{5/2})^{\beta}(t/316)^{\theta}$

$= 1.49\,(0.89)^{0.50}\,(0.09)^{0.03}$ $\quad h_s = 1.31 \times 2.5 = 3.28$ ft

Width: $W_s/D = 8.76\,(0.89)^{0.89}\,(0.09)^{0.10}$ $\quad W_s = 6.21 \times 2.5 = 15.5$ ft

Length: $L_s/D = 13.09\,(0.89)^{0.62}\,(0.09)^{0.07}$ $\quad L_s = 10.29 \times 2.5 = 25.72$

Volume: $V_s/D^3 = 42.31\,(0.89)^{2.28}\,(0.09)^{0.17}$ $\quad V_s = 21.54 \times 15.63 = 336.67$ ft^3

5. The location of the maximum scour (Figure 4.79)

$0.4\,(L_s) = 0.4\,(25.72) = 10.3$ ft downstream of the culvert outlet

4.5.4.2 Scour Hole Computation Cohesive Material: Determine the scour geometry-maximum depth, width, length, and volume of scour for an existing circular 24-inch C.M.P. discharging an estimated 40 cfs when flowing full. The downstream channel is composed of a sandy-clay material.

1. The duration of the peak discharge of 40 cfs is not known. Therefore, a peak flow duration of 30 minutes will be estimated.

2. a. The average velocity at the culvert outlet is:

$$V = Q/A = 40.0/3.14 = 12.74 \text{ fps}$$

b-e. The sandy-clay material was tested and found to have a PI of 12 and a saturated shear strength (S_v) of 240 psi. The critical tractive shear can be estimated by substituting into Equation 4.20.

$$\tau c = 0.001\ (240 + 180)\tan(30 + 1.73(12))$$
$$0.001\ (420)\tan(50.76) = 0.51\ 1b/ft^2$$

f. The modified shear number $Sn_{mod} = (\rho V^2/\tau_c)$ is:

$$Sn_{mod} = 1.94\ (12.74)^2/0.51 = 617.4$$

3. The experimental coefficients α, β, and θ from Table 4.7 are:

	α	β	θ
Depth	0.86	0.18	0.10
Width	3.55	0.17	0.07
Length	2.82	0.33	0.09
Volume	0.62	0.93	0.23

4. The scour hole dimensions are:

$$h_s/D = \alpha(\rho V^2/\tau_c)^{\beta}\ (t/316)^{\theta}$$

$$= 0.86(617.4)^{0.18}\ (0.09)^{0.10} \qquad h_s = 2.14 \times 2 = 4.30 \text{ ft}$$

$$W_s/D = 3.55(617.4)^{0.17}\ (0.09)^{0.07} \qquad W_s = 8.94 \times 2 = 17.9 \text{ ft}$$

$$L_s/D = 2.82(617.4)^{0.33}\ (0.09)^{0.09} \qquad L_s = 18.92 \times 2 = 37.8 \text{ ft}$$

$$V_s/D^3 = 0.62(617.4)^{0.93}\ (0.09)^{0.23} \qquad V_s = 140.3 \times 8 = 1122.5 \text{ ft}^3$$

5. Location of maximum depth of scour (Figure 4.79)

$0.4\ L_s = 0.4(37.8) = 15.1$ ft downstream of culvert outlet.

4.6 Rational Method for Sizing Storm Drain Systems

This section provides a method of estimating the flows needed to size a storm drain system using the Rational Equation, $Q = CIA$, as it is defined in Volume I of the *Drainage Design Manual* (see also 8 through 11, below). ***It should be stressed that the Rational Method was originally developed to estimate runoff from small areas and that the peak generated by the Rational Method cannot be hydrologically routed.***

The engineer should proceed with final hydraulic design of the system only after:

» The preliminary minor system design is completed and checked for its interaction with the major runoff;

» Reviews are made of alternatives;

» Hydrological assumptions are verified; and

» Final data is obtained on street grades and elevations.

The following paragraphs provide an outline of the the *Storm Drainage System Preliminary Design Data Form* (Figure 4.81), which was created to help the designer estimate flows for storm drain sizing. This form is for the average situation and variations will often be necessary to fit actual field conditions.

1. *Location of Design Point:* Determine design point location and list. This design point should correspond to the subbasin illustrated on the preliminary layout map.

2. *Basins:* List basins contributing runoff to this point which have not previously been analyzed.

3. *Length, ft:* Enter length of flow path between previous design point and design point under consideration.

4. *Inlet Time, minutes:* Determine the inlet time in minutes for the particular design point. For the first design point on a system, the inlet time will be equal to the time of concentration. For subsequent design points, inlet time should also be tabulated to determine if it may be of longer duration than the accumulated time of concentration from upstream basins. If the inlet time exceeds the time of concentration from the upstream basin, and the area tributary to the inlet is of sufficient magnitude, the longer inlet time should be substituted for time of concentration and used for this and subsequent basins.

5. *Street Flow Time, minutes:* Enter the appropriate flow time between the previous design point and the design point under consideration. The flow time of the street should be used if a significant portion of the flow from the above basin is carried in the street.

6. *Pipe Flow Time, minutes:* Pipe flow time should generally be used unless there is significant carry-over from above basins in the street.

7. *Time of Concentration, minutes:* The time of concentration is either inlet time or the summation of the previous design point time of concentration and the intervening flow time, whichever produces the greater peak discharge.

8. *Coefficient "C":* Rational Method Runoff Coefficient, "C," from the *Drainage Design Manual for Maricopa County, Volume I, Hydrology* (FCD 1990, Table 3.2), for the basins listed in Column 2 should be determined and listed. The "C" value should be weighted if the basins contain areas with different "C" values.

9. *Intensity "I," inches/hour:* The intensity to be applied to the basins is obtained from the time-intensity-frequency curve developed for the specific area based upon the depth-duration-frequency curves in *Drainage Design Manual for Maricopa County, Volume I, Hydrology*. The intensity is determined from the time of concentration and the design frequency for this particular design point.

10. *Area "A," acre:* The area of the basins (in acres) listed in Column 2 is tabulated here. Subtract ponding areas which do not contribute to direct runoff such as rooftop and parking lot ponding areas.

11. *Direct Runoff "Q," cfs:* Direct runoff from the tributary basins listed in Column 2 is calculated and tabulated here by multiplying Columns 8, 9, and 10 together.

12. *Other Runoff, cfs:* Runoff from other sources, such as controlled releases from rooftops, parking lots, base flows from groundwater, and any other source, is listed here.

13. *Summation Runoff, cfs:* The total of runoff from the previous design point summation plus the incremental runoff listed in Columns 11 and 12 is listed here.

14. *Street Slope, percent:* The proposed street slope is listed in this column.

15. *Street Allowable Capacity, cfs:* The allowable capacity for the street is listed in this column. Allowable capacities should be calculated in accordance with procedures set forth in Section 3.2.

16. *Pipe Slope, percent:* List the proposed pipe grade.

17. *Pipe Size, inches:* List the required pipe size to convey the quantity of flow necessary in the pipe.

18. *Pipe Capacity, cfs:* List the capacity of the pipe flowing full with the slope expressed in Column 16.

19. *Street Design, cfs:* Tabulate the quantity of flow to be carried in the street.

20. *Street Velocity, fps:* List the actual velocity of flow for the volume of runoff to be carried in the street.

21. *Pipe Design, cfs:* List the quantity of flow determined to be carried in the pipe.

22. *Pipe Velocity, fps:* Tabulate the actual velocity of flow in the pipe for design.

23. *Remarks:* Include any remarks or comments which may affect or explain the design. The allowable quantity of carry-over across the street intersections should often be listed for the minor design storm. When routing the major storm through the system, required elevations for adjacent buildings can often be listed in this column.

4.6.1 Design Example

Figure 4.82 represents a completed Storm Drainage System Preliminary Design Data form. The data contained in Figure 4.82 is intended to supplement the information shown in Figure 4.17 for the hydraulic design example in Section 4.2.5. Because junction losses are ignored in this analysis, the pipe roughness coefficient factor is to be increased by 25 percent.

All basic data are shown in Figure 4.82. The entire area is assumed to be residential and the streets are either local or collector. The design frequency is 10 years, and the points of beginning for the storm drain have been assumed. The purpose of this analysis is to demonstrate the use of the Rational Method for preliminary design of storm drains. Unless computed in the Figure 4.82, times of concentration, T_c, are assumed.

Be aware that pipe diameters may change in final design from preliminary design; however these effects are generally cancelling and can be ignored. When there is a net change of pipe diameters (non-cancelling) in 20 percent of the pipe length, the designer should reanalyze to insure system integrity (higher discharges and shorter time of concentration) or eliminate wasted investment (lower discharges and lower times of concentration). If a different type of pipe with a large difference in roughness factors is used (RCP vs. CMP), the system must be designed using both materials.

After the system has been designed according to the summation method of the Rational Method, the 100-year runoff is conveyed through the storm drain system to insure that structural flooding criteria are met for that event. Because the analysis is similar, it is not shown here. The designer should be aware that the "C" factors must be increased by 25 percent for the 100 year event in the Rational Formula (maximum = 1.0) and the flow time from point to point is determined by the length and velocity of gutter flow (as compared to the time of flow in pipes used to size the system).

Of special note is the hydraulic (final) design of the lateral which enters MH-4 from Basins 9 and 10. The hydraulic design example was done for the design conditions for the overall system. In reality, the system must also be hydraulically designed for the higher discharge (26.9 cfs) emanating from Basins 9 and 10. This is true of all laterals and inlet connector pipes.

The methodology is to determine the flow from the mainstem when the peak flow from Basins 9 and 10 occurs. This is a trial and error process starting at some point between I-1 and MH-4. This can be approximately determined by progressively subtracting the flow times (proceeding upstream) obtained in the preliminary design of the main trunk. The new starting point will be when the T_c at a design point equals the T_c after subtracting flow time to that point. Using the pipes as originally determined, new discharges are computed at the point in question to obtain the discharge in the mainstem when the higher lateral flow enters.

Figure 4.81

STORM DRAINAGE SYSTEM PRELIMINARY DESIGN DATA																						
				Flow Time									Street		Pipe			Street		Pipe		
Location of Design Point	Basins	Length ft.	Inlet Time min	Street min.	Pipe min.	Time of Concentration min.	Coefficient "C"	Intensity "I" in/hr.	Area "A" acre	Direct Runoff cfs	Other Runoff cfs	Summation Runoff cfs	Slope %	Allowable Capacity cfs	Slope %	Size in.	Capacity cfs	Design cfs	Velocity fps	Design cfs	Velocity fps	Remarks
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23

Designer ______ Date ______ Location ______

Design Assumption ______ Frequency ______ Job No ______

Figure 4.82

STORM DRAINAGE SYSTEM PRELIMINARY DESIGN DATA

Location of Design Point	Basins	Length ft.	Inlet Time min	Flow Time: Street min.	Flow Time: Pipe min	Time of Concentration min.	Coefficient "C"	Intensity "I" in/hr.	Area "A" acre	Direct Runoff cfs	Other Runoff cfs	Summation Runoff cfs	Street: Slope %	Street: Allowable Capacity cfs	Pipe: Slope %	Pipe: Size in.	Pipe: Capacity cfs	Street: Design cfs	Street: Velocity fps	Pipe: Design cfs	Pipe: Velocity fps	Remarks — Use concrete pipe n = 0.013 × 1.25 = 0.016
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23
I-1	1		10			10	0.30	4.8	1.7	2.4	–	2.4										Street grade between intersections = 0.4%
		208			1.5								0.4	2.0	0.6	12	2.4	–	–	2.4	2.3	
I-2	2		10			11.5	0.30	4.6	1.3	1.8		4.2										Since storm sewer is started,
		162			0.7								0.46	2.1	0.6	15	4.5	–	–	4.2	4.0	pipe up flow at intersections
I-3	3,4		10			12.2	0.35	4.5	2.0	3.1												
		238			0.7							7.3	1.02	3.2	1.0	18	9.5	–	–	7.3	5.8	
I-5	5,5A					12.9																Existing 18" Conc. pipe from Basin
		Need to ck Area from East Exist 18") to determine if its Tc controls analysis or if it is																				5A Approx. Capacity = 9.5cfs
		adequate to carry flow from Basin 5A																				Flow not captured in 18" is
																						carried into south gutter
Exist. 18"	Conc. 5A		10.8			10.8	0.35	4.7	8.2	13.5												Existing street is designed with
		240			0.8										1.0	18"	9.5	Cross street	street flow	9.5	5.2	no crown
I-5	5					11.6																
					∴ Tc for Basin 5A @ Inlet I-5 < Tc to Inlet I-5 from Basins 1,2,3, and 4																	
					Go back to I-5																	
												Constant flow from exist. 18"										
I-5	5,5A		11			12.9	0.30	4.4	3.8	5.0	9.5											
		67			0.2							21.8	No street		0.5	30*	27	–	–	21.8	5.8	
I-6	6		10			13.0	0.50	4.4	1.2	2.6	2.6											Carry over from Basin 5A
		314			0.7							27.0	No street		0.8	30	33			27.0	7.1	
MH-1	7		12†			13.7	0.25	4.3	13.0	14.0												
		250			0.6							41.0	No street		0.6	36	46	–		41.0	7.2	
MH-2	8		13			14.3	0.15	4.2	7.9	5.0												
		240			0.6							46.0	No street		0.6	36*	46			46.0	6.3	Only slightly pressurized
MH-3	–		–			14.9	–	–														
		100			0.3							46.0	No street		0.3	42	48			46	5.7	
MH-4	9,10					15.2		Must verify Tc from Basin 9 < Tc from Basin 1-8														

† Determined from overland flow & pipeline flow computations not shown in this example

* Denotes line sizes that were changed in final design

Designer JQA — Date 4-17-79 — Location Example

Design Assumption — Frequency 2 year — Job No 434.5

Figure 4.82, continued

STORM DRAINAGE SYSTEM PRELIMINARY DESIGN DATA

Location of Design Point	Basins	Length ft.	Inlet Time min.	Flow Time: Street min.	Flow Time: Pipe min.	Time of Concentration min.	Coefficient "C"	Intensity "I" in/hr.	Area "A" acre	Direct Runoff cfs	Other Runoff cfs	Summation Runoff cfs	Street: Slope %	Street: Allowable Capacity cfs	Pipe: Slope %	Pipe: Size in.	Pipe: Capacity cfs	Street: Design cfs	Street: Velocity fps	Pipe: Design cfs	Pipe: Velocity fps	Remarks
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23
MH-8	9		12.4			12.4	0.35	4.6	100	16.1	–											
		300			0.6							16.1	1.4	6.5	1.4	24	24.0	—	—	16.1	8.0	
I-9	10		10.0			13.0	0.35	4.4	70	108												
		85			0.3							26.9	No Street		0.3	33	26.0			26.9	4.5	Assume some pressurization possible
MH-7	—		—			13.3	—	—	—	—												
		250			0.6							26.9	No street		1.2	27	30.0			26.9	7.5	
MH-6						13.9	—	—	—	—												
		162			0.4							26.9	No street		0.8	30	33.0			26.9	7.2	
MH-4						14.3	<	15.2	–													
			Because pipe sizes are decreased in 1 segment – Redo analysis using 30" to																			
			inlet I-9 – Losses will be compensating. Steepen I-9 to MH-7																			
MH-8	9		12.4			12.4	0.35	4.6	10.0	16.1												
		300			0.6							16.1	1.4	6.5	1.4	24	24.0			16.1	8.0	
I-9	10		10.0			13.0	0.35	4.4	7.0	10.8												
		85			0.3							26.9	No street		0.5	30	25.0			26.9	5.3	
MH-7	—					13.3	—	—	—	—												
		250			0.5							26.9	No Street		1.0	30	36.0			26.9	7.8	
MH-6						13.8	—	—	—	—												
		162			0.4							26.9	No street		0.8	30	33.0			26.9	7.1	
MH-4						14.2	< 15.2															
					∴ Route from Basins 1-8 controls																	
MH-4						15.2	0.35	4.2	17.0	25.0												
	9, 10	405			1.2							71.0	No street		0.2	54*	80			71	5.5	
MH-5						16.4	—	—	—	—												
		105			0.3							71.0	No street		0.2	54	80			71	5.5	
Outlet						16.7																

* Denotes line sizes that were changed in final design

Designer JQA Date 4-17-79 Location Example

Design Assumption Frequency 2 year Job No. 434.5

4.7 References

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5

Open Channels

5.1 General

An open channel is a conveyance in which water flows with a free surface and may be natural or artificial. Natural streams usually consist of a normal or low flow channel and adjacent floodplains. For purposes of this guideline, the term open channel will include the total conveyance facility, floodplain, and stream channel.

Open channel hydraulics is of particular importance to design because of the interrelationship of channels to street and urbanization drainage. In the hydraulic analysis and design of bridges and culverts, open channel hydraulic principles are used to evaluate the effects of proposed structures on water surface profiles, flow and velocity distributions, lateral and vertical stability of the channel, stream regime, flood risk, and the potential reaction of channels to changes in variables such as urbanization; structure type, shape, and location; and scour control measures.

The hydraulic design process for open channels consists of establishing criteria, developing and evaluating alternatives, and selecting the alternative which best satisfies the established criteria. Elements that should be considered in the design process include capital investment and probable future costs, such as maintenance and flood damages to properties; traffic requirements; and the stream and floodplain environment.

Open channel design can be quite complex, requiring both specific education and extensive experience; however, when provided with specific procedures and criteria many urban applications can be successfully designed by engineers with substantially less experience. This chapter examines channel design for common urban applications, including roadside channels, channels within developments, and existing channels in urban areas or urbanizing areas that can be analyzed as "rigid." Occasionally, channel design of movable-bed and non-rigid channels will be required. These are complex, and specific design aids and descriptions of design are not included in this manual. For these applications, engineers qualified in open channel design should undertake the design. Checklists of the requirements and resources to be used for the more complex channel designs are included in Section 5.5.

5.1.1 Urban Open Channels

Urbanization causes increases in both the volume and the rate of runoff. Current practice in Maricopa County utilizes storm sewers and open channels to convey stormwater to detention or, more commonly, retention facilities. This practice substantially reduces impacts from natural occurring storms; however, the volume of runoff carried by natural streams will increase. The result can be a change in the overall stability of both natural and artificial channels. The analysis of these effects are outside the scope of this manual; however, a discussion of these effects is included in Section 5.5, as well as a checklist of technical matters to consider and a list of references which supports the checklist.

When land is developed, runoff from urban areas is concentrated to control stormwaters and provide a healthy environment. Even for small basins, concentrated runoff cannot simply be turned loose on adjacent grounds. Such action will result in erosion and the creation of a "new" urban channel; therefore, planning and design for urbanization needs to include approved disposal of newly created runoff from a development site. It is important to note that interfaces between natural and artificial channels are critical and require specific attention during design. In addition, sediment management and stream geomorphology are critical to both natural and artificial channels.

The preceding discussion simply illustrates a few requirements for design of urban channels. On larger scales, the designer may be faced with analysis or design of "non-rigid" channels; however, many urban applications employ "rigid channel" design concepts in order to gain sufficient control of urbanized stormwaters, often within a limited right of way. This chapter addresses design of rigid channels.

5.1.2 Floodplains

Maricopa County has floodplains resulting from natural channels as well as those that are created, or expanded, due to urbanization. Some of these floodplains have been identified and are being regulated by individual local governments, County agencies, the Arizona Department of Water Resources (ADWR), and the Federal Emergency Management Agency (FEMA). The Flood Control District of Maricopa County and most communities issue floodplain use permits for activities within FEMA floodplains, and drainage permits for activities located outside them. This system has evolved due to two separate statutory authorities resulting in two separate regulations. When developing designs in flood-prone areas be aware that even though the project is located in what a geomorphologist would classify a floodplain, if it has not been mapped and published by FEMA, then—from a regulatory standpoint—it is not considered a floodplain. In that case, the developer would be required to obtain a drainage permit instead of a floodplain use permit.

Regulation of floodplains has been undertaken by authority contained in the National Flood Insurance Program. Engineers designing open channels or analyzing floodplains are faced with the provisions of the program; therefore, the following short description of the program and its requirements is included.

Special Flood Hazard Areas (SFHA) have been delineated on Flood Insurance Rate Maps (FIRMs) which can be obtained from the local sponsoring agency (i.e., the

Flood Control District of Maricopa County, City of Phoenix, etc.). The 100-year flood boundaries, flood insurance rate zones, and regulatory flood elevations are shown on the FIRMs. All new development and significant modifications to existing uses must be approved by the entity responsible for regulating floodplains in the channel reach in which the development or modifications are to occur.

In instances where there is insufficient detail to accurately depict the floodplain and the various zones or when changes to existing conditions occur, a map revision process can be used. The revision process includes a Letter of Map Revision (LOMR) or a Letter of Map Amendment (LOMA). While procedurally similar, new data that shows old studies to be in error or changed conditions are addressed in a LOMR. Changes to FIRMs resulting from exclusion of individual structures and undeveloped parcels are described in a LOMA.

If construction is proposed on land within a SFHA, a conditional LOMA or LOMR may be sought from FEMA showing that the proposed structural information meets the established criteria for a standard LOMA or LOMR. After completion of the project, certified record drawing information must be submitted to FEMA to obtain an accepted LOMA or LOMR. Conditional LOMAs and LOMRs do not amend the FIRM nor do they waive the requirement to purchase flood insurance.

5.2 Definition of Symbols

The symbols defined below will be used in equations throughout Chapter 5:

Δy = Change in the water surface, ft

$\Sigma L_o/L_T$ = Ratio of the summation of the distances between rows of buildings, L_o, to the total length of the reach along a profile parallel to the direction of flow, L_T, ft/ft

A = Cross sectional area of flow, ft^2

b = Channel bottom width, ft

D = Hydraulic Depth = A/T, ft

D_i = The average diameter of a rock particle, for which "i" percent of the gradation is finer by weight.

E = Specific energy, ft

F = Froude Number

FB = Freeboard, ft

F_c = Section factor at critical depth, $ft^{3/2}$

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

G = Gradation coefficient

h_v = Velocity head, ft

n = Manning roughness coefficient (see Section 5.6)

n_o = Roughness coefficient for the area between the buildings in the floodplain (i.e., streets, yards, etc.)

n_u	=	Adjusted urban roughness coefficient
P	=	Wetted perimeter, ft
Q	=	Discharge, cfs
R	=	Hydraulic radius = A/P, ft
r_c	=	Radius of channel center-line curvature, ft (see Equation 5.12)
S_o	=	Channel bottom slope, ft/ft
T	=	Channel width along the top of the water surface, ft
V	=	Average velocity of section, fps
W_o	=	Individual widths between buildings, measured perpendicular to the direction of flow, ft
W_T	=	Total width of the floodplain, including buildings, ft
y	=	Distance from water surface to the centroid of the section, ft
Y	=	Depth of flow, ft
Y_c	=	Critical depth of flow, ft
Y_n	=	Normal depth of flow, ft

5.3 Artificial Channels

Artificial open channels are used in drainage for a wide variety of applications. The applications vary in scale from modest roadside ditches and on-site drainage to large conveyance facilities that can be several hundred to several thousand feet wide. Channel linings heavily influence other physical characteristics of open channels.

This section covers the open channel design applications more commonly encountered by civil engineers. Applications involving rivers and large washes or channels—which are considered "non-rigid"—require special design skills, and the design of these channels should not be attempted with the design techniques contained in this manual.

Rigid grade control is a requirement for using this section.

5.3.1 Route Considerations

Open channel failures frequently result from poor layout of surrounding land during the planning process. Without consideration of hydraulic parameters during the earliest phases of planning, unsafe conditions are likely to result and, often, facility and maintenance costs are excessive. A typical example of a safety problem is a high velocity channel being required in a residential area.

All natural channels are in a constant state of change. Natural channels that have small changes resulting from periods of low flows and periods of relatively high flows are considered in equilibrium. The ideal artificial channel is one that approximates a natural channel, in equilibrium, or one which has been carved over a long period of time. These channels do not have excessive velocities, and are without closely-spaced, sharp, and reverse curvatures. Artificial channels should be aligned with the entrances from and exits to hydraulic structures. In all cases, the issue of wet and dry weather safety should be a paramount consideration in route and right-of-way determinations.

Larger natural channels have low flow channels contained within the bottom width. To provide low cost maintenance, the artificial open channels should be stable for both lower and higher flow rates, which may require consideration of low flow channels to prevent accumulations of silt and reduction of stream capacity. The route should permit the use of uniform, stable channel side slopes; permit the maintenance of subcritical flow; and maintain constant channel properties such as width, side slopes, and depth. Because this condition is sometimes difficult to achieve, it can result in channels that are likely to move and, hence, result in ongoing risk and continuous maintenance.

5.3.2 Choice of Channel

The choices of channel properties available to the designer are numerous, and depend upon good hydraulic practice, environmental design, sociologic impact, basic project requirements, recognition of risks involved, and maintenance. However, from a practical standpoint, the basic choice to be made initially is whether or not the channel lining is concrete for higher velocities, soil cement, rock, or earth. In some instances, a grass lining is appropriate.

The actual choice must be made upon a variety of multi-disciplinary factors and complex considerations which include, among others:

Hydraulic

» Slope of thalweg
» Right-of-way
» Stream bed controls—bed stability
» Capacity needed
» Basin sediment yield and channel transport capacity
» Topography
» Ability to drain adjacent lands
» Geotechnical
» Groundwater levels and groundwater recharge

Structural

» Costs
» Availability of material
» Areas for excavation materials (spoil sites)

Environmental

» Neighborhood character
» Neighborhood aesthetic requirements
» Need for new green areas
» Street and traffic pattern

» Municipal or Flood Control District policies/ordinances
» Need for open space

Sociological
» Neighborhood social patterns
» Neighborhood child population
» Pedestrian traffic
» Recreation needs

5.3.2.1 Description of Channel Types: Artificial channel types can vary with the shape of the section and with the lining used for the channel bottom and banks. Typical channel lining types 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 5.1 and are discussed in detail in Section 5.5.

Some of the sections of Figure 5.1 show an optional low flow channel (discussed below), however, compound sections larger than a low flow channel may be desirable simply because they incorporate hiking trails and other recreational activities.

Concrete Lined Channels: Concrete lined channels are used primarily where right of way is limited. The channels may be designed for either subcritical or supercritical flow and generally have steep side slopes because of the limited right of way. Inherently, these channels present greater personal safety problems both in wet and dry weather (see Section 5.3.2.5). In addition, supercritical flows present greater problems to the designer in the design of the channel and in the design of appurtenances such as bridges and culverts. Channels with supercritical flow require special attention to construction joint details, changes in channel alignment, transitions, and at the interface with all hydraulic structures.

Use of concrete lined channels is discouraged in residential and recreation areas. If concrete channels are needed in these areas, fencing will probably be required to prevent access.

Concrete lined channels require reinforcing steel and minimum concrete thicknesses dictated by anticipated structural loads and the clearance requirements of steel reinforcing. It is not recommended that non-reinforced concrete be used for open channels. In addition, concrete lined channels require weep holes and/or subdrains to prevent uplift damages. Figures 5.1 and 5.2 illustrate that the top of the concrete is tied into the ground, which is required to prevent erosion caused by water entering the channel laterally, eroding the soils behind the concrete and damaging the lining. Several instances of channel failure in the Maricopa County area have occurred due to the lack of a tie-in detail at the top of a concrete channel lining. Figure 5.2 provides details for key-ins required for concrete, shotcrete and soil cement channels.

Soil Cement: Soil cement linings are composed of a thick lining of soil cement without reinforcement and have been used successfully in Maricopa County. Soil

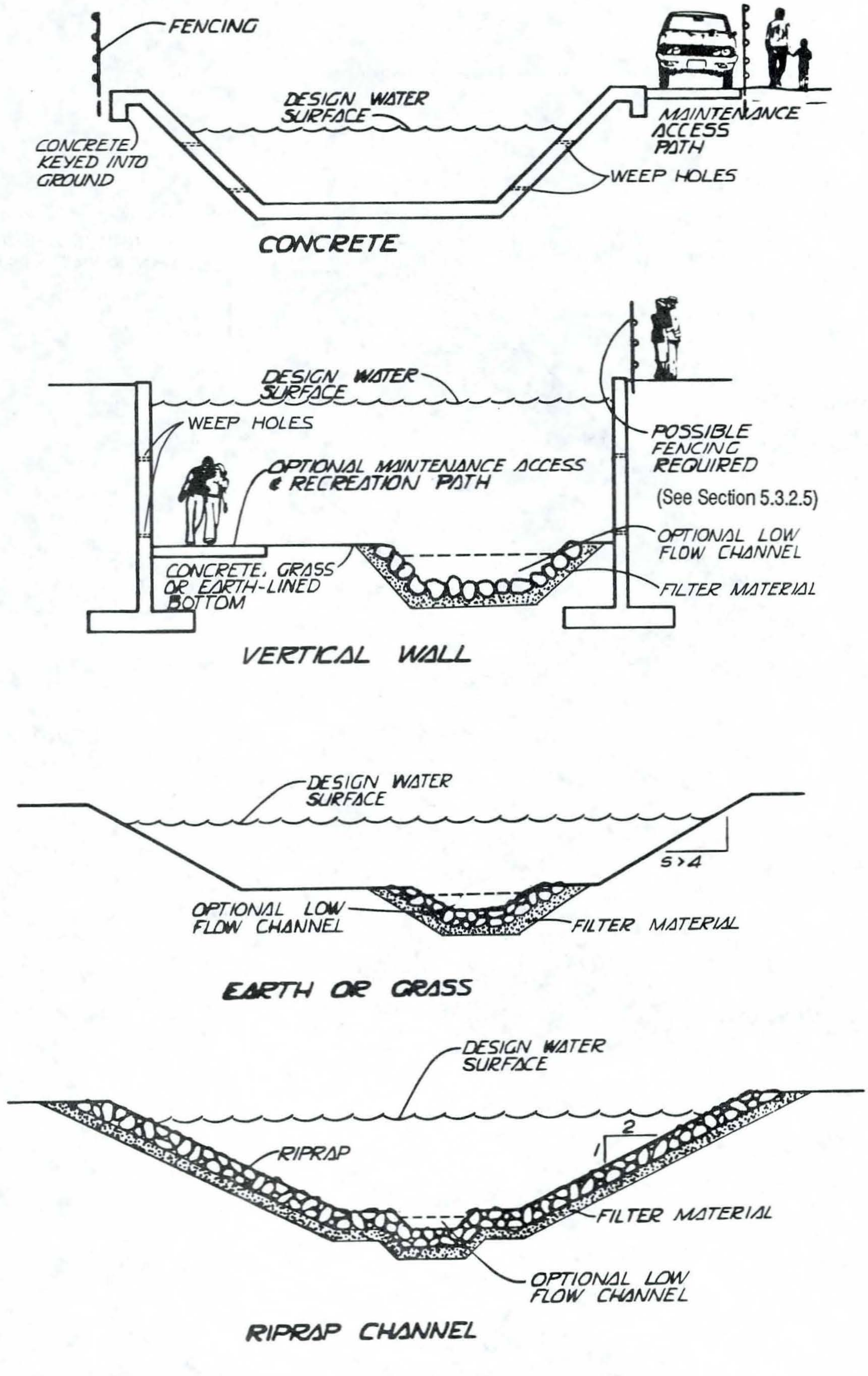


Figure 5.1
Typical Channel Sections

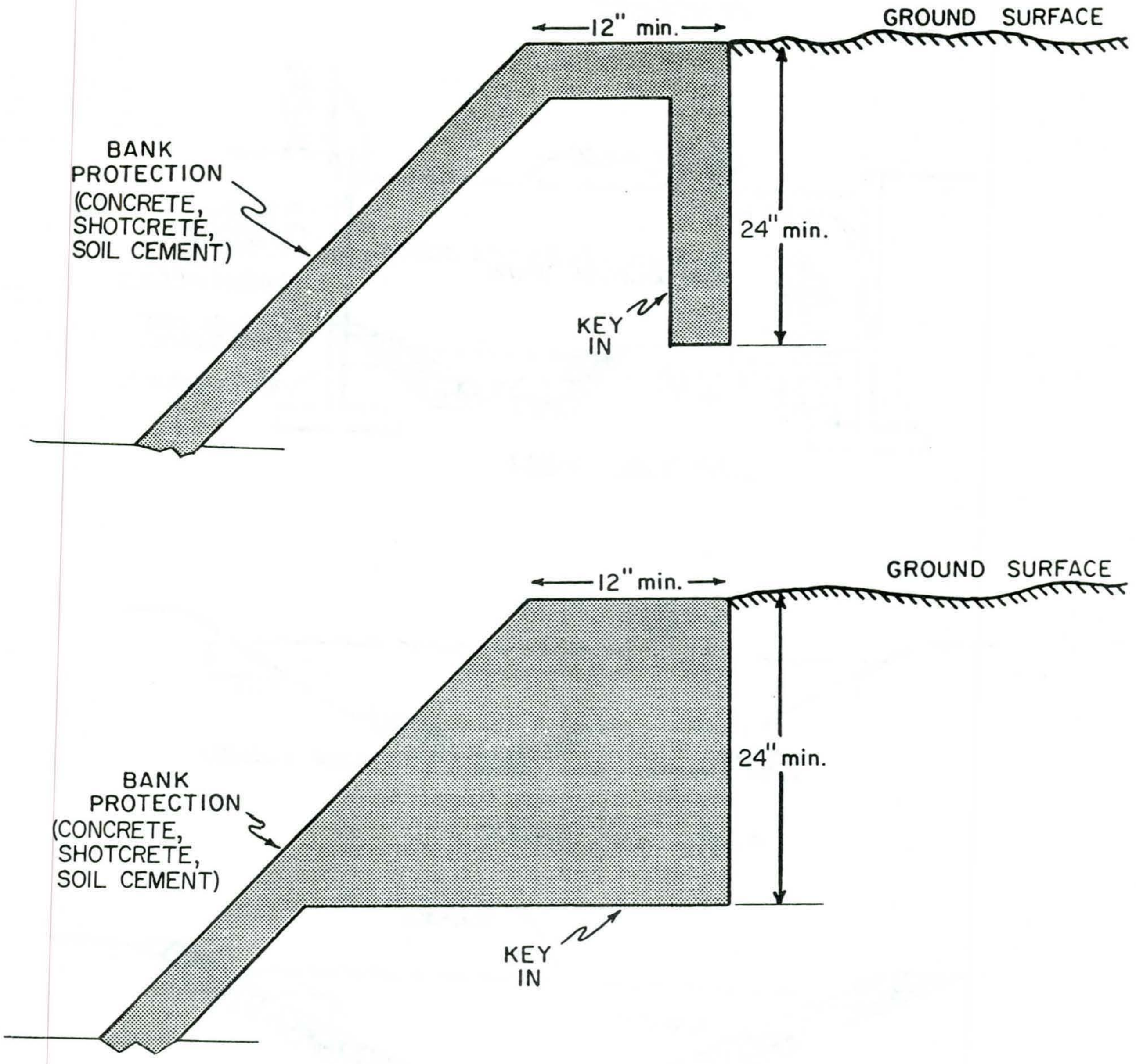


Figure 5.2
Typical Bank Protection Key-ins
(Simons, Li and Associates 1988)

cement is subject to weathering, including erosion, and may not have good life cycle traits when used in the bottom of channels. The side slopes can be steep (2 horizontal : 1 vertical); therefore, the same restrictions for concrete-lined channels should be placed on the use of soil cement channel types when they are used in residential and recreation areas.

With the possible exception of channel bottoms, soil cement can withstand higher velocities than some of the lining types. Soil cement is most likely to be used for channels with limited and restricted rights of way or as bank linings near bridges and culverts where ambient stream velocities tend to be higher than the average velocity within a channel reach. If soil cement is to be used in conjunction with an earthen or grass-lined channel bottom, control of channel grade and scour must be incorporated into the design of the channel.

Rock Lined Channels: This class of lining includes both common riprap and gabion riprap linings. In general, both of these types require placement of 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. Because of weathering and vandalism, gabion riprap should not be placed on slopes steeper than where common riprap would be used (see Section 5.4).

Riprap channels are to be designed for subcritical flow; however, because the permissible side slopes are steeper (3 horizontal: 1 vertical) than allowed for either grass lined or earth lined channels, riprap lined channels will normally be used in areas with restricted right of way. These channels will flow in the subcritical range and the rough texture may permit an individual to exit from the channel during flows, potentially permitting less restrictive safety requirements than for a typical concrete-lined channel. Design around hydraulic structures may also be less restrictive. Use of toe rock with riprap side slopes can work successfully in combination with an earth bottom. This channel is much less prone to move than a totally earth lined channel. Non-rigid design techniques must be used when earth channel bottoms are combined with riprap sides. The riprap toe protection (toe rock) should be designed to protect against anticipated scour (see Figure 5.11a, page 247).

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

Earth lined channels are designed for subcritical flow ranges. The width to depth ratios are large, the side slopes are flatter than 4 horizontal : 1 vertical, and grade control is a requirement of the design. The smaller-size range of these types of channels do not require low flow channels; however, for larger-sized channels and for channels used for pedestrian corridors, an armored low flow channel may be required to achieve effective, low-maintenance channels. Riprap toe protection may be used to increase channel stability.

Grass Lined Channels: These channels have similar properties to earth lined channels with side slopes no steeper than 4 horizontal : 1 vertical. The root structure of the grass permits higher flow velocities and smaller sections. Non-irrigated, grass lined channels will revert to earth lined channels; therefore, all references to grass lined channels will be for irrigated grass.

Because water must be conserved in the desert environment, grass lined channels are most likely to be used as part of landscaping for smaller development tracts and as part of local and regional park schemes where multiple use activities are included.

5.3.2.2 Flow Characteristics:

Design velocities for all linings should not fall below 2 fps to minimize sediment depositional problems.

Determine the maximum allowable velocity from Tables 5.1 and 5.2. Calculate the allowable capacity for the drainage ditch using Manning's formula with an appropriate n value. Manning's n values for typical channel materials are presented in Table 5.11 (page 257). If the natural channel slope would cause excessive velocity, employ drop structures, checks, riprap, or other suitable channel protection.

5.3.2.3 Embankment Protection: Depth of flow and velocity are two primary considerations for determining the appropriate type of erosion protection liner. Specific liner types and their design criteria and procedures are presented in Section 5.4.

Table 5.1
Permissible Velocities for Roadside Drainage Channels with Erodible Linings

Soils Type of Lining (Earth, No Vegetation)	Permissible Velocity, fps
Fine Sand (noncolloidal)	2.5
Sandy Loam (noncolloidal)	2.5
Silt Loam (noncolloidal)	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 (noncolloidal)	3.5
Alluvial Silts (colloidal)	5.0
Coarse Gravel (noncolloidal)	6.0
Cobbles and Shingles	5.5
Shales and Hard Pans	6.0

From: FHWA - HDS 3 & 4, Uniform Soils Classification.

Table 5.2
Roadside Channels Lined with Uniform Stand of Various Grass Cover and Well Maintained [1] [2]

Cover	Slope Range, %	Permissible Velocity, fps [3]	
		Erosion Resistant Soils	Easily Eroded Soils
Bermuda Grass[3]	0 to 5	8.0	6.0
	5 to 10	7.0	5.0
	Over 10	6.0	4.0
Buffalo Grass Kentucky Bluegrass Smooth Brome Blue Grama	0 to 5 5 to10 Over 10	7.0 6.0 5.0	5.0 4.0 3.0
Grass Mixture	0 to 5[4]	5.0	4.0
	5 to 10[4]	4.0	3.0
Lespedeza Sericea Weeping Lovegrass Yellow Bluestem Kudzu Alfalfa[3] Crabgrass	0 to 5[5]	3.5	2.5
Common Lespedeza[6] Sudangrass[6]	0 to 5[5]	3.5	2.5

(1) From Handbook of Channel Design for Soil and Water Conservation.

(2) Use velocities over 5 fps only where good covers and proper maintenance can be obtained.

(3) Typically used in Maricopa County.

(4) Do not use on slopes steeper than 10 percent.

(5) Not recommended for use on slopes steeper than 5 percent .

(6) Annuals, used on mild slopes or as temporary protection until permanent covers are established.

5.3.2.4 Low Flow Channels: The majority of storm events will be less than the design storm, resulting in frequent low flow conditions. Low flows in earth- or grass-lined trapezoidal channels will deposit sediment and develop their own pilot channel which will be meandering and could direct low flows into the channel banks causing bank erosion. Design of low flow channels will prevent meandering and will direct low flows in a controlled manner.

Rounding the channel bottom to approximate a parabolic shape will cause the centerline of the channel to act as a low flow channel. Alternatively, the channel bottom could be graded into a shallow V-shape to lower the centerline.

Because of the potential for long-term channel aggradation, base flows and crop irrigation return flows may require specific consideration. Waterways which are normally dry often have somewhat continuous low flows after urbanization because of lawn irrigation and other outside uses of water, including crop irrigation return flows for developments on the edge of urbanizing areas. Maricopa County is generally typified by low groundwater tables, porous surface materials and limited irrigation, which tend to reduce low flows and short-term problems of aggradation and wet channel bottoms.

Base flows for larger drainage basins can be a significant stability and maintenance problem for earth- or grass-bottomed channels. In this discussion, base flows can be considered as flow rates that are less than the 5- or 10-year storm events. If grass and earth channels are too wide, the low flows will tend to incise a channel within the bottom, giving rise to both higher maintenance requirements and more channel instability when larger storms occur. Because flows of sufficient size to cause a low flow channel to form may not occur for several years, the magnitude of this problem may not be observed for several years.

When calculations illustrate the need for low flow channels, it is important to provide for notches in hydraulic structures to pass low flows. Without this provision, the low flows will not be confined and local aggradation may lead to failure of hydraulic structures and channels.

5.3.2.5 Safety: Section 4.3.2.7 contains a full discussion of safety, however, with regard to open channels, it may be necessary to provide Category I safety devices—primarily fencing—to preclude wet and dry weather access to drainage facilities that can be hazardous.

> Except as subsequently provided, fencing will be required for all new concrete, shotcrete, and soil cement lined channels with side-slopes steeper than 4:1 that meet a Class A hazard as defined in Table 4.4 (page 134). Subcritical concrete, shotcrete, and soil cement lined channels having depths and bottom widths less than 3 feet and 5 feet, respectively, will not require fencing. Fencing may be required by individual entities regardless of the conditions listed in this manual.

5.3.2.6 Maintenance: Maintenance considerations are an important factor in open channel design. Grass-lined channels require irrigation and mowing. Earth-lined channels need to be kept clear of vegetation and debris. Concrete-lined channels require periodic maintenance and may also require sediment removal after intense storms. Consider the required level of maintenance and obtain assurance of the proper level of maintenance for all designs.

5.3.3 Design of Artificial Channels

This manual provides the *Simplified Design Procedure* for normally encountered design problems in open channels that can be applied only to discharges less than 2,500 cfs. When a condition is encountered that is beyond the scope of this simplified procedure, an increase in the detail of analysis is required. Section 5.6 includes design checklists for artificial and natural channels. The basis of the checklists is for the designer to address the probable important factors on each assignment. For common, controlled conditions, there are simplified approaches to design that can be used; however, when a condition occurs that requires a higher level of technical approach, the design procedures require engineers qualified in open channel design techniques and more complex procedures.

The information contained in this section is to be used as the designer proceeds through the checklist items. As often noted in this manual, the intent is to provide design approaches and support for the most commonly encountered conditions. For more complex problems, a complete checklist and recommended minimum references to support the checklist are included in Section 5.6. These references are especially helpful for those facilities involving significant sediment transport issues or complex hydraulic structures for grade control and/or energy dissipation.

5.3.3.1 Design Criteria for the Simplified Design Procedure: This section addresses properties of channel cross sections. Methods of calculating channel bottom slope, hydraulic depths, and other hydraulic characteristics follow.

The parameters listed in Table 5.3 and within this section should be used as guideline values. The criteria in Table 5.3 *can* be adjusted, but only by making request to the Flood Control District and/or other regulating agency. The request must be accompanied by a specific detailed design prepared by a registered engineer using the long-form design procedure (checklist) outlined in Section 5.5. Conversely, in the course of design, conditions requiring more stringent criteria may be found. In this instance, use the more restrictive procedures and criteria.

Table 5.3 contains the channel properties to be used by designers of open channels using the *Simplified Design Procedure*. It should be noted that the channel properties change with the design discharge, which is a result of the hydraulic characteristics of open channels. As the design discharge increases, rigid channel design becomes increasingly more difficult to achieve. Safety considerations and the force of water combine to demand greater and greater design skills as the design discharge increases.

5.3.3.2 Hydraulics of Open Channels: For a relatively long, straight, and uniform channel, normal depth (i.e., uniform flow) calculations can be used to determine the discharge capacity at varying depths for a constant cross-sectional area. However, practicing engineers working in an urban environment will rarely encounter either existing conditions or design conditions where uniform flow calculations are adequate to totally define the flow conditions associated with a given discharge. Transition sections, channel junctions or confluences, channel bends, and hydraulic structures (e.g., culverts and bridges) can create major or minor deviations from uniform flow conditions. Therefore, the engineer must consider these deviations (covered elsewhere in this manual) when designing or analyzing drainage channels.

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

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

Table 5.3
Boundary Conditions for
Artificial Channel Properties
Simplified Design Procedure
(Q < 2,500 cfs)

Type of Channel Lining (1)	Side Slopes, H:V	Maximum Velocity, fps(2)
Concrete(3)	Vertical (4)	15
Soil Cement	2:1	9 (5)
Grouted Rock	3:1	9 (5)
Riprap	3:1	9 (5)
Gabion Baskets	2:1	9 (5)
Grass (irrigated and maintained)	4:1	2.5 to 8 (6)
Earth	4:1	2.5 to 6.0

(1) The values in this table are for channel sections with the same lining material for bottom and sides. For conditions where the bottoms and sides of the channel are different, the most critical applicable criteria are to be used.

(2) Maximum velocities listed for erodible linings are to be checked in each design to assure that erosion will not occur.

(3) The concrete lining classification also includes mortar or concrete pneumatically applied (shotcrete) (see Section 5.5.2.7, page 237).

(4) When using vertical sides, refer to Chapter 6 for design of reinforcement.

(5) Guideline only. Strict limits have not been set because this manual recommends that these channels flow subcritcally (see Critical Flow, page 217).

(6) Refer to Table 5.2.

Note: 1. Compute the Freeboard using Equation 5.9 (page 221), with minimums of 1 and 2 feet for channels designed for subcritical and supercritical flow, respectively. For curved channel sections, refer to Equation 5.10 (page 220).

2. The criteria listed in this table are boundary values. The designer is responsible for determining adequacy of criteria for each specific application. For design of lining materials, analyses of soil conditions and subsurface drainage may be required by the Flood Control District and/or other regulating agencies.

uniform flow; however, uniform flow is more often a theoretical abstraction than an actuality. The engineer must be aware that uniform flow computation provides only an approximation of what will occur; however, such computations are useful for planning.

The normal depth is computed so frequently that it is convenient to use special graphs for various types of cross sections to eliminate the need for time consuming trial and error solutions (see Figures 5.13 and 5.14, pages 258 and 259).

Equations 5.1 through 5.11 are presented to aid in determining fundamental quantities in open channel flow—such as normal and critical depth and specific energy and design considerations of freeboard and minimum radius of curvature. See Section 5.2 for definitions of the symbols used in these equations.

Generally, it is necessary to apply Manning's Formula (Equation 5.1) to sections of the channel which have similar properties:

$$V = (1.49 / n)\, R^{0.67} S_o^{\,0.5} \tag{5.1}$$

Multiplying velocity by the cross sectional area of flow:

$$Q = VA \tag{5.2}$$

Because of variable channel cross sections and channel properties, uniform flow computations are rarely used solely as the basis for open channel design. Normally, a designer will use these values for conceptual level decisions. Decisions relative to preliminary and final design requirements should be made through backwater determinations (see Section 5.4).

Critical Flow: Critical flow in an open channel or covered conduit with a free water surface is characterized by several conditions. Some of them are:

» The specific energy is a minimum for a given discharge.

» The discharge is a maximum for a given specific energy.

» The velocity head is equal to half the hydraulic depth in a channel of small slope.

$$h_v = V^2/2g = D/2 \tag{5.3}$$

» The Froude Number is equal to 1.0.

With the presence of a free surface in an open channel, the force of gravity has an effect on the state of flow. The effect of gravity on open channel flow is represented by the ratio of inertial forces to gravitational forces. This ratio is known as the Froude Number (F) and is computed by rearranging Equation 5.3:

$$F = V/(gD)^{0.5} \quad = \frac{V^2}{D \cdot g} \tag{5.4}$$

For subcritical and supercritical flows F < 1 and F > 1, respectively.

The Froude Number will be used in this chapter as a design guideline. Avoid sediment deposition due to low velocities (below 2 fps).

Due to erosion and scour of erodible channels and safety concerns with excessively high velocities, the recommended upper limit of F is 2.0.

The specific energy (E) in open channels is the sum of the depth of flow and the velocity head:

$$E = Y + V^2/2g \tag{5.5}$$

A characteristic specific energy curve is made for a specific flow rate in a channel (see Figure 5.16, page 261). Two depths of flow, called alternate depths, will exist for each energy value; the lower depth will be supercritical and the higher depth will be subcritical. As the energy approaches a minimum, toward Y_c, the alternate depths will vary by small amounts. Minor changes in cross section, flow rate, channel roughness, or slope can cause the flow to pass through critical flow and on to its alternate depth—therefore, avoid the regime of flow near minimum specific energy. This flow phenomena is characterized by a wavy water surface, weak hydraulic jumps, and unsteadiness in the flow.

As a guideline, F should not fall between 0.85 and 1.3.

In the subcritical region, as F increases toward 1, the flow speeds and reduces in depth. As long as the velocity does not increase past the permissible velocity shown in Table 5.3, short-lived instability of near critical flow is acceptable. On the other hand, near critical flows in the supercritical regime may cause hydraulic jumps with the minor changes in cross section, flow rate, channel roughness, or slope, as mentioned above. This type of flow instability is undesirable for any duration, therefore supercritical flow will be permitted only if the flow is well established in the supercritical flow regime.

The guidelines in this manual restrict soil cement, grouted rock, riprap, and grass and earth lined channels to subcritical flow.

The limit to the Froude Number is thus $F \leq 0.85$. For concrete, shotcrete, and mortar lined channels, the additional range of $1.3 \leq F \leq 2.0$ is allowed.

Since critical flow is to be avoided, it is important to be able to calculate the critical depth, Y_c. Substituting Q^2/A^2 for V^2 in Equation 5.3 yields:

$$Q^2/g = A^3/T \tag{5.6}$$

Equation 5.6 is solved by successive approximation, and will only be satisfied for critical flow for the given discharge and cross section. Manipulations of Equations 5.3 through 5.6 will yield simplified expressions to determine critical depth for common prismatic channels (see *Handbook of Hydraulics*, King and Brater 1963). Only Equation 5.6 is included here because it is a general relation that will be satisfied for a channel of any cross section. The successful approximation of Y_c can be verified by checking whether Equation 5.4 has been minimized.

In applying Equation 5.6 and solving for F in Equation 5.4, if the depth is found to be at or near critical, the shape of the cross section or of the slope should be changed to achieve greater hydraulic stability.

To simplify the computation of critical flow, Figure 5.14 (page 259) gives dimensionless curves showing the relation between depth and the section factor, F_c, for rectangular, trapezoidal, and circular channels:

$$F_c = A(D)^{0.5} \tag{5.7}$$

Roughness Coefficients: Roughness coefficients (n) for use in Manning's equation vary considerably according to type of material, depth of flow, and quality of workmanship. Table 5.11 (page 257) lists roughness coefficients for pipes, earthen and natural channels and for various artificial channels.

If unsure of a specific value of roughness, the designer should check the possible range of roughness coefficients to locate potential problems.

5.3.3.3 Design of Rigid Channels: To be able to use the *Simplified Design Procedure,* the channel must be less than 2,500 cfs and "rigid." To be considered rigid, both the banks and the channel bottom must be stable. This generally results from the channel layout and from grade control.

Layout: In general, channel layout should follow existing washes, swales, or depressions.

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

This requirement applies both to the point where water becomes channelized and the point where runoff leaves the channel. This requirement is legal in nature as well as a matter of preventing erosion and property damage that would not have occurred without the drainage work being constructed. In addition, the water surface along the route *cannot* be raised so that damages occur as if improvements had not been made.

Care should be taken not to choose routes which lengthen the channel sufficiently to reduce channel slopes below that which will cause sediment deposition during low flows. Use Equations 5.1 and 5.2 to verify this condition. Likewise, channel layout can be used to reduce excessive channel slopes and the amount of grade control structures that are required.

It is most important to achieve a good channel layout in conceptual and preliminary layout of the surrounding land use. In general, the radius of curvature should not be less than three times the design flow top width.

Grade Control: There are many references to grade control requirements throughout this chapter, and it is difficult to overstate its importance. This section describes the benefits of grade control and the need for it in the design and assessment of channel stability. Actual design of man-made grade control structures is discussed in Chapter 6.

Grade control must be established as a condition of using the *Simplified Design Procedure.* It is a critical factor in the behavior of non-rigid channels. In its basic form, grade control can be any natural or man-made section of a channel that does not permit channel degradation or aggradation. Grade control is most often thought of as causing water to pass through critical depth; however, this condition is not required to establish grade control. 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. Channels with steep bottom slopes that cause channels to meander can wash out embankments, as can culverts and bridges plugged by debris or that are too small for the flood event that occurs.

Grade control and channel slope are interrelated. It does little good to establish grade control within a specific reach of open channel, when the channel downstream is headcutting or undergoing rapid deposition. When designing artificial channels, the designer needs to assess the stability of the section(s) immediately downstream from the segment under design. If there is evidence of ongoing downstream degradation, a grade control structure will be required—at a minimum—to a depth sufficient to preclude further headcutting in the channel.

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 at the end of this chapter.

For each alternative investigated, the selected channel slope should result in a stable channel, particularly for earth-lined channels. Within a reach of artificial channel, grade control structures should be used as required to meet the requirements listed in Table 5.3.

Channel Curvature: For channels with Froude Numbers less than 0.85, the ratio of the channel radius (at centerline) to the design width of the water surface shall be greater than 3.0.

Note for Froude Numbers greater than 0.85, the radius of curvature will be computed from the formula:

$$r_c \geq 4V^2T/gY \tag{5.8}$$

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. This rise in the water surface is referred to as superelevation. The shift in the velocity may cause cross-waves to form, which will persist downstream when the flow is *supercritical.* Severe erosion, deposition and reduced channel performance result from severe curvatures in channel alignment. To minimize the effect due to channel bends, channel curvature should only be used where topographic or other conditions necessitate their use. If the flow is *supercritical,* special design criteria may need to be employed to eliminate the downstream effects.

For superelevation under subcritical conditions, the following formula is generally used:

$$\Delta y = V^2 T/gr_c \tag{5.9}$$

The freeboard requirements should be added to the superelevated water surface elevation.

For supercritical flow, the disturbance caused by a bend in the channel persists downstream. Therefore, a detailed hydraulic study must be conducted to determine the effects of the channel curvature on the freeboard requirement. This section includes a discussion of channel curvature.

Freeboard: Required freeboard is computed according to the following formula:

$$FB = 0.25\,(Y + V^2/2g) \tag{5.10}$$

The minimum freeboard value for rigid channels shall be 1 foot for subcritical and 2 feet for supercritical flows.

Additional requirements for freeboard may be called for in specific cases where aggradation is substantial during a single flow event and/or superelevation must be taken into consideration.

Low Flow Channels: For channels with grass or earth bottoms, it is recommended that low flow channels (see Figure 5.1, page 209) be considered whenever the following condition exists:

$$b/(VY) \geq 1.40 \tag{5.11}$$

where V and Y are values for the 100-year flood.

The existence of frequent grade control structures may also preclude the requirement for compound channel sections; however, where grade control structures are used in conjunction with low flow channels, the hydraulic structure should be matched to pass flows within the low flow channel.

Supercritical Flow: Supercritical flow in an open channel in an urbanized area creates certain hazards which the designer must take into consideration. From a practical standpoint, it is generally unwise to have any curvature in a supercritical channel. Careful attention must be taken to insure against excessive oscillatory waves which may extend down the entire length of the channel from only minor obstructions upstream.

In a supercritical channel, there shall be no change of cross-sectional shape or area at bridges or culverts. Bridges or other structures crossing the channel must be anchored satisfactorily to withstand the full dynamic load which might be imposed upon the structure in the event of major trash plugging. Concrete linings must be protected from hydrostatic uplift forces which are often created by a high water table or momentary inflow behind the lining from localized flooding.

Backwater computation methods are applicable for computing the water surface profile or the energy gradient in channels having a supercritical flow; however, the computations must proceed in a downstream direction. The designer must take care to insure against the possibility of unanticipated hydraulic jumps forming in the channel.

Design of deflections and curves for supercritical flow is beyond the scope of this manual; refer to *Handbook of Hydraulics* (King and Brater 1963) and *Open Channel Hydraulics* (Chow 1959).

5.3.3.4 Preliminary Design: It is important that major design issues be identified and decisions about them made before proceeding into final design. In addition to the master plan consideration of the initial route, downstream control, and channel type, there are a number of technical items that must be evaluated prior to commencing final design. Some of these issues are:

» Control flow at the beginning and end of a channel reach to prevent damage to existing channels.

» Eliminate potential hydraulic jumps (this should only be an issue in concrete and shotcrete channels). Hydraulic jumps will be permitted only in planned locations at hydraulic structures (see Chapter 6).

» Minimize the use of alignment deflection in supercritical flow.

» The degree to which low flows affect channel stability and maintenance should be reflected in the chosen channel cross section.

» Determine safety requirements in conjunction with other hydraulic structures and with the inlets and outlets of conduits.

» Plan for possible secondary uses that can reduce urban costs by providing other benefits.

» Determine the need for freeboard requirements that may exceed those discussed in Table 5.3.

