THE CITY OF SAN DIEGO

Transportation & Storm Water Design Manuals

Drainage Design Manual

January 2017 Edition

The City of SAN DIEGO

THIS PAGE INTENTIONALLY LEFT BLANK FOR DOUBLE-SIDED PRINTING

Contents

APPENDICES

Figures

Tables

Equations

THIS PAGE INTENTIONALLY LEFT BLANK FOR DOUBLE-SIDED PRINTING

Chapter 1

Introduction

This Drainage Design Manual (Manual) provides policies and procedures to attain reasonable standardization of drainage design throughout the City of San Diego (City).

This Manual establishes design standards and design procedures for storm water conveyance and hydrology analysis for flood management and water quality facilities in the City. These design standards and procedures provide guidance to design engineers, developers, contractors, and others in the selection, design, construction, and maintenance of storm water conveyance facilities. This Manual covers the following topics:

1. Hydrology
2. Street Drainage and Inlets
3. Storm Drains
4. Culverts
5. Open Channels
6. Site Drainage
7. Detention Basins
8. Energy Dissipators
9. Debris Basins and Barriers
10. Desilting Basin
11. Subsurface Drainage
12. Green Infrastructure (GI)

This Manual limits its content to the planning and design of infrastructure in the context of storm water conveyance, flood management, and water quality.

1.1. Policies

Adequate designs for each project should provide for removal of runoff from the roadway or the upstream end of any development and for carrying runoff water from the upstream side of the street to the downstream side. These functions should be accomplished without causing objectionable backwater, causing excessive or increased velocities, creating damages to downstream ownerships, or unduly affecting the safe operation of traffic on the roadway. Design criteria for drainage should be selected to provide for safe operation of vehicular and pedestrian traffic and to prevent damage to any adjacent property. The goal of storm water conveyance design is to provide optimum facilities considering function versus cost.

1.1.1 Basic Objectives

The objectives are to collect, transmit, and discharge drainage in a manner to promote public safety and provide for low maintenance by:

1. Providing for the public health and safety.
2. Preventing property damage
3. Calculating the quantity and frequency of storm runoff.
4. Determining the natural points of concentration and discharge and other hydraulic controls.
5. Determining the necessity for protection from floating trash and from debris moving under water.
6. Determining the requirements for energy dissipation and slope protections.
7. Analyzing the deleterious effects of corrosive soils and waters on storm drain and structures
8. Minimizing scour and siltation of natural streambeds, canyons, and lagoons.
9. Preventing the diversion of drainage.
10. Comparing and coordinating proposed design with existing structures and systems handling the same flows.
11. Coordinating with other agencies the proposed designs for facilities.
12. Providing access for maintenance operations.
13. Providing for removal of detrimental amounts of subsurface water.
14. Designing the most efficient drainage facilities consistent with good drainage practices and considering economic considerations, ease and economy of maintenance, safety, legal obligations, and aesthetics.

1.1.2 Exceptions to Design Standards

The City recognizes that it is not possible to prescribe design standards and procedures for all situations. For instance, there are many already-developed areas within the City that do not fully conform to the drainage standards presented in this Manual. Upgrading existing facilities in previously developed areas to conform to all policies, criteria, and standards outlined in this Manual can be difficult, if not impractical, outside the context of complete redevelopment or renewal.

Exceptions may be made when the City determines that (1) the strict application of the design and procedures to a specific situation may result in unreasonable requirement for a particular project; and (2) an exception to standard drainage criteria would not be detrimental to public health, life, or safety. Such exceptions will be made by the Asset Managing Department (AMD) for the storm water conveyance system.

The design standards and procedures outlined in this Manual have been formulated so that their application in the overall planning and design of storm water conveyance and flood management facilities should be reasonable, practical, and economical.

Engineers and planners should also consider that certain critical facilities might require a higher level of protection than the minimum protection levels stated in this Manual. The City may require that critical infrastructure or emergency facilities have additional protection so their functionality will not be compromised during large flood events. Such extraordinary protection levels will be considered on a case-by-case basis.

1.1.3 Basic Policy in Drainage Design

The basic considerations are: to protect roadways and property against damage from artificial, storm, and subsurface waters; to provide for public health and safety; to minimize the effects of the proposed improvement on traffic and property; and to prevent the diversion of drainage.

Drainage facilities shall be designed for low maintenance and shall protect the environment.

1.1.4 Cooperative Drainage Project

The City may participate in cooperative projects for storm drains in accordance with Council Policy 800-04.

1.1.5 Update of Design Standards

The criteria described by this Manual shall be revised and updated as necessary to reflect advances in drainage engineering and water resources management.

1.2. Drainage Report and Green Infrastructure Sizing Reports

A drainage report and/or a Green Infrastructure Sizing report shall be required for all public improvements and private developments including but not limited to commercial, industrial, and multiple-family residential development and all subdivisions. The drainage report shall address the proposed drainage conditions as compared to the existing drainage conditions at the time of plan approval. This is to include but is not limited to hydrology studies, hydraulic design, plan and profile plots, drawings showing the existing and proposed drainage facilities, sediment analysis, and any other relevant requirements. The Green Infrastructure Sizing Report shall include the hydrology calculations, hydraulic calculations, proposed sizing of the GI, and any other information used to show the proposed GI is sized appropriately. When required by the Storm Water Standards Manual, a Storm Water Quality Management Plan (SWQMP) serves as the sizing report.

Within floodplain and floodplain fringe areas as defined by the Federal Emergency Management Agency (FEMA), the runoff criteria shall be based upon a 100-year frequency storm. Under City requirements, the minimum elevation of the finished, first floor elevation of any building is two (2) feet above the 100-year frequency flood elevation. In addition to the requirements herein, the engineer shall adhere to the FEMA regulations and guidelines for all projects located in the floodplain.

Any computer program used to support the engineer's calculations should be calibrated specifically to use the City calculations and should include good documentation of the calculations. The study should document information including input, steps followed, and key maps used. The output for the Rational Method and Modified Rational Method calculations should both include the following information for review. For overland flow, the output shall list flow distance, time of concentration (for natural and urban drainage areas), intensity, runoff coefficient (C value), acreage of the drainage area, and discharge (Q). For flow in pipes, the output shall list the roughness coefficient (n value) for the pipe, pipe length, pipe slope (or upstream elevation and downstream elevation), the time of concentration, discharge (Q), and velocity. The goal of the documentation is to clearly present the operations performed by the computer. Check with the City before using any software to determine its applicability.

All drainage reports and Green Infrastructure Sizing Reports shall be prepared and sealed by a registered civil engineer in California. This section also applies to conceptual reports. The drainage report must also include a certification page that indicates if the project requires coverage under 401/404 permits or is exempt from obtaining 401/404 permits.

The engineer is required to prepare the reports/plans in accordance with this Manual; any other applicable City manuals, ordinances, or specific published submittal requirements; and notices by the City.

1.3. Use of Standard Drawings

The following documents are to be used in conjunction with this Manual (the documents referred to be the latest versions):

1. Latest City of San Diego Standard Drawings (SDSD) and San Diego Regional Standard Drawings (SD-RSD) as adopted by the City, and
2. Standard drawings published by the County of San Diego, American Public Works Association (APWA) for Southern California, the California Department of Transportation (Caltrans), Los Angeles County Department of Public Works, and Orange County Department of Public Works may be used when the San Diego Regional Standards are silent regarding a particular design situation

In case of conflict, the SD-RSD and SDSD shall prevail. If SD-RSD and SDSD conflict, SDSD shall prevail. Other resources may also be used for the design of storm water conveyance and flood management facilities, subject to review and approval by the City.

The Standard Drawings are provided for the convenience of the design engineer but the engineer is responsible for their use.

Chapter 2

Hydrology

The design discharge depends upon many variables. Some of the more important variables are duration and intensity of rainfall; storm frequency; ground cover; and the size, imperviousness, slope, and shape of the drainage area.

2.1. Discharge Flow Methods

The designer should check with Drainage and Flood Plain Management Section, Public Works Department, to determine if there are established storm discharge flows.

If the project involves a watershed of major size or importance, flood flows may already be established through one or more of the following activities:

1. Master Plan Developments in the City and/or County
2. Studies for Development and Road Projects near the proposed project
3. Flood Insurance Studies prepared by FEMA based on existing land use at the time the study was completed. Urbanization may have caused increased flows. FEMA maps can be viewed at the SanGIS web site (www.sangis.org).
4. Recorded flows may be available from the United States Geological Survey (USGS) or the County of San Diego

If no established storm discharge flows are available, the applicable methods are:

1. Rational Method for watersheds less than 0.5 square miles – See Appendix A
2. Modified Rational Method for watersheds between 0.5 and 1.0 square miles – See Appendix A; or,
3. Natural Resources Conservation Service (NRCS) Method (formally called Soil Conservation Service (SCS) Method) for watersheds greater than 1.0 square miles – See Appendix B; or
4. Hydrologic Engineering Center (HEC) computer method.

2.2. Design Storm Frequency

Design storm frequency shall be based upon the following criteria:

1. Within floodplain and floodplain fringe areas as defined by FEMA, the runoff criteria shall be based upon a 100-year frequency storm.

2. For all drainage channels and storm water conveyance systems, which will convey drainage from a tributary area equal to or greater than one (1) square mile, the runoff criteria, shall be based upon a 100-year frequency storm.
3. For tributary areas under one (1) square mile:
    a. The storm water conveyance system shall be designed so that the combination of storm drain system capacity and overflow (streets and gutter) will be able to carry the 100-year frequency storm without damage to or flooding of adjacent existing buildings or potential building sites.
    b. The runoff criteria for the underground storm drain system shall be based upon a 50-year frequency storm.