5.3.3.5 Final Design: Unless exempted by the governing entity, 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 backwater methods, taking into consideration all losses due to changes in velocity, drops, bridge openings, and other obstructions (see Section 5.4). Other than supercritical concrete lined facilities, computations begin at a known point and extend in an upstream direction for subcritical flow; this is why the channel should be designed from a downstream direction to an upstream direction. It is necessary to show the energy gradient on all preliminary drawings to help insure against errors. Whether or not the energy gradient line is shown on the final drawings is optional.

Remember that open channel flow in urban drainage is usually non-uniform because of bridge openings, curves, and structures, so backwater computations must be used for all final channel design work (see Section 5.4).

5.3.3.6 Design of Non-Rigid Channels: Large washes and locations where urbanized channels discharge into non-urbanized areas are the most likely candidates for this type of design. Non-rigid channel design requires special design skill and experience; therefore, design parameters and procedures are not described in this manual. However, a checklist for design requirements and a list of references that address non-rigid channel design are included in Section 5.6.

Non-rigid channel design can have economic benefits through reduced channel sections using movable beds and fixed sides, or permitting a channel to seek its own equilibrium without constructing drainage facilities. In these instances, designers may be required to develop calculations to prove channel stability or to prove that the channel will maintain its course within certain specified limits.

5.4 Natural Channels

In Maricopa County, floodplains tend to be wide, braided, and not permanently fixed in one location. Furthermore, velocities tend to be high, causing channel modifications to be difficult to effect. Designers working in Maricopa County are, therefore, interested in the course of natural channels, especially where development will occur. Other design interests include the existing washes into which drainage works from development will empty, and changes that occur in natural channels due to increases in the volume of runoff that occurs from urbanization.

Floodplain analysis tends to be very complex; however, a basic understanding of the behavior of natural channels and the methods of analysis can be very useful in the overall approach to drainage design. A common requirement for many projects is determining water surface profiles.

Natural channels tend to be in a steady state of change. Mountainous streams can be rigid, yet, in a geologic framework, are in a constant state of headcutting. While some mountainous regime channels exist, the natural channels that most commonly occur in Maricopa County lie within alluvial materials that have been deposited over long periods of time.

At transitions from natural to artificial channels, an array of problems can occur. Erosion can occur where the artificial channel has a substantial increase in conveyance compared to an upstream, natural channel. Because of their steep nature, the most common problem associated with mountainous streams is the sedimentation that predictably occurs where the natural channel interfaces with artificial channels which confine flows through development. Low flow channel sections may be required in the artificial channels to move sediment. Sedimentation can be expected where attempts are made to sharply deflect the direction of flow from the naturally steep channels, but this condition should be avoided.

In the more common alluvial cases, natural channels tend to deposit sediment and meander during low flow periods (which is most of the time) and to erode and straighten channel alignments during rare events. It is in this manner that the alluvial fans have been formed. Generally, if alluvial material exists, then there is some potential for the stream to reoccupy the alluvial areas resulting from a period of high flows. Therefore, it is necessary to acknowledge the potential for a natural channel to be 'non-rigid.'

Floodplain analyses tend to overlook this tendency for a natural channel to move, and, in many instances, this is an acceptable approach; however, the strict use of this approach within urbanized areas can lead to unfortunate results. As a result, there is often a need to utilize bank protection and hydraulic structures to selectively transform a non-rigid, natural channel into a more rigid channel.

Unless a natural channel is steadily aggrading or degrading, the construction of roads with adequately sized and protected bridges will significantly limit the lateral movement capability of most channels. Notable exceptions in Maricopa County are the extensive alluvial fan areas, which are outside the scope of this manual.

Combined with bridges, the selective use of bank protection and grade control structures can prevent a natural channel from moving over a wide range of flow rates.

The entire hydrological approach to converting a natural waterway which has historically transported water from rural lands to an urban major drainage channel is so complex that applicable design criteria cannot be presented completely in this manual. It will suffice here to state that the planning for use of such channels must be undertaken with the full benefit of engineers with adequate experience in open channel flow, bed stability, and sediment transport, together with experts in related fields.

5.4.1 Analysis of Natural Channels

The investigations necessary to insure that a natural channel will be adequate are different for every waterway. Supercritical flow cannot always occur in natural channels and frequent checks should be made during the course of the backwater computations to insure that the computations do not reflect supercritical flow.

Because of the advantages which are available to a community by utilizing natural waterways for urban storm drainage purposes, the designer should consult with experts in related fields as to the methods of development. Where natural channels are used, the usual rules of freeboard depth, curvature, and other rules applicable to artificial channels do not apply. For instance, when laid out and developed for the purpose of being inundated during the major runoff peak, there can be significant advantages if the designer incorporates into the planning the overtopping of the channel and localized flooding of adjacent areas. Using natural channels requires that primary attention be given to erosive tendencies and carrying capacity adequacy. The floodplain of the waterway must be defined so that adequate zoning can take place to protect the waterway from encroachment to maintain its capacity and storage potential.

Section 5.6 contains a checklist of technical issues that need to be addressed when analyzing waterways in the vicinity of bridges and culverts. A general approach for analyzing the effectiveness of natural channels include several tasks:

» Prepare cross sections of the channel for the major design runoff.

» Investigate the bed and bank material as to the particle size classification.

» Study the stability of the channel under future conditions of flow.

» Examine channel and overbank capacity to determine adequacy for 100-year runoff.

» Examine velocities in natural channels to verify that critical velocity is not exceeded for any section.

» Define water surface limits so that the floodplain can be zoned.

» Use roughness factors ("n" values) which are representative of non-maintained channel conditions.

» Divide the channel cross sections into units of similar properties for determination of water surface profiles.

» Specify the use of drops or check dams to control water surface profile slope, particularly for more frequently occurring storm runoff.

» Prepare plans and profiles of the floodplain. Make appropriate allowances for future bridges which will raise the water surface profile and cause the floodplain to be extended.

» Evaluate the freeboard with reference to proposed non-drainage structures.

Filling of the flood fringe reduces valuable storage capacity and tends to increase downstream runoff peaks. Filling should not be used in the urban waterways where hydrographs tend to rise and fall sharply.

5.4.1.1 Requirements for Natural Channels: Washes which traverse land designated for a proposed development may be left in their natural state provided that doing so would not be in conflict with an approved master drainage plan for the area—if one exists—and provided that the development is adequately protected from flooding and erosion. According to ARS 48-3609.A, during the course of the Master Planning process, the 100-year runoff will be used to delineate a floodplain for major channels with discharges of more than 1,000 cfs and will be processed through the local government, ADWR, and FEMA.

One method of developing in the vicinity of a natural wash is to keep all structures out of its 100-year floodplain, as well as its attendant erosion-hazardous areas. Another possible method of developing in the vicinity of natural washes is to utilize part of the floodplain for development, while leaving the channel in its natural state. However, the approach would involve demonstrating that: 1) the encroachment would not adversely affect adjacent properties; and 2) the development would be located outside of any erosion-hazard areas which border the natural wash.

Encroachments into the floodplain of a natural wash are to be analyzed according to the FEMA requirements. The maximum rise in water surface shall not exceed those listed for the local regulating entity. As with all floodplain encroachments, the development must be adequately protected from flooding and erosion, and must not violate restrictions imposed by master drainage plans. At no time should an encroachment adversely affect the river's stability or adversely alter flooding conditions on adjacent properties. When encroachment is proposed within the floodplain of a major watercourse, the regulating entity may, at its discretion, request that a detailed study be performed to determine if a reduction in overbank flood storage will significantly affect downstream flood peaks.

5.4.1.2 Applicable Methodologies

Normal Depth and Velocity: If the depth and direction of the flow in an open channel are nearly constant (i.e., steady, uniform flow conditions), the flow regime is said to be "normal," and the hydraulic characteristics of the channel can be evaluated by using the Manning's Equation (Equation 5.1).

When delineating natural floodplains using the Manning's equation, it is important to ensure that the energy grade line (EGL), slopes continuously in the downhill direction. The energy grade line is defined as a line connecting points of known total head or total specific energy, E, as computed by Equation 5.5.

In those cases where the slope of the energy grade line does not nearly equal the channel bed slope, then it is not reasonable to use a uniform flow approach, and backwater calculations must be made.

Backwater Procedure: The previous section contained a brief discussion on computing normal depth, which assumes that changes in discharge, bed slope, and cross-sectional area and form, occur relatively gradually, however, sudden changes will produce additional turbulent energy losses which are not accounted for in the 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 exists, the Standard Step Method should be used for evaluating water surface profiles. The procedure for making Standard Step calculations is given in several easily obtainable textbooks or references. An example form for the method is shown in Figure 5.15 , page 260. Computer facilities are available, and the designer should perform the Standard Step calculations by using the readily available and well-documented computer program HEC-2, written and distributed by the U.S. Army Corps of Engineers.

One advantage of the Standard Step Method is that if the computation is started at an assumed elevation that is incorrect for the given discharge, the resulting flow profile will become more nearly correct with each succeeding cross section evaluated within a 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 to correct 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 tend to make the results diverge from the correct flow profile.

There are a large number of applications related to open channel flow in which the shape, or class of flow profiles are important to the designer. Figure 5.3 has been included as a reference to determine the type of flow profile that may exist in an artificial or natural channel.

5.4.1.3 Floodplain Widths and Depths in Channels with Composite Hydraulic Roughness: In the following sections, general analytical procedures are presented for evaluating floodplain hydraulics, with an emphasis on determining floodplain widths and flow depths in natural washes and constructed channels having non-uniform or composite hydraulic roughness.

Composite Channels: The cross section of a 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

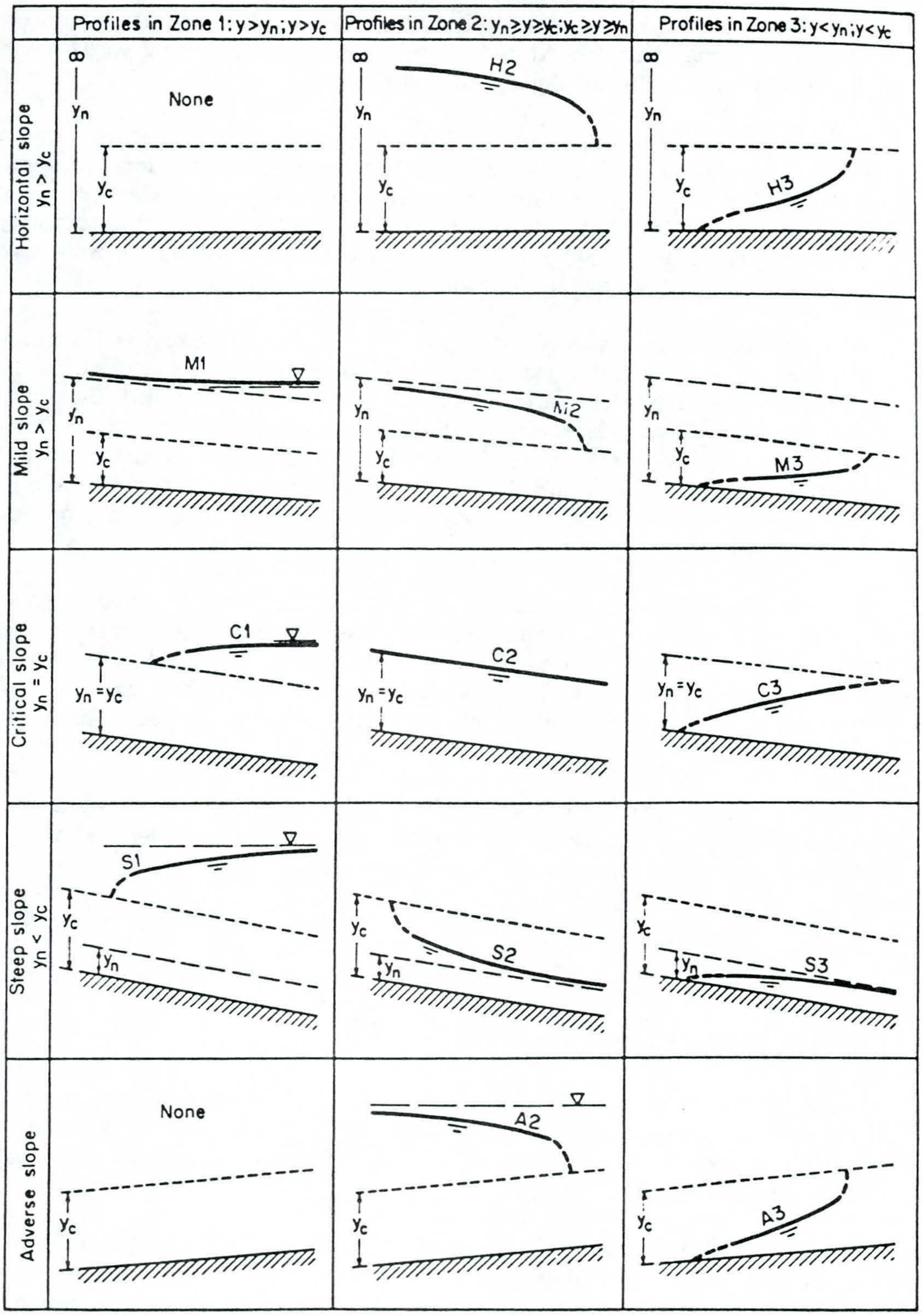


Figure 5.3
Classification of Flow Portion of Gradually Varied Flow
(Chow 1959)

example, an alluvial channel may have a primary, sand-bed channel which is bounded on both sides by densely-vegetated, overbank floodplains, or a flooded street section may be bounded on both sides by landscaped front yards having shallower flood depths and slower flow velocities.

In composite channels like these, the discharge is computed for each sub-section 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.

Manning Roughness Coefficients: Manning roughness coefficients (see Table 5.11, page 257), for use in water surface profile calculations, should be carefully estimated by experienced engineers. The estimates should include consideration that roughness may vary with flood stage, depending on such factors as the width-depth ratio of the wash; presence of vegetation in the main channel and the overbank areas; the types of materials making up the channel bed; and the degree of meandering. Additional information concerning Manning roughness coefficients may be found in Chapter 4.

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 roughness coefficients for the overbank areas. When faced with such a 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: use the adjusted urban roughness coefficient, n_u, with the total cross-sectional area, (Hejl 1977). See Figure 5.4.

$$n_u = n_o\,(1.5\,(W_T/\Sigma W_o) + (1 - W_T/\Sigma W_o)\,\Sigma L_o/L_T - 0.5) \qquad \textbf{(5.12)}$$

where all coefficients are as defined in Section 5.2.

5.4.1.4 Related Issues

Maintenance: In many instances, specific maintenance access requirements are required by the local entity and/or the Flood Control District. In planning and designing open channels, the designer should determine these requirements at the outset of the project.

Development in Floodplains: Development in areas outside of the floodway in designated floodplains is permissible as long as zoning regulations are met. The advance planning of developments that may be affected by the floodplain regulations requires a thorough understanding of floodplain regulations and drainage standards and may also require an engineering background in open channel hydraulics, river mechanics, and sediment transport.

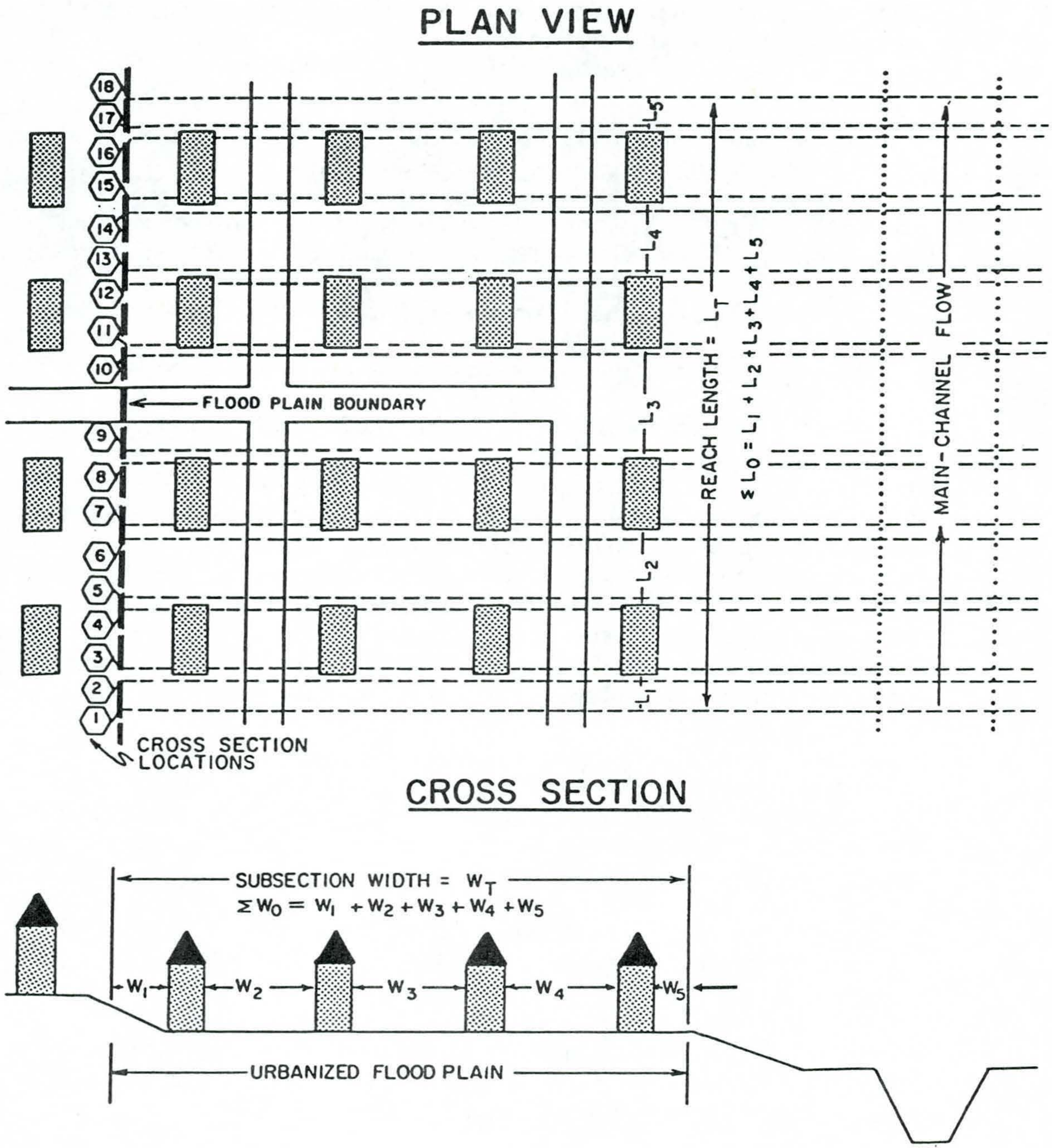


Figure 5.4

Diagram of Idealized Urban Floodplain

(Simon, Li and Associates 1988)

5.5 Channel Linings

The type of channel lining which 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, and compatibility with existing improvements at the site. The following lining types are acceptable within their range of applicability.

5.5.1 Soil Cement

Soil cement has been shown to be an effective and economical method for slope protection and channel lining in the Maricopa County area.

5.5.1.1 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 contain no material retained on a 2-inch (50 mm) sieve;

2. At least 55 percent of the material pass the No. 4 (4.75 mm) sieve; and

3. Between 5 percent and 35 percent pass the No. 200 (0.074 mm) sieve.

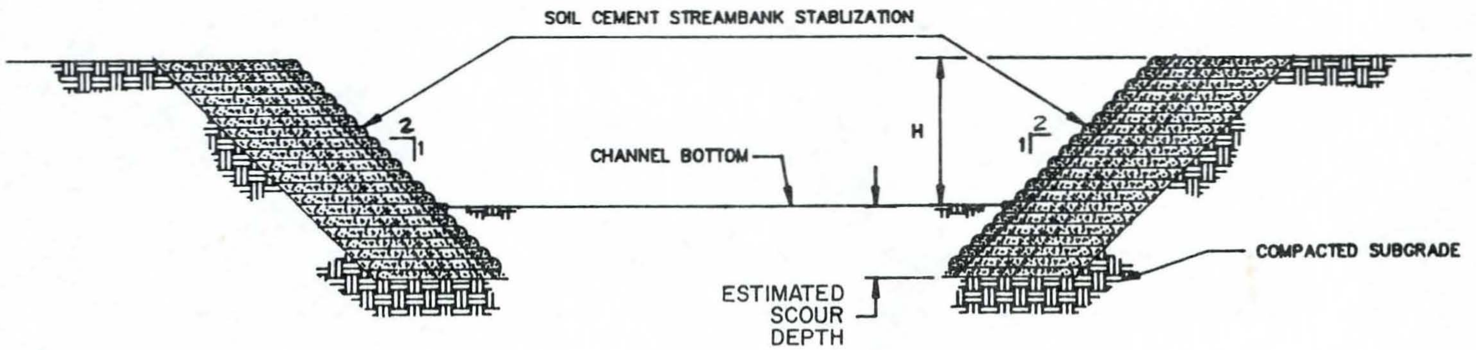
If the amount of material passing the No. 200 sieve exceeds 35 percent, the addition of a coarser material may be necessary. Soils containing more than 35 percent material passing the No. 200 sieve may be used if the fines are non-plastic and can be adequately mixed with cement. Standard laboratory tests (ASTM D558, D559 and D560) 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 on-site soil materials, or soils borrowed from nearby areas, are mixed with Portland cement and water and transported to the site for placement and compaction. Mixed in-place construction has also been used successfully.

5.5.1.2 Design of Soil Cement Linings: Figure 5.5 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 a 7- to 9-foot minimum



Soil Cement Streambank Stabilization
Typical Section
(Simons, Li and Associates 1987)

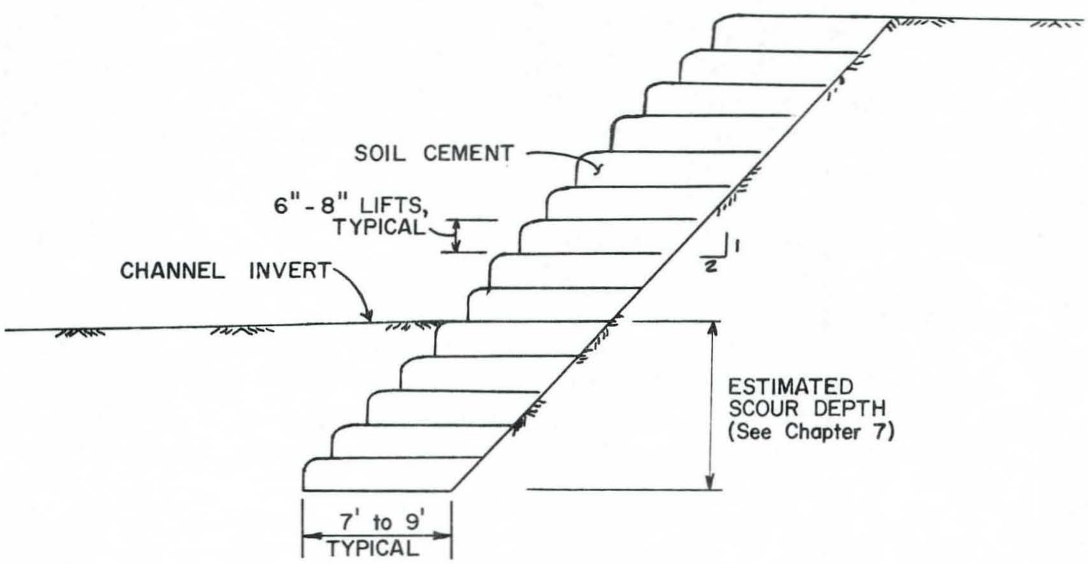


Figure 5.5
Soil Cement Placement Detail
(Not to Scale)

working width is required for placement and compaction of the soil cement layers by standard highway construction equipment. Figure 5.6 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 8 feet, a side slope of 3:1 and 6-inch thick layers, the resulting minimum thickness of facing would be about 2 feet, measured normal to the slope.

Soil cement may also be placed in layers parallel to the slope, as shown in Figure 5.7. The maximum side slope for this "slope paving" technique is 4:1, with flatter side slopes preferred.

The minimum thickness for the soil cement lining shall be 12-inches (two 6-inch lifts).

An important consideration in the design of the soil cement facing is to ensure 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.

To protect against undermining of the soil cement layer by lateral inflows, the top of the lining should be keyed into the ground, as shown in Figure 5.2. As with any impervious channel lining system, seepage and related uplift forces should be considered and appropriate counter-measures provided, such as weep holes or subdrains. Tributary storm drain 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 scour.

5.5.2 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; and 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 which 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.

5.5.2.1 Flow Regime Considerations: Concrete lined channels are usually designed for flow conditions where the Froude Number exceeds 1.3 and/or when velocities exceed five feet per second for earth lined channels. Froude Numbers for supercritical flow shall be greater than 1.3 and less than 2.0. Critical flow conditions, where $0.85 \leq F \leq 1.3$, are very unstable, and must be avoided.

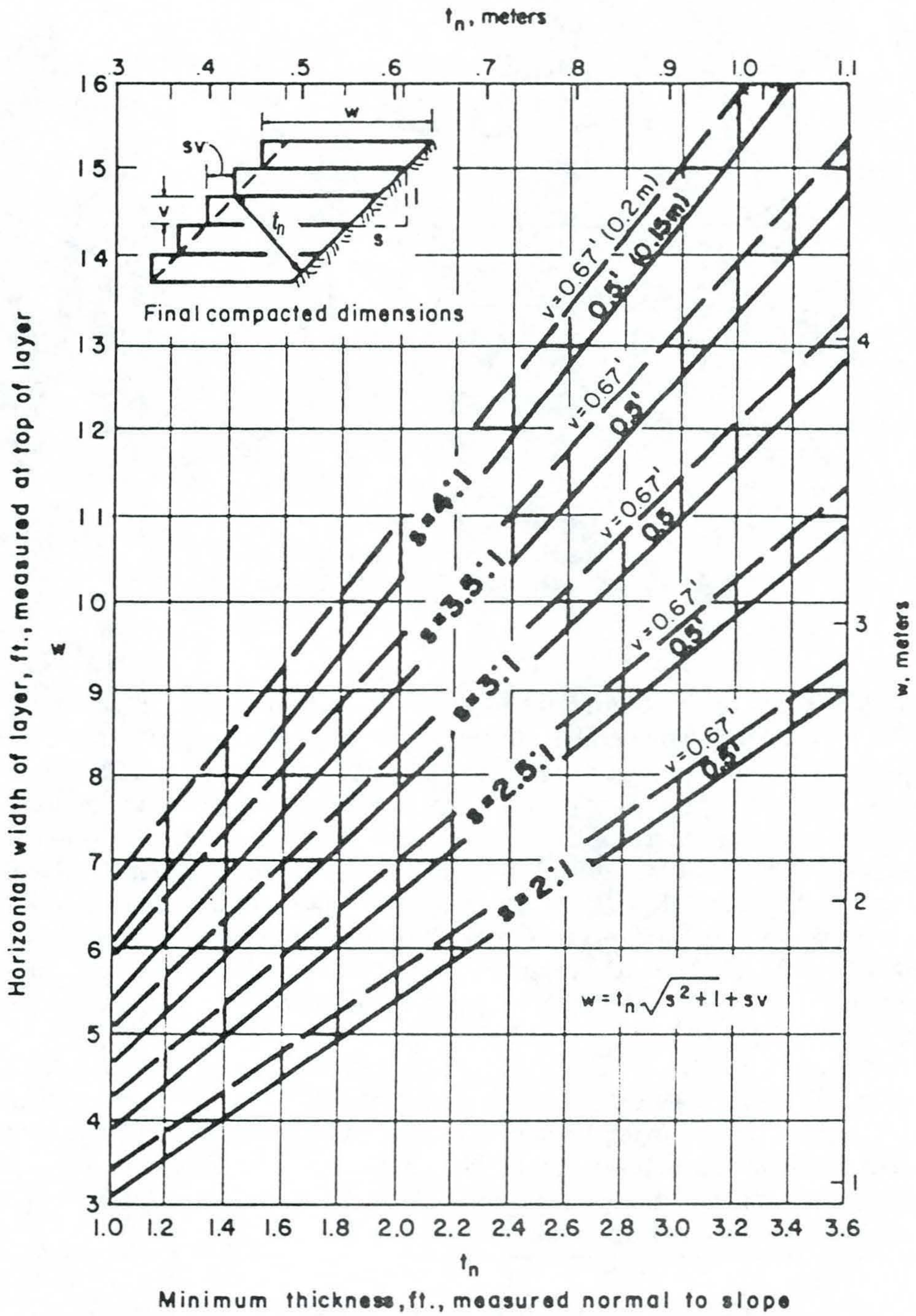


Figure 5.6
Relationship of Slope, Facing Thickness, Layer Thickness and Horizontal Layer Width for Soil Cement Lining
(Portland Cement Association, Undated)

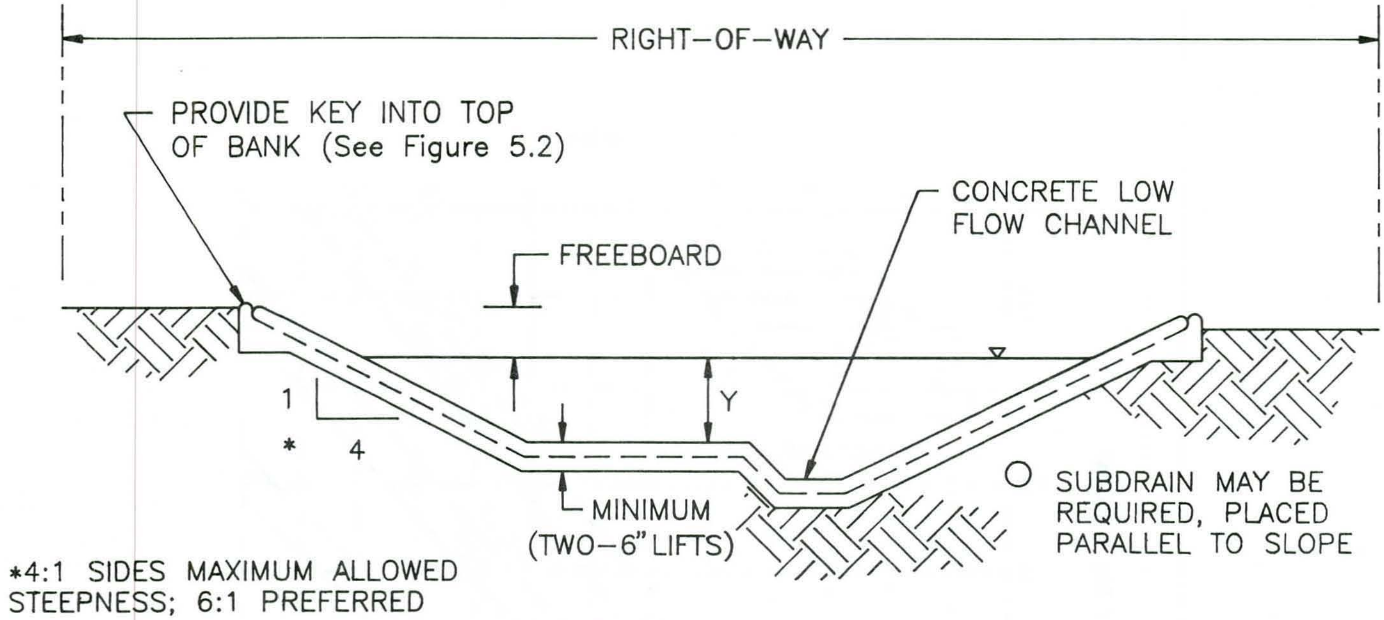


Figure 5.7
Soil Cement Lined Channel
(Excluding Control Structures)
(Pima County Department of Transportation and Flood Control District 1984)

Supercritical flow in an open channel in an urbanized area creates certain hazards which the designer must take into consideration. From a practical standpoint it is generally unwise to have any curvature in a supercritical channel. Careful attention must be taken to prevent or control excessive oscillatory waves which may extend down 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 channels carrying supercritical flow shall be lined with continuously reinforced concrete extending both longitudinally and laterally. There shall be no reduction in cross sectional area at bridges or culverts. Freeboard shall be adequate to provide a suitable safety margin—at least 2 feet—or an additional capacity of approximately one-third of the design flow.

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

5.5.2.2 Lining Criteria: Generally, if slopes steeper than 2 : 1 are used, then safety and structural requirements become a primary concern. The thickness of the lining should be a minimum of 3-inches depending on channel capacity and stability against hydraulic forces and other forces acting on the channel. Design of the lining should 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 overbanks as shown on Figure 5.2, page 210.

5.5.2.3 Roughness Coefficient: The roughness coefficient for a concrete lining can vary from 0.013 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.

5.5.2.4 Bedding: 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 channel lining with concrete.

5.5.2.5 Transitions: 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 to ensure control of flood flows, prevent undermining of the lining and reduce turbulence. Transitions should be lined with concrete or riprap 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 specialized analyses that are beyond the scope of this manual. References are listed at the end of this chapter.

5.5.2.6 Underdrainage: 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 which 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 5.8 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 underdrains refer to *Lining for Irrigation Canals* (USBR, undated).

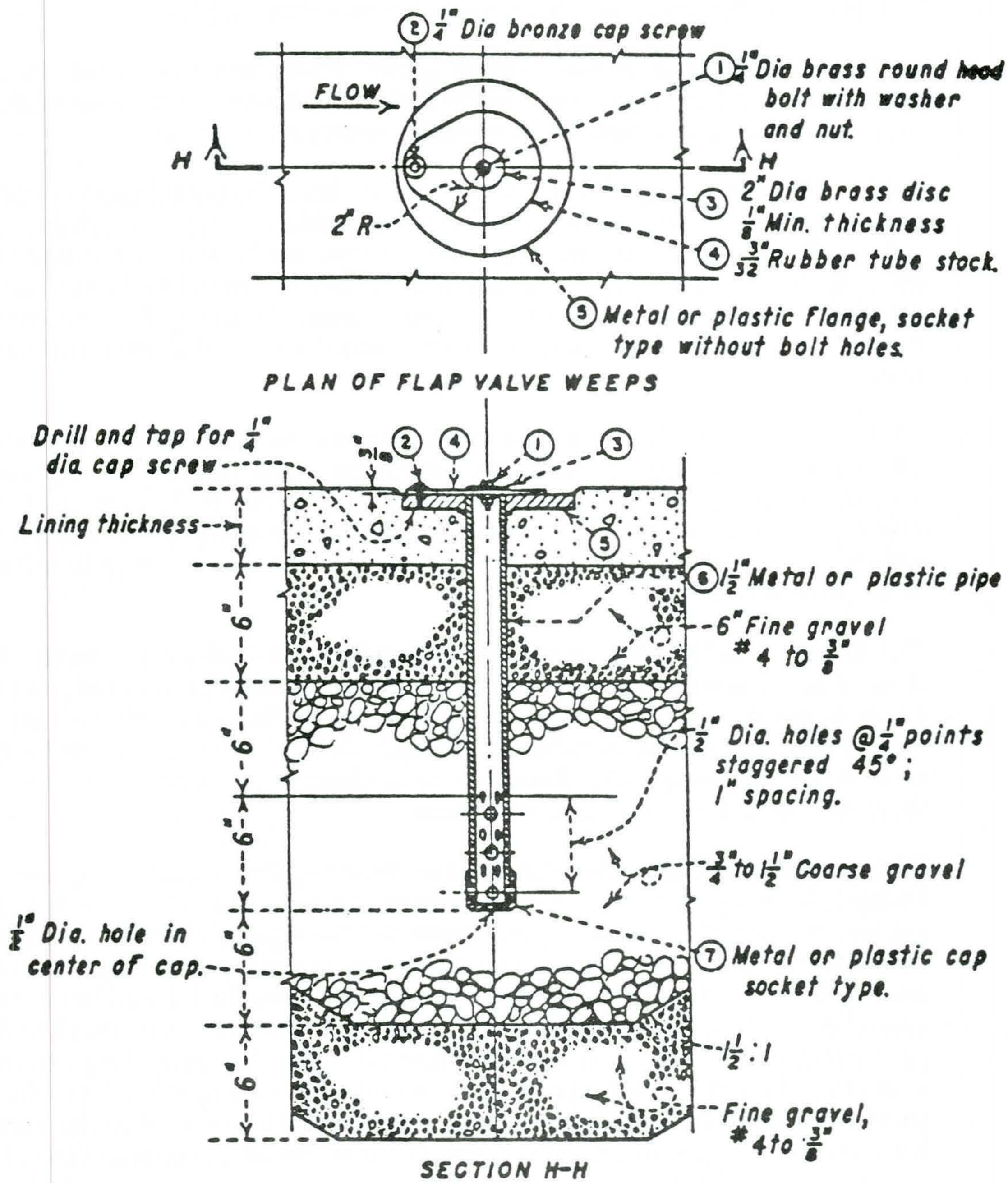


Figure 5.8
Flap Valve Installation for a Channel Underdrain
(Simons, Li and Associates 1981)

Where a lesser degree of seepage control is warranted, weep holes spaced at appropriate intervals may be used. When embankment safety may be compromised or when ground levels may be raised by draining 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.

5.5.2.7 Shotcrete: 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.

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 in Section 5.5.2 apply in the design of shotcrete channels.

The designer must adhere to the local regulations of Section 525 of the *Uniform Standard Specifications for Public Works Construction* by the Maricopa Association of Governments (MAG 1979, rev. 1984), and Section 912 of the Arizona Department of Transportation's *Standard Specifications for Road and Bridge Design Construction* (1990). The nozzle operator should be certifiable under ACI 506.

5.5.3 Riprap Lined Channels

Graded 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; and

» river conditions.

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

5.5.3.1 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 Maricopa 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. Rock derived from igneous and metamorphic sources provides 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 (see Section 5.6) and absorption test (see ASTM C127). Based on these tests, the durability absorption ratio (DAR) is computed as follows:

$$DAR = \frac{Durability\ Index}{Percent\ Absorption + 1} \qquad \textbf{(5.13)}$$

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; and,

   b. Durability index 51 or less, material is rejected.

Shape: There are two basic shape criteria. First, the stones should be angular. Second, not more than 25 percent of the stones should have a length more than 2.5 times the breadth. 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.

5.5.3.2 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}. A gradation coefficient, G, of 1.5 is required.

$$G = 0.5\,(D_{84}/D_{50} + D_{50}/D_{16}) \tag{5.14}$$

Table 5.4 provides design gradations for specified classes of 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.

Thickness: The riprap-layer thickness is to be at least 1.5 times the D_{100} value, but need not exceed twice the D_{100} value. The thickness is measured perpendicular to the slope upon which the riprap is placed.

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:

$$(D_{15})\,filter \leq 5(D_{85})\,bank \tag{5.15a}$$

$$4(D_{15})\,bank \leq (D_{15})\,filter \tag{5.15b}$$

$$(D_{50})\,filter \leq 25\,(D_{50})\,bank \tag{5.15c}$$

If the inequalities are satisfied by the riprap itself, then no filter blanket is required. If the difference between the bank 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 5.5 outlines recommended standard gradations.

Table 5.4
Design Gradation for Specified Classes of Riprap

Percent Passing	Size	D_{50} Class, Inches						
		6	8	12	18	24	30	36
100 to 90	2.0 D_{50}	12	16	24	36	48	60	72
85 to 70	1.5 D_{50}	9	12	18	27	36	45	54
50 to 30	1.0 D_{50}	6	8	12	18	24	30	36
15 to 5	0.67 D_{50}	4	5	8	12	16	20	24
5 to 0	0.33 D_{50}	2	3	4	6	8	10	12

Table 5.5
Gradation for Gravel Bedding
(Simons, Li and Associates 1989)

Standard Sieve Size	Type I	Type II
	(Percent Passing by Weight)	
3 inches	—	90 to 100
1-1/2 inches	—	—
3/4 inch	—	20 to 90
3/8 inch	100	—
#4	95 to 100	0 to 20
#16	45 to 80	—
#50	10 to 30	—
#100	2 to 10	—
#200	0 to 2	0 to 3

The Type-I and Type-II bedding specifications shown in Table 5.5 were developed using the criteria given in Equation 5.14, 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 5.5 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 5.6.

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 carefully 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.

Table 5.6
Thickness Requirements for Gravel Bedding

Riprap Classification	Minimum Bedding Thickness, inches		
	Fine Grain Native Soils		Coarse Grain Native Soils
	Type I	Type II	Type II
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

5.5.3.3 Hydraulic Design Requirements

General: Channel linings constructed of placed, graded riprap or wire enclosed rock to control channel erosion have been found to be cost effective where channel reaches are relatively short (0.25 mile or less) and where a nearby source of quality rock is available.

Situations where riprap or wire enclosed rock 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 enclosed rock linings; and

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

This section presents design requirements for graded riprap, while Section 5.5.5.3 contains additional design considerations specifically related to wire enclosed rock. Both Sections are valid only for subcritical flow conditions where the Froude Number is 0.85 or less.

Riprap Sizing: The procedures described in *Hydraulic Design of Flood Control Channels* (Corps of Engineers, EM-1601, 1970 or latest revision) shall be used for the design of graded riprap channel linings or bank protection. The riprap design procedures include two concepts for the design of a stable channel lining. The two concepts are estimation of permissible velocity, and estimation of permissible shear.

The Corps riprap design procedure focuses on determination of the proposed channel roughness. From this, boundary shear is determined and then compared with the design shear that the riprap is able to resist. The design is a trial-and-error process with different stone size, in terms of D_{50}, and riprap thickness being adjusted so that the estimated boundary shear is less than the riprap design shear.

The Corps design procedures assume riprap material that is predominantly angular in shape. Cobbles with rounded edges have a smaller angle of repose, as shown in

Figure 5.9. Since the internal friction angle of a graded mass of cobbles is less than an equal mass of angular rock, the cobble mass is more likely to be eroded by channel flow. Use of cobbles or rounded rock for channel lining shall incorporate suitable design modifications in terms of side slope, median rock size and layer thickness, and shall be approved by the appropriate jurisdictional agency.

Riprap linings shall have a minimum thickness of 1.5 times D_{100}. In urban areas, riprap having a D_{50} less than one foot should be buried and revegetated to protect the riprap lining from vandalism.

5.5.4 Grouted Rock

5.5.4.1 General: Grouted rock is a structural lining comprised of rock that is interlocked and bound together by means of concrete grout injected into the void spaces. Grouted rock provides a stable lining similar to concrete with the added advantage of a higher roughness factor due to the rock surfaces projecting above the grout layer.

5.5.4.2 Materials

Rock: Rock for grouting should conform to the property requirements described in Section 5.5.3.1. Graded riprap should not be used for grouting, as the smaller rock in a graded mix occupies the void spaces to be filled with grout. Rock should be specified as a single layer of a minimum dimension rock. As an alternative, a class of riprap from Table 5.4 may be specified, with the requirement that rock smaller than the D_{50} size be removed. Additional details on grouted rock may be found in Chapter 6.

Grout: The grout mix should be specified to provide the strength and durability required to meet the specific application. The minimum 28-day compressive strength shall be 2,000 psi and the slump shall be within a range of 4 to 7 inches. The stone aggregate should conform to the gradation requirements of Size Number 8 course aggregate (3/8 inch to No. 8) as specified in ASTM C-33. A maximum of 30 percent of the cementous material may be fly ash (ASTM C-618, Type C or F). Fiber reinforcement may be added to the grout to provide additional control of shrinkage and cracking. Refer to Chapter 6 for additional information on grout for rock.

5.5.4.3 Design Considerations: Since grouted rock is a structural lining similar to reinforced concrete, it is subject to the same design considerations. Rock must be sized for the anticipated hydraulic design conditions. Foundation conditions must be evaluated and provisions made for underdrainage and seepage control. If only bank protection is to be provided, the grouted rock protection must extend below the channel invert to a depth below the estimated depth of scour. Refer to Chapter 7 for additional discussion on channel scour. Construction considerations for grouted rock are presented in Chapter 6.

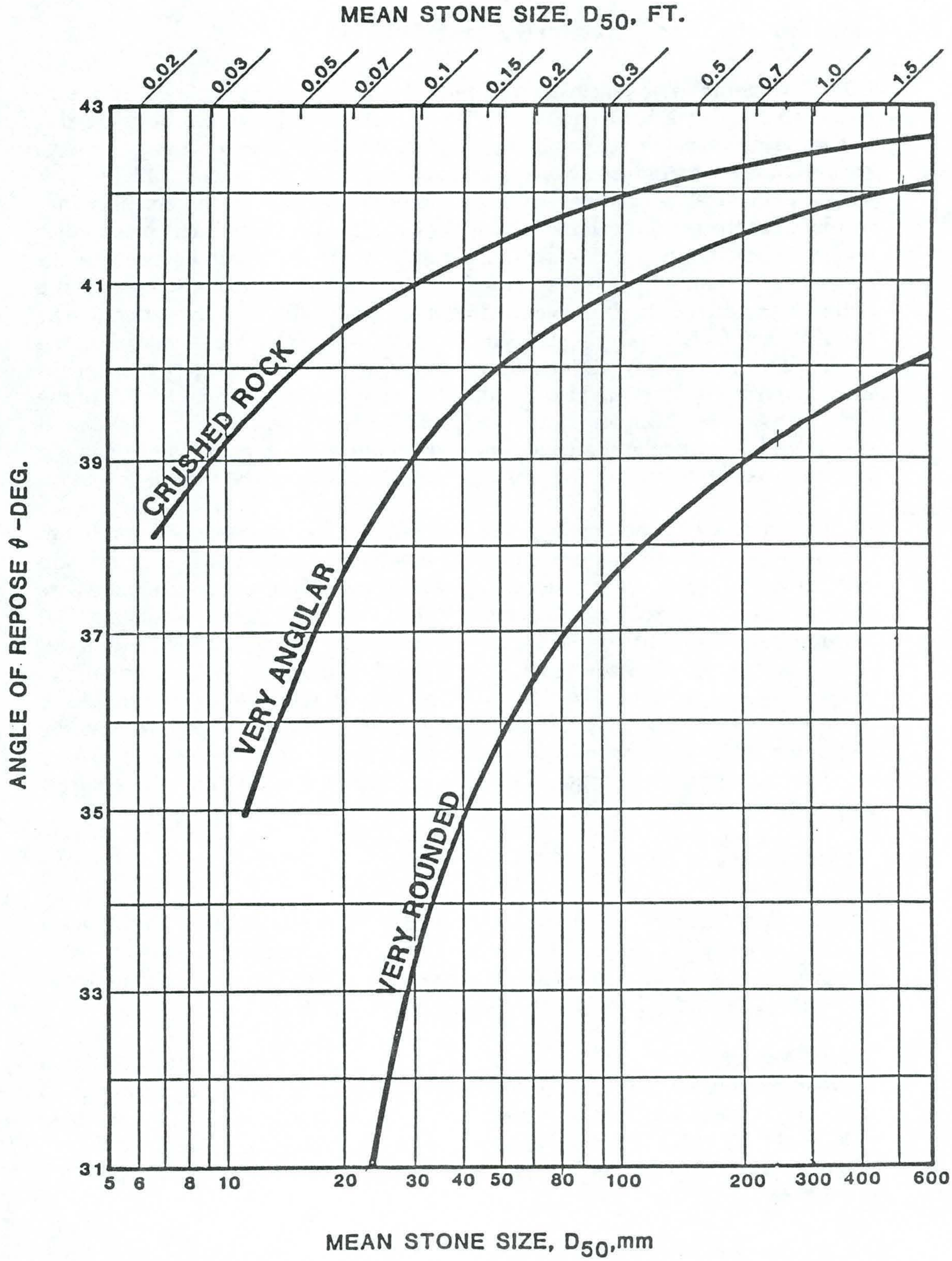


Figure 5.9
Angle of Repose of Riprap in Terms of Mean Size and Shape of Stone
(FHWA 1988b)

5.5.5 Wire Enclosed Rock (Gabion Baskets)

5.5.5.1 General: Wire enclosed rock refers 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 ordinary 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, is considered to be about 15 years. Water carrying silt, sand or gravel can reduce the service life of the wire. Also, water which 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.

Wire enclosed rock is 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. Mattresses on sloping surfaces must be securely anchored to the surface of the soil as discussed in Section 5.5.5.3.

5.5.5.2 Materials

Rock and Wire Enclosure Requirements: Rock filler for the wire baskets should meet the rock property requirements for ordinary riprap. Minimum rock sizes and basket characteristics are shown in Table 5.7. The maximum stone size should not exceed two-thirds basket depth or 12 inches, whichever is smaller.

Bedding Requirements: Long term stability of riprap and gabion 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 5.5.3.2 for gravel bedding or filter design.

5.5.5.3 Design Considerations: The geometric properties of wire enclosed rock permit placement in areas where ordinary riprap is either difficult or impractical to place. Proper design and construction is important to successful operation and lifetime performance.

Table 5.7
Gabion Baskets and Slope Mattresses

Letter Code or Size	Length, ft	Width, ft	Depth, ft	Number of Diaphragms	Capacity, yd^3	Minimum Rock Dimensions, inches
Gabion Baskets						
A	6	3	3	1	2	4
B	9	3	3	2	3	4
C	13	3	3	3	4	4
D	6	3	1	1	1	4
E	6	3	1	2	1.5	4
F	9	3	1	3	2	4
G	13	3	1	1	0.66	4
H	9	3	1	2	1	4
I	13	3	1	3	1.33	4
Gabion Slope Mattresses						
T	9	6	0.75	5	1.80	3
U	12	6	0.75	6	2.20	3

Slope Mattress Lining: Figure 5.10 shows a typical configuration for a gabion slope mattress channel lining. Mattresses and flat gabions on channel side slopes need to be tied to the banks by 1-inch diameter steel pipes driven 4 feet into tight, solid soil (clay) and 6 feet into loose soil (sand). The pipes should be located at the inside corners of basket diaphragms along an upslope (highest) basket wall, so that the 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 six feet along and down the slope, for 2:1 slopes and every 9 feet along and down the slope for slopes flatter 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 least every 12 feet. Counterforts should be keyed at least 12 inches into the existing banks with slope mattress linings to counteract longitudinal movement.

5.5.6 Toe Protection

5.5.6.1 General: Scour at a poorly designed toe of a channel bank is one of the major causes of failure of riprap or gabion protection measures. These failures result when the foundation of the bank protection measure is undermined by scour at the toe resulting from local scour at high flows 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 Section 5.8. Means of providing protection for the maximum scour—once it has been determined—are presented in this section.

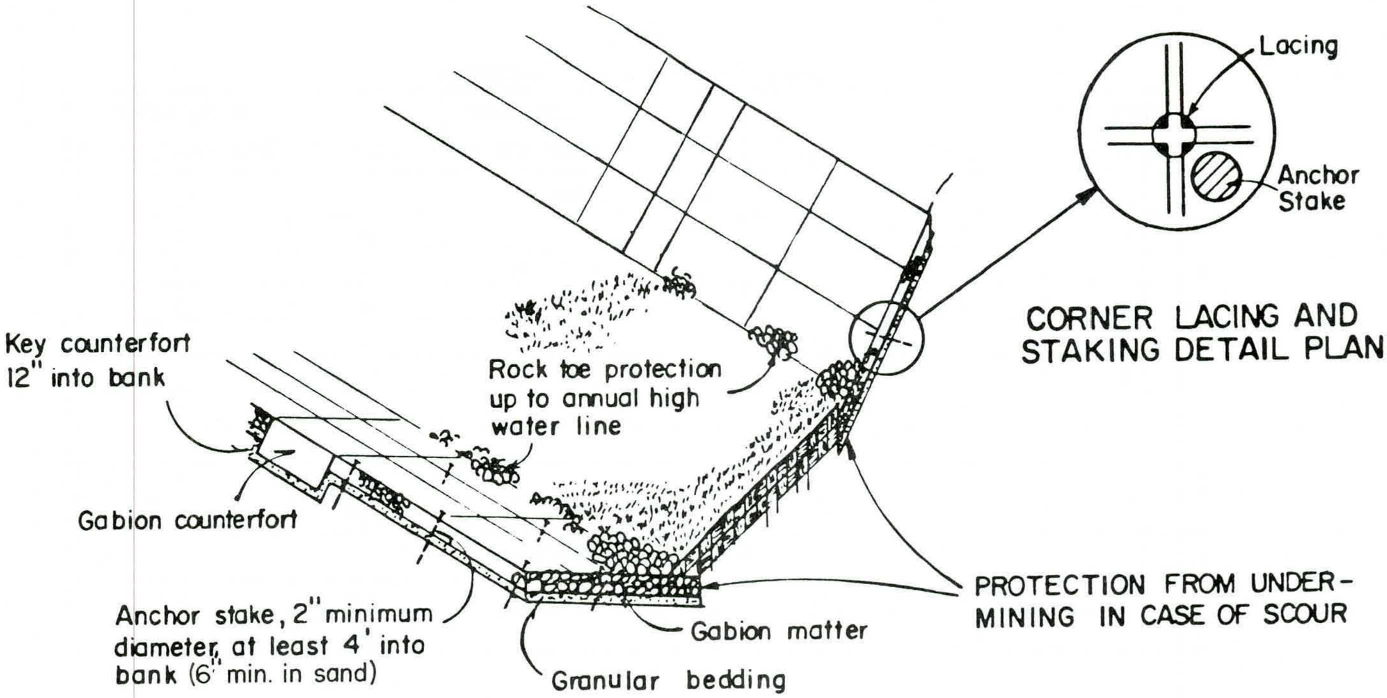


Figure 5.10
Slope Mattress Lining
(Wright-McLaughlin Engineers 1969)

5.5.6.2 Design Guidelines: The two methods of providing toe protection in alluvial channels are:

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

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. The first method requires extension of the bank protection into the excavated channel bed and is primarily used for placement in the dry 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 downslope, as scour occurs, to form a protective cover. Figure 5.11a shows the desirable configuration for a weighted riprap toe. Less frequently used materials are gabion mattresses (see Figure 5.11b). 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

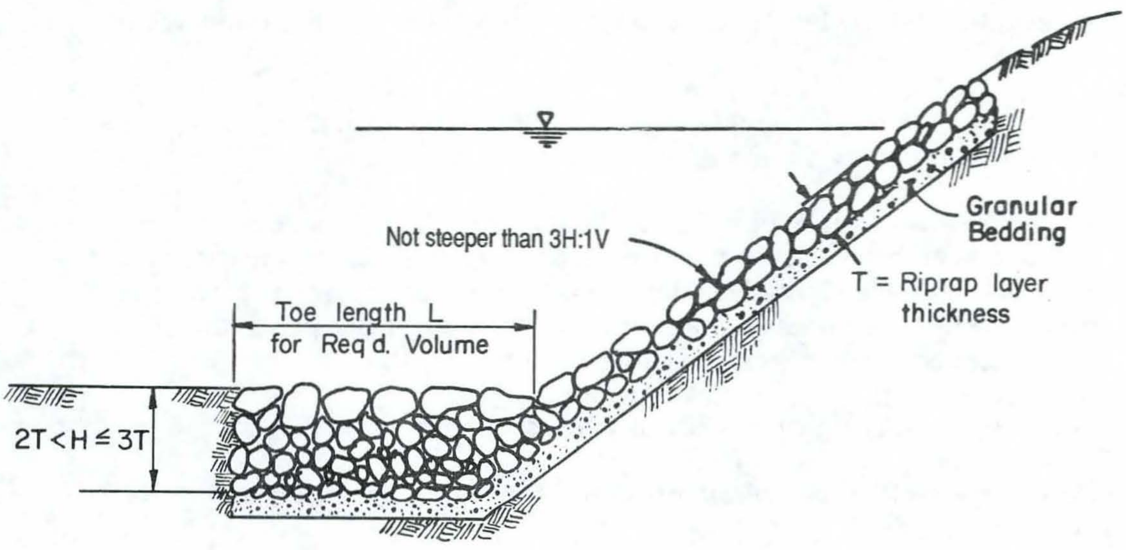


FIGURE 5.11a

TOE PROTECTION RIPRAP CHANNEL LINING

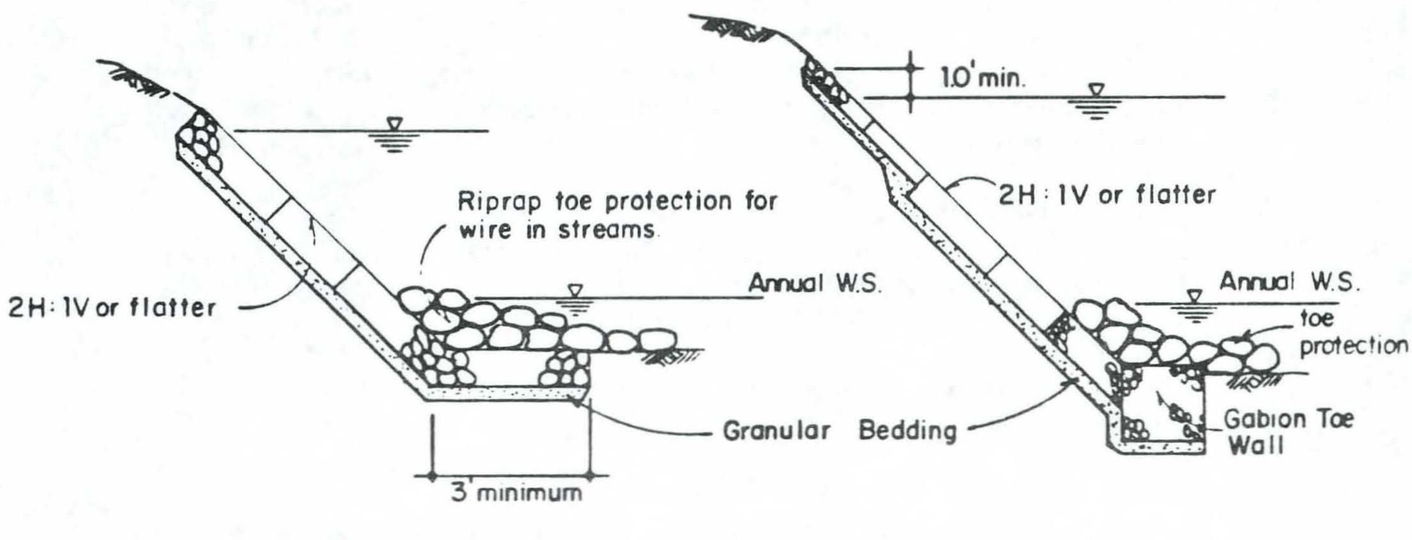


FIGURE 5.11b

FIGURE 5.11c

Figure 5.11
Toe Protection Wire Enclosed Rock Lining
(Adapted from: Wright-McLaughlin Engineers 1969)

of Engineers (December, 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 thickness of the riprap bank protection. The recommended toe shape is shown in Figure 5.11a.