2.3. Soil Type

For storm drain, culverts, channels, and all associated structures, Type D soil shall be used for all areas.

2.4. Other Requirements

1. Design runoff for drainage and flood control facilities within the City shall be based upon full development of the watershed area in accordance with the land uses shown on the City of San Diego, Progress Guide and General Plan.
2. When determining criteria for floodplain management and flood proofing, design runoff within the City shall be based upon existing conditions in accordance with the City Floodplain Management Requirements and FEMA Regulations.
3. Under City requirements, the minimum elevation of the finished, first floor elevation of any building is 2 feet above the 100-year frequency flood elevation.

2.5. Water Quality Considerations

Requirements for hydrologic studies specific to the design of pollution prevention controls and hydromodification management controls are detailed in the Storm Water Standards. Where the Storm Water Standards specify modifications to the guidelines stated herein on discharge flow methods, design storm frequency, or soil type, the modifications shall supersede these but only for the purposes stated in the Storm Water Standards. Where the Storm Water Standards does not specify a modification, the guidance found here in Chapter 2 shall apply.

Chapter 3

Street Drainage, Cleanouts, and Inlets

Roadway drainage and inlet design are critical components of a storm water conveyance system. The surface drainage system must be consistent with the capacity of the storm conveyance system immediately downstream (discussed in Section 3.2). The design of roadway drainage and inlets are guided by the following principles:

1. Promote the safe passage of pedestrian and vehicular traffic, maintain public safety, and manage flooding during storm events.
2. Minimize capital and maintenance costs of the storm water conveyance system.

This chapter summarizes the general design criteria for roadway drainage and inlet design and describes the methods to apply when designing these conveyance systems.

3.1. Design Criteria

The capacity of surface drainage system (street capacity and inlet interception capacity) must be consistent with that of the storm water conveyance system immediately downstream. The roadway conveyance capacity is constrained by the width, depth, and velocity of flow.

When replaced, a storm water conveyance system shall be replaced from structure to structure. Patches and partial replacements only allowed on a case-by-case basis at the discretion of the AMD even when done as part of an emergency project or a project for another asset.

3.1.1 Roadway Drainage

The roadway drainage requirements include the following:

1. Maximum allowable drainage on any street section shall be based on Section 3.2.
2. Street drainage shall be intercepted with an underground storm drain prior to any street grade of ten percent (10%) or greater.
3. Design for drainage on any multi-lane street section shall require retention of the drainage flows in such a manner that one lane in each direction of traffic will be clear of flow at all times for a 50-year frequency storm.
4. The minimum street and gutter grade permitted is 0.6 percent.
5. All streets shall have a nominal two percent (2%) crown unless superelevated.

Section 3.2 discusses the calculation of flow width, depth, and velocity.

3.1.2 Inlets

3.1.2.1 Inlet Location

Mandatory Inlet Location

Storm drain inlets must be placed at prescribed locations in order to protect the public safety and provide a functional storm water conveyance system. These locations include:

1. Locations where flow in roadway cross-section exceeds the limitations prescribed in Section 3.1.1, above;
2. Low points in the roadway profile, such as sumps;
3. The upstream side of superelevated roadway cross-sections (located in a manner such that no more than 0.1 acre contributes to flow crossing traffic lanes);
4. Superelevated roadway transition sections where concentrated flows are not permitted to cross traveled lanes under the design storm frequency for the street. Median inlets shall be designed and spaced so the lane adjacent to the median (number one lane or fast lane of traffic adjacent to the median) is free from drainage flow for the design storm frequency;
5. Tee intersections where the terminating or side street grade exceeds five percent (5%) or for flatter grades when the gutter flow will encroach or overrun the cross gutter;
6. Subdivision boundaries (for flows in the public right-of-way); and
7. Intersections where flow is directed towards the intersection without relief from a cross gutter.

Recommended Inlet Location

The designer may desire to locate inlets at locations in addition to the minimally prescribed locations in order to improve the function of a storm drain system. These locations include:

1. Locations where there is a change to the roadway cross-slope; and
2. Downstream of changes in roadway longitudinal slope, especially in locations of reduced roadway grade to prevent sedimentation and increase safety. Locating inlets at the downstream side of longitudinal roadway grade changes allow the designer to utilize the higher depth developed by the reduced longitudinal grade in the roadway. Because water depth changes gradually due to momentum, it is more efficient to locate the inlet a minimum distance from the beginning point of the vertical curve (Figure 3–1).

The designer shall locate inlets in a manner to minimize the necessity of cross-gutters and shall avoid locating inlets where they would necessitate a local depression at the median.

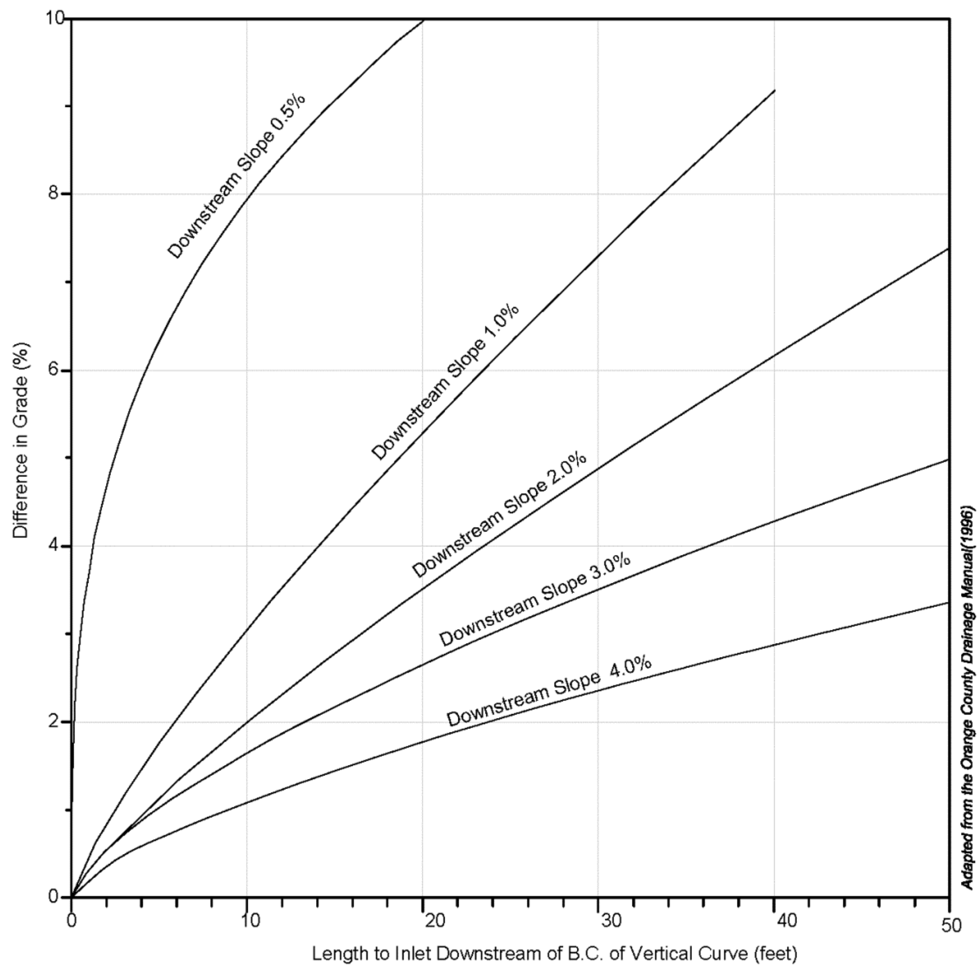


Figure 3-1. Recommended Inlet Location Downstream of Longitudinal Grade Change

Local Street Intersections with Arterial or Collector Streets

Where local streets intersect arterial or collector streets, the grade of the arterial or collector street shall continue uninterrupted. For drainage purposes, a cross gutter may be used perpendicular to the local street to convey flow across the intersection when necessary. The cross gutter shall be sufficient to convey runoff across the intersection with a spread equivalent to that allowed on the street.

Collector or Arterial Street Intersections

In the case where two collector or arterial streets intersect, the longitudinal grade of the more major street shall be maintained as much as possible. No form of cross gutter shall be constructed across major streets for drainage purposes. Cross gutters across the intersection may be considered for collector streets, but only in rare cases and with City approval.

3.1.2.2 Inlet Capacity

The basic criteria for storm drain inlet design shall be that any inlet will be sized to accept one hundred percent (100%) of the drainage received without bypass for the design storm frequency required for the system.

Storm flows in the public right-of-way should be picked up at subdivision boundaries.

All inlets shall be designed so that the energy gradient is a minimum of six (6) inches lower than the gutter grade or grate of grated inlets, whichever is lower.

3.1.2.3 Standard Drawing Types

Type of inlet to be used when an inlet is required shall be as described below. These may be used at any approved location in the curb on grades up to five percent (5%). Grades in excess of five percent may necessitate the use of a concrete apron within the parking lane, additional inlets, or other special inlet design as approved by the City.