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.

5.5.7 Grass Lined Channels

Appropriate grass cover for Maricopa County includes: alfalfa for dry land and Bermuda grass for irrigated applications. Establishment or temporary irrigation will be required in most situations (see Table 5.2).

The designer should coordinate work with the appropriate jurisdictional agency to provide the most acceptable grass cover and irrigation method.

5.5.8 Other Forms of Channel Stabilization

Other, less common forms of channel stabilization may also be acceptable, provided that it can be demonstrated that the particular stabilization method proposed is capable of withstanding the hydraulic conditions which can occur within the channel.

5.6 Design Procedures

This section provides the user of this manual with checklists (procedures) to be followed during design, and with examples of design computations that can be followed in order to clarify the procedures that are used.

5.6.1 Artificial Channels

Table 5.8 is the checklist to be employed in the design of artificial channels. Most applications in Maricopa County will be able to follow the Simplified Design Procedure. Should conditions fall outside of the *Simplified Design Procedure*, the more rigorous procedure, as shown in the remaining steps, must be completed. In some instances, the conditions may be so complex that the design procedure for natural channels (see Section 5.6.2) will be required. Determination of water surface profiles is not always required; however, such a determination is necessary in all instances of spatially varied flow (i.e. side channel spillways) and where obstructions cross the channel (i.e. culverts, bridges, grade control structures, etc.

5.6.1.1 Conditions for Using the Simplified Design Procedure: The steps required for the *Simplified Design Procedure* are marked with an asterisk in Table 5.8 and can be used to design open channels that service minor drains and for major drains that meet the following conditions:

» Grade control established, and

» Design parameters within those listed in Table 5.3.

5.6.1.2 Conditions Beyond the Scope of the Simplified Design Procedure: When the conditions of the previous subsection cannot be met, it will be necessary to undertake a more detailed analysis that includes all of the steps listed in Tables 5.9 and 5.10. Table 5.9 addresses analytic requirements of natural channels and modifications that may be necessary to accommodate crossings of the waterways, such as bridges and culverts.

5.6.2 Natural Channels

Detailed analyses of natural channels and non-rigid artificial channels are beyond the scope of this manual; however, some understanding of the aspects of the analysis for non-rigid channels is necessary to proceed with the design of rigid channels and to provide initial analyses during the master planning phases, especially for lands adjacent to natural channels. Tables 5.9 and 5.10 are included to provide prospective designers with a comprehensive list of subjects that must be considered on adjacent developments and to plan effective countermeasures to potential adverse impacts.

In addition to normal resources a designer may have for analyzing non-rigid channels, four specific resources that were used to develop Table 5.10 are listed at the bottom of that table. Examination of these documents reveals the potential complexity of analysis of natural channels in Maricopa County. Few naturally occurring channels will have sufficient grade controls to be in equilibrium over the expected range of flows. This condition can have significant impacts on the design of related structures, such as culverts, hydraulic drops, and bridges. In addition, the

Table 5.8
Design Checklist for Artificial Channels

	Item	Section Reference
Simplified Design Procedures		
*	When Simplified Procedures can be Used	5.6.1.1
	When More Thorough Analysis is Required	5.6.1.2
Initial Data		
*	Existing Structures	2.3
*	Existing Channel Characteristics	2.3, 5.3.1, 5.3.3.1
*	Existing Grade Control	2.3, 5.3.1, 5.3.3.3
*	Existing Flood Performance Characteristics	2.3
*	Scour Observations	5.5.6
*	Existing Stream Development	2.3
*	Land Use Changes	2.3
	Flood History	Drainage Design Manual, Volume I
	Rainfall/Runoff Relationships	Drainage Design Manual, Volume I
Possible Components and Strategies		
*	Channels	5.3.2, 5.5
*	Alignment	5.3.3.3, 5.3.3.4
*	Grade Control Structures	2.3, 5.3.3.3, Chapter 6
Consideration for Right of Way		
	Migration	
*	Water Level	2.3
Economic and Alternative Analysis		
*	Designation of Significantly Different Concepts	
	* Type of Lining	5.3.2.1, 5.5
	* Type of Cross Section	5.3.2.1, 5.3.3.3
	* Channel Alignment	5.3.1, 5.3.3.3
	* Location of Grade Control(s)	5.3.2.5, 5.3.2.4, 5.3.3.3
*	Hydrologic and Hydraulic Detailing of Alternatives	5.3.3.2, Drainage Design Manual, Volume I
	Least Total Expected Cost Evaluation	NIC
	Extreme Flood Evaluation of Components and Alternatives	NIC
	Environmental Considerations	2.3
*	Documentation and Comprehensive Evaluation	2.3
*	Safety Requirements	5.3.2.5, Chapter 4
Hydraulic Analysis		
*	Determination of Control	5.3.2.4, Chapter 6
	Determination of Type of Flow Profile	5.4.1
*	Normal Depth Calculations	5.3.3.2
*	Water Surface Profile Calculations	5.4.1.2
*	Bridge Hydraulics	Chapter 7
*	Channel Lining	5.5
	Supercritical Channel Hydraulics	5.3.3.3
	Superelevation	5.3.3.3
*	Drop Structure Hydraulics	5.5
	Physical Hydraulic Models	5.4
*	Low Flow Channel	5.3.3.3
Sediment Transport Analysis		
	Required when Simplified Design Procedure cannot be used— Reference to Natural Channels	Table 7.1
Additional Considerations		
	Permanent Record	
	Post Construction Data	
	Normal Inspection (References)	

Table 5.9
Design Checklist for Natural Channels

Item	Section Reference
Initial Data	
Existing Structures	
Channel Characteristics	5.4.1.2, 5.4.1.3
Existing Flood Performance Characteristics	5.4
Existing Grade Control	5.4
Scour Observations	5.3.2.1, 5.5.6
Existing Stream Development	5.6.2
— Dams, Diversions	
— Flood Control	
— Mining	
Flood History	Drainage Design Manual, Volume I
Rainfall/Runoff Relationships	Drainage Design Manual, Volume I
Possible Components And Strategies	
Channels	5.4.5.5
Bridge Components	5.4.1, 5.6.2
River Alignment Control Strategies or Mitigation	5.4
Alignment Control Structures	Chapter 6
Grade Control Structures	Chapter 6
Non-structural Measures (Easements, Acquisition, Litigation)	
Economic And Alternative Analysis	
Designation of Significantly Different Concepts	5.4.5.5
Hydrological and Hydraulic Detailing of Alternatives	5.3.3.2, Drainage Design Manual, Volume I
Least Total Expected Cost Evaluation	
Extreme Flood Evaluation of Components and Alternatives	
Environmental Considerations	
Documentation and Comprehensive Evaluation	
Hydraulic Analysis	
Determination of Control	
Determination of Type of Flow Profile	Figure 5.3, 5.4.1
Normal Depth Calculations	5.3.3.2
Water Surface Profile Calculations [Usually HEC-2]	5.4.1.2
Bridge Hydraulics	Chapter 7
Sand Bed Formation Determination	
Sand Bed Roughness	
Cobble, Boulder, or Riprap Roughness Determination	5.5.3
Vegetation or Combination Lining Roughness	5.6.3.1
Dune and Antidune Height	
Supercritical Channel Hydraulics	5.3.3.3
Drop Hydraulics	5.5.6.3
Average Characteristics	
Physical Hydraulic Models	
Sediment Transport Analysis	Table 5.10, Table 7.1
Additional Considerations	
Permanent Record	
Post-construction Data	
Normal Inspections (References)	

Table 5.10
Design Checklist for Natural Channels

Level I Sediment Transport Analysis
Data Requirements
Determination of Plan Form Characteristics
Lane Relation and other Geomorphic Relationships
Aerial Photograph Interpretations
Bed and Bank Material Analysis
Land Use Changes
Flood History
Rainfall/Runoff Relationships
Level II Sediment Transport Analysis
Data Requirements
Watershed Sediment Yield
Detailed Bed and Bank Material
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
Level III Sediment Transport Analysis
Data Inventory Modeling
Watershed Sediment Modeling
Instream Mining Response
Single Event Stream Bed Modeling
Long Term Bed Modelling

Resources

1. Laursen and Duffy1980.

2. Richardson 1988.

3. Sabol, Nordin, and Richardson 1988.

4. Simons, Li and Associates 1985.

designer should be alert to practices that are common within channels, such as gravel mining, that can have significant impact on structures and on the overall stability of the channel. It is very common for natural channels to move laterally during larger storm events, and this potential condition can result in serious consequences to adjacent land uses.

5.6.3 Design Example

5.6.3.1 Example: Improve a small earth lined channel with incised low flow, bank and edge erosion $S_o = 0.006$ ft/ft, partially vegetated sandy silt material to convey 100-year design flow, $Q_{100} = 650$ cfs. (See Figure 5.12.)

Given:

» Available channel width of 50 feet, with approximately 3 feet depth

» Provide grass-lined main channel with concrete-lined low flow (see Section 8.2.2.6)

» Use 4:1 side slopes and provide minimum freeboard allowance.

Step 1: Check channel capacity.

A. Solve Manning Equation (Equation 5.1)

1. Find cross-sectional area of flow [Total area (A_T) = area of low flow channel (A_{lf}) + area of main channel (A_{mc})]

$$A_T = A_{lf} + A_{mc}$$

$$= (1.5)(5) + 3((18+42)/2)$$

$$= 7.5 + 90$$

$$A_T = 97.5 \text{ sf}$$

2. Find wetted perimeter and indicate a 4" thickness for low flow wall.

$$P_T = P_{lf} + P_{mc}$$

$$= \left(2(0.33) + 2(1.5) + 5\right) + \left((18 - 5.67) + 2(3)(1^2 + 4^2)^{0.5}\right)$$

$$= 8.67 + (12.33 + 24.7)$$

$$P_T = 45.7 \text{ ft}$$

3. Find hydraulic radius

$$R = \frac{A_T}{P_T}$$

$$= \frac{97.5}{45.7}$$

$$= 2.13 \text{ ft}$$

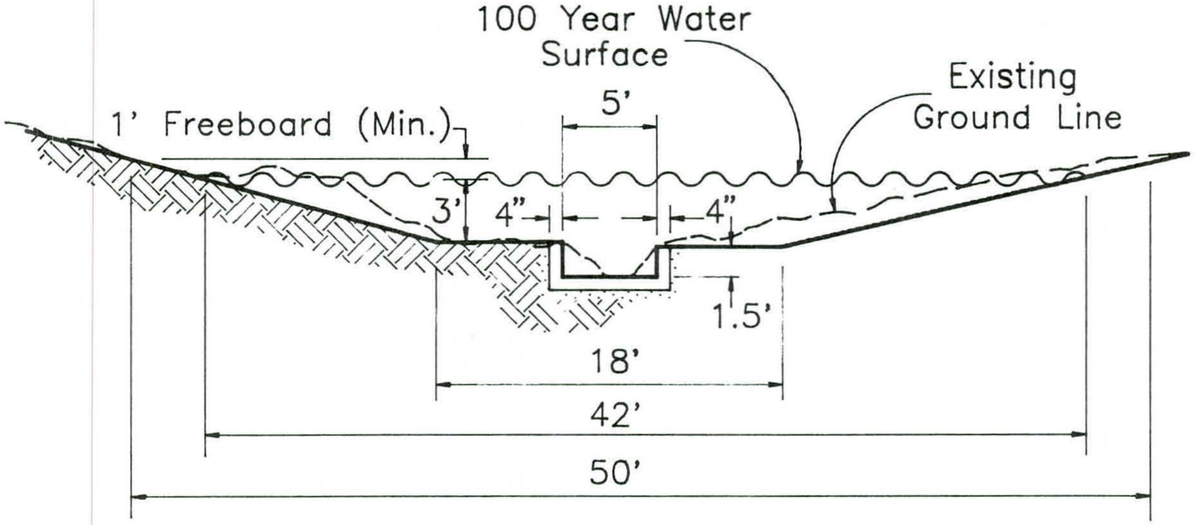


Figure 5.12
Channel Design Example

4. Determine Manning's "n" from Table 5.11.
   Find composite "n" value:
   Concrete lined low flow: n = 0.015
   Grass-lined main channel: n = 0.025

$$n = \left[\Sigma_1^{\ N}(P_N\, n_N^{\ 1.5})\,/\,P\right]^{0.67} \qquad \text{(Chow, 1959)}$$

$$= \left[\,(P_{lf})(n_{lf}^{\ 1.5}) + (P_{mc})\,(n_{mc}^{\ 1.5})\,/\,P_T\right]^{0.67}$$

$$= \left[8.67(0.015)^{1.5} + 37.0(0.025)^{1.5}\,/\,45.7\right]^{0.67}$$

$$= [(0.016 + 0.15)\,/\,45.7]^{\,0.67}$$

$$n = 0.023$$

5. Substitute values to solve for slope
   Multiply Equation 5.1 by Q and rearrange :

$$Q = (1.49\,/\,n)\;A_T\,R^{\,0.67}\;S_o^{\ 0.5}$$

$$S_o = \left[Qn\,/\,(1.49\;A_T\,R^{\,0.67})\right]^2$$

$$= \left[650\;(0.023)\,/\,(1.49)\;(97.5)\;(2.13)^{\,0.67}\right]^2$$

$$S_o = 0.0038 \text{ ft/ft}$$

Since a channel bottom slope of 0.0038 ft/ft is sufficient to convey the design flow of 650 cfs, the steeper existing S_o of 0.006 ft/ft will convey the flow with a smaller cross sectional area. Equation 5.1 can be solved for $A_T R^{0.67}$ to determine the actual cross section of flow:

$$A_T R^{0.67} = Q_n/1.49 S_o{}^{0.5} = 650(0.023)/1.49(0.006)^{0.5} = 129.5 \text{ ft}^{2.67}$$

By trial and error,

Y_n = 2.65 ft,
A_T = 7.5 + 2.65((18 + 39.2)/2) = 83.3 sf,
P_T = 8.67 + ((18 – 5.67) + 2(2.65) $(1^2 + 4^2)^{0.5}$) = 42.9 ft, and
R = A_T/P_T = 83.3/42.9 = 1.94 ft.

Flow along the channel at S_o = 0.006 ft/ft has reduced the water depth by 0.35 ft. Note that the composite "n" value has not changed with the new values of P_{lf}, P_{mc}, and P_T.

B. Check velocity and Froude Number

1. $V = Q / A$
   $= 650 / 83.3$

   V= 7.8 fps < 8 fps allowable for Bermuda grass lined channels with erosion resistant soil only.

2. Check Froude Number

$$F = V / (gD)^{0.5}$$

$$= \frac{7.8}{\left((32.2)(\frac{83.3}{39.2})\right)^{0.5}}$$

$$F = 0.94 > 0.85$$

The channel is under near critical flow conditions and will not be stable at a bottom channel slope of 0.006 ft/ft for the design flow. One solution is to provide grade control structures to maintain S_o = 0.0038 ft/ft, thereby having V = QA = 650/97.5 = 6.7 fps and F = $V/(gD)^{0.5}$ = $6.7/((32.2)(97.5/42))^{0.5}$ = 0.77, which is within the acceptable range of subcritical flow. See Chapter 6 for grade control structures.

3. Check channel transitions (see Chapter 6).

C. Confirm freeboard requirement

$$FB = 0.25\left(Y + V^2/2g\right) \quad (5.9)$$

$$= 0.25\left(3 + 6.7^2/2(32.2)\right)$$

$$= 0.25(3.7)$$

$$FB = 0.92 \text{ ft}$$

$$= 1 \text{ ft minimum}$$

Summary:

» Grass lined channel with 4:1 side slopes

» Velocity = 6.7 fps; F = 0.77, subcritical flow. See Table 5.2 for allowable grass and soil types.

» Channel slope = 0.0038 ft/ft < 0.006 ft/ft (existing)

» Provide grade control

» Provide 1 foot minimum freeboard allowance

» Check flow velocities and hydraulic properties for other flows anticipated

5.7 Design Aids

Table 5.11 and Figures 5.13 through 5.16 are provided here to help in designing open channels.

Table 5.11
Manning's Roughness Coefficients*

Channel Material	Roughness Coefficient (n)		
	Minimum	Normal	Maximum
Corrugated metal	0.021	0.025	0.030
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
Asphalt (use maximum value when cars are present)	0.013	0.016	0.020
Soil Cement	0.018	0.020	0.025
Constructed channels with earth or sand bottom and sides of:			
Clean earth; straight	0.018	0.022	0.025
Earth with grass and weeds	0.020	0.025	0.030
Earth with 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
Dry rubble or riprap	0.023	0.032	0.036
Natural channels with sand bottom and sides of:			
Trees and shrubs	0.025	0.035	0.045
Rock	0.024	0.032	0.040
Natural channel with rock bottom	0.040	0.060	0.090
Overbank Floodplains			
Desert brush	normal density	0.040	0.060
Dense vegetation	0.070	0.100	0.160

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

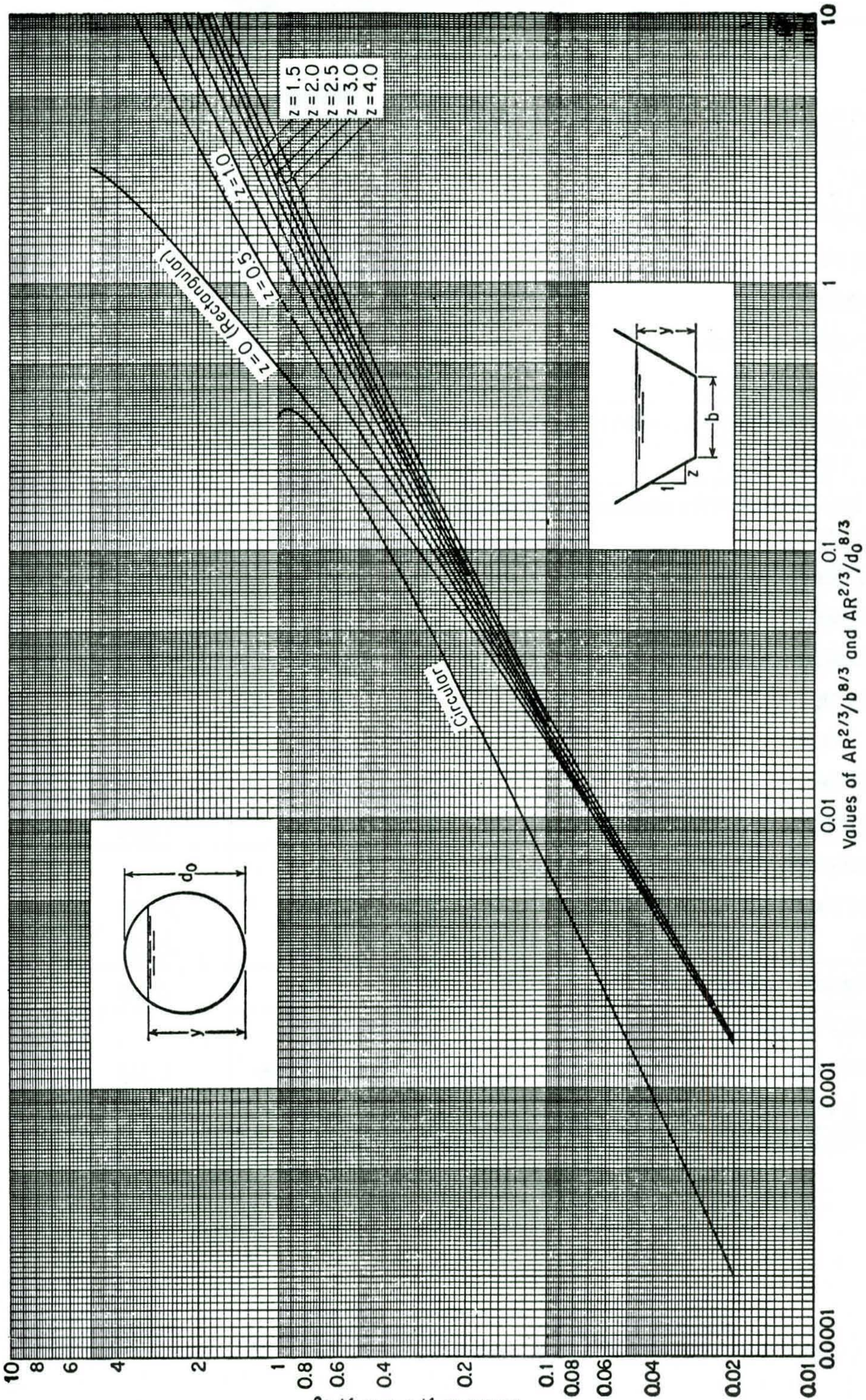


Figure 5.13
Curves for Determining the Normal Depth for Uniform Flow in Open Channels
(Wright-McLaughlin Engineers 1969)

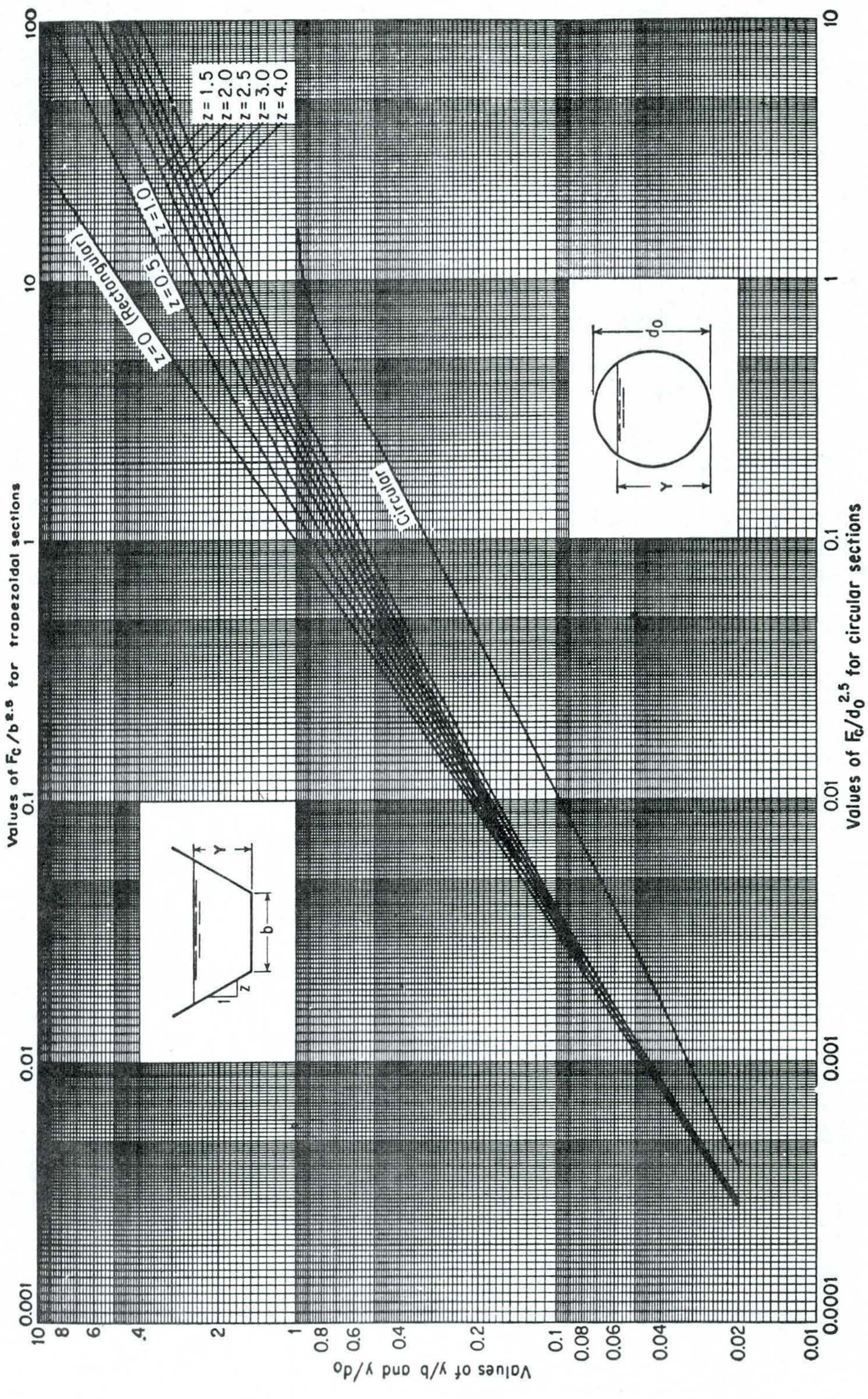


Figure 5.14
Curves for Determining the Critical Depth in Open Channels
(Wright-McLaughlin Engineers 1969)

Figure 5.15
Standard Step Backwater Form Example

CLIENT ______________________
JOB TITLE ______________________
JOB NUMBER ______________ PHASE __________
COMPUTATIONS BY ______________ DATE __________
CHECKED BY ______________ DATE __________

WATER SURFACE PROFILE COMPUTATIONS
STREAM ______________________
LOCATION ______________________
REACH ______________________
SHEET ________ OF ________ PAGE ________

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
SECTION	REACH LENGTH L	ASSUMED W. S. ELEVATION	AREA A	$K_d = \frac{1.486}{n} AR^{2/3}$	$K_d / L^{1/2}$	$\left(\frac{Q}{\Sigma \frac{K_d}{L^{1/2}}}\right)^2 = h_f$	$(K_d/L^{1/2})(h_f)^{1/2} = Q$	V	$V^2 Q$	h_v	$h_{v_1} - h_{v_2}$	OTHER LOSSES	MEAN h_f	TOTAL LOSS	ΔH	WATER SURFACE ELEVATION

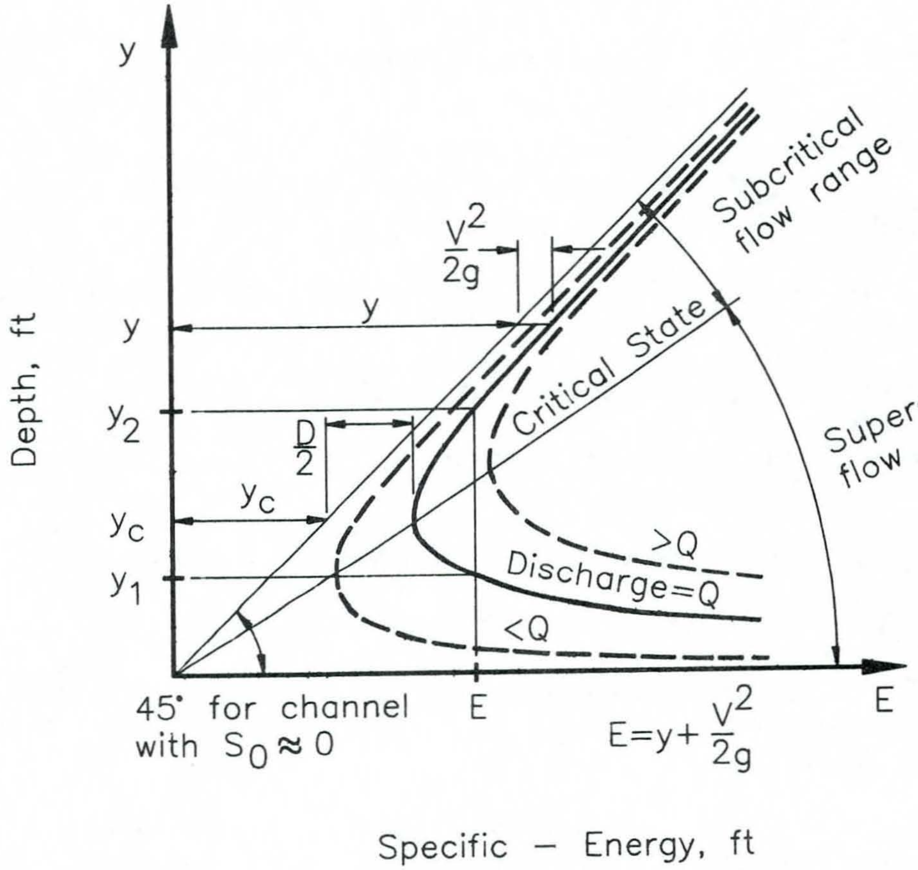


Figure 5.16
Specific Energy Curve
(Chow 1959)

5.8 References

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6

Hydraulic Structures

6.1 Use of Structures in Drainage

Hydraulic structures are used in storm drainage 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.

Hydraulic structures increase the cost of drainage facilities, and their use should be limited by careful and thorough hydraulic engineering practices to specific 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, low flow check 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 of all structures.

This chapter is oriented toward control structures for drainage channels and for conduit outlets—in contrast to dams, spillways or specialized conveyance measures. Refer to Chapter 4 for culvert and closed conduit design, Chapter 5 for open channel design, and Chapter 7 for bridge design.

6.1.1 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.

6.1.2 Conduit Outlet Structures

Energy dissipation structures are necessary at the outlets of culverts or storm sewers 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 regime. Outlet structures may be preformed rock riprap stilling basin or reinforced concrete structures such as impact basin baffle chutes.

6.1.3 Special Channel Structures

Bridges, spur dikes, channel transitions, constrictions and bends, and structures for lined channels and for long conduits are examples of hydraulic structures for special applications.

6.1.3.1 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 increased water surface 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. Refer to Chapter 7 for further discussion on bridges.

6.1.3.2 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, or to purposely accelerate the flow to be carried by a specialized high velocity conveyance structure. Constrictions are designed to restrict and reduce the conveyance along a short reach. Examples are a bridge with roadway approach embankments that significantly encroach into a floodplain, or a structure designed to raise the upstream water surface to force spill into an off-channel storage facility. 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.

6.1.3.3 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 can occur which can require special structural and other design considerations.

6.1.4 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 engineer's judgement of an appropriate factor of safety should be carefully analyzed for each structure 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: *Preliminary Engineering Manual, Civil Works Construction* [U.S. Army Corps of Engineers, January 1948]; *Design Manual, Foundations and Earth Structures* [Department of the Navy, May 1982]; *Design of Small Dams* [USBR 1977]; *Design of Gravity Dams* [USBR 1976]; and *Drainage of Roadside Channels with Flexible Linings* [FHWA March 1986].

6.2 Definition of Symbols

The following symbols are used in equations throughout Chapter 6:

α	=	Angular variation of sidewall with respect to channel centerline
α_1	=	Kinetic energy coefficient
β	=	Standing wave front angle
ρ	=	Density of water, 1.94 lbs sec^2/ft^4
τ	=	Shear stress on the bed caused by the flow of water, psi
γ	=	Specific weight, lb/ft^3
A	=	Area (subscripts as shown in Figures), ft^2
b	=	Bottom width, ft
b_t	=	Trickle channel width, ft
B	=	Basin depth below downstream channel, ft
C	=	Coefficient relating to superelevation of water surface that occurs at a bend
C_d	=	Drag force coefficient
C_p	=	Coefficient of mean pressure fluctuations from mean pressure levels in a hydraulic jump
$C_{p\text{-}max}$	=	Coefficient of maximum pressure fluctuations from mean pressures level in a hydraulic jump
C_w	=	Lane's Weighted-creep ratio
d_2	=	Depth of basin tailwater = Y_2 + B, ft
d_{50}	=	Mean diameter of particle (stone), ft, mm
d_s	=	Depth of scour, ft
D	=	Jet plunge height, ft

D_b	=	Bedding layer thickness, ft
D_g	=	Grout depth, ft
D_{jm}	=	Distance to the hydraulic jump, main channel, ft
D_{jt}	=	Distance to the hydraulic jump, trickle channel, ft
D_n	=	Drop Number
D_r	=	Rock depth, ft
EGL_m	=	Energy grade line along main portion of drop
EGL_t	=	Energy grade line along trickle channel through a drop
El_c	=	Water surface elevation of criteria depth at the crest of a drop
El_m	=	Elevation of crest of a drop at main channel invert drop
El_t	=	Elevation of crest of a drop at trickle channel invert
F	=	Force, lb or Froude number = $V/(gy)^{0.5}$
F_1	=	Froude Number upstream of the jump
F_j	=	Impact force of flow jet, lb
Fm	=	Momentum force, lb
F_s	=	Specific force, ft^2
g	=	Acceleration due to gravity, 32.2 ft/sec^2
h	=	Height of the wingwalls above the main crest, ft
h_l*	=	Total backwater, ft
h_L	=	Head loss, ft
h_V	=	Velocity head, ft
H	=	Differential head between analysis points, ft
H_{cw}	=	Height of seepage cutoff, ft
H_d	=	Desired drop across structure, ft
H_m	=	Total energy head at the crest of the main drop, ft
H_s	=	Differential head, usually at a drop; the difference between the upstream water surface (normal depth) to the downstream tailwater, or the head difference between analysis points (e.g. to point of supercritical flow minimum depth), ft
H_t	=	Total energy at the crest of the trickle channel, ft
H_w	=	Head on structure for weighted creep ration, (headwater-tailwater), ft
L_d	=	Length at a vertical hard drop, from the crest wall to the point where the flow nappe contacts the basin floor, ft
L_{dm}	=	Nappe length, main channel, ft
L_{dt}	=	Nappe length, trickle channel, ft
L_H	=	Horizontal creep distance along contact surfaces less than 45 degrees, ft
L_j	=	Length of the hydraulic jump (approximately 6 Y_2), ft

L_V	=	Vertical creep distance along any contact surfaces greater than 45 degrees, ft
L_{Bm}	=	Design basin length, main channel, ft
L_{Bt}	=	Design basin length, trickle channel, ft
M	=	Mass rate of flow, lb sec/ft = ρQ
n	=	Manning roughness coefficient
P	=	For a vertical drop structure, the height of the weir crest above the approach channel, ft
ΔP	=	Maximum pressure fluctuation at a given location within a hydraulic jump, psi
q_D	=	Design unit discharge, cfs/ft
q	=	Unit discharge $> y^{3/2} g^{1/2}$, cfs/ft
q_c	=	Discharge per unit width of crest, cfs/ft
q_m	=	Unit discharge in the main channel at drop, $ft^2/sec^{1/2} = q^{1/2} y_{cm}^{3/2}$
q_t	=	Unit discharge in the trickle channel at drop, , $ft^2/sec^{1/2} = q^{1/2} y_{ct}^{3/2}$
Q	=	Maximum design discharge, cfs
r	=	Channel centerline radius of curvature, ft
R	=	Hydraulic radius, ft
S_o	=	Bed or drop slope (S is also used), ft/ft
T	=	Top width of flow in the channel, ft
v	=	Velocity, fps
V	=	Maximum exit velocity, fps
W	=	Chute width, ft
y	=	Depth of flow, ft
y_c	=	Critical flow depth, ft
y_{cm}	=	Critical depth at a drop in the main channel, ft
y_{ct}	=	Critical depth at a drop in the trickle channel, ft
y_f	=	Vertical fall at a drop, ft
y_n	=	Normal depth, ft
y_p	=	At a vertical drop, the pool depth under the nappe just below the crest, ft
Y_1	=	At a vertical hard drop, the depth of flow immediately downstream of the point where the nappe contacts the basin, or at the toe of a sloping drop, ft
Y_{1u}	=	Initial (upstream) flow depth, ft
Y_2	=	The tailwater depth required to cause a jump to form immediately downstream of the initial depth location for Y_1, ft
Y_{2D}	=	Sequent (downstream) flow depth, ft
$(Y_2)m$	=	Sequent depth, main channel, ft

$(Y_2)t$	=	Sequent depth, trickle channel, ft
Y_f	=	Effective fall height from the crest to the basin floor, ft
Y_p	=	Pool depth under the nappe downstream of the crest, ft
Y_{tw}	=	Actual tailwater depth present downstream of the drop, ft
z	=	For a vertical drop structure, the difference in the bed elevations of the approach channel at the weir and the downstream channel at the end of the structure, ft, $= D - P - 0.67d_s$
Z	=	Channel side slope vertical ratio distance, ft/ft
Z_f	=	Drop face slope, ft/ft

6.3 Channel Drop Structures

6.3.1 General

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 criteria is on drop structures with design flows up to 10,000 cfs. Flows less than 500 cfs are the usual range for check structures. Check structures have additional considerations because major flooding goes around the structure abutments, typically in a much wider floodplain.

6.3.1.1 Basic Components of a Drop Structure: Figure 6.1 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.

6.3.1.2 Design Considerations: In addition to hydraulic performance (discussed in Section 6.3.2), a number of other considerations affect the selection of an appropriate drop structure for a particular application.

Soil and Foundation Conditions: Geotechnical investigations should be completed to identify the characteristics of the on-site 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 include ease of access to the crest and stilling basin areas, designs for vandal resistance, designs that eliminate trapped (ponded) water and provision of landscaped or grassed slopes that are easily maintained.

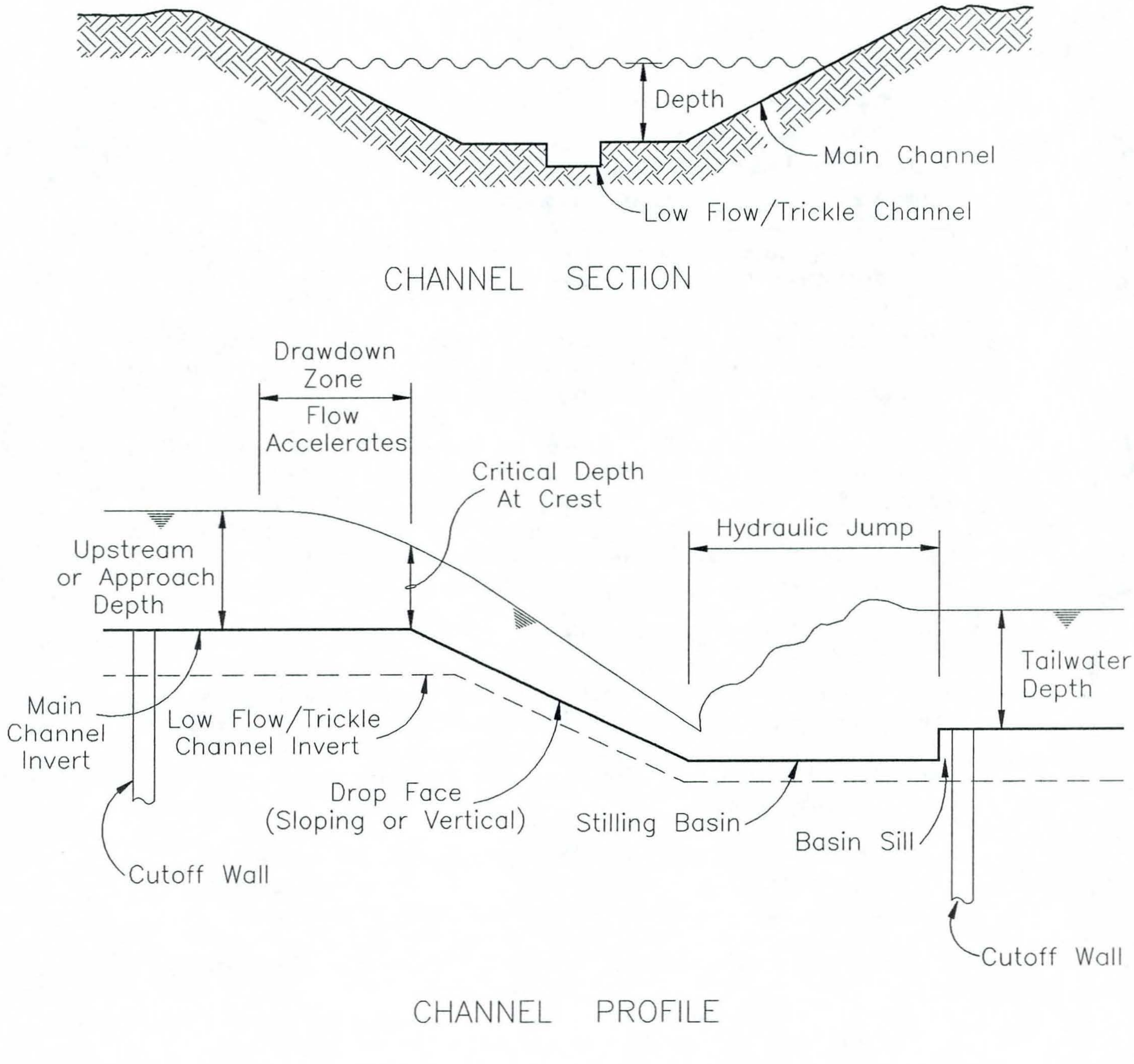


Figure 6.1
Typical Drop Structure Components
(Adapted from McLaughlin Water Engineers, Ltd. 1989)

Sociological Considerations: These include public acceptability issues such as safety, visual appearance, mosquito breeding in ponded water, etc.

6.3.1.3 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
» Low Flow Check Structures

Figure 6.2 shows schematic profiles of each type.

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 to assure their quality and integrity. In addition, drop structures should also 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 6.3.3 presents considerations for the selection of the appropriate type of drop structure for particular application or site conditions.

6.3.2 Hydraulic Analysis Considerations

6.3.2.1 General 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 6.3.3).

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

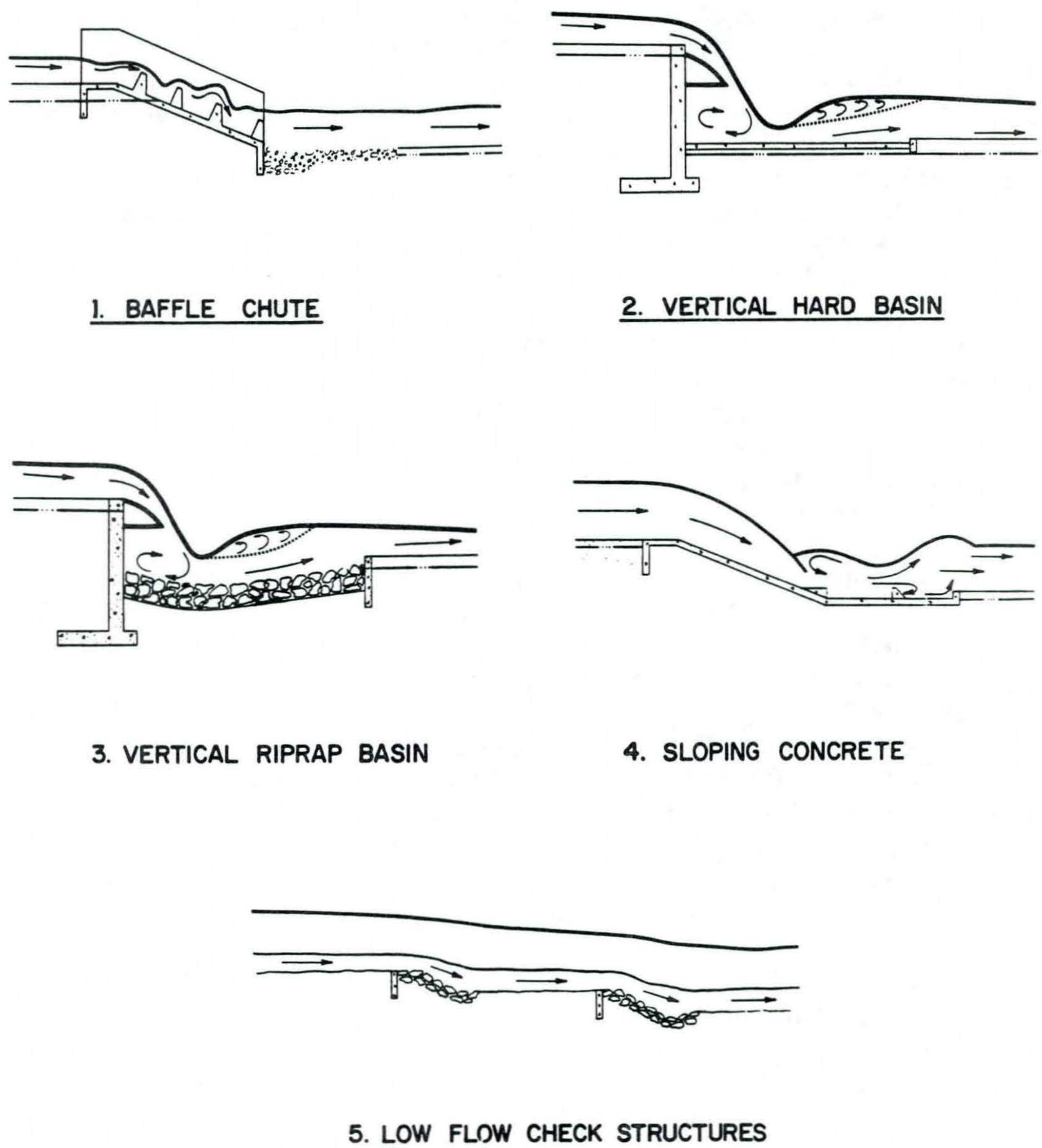


Figure 6.2
Drop Structure Types
(McLaughlin Water Engineers, Ltd. 1986)

drop to determine the extent of protection required, as well as the potential problems/solutions for each. (See Section 6.3.2.3 for discussion.)

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.

6.3.2.2 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 6.3 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,

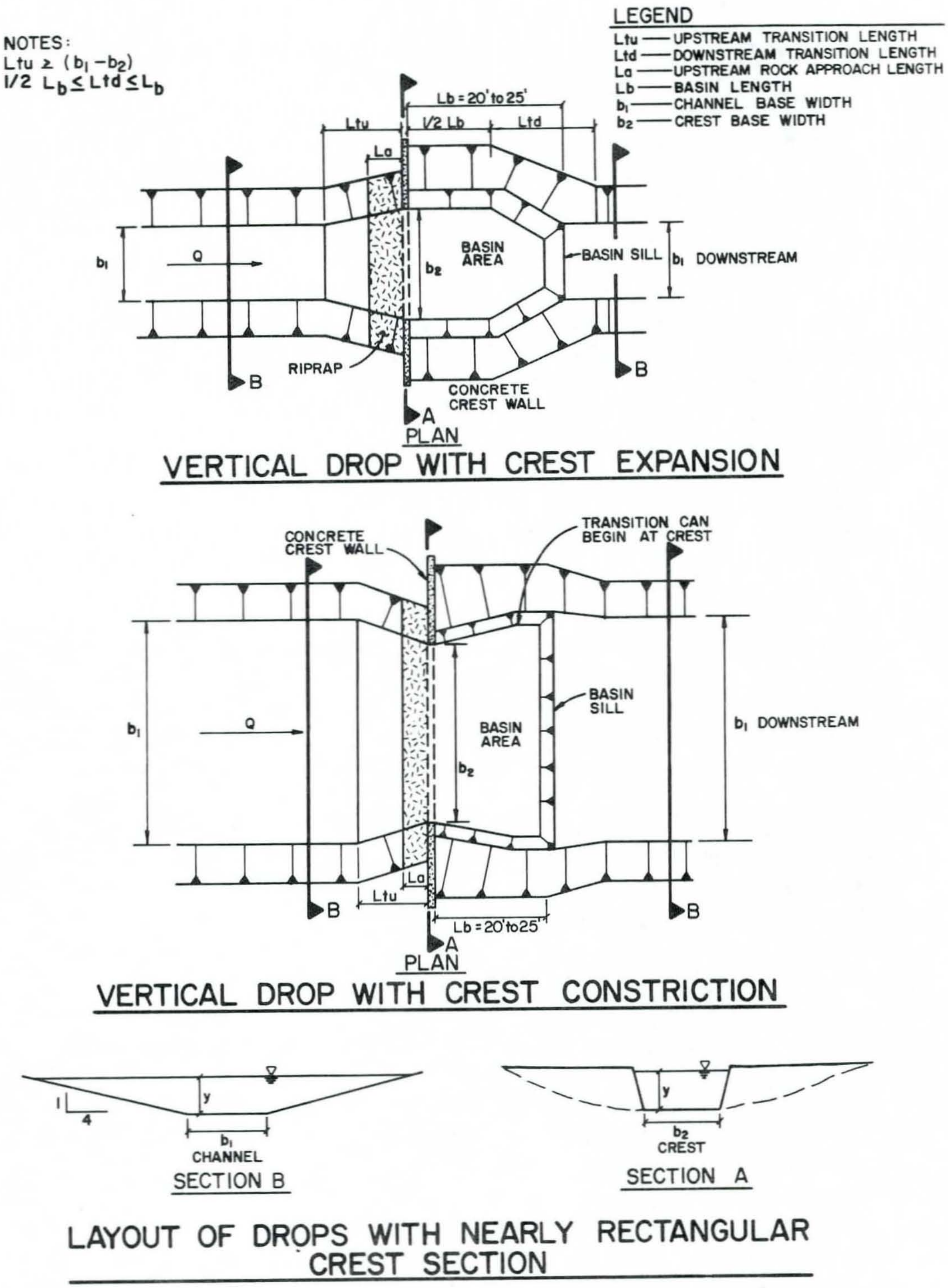


Figure 6.3
Typical Vertical Drop Crest Configurations
(McLaughlin Water Engineers, Ltd. 1989)

2. To optimize crest and basin dimensions to achieve the most cost-effective 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.

Sloping Abutments at Drop Crest: Figure 6.4 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 in Section 6.3.2.4.

6.3.2.3 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. 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 its 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 is determined by comparison of the specific force, F_s, above and below the toe of the drop, using the following equation:

$$F_s = q^2 / gy + y^2 / 2 \tag{6.1}$$

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

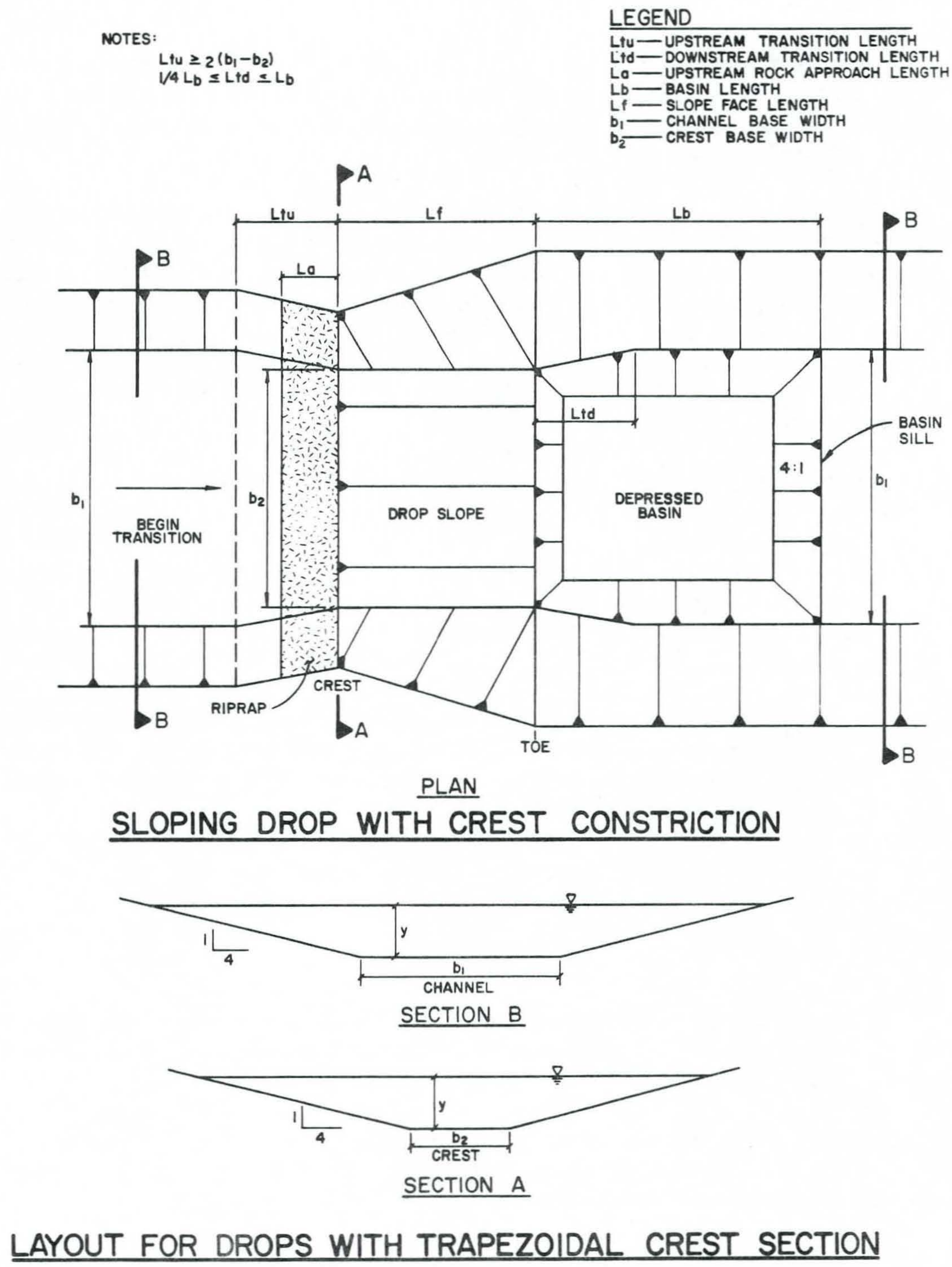


Figure 6.4
Typical Sloping Drop Crest Configuration
(McLaughlin Water Engineers, Ltd. 1989)

depressing the basin to a depth nearly as low as the downstream trickle channel elevation.

Analyses 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 6.5). 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 assure that the stilling basin length is sufficient to contain the jump.

An additional discussion of the hydraulic jump phenomenon is presented in Section 6.8.

6.3.2.4 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:

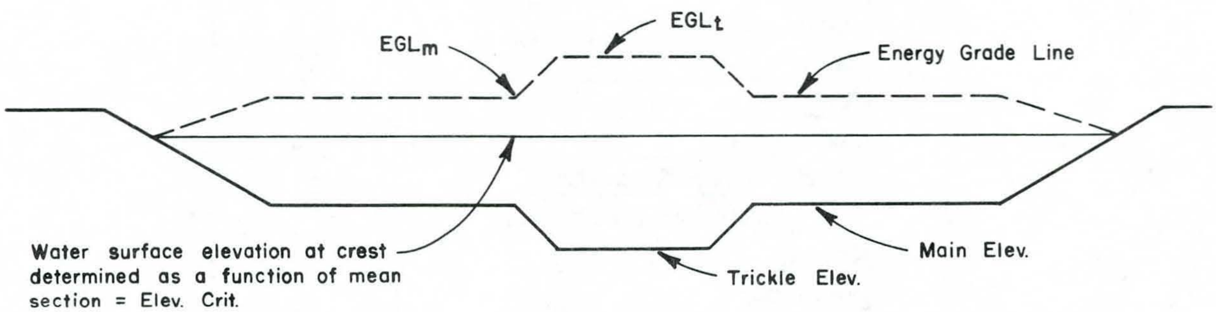
$$C_w = \frac{(L_H + 3L_V)}{3H} \tag{6.2}$$

3. Lane's recommended weighted-creep ratios are given for various foundation materials in Table 6.1. A definition sketch of the variables in Equation 6.2 is presented in Figure 6.6.

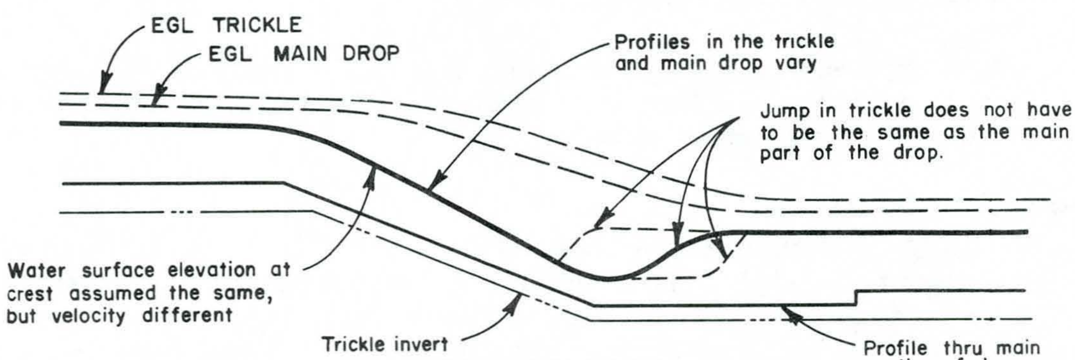
4. Reverse filter drains, weep holes, and pipe drains are aids to provide security from seepage, and recommended safe weighted-creep head ratios may be reduced as much as 10 percent, if used.

5. Care must be exercised to insure 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.



HYDRAULIC ANALYSIS
SECTION AT CREST OF DROP



HYDRAULIC ANALYSIS
PROFILE FOR SLOPING DROP

Figure 6.5
Typical Section and Profile for Sloping Drop
(McLaughlin Water Engineers, Ltd. 1989)

Table 6.1
Lane's Weighted-Creep: Recommended Ratios

Material	C_W 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

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 1974). 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.