1. Type “A” Curb Inlet – May only be used where there is no room behind the curb (due to existing conditions) to accommodate other types of curb inlets, or will provide the most efficient design for a particular problem.
2. Type “B” Curb Inlet – To be used as the basic inlet to intercept street drainage.
3. Type “C” Curb Inlet – Not permitted where street grade is less than five percent (5%). May be used to gain additional inlet capacity of two CFS maximum when a Type “A” or “B” curb inlet is slightly insufficient and combined with a street grade over five percent (5%).
4. Type “D” Curb Inlet – Not permitted in the City.
5. Type “E” Curb Inlet – Not permitted in the City.
6. Type “F” Catch Basin – To be used to intercept surface drainage from ditches or swales outside of traveled ways. Not permitted adjacent to sidewalks, bikeways, or trails for public use.
7. Type “G” Catch Basin – Not to be used; use Type “I” catch basin.
8. Type “H” Catch Basin – Not permitted in the public right-of-way.
9. Type “I” Catch Basin – May be used in alleys. Other uses are only allowed under special conditions and with prior approval.
10. Inlet aprons shall be limited to parking lanes only.
11. Use curb inlet – Type “J” Median (SDD-118) for center median inlets (four feet maximum opening, L=5’ Max.). Do not depress the gutter beyond the lip of the gutter (i.e., not into the driving lane).
12. Sidewalk Underdrains – Use type “A” curb outlet (D-25) for 0.5 cfs to 4.0 cfs.; use sidewalk underdrain(s) pipe (D-27) for flows up to 0.5 cfs.
13. Minimum inlet opening length is four feet, L=5’.
14. Maximum inlet opening is twenty feet, L=21’.

15. Any wing on an inlet shall be constructed on the upstream side.

3.1.2.4 Inlet Depression

When a shoulder or parking lane separates the curb inlet from traffic lanes, a maximum depression depth of 0.33 feet (4 inches) is allowed; when an inlet is adjacent to traffic lanes (for instance, at medians), 0.17 feet (2 inches) of depression is allowed.

3.2. Design Procedure

3.2.1 Gutter Flow

This Manual calculates flow depth and velocity under the assumption of uniform flow for typical roadway cross-sections with six-inch and eight-inch curb heights, standard gutter widths, and two percent (2%) cross-slope. The conveyance is determined using the Uniform Flow (Manning) Equation (Equation 3-1).

Equation 3-1. Uniform Flow (Manning) Equation

$$Q = \frac{1.49}{n} AR^{2/3}S^{1/2}$$

where:

Q	=	discharge (ft^3/s)
n	=	Manning roughness coefficient (dimensionless)
A	=	cross sectional area of the flow (ft^2)
R	=	hydraulic/wetted radius (ft)
S	=	longitudinal gutter slope (not roadway cross-slope; ft/ft)

Figures 3–2 and 3–3 provide the depth and velocity of flow versus the longitudinal slope and discharge for standard six-inch and eight-inch curb and gutter configurations, respectively. The designer is referred to the Federal Highway Administration's (FHWA) Urban Drainage Design Manual (HEC 22) for guidance for other curb and gutter configurations. Curb and gutter configurations other than a San Diego Regional Standard six- inch curb and gutter require prior approval from the City.

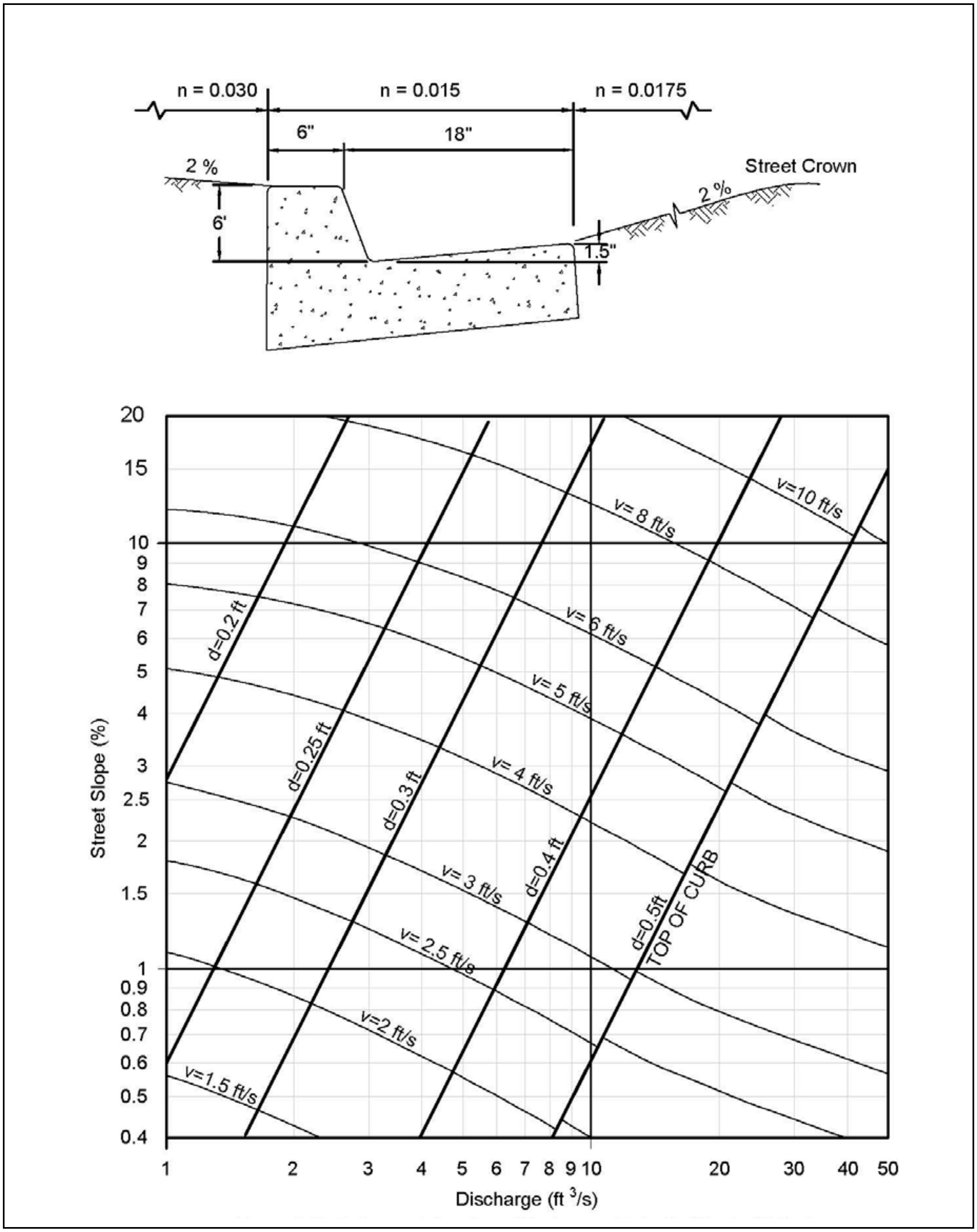


Figure 3-2: Gutter and Roadway Discharge-Velocity Chart (6" Curb)

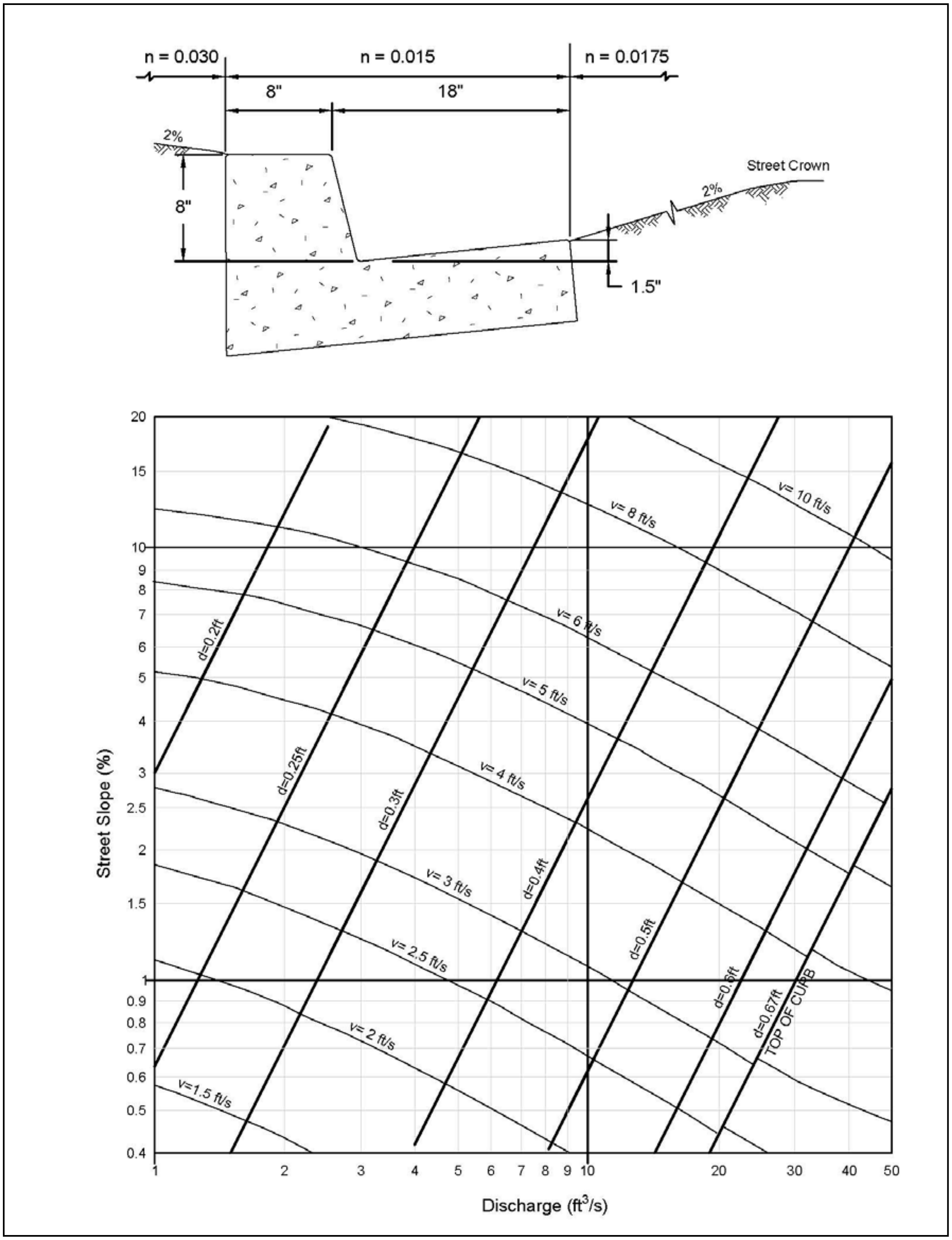


Figure 3-3: Gutter and Roadway Discharge-Velocity Chart (8" Curb)

3.2.2 Inlet Design

3.2.2.1 Curb Inlets on Grade

Full Interception

The capacity of a curb inlet on continuous grade depends on gutter slope, depth of flow in the gutter, the dimensions of the curb opening, and the amount of depression at the catch basin. Equation 3–2 describes the capacity of a curb inlet assuming full (100 %) interception.