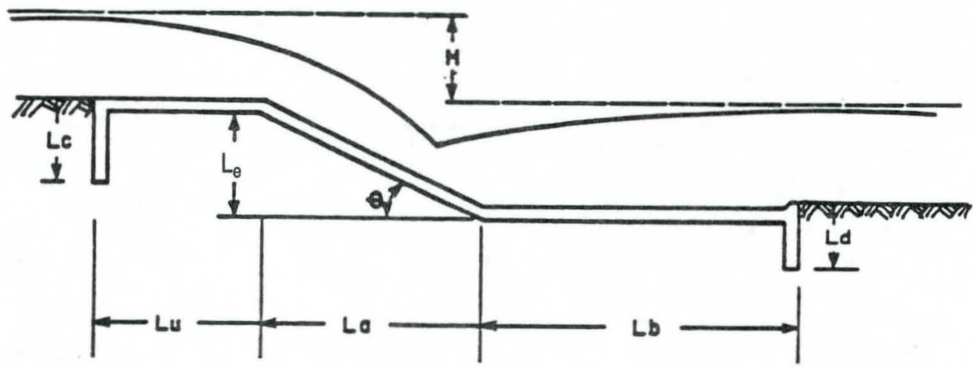
Locating a seepage cutoff wall upstream of the crest of a drop structure and using horizontal impervious blankets can be effective. It is also very important to control lateral seepage around the structure.

6.3.3 Drop Selection

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.

6.3.3.1 Surface Flow Hydraulic System: 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 6.3.2. Additional guidelines are also contained in Section 6.3.4.

6.3.3.2 Foundation and Seepage Control Systems: Table 6.2 presents some typical foundation conditions and control systems typically used for various drop heights. Table 6.2 is presented only as a guide. The hydraulic engineer must



$$\text{For } \theta < 45^\circ \quad — \quad \Sigma L_H = L_u + L_a + L_b$$

$$\Sigma L_V = 2(L_c + L_d)$$

$$\text{For } \theta > 45^\circ \quad — \quad \Sigma L_H = L_u + L_b$$

$$\Sigma L_V = 2(L_c + L_d) + L_e$$

where:

L_H	=	Horizontal seepage length, ft
L_V	=	Vertical seepage length, ft
L_u	=	Horizontal length from crest to upstream cutoff wall, ft
L_a	=	Horizontal length along face of drop structure, ft
L_b	=	Horizontal length along downstream basin, ft
L_c	=	Depth of cutoff wall, ft
L_d	=	Depth of cutoff at downstream end of basin, ft
L_e	=	Vertical drop height, ft

Figure 6.6
Definition Sketch for Weighted-Creep Theory for Nonporous Liner and Cutoffs
(Adapted from: Simons, Li and Associates 1981)

Table 6.2
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/SwB*	S/SwB*
	CTc	CTc/ST	ST	ST
	CTf	CTf/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 sheetpile support)

** (excavate into 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

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 on-site 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.

6.3.3.3 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 4 feet high versus the cost of two structures, each 2 feet high.

6.3.3.4 Construction Considerations: The selection of a drop and its foundation may also be tempered by construction difficulty, location, access, and material availability/delivery. Table 6.3 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.

6.3.4 Design Guidelines for Drop Structures

6.3.4.1 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, *Hydraulic Design of Stilling Basins and Energy Dissipators* (Peterka 1958), and *Design of Small Canal Structures* (Aisenbrey, et al 1974), that should be used for the design of these structures. Recent references include *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

Table 6.3
Construction Components Concerns and Quality Control Measures of Drop Structures

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.
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.
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 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 exessive 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 thickness 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 throughly 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 boulers should be placed with a gradall or multi-prong grapple device for ease of 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 6.3 (continued)

Type	Quality Concerns	Quality Control Measures and Inspection
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.
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 manufacturers instructions.
Seepage Cutoff 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.
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 inspection 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.

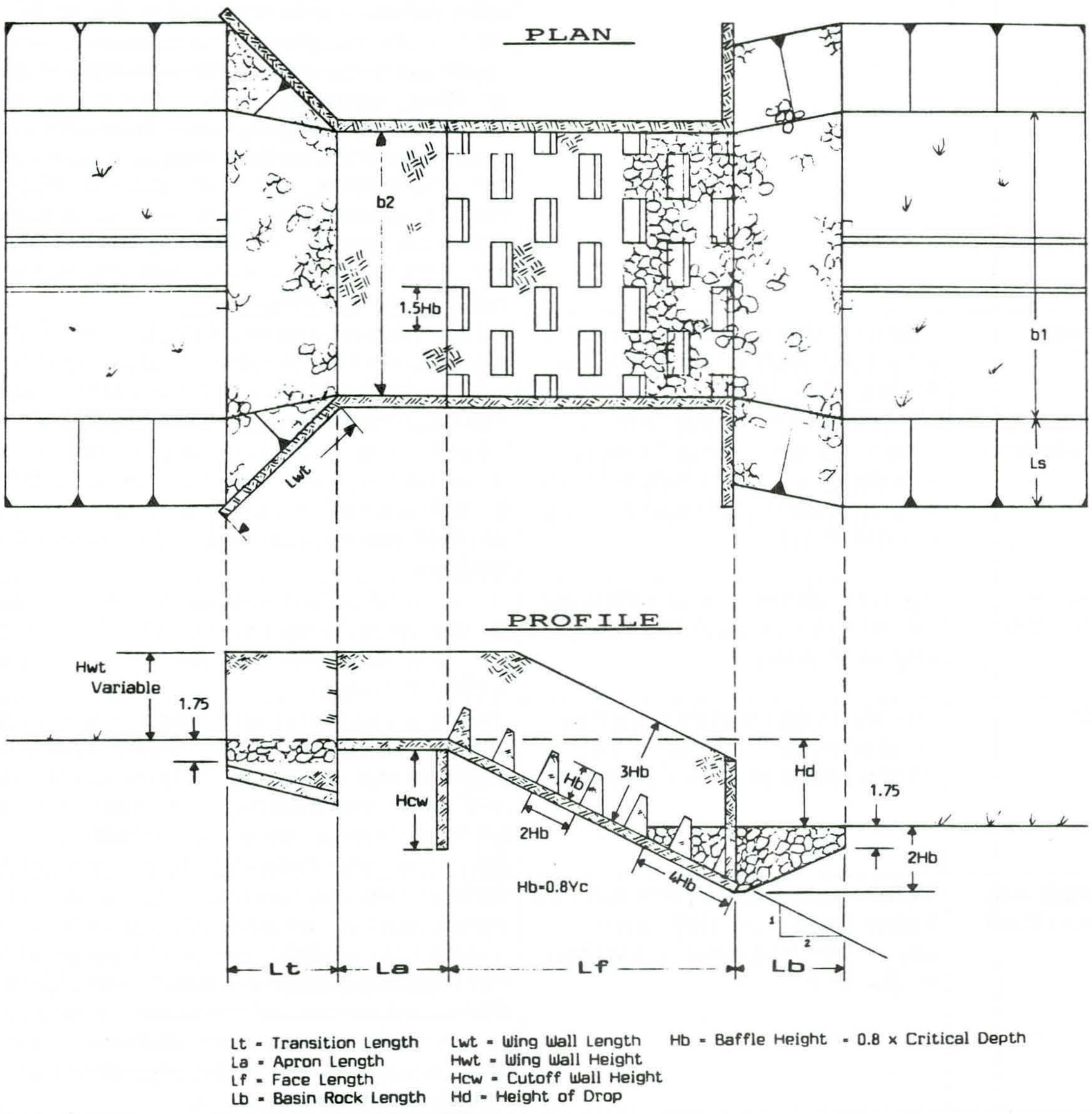


Figure 6.7
Baffle Chute Drop
(McLaughlin Water Engineers, Ltd. 1989)

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.

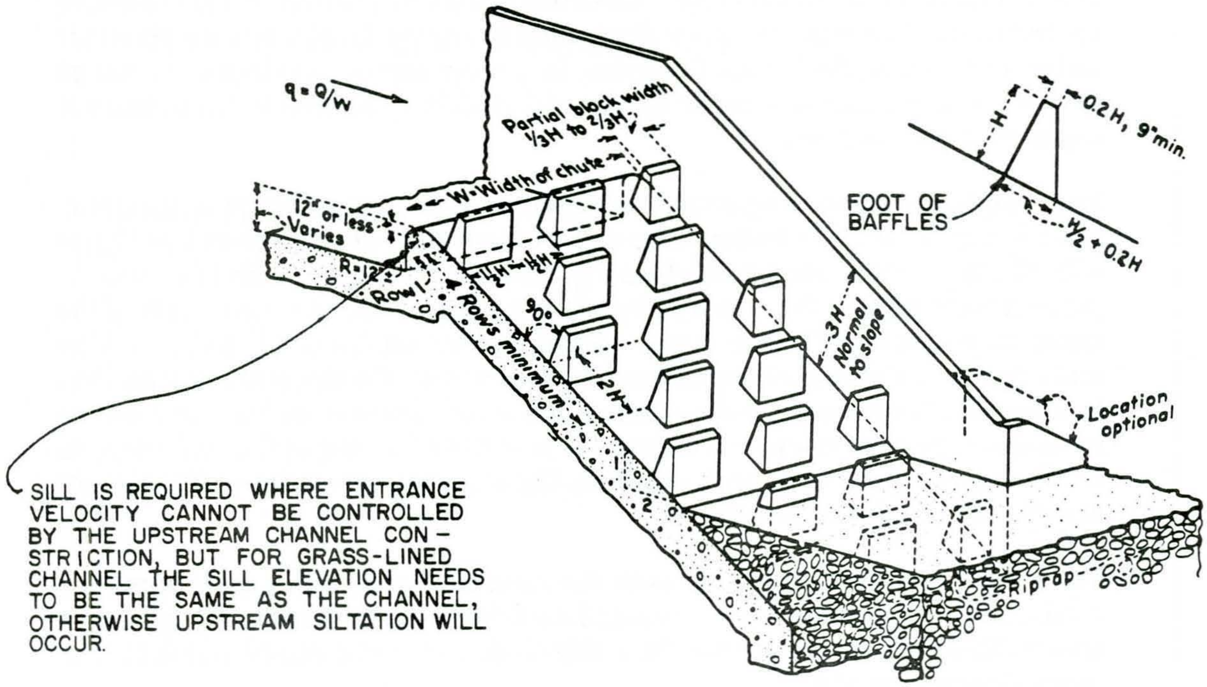
Typical design consists of upstream transition walls, a rectangular approach chute, a sloping apron of 2:1, or flatter, slope with multiple rows of baffle piers (see Figure 6.7). The toe of the chute extends below grade and is backfilled with loose rock to prevent undermining the structure by eddy currents 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.

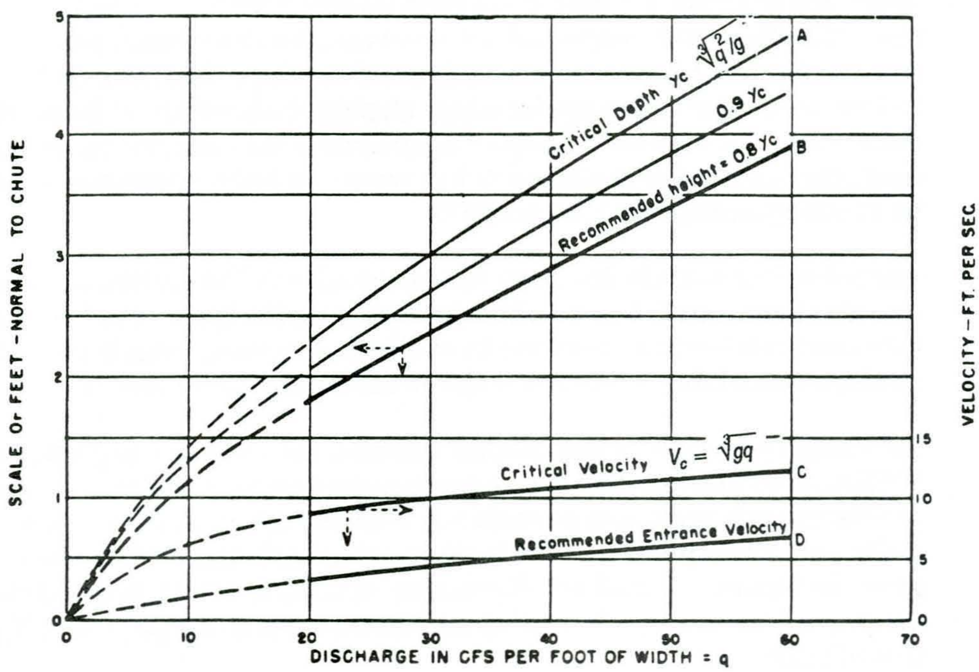
This design is quite flexible in adaption, once the hydraulic principles are understood. For example, the design has been modified for low drops by locating two rows of baffles on the slope and two rows on a horizontal extension of the chute. Another approach has been to use a flatter chute slope than 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 (Simons 1982).

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 Figures 6.8 and 6.9. Remaining structural design parameters must be determined for specific site conditions. Recommended design procedures are discussed below.



(A) USBR ISOMETRIC



(B) DESIGN CRITERIA

Figure 6.8
Baffle Chute Design Criteria
(Adapted from: Peterka 1958)

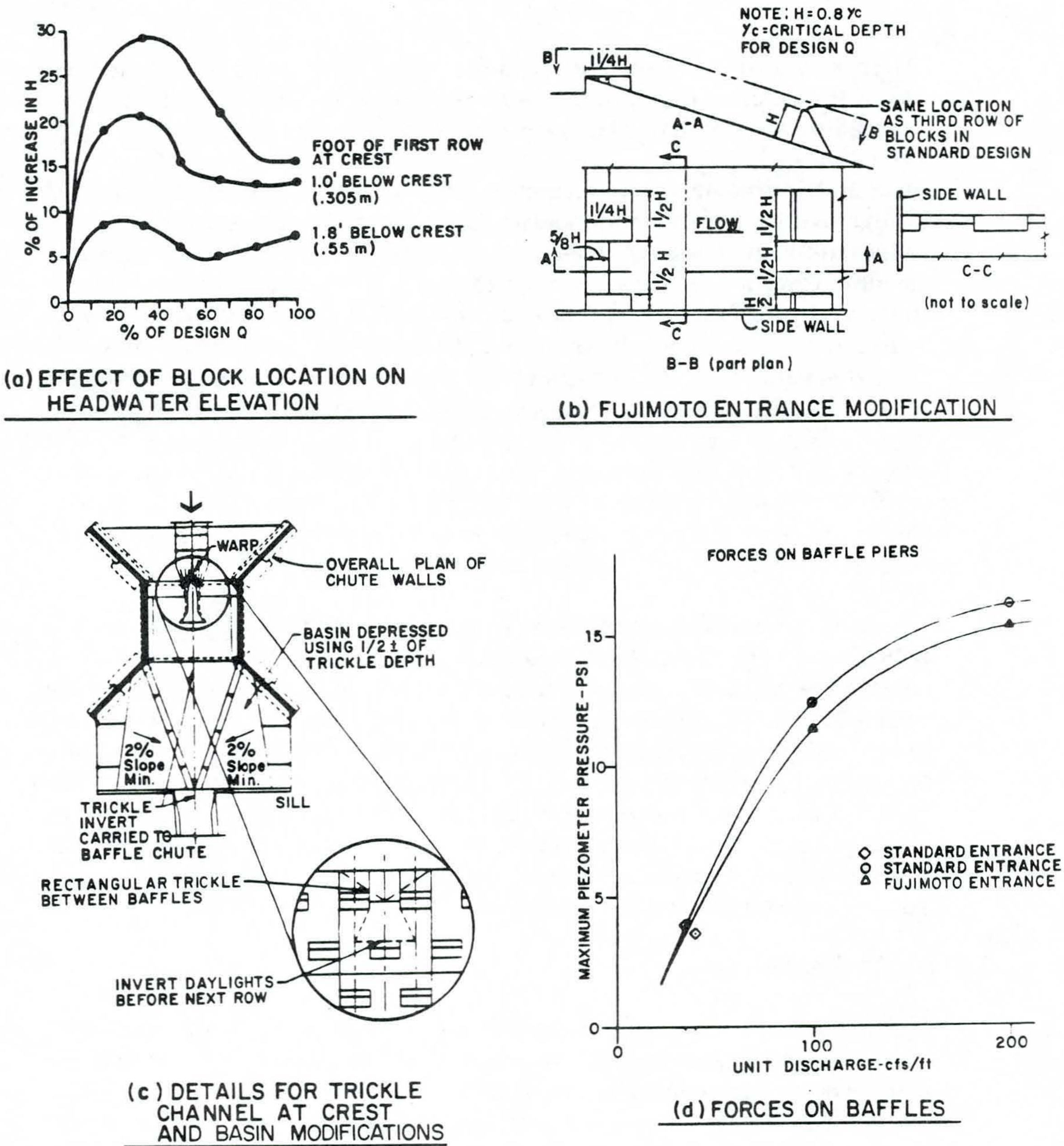


Figure 6.9
Baffle Chute Crest Modifications and Forces
(Adapted from: Peterka 1958)

General Hydraulic Design Procedure:

1. Determine the maximum inflow rate and the design unit discharge:

$$q_D = Q/W \tag{6.3}$$

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.

2. 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.

3. 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 6.8(b) gives the USBR recommended entrance (channel) velocity. In a typical grass-lined 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 6.9(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 6.9(c) illustrates one method of designing the low flow channel through the crest.

2. The conventional design shown in Figure 6.8(b) results in the top elevation of the baffles being higher than the crest, which causes a higher backwater surface effect upstream. Figure 6.9(a) may be used to estimate 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 6.9(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:

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 6.8(b) may be extrapolated:

$$y_c = (q^2/g)^{0.33} \tag{6.4}$$

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 4, below).

   Baffle pier widths and spaces should equal, preferably, about 1.5 H 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 $2H_b$ 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 are 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 6.3.2.4).

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 6.9(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 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 straight forward to construct. Steel, formwork, concrete placement and finish, and backfill require periodic inspection.

6.3.4.2 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 is limited to 3 feet for safety considerations.

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 6.10 shows a vertical drop with a grouted boulder basin (concrete may also be used), and illustrates several important design considerations.

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. (Refer to Section 6.3.2.2.)

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 area as discussed in Section 6.3.2.3. Add subscript *t* for the trickle channel area and subscript *m* for the main channel area in the following equations.

   Refer to Figure 6.11 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

$$L_b \geq L_d + D_j + 9.6Y_2 \qquad \textbf{(6.5)}$$

   When a hydraulic jump occurs immediately where the nappe hits the basin floor, the following variables are defined:

$$L_d / Y_f = 4.3\, D_n^{\;0.27} \qquad \textbf{(6.6a)}$$

   where

$$D_n = q_c^2/(gY_f^{\,3}) \qquad \textbf{(6.6b)}$$

$$Y_p / Y_f = 1.0\, D_n^{\;0.22} \qquad \textbf{(6.7)}$$

$$Y_1 / Y_f = 0.54\, D_n^{\;0.425} \qquad \textbf{(6.8)}$$

$$Y_2 / Y_f = 1.66\, D_n^{\;0.27} \qquad \textbf{(6.9)}$$

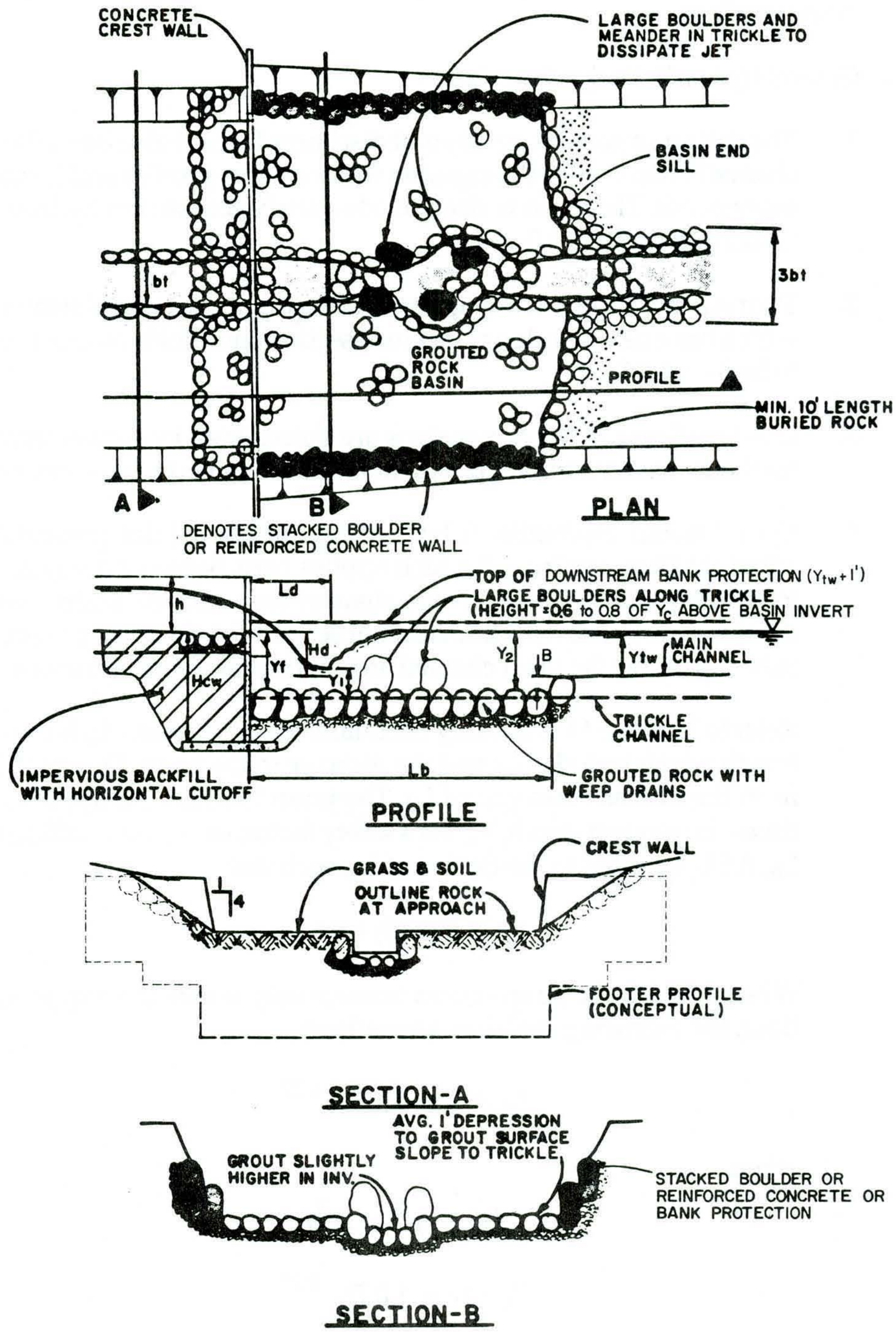


Figure 6.10
Vertical Hard Basin Drop
(McLaughlin Water Engineers, Ltd. 1989)

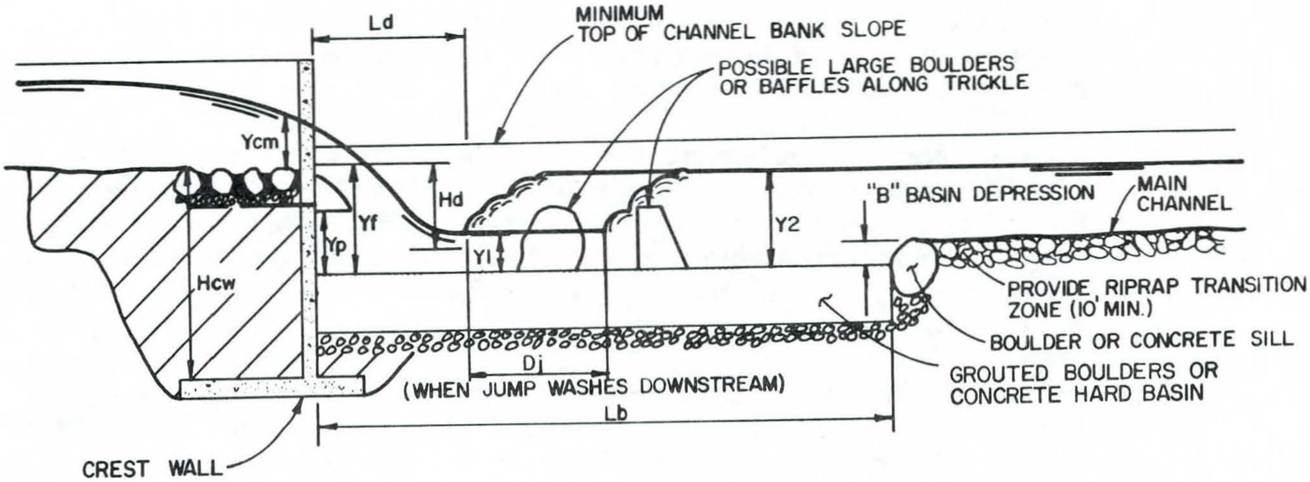


Figure 6.11
Vertical Drop Hydraulic System
(McLaughlin Water Engineers, Ltd. 1989)

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:

$$L_{bm} = L_{dm} + D_{jm} + 0.60\,(6Y_2)_m \tag{6.10}$$

   b. At the trickle zone, standard design:

$$L_{bt} = L_{dt} + D_{jt} + 0.60\,(6Y_2)_m \tag{6.11}$$

   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:

$$L_{bt} = L_{dt} + 0.60\,(6Y_2)_t \tag{6.12}$$

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.

6.3.4.3 Vertical Riprap Basin Drops: As shown in Figure 6.12, 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 is limited to 3 feet due to safety considerations and the practicality of obtaining large basin riprap for higher drops. Submergence by high tailwater can limit the dissipation efficiency.

The hydraulic design was developed through model testing by C.D. Smith and D.K. Strang in 1967 (*Scour in Stone Beds*) and design procedures were further developed by M.A. 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 in Section 6.3.4.2 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 6.12 that:

$$H_d = D - 0.67\, d_s \qquad \textbf{(6.13)}$$

The designer must find the combination of rock size and jet plunge height D that gives a depth of scour which balances Equation 6.13. The relation between rock size d_{50}, jet plunge height D, head on the weir, H, ($H = 1.5Y_c$) and depth of scour d_s is given in Figure 6.13. As these values will be different in the main drop and the trickle, the design d_{50} and/or d_s will vary.

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 0.3D. 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.

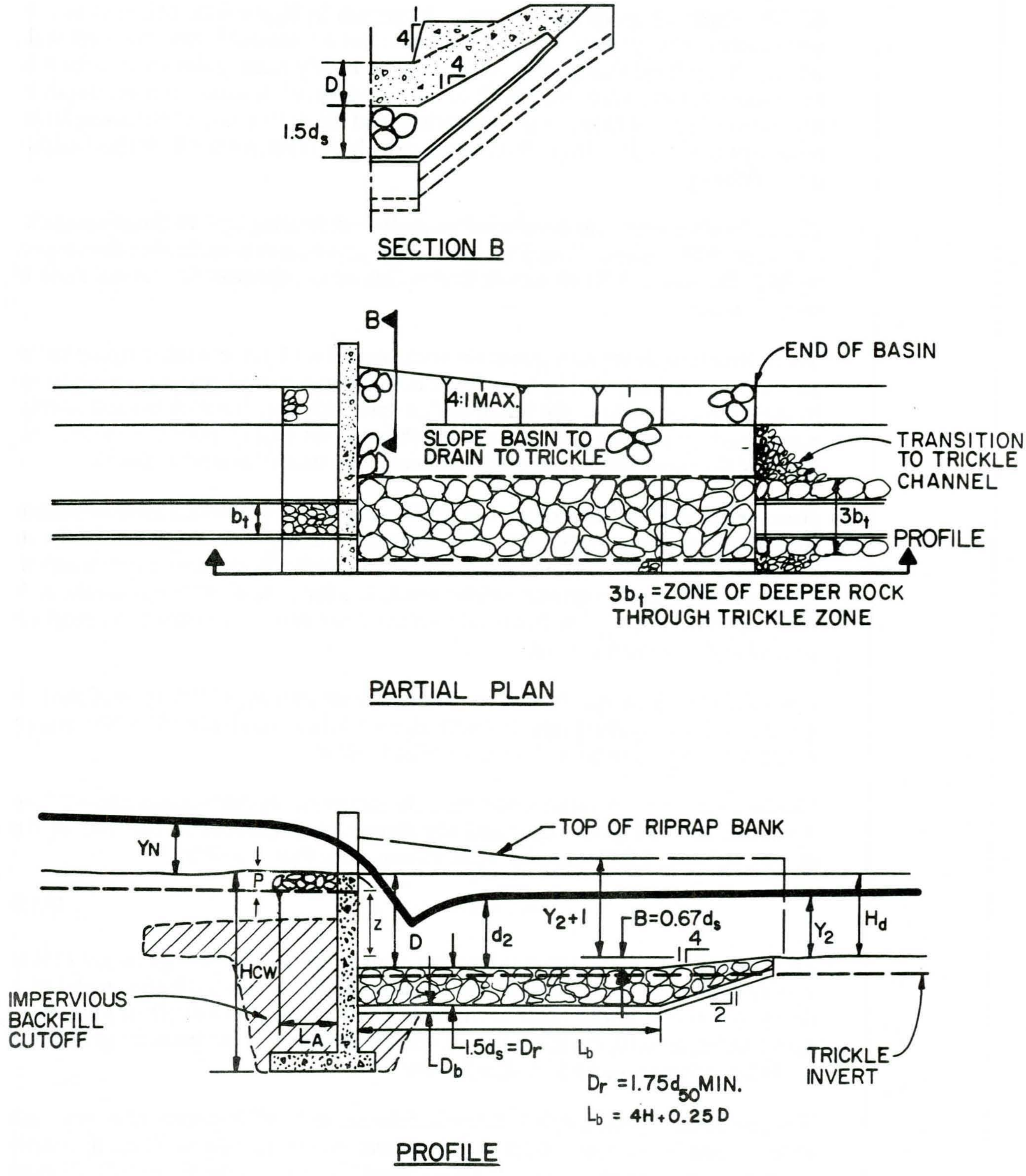


Figure 6.12
Vertical Riprap Basin Drop
(Stevens 1981)

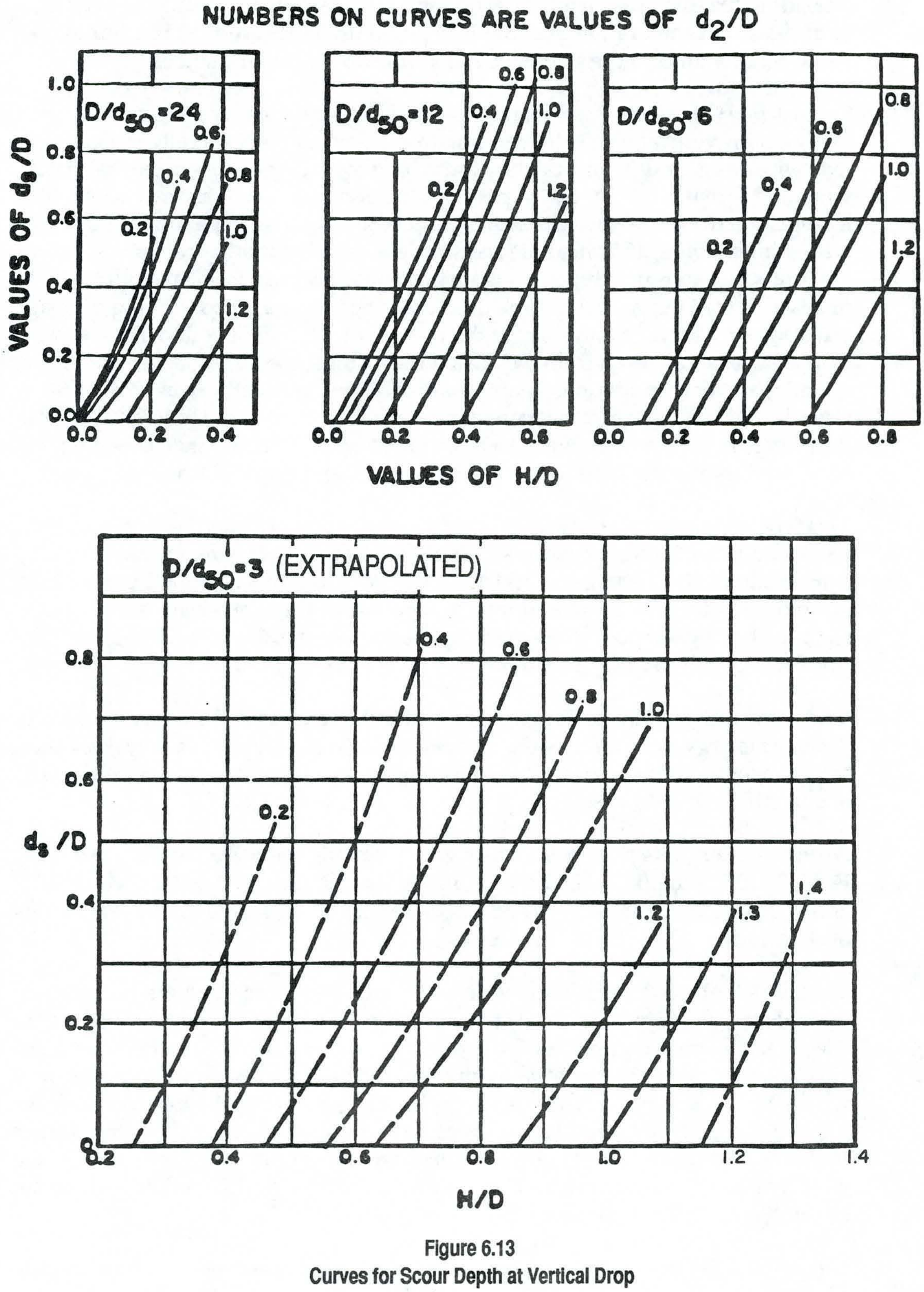


Figure 6.13
Curves for Scour Depth at Vertical Drop
(Stevens 1981)

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 pile 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 straight forward. 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.

6.3.4.4 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 (Corry et al, 1975; and Peterka 1958). These drops and associated basins are suited for different kinds of situations.

The Saint Anthony Falls Stilling Basin 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 would be 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 6.14 illustrates the various types of stilling basins for use with sloping concrete drops.

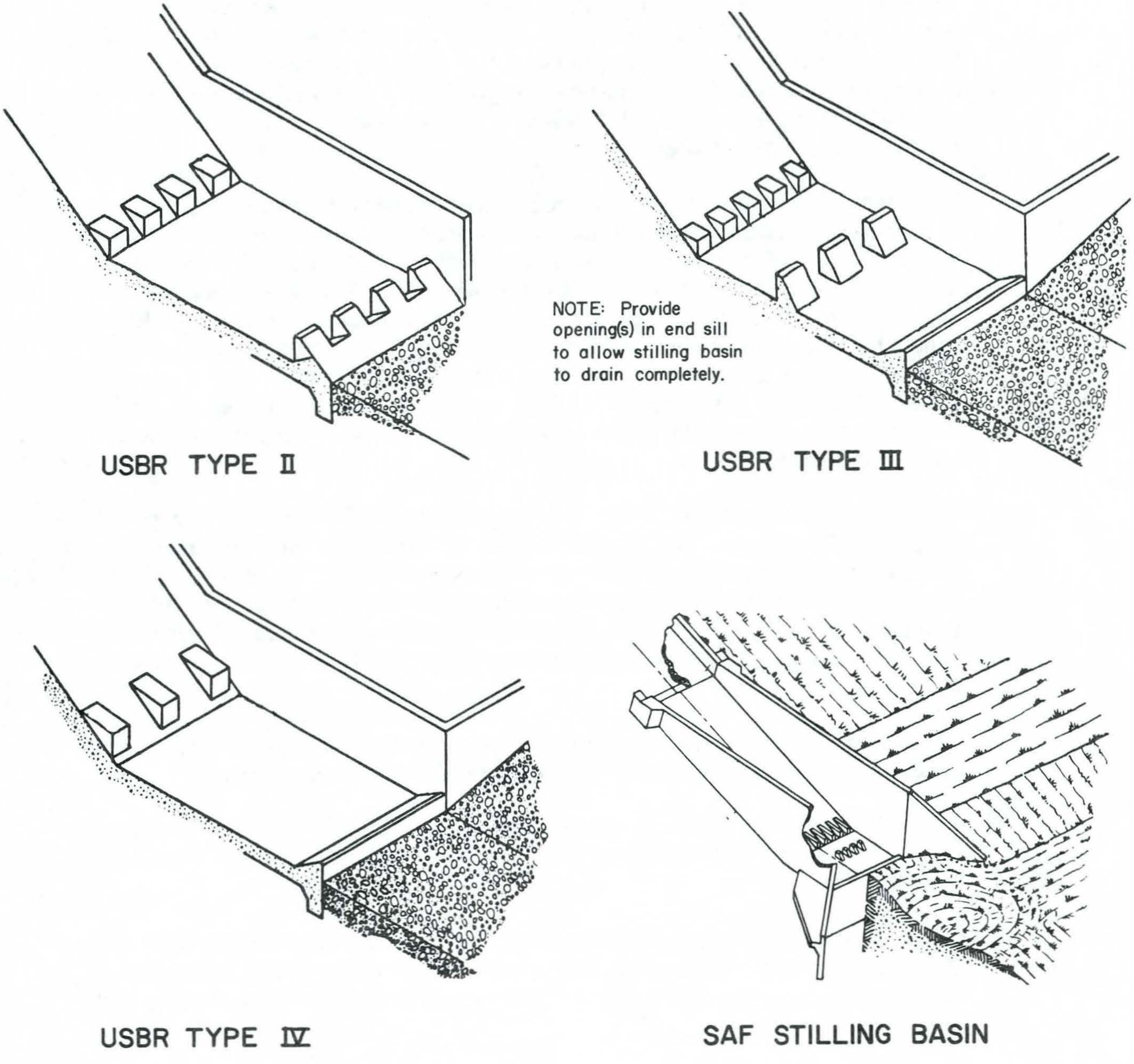


Figure 6.14
Stilling Basins for Sloping Concrete Drops
(Adapted from: Corry, et al 1975)

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* (Corry et al, 1975) and *Hydraulic Design of Stilling Basins and Energy Dissipators* (Peterka 1958).

Analysis of channel approach and crest hydraulics generally follows the guidelines presented in Section 6.3.2. 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 6.3.2.4), 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.

6.3.4.5 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 Maricopa 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.

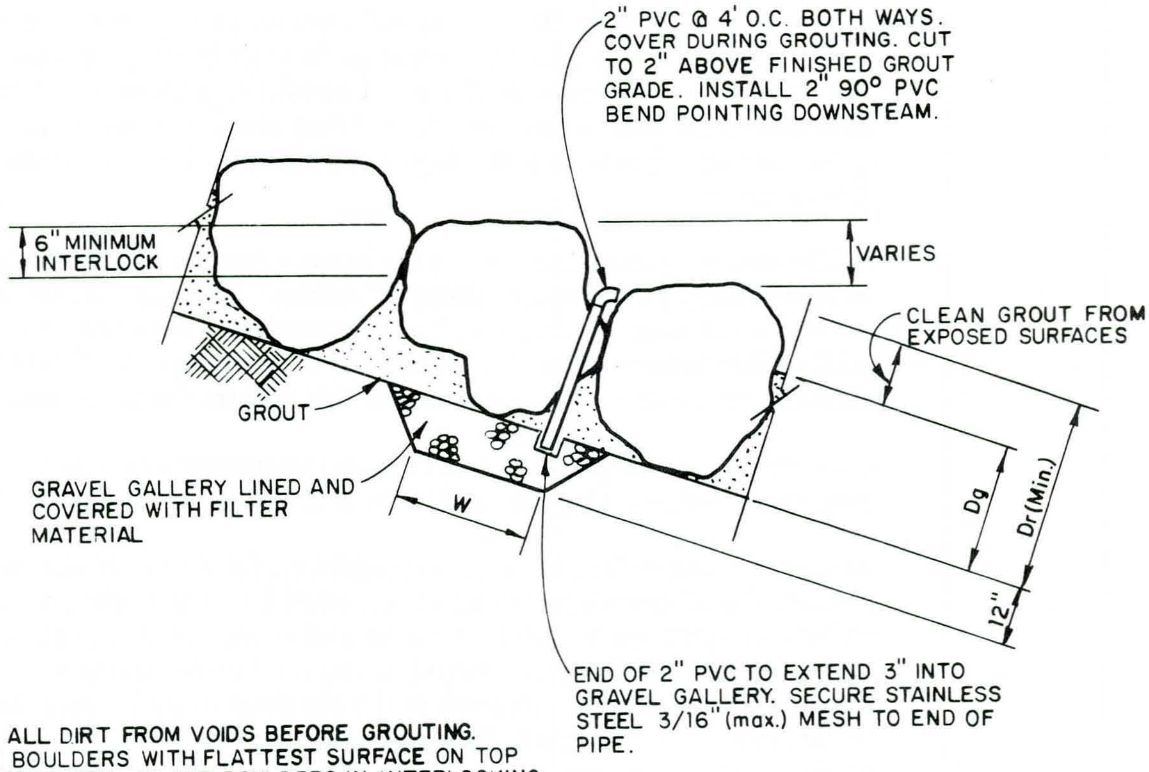
Addition of roughness elements on the face of a sloping concrete drop can provide increased energy dissipation. "Stepped" concrete has been successfully applied at spillways and drop structures. Roller compacted concrete is a methodology that can achieve the stairstep geometry on the face of a sloping drop. Reinforced concrete steps can be constructed by standard construction methods on small structures.

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 6.15).

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 manpower 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 and placed with a large flat surface horizontal has been demonstrated to be successful (see Taggart 1986).

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.



NOTES:

1) CLEAN ALL DIRT FROM VOIDS BEFORE GROUTING.
2) PLACE BOULDERS WITH FLATTEST SURFACE ON TOP AND HORIZONTAL. PLACE BOULDERS IN INTERLOCKING FASHION, TOUCHING ADJACENT BOULDERS AND MINIMIZING VOIDS.
3) DEPTH OF ROCK (Dr) AND DEPTH OF GROUT (Dg) AS CALLED OUT ON PLANS.
4) GROUT TO PENETRATE FULL DEPTH OF ROCK AND FILL ALL VOIDS.
5) CLEAN ALL GROUT FROM EXPOSED SURFACES.
6) GRAVEL GALLERIES ARE CALLED OUT ON PLANS AS "W" (width of gallery) GRAVEL GALLERY (ies) "S" (spacing between galleries) O.C.

Figure 6.15
Grouted Boulder Placement
(McLaughlin Water Engineers, Ltd. 1989)

Other USBR Basins: Some other stilling basins developed by the USBR (Peterka 1958) 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.

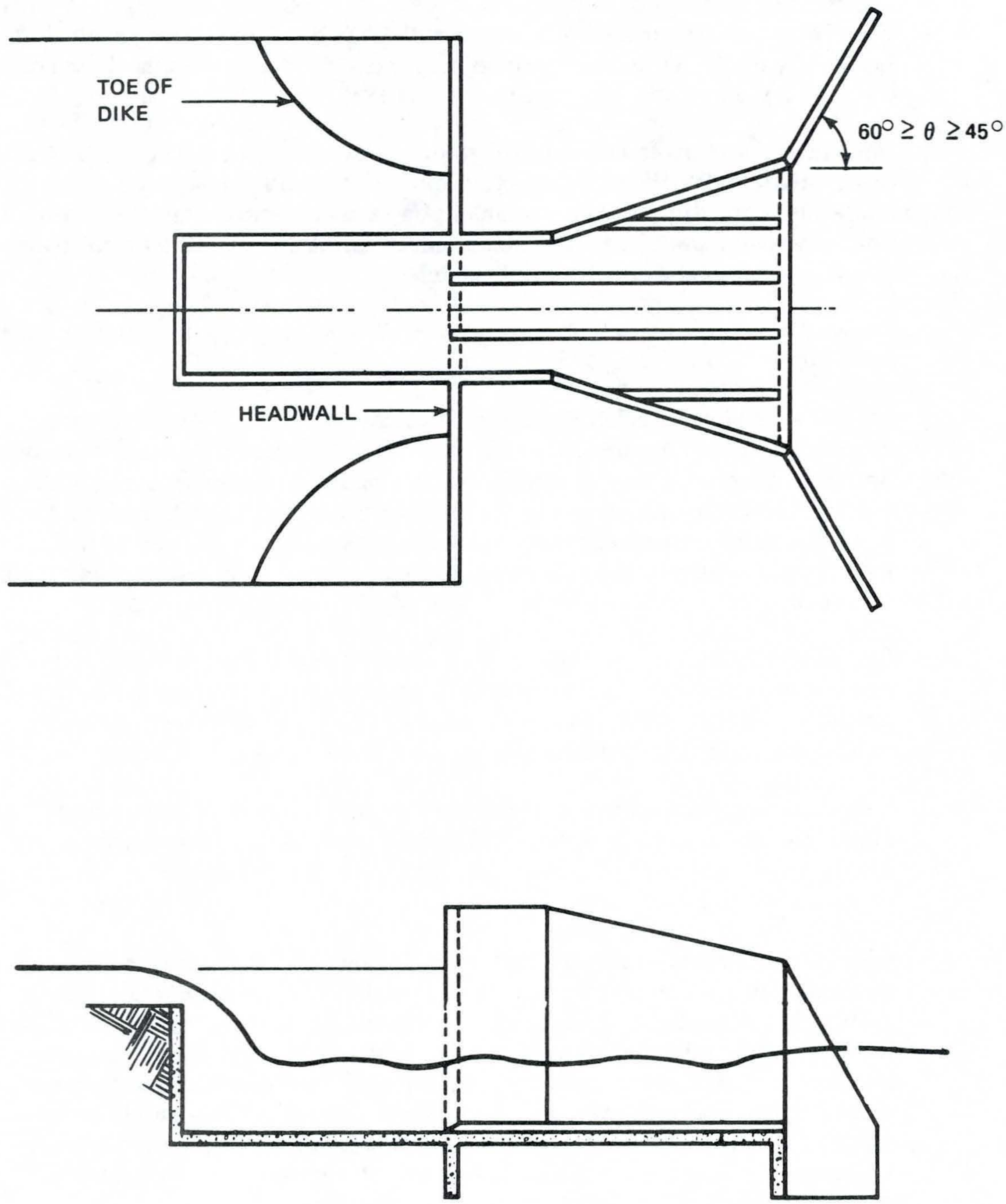


Figure 6.16
Box Inlet Drop Structure
(Adapted from: Corry, et al 1975)

Box Inlet Drop Structure: The box inlet drop structure may be described as a rectangular box open at the top and downstream end (Figure 6.16). Water is directed to the crest of the box inlet by earth dikes and headwalls. A 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.

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* (Corry, et al 1975).

6.3.4.6 Low Flow Check Structures: Low flow check structures and associated erosion control techniques can be effective in stabilizing natural channels and other unlined channels. With the advent of floodplain management and regulation, private developers are frequently directed to preserve the floodplain. Unfortunately, urbanization creates more frequent and sustained flows. The overall floodplain may remain relatively stable, but the low flow channel becomes more susceptible to erosion.

Stream stabilization of base flow channels involves determination of the stable slope and configuration for a variety of frequent runoff rates, with particular emphasis on the dominant discharge (mean annual flood). Local soils, bed materials, and sediment gradation must be considered.

Low flow check structures are designed to provide control points and establish/maintain stable bed slopes within the base flow channel. Other options include low flow (trickle) channel lining, toe riprap, control sills across the floodplain, revetments, and groins.

Check structures are frequently submerged during higher flood events. The application and sizing is complex because of the need to address a wide range of flows. Although the checks may stabilize the channel for low flows, they may be in jeopardy from mid- to high-range flows as water goes around the check abutments. Extensive care is needed with seepage cutoff and abutments that key far back into areas that are less likely to be damaged during high flows. Care should be taken to have a depressed stilling area to avoid a secondary drop at the end to the drop. In any case, ongoing maintenance of check structures will be likely and should be considered in the design so later repairs are practicable.

6.4 Conduit Outlet Structures

6.4.1 General

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. Outlets 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.

6.4.2 Riprap Protection at Conduit Outlets

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* (Corry, et al 1975), 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 the dimensions 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, and the scour hole functioned very efficiently as an energy dissipator. The concentrated flow at the culvert brink plunged into the hole, a jump formed against the downstream extremity of the scour hole, and 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 in Figure 6.17. Principal features of the basin 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 greater than 2 and less than 4.

» The length of the energy dissipating pool, L_s, is 10 h_s or $3W_o$ whichever is larger. The overall length of the basin, L_b, is 15 h_s or $4W_o$ whichever is larger.

6.4.2.1 General Hydraulic Design Procedure

1. Estimate the flow properties at the brink of the culvert. Establish the brink invert elevation such that $TW/y_o \leq 0.75$ for design discharge.

2. For subcritical flow conditions (culvert set on mild or horizontal slope), use Figure 6.18 or Figure 6.19 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 ($y_e = (A/2)^{1/2}$). 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 6.20, 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 6.17.

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 6.17), 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.

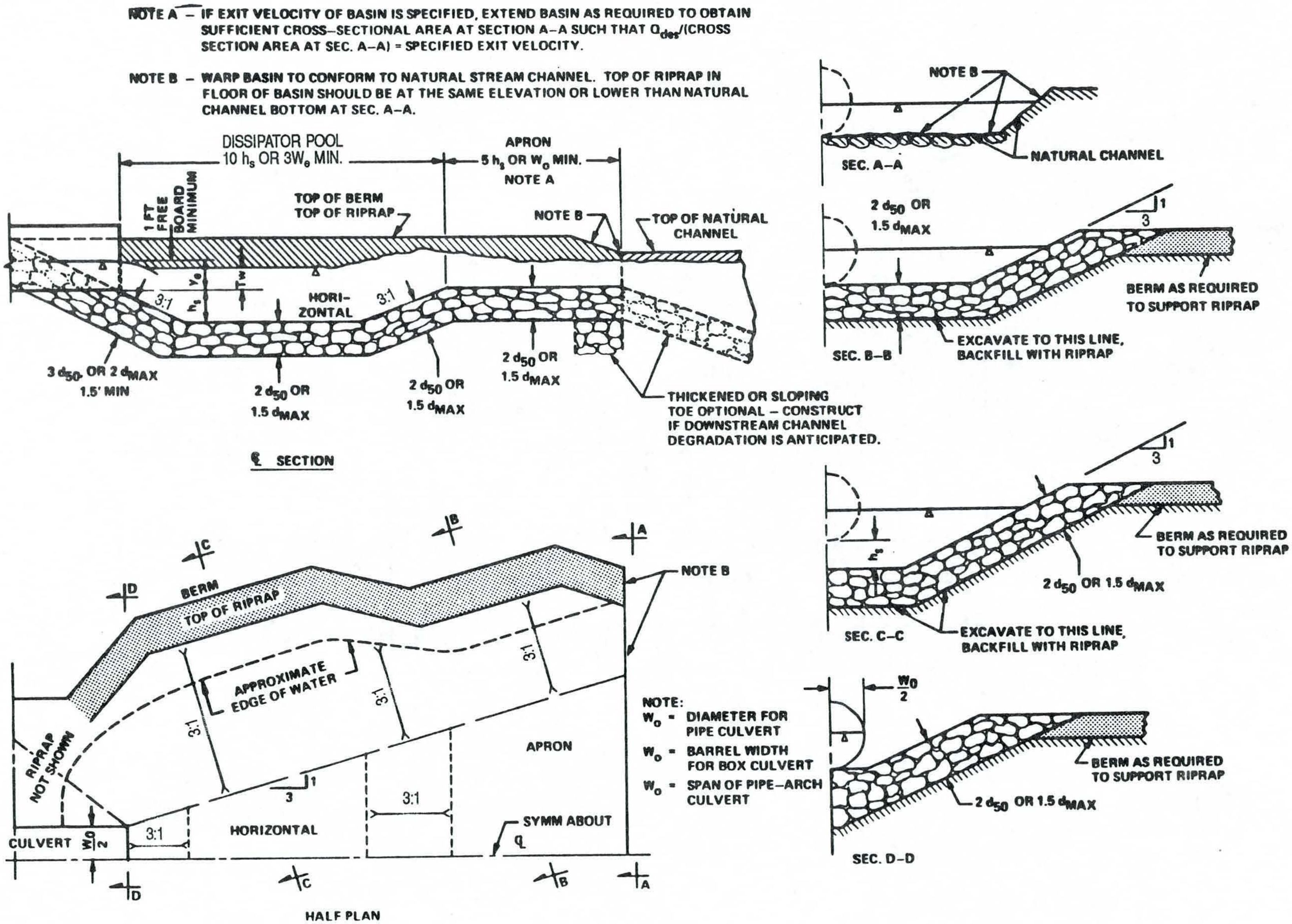


Figure 6.17
Details of Riprapped Culvert Energy Basin
(Corry, et al 1975)

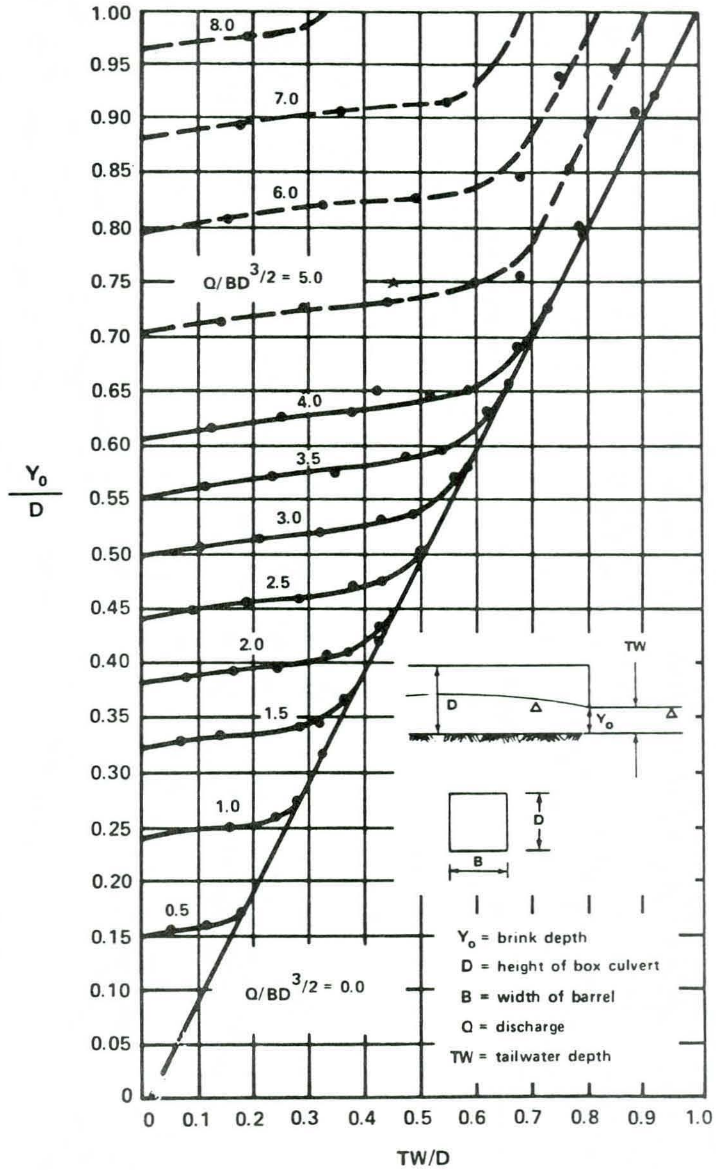


Figure 6.18

Dimensionless Rating Curves for the Outlets of Rectangular Culverts on Horizontal and Mild Slopes

(Simons, et al 1970)

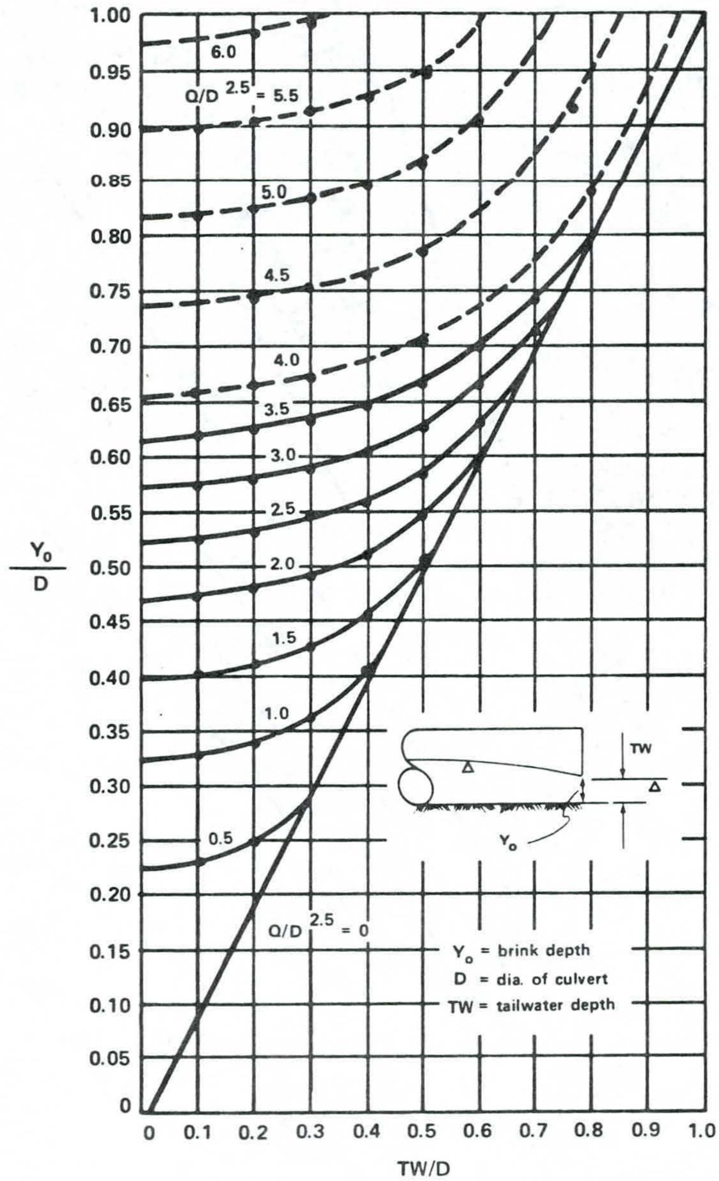


Figure 6.19
Dimensionless Rating Curves for the Outlets of Circular Culverts on Horizontal and Mild Slopes
(Simons, et al 1970)

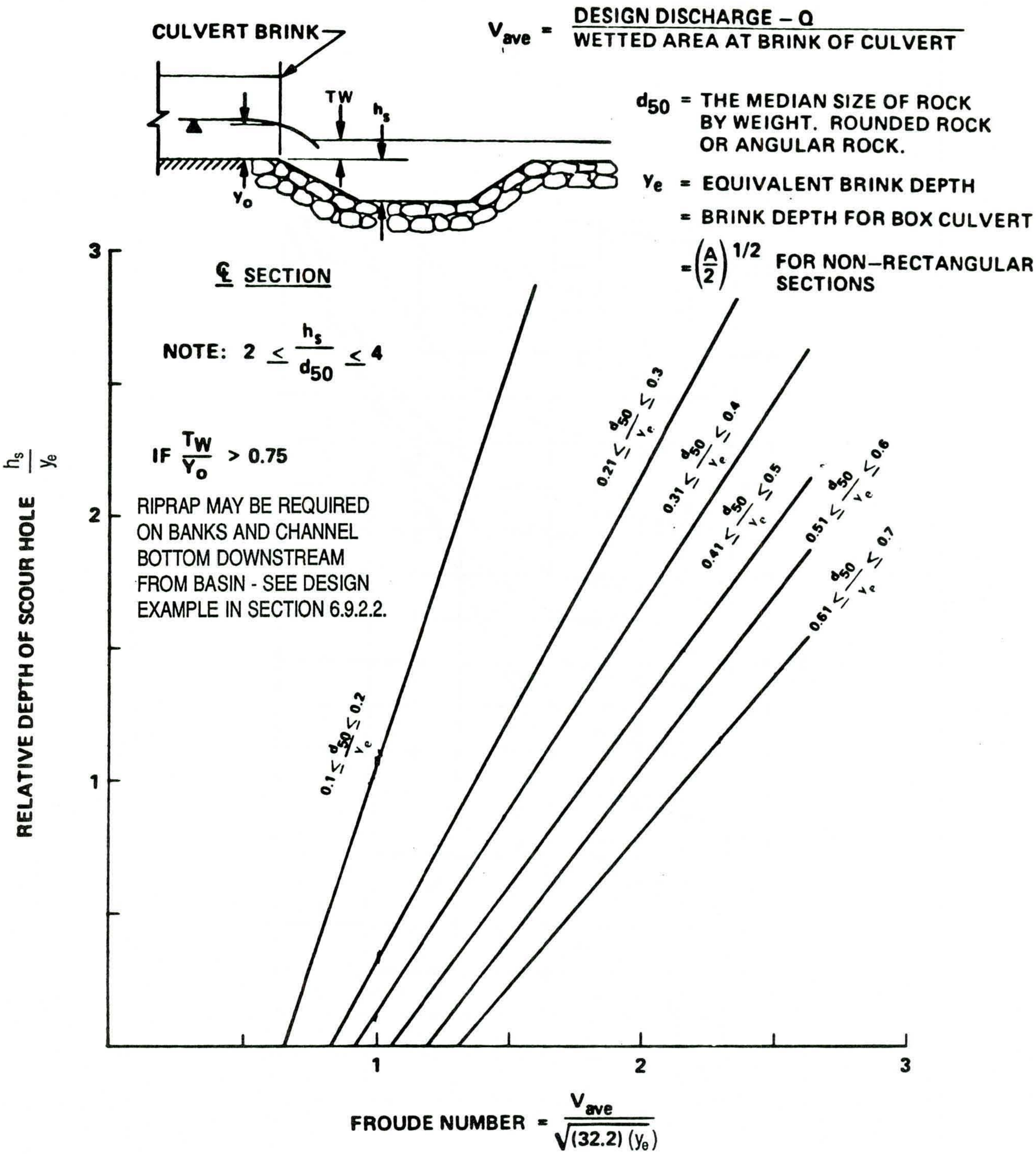


Figure 6.20
Relative Depth of Scour Hole versus Froude Number at Brink of Culvert with Relative Size of Riprap as a Third Variable
(Corry, et al 1975)

8. If high tailwater is a possibility and erosion protection is necessary for the downstream channel, the following design procedure is suggested:

   Design a conventional basin for low tailwater conditions in accordance with the instructions above. Estimate centerline velocity at a series of downstream cross sections using the information shown in Figure 6.21. Shape downstream channel and size riprap using guidelines presented in Chapter 5 and the stream velocities obtained above.