Equation 3-2. Capacity of Curb Inlet

$$\frac{Q}{L_T} = 0.7\,(a+y)^{3/2}$$

where:

Q	=	interception capacity of the curb inlet (ft^3/s)
y	=	depth of flow approaching the curb inlet (ft; maximum of $y = 0.4$)
a	=	depth of depression of curb at inlet (ft; use $a=0.33$)
L_T	=	length of clear opening of inlet for total interception (ft)

Figure 3–4 illustrates the relationship between interception capacity, depth of approaching flow, and curb inlet depression, and may be used to determine curb inlet interception capacity.

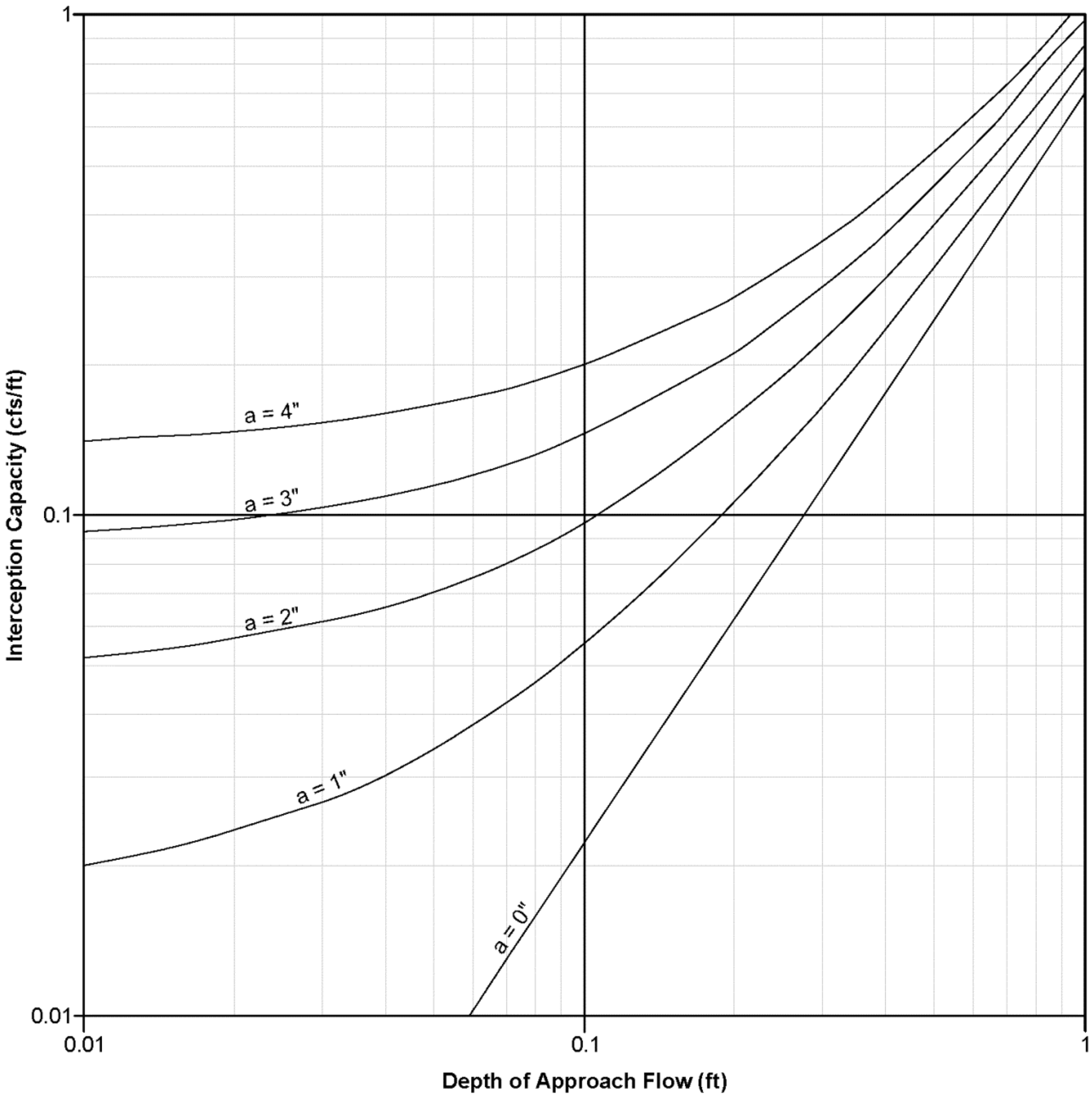


Figure 3-4. Interception Capacity of Curb Opening Inlet on Continuous Grade

Section 3.2.1 describes the method used to calculate the depth of flow approaching the curb inlet. Maximum y values are discussed in Section 3.1.2.4. Inlet opening guidelines are discussed in Section 3.1.2.3. The total length of the curb inlet (L) includes the length of the upstream and downstream face of the curb inlet (equal to an additional 1 foot of length for a SD-RSD Type B inlet).

Partial Interception

Partial interception is not permitted; use full interception as described above.

3.2.2.2 Curb Inlets in Sag

Curb inlets in sags or sump locations operate as weirs at shallow depths and operate as orifices as water depth increases. The designer shall estimate the capacity of the inlet under each condition and adopt a design capacity equal to the smaller of the two results. When designing the size of a facility, the designer shall use the larger of the sizes obtained by solving for the two conditions.

Inlets in sumps act as weirs for shallow depths, which can be described using Equation 3–3.

Equation 3-3. Shallow Depth Weir

$$Q=C_W L_W d^{3/2}$$

where:

Q	=	inlet capacity (ft^3/s)
C_W	=	weir discharge coefficient
L_W	=	weir length (ft)
d	=	flow depth (ft)

Table 3–1 presents appropriate weir coefficient values and lengths for various inlet types.

Table 3-1: Weir Discharge Coefficients for Inlets in Sag Locations

Inlet Type	Coefficient (C_W)	Weir Length (L_W)	Equation Valid
Grate Inlet Against Curb	3.00	L + 2W [(1)]	$d<1.79(A_O/ L_W)$
Grate Inlet, Flow from All Sides	3.00	2(L + W) [(1)]	$d<1.79(A_O/ L_W)$
Curb Opening Inlet	3.00	L'	d<h
Depressed Curb Opening Inlets Less than L'=12 feet [(2)]	3.00	L' + 1.8W	d<h
Slotted Inlets	2.48	L [(1)]	d< 0.2 ft

(1) Weir length shall be reduced to account for clogging.

(2) "Depressed Curb Opening Inlets" refers to curb inlets with depression larger the width of the gutter (for example, SD-RSD No. D-3B, "Concrete Apron for Curb Inlet"). The width (W) of the curb opening depression is measured perpendicular to the face of the curb opening.
Where: L = curb opening length (feet); L' = length of clear opening of installed inlet (feet); and A_o = orifice area.

At higher flow depths, curb inlets operate as an orifice (Equation 3–4).

Equation 3-4. Higher Flow Depth Curb Inlet

$$Q = 0.67hL(2gd_o)^{1/2}$$

where:

Q	=	inlet capacity (ft^3/s)
h	=	curb opening height (ft)
L	=	curb opening length (ft)
g	=	gravitational acceleration (32.2 ft/s^2)
d_o	=	effective depth of flow at curb face (ft)

The effective depth of flow at the curb face includes the curb depression and must be adjusted for the curb inlet throat configuration. The San Diego Regional Standard curb inlet opening (SD-RSD No. SDD 102) has an inclined throat; therefore, the effective depth of flow at the curb face is given by Equation 3–5.

Equation 3-5. Effective Depth of Flow at Curb Face

$$d_o = (y+a) - \frac{h}{2}\sin\theta$$

where:

y	=	depth of flow in adjacent gutter (ft)
a	=	curb inlet depression (ft)
$(h/2)\sin\theta$	=	adjustment for curb inlet throat width (h) and angle of throat incline (θ). For a standard 6-inch curb inlet opening with a 4-inch depression (SD-RSD No. SDD 102), $(h/2)\sin\theta$ =3.1 inches (0.26 ft)

Table 3–2 presents appropriate orifice coefficient values and lengths for various inlet types. In general, if an inlet is functioning as an orifice, the depth of flow is very deep and it is recommended that the design of the inlet be re-considered to avoid this condition.

Table 3-2. Orifice Coefficients for Inlets in Sag Locations

Inlet Type	Coefficient (C_o)	Orifice Area (A_o)	Equation Valid
Grate Inlet	0.67	Clear Opening Area [(1), (2)]	$d > 1.79(A_O/L_W)$
Curb Opening Inlet	0.67	hL	$d + (h/2) > 1.4h$
Slotted Inlets	0.80	LW [(2)]	$d < 0.4$ ft

(1) Actual grate opening area for SD-RSD No. D-15 Drainage Structure Grate is A_o=4.7 ft^2.

(2) Orifice area shall be reduced by 50 percent to account for clogging

3.2.2.3 Grated Inlets on Grade

The actual length and width of the grate is the overall dimension of the grate less the width of any bars or vanes. A single San Diego Regional Standard No. D-15 grate has an actual length of L=3.0 ft and an actual width of W=1.6 ft. To account for the effects of clogging of grated inlets on a grade, the actual length and width of the grate is reduced by a factor of fifty percent (C=0.50). Therefore, the effective length and width of a SD-RSD D 15 grate inlet are L_e=1.5 ft and W_e=0.8 ft, respectively. The FHWA Urban Drainage Design Manual (HEC 22) provides guidance for other grate types and configurations.

The procedure to determine the interception capacity of a grated inlet on grade is as follows:

Step 1. Divide the flow approaching the inlet ($Q_{APPROACH}$) into frontal discharge (Q_W) and side flow (Q_S), (i.e., flow exceeding the width of the grate). A San Diego Regional Standard No. D 15 grate has an effective width W_e=0.8 ft. This is determined by Equation 3–6.

Equation 3-6. Interception Capacity of Grated Inlet on a Grade

$$Q_W = Q_{APPROACH}\left[1-\left(1-\frac{W_e}{T}\right)^{2.67}\right]$$

$$Q_s = Q_{APPROACH} - Q_W$$

where:

Q_W	=	portion of approaching flow within the width of the grate (ft^3/s)
$Q_{APPROACH}$	=	total flow approaching the grate (ft^3/s)
T	=	total spread of water in the roadway (ft)
Q_S	=	side discharge (ft^3/s)
W_e	=	effective width of the grate (ft)

Step 2. Compare the approach velocity to the splash-over velocity. The splash-over velocity for a single San Diego Regional Standard No. D-15 grate is 2.0 ft/second. The designer is referred to the FHWA Urban Drainage Design Manual (HEC 22) for guidance for other grate types and configurations. When the approach velocity is less than the splash-over velocity, it can be assumed that the grate intercepts all of the approaching frontal discharge. When the approach velocity exceeds the splash-over velocity, calculate the amount of Q_W intercepted by the grate. This is determined by Equation 3–7.

Equation 3-7. Frontal Discharge

$$Q_{INTERCEPT,\ FRONT}=(1.0-0.09(V-V_O))Q_W$$

where:

$Q_{INTERCEPT,FRONT}$	=	frontal discharge intercepted by grated inlet (ft^3/s)
V	=	velocity of flow approaching inlet (ft/s)
V_O	=	splash-over velocity (V_O=2.0 ft/s for a standard D-15 grate)
Q_W	=	frontal flow approaching the grated inlet (ft^3/s)

Step 3. Determine the amount of side flow (Q_{SIDE}) that is intercepted by the grate. This is determined by Equation 3–8.

Equation 3-8. Side Discharge

$$Q_{INTERCEPT,SIDE}=\frac{Q_{SIDE}}{\left(1+\frac{0.15V^{1.8}}{S_x L_e{}^{2.3}}\right)}$$

where:

$Q_{INTERCEPT,\ SIDE}$	=	side discharge intercepted by grated inlet (ft^3/s)
Q_{SIDE}	=	side flow (i.e., flow outside the width of the grate) (ft^3/s)
V	=	velocity of flow approaching inlet (ft/s)
S_x	=	street cross slope (not the longitudinal slope of gutter; ft/ft)
L_e	=	effective length of the grate (ft). A San Diego Regional Standard No. D-15 grate has an effective length of L_e=1.5 ft. The Federal Highway Administration's Urban Drainage Design Manual (HEC-22) provides guidance for other grate types and configurations.

Step 4. Calculate the amount of flow intercepted by the inlet and the bypass flow, and apply the bypass flow to the roadway flow calculations and inlet capacity calculations downstream. This is determined by Equation 3–9.

Equation 3-9. Total Intercepted Discharge

$$Q_{INTERCEPT,TOTAL}=Q_{INTERCEPT,SIDE}+Q_{INTERCEPT,FRONT}$$

3.2.2.4 Grated Inlets in Sag

A grated inlet in a sag location operates as a weir at shallower depths and as an orifice at larger depths. The designer shall estimate the capacity of the inlet under both weir flow and orifice flow conditions, then adopt a design capacity equal to the smaller of the two results. Figure 3–5 provides a nomograph for calculating the capacity of grated inlets in sag locations.

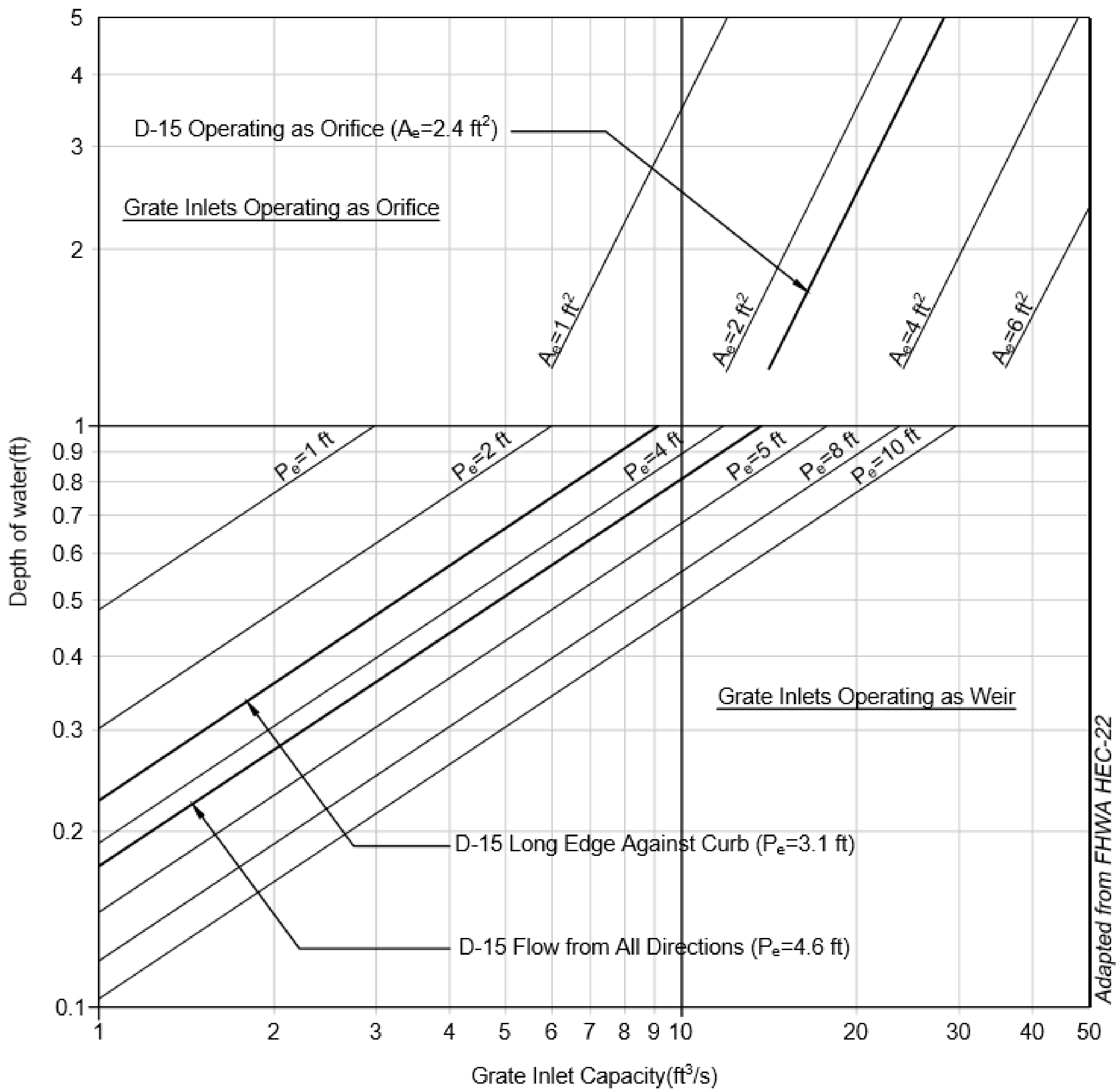


Figure 3-5. Capacity of Grate Inlets in Sag Locations

Step 1. Calculate the capacity of a grate inlet operating as a weir using the weir equation (Equation 3–10) with a length equivalent to perimeter of the grate. When the grate is located next to a curb, disregard the length of the grate against the curb.

Equation 3-10. Grate Inlet Capacity

$$Q=C_W P_e d^{3/2}$$

where:

Q	=	inlet capacity of the grated inlet (ft^3/s)
C_W	=	weir coefficient (C_W=3.0 for U.S. Traditional Units)
P_e	=	effective grate perimeter length (ft.)
d	=	flow depth approaching inlet (ft.)

To account for the effects of clogging of a grated inlet operating as a weir, a clogging factor of fifty percent (C_L=0.50) shall be applied to the actual (unclogged) perimeter of the grate (*P*). This is determined by Equation 3–11.