Additional information regarding design of riprap basins for conduit outlets may be found in *Hydraulic Design of Energy Dissipators for Culverts and Channels* (Corry, et al, 1975).

6.4.3 Concrete Outlet Structures

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 that are empty or have low water levels.

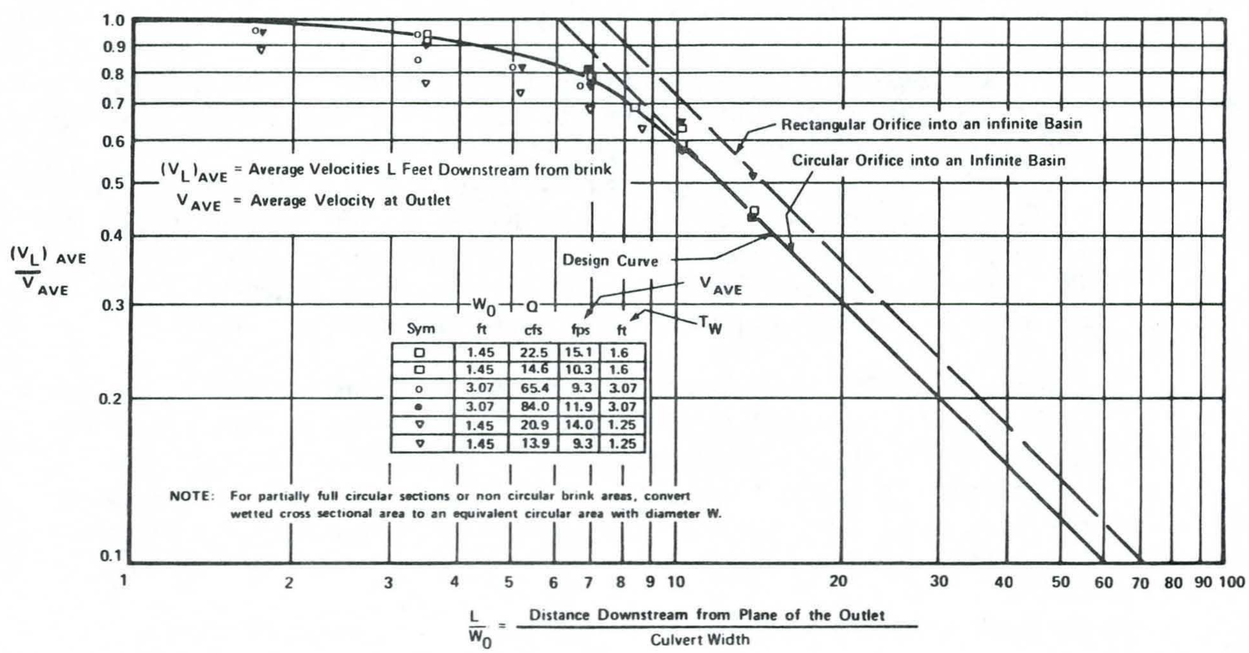


Figure 6.21
Distribution of Centerline Velocity for Flow from Submerged Outlets
(To be used for predicting channel velocities downstream from culvert outlets where high tailwater prevails.)
(Simons, et al 1970; and Corry, et al 1975)

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.

6.4.3.1 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 1958). Additional structural details are provided in *Design of Small Canal Structures* (Aisenbrey, et al 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 criteria 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.

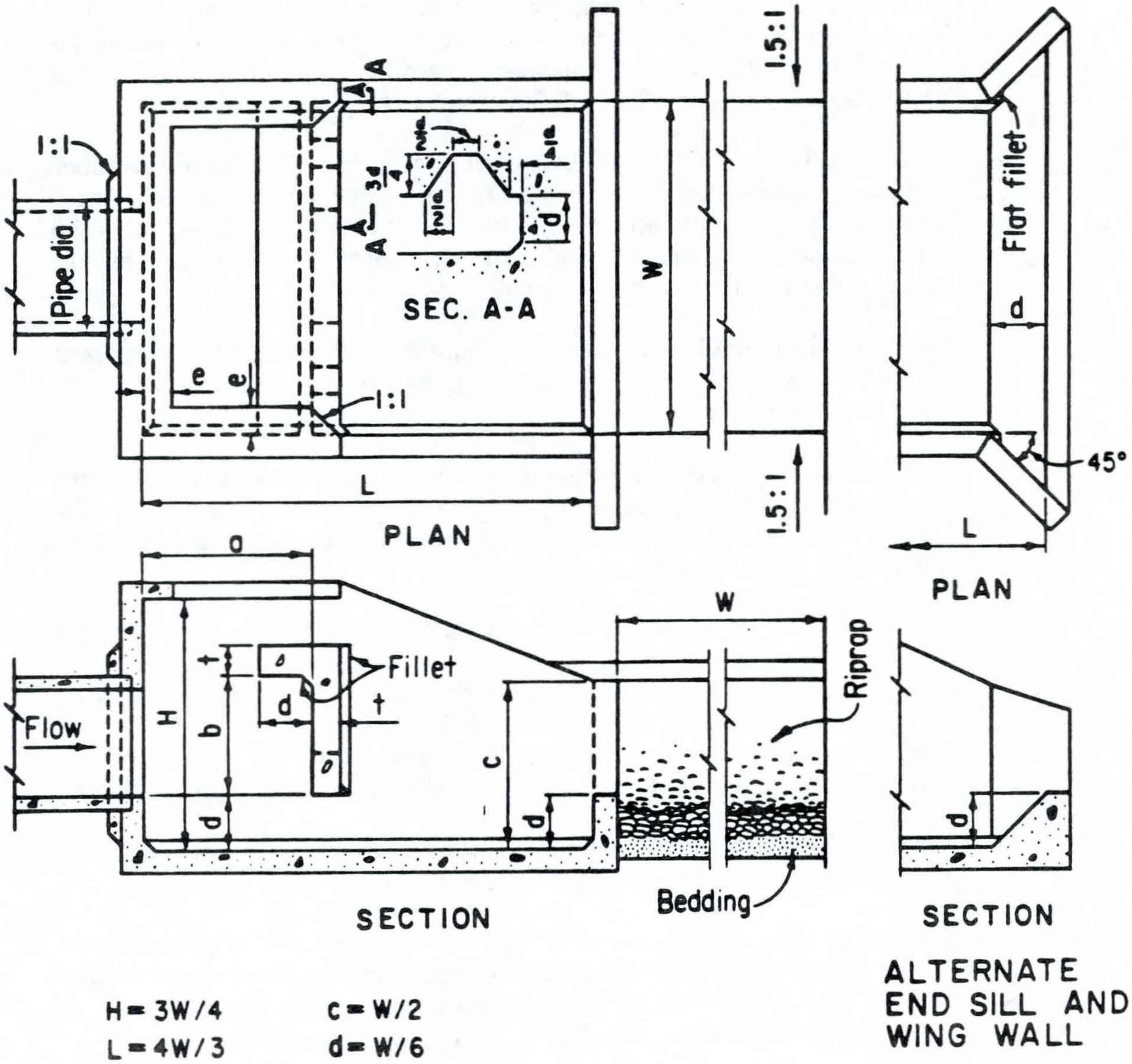
Energy dissipation is accomplished through momentum transfer as flow entering the basin impacts a large overhanging baffle. At high flow, further dissipation is produced as water builds up behind the baffle to form a highly turbulent backwater zone. Flow is then re-directed under the baffle to the open 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.

The generalized design configuration (Figure 6.22) 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.

General Hydraulic Design Procedure for Stilling Basins (see Figure 6.22):

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 6.8. 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 $= V/(gD)^{0.5}$.



H = 3W/4	c = W/2
L = 4W/3	d = W/6
a = W/2	e = W/12
b = 3W/8	t = W/12 suggested minimum
	Riprap stone size diameter = W/20

NOTE: See Figure 6.23 for W

Figure 6.22
General Design of the USBR Type VI Impact Stilling Basin
(Adapted from: Peterka, et al 1958)

2. The entrance pipe should be turned 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 6.23. The remaining dimensions are proportional to the basin width according to the legend in Figure 6.22. 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:

$$F_m = \rho V Q = 1.94\, VQ \tag{6.14}$$

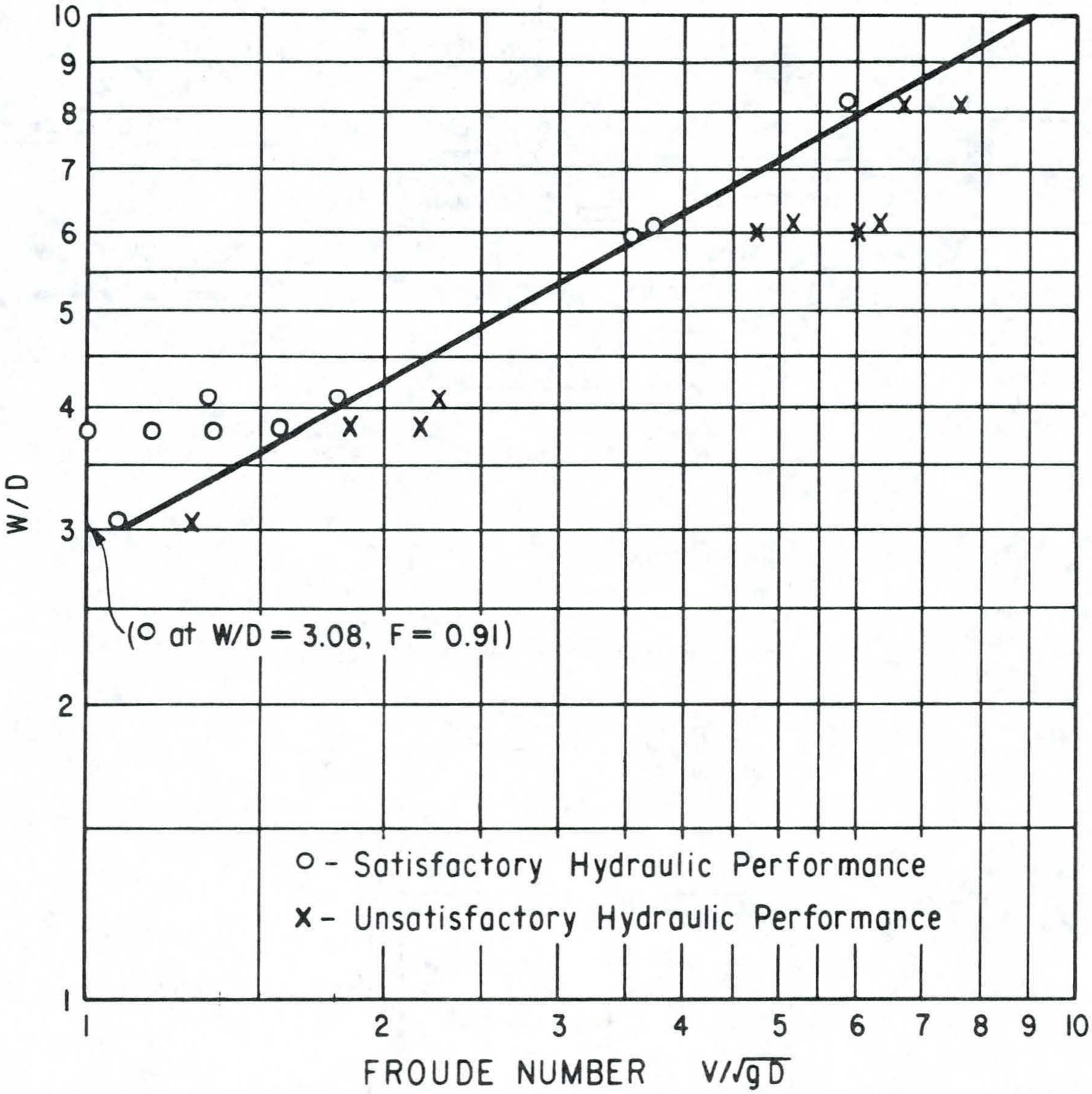
6. Riprap with a minimum D_{50} of 18 inches should be provided in the receiving channel from the end sill to a minimum distance equal to the basin width. The depth of rock should be equal to the sill height or at least 2.5 feet. Rock may be buried below finished grades and the area vegetated as desired to match the site.

7. The alternate end sill and wingwall shown in Figure 6.22 is recommended for all grass lined channels to reduce the scour potential below the sill wall.

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 6.24.

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.



"W" is the inside width of the basin.
"D" represents the depth of flow entering the basin and is the square root of the flow area at the conduit outlet.
"V" is the velocity of the incoming flow.
The tailwater depth is uncontrolled.

Figure 6.23
Design Width of the USBR Type VI Basin
(Adapted from: Peterka, et al 1958)

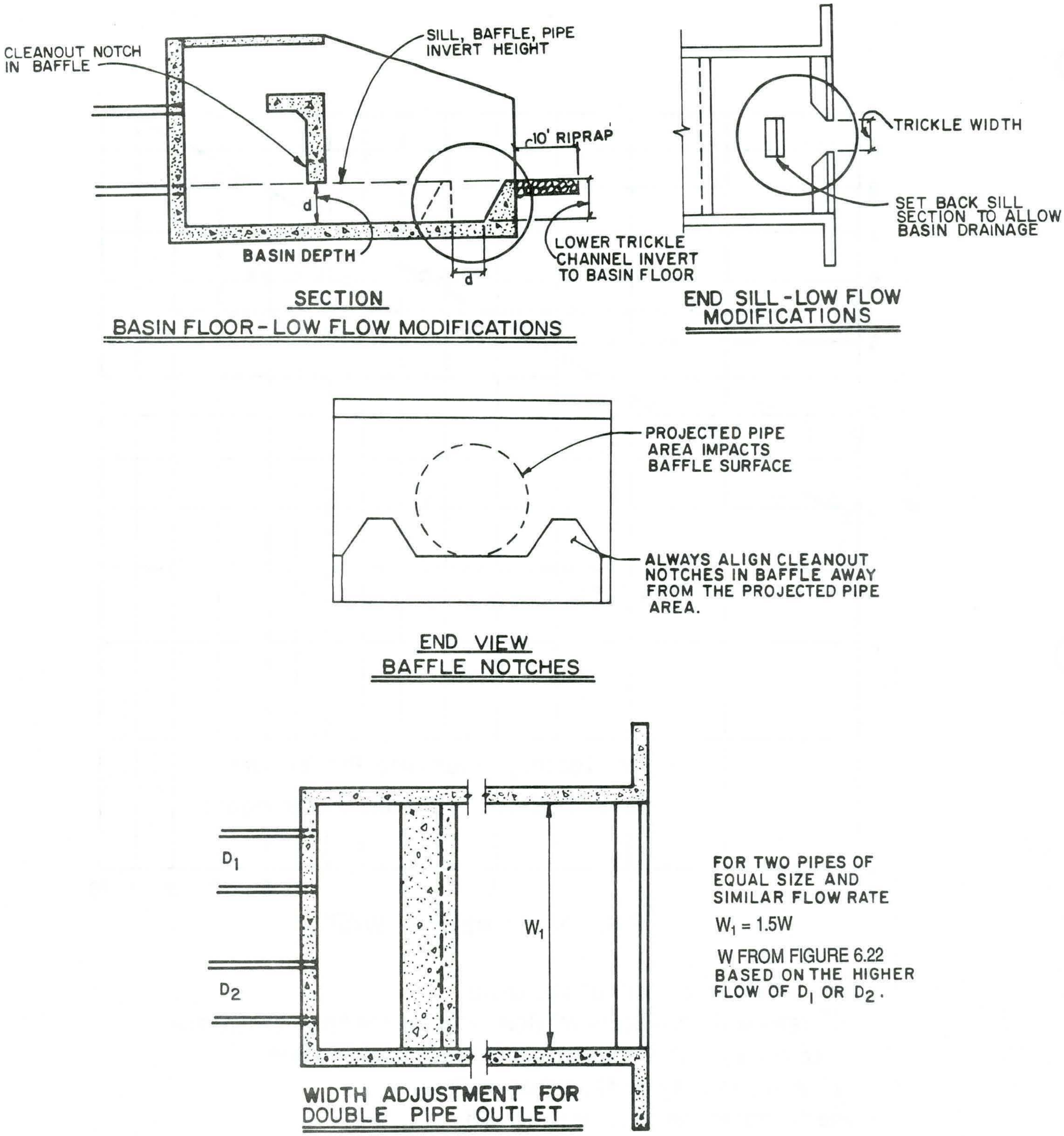


Figure 6.24
Modifications to Impact Stilling Basin
(To Allow Basin Drainage for Urban Applications)
(McLaughlin Water Engineers, Ltd. 1989)

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 6.22) 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 6.24.

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 6.24.

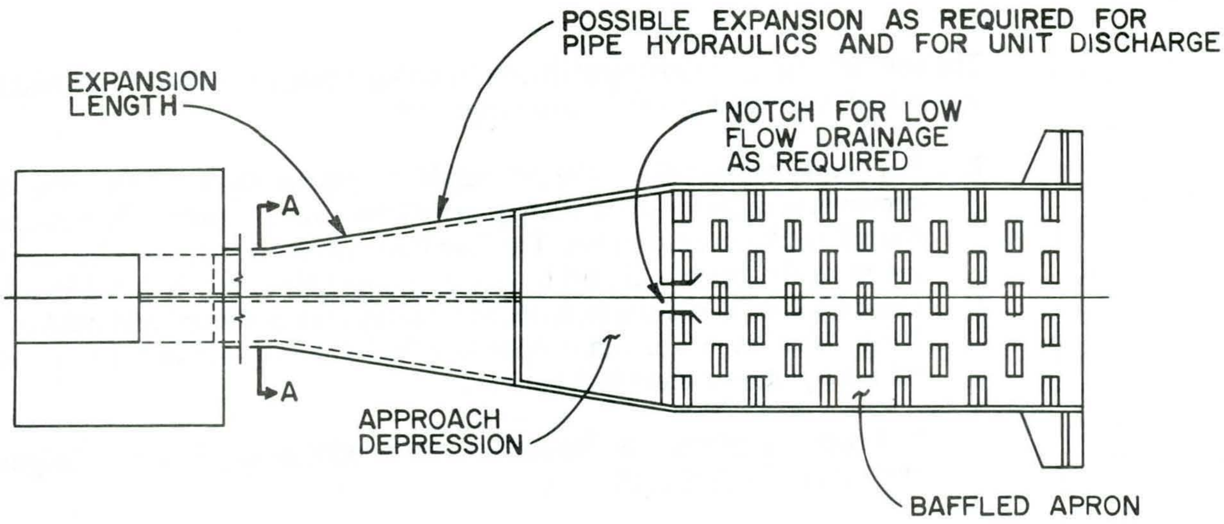
The total width of a combined dual inlet basin can be reduced to three-fourths of the total width for separate basins. For example, if the design width for each pipe is w, the combined basin width would be 1.5 w.

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.

6.4.3.2 Baffle Chute Energy Dissipator: The baffle chute developed by Peterka (1958) 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 6.25 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 6.3.4.1. The remaining



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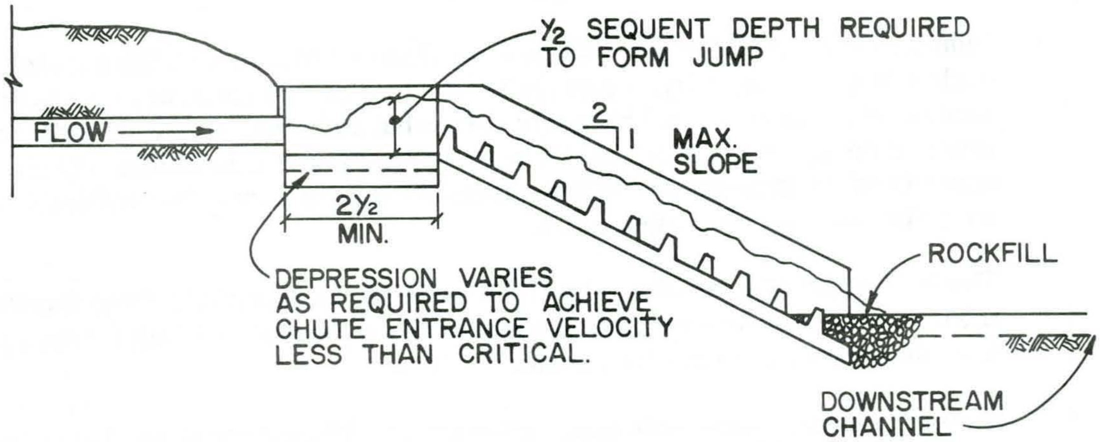
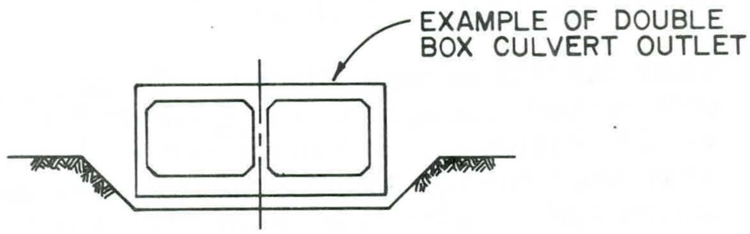


PROFILE



SECTION A-A

Figure 6.25
Baffle Chute at Conduit Outlet
(Adapted from: Peterka, et al 1958)

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.

6.5 Special Channel Structures

6.5.1 Channel Transitions

A flow transition is a change of open channel flow cross section designated to be accomplished in a short distance with a minimum amount of flow disturbance. Types of transitions are illustrated in Figure 6.26. Of these, the abrupt (headwall) and the straight line (wingwall) are the most common.

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 4.

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.

Outlet transitions (expansions) 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* (Corry, et al 1975)

6.5.2 Supercritical Flow Structures

6.5.2.1 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 detention reservoirs for a control structure and/or a spillway.

Acceleration chutes are potentially hazardous if inadequately planned and designed (see USBR 1974; Peterka 1958; 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 above address acceleration chutes in greater detail than can be discussed in this manual. Refer to these publications for a detailed analysis.

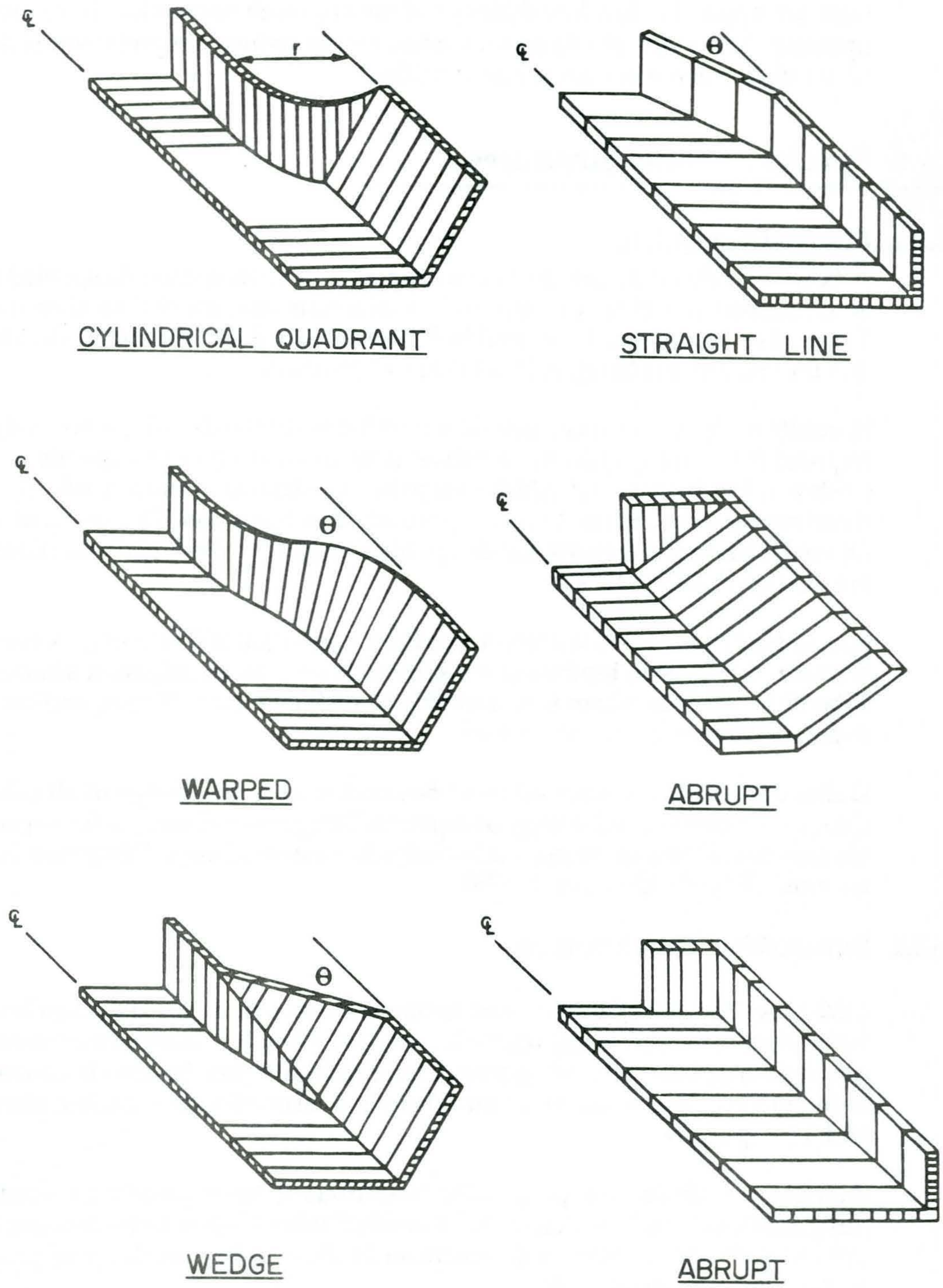


Figure 6.26
Channel Transition Types
(Adapted from: Corry, et al 1975)

Chutes have four component parts:

» Inlet

» Vertical Curve Section

» Concrete, Steeply Sloped Channel

» 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 detention 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.

6.5.2.2 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* (Corps of Engineers 1970b).

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 bind.

$$F = M\Delta v \qquad \textbf{(6.15)}$$

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.

$$\Delta P = \frac{P}{2}(\Delta(v^2)) \tag{6.16}$$

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.

6.6 Safety

Hydraulic structures constructed in Maricopa County will usually be subject to public access. Designs for hydraulic structures must address the issue of safety. First, signage must be provided to identify the potential hazard of flooding or dangerous flow measures to the public. Second, appropriate measures must be designed to keep the public away from hazardous locations. For example, vertical drop structures should not exceed 3 feet in height, and adequate fencing or railings must be provided along all other walls, such as wing walls or training walls. Additional considerations for safety are discussed in Chapter 4.

6.7 Operation, Maintenance and Aesthetic Considerations

6.7.1 Operation and Maintenance

Hydraulic structures should be designed so they can be maintained. As with other drainage facilities, maintenance operations will consist of scheduled and unscheduled operations. Scheduled operations include mowing, debris removal, graffiti removal, and rock replacement. Unscheduled operations are those which follow a storm event and include debris removal, rock replacement, erosion repair, fence or railing repair and other activities for which the frequency and scope cannot be predicted. Some maintenance considerations appropriate for hydraulic structures are presented below. Access to key areas (i.e. crest area, stilling basin area) for maintenance equipment and personnel is the primary consideration common to all structure types.

» A 4:1 slope is recommended as a minimum for mowing equipment on landscaped or grass bank and transition slopes. The local jurisdictional agency should be consulted regarding special circumstances for specific site constraints where a steeper slope may be necessary.

» Transition areas upstream and downstream of the structures should be designed to drain completely. This applies particularly to stilling basins.

» Selection and placement of rock for a stilling basin or upstream of a drop crest should consider a size range not easily displaced by flow as well as one not easily moved by vandalism. Grouted boulders are a suitable alternative.

» Open channels are recommended in lieu of pipes for conveyance of low flows through the drop structure area. Pipes should be no smaller than 24 inches in diameter. Low flow channels are preferred over pipes which may plug or frequently overtop, leading to additional maintenance problems.

» Riprap should be provided at likely scour areas that are relatively expensive to access and repair later.

6.7.2 Structure Aesthetics

6.7.2.1 General: Aesthetics, safety, recreation, and overall integration with nearby land uses are important aspects in the design of hydraulic structures. The design and planning, construction, and maintenance of hydraulic structures and natural drainageways in an urban setting all offer opportunities for promoting aesthetic design and habitat features. Maximizing functional uses while improving visual quality requires good planning from the onset of the project, and the coordinated efforts of the owner/client, engineer, landscape architect, and planner.

The significance of providing an aesthetic and visually appealing project depends on the number, type, and frequency of viewer; the viewing angle; project location; and the overall environment of the project area. Aesthetic considerations are site and project specific.

The combination and diversity of forms, lines, colors, and textures create the visual experience. Material selection and landscape design can provide visual character and create interesting spaces in and around hydraulic structures.

6.7.2.2 Open Spaces and Parks: Creative planning concepts in urban and urbanizing areas, particularly in residential areas, emphasize multiple uses of flood control, recreation, and open spaces. Cluster housing and good subdivision planning may be coordinated to offer opportunities to maintain the natural habitat characteristics of the drainageway while fulfilling open space and recreation requirements.

Multiple use of flood control structures and open space parks has proven to be an effective and aesthetic land use combination. Athletic fields and detention areas which remain dry most of the time have been used in many communities. The design of overflow structures and crest controls can be combined with concrete pathways to blend with a park lined environment.

6.7.2.3 Materials: A variety of materials and finishes are available for use in hydraulic structures. Concrete color additions, exposed aggregates and form liners can be used to create visual interest to otherwise stark walls. The location of expansion and control joints in combination with reveals can be used to create effective design detailing of headwalls and abutments. Rock and vegetation can be used for bank stability and erosion protection around structures to provide visual contrast and diversity, and spatial character.

6.8 Design Aids

6.8.1 Hydraulic Jump

With the exception of the baffle chute drop, all of the drop structures described herein 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 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). The various types of hydraulic jumps that can occur are listed in Table 6.4.

6.8.1.1 Jump Height: The depth of flow immediately downstream of a hydraulic jump is referred to as the sequent depth. The sequent depth in rectangular channels can be computed by use of the following equation:

$$Y_{2D} = \tfrac{1}{2} Y_{1u} \left[(1 + 8F_1{}^2)^{1/2} - 1 \right] \tag{6.17}$$

The solution for sequent depth in trapezoidal channels can be obtained from a trial-and-error solution of Equation 6.18, which is derived from momentum equations. It is also acceptable, for design purposes, to determine the sequent depth in

Table 6.4
Types of Hydraulic Jumps

Upstream Froude Number	Type of Jump	Energy Loss, %
1 < F ≤ 1.7	Undular Jump	0.5
1.7 < F ≤ 2.5	Weak Jump	5 to 18
2.5 < F ≤ 4.5	Oscillating Jump	18 to 44
4.5 < F ≤ 9	Steady Jump	44 to 70
9 < F	Strong Jump	70 to 85

trapezoidal channels from Equation 6.17. Equation 6.19 is much simpler to solve and produces only slightly greater values for sequent depth than does Equation 6.18.

$$\frac{ZY_{1u}^{\ 3}}{3} + \frac{bY_{1u}^{\ 2}}{2} + \frac{Q}{gA_1} = \frac{ZY_{2D}^{\ 3}}{3} + \frac{bY_{2D}^{\ 2}}{2} + \frac{Q}{gA_2} \tag{6.18}$$

Figures 6.27 and 6.28 provide graphs of hydraulic jumps for a horizontal rectangular channel and a horizontal trapezoidal channel, respectively.

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 Figures 6.29 and 6.30.

6.8.1.2 Surface Profile: The surface profile of a hydraulic jump may be needed to design the profile of extra bank protection, or training walls for containment of the jump. The surface profile can be determined from Figure 6.31.

6.8.1.3 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 6.19. 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 6.1 (Section 6.3.2.3) 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 force of the supercritical approach flow.

6.8.1.4 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 6.17. Therefore, the height of this wave should be determined as follows:

$$(Y_{2D} - Y_{1u}) / Y_{1D} = F_1^{\ 2} - 1 \tag{6.19}$$

where all terms are as previously described.

6.8.2 Design Charts and Figures

Table 6.5 (page 333) has been included as an additional aid to the user of this manual.

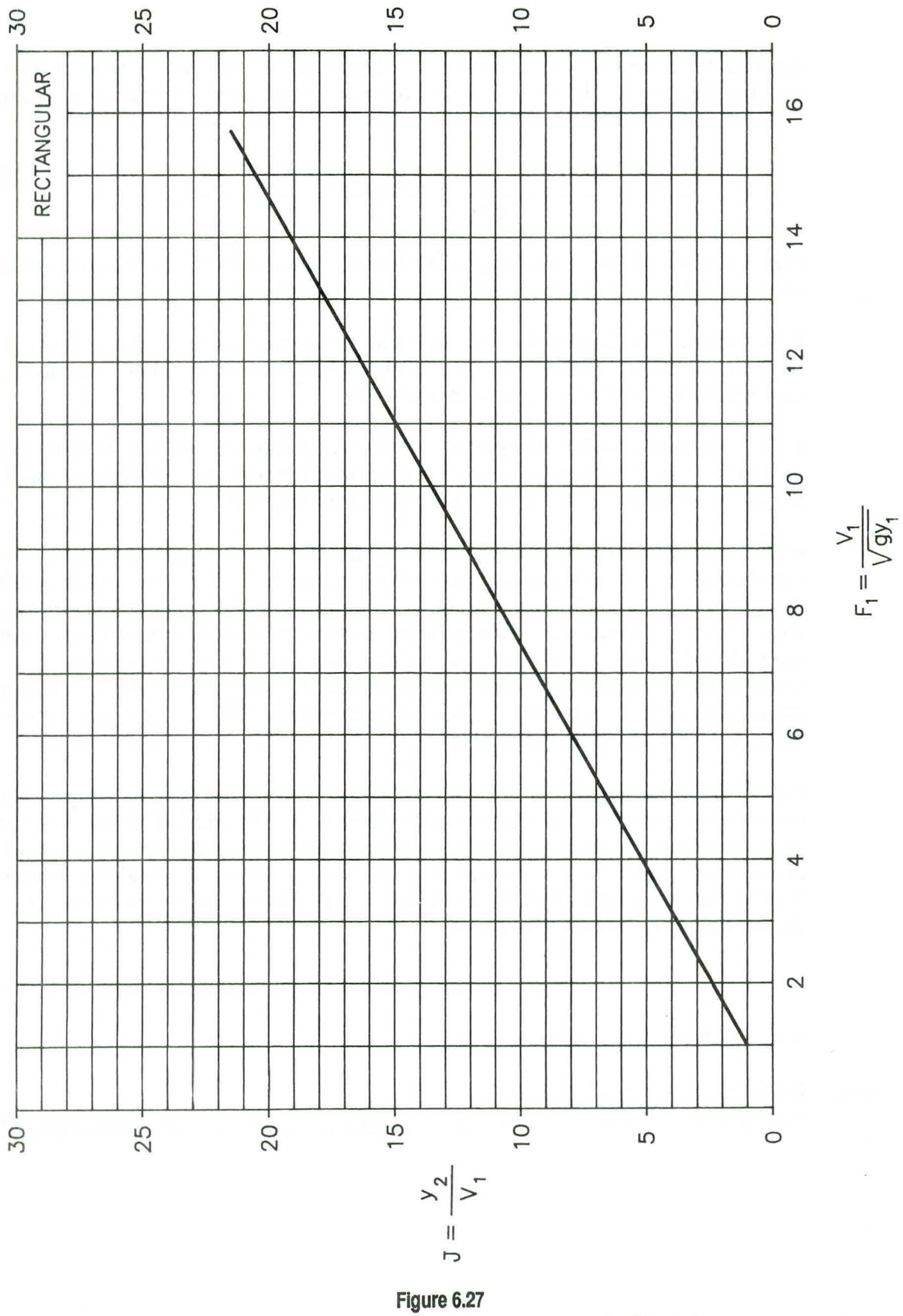


Figure 6.27

Height of a Hydraulic Jump for a Horizontal Rectangular Channel

(Simons, Li and Associates, Inc. 1988)

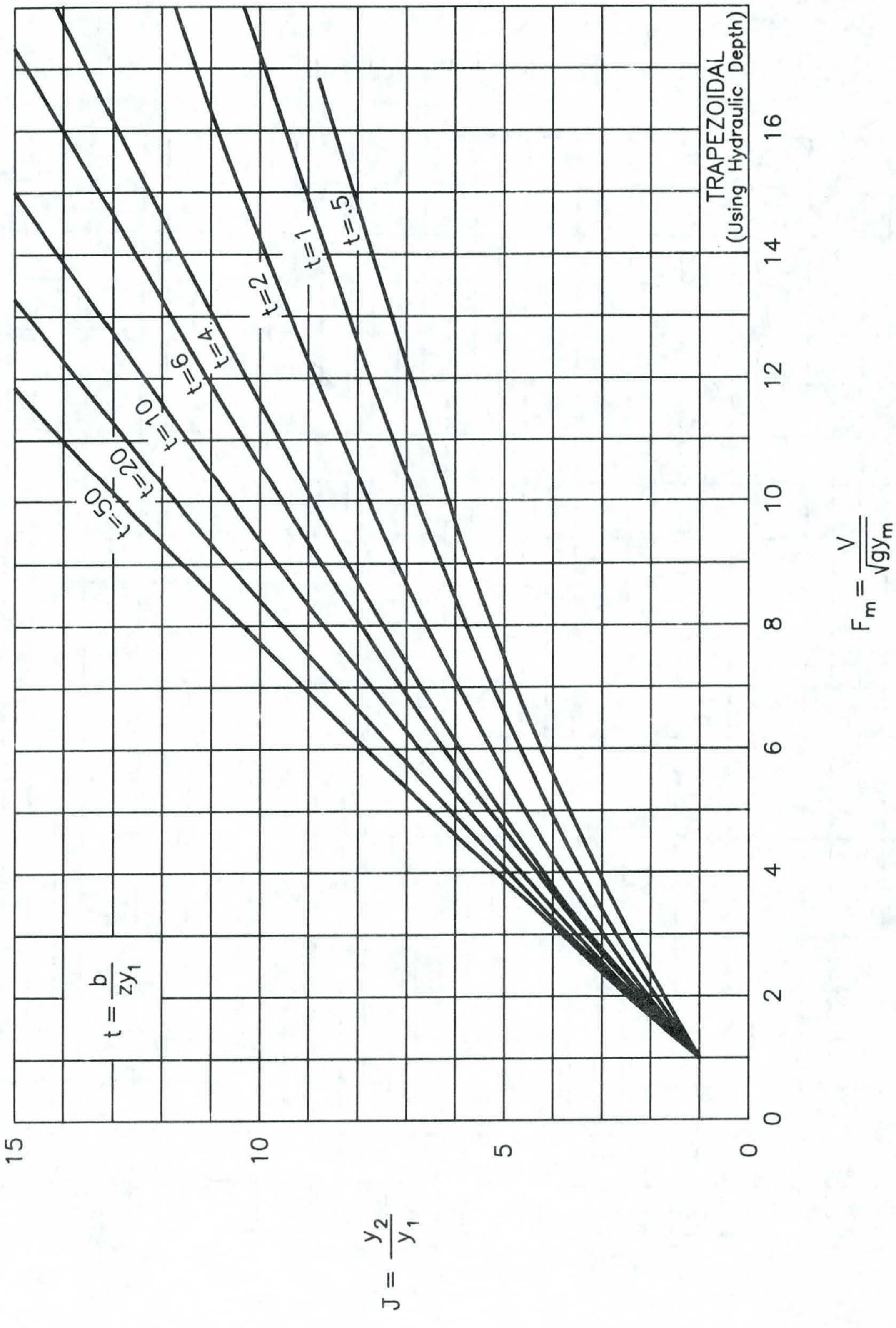


Figure 6.28
Height of a Hydraulic Jump for a Horizontal Trapezoidal Channel
(Using Hydraulic Depth)
(Simons, Li and Associates, Inc. 1988)

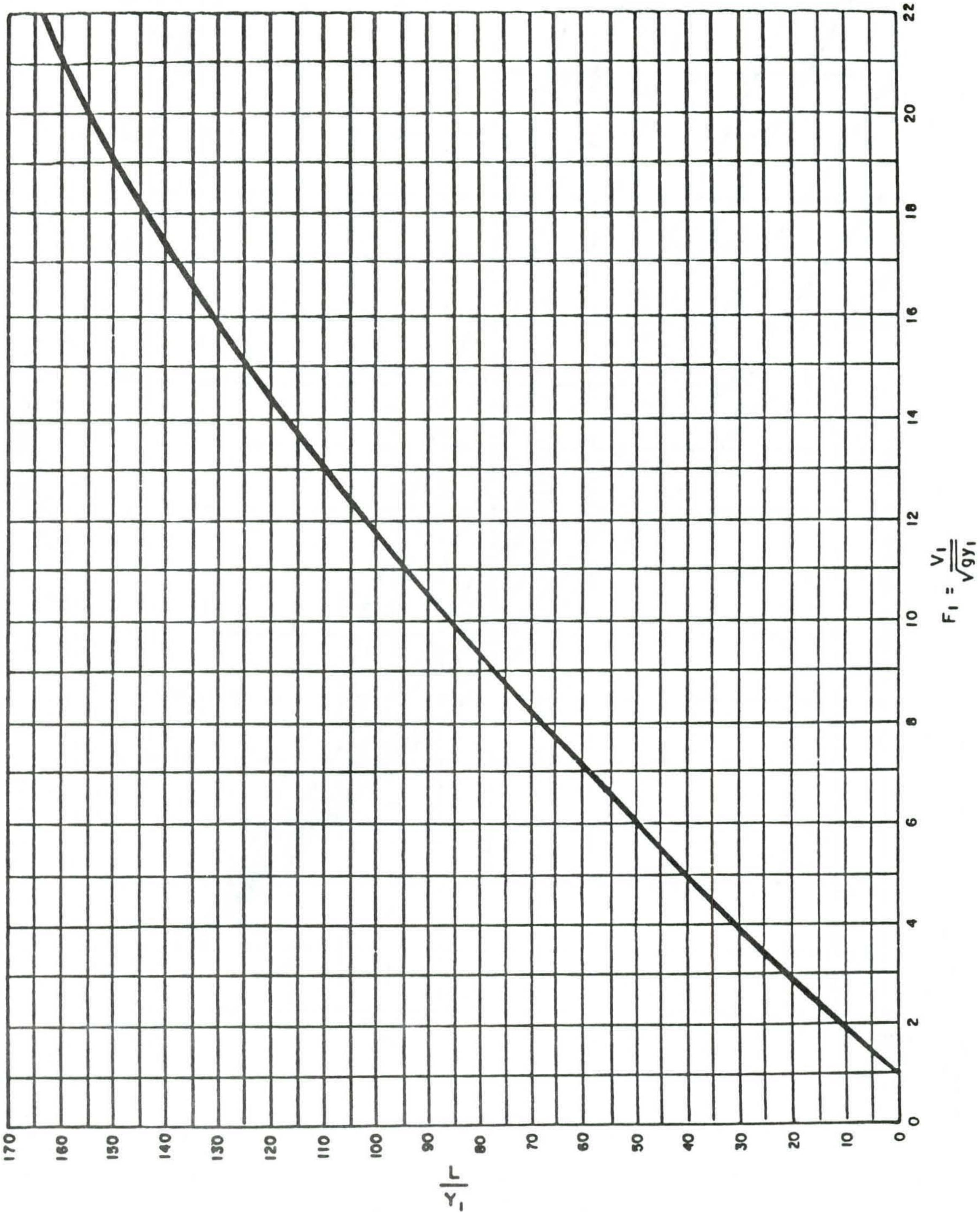


Figure 6.29
Length of a Hydraulic Jump for Rectangular Channels
(Simons, Li and Associates, Inc. 1988)

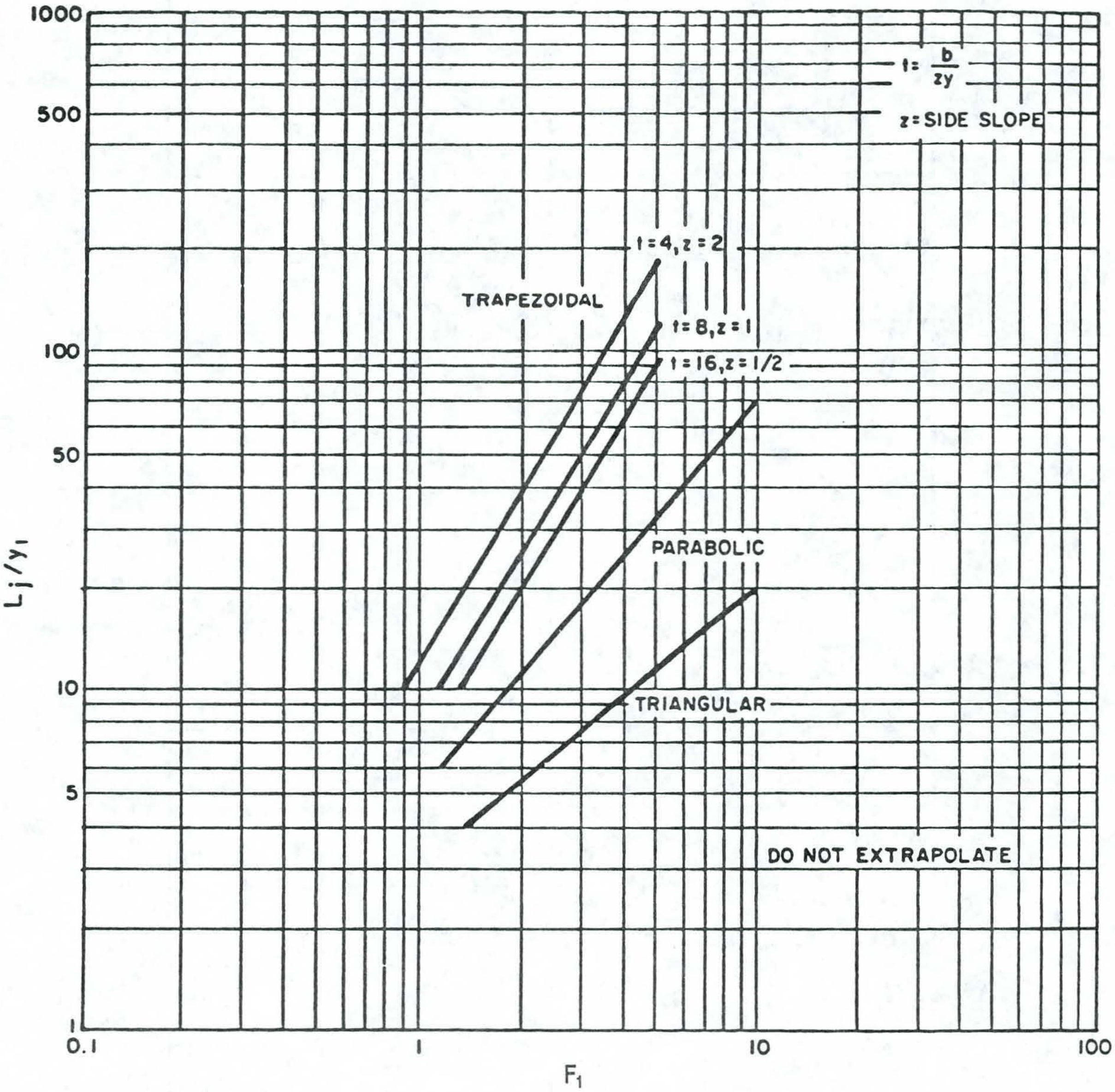


Figure 6.30
Length of a Hydraulic Jump for Non-Rectangular Channels
(Simons, Li and Associates, Inc. 1988)

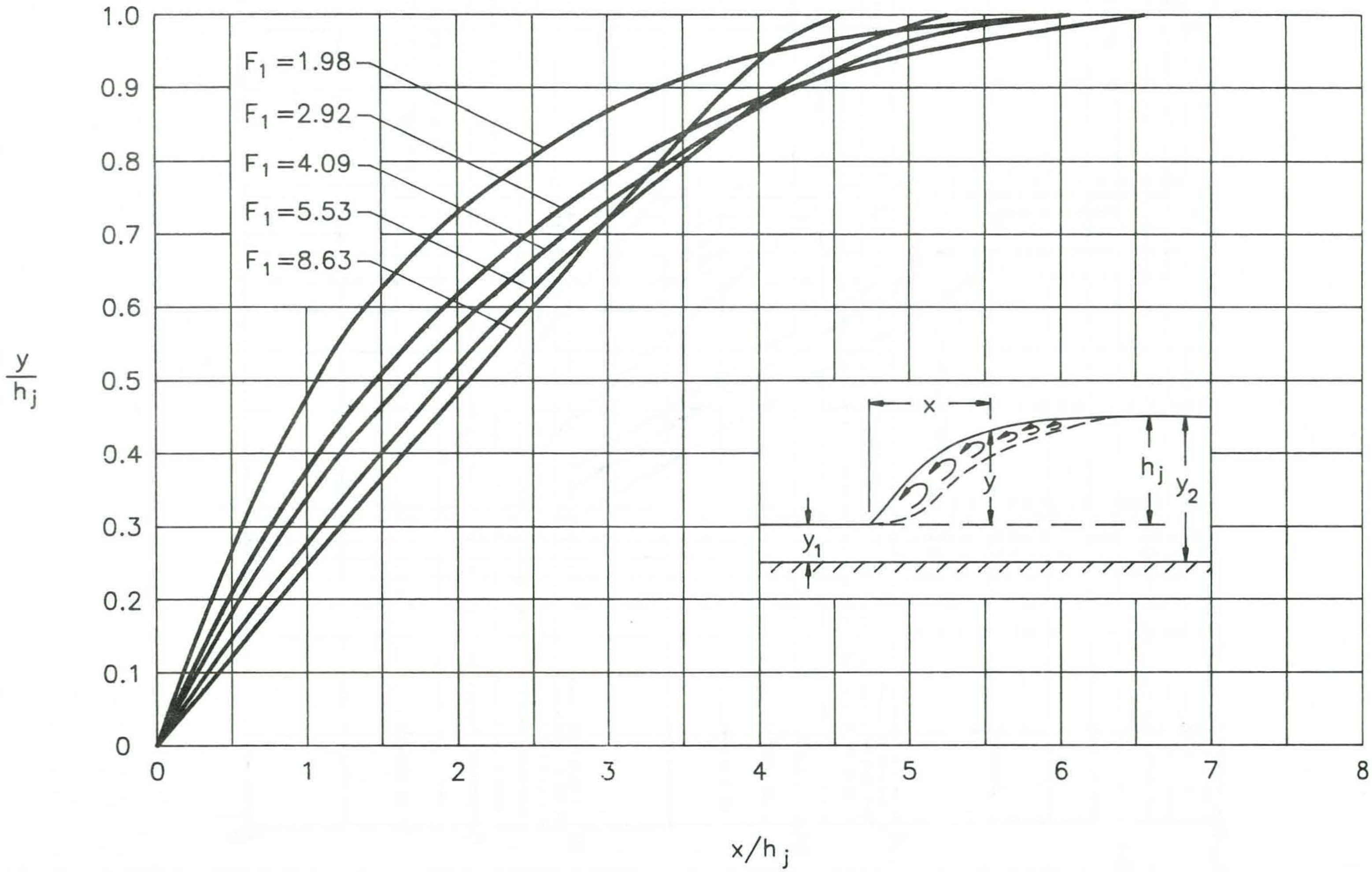


Figure 6.31
Surface Profile of a Hydraulic Jump
(Simons, Li and Associates, Inc. 1988)

Table 6.5
Uniform Flow In Circular Sections Flowing Partly Full
(Corry, et al 1975)

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

6.9 Design Examples

6.9.1 Open Channel Drop Structures

Design Example for Vertical Drop with Riprap Basin (Stevens 1981)

A rock riprap vertical drop structure is to be used to drop the flow in the drainage channel a vertical distance of 4 ft. See Figure 6.12.

Approach Conditions: The approach channel is trapezoidal in cross-section with:

- » bed width is $B_o = 10$ ft
- » side slopes are 2 horizontal to 1 vertical
- » design discharge is $Q = 360$ cfs
- » the depth of flow is $y_o = 4.00$ ft
- » the average velocity is $V_o = 5.00$ fps
- » the specific energy in the approach channel is

$$E_o = y_o + V_o^{\,2}/2g = 4.00 + 0.39 = 4.39 \text{ ft}$$

Weir Crest: Make the length of the weir $B = 10$ ft.

The head on the weir required to pass the design flow is as follows (see Table 8.2):

$$H = [Q/(5.67B)]^{0.67}$$

$$H = [360/(5.67 \times 10)]^{0.67} = 3.43 \text{ ft}$$

The height of the weir crest is:

$$P = E_o - H$$

$$P = 4.39 - 3.43 = 0.96 \text{ ft}$$

If drawdown can be tolerated in the approach channel, the weir crest should be placed at the bed level of the approach channel; that is $P = 0$. If not, make $P = 1.0$ ft. Here use $P = 1.0$ ft.

The height of the wingwalls is:

$$h = H = 3.5 \text{ ft}$$

Basin: The problem is to find the drop height D and rock size d_m such that z is 4.0 ft. The depth of flow leaving the basin is:

$$y_2 = y_o = 4.0 \text{ ft}$$

Start by using the largest rock size that has been tested in the model. That is:

$$D / d_{50} = 6.0$$

First, try D = 6.0 ft, so that d_{50} = 1.0 ft, and H/D = 0.57.