Equation 3-11. Grate Perimeter Length

$$P_e=(1-C_L)P$$

where:

P_e	=	effective grate perimeter length (ft)
C_L	=	clogging factor (C_L=0.50)
P	=	actual grate perimeter (ft.; i.e., the perimeter less the total width of bars or vanes), P=2W+L for grates next to a curb, and P=2(L+W) for grates with flow approaching from all sides

Step 2. Calculate the capacity of a grate inlet operating as an orifice. Use the orifice equation (Equation 3–12) assuming the clear opening of the grate reduced by a clogging factor C_A=0.50. A San Diego Regional Standard No. D-15 grate has an actual clear opening of A=4.7 ft^2. The FHWA Urban Drainage Design Manual (HEC 22) provides guidance for other grate types and configurations.

Equation 3-12. Capacity of Grate Inlet Operating as Orifice

$$Q=C_O A_e (2gd)^{1/2}$$

$$A_e=(1-C_A)A$$

where:

Q	=	inlet capacity of the grated inlet (ft^3/s)
C_O	=	orifice coefficient (C_O=0.67 for U.S. Traditional Units)
g	=	gravitational acceleration (32.2 ft/s^2)
d	=	flow depth above inlet (ft)
A_e	=	effective grate area (ft^2)
C_A	=	area clogging factor (C_A=0.50)
A	=	actual opening area of the grate inlet (i.e., the total area less the area of bars or vanes)

Step 3. Use the more conservative of the two results.

3.2.2.5 Combination Inlets

The procedure for calculating the capacity of a combination of a curb opening inlet and grate inlet assumes that the curb opening inlet is placed upstream of the grated inlet.

Step 1. Determine the portion of flow intercepted by the curb-opening part of the inlet (see Section 3.2.2.1).

Step 2. Determine the depth, width, and velocity of the flow that bypasses the curb opening part of the inlet.

Step 3. Determine the portion of flow intercepted by the grated inlet (see Section 3.2.2.2).

3.2.2.6 Slotted Inlets

Slotted drains may be used in certain approved, select locations or to increase the capacity of curb inlets where justified, providing they meet the following conditions:

1. Minimum pipe size shall be eighteen inches (18”).
2. Minimum pipe grade shall be five percent (5%).
3. Minimum slot height shall be six inches (6”).
4. Pipe shall be 16 gage or heavier.
5. Pipe shall conform to the minimum allowable service life for underground conduits (see Chapter 4, “Storm Drains”).
6. All drain pipes/conduits shall be designed to withstand an H-20 loading.
7. Maximum length of any one run of slotted pipe shall be sixty feet (60’).
8. The slotted pipe trench shall be backfilled and encased from below the bottom of the pipe with 420-B-2500 concrete to the subgrade of the final surface course of the traveled way.
9. Length of slotted pipe required may be determined from Figure 3–6 and Figure 3–7.
10. Use of slotted drain pipe should be discouraged in areas of heavy pedestrian traffic. Expanded wire mesh heel guards shall be attached across the top of the open slot when pipe is approved in pedestrian traffic areas.
11. Slotted drain pipes shall be used parallel to concrete median barriers for drainage pickup. These shall be collected in a Type I inlet (grate must be anchored).
12. A cleanout shall be installed at one end of a run of slotted drain pipe.

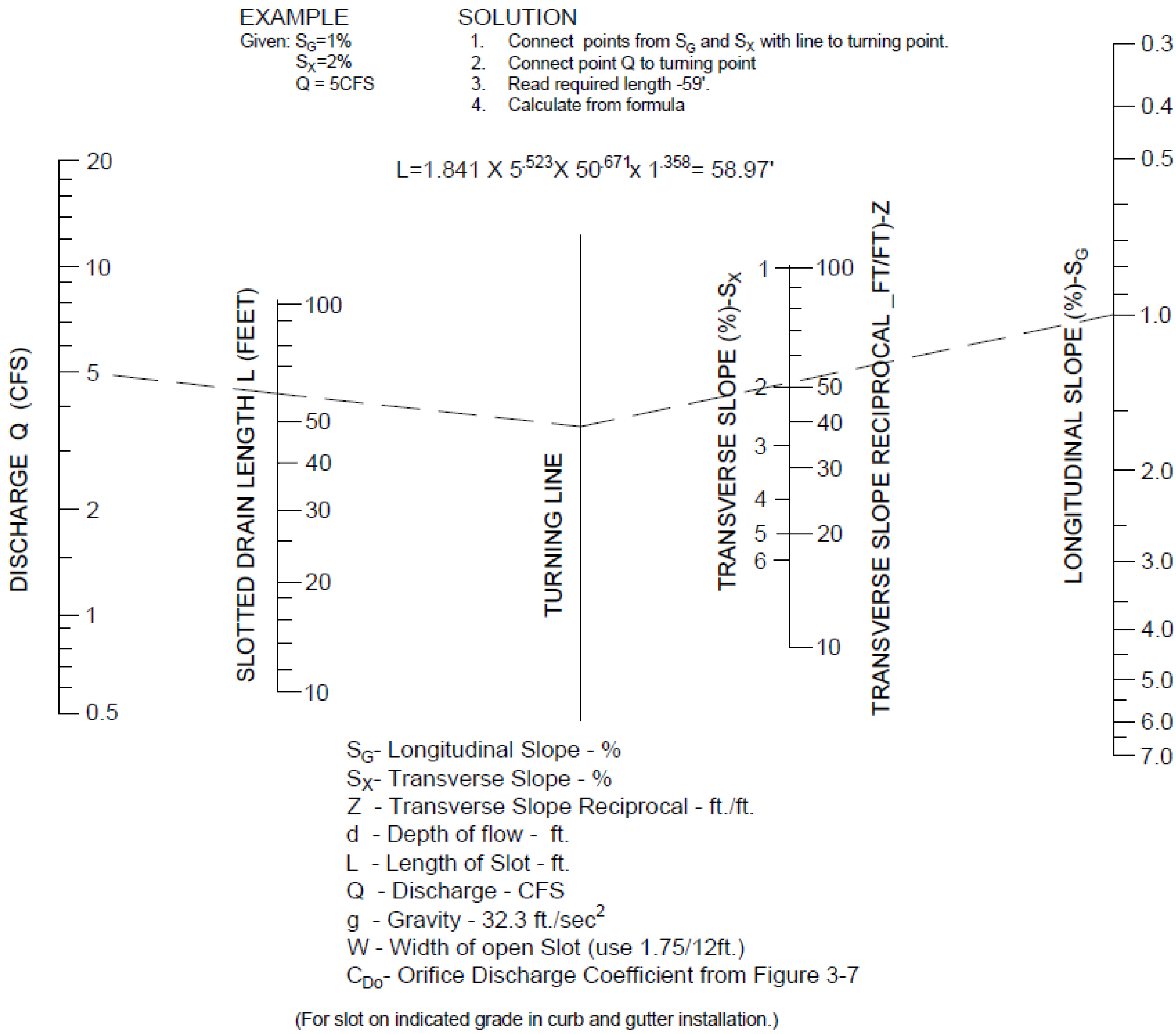
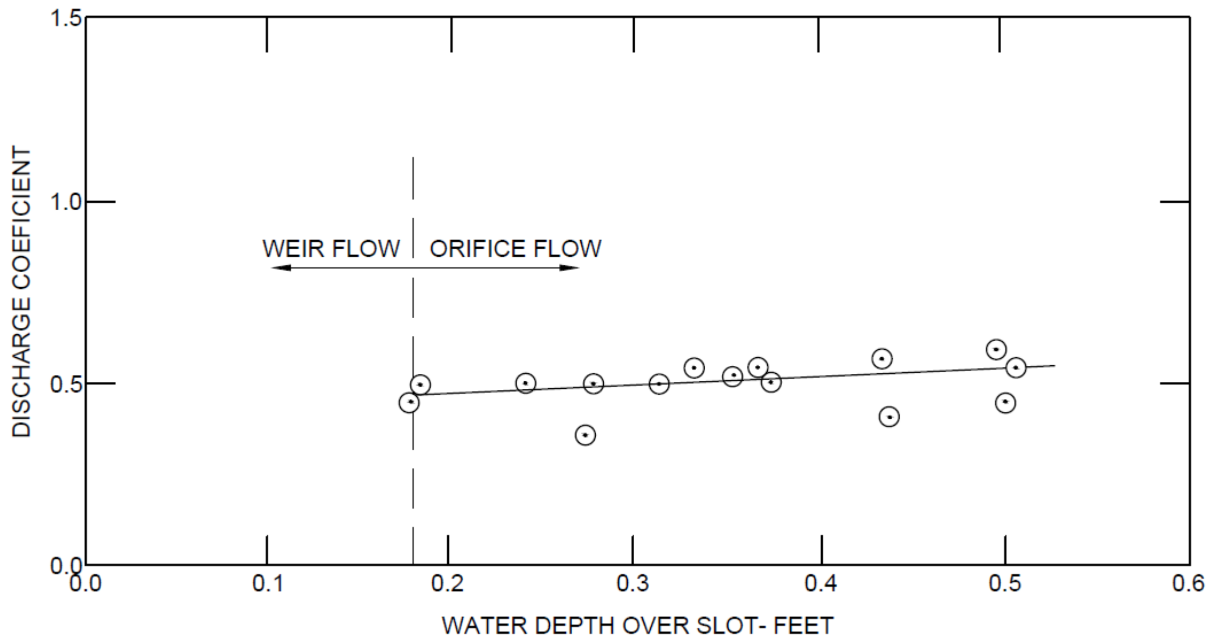


Figure 3-6. Design Length of Slot Opening - Slotted Drain Pipe Nomograph



Length of Grate = $Q / C_{DW} d^{\frac{3}{2}}$ where $C_{DW} = 3.125$
(For weir flow, d = 0.18 ft)

Length of Grate = $Q / C_{DO} W \sqrt{2gd}$
(For orifice flow, d > 0.18ft)

(For flatter grades than those shown in Figure 3–6 or for other than curb and gutter installation.)