Now a trial and error solution is required to find the depth d_2 that satisfies both the curves in Figure 6.13 (page 299) and the equation:

$$d_2 = Y_2 + 0.67 d_s$$

Here d_s is the depth of scour, in feet. The calculations are summarized in the Table 6.6.

The procedure is to select a trial value of d_2, compute d_2/D, obtain d_s/D from Figure 6.13, and then compute d_2 from the above equation. When the trial value and the computed value of d_2 agree, that is one solution. For the first, try:

D	=	6.0 ft
d_2	=	5.5 ft and
z	=	6.0 - 1.0 - 1.5 = 3.5 ft, which is less than the required drop (4 ft).

Therefore, a larger D is needed. In this case, a value of 8 ft is assumed while d_s = 1.3 ft to maintain the D/d_s ratio of 6. This results in d_2 = 5.5 ft and z = 5.5 ft, which is too large.

These two trial results for D and z are graphically plotted in Figure 6.32 so that a new (and hopefully) final value for D can be selected to yield the desired drop, z = 4.0 ft. The interpretation is that D = 6.5 ft which, as shown in Table 6.6, turns out to be the required result. The design values are:

D	=	6.5 ft
P	=	1.0 ft
d_{50}	=	1.1 ft
d_s	=	2.2 ft

From Figure 6.12, the length of the basin is calculated as follows:

$$L_b = 4H + 0.25D$$

$$L_b = 4 \times 3.43 + 0.25 \times 6.5$$

$$= 15.5 \text{ ft}$$

The minimum depth of rock in the plunge pool is 1.5 d_s = 3.3 ft (Figure 6.12), and the minimum thickness of the riprap on the side slopes is t = 2.0 d_{50} = 2.20 ft (Figure 6.17).

Table 6.6
Calculations of Basin Dimensions

D, ft	d_{50}, ft	H/D	Trial d_2, ft	d_2/D	d_s/D	d_s, ft	0.67 d_s, ft	Resulting d_2, ft
6.0	1.0	0.57	5.0	0.83	0.46	2.76	1.85	5.85
			6.0	1.00	0.30	1.80	1.21	5.21
			5.5	0.92	0.38	2.28	1.53	5.53 ok
$z = 6.0 - 1.0 - 1.5 = 3.5$ ft $\neq 4.0$ ft								
8.0	1.3	0.43	7.0	0.88	0.18	1.44	0.96	4.96
			6.0	0.75	0.25	2.00	1.33	5.33
			5.5	0.69	0.28	2.24	1.49	5.49 ok
$z = 8.0 - 1.0 - 1.5 = 5.5$ ft $\neq 4.0$ ft								
6.5	1.1	0.53	6.0	0.92	0.28	1.82	1.21	5.21
			5.5	0.85	0.33	2.15	1.44	5.44 ok
$z = 6.5 - 1.0 - 1.4 = 4.1 \approx 4.0$ft								

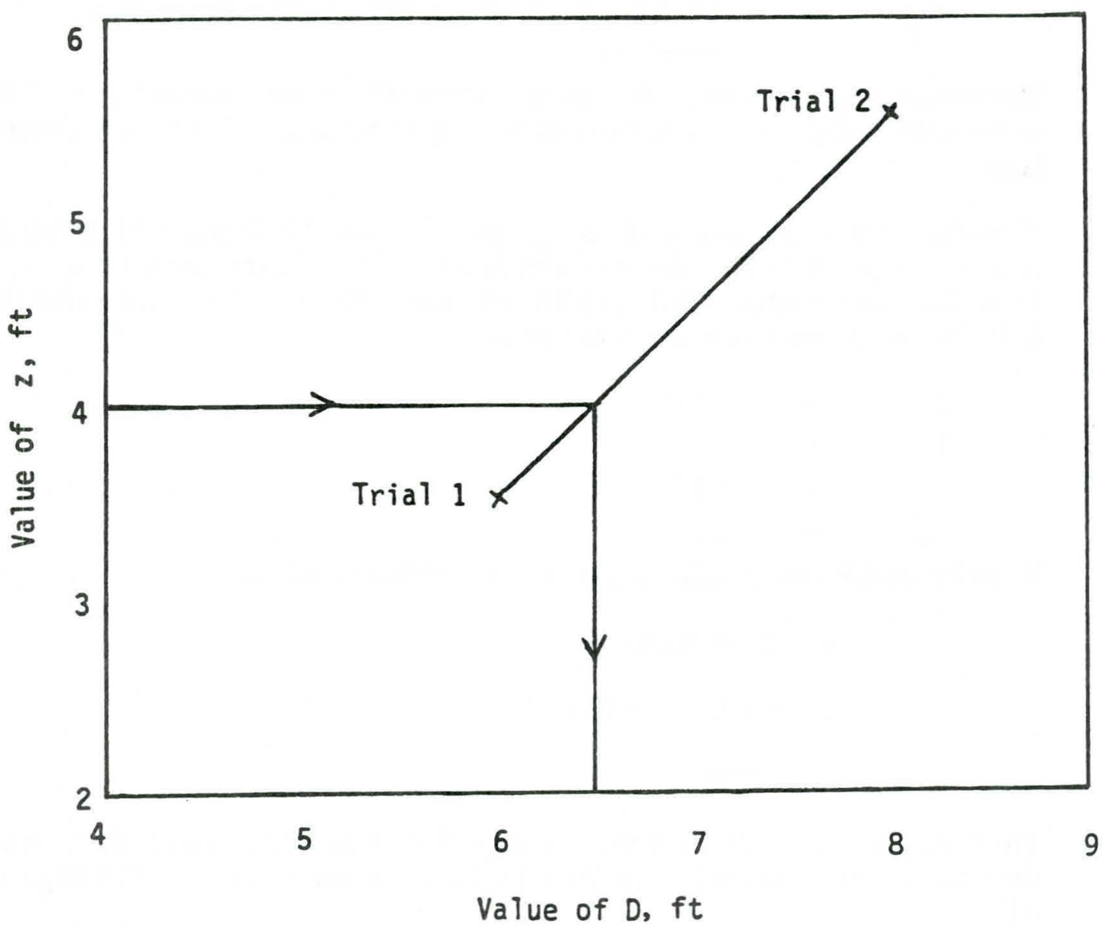


Figure 6.32
Solution for Vertical Drop with Riprap Basin Design Example

A gravel filter is necessary between the soil on the bottom of the plunge pool and on the cut slopes and the riprap. Usually 6 inches of well-graded pit-run gravel (minus 3 inches) is all that is required for a filter if the riprap is well graded.

6.9.2 Conduit Outlet Protection
Design Examples for Riprap Basins (Corry, et al 1975)

6.9.2.1 Design Example 1

Given: 8 ft by 6 ft box culvert
$Q = 800$ cfs
supercritical flow in culvert
normal flow depth = brink depth
$y_o = 4$ ft
Tailwater depth (TW) = 2.8 ft

Find: Riprap basin dimensions for these conditions.

Solution:

1. $y_o = y_e$ for rectangular section, $y_e = 4$ ft
2. $V_o = Q/A = 800/(4)(8) = 25$ fps
3. $F = V_o/[(32.2)(y_e)]^{0.5} = 25/[(32.2)(4)]^{0.5} = 2.20$
4. $TW/y_e = 2.8/4.0 = 0.7$; $TW/y_e < 0.75$ OK
5. Try $d_{50}/y_e = 0.45$, $d_{50} = (0.45)(4) = 1.80$ ft

   From Figure 6.20, $h_s/y_e = 1.6$

   $h_s = (4)(1.6) = 6.4$ ft
   $h_s/d_{50} = 6.4/1.8 = 3.6$ ft $\quad 2 < h_s/d_{50} < 4$ OK
6. From Figure 6.17:

   $L_s = (10)(6.4) = 64$ ft
   $L_s \text{ min} = (3)(W_o) = (3)(8) = 24$ ft, use $L_s = 64$ ft
   $L_B = (15)(6.4) = 96$ ft
   $L_B \text{ min} = (4)(W_o) = (4)(8) = 32$ ft, use $L_B = 96$ ft

Other basin dimensions designed in accordance with details are shown in Figure 6.17 (page 309).

6.9.2.2 Design Example 2

Given: 8 ft by 6 ft box culvert
$Q = 800$ cfs
supercritical flow in culvert
normal flow depth = brink depth
$y_o = 4$ ft
Tailwater depth (TW) = 4.2 ft
Downstream channel can tolerate 7 fps for design discharge

Find: Riprap basin dimensions for these conditions.

Solution:

Note: High tailwater depth, $TW/y_o = 1.05 > 0.75$.

1. Design riprap basin using steps 1 through 6 of Design Example 1.

   $d_{50} = 1.8$ ft; $h_s = 6.4$ ft; $L_s = 64$ ft; $L_B = 96$ ft

2. Design riprap for downstream channel. Use Figure 6.21 to estimate the average velocity along the channel. Compute the equivalent circular diameter, D_e, for the brink area, A, from:

   $$A = 3.14\,D_e^2/4 = 2y_o^2$$

   $$W_o = (4)(8) = 32\text{ ft}^2$$

   $$D_e = (32(4)/3.14)^{0.5} = 6.4\text{ ft}$$

   $$V_o = 25\text{ fps (Design Example 1)}$$

L/D_e	L (compute)	V_L/V_o (Figure 6.21)	V_L
10	64	0.59	14.7
15	96	0.36	9.0
20	128	0.30	7.5
21	135	0.28	7.0

The channel should be lined with the same size rock used for the basin. Protection must extend at least 135 feet downstream from the culvert brink.

6.9.2.3 Design Example 3

Given: 6 feet diameter cmp
$Q = 135$ cfs
$S_o = 0.004$
Mannings $n = 0.024$
Normal depth in pipe for $Q = 135$ cfs is 4.5 feet
Normal velocity is 5.9 fps
Flow is subcritical
Tailwater depth (TW) is 2.0 ft

Find: Riprap basin dimensions for these conditions.

Solution:

1. Determine y_o and V_o:

   $Q/D^{2.5} = 135/(6)^{2.5} = 1.53$

   $TW/D = 2.0/6 = 0.33$

   From Figure 6.19, $y_o/D = 0.45$

   $y_o = (0.45)(6) = 2.7$ ft

   $TW/y_o = 2.0/2.70 = 0.74, \quad TW/y_o < 0.75$ OK

   Brink Area (A) for $y_o/D = 0.45$ is:

   $A = (0.343)(36) = 12.3\ ft^2$ [0.343 is from Table 6.5]

   $V_o = Q/A = 135/12.3 = 11.0$ fps

2. $y_e = (A/2)^{0.5} = (12.3/2)^{0.5} = 2.48$ ft

3. $F = V_o/[(32.2)(y_e)]^{0.5} = 11/[(32.2)(2.48)]^{0.5} = 1.23$

4. Try $d_{50}/y_e = 0.25, \quad d_{50} = (0.25)(2.48) = 0.62$ ft

   From Figure 6.20, $h_s/y_e = 0.75, \quad h_s = (0.75)(2.48) = 1.86$ ft

   check: $h_s/d_{50} = 1.86/0.62 = 3, \quad 2 < h_s/d_{50} < 4$ OK

5. $L_s = (10)(h_s) = (10)(1.86) = 18.6$ ft
   or
   $L_s = (3)(W_o) = (3)(6) = 18$ ft, Use $L_s = 18.6$ ft

   $L_B = (15)(h_s) = (15)(1.86) = 27.9$ ft
   or
   $L_B = (4)(W_o) = (4)(6) = 24$ ft, Use $L_B = 27.9$ ft

   $d_{50} = 0.62$ ft, Use $d_{50} = 8$ inches

Other basin dimensions designed in accordance with details are shown on Figure 6.17 (page 309).

6.10 References

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Notes

7

Bridges

7.1 Introduction

This chapter presents an 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. 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.

7.2 Definition of Symbols

The following symbols are used in equations throughout Chapter 7:

α_1 = Kinetic energy coefficient = $\Sigma(qv^2)/QV_1^2$

α_2 = Kinetic energy coefficient to correct for non-uniform velocity distribution under the bridge = $\Sigma(qv^2)/QV^2_{n2}$

A_1 = Total water area at Section 1, including that produced by the backwater, ft^2

A_4 = Water area at Section 4, where normal stage is reestablished, ft^2

A_{n2} = Gross water area in constriction measured below normal stage, ft^2

A_p = Water area occupied by piers, ft^2

g	=	Acceleration due to gravity, 32.2 ft/sec^2
h_1*	=	Total backwater depth, ft
$K*$	=	Total backwater coefficient
K_b	=	Base curve coefficient
ΔK	=	Incremental backwater coefficient
ΔK_p	=	Incremental backwater coefficient for piers
M	=	Bridge opening ratio
σ	=	Correction factor, for $M < 1$
q	=	Discharge in the same subsection, cfs
Q	=	Total discharge in river, cfs
v	=	Average velocity in a subsection, fps
V_1	=	Average velocity in river at Section 1 = Q/A_1, fps
V_{n2}	=	Average velocity in constriction = Q/A_{n2}, fps

7.3 Design Approach

The method of planning for bridge openings must include water surface profile analyses of the channel for the major storm runoff and other design frequencies as may be appropriate. Once the water surface profiles have been calculated, the maximum reasonable effect on the channel flow by the bridge should be determined.

7.3.1 Hydraulic Analysis

The hydraulic analyses of pre- and post-bridge conditions can be performed using a computerized step-backwater model. The HEC-2 program developed by the U.S. Army Corps of Engineers (COE, Water Surface Profiles, Users Manual, 1990) is the most common backwater computation software and is used nationwide. Cross sections for the model must be taken 1) at a sufficient distance downstream of the bridge so that the bridge has no effect upon flow characteristics; 2) immediately downstream of the bridge opening; 3) immediately upstream of the bridge opening; and 4) a sufficient distance upstream of the bridge opening to evaluate the structure's effect upon flow characteristics.

Choose coefficients carefully for calculation of losses for expansion, contraction and intermediate piers that are appropriate for the bridge structure under consideration. The HEC-2 bridge routines require meticulous input preparation for proper computer analysis. Care should be taken to review input data and to examine results thoroughly for reasonableness.

Another methodology for hydraulic analysis of bridge crossings is that described in *Hydraulics of Bridge Waterways* (FHWA 1978b) and paraphrased below. Figure 7.1 illustrates a bridge crossing normal to a stream with wingwall abutments and embankment approaches that encroach into the floodplain. For detailed information, refer to the FHWA document cited above.

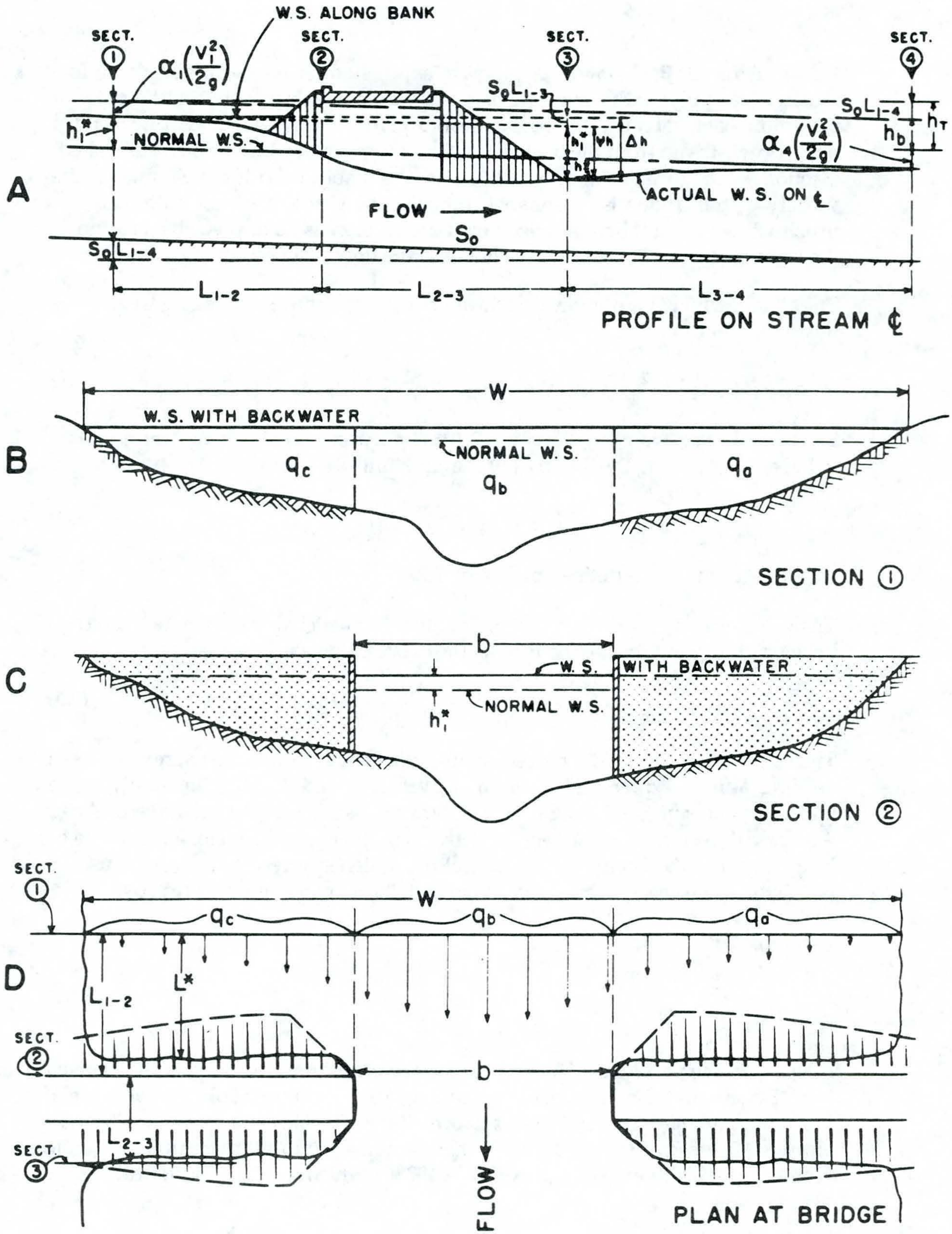


Figure 7.1
Normal Bridge Crossing Designation
(FHWA 1978b)

7.3.1.1 Effect of Backwater: A practical expression for backwater has been formulated by applying the principle of conservation of energy between the point of maximum backwater upstream from the bridge, (see Figure 7.1A, Section 1); and, a point downstream from the bridge at which normal stage has been reestablished, (Section 4 of Figure 7.1A). The equation is reasonably valid if the channel in the vicinity of the bridge is reasonably uniform, the gradient of the bottom is approximately constant between Sections 1 and 4, there is no appreciable erosion of the bed in the constriction due to scour, and the flow is subcritical.

The equation for computation of backwater upstream from a bridge constricting the flow is as follows:

$$h_1^* = K^*\alpha_2 [(V^2_{n2})/2g] + \alpha_1[(A_{n2}/A_4)^2 - (A_{n2}/A_1)^2](V^2_{n2})/2g \qquad \textbf{(7.1)}$$

To compute backwater by Equation 7.1 it is necessary to obtain the approximate value of h_1^* by using the first part of the equation:

$$h_1^* = K^*\alpha_2 (V^2_{n2}/2g) \qquad \textbf{(7.2)}$$

where all terms are as defined in Section 7.2.

The value of A_1 in the second part of Equation 7.1, which depends on h_1^*, can then be determined and the second term of the expression evaluated:

$$\alpha_1\left[(A_{n2}/A_4)^2 - (A_{n2}/A_1)^2\right](V^2_{n2}/2g) \qquad \textbf{(7.3)}$$

This part of the equation represents the difference in kinetic energy between Sections 4 and 1, expressed in terms of the velocity head $V^2_{n2}/2g$. Equation 7.1 may appear cumbersome, but it was set up as shown to permit omission of the second part when the difference in kinetic energy between Sections 4 and 1 is small enough to be insignificant in the final result. To permit the designer to readily recognize cases in which the kinetic energy term may be ignored, the following guides are provided:

$M > 0.7$,
$V_{n2} < 7$ fps, and
$K^*V^2_{n2}/2g < 0.5$ ft

If values in the problem at hand meet all three conditions, the backwater obtained from Equation 7.2 can be considered sufficiently accurate. Should one or more of the values not meet the conditions set forth, it is advisable to use Equation 7.1 in its entirety. The use of the guides is further demonstrated in the examples given in Hydraulics of Bridge Waterways (FHWA 1978b), which should be used in all bridge design work.

The value of the total backwater coefficient K^*, which has been determined experimentally, varies with:

1. Stream constriction as measured by bridge opening ratio M;

2. Type of bridge abutment—wingwall, spill through, etc.;

3. Number, size, shape, and orientation of piers in the constriction;

4. Eccentricity, or asymmetric position of bridge within the floodplains; and

5. Skew (bridge crosses floodplain at other than 90 degree angle).

The total backwater coefficient K^* consists of a base curve coefficient K_b, to which is added incremental coefficients to account for the effect of piers, eccentricity, and skew. The value of K^* is primarily dependent on the degree of constriction of the flow but also changes to a limited degree with the other factors.

7.3.1.2 Effect of M and Abutment Shape (Base Curves): Figure 7.2 shows the base curve for backwater coefficient, K_b, plotted with respect to the opening ratio, M, for several wingwall abutments and a vertical wall type. Note how the coefficient K_b increases with channel constriction. The several curves represent different angles of wingwall as can be identified by the accompanying sketches; the lower curves, represent the better hydraulic shapes.

Figure 7.3 shows the relation between the backwater coefficient K_b and M, for spill through abutments, for three embankment slopes. A comparison of the three curves indicates that the coefficient is little affected by embankment slope. Figure 7.3 and Figure 7.3 will be designated "base curves" and K_b will be referred to as the "base curve coefficient." The base curve coefficients apply to normal crossings for specific abutment shapes, but do not include the effect of piers, eccentricity, or skew.

7.3.1.3 Effect of Piers (Normal Crossings): The effect produced on the backwater by introduction of piers in a bridge constriction has been treated as an incremental backwater coefficient designated ΔK_p. ΔK_p is added to the base curve coefficient when piers are a factor. The value of ΔK_p is dependent on the ratio that the area of the piers bears to the gross area of the bridge opening, the type of piers (or piling in the case of pile bents), the value of the bridge opening ratio M, and the angularity of the piers with the direction of flood flow. The ratio of the water area occupied by piers (A_p) to the gross water area of the constriction (A_{n2})—both being based on the normal water surface—has been assigned the letter J:

$$J = A_p / A_{n2} \qquad \textbf{(7.4)}$$

In computing the gross water area, A_{n2}, the presence of piers in the constriction is ignored. The incremental backwater coefficient for the more common types of piers and pile bents can be obtained from Figure 7.4. Enter the proper value of J on Chart A of Figure 7.4 and read ΔK; obtain the correction factor σ from Chart B of Figure 7.4 for opening ratios other than unity. The incremental backwater coefficient is then:

$$K_p = \Delta K \sigma \qquad \textbf{(7.5)}$$

The incremental backwater coefficients for piers can, for all practical purposes, be considered independent of diameter, width, or spacing, but should be increased if

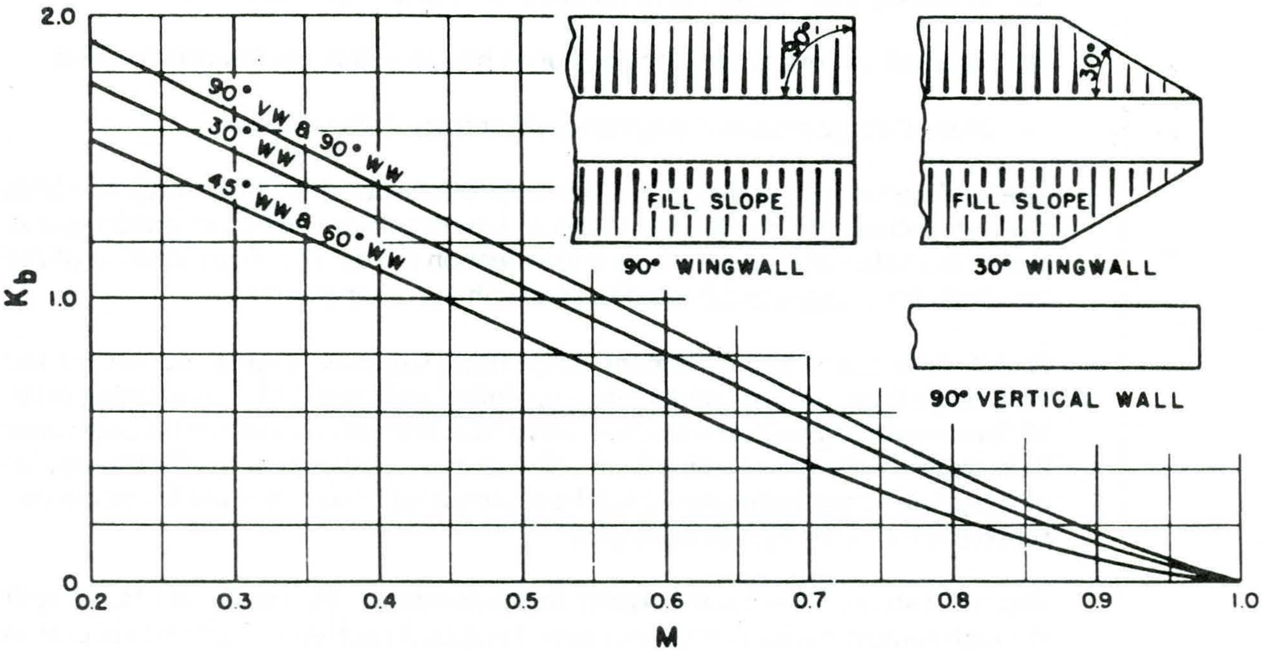


Figure 7.2
Base Curves for Wingwall Abutments

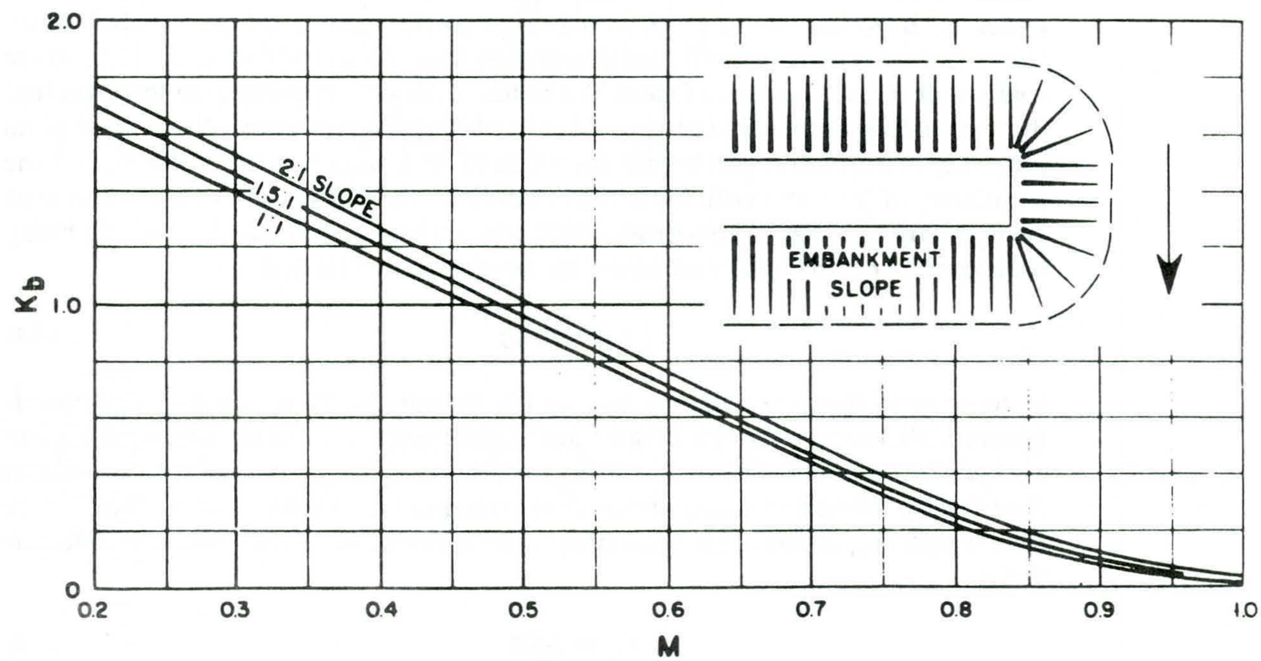


Figure 7.3
Base Curves for Spillthrough Abutments
(FHWA 1978b)

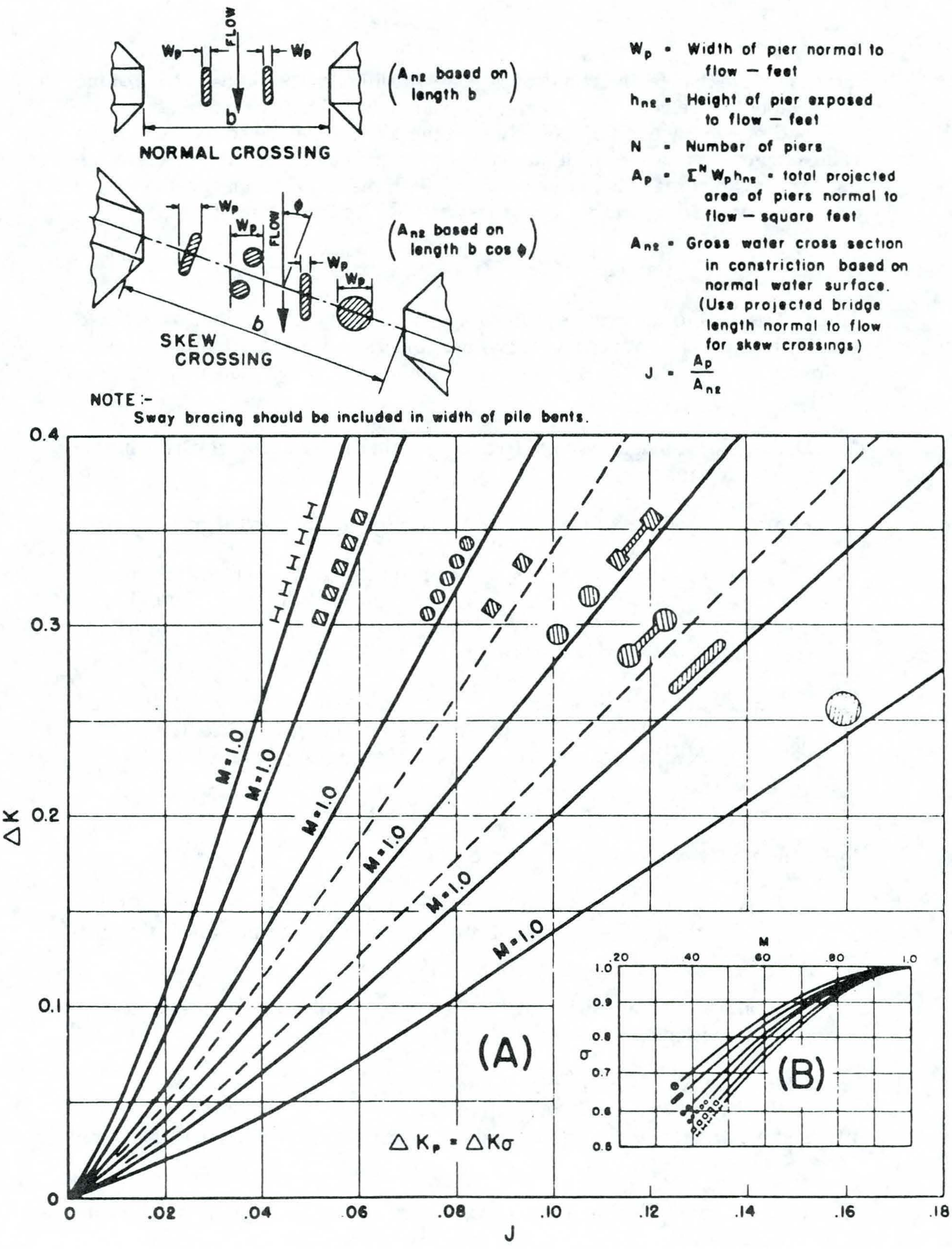


Figure 7.4
Incremental Backwater Coefficient for Piers
(FHWA 1978b)

there are more than 5 piles in a bent. A bent with 10 piles should be given a value of ΔK_p about 20 percent higher than those shown from bents with 5 piles. If there is a good possibility of debris collecting on the piers, it may be advisable to use a value greater than the pier width to account for debris blockage. However, modeling of debris blockage should be reviewed with the jurisdictional agency. For a normal crossing with piers, the total backwater coefficient becomes:

$$K^* = K_b \text{ (from Figure 7.2 or 7.3)} + \Delta K_p \text{ (from Figure 7.4)} \tag{7.6}$$

7.3.1.4 Design Procedure: The following brief step-by-step outline is for determining backwater produced by a bridge constriction. Detailed procedures illustrated by examples are presented in *Hydraulics of Bridge Waterways* (FHWA 1978b).

1. Determine the magnitude and frequency of the discharge for which the bridge is to be designed.

2. Determine the stage of the stream at the bridge site for the design discharge.

3. Plot representative cross section of stream for design discharge at Section 1, if not already done under Step 2. If stream channel is essentially straight and cross section substantially uniform in the vicinity of the bridge, the natural cross section of the bridge site may be used for this purpose.

4. Subdivide above cross section according to marked changes in depth of flow and roughness. Assign values of Manning roughness coefficient, n, to each subsection. Careful judgement is necessary in selecting these values.

5. Compute conveyance and discharge in each subsection.

6. Determine value of kinetic energy coefficients.

7. Plot natural cross section under proposed bridge based on normal water surface for design discharge, and computed gross water area (including area occupied by piers).

8. Compute bridge opening ratio, M, observing modified procedure for skewed bridge crossings.

9. Obtain value of K_b from appropriate base curve.

10. If piers are involved, compute value of J and obtain incremental coefficient ΔK_p.

11. If eccentricity is severe, compute value of eccentricity and obtain incremental coefficient, ΔK_e.

12. If a skewed crossing is involved, observe proper procedure in previous steps, then obtain incremental coefficient, ΔK_s, for proper abutment type.

13. Determine total backwater coefficient K* by adding incremental coefficients to base curve coefficient K_b.

14. Compute backwater by Equation 7.1.

15. Determine distance upstream to where the backwater effect is negligible.

7.3.2 Other Hydraulic 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.

7.3.2.1 Scour: To determine scour at bridges, refer to *Predicting Scour at Bridge Piers* (Laursen, February 1980) and *HEC No. 18, Scour at Bridges* (FHWA 1989—draft form). Total scour at a bridge crossing consists of three components which are generally cumulative, and a fourth, lateral stream migration, which can move the general bed grade horizontally to a new location. The first three components are:

Long Term Aggradation or Degradation: This is a variation to river bed elevation, usually occurring over long periods of time due to changes in controls, such as dams and in-stream mining. Such variation can result in modification of sediment discharge and river geomorphology, such as a departure from a meandering to a braided stream. The changes may be natural or man-induced, but are far more often documented as the latter (FHWA 1978a).

Long term bed elevation changes (aggradation or degradation) may be the natural trend of the stream or may be the result of some modification to the stream or watershed condition. Factors that affect long term bed elevation changes are: dams and reservoirs (upstream or downstream of the bridge), changes in watershed land use (urbanized, deforestation, etc.), channelization, cutoff of a meander bend (natural or man-made), changes in the downstream base level (control) of the bridge reach, gravel mining from the stream bed, diversion of water into or out of the stream, natural lowering of the total system, movement of a bend or bridge location in reference to stream platform, and stream movement in relation to the bridge crossing.

General Scour: 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 caused by the local area geometry and water surface controls.

General scour results from the acceleration of the flow due to either a natural or bridge contraction or both (contraction scour). General scour may also result from the location of the bridge on the stream, such as, its location with respect to a stream bend or its location upstream from the confluence with another stream. In the latter case, the elevation of the downstream water surface will affect the backwater on the bridge, hence, the velocity and scour. General scour may occur during the passage of a flood and the river may fill in as the flood recedes, thus it may not be directly evident; whereas, degradation always results in an evident change that is largely irreversible (unless the bed elevation is corrected).

General scour from a contraction usually occurs when the normal flow area of a stream is decreased either by a natural constriction or 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 which cut off the overland flow that normally goes across the floodplain during high flow. This latter case also can cause clear-water scour (defined further under Local 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.

Armoring: Armoring occurs on a stream or in a scour hole when the forces of the water during a particular flood are unable to move the larger sizes of the bed material. This protects the underlying material from movement. Scour around an abutment or pier may initially occur but as the scour hole deepens the coarsest bed material may move down in the hole and protect the bed so that the full scour potential is not reached.

Lateral Stream Migration: In addition to the above, lateral shifting of the stream may also erode the approach roadway to a bridge and change the angle of the flow in the waterway at the bridge crossing, causing a change in the total scour.

Local Scour: This is the scour that occurs at a pier or abutment as the result of the pier or abutment obstructing the flow. This type of scour only occurs on a small portion of the channel width, where the obstructions to the flow cause local current accelerations creating vortices that remove the material around them.

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.

Generally, local scour depths are much larger than the other two. But, if there are major changes in stream conditions, such as a large dam built upstream or downstream of the bridge or severe straightening of the stream, long term bed elevation changes can be the larger element in the total scour.

Types of local scour are:

» *Clear-water scour:* Clear-water scour occurs when there is no movement of the bed material of the stream upstream of the crossing but the acceleration of the flow and vortices created by the piers or abutments causes the material at their base to move.

» *Live-bed scour:* Live-bed scour occurs when the bed material upstream of the crossing is also moving.

Table 7.1 presents a checklist of potential problems relating to channel movement/scour and the causative factors which should be examined (see also Tables 5.8, 5.9, and 5.10 (pages 250 through 252) for the Design Checklists).

7.3.2.2 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 structure. 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.

The minimum freeboard—in feet—for new bridges designed for the 50-year peak storm or greater is the velocity head ($V^2/2g$) plus two feet, where V represents the average velocity of flow in the channel approaching the bridge. Both designs should also be checked hydraulicly for the 100-year storm. For bridges designed for less than the 50-year storm, the freeboard criteria in Section 5.3.3.3 applies.

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.

A new bridge replacement will not be permitted to create a rise in the existing flood water surface, to cause an increase in lateral extent of the floodplain, or to otherwise worsen existing flood conditions.

Supercritical Flow: For the special condition of supercritical flow within a lined channel, a 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 clear and permit the flow to pass unimpeded and unchanged in cross section.

Large Culvert Structures: When large culverts are used in lieu of bridges, the design approach often differs. For hydraulic analysis of culverts, refer to Chapter 4.

Table 7.1
Checklist of Potential Problems and Factors to be Examined for Channel Movement/Scour
(See also Tables 5.8, 5.9, and 5.10)

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
- » Clear-water scour
- » Bends
  - Natural stream constriction
  - Long approaches to the bridge over the floodplain
  - Berms from sediment deposits
  - Island or bar formations
  - Debris
  - Growth of vegetation in floodplain or channel
  - Bed and sediment characteristics

Armoring

Lateral Migration

Local Scour

- » Width of pier
- » Projected length of abutment into flow
- » Length of pier
- » Velocity of approach flow
- » Size of bed material
- » Angle of approach flow to the pier or abutment
- » Shape of pier or abutment
- » Bed configuration
- » Debris
- » Clear-water scour
- » Live-bed scour

7.4 References

FHWA, see U.S. Department of Transportation Federal Highway Administration

Richardson, E.V., 1988, *Highways in the River Environment*, U.S. Department of Transportation Federal Highway Administration.

U.S. Army Corps of Engineers Hydrologic Engineering Center, 1990, *HEC-2 Water Surfaces Profiles, Users Manual.*

U.S. Department of Transportation Federal Highway Administration, 1978a, *Countermeasures for Hydraulic Problems at Bridges*, Volume II, U.S. Geological Survey, Water Resources Division, PB-297-685.

——, 1978b, *Hydraulics of Bridge Waterways*, Hydraulic Design Series No. 1.

Notes

8

Detention/Retention

8.1 Introduction

As part of a total system of urban stormwater management, detention and retention facilities are man-made storage measures intended to mitigate the negative impacts of urbanization on storm drainage, which include:

» Increased peak flow rates.

» Loss of natural depression storage.

» Reduction of infiltration capacity in a drainage basin.

» Reduction of natural vegetation, which, in a natural state, reduces storm runoff through the process of interception.

» Increased pollutant load in surface runoff.

This chapter presents the engineering methodologies and details associated with the planning, analysis and design of detention and retention facilities within Maricopa County, Arizona. The guidelines herein are intended to achieve the following goals:

1. Design of detention/retention facilities that satisfy the ordinance provisions of Maricopa County and/or the individual jurisdictional agencies within the County with regard to hydraulic function and maintainability;

2. Design of detention/retention facilities that are amenities, and, where possible, incorporate multiple-use concepts; and

3. Design of facilities that will not jeopardize the quality of surface water or groundwater resources.

8.1.1 Interaction with Other Components of a Drainage System

Detention and retention facilities are components of an overall stormwater management system that is also comprised of natural and man-made channels, storm sewers, inlets, streets and other drainage structures. Their purpose is to provide temporary storage of the excess runoff from development areas and to control the increased peak rates of runoff. Proper planning and design of detention/retention facilities must consider the interaction of storage with the other components of the drainage system.

The greater the number of storage facilities in a system, the more complex is the analysis of the interaction of the various outflows. Often the increased costs of construction and maintenance of a large number of smaller storage facilities offset any savings in reduced sizes of storm sewers downstream. Planning efforts should be oriented toward minimizing the number of storage facilities in a drainage basin.

As part of the planning and design process, the engineer must verify that releases from the detention/retention facility will not adversely impact downstream conditions in terms of both manner and quantity of flow. Conditions such as peak flow, velocity, flow concentration, prolongation of flow and quality of discharge are factors to be considered.

8.1.2 Limitations on Use of Detention/Retention Facilities

The requirement for a development to provide storage of excess runoff by detention or retention facilities shall not be waived unless determined otherwise by the jurisdictional agency on a case by case basis.

In general, storage facilities are to be located so they can intercept the flow from the entire development area. If portions of the area cannot drain to a single storage facility, then additional facilities may be added to provide control of those areas as approved by the appropriate jurisdictional agency. The objective is to provide storage of excess runoff with a minimum number of detention/retention facilities located at optimum points within a development area. Whenever possible, the facilities shall be designed for multiple uses, such as parks or other recreational facilities, to offset the cost of open space and to encourage improved maintenance.

Residential developments shall have no single lot storage and the design of common facilities shall not assume any individual lot on-site storage, unless approved by the jurisdictional agency. Developments with Homeowner's Associations shall locate their facilities in private drainage tracts or public sites dedicated by the developer, in accordance with requirements determined by the jurisdictional agency. The private facilities shall be maintained by the Homeowner's Association. Public tracts shall be maintained by the jurisdictional agency. Common storage facilities from single family developments without a Homeowner's Association and with public streets shall have maintenance provisions determined by the jurisdictional agency. The number and location of storage facilities within a development is to be approved by the jurisdictional agency. Dedication to the public may require the inclusion of recreational facilities or other features deemed necessary by the jurisdictional agency.

Single lot, non-residential developments that are not served by a public storage facility shall provide the required storage on the lot itself without depressing the right-of-way area.

8.1.2.1 Regional Detention/Retention Facilities Regional detention/retention facilities are large storage facilities located at strategic sites within a drainage basin to provide control of excess runoff with an optimum (and minimum) number of storage facilities to achieve the most cost-effective drainage system. Advantages of this type of facility include the following:

» The siting and design of regional storage facilities are normally incorporated as part of an overall drainage master plan. Thus, alternative siting combinations and their respective hydraulic routing effects can be investigated. Storage alternatives can be evaluated with other factors (i.e., conveyance system, land and maintenance costs), to arrive at an optimal solution for the drainage basin.

» Operation and maintenance costs are reduced. Maintenance of regional facilities is typically the responsibility of the jurisdictional agency. The reduced cost of operation and maintenance often can offset the increased cost of tributary storm sewers which must be sized to carry higher peak rates of flow.

» Regional facilities are more effective and reliable because they are planned, designed and maintained as part of a total drainage system. On-site facilities can be less reliable and less effective because they are constructed randomly as a basin develops and because maintenance efforts can vary. The result of on-site facilities is a higher percentage of malfunction.

Advance planning is the key element in the regional approach to stormwater detention/retention. The jurisdictional agencies within Maricopa County have agreed that basin-wide master drainage planning is necessary for the development of cost-effective systems for stormwater management. Planning for regional detention/retention facilities for a drainage basin typically includes:

1. Development of an optimum drainage master plan for the basin; the jurisdictional agencies have agreed that master drainage planning efforts will continue, in order to achieve efficient, cost-effective drainage systems and to ensure that multiple use opportunities are preserved.

2. Multi-jurisdictional cooperation, because natural drainage basins do not necessarily follow jurisdictional boundaries.

3. Participation by property owners, developers, engineers and the general public.

4. A plan for implementation that incorporates construction phasing of facilities.

5. Establishing a framework for fair and equitable financing of capital and maintenance costs.

8.2 Design Criteria

This section presents certain guidelines, procedures and criteria to be used in the analysis and design of detention and retention facilities. Where specific policies and criteria vary, the engineer should contact the specific jurisdiction in which he/she is preparing a design.

8.2.1 Criteria for Retention Facilities

The following general criteria apply to the design of stormwater retention facilities:

8.2.1.1 Design Frequency

Retention facilities shall be designed to retain the peak flow and volume of runoff from rainfall events up to and including the 100-year, 2-hour duration storm falling within the boundaries of the proposed development.

8.2.1.2 Hydrology: Procedures and criteria for development of inflow hydrographs for retention facilities are described in the *Drainage Design Manual for Maricopa County, Volume I, Hydrology* (Flood Control District 1990). Some jurisdictional agencies have developed simplified equations for determining the volume required for retention. The engineer should verify the methodology for calculation of the required storage volume with the appropriate jurisdiction. Off-site flows may not be routed through a retention facility unless specifically approved by the appropriate jurisdictional agency.

8.2.1.3 Siting and Geometry: With respect to siting, retention facilities which utilize a method of subsurface disposal shall be located such that the infiltration surface will be a specific distance, both horizontal and vertical, from any functioning water well. The appropriate jurisdiction should be contacted regarding regulations governing the siting of such facilities near wells or near the static groundwater table.

Basic requirements regarding facility shape, side slopes, depth and bottom configuration are provided below. Additional details are presented in Sections 8.3, 8.4, and 8.5 in conjunction with guidelines regarding safety, operation and maintenance, aesthetic, and multiple use considerations.

Shape: As a general rule, curvilinear, irregularly shaped facilities will have the most natural character. A wide range of shapes can be considered and utilized to integrate the detention facility with the surrounding site development. Smooth curves should be used in the plan layout of the grading for the facility.

Side Slopes: Where grass is intended to be established, side slopes shall not be steeper than 4 horizontal : 1 vertical. Where other protection measures are intended, such as shrub planting, rock riprap or other structural measures, slopes shall not exceed 3 horizontal : 1 vertical unless approved by the appropriate jurisdiction. Where slopes abut the street right-of-way, the minimum slope shall be 4 horizontal : 1 vertical regardless of surface treatment. Some jurisdictions require a flatter slope. The engineer should verify the slope requirement prior to commencing design. Transitions from slopes to level ground at the top and bottom of a facility shall be smooth curves. In all cases, slopes must be designed to allow for safe operation of maintenance equipment. Refer to Section 8.4 for provision of maintenance access. Side slope design should be done with the visual character of the completed facility in mind. A more natural appearance can be achieved by varying side slopes within a detention area.

Depth and Bottom Configuration: Maximum ponding depth and freeboard requirements vary within Maricopa County and specific criteria for such must be verified by the engineer with the appropriate jurisdictional agency. With respect to grading, deep facilities should be avoided, if possible. For facilities in excess of six feet deep, consideration should be given to the use of flatter side slopes or the provision of intermediate benches along side slopes.

The minimum cross slope in the bottom of a retention facility shall be 1 percent.

8.2.1.4 Drain Time

The design of all retention facilities shall be such that the stored runoff shall be discharged completely from the facility within 36 hours following the storm event.

8.2.1.5 Lining/Surface Treatment: In keeping with the goal of retention facilities as amenities that incorporate multiple use concepts where possible, grass and/or landscape plantings are preferred surface treatments. As a general rule, grass and plant species used for landscape development and revegetation should be native to Maricopa County. A registered landscape architect should prepare the landscape design with consideration toward use of plant species appropriate for the level and frequency of inundation of the facility. Permanent irrigation systems are required for grass areas and most types of basin revegetation and landscaping. However, use of native and drought tolerant species (including seeding) may only require a temporary system to obtain effective germination and establishment. Whether permanent or temporary, that portion of the irrigation system within the flood zone must be designed to tolerate inundation and silt accumulations.

The use of inert materials is appropriate for stabilization and erosion control where steep slopes are unavoidable, along channels, at inflow points, at the outlet control structure and any other location where flowing water may threaten stability. Use of these materials should be properly engineered and should respond to aesthetic considerations. Inert materials for erosion control include:

» Loose rock riprap with a specific, engineered gradation

» Loose or grouted boulders (minimum dimension 18 inches and larger)

» River stone

» Gabions

» Soil cement and concrete

Designs that combine landscape planting with the use of inert materials are recommended. Voids can be designed within the inert material to allow installation of plants. The result is a durable and attractive method of protection.

8.2.1.6 Low Flow Channels: Low flow channels may be appropriate for retention facilities that dispose of stored runoff via a positive outlet. A jurisdiction may require provision of a low flow channel in a retention facility to ensure that the facility drains completely. Refer to Section 8.2.2.6, for guidelines regarding low flow channels.

8.2.1.7 Retention Facility Inlet and Outlet Structures: Conveyance of runoff into a retention facility often involves directing the inflow down a slope into the storage area. The design of an inlet structure shall be such that inflow is directed into the facility in a non-erosive manner and without adverse impacts to the retention facility or to upstream areas. The engineer is referred to analysis methods presented in Chapter 6 for the design of inlet structures.

Retention facilities shall be drained by either a positive gravity outlet, a pump station, or by subsurface disposal measures. To facilitate maintenance, the minimum allowable pipe size for the primary outlet pipeline from a retention facility shall be 12-inches in diameter. If the flow capacity must be further reduced, an orifice plate may be attached as shown in Figure 8.7a (page 375). Additional details regarding pipe outlets, including trash racks and outlet energy dissipation are presented in Section 8.2.2.7. General guidelines for the design of pump stations are contained in Chapter 9. Outflow rates from gravity or pumped outlets shall not exceed the capacity of the downstream drainage system. The engineer shall make all investigations necessary to document that no adverse impacts to downstream areas will occur as a result of releases from a retention facility. An emergency spillway shall be provided for all retention facilities as described in Section 8.2.2.7.

8.2.1.8 Subsurface Disposal: The primary methods of underground disposal of stormwater runoff at retention facilities are engineered basin floors and dry wells. Infiltration rates of basin floors or dry wells shall not be used in determining outflow rates in flood-routing procedures.

Engineered Basin Floors: Analysis and design of the bottom of a retention facility intended for subsurface disposal is detailed in *Underground Disposal of Stormwater Runoff* (FHWA 1980); refer to this publication for specific design criteria.

Dry Wells: Dry wells may be used for subsurface disposal of stormwater, if approved by the jurisdictional agency, and if criteria such as subsurface strata permeability, groundwater levels and maintenance can be satisfactorily addressed. The main cause of dry well failure is clogging of the transmission media (gravel) by silt and debris. Failure can be hastened by poor maintenance. Figure 8.1 shows a typical dry well installation, while Figure 8.2 shows examples of surface treatments.

The following list of general requirements and criteria shall be used in the design and construction of dry wells (or other methods of subsurface disposal of stormwater). In addition, the engineer is referred to specific dry well policies of the applicable jurisdictional agency within Maricopa County.

» The feasibility of subsurface disposal of stormwater at a site must be documented by field investigations and a report by a registered civil engineer. Field investigations shall include percolation tests to obtain permeability rates for use in the design of the retention facility. The accepted design disposal rate for a dry well is not to exceed 0.1 cfs per well unless a greater rate can be supported by a detailed, certified soils report. Should the soils report indicate a higher rate, a conservative value of 50 percent of the higher rate (not to exceed 0.5 cfs) shall be used to compensate for deterioration over time. The infiltration surface of the subsurface disposal facility must be located a specified minimum distance from the static groundwater table, both horizontally and vertically, depending on the

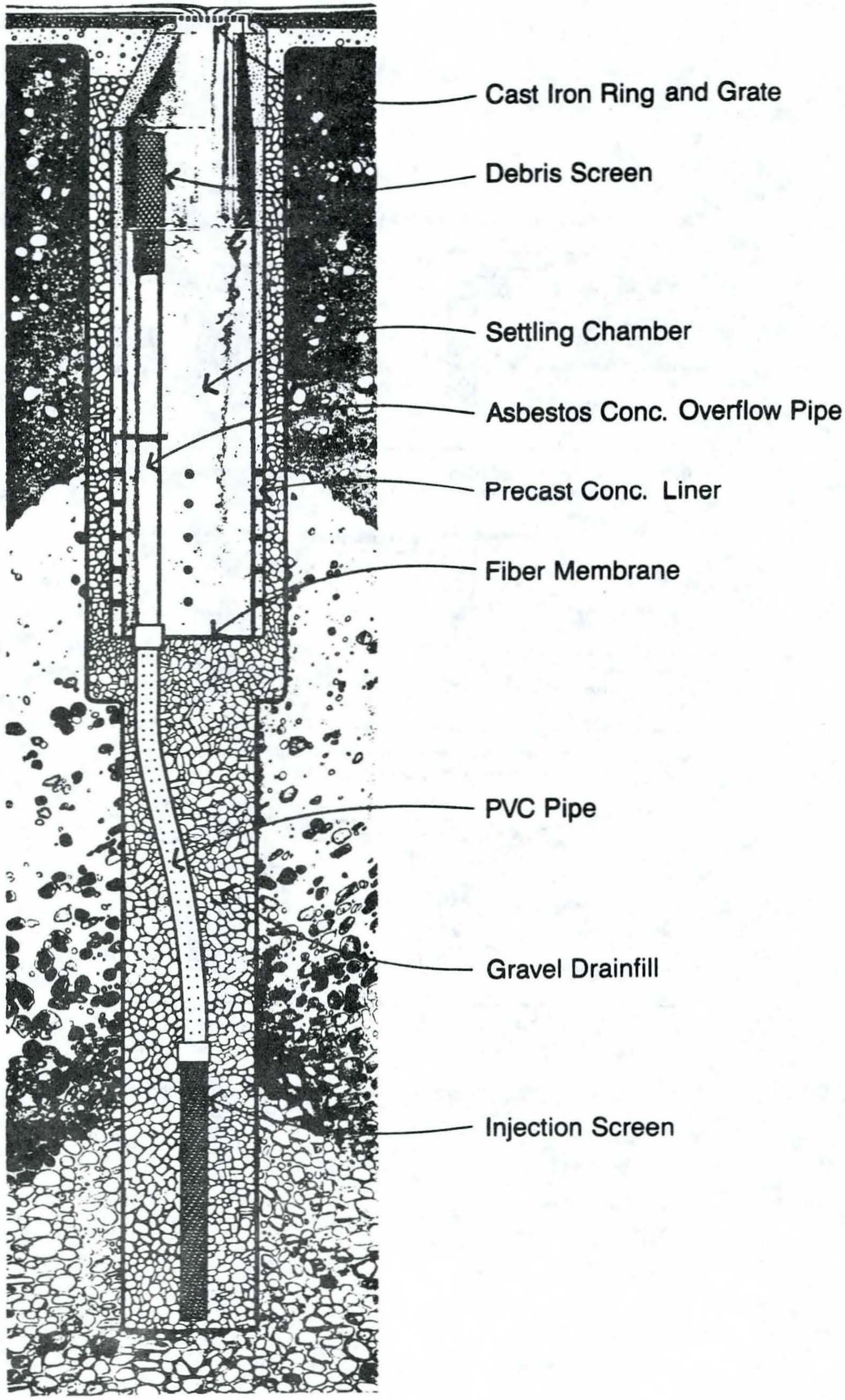
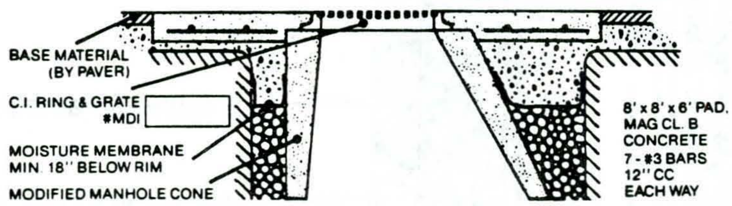
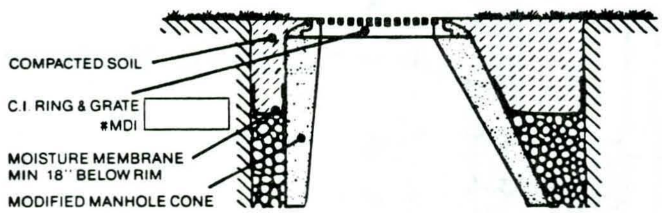


Figure 8.1
Typical Dry Well Installation

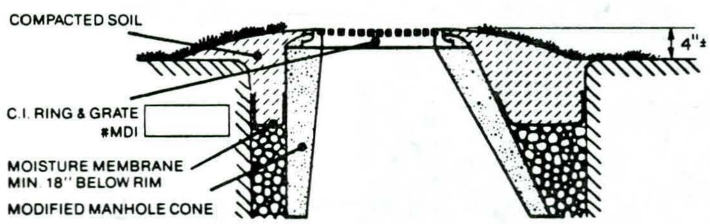
A. Adds a concrete pad for heavy traffic areas.



B. For landscaped retention ponds and planters. No paving or pad. In areas where silt might flow to drywell, use C.



C. Use in landscaped retention/detention basins or where heavy silt flow is anticipated. Height should be 4"±.



D. A special design where unstable soil conditions could cause surface subsidence. Also installed with connecting pipes and trenches.

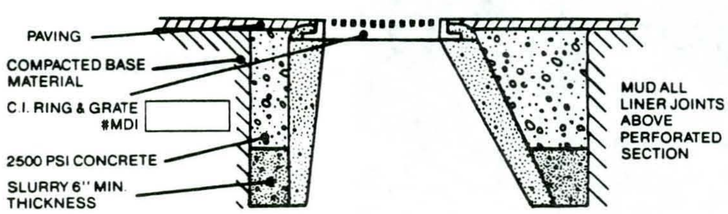


Figure 8.2
Typical Dry Well Surface Treatments
(McGuckin Drilling Inc. 1987)

type of development proposed. The appropriate jurisdictional agency should be contacted for specific criteria regarding this item.

» The design of a dry well must include provisions for trapping sediment within a settling chamber. This measure will significantly increase both the efficiency and useful life of the well. Once a year, at a minimum, the settling chamber should be inspected, and it should also be inspected after any major inflow to the dry well. Sediment shall be removed from the chamber at such a time that approximately one-half of its capacity is filled. This level of sediment buildup shall be clearly marked on the inside of the settling chamber. All sediment removed from a settling chamber shall be disposed of either at an authorized sanitary landfill or at any other suitable location approved by the governing jurisdiction.

» A test well shall be installed for any retention facility utilizing dry wells for stormwater disposal. Upon approval of performance, this test well may then be used as one of the functioning dry wells within the retention facility. For purposes of design, the "initial" well-injection rates (determined from the test well) shall be multiplied by the factor 0.5 in order to establish "aged" well-injection rates to be used for purposes of determining the required number of dry wells ultimately needed within the facility.

» Infiltration rates of dry wells shall not be used in determining outflow rates in flood-routing procedures. Any retention facility which relies solely upon infiltration as its method of drainage shall be sized to contain the maximum storage volume that would be required without considering an outflow rate.

» Disposal methods using infiltration shall not be permitted for stormwater runoff which carries significant concentrations of sediment. This includes stormwater runoff flowing through sand bed channels, as well as stormwater runoff emanating from a predominantly natural watershed.

» During site development, all dry wells shall be securely covered with filter cloth or other material to prevent the introduction of excessive sediment into the settling chamber.

» Retention of runoff emanating from industrial developments and infiltration of runoff to the subsurface will be handled on a case-by-case basis by the appropriate reviewing agency.

» Runoff stored in a retention facility shall be completely drained from the facility within a maximum time period of 36 hours following the storm event. Dry wells that cease to drain a facility in a 36-hour period shall be replaced by the owner with new ones, unless an alternate method of drainage is available.

8.2.2 Criteria For Detention Facilities

8.2.2.1 Design Frequency: Stormwater detention facilities incorporated within new developments shall be designed to ensure that, at a minimum, the post-development 100-year, 2-hour duration peak discharge from the site will not exceed the pre-development conditions (see Figure 8.3, 100-year 2-hour Isopluvial Map). In jurisdictions where multi-frequency control is required, the design shall be prepared to regulate the peak discharge rates for one or more storm events in addition to the 100-year storm. Specific multi-frequency events shall be verified with the appropriate jurisdiction.

8.2.2.2 Hydrology: For a typical stormwater detention facility, there are three variables to be considered in flood routing through the structure:

1. Inflow to the facility, which varies as a function of time;

2. Outflow from the facility, which varies as a function of time; and

3. Storage which is the result of the difference between the inflow and outflow for a period of time or time interval.

This is shown graphically in Figure 8.4.

The outflow hydrograph from a proposed stormwater detention facility shall be determined using the "Storage Indication" or "Modified Puls" method of flood routing. See Section 8.7 for a detailed description of this routing procedure. Other similar hydrologic routing methods may also be used, provided that the chosen method is first approved by the appropriate review agency. If a computer program for flood/reservoir routing is intended to be used, documentation of the program shall be submitted to the appropriate review agency prior to commencing design. Non-tributary flows may not be routed through a detention facility unless specifically approved by the jurisdictional agency.

Detention ponds in series (i.e., when the discharge of one facility becomes the inflow of another) are complex and require special consideration and design by a hydraulic engineer. If such a system is unavoidable, the engineer must submit a hydrologic analysis which demonstrates the system's adequacy. This analysis must incorporate the construction of hydrographs for all inflow and outflow components.

8.2.2.3 Siting and Geometry: Refer to Section 8.2.1.3, for guidelines relating to facility geometry. The minimum cross slope in the bottom of a detention facility shall be 1 percent. The bottom shall be designed to drain to a low flow channel (described below).

8.2.2.4 Drain Time: The design of all detention facilities shall be such that the stored runoff shall be discharged completely from the facility within 36 hours following the storm event.

8.2.2.5 Lining/Surface Treatment: Refer to Section 8.2.1.5 for guidelines regarding surface treatments for detention facilities.

8.2.2.6 Low Flow Channels: A low flow channel is required in the bottom of a detention facility to provide positive routing of drainage to the primary outlet structure. An example of a rectangular concrete low flow channel is provided in Figure 8.5. The engineer will provide design of the reinforcement of the channel. The channel shall have a 0.5 percent maximum longitudinal slope. Alternative low flow channel designs may be considered at the discretion of the individual jurisdictional agency, however, use of loose rock or other movable materials is specifically not permitted.

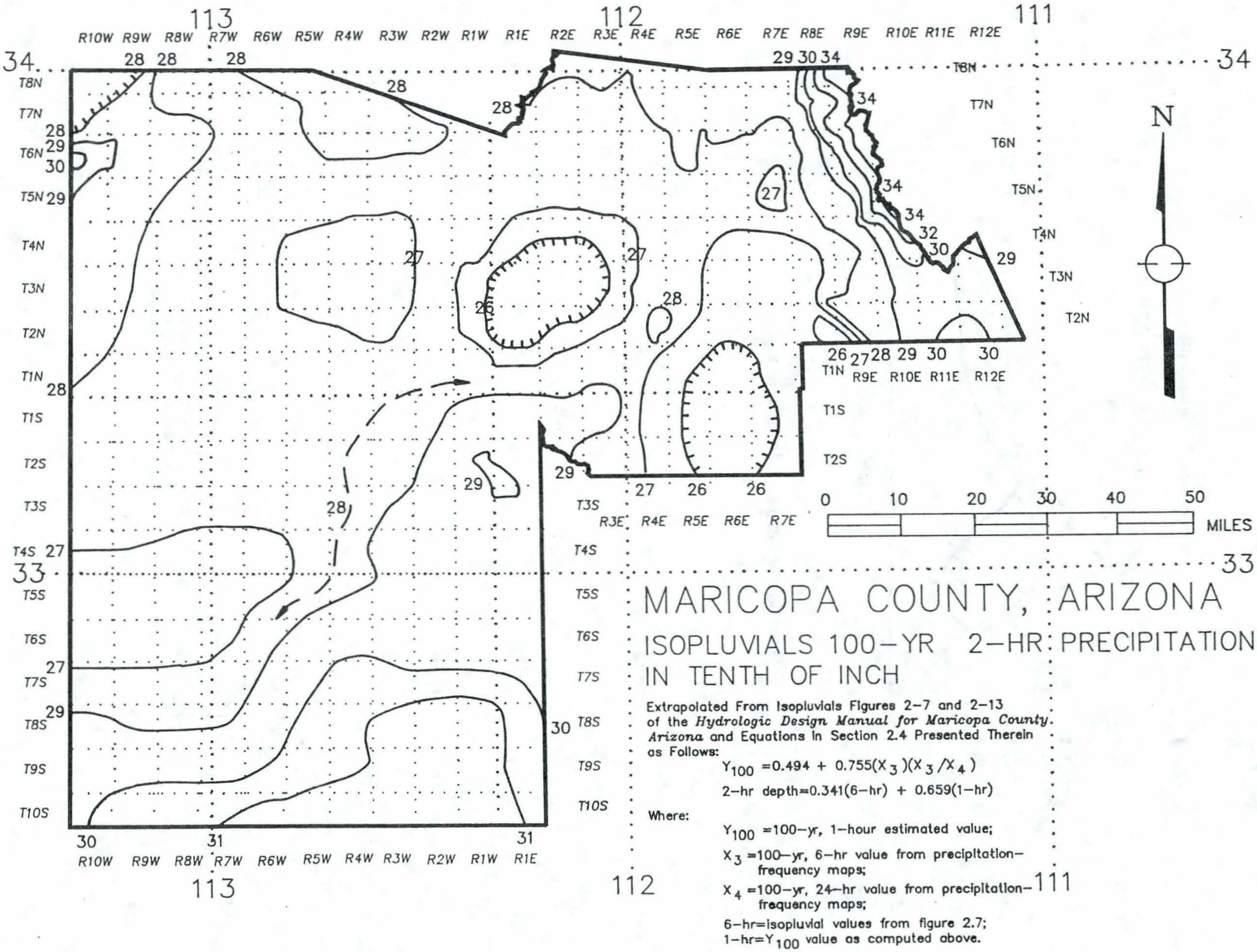


Figure 8.3
Isopluvial 100-Year, 2-Hour Precipitation

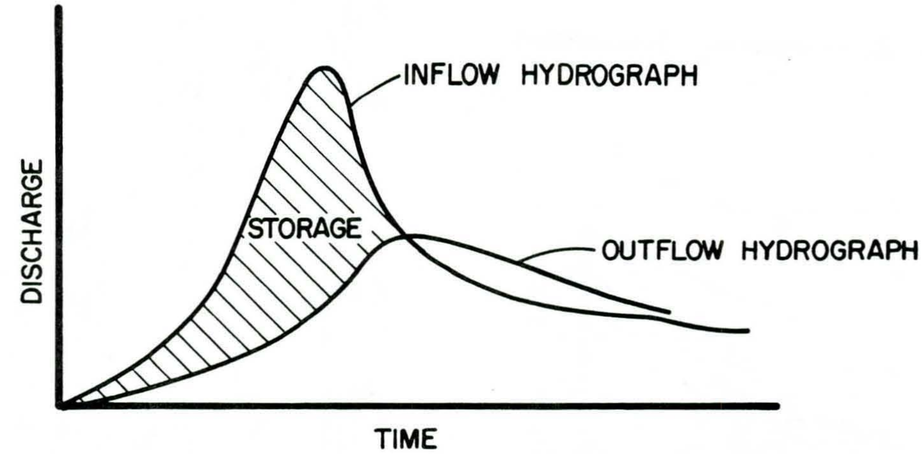


Figure 8.4
Flood Routing (Inflow and Outflow Hydrograph)

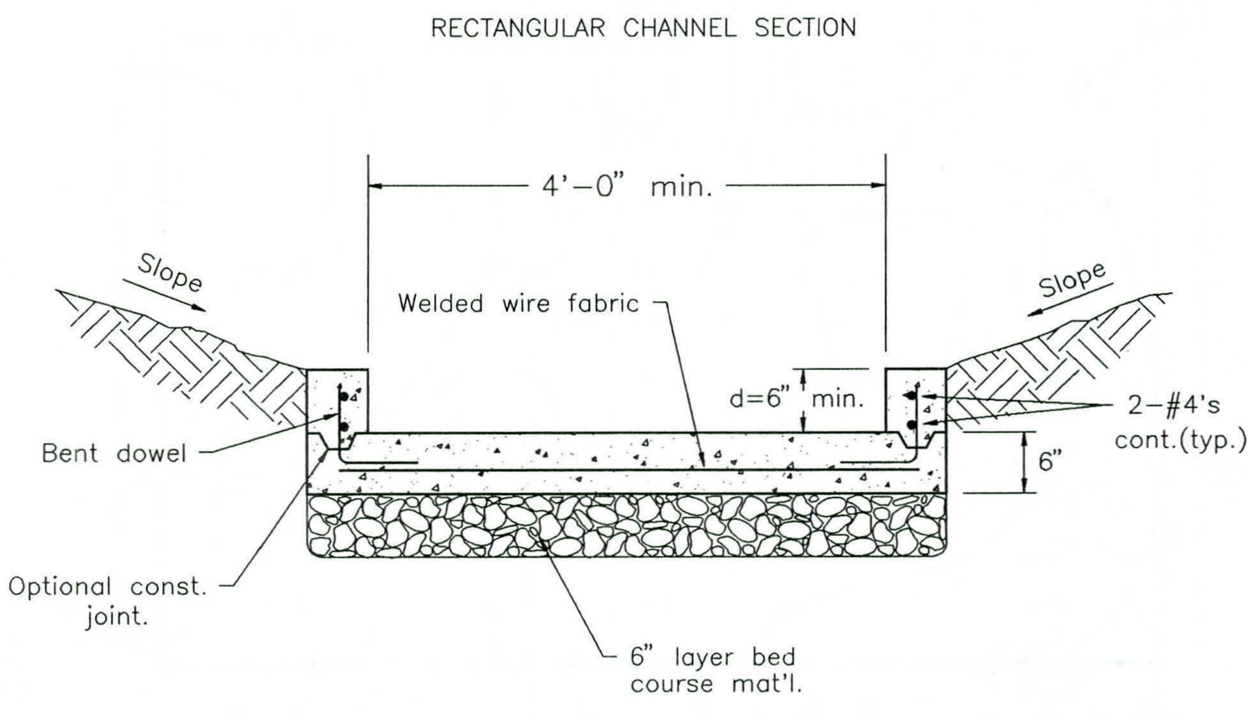


Figure 8.5
Rectangular Concrete Channel Section
(Adapted from: WRC Engineering, Inc. 1985)

8.2.2.7 Detention Facility Inlet and Outlet Structures: Inlet structures shall be designed to convey runoff into a detention facility in a non-erosive manner and without adverse impacts to the facility or to upstream areas. Analysis methods presented in Chapter 6 shall be used for designing inlet structures.

Outlet structures are an important component of stormwater detention facilities since they control the rates of release from the facility, the water depth, and storage volume in the facility. Outlet structures are classified as: 1) primary outlet structures that provide the hydraulic control for the specific design event(s) required by the jurisdictional agency; and 2) emergency spillways that provide safe routes, typically via surface overflow, for storm events in excess of the design frequency or in the case of debris blockage or malfunction of the primary outlet structure.

Primary Outlet Structures: Within Maricopa County, jurisdictional agencies may require attenuation of a single frequency storm or a number of frequencies. Refer to the specific requirements of the jurisdiction where the design is being prepared; however, two-stage and multi-stage control structures are becoming more widely used. Figure 8.6 presents examples of single frequency and multi-frequency outlet control structures. The minimum allowable pipe size for primary outlet structures is 12-inches in diameter. If the flow capacity of an outlet pipe must be further reduced, an orifice plate may be attached, as shown on Figure 8.7(a). The orifice plate must be constructed of heavy, galvanized steel and attached by tamper-proof bolts. Other outlet configurations may be allowed provided they meet the requirements of the permitted release rates at the required volume and include proper provisions for maintenance and reliability.

Primary outlet structures, particularly those controlling multiple storm events, are often special design structures unique to specific site applications. Consideration must be given to structural adequacy and flotation under hydrostatic loads.

- *Trash Racks:* Trash racks shall be provided for pipe and orifice outlets. The trash rack assembly shall be hinged or removable to allow access to the outlet construction. The mesh or bar screen shall be fabricated of steel designed to withstand the hydrostatic load resulting from the 100-year design ponding with screen openings blocked. The rack assembly shall be galvanized steel or steel with a protective coating suitable for exposure to sunlight, as well as submerged conditions. Figure 8.7(b) provides guidelines for determining the open area requirements for trash racks. An anti-vortex device should be included with the trash rack design if vortices are anticipated which could affect hydraulic efficiency and cause erosion of adjacent earth slopes.

- *Energy Dissipation at Outlet:* Adequate energy dissipation measures shall be provided at the downstream end of primary outlet structures. Such measures shall be designed to control local scour at the pipe outlet and to reduce velocities to pre-development conditions prior to exiting onto the downstream property.

Emergency Spillways: Emergency spillways are normally surface overflow weirs, channels, or combinations thereof, provided for the safe overflow and routing of floodwaters under unusual circumstances. Such situations include the blockage or malfunction of the primary outlet structure or the occurrence of a storm event larger

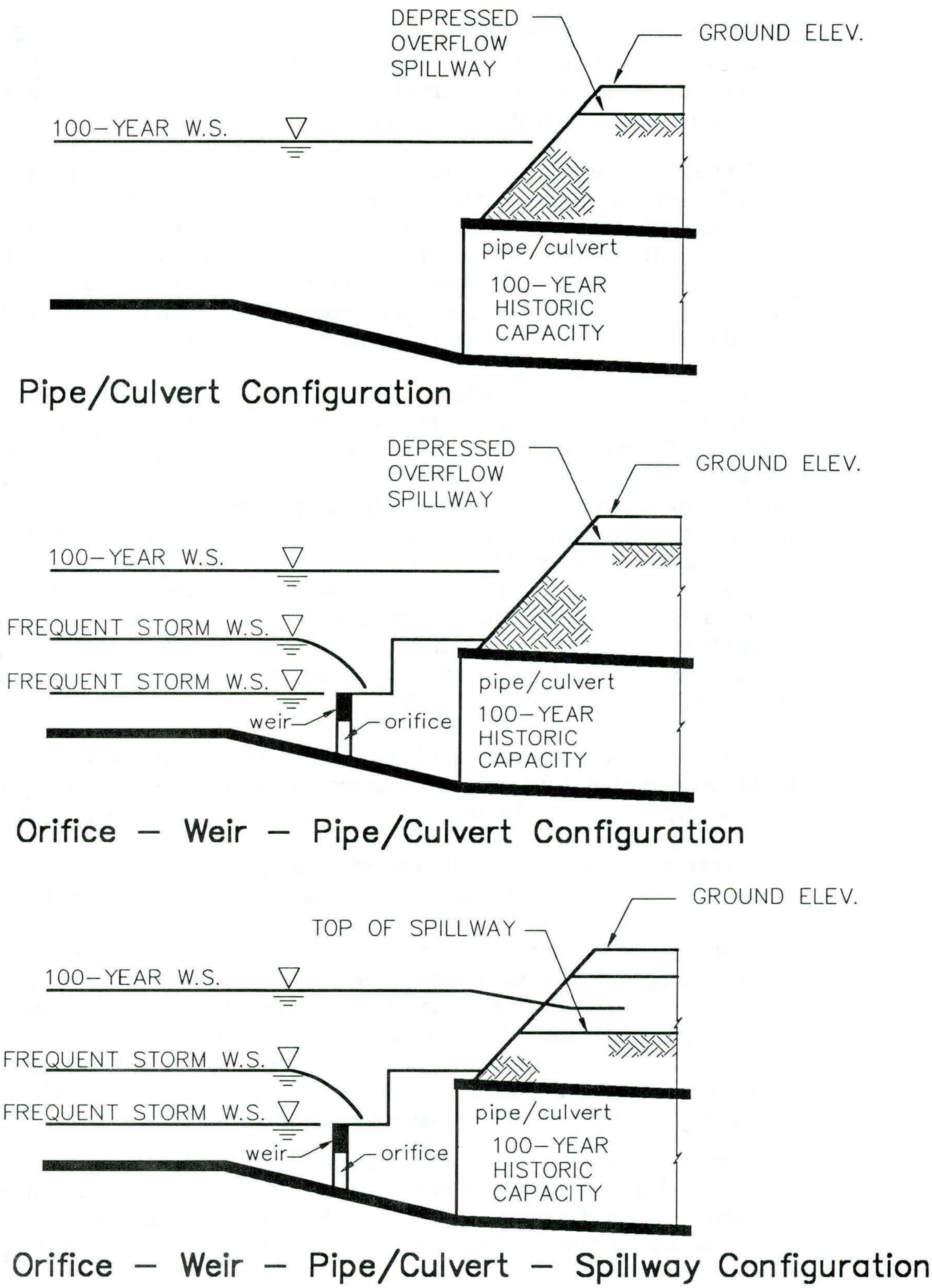
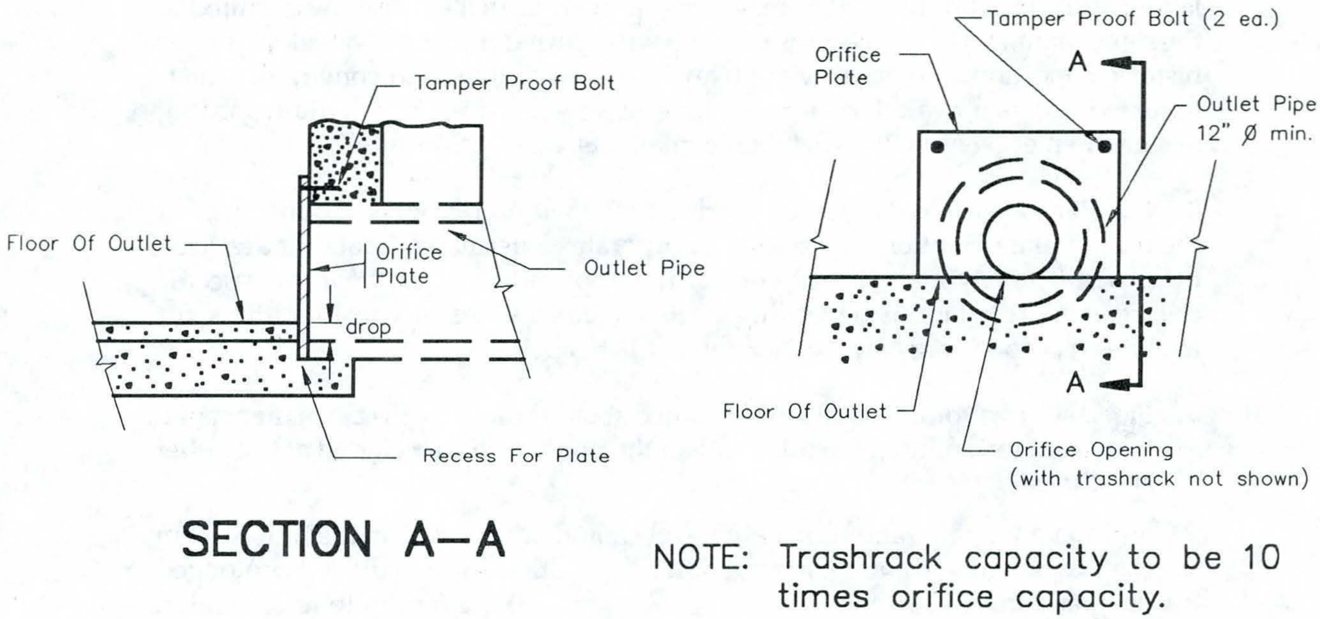
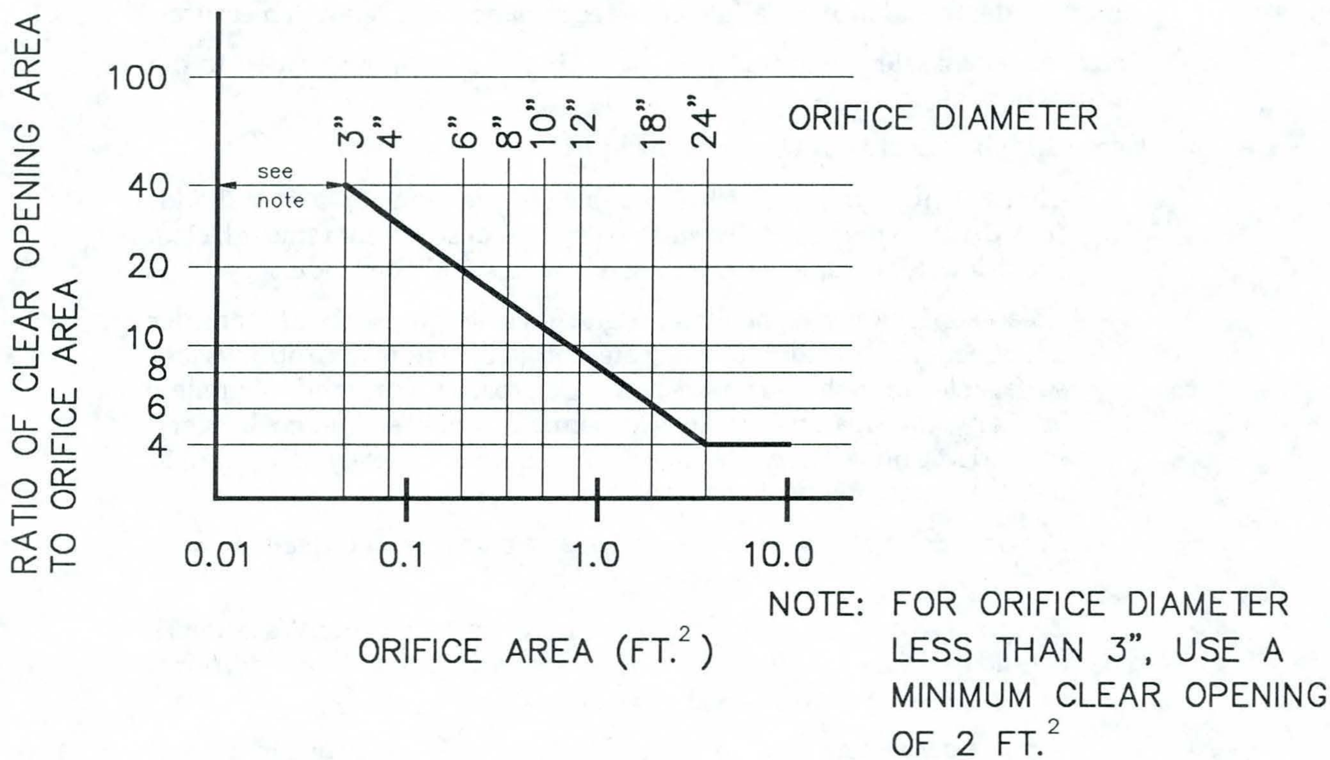


Figure 8.6
Examples of Primary Outlet Structures
(from: *Stormwater Detention/Retention Manual*, Pima County Department of Transportation and Flood Control District, City of Tucson, Simons Li and Associates, Undated.)



(a) Orifice Plate Details



(b) Trashrack Area Requirements

Figure 8.7
Detention Facility Outlet Details
(Adapted from: WRC Engineering 1987)

than that for which the facility was designed. Consideration must be given to the layout and configuration of the emergency spillway so that excess flow is routed in the same manner and direction as would have occurred under pre-development or historic conditions. Emergency spillways must be designed to convey the unattenuated 100-year peak discharge at non-erosive velocities. For criteria regarding design of emergency spillways for embankments, refer to Section 8.2.4.

8.2.2.8 Permanent Pools: Certain jurisdictions within Maricopa County permit the design of a detention facility that incorporates a permanent pool for aesthetic purposes. The engineer should contact the appropriate jurisdiction for specific criteria and regulations regarding such facilities. General considerations for facilities incorporating permanent pools are listed below:

» Flood storage volume shall be maintained above the level of the permanent pool. Provision for draining the full depth of the pond shall be included at the outlet structure.

» Maintenance of a minimum water level should be ensured by the inflow from the watershed and/or by augmentation from other sources during prolonged dry periods and by the capability of the bottom of the facility to retain water. Seepage and evaporation losses should also be considered.

» Maintain water quality and minimize algae growth by designing for sufficent minimum depth and incorporating use of recirculation and aeration measures.

» Consider public safety as primary in the design of all features related to the permanent pool.

» Geometric characteristics of the pond include:

  - Choose bottom lining material suitable for retention of water and with consideration toward maintenance (i.e., ease of sediment removal, etc.). Provisions for completely draining the pond should be made.

  - Create aesthetic yet maintainable edges. Edge design also should consider the effect of drawdown of the water surface. That is, a drop in water surface elevation should not create a wide expanse of unsightly shoreline. Similarly, the area surrounding the permanent pool should be designed for periodic inundation. The area should drain completely and return to a stable surface following a flood event.

  - Provision of stable side slopes above and below the permanent water surface.

  - The pond edge shall be designed to minimize safety hazards. Water depth should be limited to 1.5 to 2 feet within 8 feet of the shoreline. Additional safety measures are discussed in Section 8.3.

  - Resolve permanent pool water depth issues versus safety needs; a 3-foot depth at shoreline required to limit pond edge vegetation growth exceeds the recommended pond edge depth (1.5 to 2.0 feet). Therefore, other safety measures must be considered.

» The design should consider measures to minimize sediment inflow to the pond. Once sediment has entered the permanent pond, then removal can be expensive and may require draining the pond. Erosion should ideally be controlled at the source or by mitigation measures along the incoming channel. However, if such measures are not feasible, a sediment trap should be designed at the pond inflow location to intercept the majority of the incoming sediment and to facilitate removal (see Section 8.6.4).

» If the detention facility and permanent pool are created by a retaining structure, such as an earth embankment, then the design guidelines for the embankments shall be followed, with particular emphasis on seepage control and embankment stability (see Section 8.2.4).

» Potential impacts downstream shall be considered. The designer should be aware that an impoundment may improve, worsen or maintain existing downstream flow characteristics, and that any changes, even apparent improvements, may be viewed as infringements of downstream riparian rights.

» Since a permanent pool is most often desired for creation of a focal amenity for a development, it is appropriate that a registered landscape architect work in conjunction with the engineer to achieve an aesthetic design with consideration of costs of construction and maintenance.

8.2.3 Criteria for Special Detention/Retention Methods

Special methods for stormwater detention / retention include underground storage, conveyance storage, roadway embankment storage, and storage in parking lots, pedestrian plazas, courtyards and common areas.

The use of rooftops as storage areas for runoff is **not permitted** in Maricopa County.

Application of the special measures discussed below is regulated according to specific jurisdictions within Maricopa County. Contact the local jurisdiction before beginning to design using any of these methods. Since the following methods often result in facilities near buildings, it should be emphasized that finished floor elevations of structures shall be a minimum of one foot above the 100-year water surface of any detention / retention facility.

8.2.3.1 Underground Storage: This type of storage involves the construction of underground tanks, pipes, or vaults which accept stormwater runoff by means of inlets and storm drain pipes. Due to the high cost of this type of installation, it is generally limited to high-density developments, where surface storage is not feasible due either to the scarcity or high cost of land—or both.

Underground storage facilities must be provided with some method of drainage (i.e. gravity drains, pumps, or infiltration). In all cases, manholes (or some other means of access to the underground storage facilities) must be provided for maintenance purposes.

8.2.3.2 Conveyance Storage: During the period that channels and floodplains are filling with runoff, the stormwater is being stored in transient form. This type of storage is known as conveyance storage. Construction of slow velocity channels with large cross sectional areas assists in the accomplishment of such storage.

Conveyance storage systems are usually feasible only on large projects, and require detailed dynamic modeling for analysis.

8.2.3.3 Roadway Embankment Storage: When feasible, use of roadway fill slopes as an embankment for a detention basin provides an economical means of stormwater storage. Special considerations must be given both to the stability of the embankment and to the protection of the embankment from erosion. Additionally, State of Arizona dam safety requirements may need to be addressed if the embankment height and/or the potential storage volume exceeds certain limits (see Section 8.2.4).

8.2.3.4 Parking Lot Storage: Using parking lots for detention/retention is a special case of surface storage. It is an economical option for meeting detention/retention requirements in high density commercial and industrial developments. Planning of areas within a parking lot which will accept ponding should be such that pedestrians are inconvenienced as little as possible. Refer to local jurisdictional standards on the percentage of the parking lot to be used as retention area and the allowable ponded depth. Deeper ponding should be confined to remote areas of parking lots, whenever possible. The maximum depth of ponded water within any parking lot location shall be one foot (1 ft). Drainage of parking lots can be accomplished by means of dry wells (if permitted), curb openings, weirs, storm drains, orifices in walls, or gated outlets.

The minimum longitudinal slope permitted within parking lot storage facilities is 0.005 ft/ft, unless concrete valley gutters are provided. With concrete valley gutters, a minimum longitudinal slope of 0.002 ft/ft may be permitted.

8.2.3.5 Storage in Plazas, Courtyards and Common Areas: Landscaped common areas, pedestrian plazas and courtyards, which are typically provided in conjunction with high density residential, commercial and office developments, provide opportunities for multiple use as stormwater detention/retention facilities. Such facilities should be designed to minimize public inconvenience, especially during frequent storm events. Public safety issues are also very important with this type of facility (see Section 8.3). Positive drainage to the outlet structures and trash/debris control must be provided to assure that the facility drains completely and efficiently.

8.2.4 Embankment Design Criteria

Whenever possible, detention/retention facilities should be constructed with the storage volume located entirely below the natural ground surface adjacent to the basin. However, in some instances this may not be possible, and embankments may be necessary to provide the required storage volume. Since the use of embankments may create a potential downstream flood hazard due to failure of the embankment, the following design considerations must be addressed in conjunction with their use. For additional information and guidelines for the design of embankments for detention/retention facilities, refer to *Design of Small Dams* (USBR 1987).

8.2.4.1 State Dam Safety Requirements: The Arizona Department of Water Resources (ADWR), Division of Safety of Dams, has legal jurisdiction over all dams (embankments) which exceed certain height and storage limits. ADWR defines a jurisdictional dam as "either 25 feet or more in height or stores more than 50 acre-feet. If it is less than six feet in height regardless of storage capacity or does not store more than 15 acre-feet regardless of height, it is not in jurisdiction."

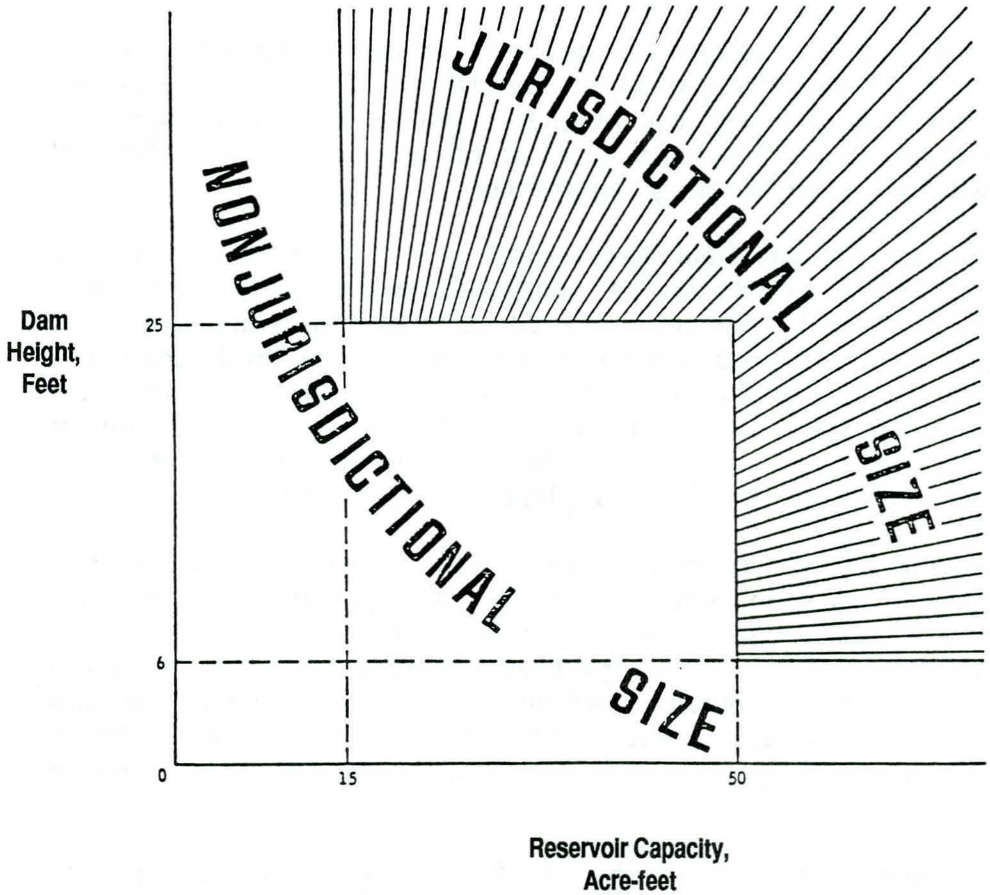
Figure 8.8 graphically illustrates the difference between a jurisdictional and a non-jurisdictional dam. ADWR should be contacted regarding specific dam-safety requirements in conjunction with the design of any embankment which might come under their jurisdiction. Those which do fall within the jurisdiction of ADWR shall comply with their applicable design requirements.

8.2.4.2 Non-jurisdictional Dam Design: Embankments for detention/retention facilities in Maricopa County that are "non-jurisdictional" according to the state criteria will generally be classified by the state as small dams with an associated hazard potential. The hazard potential classification is related to the conditions downstream of the dam. In the urban environment of Maricopa County, the potential for probable loss of life and excessive damage to development downstream (existing or future) is great. Therefore, all dams for detention/retention facilities will be considered as having high hazard potential.

The design reports, calculations, plans and specifications for construction of an embankment for a detention or retention facility shall be prepared by, or under the direction of, a professional engineer registered under the laws of Arizona, and having proficiency in civil engineering as related to dam technology. The engineer should check with the appropriate jurisdiction for specific submittal requirements for embankment dam designs. Figure 8.9 shows a typical section of an embankment dam with common components applicable to a typical detention or retention facility.

8.2.4.3 Geotechnical Engineering Studies: A geotechnical engineering study shall be conducted prior to the design of any dam. The study shall provide information on the dam site conditions such as the dam foundation and abutments (valley floor and sides), and shall provide evaluation of soil materials proposed for construction of the dam. Samples obtained from borings and exploratory pits can be tested under laboratory conditions to evaluate more precisely the soil and rock classification properties, strength, permeability, compatibility and other specialized tests pertinent to the specific project conditions. Analyses shall be conducted to evaluate conditions such as embankment slope, foundation stability, embankment and foundation seepage, internal and external erosion potential and embankment settlement. The results of these analyses are used to develop criteria for economic and safe design and construction of embankment dams. These criteria include the types and zones of embankment fill materials based on using available borrow materials, upstream and downstream embankment slopes, and recommended measures for control of seepage.

8.2.4.4 Emergency Spillway: All embankment dams for detention/retention facilities shall incorporate an emergency overflow spillway for the safe overflow and routing of floodwaters under unusual circumstances. Such conditions include the blockage or malfunction of the primary outlet structure or the occurrence of a storm event larger than that for which the facility was designed. Floodwaters that might otherwise overtop the embankment shall exit the facility via the emergency



A JURISDICTIONAL DAM is either twenty-five feet or more in height or stores more than fifty acre-feet. If it is less than six feet in height regardless of storage capacity or does not store more than fifteen acre-feet regardless of height, it is not in jurisdiction.

THE HEIGHT is the vertical distance from the lowest elevation of the outside limit of the dam at its intersection with the natural ground surface to the spillway crest elevation.

THE CAPACITY is the maximum storage, in acre-feet, which can be impounded by the dam when there is no discharge of water.

Figure 8.8
State of Arizona Jurisdictional Dam Definition
(Adapted from: State of Arizona Department of Water Resources, Safety of Dams Section.)

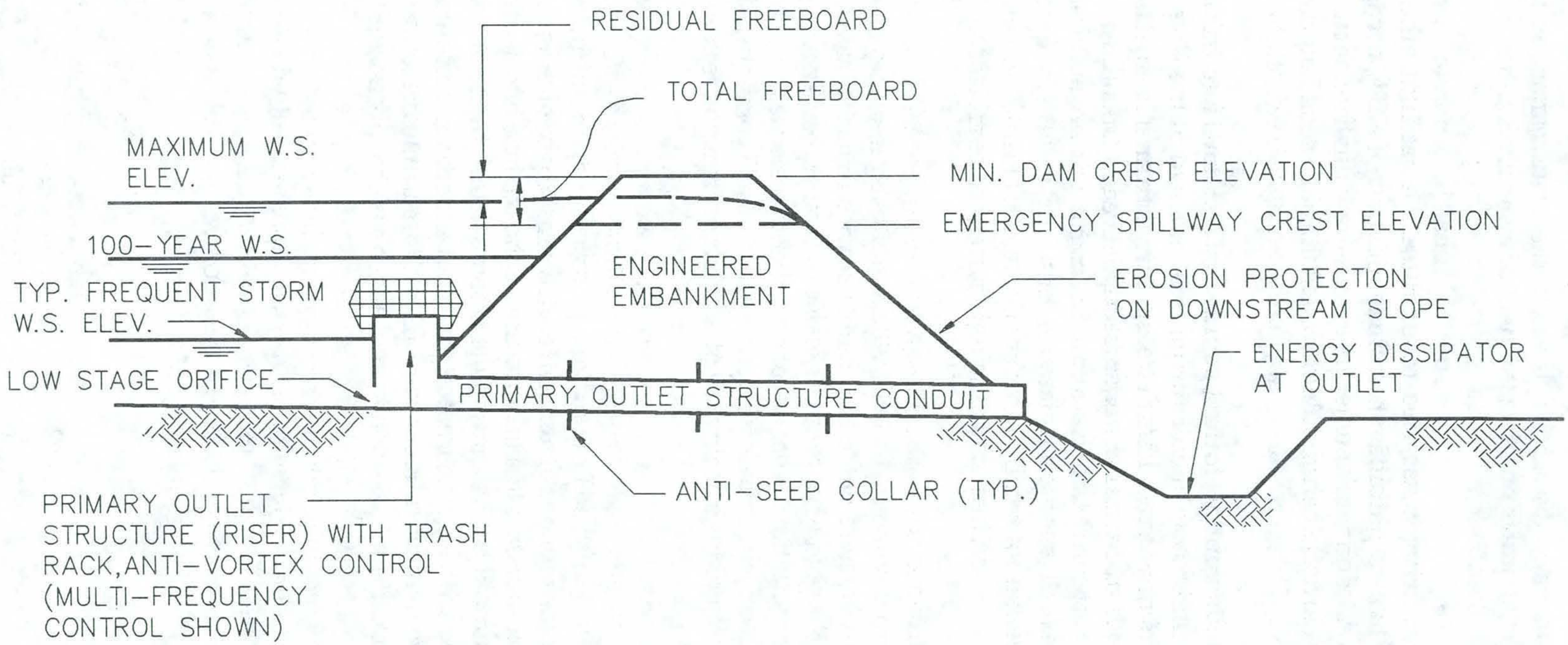


Figure 8.9
Typical Section of a Detention/Retention Facility Embankment Dam

spillway and flow downstream in the same manner and direction as would have occurred under pre-development or historic conditions.

The design of emergency spillways shall incorporate adequate erosion control and energy dissipating measures to insure the stability of the embankment. Due to the high hazard potential of embankment facilities in Maricopa County, the minimum design standard for emergency spillways shall be as indicated in Table 8.1. Total freeboard and residual freeboard dimensions shall conform to the applicable ADWR design requirements.

8.2.4.5 Primary Outlet Structure: The primary outlet structure is the main outlet structure by which stormwater is discharged from a detention/retention facility. It is typically a closed conduit structure with an inlet specifically designed to control a single frequency storm or multiple events depending on the requirements of the specific jurisdiction. Examples of typical primary outlet structures and discussions of related components such as trash racks and energy dissipation structures are presented in Section 8.2.2.7 (Figures 8.6 and 8.7). Special consideration must be given to seepage control along outlet conduits within an embankment dam, as discussed below.

8.2.4.6 Seepage: There are basically two categories of seepage considerations in embankment dam design. The primary concern is that seepage does not adversely affect the integrity or stability of the dam. The other category, water storage loss, is something the owner is usually most concerned about. This category relates to design of additional seepage control measures as required to maintain a permanent pool for reuse (water harvesting), or aesthetic or recreational purposes. Analyses shall be conducted in the following areas at a minimum, to address control of seepage for the primary function of detention or retention of stormwater.

Foundation: The flow of water through a pervious foundation produces seepage forces as a result of the friction between the percolating water and the soil medium. As the water percolates upward at the toe of the embankment, the seepage forces lift the soil by reducing its effective weight. In certain cases, this "piping" of the foundation soil can result in the failure of an embankment. A very common approach used is to excavate a cutoff trench into the foundation strata, typically into an impervious layer. The trench is then carefully backfilled with relatively impervious material.

Embankment: Seepage through an embankment will occur, even with the tightest materials. On the upstream side of the dam, the embankment soils will reflect a water level equal to the impounded water level. As the water seeps through the

Table 8.1
Emergency Spillway Design Capacity Requirements for an Embankment Dam

Dam Height	Spillway Design Capacity
$H \leq 6$ ft	Unattenuated 100-year inflow
6 ft $<$ H $<$ 25 ft	1/2 Probable Maximum Flood

dam, its pressure reduces and the water level drops. Design of the embankment should be such that seepage at the downstream toe occurs with no residual pressure. If the seepage were excessive, or were to emerge at an unplanned higher location, then erosion could begin at the discharge point and rapidly remove materials from within the embankment. Toe drains are typically designed to intercept the planned seepage flow, preventing nuisance conditions and enhancing slope stability.

Slope Stability: Combined with seepage analysis, slope stability analysis is critical. The forces pushing a mass of soil are analyzed with respect to the force resisting that movement. A related problem is slope stability during conditions of rapid change. A common concern is during a rapid drawdown, such as when operational problems with outlet works or seepage occur. With such operational problems, pressures in the soil may cause the slopes to fail during drawdown.

Piping along Boundary Conditions: Wherever there are boundary conditions, such as along an outlet conduit, spillway wall, cutoff trench or more subtle situations (such as layers of fill that have been rolled to a smooth hard surface), there is the potential of creating a more direct route for piping. The water flows at a higher erosive rate because it has a shorter, more efficient route. The technique that is often used along conduits and walls is to construct cutoff collars which extend laterally at intervals into the trench or embankment. When a much longer flow path is created, piping is minimized.

8.3 Safety

Public access and safety are inherent elements in the design of a detention or retention facility. These elements are of primary importance, particularly in the case of multiple-use facilities where public use is encouraged in areas subject to potential flooding.

Safety at detention/retention facilities is addressed in two ways. The first relates to the need to identify and communicate potential hazards to the public. For example, with proper signage, users can be made aware of the existence of potential hazards, such as flooding, high velocity flows, etc.

The second relates to the design and maintenance of the facility. Appropriate steps must be taken to mitigate potentially dangerous conditions. Where the dangerous condition cannot be prevented, appropriate measures must be implemented to keep users away from hazardous locations. If these safety concerns are appropriately addressed, there is no reason why public use of detention/retention basins should not be allowed and encouraged.

8.3.1 Identification of Potential Flood Hazard

Signs will be the principal means by which users of the facility are advised of potential flood hazards. Signs should be provided at all designated entryways. They should also be provided at intervals (approximately 100 feet) around the perimeter of the facility to inform visitors who might gain access at other than designated entrances.

In addition to entry and perimeter signs, signs should be installed within the facility. These signs should restate the potential flood hazard and should provide directions for appropriate routes out of the basin area should flooding occur.

Signs should indicate that rain in the immediate area is not a prerequisite for high water levels. They should indicate that storms elsewhere in the watershed can result in flooding, and in some cases, a wall of water may enter the facility. An interpretive display sign illustrating this phenomena may be appropriate at each major entrance.

To be effective in communicating the potential flood hazard to the public, signs must be in place when and where needed. Consideration must be given to durability and vandal-resistance. Materials, fasteners, installation techniques, mounting heights, etc. must be evaluated in the design of important warning signs.

In some instances, warning devices other than signs may be appropriate and necessary. An audible alarm or a system of flashing lights, with a remote sensor activated by floodwater in an upstream channel, might be considered. This might be necessary in locations where watershed and inlet characteristics could result in rapid filling of the facility.

Where parking lots are designed such that floodwaters will inundate the area, signs should be posted to alert users. Overnight or long-term parking should be prohibited.

8.3.2 Inlet Structures

Often higher flood flow is directed into a multiple-use facility by an overflow side channel spillway or by a drop structure. A large volume of water entering the facility at high velocity can literally wash away an individual who is on or near the inlet structure. The design of an inlet that minimizes the velocity of incoming water will greatly enhance safety and should be included in the criteria for inlet structure design. Railing or fencing shall be designed at the top of all structural walls.

It is also important to design inlet structures so that they do not become attractive nuisances. They should not be suitable for potentially dangerous activities such as skateboarding and motor-cross biking. In this regard, a rough textured surface might be more appropriate than a smooth, troweled-finish concrete surface. Features that preclude inappropriate uses of inlet structures should be used.

As noted above, signs located around the inlet structure can inform the public of potential hazards associated with the area. Informative and well-maintained signs will be critically important in basin inlet areas.

8.3.3 Outlet Structures and Spillways

There are two elements of the safety issue as related to outlet structures and spillways. The first deals with the safety of the user during flood conditions. The second deals with the uninterrupted operation of the outlet or drain required for the release of impounded water.

User safety must be of primary concern with the design of outlets or drains. They must be designed so that it is not possible for a user to be washed into an outlet pipeline and trapped. This is particularly important when considering children using the outlet structures as a playground.

A properly designed trash rack can prevent clogging by debris as well as prevent a person from being swept into the outlet structure and pipeline. In addition, where hydraulic conditions at the outlet structure can lead to the formation of a vortex, the design should include anti-vortex protection. It is important to note, however, that an outlet structure is not a safe structure during flood conditions, whether it is a horizontal pipe outlet or a riser type structure mounted to a horizontal pipeline. Powerful inlet velocities can draw a person underwater at the outlet structure regardless of the existence of a trash rack or grate. Signage is important to alert the public of this danger.

All site furnishings, such as benches, trash receptacles, and picnic tables must be secured to prevent them from becoming waterborne-debris which could clog the outlet structure.

Safety must also be considered downstream of outlet structures. Release flows, even though they may be controlled, can present a hazard. Specific conditions downstream of an outlet must be evaluated in terms of safety. To protect the public, structural walls shall have fencing or railing along the top of an outlet structure.

8.3.4 Safety Within the Facility

The principal factors associated with safety inside a detention/retention facility are user education, advance warning, potential water depth, slopes, routes out of flooded areas, and time to drain.

User education is a fundamental element in safety design for a detention/retention facility. Clear, concise signage with illustrative graphics can inform the public of the primary flood control purpose of the facility and describe the various features and their potential danger during a flood. Advance warning (alarms or lights triggered by upstream water levels) should be considered for multiple-use facilities, particularly where flash flooding and rapid basin inflow is possible.

Safety concerns increase with an increase in potential water depth. A facility with a potential water depth of 2 to 3 feet (less than the head height of most users) is typically less dangerous than a facility with a potential water depth of 5 to 6 feet, or more. For reasons of safety, potential water depth in detention/retention facilities should be kept to a minimum. When possible, potential water depth should be kept to 3 feet or less.

In all facilities, regardless of depth, slopes in flood-prone areas should be kept as shallow as possible. This will allow users who find themselves caught in flooded areas (or users who deliberately enter flooded areas) to walk out and up to non-flooded zones. It is recommended that slopes in flood-prone areas not exceed 6 horizontal : 1 vertical.

In addition to slopes, consideration should be given to bottom conditions in flood-prone areas. Soils that provide firm footing when saturated are safer than soils which do not. In severe cases of unsuitable soils, partial or total removal may be necessary.

In addition to gentle slopes, routes out of flood-prone areas must be provided. Barriers that could trap a user in a flood-prone area must be avoided. Safe, well-signed exit routes that are negotiable under wet conditions must be developed.

8.4 Operation and Maintenance

There are two major components to the maintenance of a detention/retention facility. The first is to design a facility that is maintainable, and the second is the physical work required to keep the facility operating as designed and constructed. Maintenance of a detention/retention facility falls into two categories; scheduled and unscheduled. Scheduled maintenance includes those activities such as mowing, pruning, and trash removal. These activities can be predicted and can be performed on a regular basis.

Unscheduled maintenance will involve the repair of facilities after storms and flooding. The frequency and scope of this type of maintenance cannot be predicted. Examples are:

1. Embankment repair to keep erosion or rock riprap or earth fill sloughing from weakening the dam structure.

2. Debris removal during and following storms.

3. Inlet and outlet channel repairs to halt erosion and maintain hydraulic capacity.

4. Inlet and outlet structure repair to insure that the facility will function as intended.

   It is important that adequate funding be provided for unscheduled maintenance such that repairs can be made immediately after flood or inundation damage occurs.

8.4.1 Design Considerations

The following is an outline of design considerations and recommendations which facilitate specific maintenance activities.

8.4.1.1 Access: Access roads for service and maintenance vehicles should be maintained to allow for equipment access to the facility, whenever needed. Access control gates should be provided if restricted access is required.

Design recommendations:

» Access ramps into the facility shall be graded at 10 percent or less. Turning radii shall be 50 feet or greater. Access ramps shall be designed for vehicle wheel capacities not less than 12,000 lbs.

» Service drives and gates shall be located in readily accessible, but inconspicuous, locations so as to not encourage unauthorized use.

» Design access control gates and adjacent areas shall be as secure as economically feasible. Initial expenditures for access control can save significant costs in future repairs.

8.4.1.2 Sediment Removal: Sediment will inevitably be deposited in the detention/retention facility. Conditions will be worst during years when construction activity in the watershed is greatest.

Design recommendations:

» Provide stilling basins or fore-basin collection points where most sediment will be deposited (see Section 8.6.4).

» Provide controlled vehicular access into the facility for trucks and front-end loaders.

8.4.1.3 Repair of Eroded Slopes: Immediate repair of eroded slopes can minimize the ultimate cost for this activity. Small areas can be repaired by hand with on-site materials. Large eroded areas are much more difficult and expensive to correct because they may require larger equipment and placement of imported material.

Design recommendations:

» Keep side slopes to minimum percentages to reduce likelihood of erosion.

» Provide vegetative or inert material cover on all slopes to minimize erosion.

» Adequately protect slopes subject to moving water or foot traffic. Make detailed evaluation of anticipated conditions and design protection accordingly. Use collector ditches for on-site drainage at the top of slopes.

8.4.1.4 Weed Control: Weed growth can adversely affect the use, appearance, and hydraulic characteristics of a basin. Therefore, weed growth shall be controlled. Extensive use of herbicides in basins where the primary or secondary purpose is groundwater recharge is not acceptable.

Design recommendations:

» Plant or seed all non-paved areas in and around the basin to establish a vegetation cover. Weed infestation is much less likely in areas which have a cover of desirable plants than on disturbed or untreated areas.

» Design basins to allow all areas, including slopes, to be accessible by equipment such as flail mowers which can cut or remove weed growth.

8.4.1.5 Maintenance of Low Flow Channels and Drainage Structures: In-basin drainage structures and facilities must be maintained to insure their proper operation. Design can influence maintenance requirements.

Design recommendations:

» Provide access to channels for front-end loaders and hauling equipment. Provide accessible areas, free of trees, to accommodate equipment movement.

- » Provide energy dissipators to prevent damage to the channel or drainage structures during high inflow conditions.
- » Design structures so that they will not collect debris which could impact proper operation.

8.4.1.6 Landscape Maintenance: Some degree of plant and landscape maintenance will be required even when native, drought-tolerant species are planted.

Design recommendations:

- » Select species with growth habits that minimize pruning and trimming or other maintenance requirements.
- » Specify and use the largest plants within budgetary constraints. This can minimize potential damage during initial growth seasons.
- » Space trees or plant masses for maintenance and equipment access.

8.4.1.7 Irrigation System Maintenance: Maintenance considerations of irrigation systems are critical, particularly when a permanent irrigation system is installed.

Design recommendations:

- » Specify and use equipment that will continue to operate when "contaminated" with sand or other soil deposition. For example, large sprinkler head orifices, verses drip emitters, are less likely to clog when lake or well water is used for irrigation.
- » Zone and layout system to avoid crossing channels where scour and erosion are likely to occur.
- » If required, increase depth of bury or encase pipelines in concrete (particularly mainlines) that cross channels that are likely to be eroded.
- » Install control equipment (other than remote control valves) in areas not subject to stormwater inundation.

8.4.1.8 Sign, Wall, and Fence Maintenance: For the protection of the public, informational signs and fences must be maintained and kept in good repair.

Design recommendations:

- » Use signs that are made of aluminum or other durable material that does not corrode or cannot be burned.
- » Secure signs to posts or standards with tamper-proof fasteners. Use posts or standards that will not be damaged by anticipated flooding or vandalism.
- » Locate fences away from areas likely to collect debris and act as dams to incoming water or water moving within the basin.
- » Design fences, gates, walls, etc., to minimize damage or accidental opening during normal area use or by flooding.

» In non-critical areas, design fences with an open or "clear-space" at grade to allow shallow water and debris to flow or blow under them.

» Design fences, such as backstops, with break-away or swing-away panels so flow is not impeded through the basin.

8.5 Multiple-use Concepts and Aesthetic Design Guidelines

A goal in Maricopa County is to design detention/retention facilities as amenities and, where possible, to incorporate multiple-use concepts. Flood control functions and other uses in detention/retention facilities are generally compatible. Rationale for multiple-use facilities includes decreased facility costs and an increased community acceptance. Combining flood storage with recreation uses or other community facilities on a single site decreases total costs for land acquisition and site development. The development of detention/retention facilities as parks or urban green space increases the acceptance by area residents and encourages better overall maintenance. If appropriately designed, use conflict is a minor concern.

The planning and development of facilities for multiple-use requires cooperation between the engineer, a qualified landscape architect, intergovernmental agencies, community organizations, park and recreation departments, and risk management agencies.

8.5.1 Potential Uses

Appropriate uses for detention/retention facilities in Maricopa County include active and passive recreation, urban green space, water amenities, water harvesting, and groundwater recharge. Use(s) in addition to flood control should address specific community needs and be clearly identified before the facility is designed.