Figure 3-7. Orifice Discharge Coefficient C_{DO}

When located on a grade, slotted inlets function as a side-flow weir, much like curb-opening inlets. The FHWA suggests that the hydraulic capacity of slotted inlets corresponds closely to the hydraulic capacity of curb-opening inlets when the slot openings are greater than 1.75 inches. Therefore, the designer may use the equations developed for curb opening inlets presented in Section 3.2.2.1 when the slot openings are greater than 1.75 inches. When located in a sump, slotted inlets can function either as a weir or as an orifice. As with grated inlets, the designer shall estimate the capacity of the inlet under both weir flow and orifice flow conditions, then adopt a design capacity equal to the smaller of the two results. Table 3–1 and 3–2 provides guidance for the appropriate coefficients to apply in the weir and orifice equations. Figure 3–8 presents a nomograph for the design of slotted inlets in sumps.

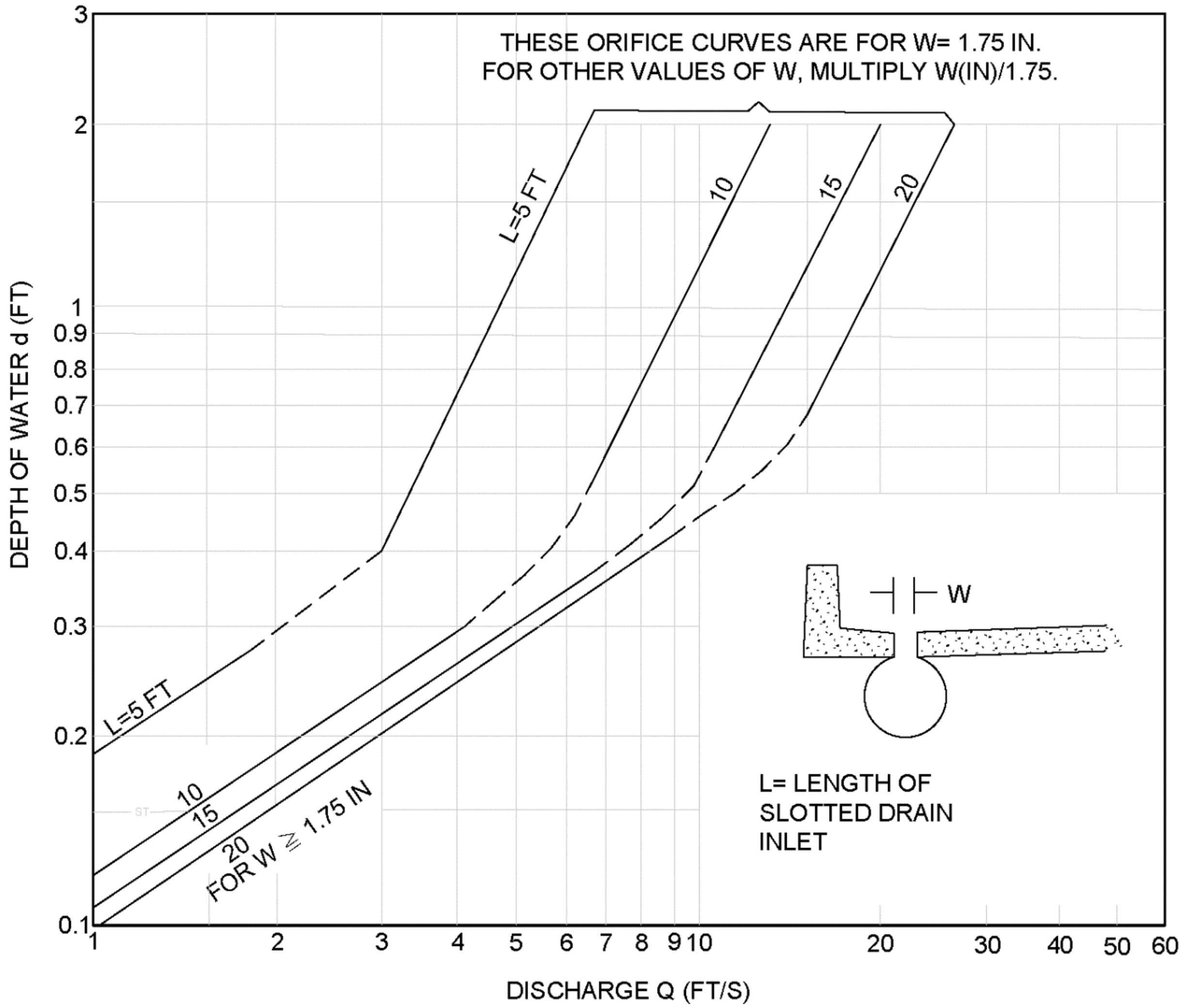


Figure 3-8. Slotted Drain Inlet Capacity in Sump Locations

As with grate inlets, a clogging factor (C_L) shall be applied to the actual (unclogged) length of a slotted inlet (L) (Equation 3 –13).

Equation 3-13. Grate Perimeter Length

$$L_e=(1-C_L)L$$

where:

L_e	=	effective grate perimeter length
C_L	=	clogging factor (C_L=0.50)
L	=	actual (unclogged) length of the slotted inlet (ft)

THIS PAGE INTENTIONALLY LEFT BLANK FOR DOUBLE-SIDED PRINTING

Chapter 4

Storm Drains

Underground conduits operate in conjunction with surface drainage to maintain public safety and manage flooding during storm events. The entire storm water conveyance system (underground conduits and street surface improvements) must have the capacity to convey the peak discharge from a 100-year design event without affecting property located adjacent to the right-of-way. Street drainage systems shall meet the criteria regarding the maximum flow width, depth, and velocity as described in Chapter 3 of this Manual. To satisfy these criteria, it is often necessary to supplement surface drainage with underground conveyance. This chapter summarizes the general design criteria for underground drainage conduits in the City of San Diego and describes the methods to apply when designing these systems.

4.1. Design Criteria

4.1.1 Hydraulic Capacity

Storm drains shall have the capacity to convey the discharge from the Design Storm Frequency as defined in Section 2.2.

The conduit shall convey the design flow with the hydraulic grade line (HGL) maintaining a minimum freeboard of 1 foot below the ground surface or gutter flow line during the design event.

Storm drains draining the public right-of-way shall not be less than 18 inches in diameter. The cross-sectional area of the pipe shall not decrease when proceeding down gradient within the storm drain system. Diversion of drainage is not allowed (i.e., the discharge point and all inlets of a storm drain system shall be within the same watershed).

This Manual references its design criteria and procedures to storm drain conduit with a circular cross-section. These criteria and procedures can be adapted to other cross-section shapes (e.g., arches, other non-circular or non-rectangular shapes) by comparing their section factor ($AR^{2/3}$).

4.1.2 Manning Roughness Coefficient

Appendix C provides a table of recommended Manning Roughness Coefficients for underground conduits.

4.1.3 Alignment and Curvature

4.1.3.1 Horizontal Alignment

Storm drains shall adhere to a straight alignment or a circular curve of uniform radius within the same run of pipe (i.e., from one clean-out, inlet, or other drainage structure to another). If curved, the storm drain shall follow the alignment of overlying streets whenever reasonable. All storm drains within a slope shall be aligned perpendicular to the slope contours. Provide a flat access area over all public storm drains.

The horizontal alignment of a storm drain system shall maintain a minimum horizontal clearance of no less than ten feet (10’) (outside diameter to outside diameter) from sanitary sewer lines and five feet (5’) (outside diameter to outside diameter) from potable water mains, reclaimed water mains, and other storm drains unless prior approval from the City is obtained.

The material type, length of pipe segments, and bevel of joints limit the curvature of the storm drain. Appendix D presents additional information on pipe alignment based on pipe characteristics.

When designing the junction of two storm drains, priority shall be given to the larger of the connecting storm drains. Flow from the smaller storm drain shall not oppose the flow in the main line without prior approval from the City. Specifically, when the angle of confluence (φ) is measured from the centerline of the main line, the angle of confluence shall be less than or equal to 90 degrees at all times. Figure 4–1 illustrates the definition of angle of confluence used in this Manual. The angle of confluence shall be further limited to 60 degrees or less in cases where:

1. The smaller pipe is 36 inches in diameter or larger; or
2. The flow from the smaller pipe is greater than or equal to 10 percent of the main-line flow.