8.5.1.1 Active Recreation: Active recreation includes a wide range of organized and unstructured activities that involve some type of physical movement. This type of recreational activity—both individual and group—generally requires larger areas than passive recreation uses. Because of their size, regional detention/retention facilities can provide more opportunities for group sports with large space requirements. Field sports (soccer, football, baseball) require areas with standardized dimensions.

8.5.1.2 Passive Recreation: Passive recreation generally involves individuals or small groups and a minimal amount of physical activity. Typically, passive recreation does not require large open spaces, and is, therefore, appropriate for both large and small detention/retention facilities.

8.5.1.3 Detention/Retention Facilities as Water Amenities: Facilities that incorporate a permanent pool can provide physical and psychological relief from the hot desert environment. The use of a permanent pool for detention/retention facilities is limited strictly to a visual amenity because body contact activities, such as swimming or wading, are specifically excluded.

8.5.1.4 Urban Green Space: Urban green space provides a visual resource within the community. As urbanization continues, the value of green space will increase. Green space provides visual breaks from the urban environment, acts as a filter to clean the air and can reduce erosion from wind and rain. Landscape materials in a detention/retention facility should respond to the recessed nature of the land form, the scale of the facility and the occurrence of frequent flooding.

The use of native and non-native, drought-tolerant species for landscape planting is highly recommended. The following basic zones should be considered in the landscape design for a detention/retention facility.

Channels: These are areas where there will be flowing water. Planting in these areas should be limited to grasses, groundcovers and low growing shrubs, with preference given to vegetation with flexible branching and resilient growth habits.

Basin Areas: There may be inundation and standing water in basin areas at some time during the year. Choice of plant materials should reflect these conditions. Trees, shrubs and grasses can be planted judiciously in these zones.

Elevated Areas: These areas may be occasionally inundated. The choice of plant material will depend on the use assigned to the area. Trees, shrubs and grasses can be planted and more easily maintained in areas of higher ground elevation.

8.5.1.5 Water Harvesting for Reuse or Recharge: A basic water harvesting system consists of three components: collection, storage and dispersion. Since stormwater detention/retention facilities will already be designed to collect and store runoff, some simple additions may allow harvesting the water for reuse. All applicable requirements of the Health Department and the Arizona Department of Water Resources must be met in addition to the normal review requirements.

When reusing stormwater for such things as on-site landscape irrigation, the facility must be lined by an impermeable membrane or by treating the soils to increase impermeability with native or imported clay or other measures. The local jurisdiction must be contacted regarding the acceptability of soil treatment measures in terms of the effect on water quality. Grading of the surrounding site should optimize runoff to the storage facility. An evaporation control mechanism may be appropriate for a surface storage system. Dispersion of water is typically achieved by pumping from the pond for irrigation.

A facility may be designed specifically to augment the groundwater aquifer. No formal dispersion is required other than methods to maximize the potential for water to percolate through the subsurface to the groundwater table. Thus, the facility should be designed to maximize the surface contact area between the stored water and the soil. Potential siltation problems must be addressed by providing a settling basin at the inlet or by other suitable measures.

Runoff water stored for recharge or reuse purposes does not contribute to detention/retention requirements. Adequate storage for detention/retention must be provided at all times, in addition to the volume provided for harvesting water.

8.6 Water Quality

8.6.1 Introduction

Urban runoff is distinguished from undeveloped area runoff in two principal ways: it occurs at greater discharge rates and volumes, and it contains varying but commonly higher concentrations of toxic substances, bacteria, and dissolved organic matter. Detention/retention facilities can play a significant role in mitigating the pollution problems associated with urban runoff.

8.6.2 Major Pollutants and Their Sources

Major pollutants associated with urban runoff include the following:

Sediment: Construction activities associated with urbanization and poor agricultural practices result in erosion and sedimentation.

Suspended Materials: Particulate matter and floating material, such as oils and scum, are included as suspended material. Suspended solid concentration in urban runoff may be 2 to 3 times that found in domestic waste.

Oxygen Demanding Materials: These include degradable organic matter and certain nitrogen compounds that consume the available dissolved oxygen as they degrade. The biochemical oxygen demand of stormwater runoff is usually in the 20 to 30 mg/l range, almost the same range as sewage effluent after secondary treatment.

Pathogenic Bacteria and Viruses: These include coliform, fecal coliform, and fecal streptococci, the same pathogenic bacteria and viruses found in domestic sewage.

Toxic Substances: These include heavy metals and a full range of EPA designated pollutants. The EPA list contains approximately 100 primarily organic substances, such as TCE.

Studies show that the areas contributing the greatest amounts of pollution are those with highly erodible surface conditions, such as plowed land or construction sites, or those areas characterized by highly impermeable surfaces, such as shopping malls, industrial areas and large housing complexes. Runoff from vehicular right-of-way (which accounts for over 20 percent of some urban lands), will contain hydrocarbons, other organics, and a diminishing—but still significant—amount of lead. Fertilizers and pesticides are transported by runoff from residential and agricultural areas.

8.6.3 Role of Detention/Retention Facilities in Water Quality Control

Most pollutants of concern have a high affinity for suspended solids in runoff and for soil particles. Thus, the most logical way to achieve pollutant removal is through sedimentation and infiltration. Consequently, detention/retention facility design for water quality control should maximize settling to the extent possible. This consideration may alter typical design features. In general, quiescent conditions and infiltration should be maximized while short-circuiting should be minimized. Design techniques that will accomplish these objectives are:

» Using long, narrow basin configurations, i.e., width to length ratios of 2:1 to 3:1, with the length measured along a line between the inlet and outlet.

» Installing inlet and outlet structures at extreme ends of the basin.

» Using baffles or flow retarders.

» Constructing ponds in two stages.

» Using riser outflow structures instead of ground level pipes to maintain a slow-draining pool encouraging infiltration.

» Developing a grass cover for the basin floor.

» Using underground tile drains for outlet discharge to provide soil filtration of the runoff.

Using wet rather than dry ponds will generally improve quiescent conditions, maximize infiltration, and provide a degree of biological treatment.

8.6.4 Method for Control of Sedimentation

Sediment removal within a detention/retention facility may be facilitated by the use of a "sediment trap" at the inlet, which will concentrate the majority of the incoming sediment bed load to a small portion of the facility. Sediment traps should be provided in conjunction with all detention/retention facilities which are intended as multi-use facilities. Figure 8.10 is a conceptual sketch of a typical detention basin sediment trap. The following list provides guidelines for the design of efficient sediment traps.

1. An additional sedimentation volume should be provided within the sediment trap at an elevation below the invert of the inflow channel.

2. The length/width ratio of the sediment trap should be a minimum of 2:1, with the length measured along a line between the inlet and outlet.

3. The basin shape should be wedge-shaped, with the narrow end located at the inlet to the basin (see Figure 8.10).

4. Provisions for total drainage and accumulated sediment removal of the sediment trap must be provided. Maintenance access should also be provided and designed to accommodate heavy trucks and other equipment necessary for removal of accumulated sediment.

8.7 Flood Routing

8.7.1 Flood Routing through Detention Facilities by the Storage-Indication Method (SCS National Engineering Handbook, Section 4)

Characteristically, the storage of a reservoir is closely related to its outflow rate. In reservoir routing methods, the storage-discharge relation is used for repeatedly solving the continuity equation; each solution is a step delineating the outflow hydrograph. Numerous computer software programs (such as HEC-1) have been

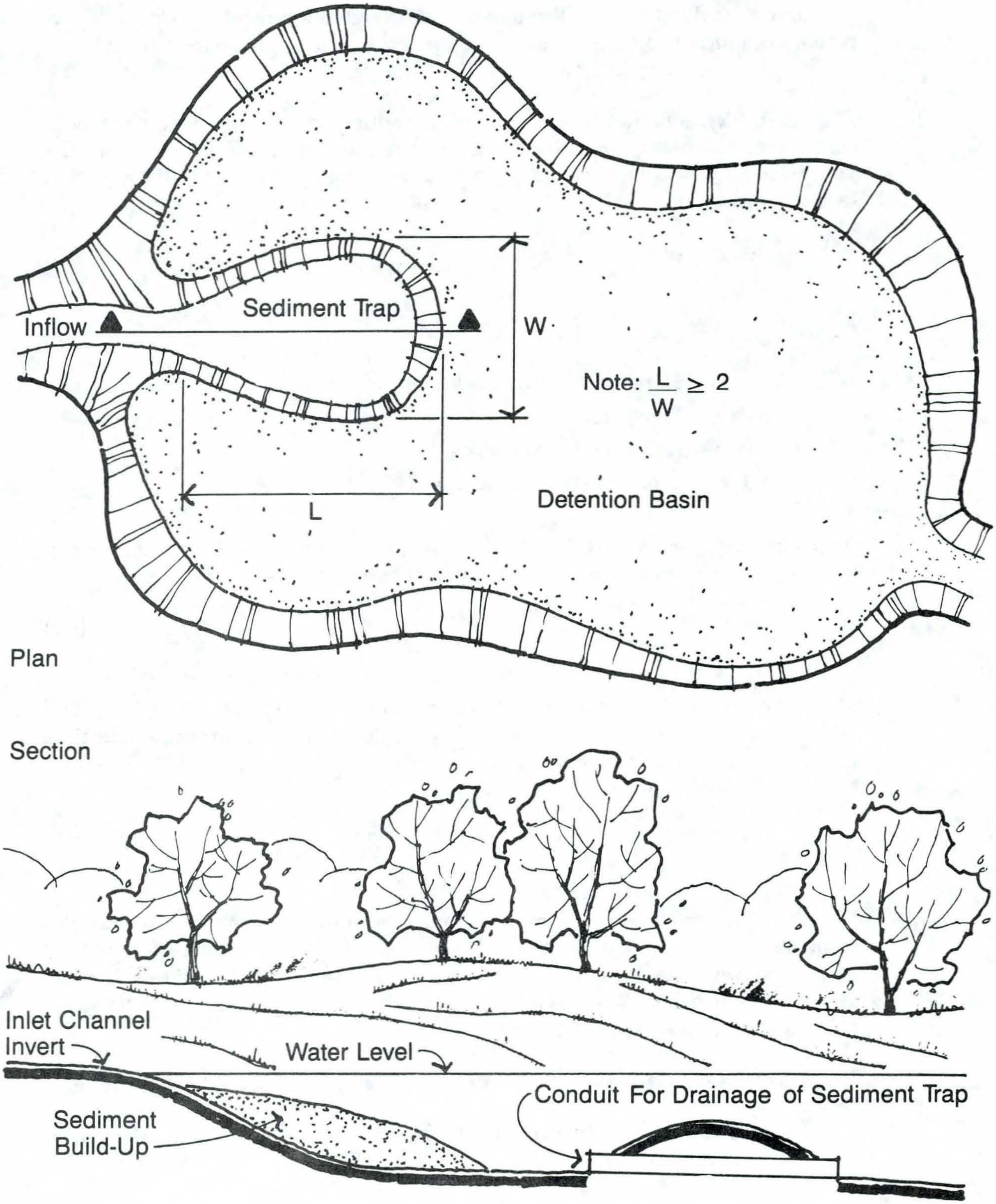


Figure 8.10
Sediment Trap Concept
(from: *Stormwater Detention/Retention Manual*, Pima County Department of Transportation and Flood Control District, City of Tucson, Simons Li and Associates, Undated.)

developed for flood routing through detention/retention facilities. Use of a particular computer program should be approved by the appropriate jurisdictional agency prior to its application on a particular project.

The continuity equation used in reservoir routing methods is concerned with conservation of mass: for a given time interval, the volume of inflow minus the volume of outflow equals the change in volume of storage. The continuity equation for a reservoir is:

$$\frac{dS}{dt} = I(t) - O(t) \tag{8.1}$$

where:

S	=	Storage volume in the system, cfs-hrs or ft^3
t	=	Time, hrs
I(t)	=	Inflow to the system, cfs
O(t)	=	Outflow from the system, cfs

If the time is broken into intervals of duration Δt and indexed by j, Equation 8.1 can be rewritten for the change in storage over the interval:

$$S_{j+1} - S_j = \frac{I_j + I_{j+1}}{2}\Delta t - \frac{O_j + O_{j+1}}{2}\Delta t \tag{8.2}$$

The values of I_j and I_{j+1} are obtained from an inflow hydrograph. The values of Oj and S_j are obtained at the jth time interval from calculation during the previous time interval. Equation 8.3 results from multiplying Equation 8.2 through by $2/\Delta t$ and by isolating the unkowns O_{j+1} and S_{j+1}:

$$\left(\frac{2S_{j+1}}{\Delta t} + O_{j+1}\right) = (I_j + I_{j+1}) + \left(\frac{2S_j}{\Delta t} - O_j\right) \tag{8.3}$$

Equation 8.3 can be used to facilitate the storage-outflow function solution in tabular form (see sample problem, Section 8.7.2). In order to calculate the outflow (Oj+1) from Equation 8.3, a storage outflow function relating $2S/\Delta t + O$ and O is needed. The method for developing this relationship using elevation-storage and elevation-outflow data is shown in Figure 8.11

The following steps are used in the Storage-Indication Method of flood-routing:

1. Develop the inflow hydrograph (refer to the *Drainage Design Manual for Maricopa County, Volume I, Hydrology*, FCD 1989).

2. Develop a elevation-storage relationship (Figure 8.11a) for the structure. The storage will normally be developed in acre-feet which will then be converted to cubic feet in the working table (see Table 8.3, page 399).

3. Develop a elevation-discharge relationship (Figure 8.11b) for the structure from hydraulic equations relating head and discharge for various types of spillways and outlet works. Table 8.2 includes equations that can be used. For a discussion on the values of C, Co, and Cd, see *Design of Small Dams* (USBR 1987).

4. Select the routing interval, Δt. The shorter the interval selected, the more precise the results will be.

5. Using the results of steps 2 and 3, make a four-column table with the following headings: (1) Elevation H, ft; (2) Storage S, ft^3; (3) Discharge O, cfs; and (4) $2S/\Delta t + O$. For an example, see Table 8.5, page 400.

6. Plot the value of $2S/\Delta t + O$ on the horizontal axis of a graph with the value of the outflow, O, on the vertical axis (see Figure 8.11c). Figure 8.12, page 400, represents such a graph.

7. Compute the value of $2S_{j+1}/\Delta t + O_{j+1}$ using Equation 8.3. All of the terms on the right side of Equation 8.3 are known for time interval j. Obtain the values of I_j and I_{j+1} from the inflow hydrograph (Step 1), as is done in Table 8.6.

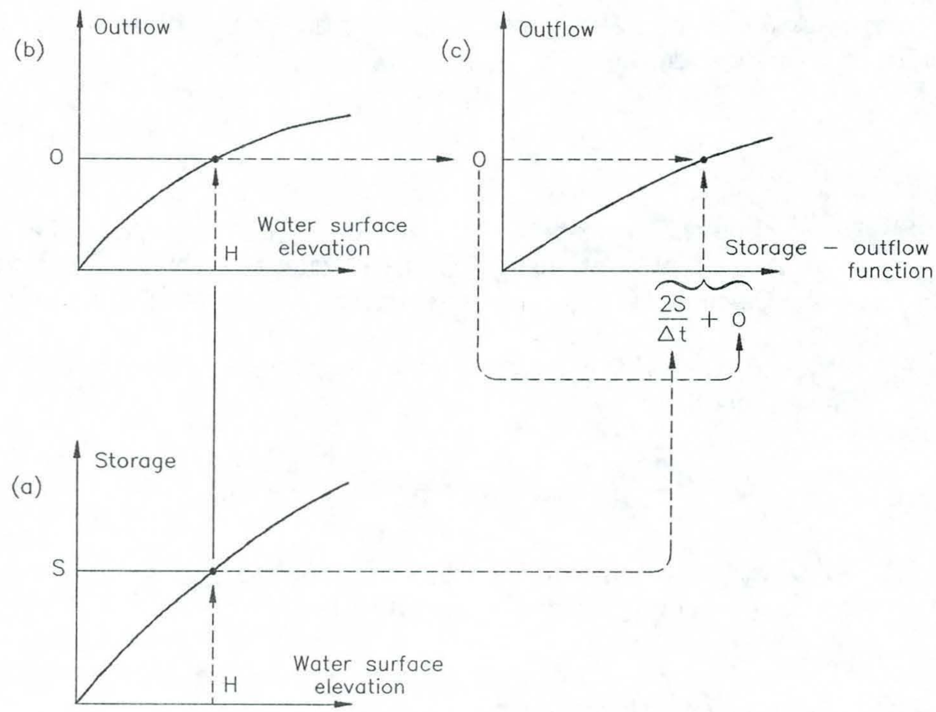
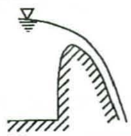
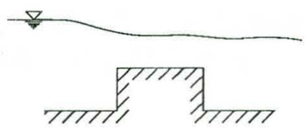


Figure 8.11

Development of the Storage-Outflow Function for Level Pool Routing on the Basis of Storage-Elevation and Elevation-Outflow Curves

(from: Applied Hydrology, Chow et al 1988)

Table 8.2
Spillway Discharge Equations
(from *Applied Hydrology*, Chow et al 1988)

Spillway Type	Equation	Notation
Uncontrolled overflow ogee crest 	$O = CLH^{3/2}$	O = Discharge, cfs C = Discharge coefficient[1] L = Effective length of crest H = Total head on the crest including velocity of approach head
Broad-crested weir 	$O = C_oLH^{3/2}$	C_o = Discharge coefficient[2] L = Effective length of crest H = Total head on the crest including velocity of approach head
Culvert (submerged inlet control)	$O = C_dA\sqrt{2gH}$	A = Cross-sectional area C_d = Discharge coefficient[3]

Source: *Handbook of Hydraulics*, 6th Edition, Brater and King 1982.

(1) C value for ogee crest varies from 3.1 to 3.9 depending upon the head, the depth of approach, the slope of upstream face, and the configuration of the downstream apron.

(2) C_o value for broad-crested weir varies from 2.3 to 3.3 depending upon the head, the breadth of weir crest, and the shape of the upstream corner.

(3) C_d value for culvert varies from 0.2 to 0.9 depending upon the head, pipe size, pipe length, material of the pipe, and the shape of the inlet edge.

8. Determine the corresponding value of O_{j+1} to $2S_{j+1}/\Delta t + O_{j+1}$ from the storage-outflow relationship $2S/\Delta t + O$ versus O. This can be done by either using the plot of step 6 or by linear interpolation of tabular values from step 5.

9. Calculate the value of $2S_{j+1}/\Delta t - O_{j+1}$ to set up the data required for the next time interval by:

$$\left(\frac{2S_{j+1}}{\Delta t} - O_{j+1}\right) = \left(\frac{2S_{j+1}}{\Delta t} + O_{j+1}\right) - 2O_{j+1} \tag{8.4}$$

10. Repeat the computation for subsequent routing periods and plot the inflow and outflow hydrographs. See Figure 8.13.

Steps 7, 8, and 9 are demonstrated in Table 8.6. With the exception of Step 8, all of these steps can be easily performed by using a spreadsheet.

8.7.2 Flood Routing Sample Problem

A detention basin is proposed. Determine the peak outflow discharge and the peak water surface elevation in the basin.

8.7.2.1 Given:

1. Inflow Hydrograph (plotted on Figure 8.13, page 402).

Time, hrs	Inflow, cfs	Time, hrs	Inflow, cfs
0.00	0	2.00	250
0.25	10	2.25	160
0.50	25	2.50	110
0.75	50	2.75	70
1.00	100	3.00	40
1.25	220	3.25	20
1.50	610	3.50	10
1.75	450	3.75	0

2. Stage-Surface Area Relationship

Elevation, ft	Surface Area, Acres
1100	4.6
1100.5	4.8
1101	5.2
1101.5	5.4
1102	5.6
1103	5.8
1104	6.2
1105	6.6
1106	7.0
1107	7.5
1108	7.8

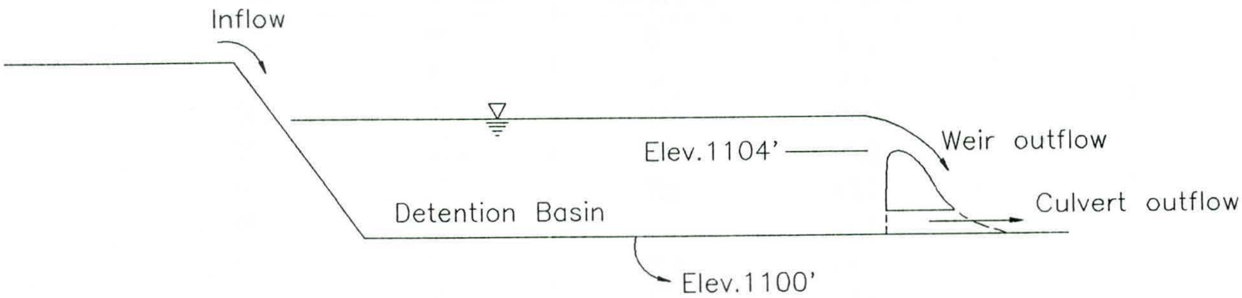
3. Outflow Structures

   a. Principal Spillway, ogee crest

      Discharge coefficient = 3.5
      Width of weir = 20 ft
      Elevation of weir crest = 1104 ft

b. Low Flow Structure, culvert (corrugated metal pipe)

Discharge coefficient = 0.5
Diameter = 15 inches = 1.25 ft
Elevation of culvert inlet = 1100 ft
Elevation of culvert center = 1100.63 ft (used to determine H_2)



An inflow hydrograph for this sample problem is given, so the solution procedure begins with Step 2. However, for a practical problem, the infow hydrograph needs to be developed for Step 1.

8.7.2.2 Solution:

Step 2. Develop a Water Surface Elevation-Storage Relationship, as in Table 8.3.

Step 3. Develop a Water Surface Elevation-Discharge Relationship, as in Table 8.4

Weir Flow $O_1 = C_1 L H_1^{3/2} = 3.5 \times 20 \times H_1^{3/2}$

Culvert Flow $O_2 = C_2 A \sqrt{2gH_2} = 0.5 \times \frac{\pi \times 1.25^2}{4} \times \sqrt{2 \times 32.2 \times H_2}$

Total Outflow $O = O_1 + O_2$

Step 4. $\Delta t = 0.25$ hrs = 900 seconds.

Step 5. Develop a Storage-Outflow Relationship for a detention reservoir, as in Table 8.5.

Step 6. Plot the value of $2S/\Delta t + O$ on the horizontal axis of a graph with the value of the outflow O on the vertical axis. See Figure 8.12.

Table 8.3
Water Surface Elevation-Storage Relationship

Elevation	Surface Area, Acres	Average Surface Area, Acres	Difference in Elevation, ft	Interval Storage, Acre-ft	Storage, Acre-ft	Storage, ft^3*
1100.0	4.6				0	0
		4.7	0.5	2.35		
1100.5	4.8				2.35	102,370
		5.0	0.5	2.5		
1101.0	5.2				4.85	211,270
		5.3	0.5	2.65		
1101.5	5.4				7.5	326,700
		5.5	0.5	2.75		
1102.0	5.6				10.25	446,490
		5.7	1.0	5.7		
1103.0	5.8				15.95	694,780
		6.0	1.0	6.0		
1104.0	6.2				21.95	956,140
		6.4	1.0	6.4		
1105.0	6.6				28.35	1,234,930
		6.8	1.0	6.8		
1106.0	7.0				35.15	1,531,130
		7.25	1.0	7.25		
1107.0	7.5				42.4	1,846,940
		7.65	1.0	7.65		
1108.0	7.8				50.05	2,180,180

*1 Acre-ft = 43,560 ft^3

Table 8.4
Water Surface Elevation-Discharge Relationship

Water Surface Elevation	Weir		Culvert		Discharge, cfs
	H_1	O_1	H_2	O_2	
1100.0	0	0	0	0	0
1100.5	0	0	0	0	0
1101.0	0	0	0.37	3	3
1101.5	0	0	0.87	5	5
1102.0	0	0	1.37	6	6
1103.0	0	0	2.37	8	8
1104.0	0	0	3.37	9	9
1105.0	1	70	4.37	10	80
1106.0	2	198	5.37	11	209
1107.0	3	364	6.37	12	376
1108.0	4	560	7.37	13	573

Table 8.5
Storage-Outflow Relationship for a Detention Reservoir

Elevation H, ft	Storage S, ft^3	Discharge O, cfs	$(2S/\Delta t) + O$, cfs
1100.0	0	0	0
1100.5	102,370	0	227
1101.0	211,270	3	472
1101.5	326,700	5	731
1102.0	446,490	6	998
1103.0	694,780	8	1,552
1104.0	956,140	9	2,134
1105.0	1,234,930	80	2,824
1106.0	1,531,130	209	3,612
1107.0	1,846,940	376	4,480
1108.0	2,180,180	573	5,418

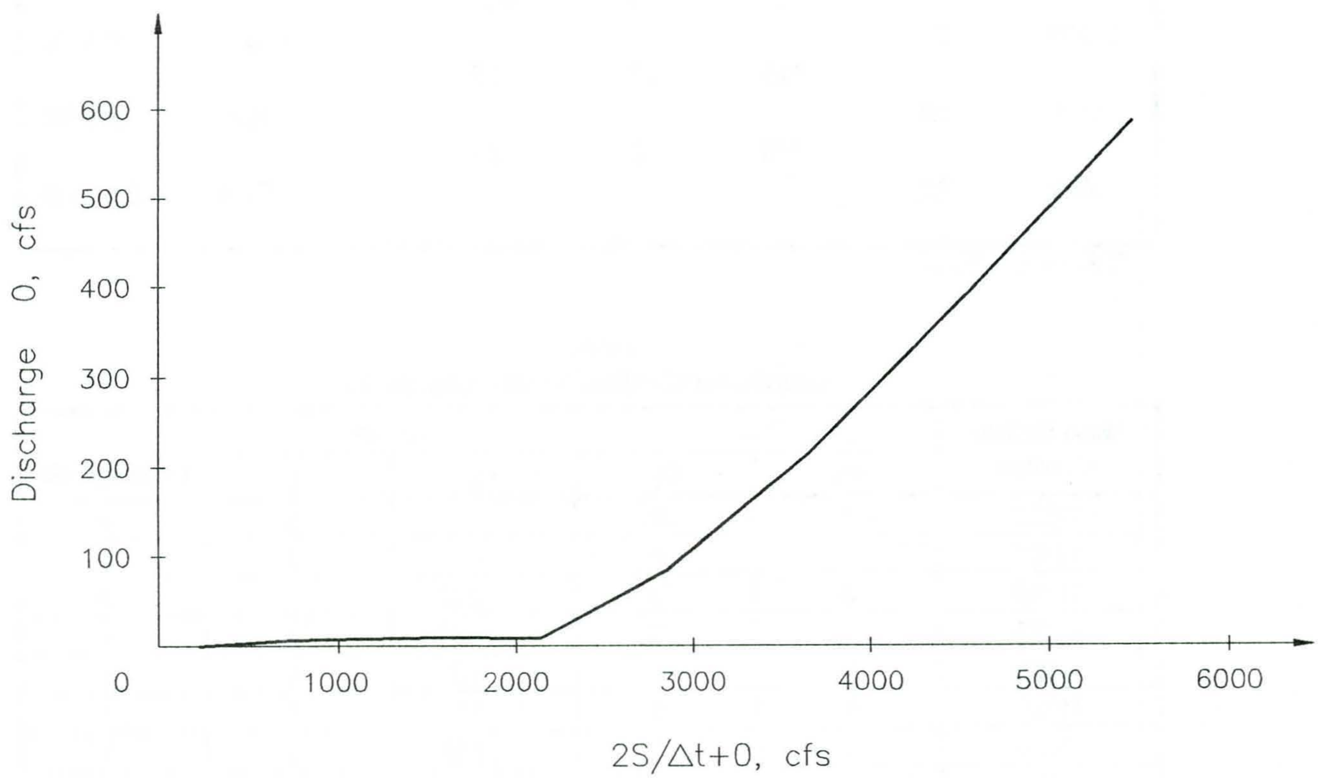


Figure 8.12
Storage-Outflow Function for Sample Problem

Steps 7 through 9:

Table 8.6
Routing of Flow through Detention Basin

Time, hrs	Inflow, cfs	$I_j + I_{j+1}$, cfs	$\frac{2S_j}{\Delta t} - O_j$, cfs	$\frac{2S_{j+1}}{\Delta t} - O_{j+1}$, cfs	Outflow, cfs
0.00	0		0	Step 7	Step 8 0
0.25	10	10	10	10	-2x 0
0.50	25	35	45	45	Step 9 0
0.75	50	75	120	120	0
1.00	100	150	268	270	1
1.25	220	320	580	588	4
1.50	610	830	1,394	1,410	8
1.75	450	1,060	2,370	2,454	42
2.00	250	700	2,830	3,070	120
2.25	160	410	2,942	3,240	149
2.50	110	270	2,928	3,212	142
2.75	70	180	2,858	3,108	125
3.00	40	110	2,762	2,968	103
3.25	20	60	2,662	2,822	80
3.50	10	30	2,556	2,692	68
3.75	0	10	2,450	2,566	53
4.00		0	2,368	2,450	41
4.25			2,304	2,368	32
4.50				2,304	26

Step 10. Plot the inflow and outflow hydrographs.

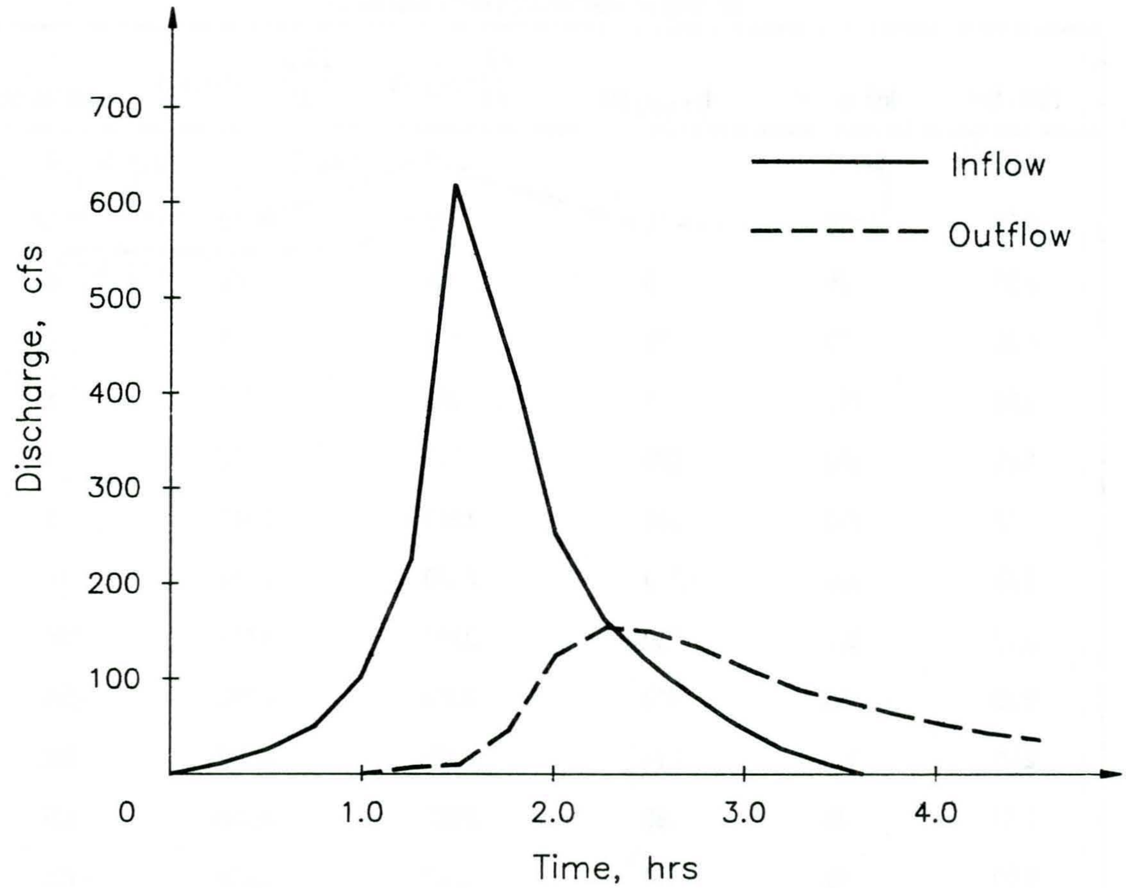


Figure 8.13
Comparison of Inflow and Outflow Hydrographs

Peak Outflow Discharge: $O_{peak} = 149$ cfs (from Table 8.6).

Peak Water Surface Elevation: By linear interpolation of values in Table 8.5 and using O_{peak},

$$H_{peak} = 1105 + \frac{(149-80)}{(209-80)} \times (1106-1105) = 1105.53 \text{ ft}$$

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9 Pump Stations

9.1 Introduction

Stormwater pump stations are used where gravity discharge is not feasible or for metering flow out of detention/retention facilities. When used independently of a detention/retention facility, storage should be incorporated into pump station facilities to reduce pump cycling and hence the initial capital and long-term operational costs of the pump station.

The actual design of pump stations involves several technical disciplines, and the approach to the design is often dependent on the size of the facility and the consequences of any type of system failure. A pumping facility failure serving a major interchange and adjacent major development causing millions of dollars of damages demands greater reliability than a small pumping facility that drains a retention pond with a 36-hour disposal time and overflows result in small increases in water depth of an adjacent street. This chapter provides only an overview of the conditions that should be considered in the design of stormwater pumping facilities.

Stormwater pump stations may be either dry pit or wet pit facilities. In the latter type of facility, the pumps are submersible and are located in a wet well. In the former type, centrifugal pumps are located in a dry pump room and generally use a wet well to modulate the incoming flows (a form of storage). For small pump stations, the pump may be located in an inlet or a manhole-type wet well.

For a rigorous discussion of the design of stormwater pump stations, refer to *Manual for Highway Stormwater Pumping Stations* (FHWA 1982).

9.2 Design Approach

The design approach addresses two conditions, the criteria that are to be applied and a check list of conditions that should be considered in the design of pumping facilities (see Table 9.1).

9.2.1 Criteria

The use of pumping facilities for stormwater is discouraged and will be considered only on express agreement by the jurisdictional agency. Unless excepted by the jurisdictional agency, the following criteria should be applied to the design of stormwater pump stations:

1. Pumping facilities (excluding components whose design requires submersion) will be set at an elevation at or above the anticipated level of the 100-year event, considering that a total power failure may occur.

2. Pumps shall be capable of handling solids to a minimum of 3-inches. Consideration for handling smaller solids can be made for pumping facilities that serve storage facilities.

3. Screening devices will not be used at the entrances to the pump stations. Grates will be used on each catch basin.

4. Required calculations include: a) total dynamic head; b) net positive suction head; c) head capacity curves (parallel operation); and d) mass flow curves.

5. Controls will provide for automatic and manual operations and will have communications to permit transmission of failure signals to designated reporting locations.

6. A potable water supply with back-flow prevention and hose bibs should be provided to aid in removal of silt and trash.

7. A ventilation system will provide intermittent ventilation of wet-wells.

8. Plugging factors will be used on inlets of pipe systems that are tributary to pump stations.

9. Facilities not associated with retention facilities will provide storage to the maximum practical extent to aid in efficient operation of the system.

10. A redundant pumping system may be required, particularly at small installations.

11. The site layout shall consider adequate access for maintenance vehicles.

12. Generally, stormwater pump stations should not be privately maintained.

Table 9.1
Design Checklist for Pump Stations

Initial Data
» Contributing Drainage Basin
» Location of Outfall
» Capacity of Outfall
» Probable Growth in the Contributing Basin
» Inflow Hydrographs
Possible Components
» Source of Power
» Pumps
» Intakes and Catch Basins
» Controls
» Storage
» Debris Handling
» Potable Water Supply
» Testing
» Hoisting Equipment
» Ventilation
» Control of Hazardous Materials
Hydrology
Economic and Alternative Analysis
» Designation of Significantly Different Concepts
» Hydrologic and Hydraulic Detailing of Alternatives
» Cost Evaluation
» Extreme Event Evaluation of Components and Alternatives
» Environmental Considerations
» Documentation and Comprehensive Evaluation
Hydraulic Analysis
» Mass Curve Routing
» Pump Characteristics
» Pipe Losses
» Miscellaneous Losses
Sediment Transport
Additional Considerations

9.3 References

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Glossary

1/2 PMF: The flood hydrograph with ordinates equal to one-half the corresponding ordinates of the Probable Maximum Flood Hydrograph.

100-Year Flood: A flood stage or height that, statistically, has one percent chance of being equaled or exceeded in any given year. The 100-year flood is often referred to as the base flood.

Abutments: Walls supporting the end of a bridge or span, and sustaining the pressure of the abutting earth. In a drop structure, the walls which form the sides of the crest of the drop. In some structures, wingwalls (transition walls) extend upstream of the abutment walls to create a smooth transition from the upstream channel.

Aggradation: A progressive buildup or raising of the channel bed due to sediment deposition. Permanent or continuous aggradation is an indicator that a change in the stream's discharge and sediment load characteristics is taking place, see Degradation.

Alluvium: Unconsolidated material deposited by a stream in a channel, floodplain, alluvial fan, or delta.

Armor: Surfacing of channel bed, banks, or embankment slope to resist erosion.

Armoring: (a) Natural process whereby an erosion-resistant layer of relatively large particles is formed on a streambank due to the removal of finer particles by streamflow. (b) Placement of a covering on a streambank to prevent erosion.

Arterial Street System: The arterial system should carry a major portion of trips entering and leaving the urban area, as well as the majority of movements through the central city. Frequently, the arterial system will carry important intra-urban as well as intercity bus routes. Arterials are typically located on one-mile intervals on section lines.

Baffle Chute: A type of drop structure or outlet structure that incorporates baffles for energy dissipation.

Baffles: Deflector vanes, blocks, guides, grids, gratings or similar devices constructed to: 1) check or effect a more uniform distribution of velocities; 2) dissipate energy; 3) divert, guide, or agitate flow; and 4) check eddy currents.

Basin Area: The area which contributes stormwater to a concentration point such as a lake, stream, or drainage system.

Basin Floor: The bottom of a stormwater retention facility which has been specifically designed for the purpose of disposing stored runoff following a storm event by the process of infiltration into the subsurface.

Basin Sediment Yield: The total sediment outflow from a watershed or a drainage area at a point or reference and in a specified time period. This outflow is equal to the sediment discharge from the drainage area.

Bed Material: Material found on the bed of a stream (may be transported as bed load or in suspension).

Bed Sediment Discharge: The part of the total sediment discharge that is composed of grain sizes found in the bed and is equal to the transport capability of the flow.

Braided Stream: A stream whose flow is divided at normal stage by small mid-channel bars or small islands; the individual width of bars and islands is less than about three times the water width; a braided stream has the aspect of a single large channel within which are subordinate channels.

Bridge Low-chord: The elevation of the lowest portion of the bridge deck structure used in determining the area of the bridge opening available for flow conveyance.

Catch Basin: A chamber or well, usually built at the curb line of a street, for the admission of surface water to a storm sewer or sub-drain.

Channel Failure: Sudden collapse of a channel due to an unstable condition, such as the removal of a bank by scour.

Channel Reach: A segment of stream length that is arbitrarily bounded for purposes of study.

Channel Stabilization: Methods of achieving slope and cross-section which allow a channel to transport the water and sediment delivered from the upstream watershed without aggradation or streambank erosion.

Check Dam: A low dam or weir across a channel,for the diversion of irrigation. Also used herein for a low dam to control stream gradient, typically associated with small streams or the low flow channel of a floodplain or other channel.

Check Structure: A small drop structure constructed in the low flow portion of a channel for the purpose of controlling stream gradient.

Clear Zone: The roadside border area, starting at the edge of the traveled way, available for safe use by errant vehicles.

Clear-water Scour: Scour which occurs when there is no movement of the bed material of the stream upstream of the crossing, but occurs as a result of acceleration of the flow and vortices created by piers or abutments causing material at their base to move.

Collector Street System: Collector streets may penetrate neighborhoods and may carry a minor amount of through traffic.

Contraction Scour: General scour resulting from the acceleration of flow due to a natural channel constriction or bridge contraction.

Crest: That portion of the drop structure which controls the gradient of the upstream channel. In a vertical drop structure the crest is a wall typically constructed of reinforced concrete or sheet pile. In a sloping drop structure, the crest is the portion of the drop at the top of the slope and usually incorporates a buried cutoff wall for seepage control.

Critical Depth: The depth at which a given discharge flows in a given channel with a minimum specific energy. For depths greater and lower than critical, the flow is said to be subcritical and supercritical, respectively.

Critical Flow: Flow at critical depth.

Culvert: A hydraulically short conduit which conveys surface water runoff through a roadway embankment or through some other type of flow obstruction. Culverts are constructed from a variety of materials and are available in many different shapes and configurations. Culvert selection factors include roadway profiles, channel characteristics, flood damage evaluations, construction and maintenance costs, and estimates of service life.

Degradation: A progressive lowering of the channel bed due to scour. Permanent or continuing degradation is an indicator that a change in the stream's discharge and sediment load characteristics is taking place, see Aggradation.

Design Discharge: Maximum flow a structure or channel is expected to accommodate without contradicting the adopted design constraints.

Design Frequency: The nth-year storm for which it is expected that the structure or facility designed for that storm would experience an actual hydrological event of a given or greater magnitude, once, on average, in n years. For example, a 50-year storm has a 2 percent chance of occuring in any given year. Also called the return period, excedence interval, or recurrance interval.

Discharge: Volume of water passing through a channel during a given time.

Drainage Basin: A geographical area which contributes surface runoff to a particular concentration point. The terms "drainage basin", "tributary area" and "watershed" are used interchangeably.

Drainageway: A route or watercourse along which storm runoff moves, or may move, to drain a catchment area.

Drop Structure: A structure constructed in a conduit, canal, or open channel for the purpose of gradient (bottom slope) control.

Dry Well: An engineered subsurface chamber designed to accept surface runoff and allow it to drain into the subsurface strata.

Embankment: A man-made earth fill structure constructed for the purpose of impounding water.

Emergency Spillway: An outflow spillway from a stormwater detention/retention facility that provides for the safe overflow of floodwaters for storm events in excess of the design capacity of the Primary Outlet Structure, or in the event of malfunction or debris blockage of the Primary Outlet Structure.

Energy Grade Line (EGL): An inclined line representing the total energy of the flowing water. For an open channel, the EGL is above the **water surface** by a value of the velocity head. In a closed pressure conduit, the EGL is above the **pressure head line** by a value of the velocity head. See Hydraulic Grade Line and Figure 4.4.

Equilibrium: The state of balance of natural channels between hydraulic forces or actions. Equilibrium occurs when the streambed has achieved a graded condition when the slope and energy of the stream are just sufficient to transport material delivered to it. Natural channels which have small changes resulting from periods of low and high flows are considered in equilibrium.

Erosion: Displacement of soil particles on the land surface due to water or wind action.

Filter: Layer of fabric, sand, gravel, or graded rock placed (or developed naturally where suitable in-place materials exist), between the bank revetment and soil for one or more of three purposes: 1) to prevent the soil from moving through the revetment by piping, extrusion, or erosion; 2) to prevent the revetment from sinking into the soil; and 3) to permit natural seepage from the streambank, thus preventing buildup of excessive hydrostatic pressure.

Filter Blanket: A layer of graded, intermediate-size gravel placed between fine-grained material and riprap, to prevent wash-out of the finer material.

Filter Fabric: Fabric of synthetic strands that serves the same purpose as granular filter blanket.

Fine Sediment Load (or Washload): That part of the total sediment load that is composed of particle sizes finer than those represented in the bed. Normally, the fine-sediment load is finer than 0.062 mm for a sand-bed channel. Silt, clay, and sand could be considered fine sediment load in a coarse gravel and cobble bed channel. The washload generally comes from the watershed.

Flood Fringe: A regulatory district within the floodplain but outside the floodway district.

Flood Peak: The largest value of the runoff flow which occurs during a flood event, as observed at a particular point in the drainage basin.

Flood Routing: The mathematical simulation of a flood wave as it moves downstream along a watercourse or through a detention/retention facility.

Floodplain: A flood-prone area is identified on FEMA flood insurance rate maps generally containing a floodway fringe district and floodway district or areas of land adjoining or near the channel of a watercourse which have been, or may be, covered by floodwaters. A floodplain functions as a temporary channel or reservoir for overbank flows.

Floodway: A specific regulatory district within the floodplain as identified on FEMA flood hazard boundary maps; or the channel of a river or other watercourse and the adjacent land area necessary to discharge the 100-year flood without cumulatively increasing the water surface by more than one foot and without creating hazardous velocities of floodwaters. Normally used only when developing long reaches of floodplain mapping.

Freeboard: The vertical distance above a design water surface elevation that is provided as a contingency or allowance for waves, surges, water-borne debris or other factors.

Froude Number: A dimensionless number (expressed as $V/(gy)^{0.5}$) that represents the ratio of inertial to gravitational forces. High Froude numbers (values greater than 1) indicate supercritical flow with associated high velocity and scour potential.

Gabion or Wire-Enclosed Basket: A basket or compartmented rectangular container made of steel wire mesh. When filled with cobbles or rock of suitable size, the gabion becomes a flexible and permeable block with which flow-control structures can be built.

General Scour: Scour in a channel or on a floodplain that is not localized at a pier, abutment, or other obstruction to flow. In a channel, general scour usually affects all or most of the channel width.

Geomorphology: That branch of both physiography and geology that deals with the form of the earth, the general configuration of its surface, and the changes that take place due to erosion of the primary elements and in the buildup of erosional debris.

Grade Control Structure (sill, check dam): A structure across a stream channel placed bank to bank (usually with its central axis perpendicular to flow) to control bed slope and prevent scour or headcutting.

Gradient: The rate of change of a characteristic per unit of length. The term is usually applied to such things as channel/stream bed slope elevation, conduit invert elevation, velocity, pressure, etc.

Guide Bank: A dike extending upstream from the approach embankment at either or both sides of the bridge opening to direct the flow through the opening. Some guide banks extend downstream from the bridge.

Gunite: Term formerly used for dry-mix mortar shotcrete.

Headcutting: Channel bottom erosion moving upstream along a waterway indicating that a readjustment of the channel's slope and its discharge and sediment load characteristics is taking place. Headcutting is evidenced by the presence of abrupt vertical drops in the stream bottom or rapidly moving water through an otherwise placid stream. Headcutting often leaves stream banks in an unstable condition as it progresses along the channel.

Hydraulic Grade Line (HGL): For an open channel, it is coincident with the water surface. In a closed pressure conduit, it is the line representing the pressure head of the conduit. HGL will always be EGL minus the velocity head. See Energy Grade Line and Figure 4.4.

Hydraulic Jump: The hydraulic jump is an abrupt rise in the water surface which occurs in an open channel when water flowing at supercritical velocity is retarded by water flowing at subcritical velocity or a stationary pool. The transition through the jump results in a marked change in energy, evidenced by turbulence of the flow within the area of the jump. The hydraulic jump is often used as a means of energy dissipation.

Hydraulic Structures: The facilities used to impound, accommodate, convey or control the flow of water, such as dams, weirs, intakes, culverts, channels, and bridges.

Hydrograph: The functional relationship between time and flow discharge, as observed at a particular point within a drainage basin. In the case of a detention/retention facility, an Inflow Hydrograph depicts the relationship of time and runoff inflow to the facility, and an Outflow Hydrograph is a graph of flow discharge from the facility versus time.

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

Incised Stream: A stream that flows in an incised channel with high banks. Stream banks that stand more than 15 feet above the water surface at normal stage are regarded as high banks.

Infiltration: The movement of water into and through the soil.

Invert: The lowest point in the channel cross section or at flow control devices such as drop structures, dams, or outlet structures, see Thalweg.

Jurisdiction or Jurisdictional Agency: Maricopa County, the Flood Control District of Maricopa County, and the incorporated municipalities within Maricopa County.

Lateral Stream Migration: Change in position of a channel by lateral erosion of one bank and simultaneous accretion of the opposite bank. Movement in which the material has a dominate lateral component.

Launching: Release of undercut material (stone riprap, rubble, slag, etc.) downslope; if sufficient material accumulates on the streambank face, the slope can become effectively armored.

Live-bed Scour: Scour which occurs when the bed material upstream of the crossing is also moving.

Local Aggradation: Aggradation in a channel or on a floodplain that is localized at a pier, abutment, or other obstruction to flow.

Local Scour: Scour in a channel or on a floodplain that is localized at a pier, abutment, or other obstruction to flow.

Local Street System: The local street system comprises all facilities not on one of the higher systems. It offers the lowest level of mobility and usually contains no bus routes. Service to through traffic movement usually is deliberately discouraged.

Low Flow Channel: A channel within a larger channel which typically carries low and/or normal flows.

Major drains: Include natural and man-made channels and conduits that serve watershed areas from 160 acres to about 10 square miles.

Master Planning: A "systems" approach to the planning of facilities, programs and management organizations for comprehensive control and use of stormwater within a defined geographical area or drainage basin.

Meandering Channel: A channel exhibiting a characteristic process of bank erosion and point bar deposition associated with systematically shifting meanders.

Median Diameter: The particle diameter at the 50 percentile point on a size distribution curve such that half of the particles (by weight for samples of sand, silt or clay and by actual measurement for samples of gravel and riprap) are larger and half are smaller. The median diameter is denoted D_{50}.

Minor drains: Serve watershed areas up to 160 acres and are normally the drains associated with subdivision development.

Multi-purpose Facility: A detention or retention facility that provides benefits in addition to the primary function of flood control. Such benefits may include recreation, parking, visual buffers or water harvesting.

Nappe: The sheet or curtain of water overflowing a weir or dam. When freely overflowing the crest of a structure, it usually has a well-defined upper and lower surface.

Off-stream Detention/Retention Facility: A facility that is located near or adjacent to a watercourse (i.e., the stream does not flow directly into the facility). Inflow to the facility is typically accomplished by means of side weirs. It is also referred to as an Off-line Detention/Retention Facility.

On-site Detention/Retention: The temporary storage of excess storm runoff in the upper area of a drainage basin. This type of facility is typically or within a subdivision, primarily by an individual development and generally irrespective of watershed features.

On-stream Detention/Retention Facility: A facility that is located within the path of a stream or watercourse, and thereby intercepts the entire flow from the upstream drainage basin. It is also referred to as an On-line Detention/Retention Facility.

Orifice: A hole in the outlet structure of a stormwater storage facility sized to drain the facility at a specific rate of flow.

Outlet Structure: A hydraulic structure placed at the outlet of a conduit, open channel, spillway, etc., for the purpose of dissipating energy and providing a transition to the channel or conduit downstream. Outlet structures may consist of culverts, weirs, orifices (gated or un-gated), dry wells, or any combination thereof.

Plunge Pool: An energy dissipation device placed downstream of a conduit, channel or vertical wall drop structure. The plunge pool basin is typically lined with rock riprap, concrete or other protective covering and dissipates the energy of free falling water through impact and turbulence.

Pressure Head: In a closed pressure conduit, it represents the energy per unit weight stored in the fluid by virtue of the fluid being under pressure expressed as P/γ. Generally having the units of feet. In an open channel, the pressure head is everywhere zero.

Primary Outlet Structure: Also known as the Primary Spillway or Principal Spillway, it is the main outlet structure by which stormwater is discharged from the detention/retention facility.

Probable Maximum Flood (PMF): The flood runoff that may be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in the region.

Pump Station: A facility housing stormwater pumps, controls, power plants and their appurtenances.

Regional Detention/Retention: The temporary storage of excess runoff by means of large storage facilities located at strategic sites within a drainage basin. Sites are generally planned to provide control of excess runoff from an entire drainage basin with an optimum (presumably a minimum) number of storage facilities to achieve the most cost-effective drainage system. Regional detention/retention sites are normally maintained by a public or quasi-public agency.

Regional drains: The main outfalls for drainage. They serve watershed areas generally greater than 10 square miles, and include rivers and washes.

Residual Freeboard: For an embankment dam, the vertical distance between the maximum water surface elevation and the minimum dam crest elevation.

Retention Basin: A basin or reservoir wherein water is stored for regulating a flood, however, it does not have gravity-flow outlets for outflows during floods as detention basins do. The stored water must be disposed by some other means such as by infiltration into soil, evaporation, injection (or dry) wells, or pumping systems.

Reverse Filter Drain: An engineered granular filter placed at weep hole locations on hydraulic structures to collect and direct groundwater to the weep holes to relieve uplift pressures and other adverse effects of uncontrolled seepage water.

Riprap Toe Protection: In the restricted sense, layer or facing or broken rock or concrete dumped or placed at the toe of a channel to protect a structure or embankment from erosion; also the broken rock or concrete suitable for such use. Riprap has also been applied to almost all kinds of armor, including wire-enclosed riprap, grouted riprap, sacked concrete, and concrete slabs.

Runoff: The portion of precipitation on land that ultimately reaches streams; especially water from rain or melted snow that flows over the ground surface.

Scour: Erosion due to flowing water, usually considered as being localized as opposed to general bed degradation.

Sediment (or Fluvial Sediment): Fragmental material transported, suspended, or deposited by water.

Sediment Discharge: The quantity of sediment that is carried past any cross section of a stream in a unit of time. Discharge may be limited to certain sizes of sediment or to a specific part of the cross section.

Sediment Trap: An area within a stormwater detention/retention facility which is designed to trap the majority of incoming sediments for the purpose of facilitating maintenance.

Seepage: The movement of water through pores and voids of pervious material such as soil, gravel, synthetic filter media, etc.

Seepage Cutoff Wall: An impervious subsurface barrier constructed of clay, concrete or synthetic material for the purpose of increasing the length of travel of subsurface water flow and thereby reducing and/or controlling the action of such flows (for example, uplift forces) at hydraulic structures.

Shotcrete: Mortar or concrete pneumatically projected at high velocities onto a surface.

Sill: A raised edge at the downstream end of a stilling basin. The sill typically has a notch or opening to allow normal stream flows to pass through and/or to allow the basin to drain completely following a storm.

Slope Paving: Covering of a channel bank or bed with stones or concrete.

Soil-Cement: A designed mixture of soil and portland cement compacted at a proper water content to form a veneer or structure which when placed on a streambed or bank can prevent erosion. Also referred to as Cement Stabilized Alluvium.

Spillthrough Abutment: A bridge abutment having a fill slope on the streamward side.

Spillway: (a) A low-level passage serving a dam or reservoir through which surplus water may be discharged; usually an open ditch around the end of a dam, or a gateway or a pipe in a dam. (b) An outlet pipe, flume, or channel serving to discharge water from a ditch, ditch check, gutter or embankment protector.

Stage: The depth of water within a stormwater storage facility, as measured above an established datum.

Storage Reservoir of Pump Station: A reservoir wherein peak flows from storm drains are stored for reducing capacity requirements of the pump station to pump runoff to an appropriate outlet.

Storm Drainage System: A drainage system for collecting runoff of stormwater on highways and removing it to appropriate outlets. The system includes inlets, catch basins, storm sewers, main drains, storage reservoirs, detention basins and pump stations.

Stormwater Detention Facility: A stormwater storage facility which temporarily stores surface runoff and releases it at a controlled rate through a positive outlet.

Stormwater Retention Facility: A stormwater storage facility which stores surface runoff. Stored water is infiltrated into the subsurface or released to the downstream drainage system or watercourse (via a gravity outlet or pump) after the storm event.

Streambank Erosion: Removal of soil particles from a bank surface due primarily to water action. Other factors such as weathering, ice and debris abrasion, chemical reactions, and land use changes may also directly or indirectly lead to streambank erosion.

Streambank Protection: Any technique used to prevent erosion or failure of a streambank.

Subdrain: An underground conduit, usually perforated and surrounded by an engineered granular filter material that is designed to permit infiltration for the purpose of collecting and conveying groundwater.

Subgrade Erosion: Erosion of the material underlying that portion of the stream bed which is subject to direct action of the flow.

Subsurface Disposal: Drainage of stormwater runoff into the subsurface by the process of infiltration. This is typically accomplished through the use of dry wells, engineered basin floors, etc.

Tailwater: The water surface elevation in the channel downstream of a hydraulic structure.

Thalweg: The line extending down a channel that follows the lowest elevation of the bed. Not to be confused with the channels's centerline, see Invert.

Total Freeboard: For an embankment dam, the vertical distance between the emergency spillway crest and the minimum crest elevation of the dam.

Total Sediment Discharge: The sum of suspended sediment discharge and bedload discharge or the sum of bed material discharge and washload discharge of a stream.

Transport Rate: Rate at which sediment particles are carried when hydraulic conditions exceed the critical condition for motion. Transport rates are calculated analytically by the use of transport functions.

Trash Rack: A metal bar or grate structure located at the outlet structure of a stormwater detention/retention facility and designed to prevent blockage of the outlet structure by water-borne debris.

Trickle Channel: Also called the low flow channel, the trickle channel is that portion of a major channel which is sized to carry the normal low flow.

Underdrain: See Subdrain.

Uniform Flow: Flow of constant water area, depth, discharge, and average velocity through a reach of a channel.

Uplift Pressure: Pressure caused by uncontrolled seepage or groundwater flow beneath a structural slab which can lead to cracking and displacement of the structure.

Velocity Head: Represents the kinetic energy of the flowing fluid generally expressed as $V^2/2g$ in feet, but actually is the energy per pound of flowing fluid.

Vortex: Local current accelerations which cause a whirling or circular motion that tends to form a cavity or vacuum at its center, thus moving sediment toward the cavity.

Watershed: An area confined by drainage divides, often having only one outlet for discharge.

Weep Hole: Openings in an impermeable wall or revetment to relieve the neutral stress or pore water pressure. Weep holes are typically combined with reverse filter drains to form a total system for seepage control.

Weir: A notch of regular form through which water flows. A weir may be a depression or notch in the side of an outlet structure or a depression of specific shape in the embankment of a stormwater storage facility. Classified in accordance with the shape of the notch, there are rectangular weirs, V-notch weirs, trapezoidal weirs and parabolic weirs.