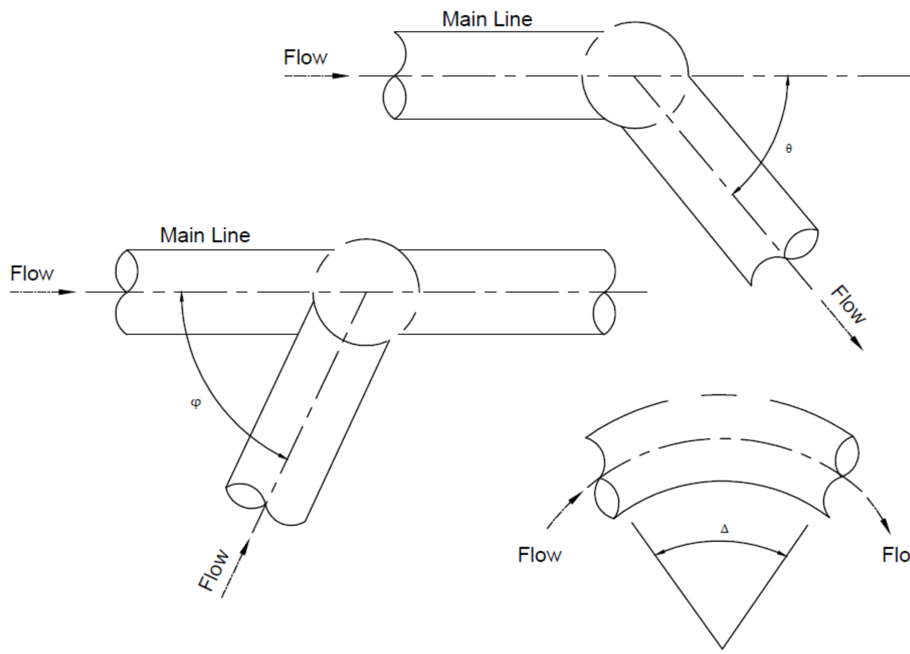


Figure 4-1. Definition Sketch for Angle of Deflection (θ), Angle of Confluence (φ), and Bend Radius (Δ)

4.1.3.2 Vertical Alignment

The vertical alignment of a storm drain systems shall (1) maintain a depth of cover to sufficient to avoid damage to the facility from overhead traffic loads; (2) minimize conflicts with other underground utilities with a minimum vertical separation of one [1] foot between the outside diameter of a storm drain and the outside diameter of other utilities; and (3) minimize potential buoyancy problems in cases where a groundwater table is present.

Storm drain conduit shall be protected from surface disturbances and displacements with soil or other cover. The minimum soil cover above a storm drain facility depends on storm drain material type and strength, size of the conduit, cover material, bedding conditions, and traffic loading. The designer shall determine the required cover based on project conditions, maintaining a minimum soil cover of 2 feet, or 1 foot below a pavement subgrade, whichever is greater.

The maximum soil cover above a storm drain facility depends on storm drain material type and strength, size of the conduit, cover material, bedding conditions, and traffic loading. Appendix D provides maximum soil loading for reinforced concrete pipe; the designer may specify alternate materials with prior City approval and demonstration that the material is adequate for the design load. The designer shall confirm that the design strength of the conduit will be adequate for the soil loading conditions. When there is more than 15 feet of soil cover, special design conditions may apply (see “Deep-Cover Culverts” Section 4.1.5 and “Minimum Easement Widths,” Section 4.1.6).

Best design practice for the vertical alignment within cleanouts, junction structures, or equivalent drainage structures is to provide a minimum of 0.1 foot of fall across the structure. When increasing the pipe diameter in the downgrade direction, the crowns (soffits) of the incoming and outgoing storm drains are to be matched.

4.1.4 Cleanouts

Cleanouts are structures that allow access for maintenance of a storm drain facility. For purposes of design, the definition of a cleanout under this section shall mean those structures designated as a cleanout, curb inlet, or catch basin in the City of San Diego Standard Drawings. The designer shall specify cleanouts at prescribed locations within a storm drain facility and at specific locations in relation to the horizontal and vertical curvature of a pipe alignment.

Inlet structures may be used as clean-outs for pipes less than or equal to 36 inches in diameter. Pipes larger than 36 inches in diameter require separate clean-out structures or inlets structures specially modified to provide for maintenance of the facilities. When an inlet structure is used as a clean-out structure, particular attention should be paid to the hydraulic grade line to maintain 1 foot of freeboard below the inlet flow line.

4.1.4.1 Standard Drawing Types

1. Type “A” cleanout – To be used as the basic cleanout
2. Type “B” cleanout – To be used only with City approval

4.1.4.2 General Location

Cleanouts shall be located at prescribed locations within a storm drain alignment to provide a maintainable drainage system:

1. At the point where a storm drain facility transfers from private to public maintenance;
2. At the point where a storm drain facility enters or exits a public right-of-way;
3. At points where the storm drain size, material, slope, or direction changes; and
4. At regular intervals along the storm drain alignment as prescribed in Table 4–1.

Table 4-1. Maximum Cleanout Spacing

Pipe Size	Maximum Cleanout Spacing
< 30"	300 ft.
< 42"	400 ft.
< 60"	600 ft.
≥ 60"	800 ft.

4.1.4.3 Horizontal Curves and Angle Points

Cleanouts shall be located as follows:

1. At the end of all horizontal curves in the conduit while maintaining the maximum spacing;
2. At the Point of Compound Curvature (PCC) and/or Point of Reverse Curvature (PRC) of curves;
3. At all horizontal angle points, except a single horizontal angle point of 10 degrees or less in cases where
   a. the angle point does not combine a horizontal and vertical curve; and
   b. the angle point is located within forty feet (40') of another cleanout or outfall; and
   c. abrasive bed load materials, under relatively high velocities (15 FPS or greater) will not occur.

4.1.4.4 Vertical Curves and/or Angle Points

Cleanouts shall be located as follows:

1. Normally on sag vertical curves, the cleanout should be located at the end connecting the flattest grade;
2. All vertical angle points, except
   a. A single vertical angle point of 10 degrees or less in cases where
      1) the angle point does not connect a horizontal and vertical curve; and
      2) the angle point is located within 10 feet of another cleanout or outfall; and

3) abrasive bed load materials, under relatively high velocities (15 FPS or greater) will not occur.

b. A single angle of deflection of 30 degrees within ten feet (10') of an outfall may be approved, provided a factory manufactured elbow (10 degrees maximum angle points) is used. Should the conduit ever be extended from this outfall, a cleanout shall be provided at this point. Should the direction of the pipe change more than a total of 30 degrees in the vertical or horizontal direction from the top of slope to bottom of slope, a cleanout is required at the bottom angle point.

The design shall make sure that the energy gradient is six inches (6") lower than the bottom of the roof slab on a cleanout.

4.1.4.5 Concrete Lugs

Concrete lugs shall not be used in lieu of cleanout.

4.1.5 Deep-Cover Conduits and Culverts

Depth of cover over fifteen feet (15') shall be allowed by necessity only. All drains shall utilize every possible means to minimize excessive cover. This may require offsite reconstruction or filling of upstream low areas. Larger pipe sizes to reduce pipe depth shall be used.

1. When pipe is under cover of 15 feet to 25 feet, the storm drain shall have a 100-year minimum life.
2. When pipe is under cover of 25 feet or more, the storm drain shall meet the following requirements:
    a. Be over-sized by 6 inches in order to provide for future interior repairs or re-lining;
    b. Have a 100-year minimum life;
    c. Pipe runs shall be straight with no curves or angle points.

4.1.6 Easements and Access Roads

4.1.6.1 Minimum Easement Width

Table 4–2 lists the minimum easement width required for underground storm water conveyance facilities.

Table 4-2: Minimum Easement Widths

Pipe Diameter or Equivalent	Minimum Easement Width
< 36"	15 ft.
≤ 60"	20 ft.
≤ 84"	25 ft.
≤ 108"	30 ft.
> 108"	40 ft.

These minimum easement widths assume conventional pipe installation with a maximum cover of 15 feet. Storm drains with cover between 15 feet and 25 feet will require an additional 2 feet of easement width for every foot of cover over 15 feet. Storm drains with cover deeper than 25 feet or other special conditions may warrant additional easement width and require City consultation and approval.

The minimum easement widths also assume that excavation of the pipes for replacement purposes can be accomplished while remaining in compliance with all OSHA (Occupational Safety and Health Administration) and Cal-OSHA requirements regarding the use of standard shoring techniques. Easement widths that need to be wider than the minimum to accommodate standard shoring shall be reviewed and approved by the asset owner. Special shoring and bracing techniques are not permitted as an alternative to providing wider easement widths since these methods cannot always be utilized in emergency situations.

4.1.6.2 Easement Alignment

For new development, storm drains and easements shall be located entirely within one lot or parcel and shall be aligned longitudinally adjacent to the parcel or lot line unless doing so is in conflict with the design standards set forth in other sections in this Chapter. In existing developments, storm drains easements shall follow lot or parcel lines to the maximum extent practicable.

All storm drain easements shall have physical access from the public right-of-way. Easements and access roads shall overlap to the maximum extent practicable.

Where storm drain easement alignment is interrupted (e.g., by a slope or curb), a non-contiguous easement may be proposed. Separate easements and roads may be proposed for each storm water pipe or structure.

If access is available from only one point, easements shall be at least 20 feet wide beginning 30 feet from the last storm drain structure to allow vehicles space to turn around.

4.1.6.3 Easement Encroachments

Encroachment into a storm drain easement shall be consistent with City Council Policy 700-18. In general, permanent structural improvements shall not be constructed on a storm drain easement. Facilities such as parking lots, recreation fields and trails, maintenance access roads, and fencing may be permitted and approved at the discretion of the AMD. In no case shall an encroachment impede access to a storm drain structure or be located on top of a structure so that the structure cannot be accessed.

When a permanent structure is located in an existing easement, the storm drain and easement may be relocated if an alternative location is acceptable to the AMD. Upon acceptance of the relocation, the existing storm drain and easement shall be abandoned. All cost for the relocation of new storm drain and easement and abandonment of the existing storm drain and easement shall be the responsibility of the requesting party.

4.1.6.4 Shared Easements

It is preferred that storm drain easements be established exclusively for storm drainage facilities. Joint use easements with other wet utilities (i.e., potable water and sanitary sewer) will be permitted when circumstances warrant, so long as the facilities within the joint-use easement maintain the minimum separation between wet utilities as outlined in Section 4.1.3. For easements containing more than one utility, an additional 10 feet of width shall be established for the storm drain.

4.1.6.5 Structures Adjacent to Easements

To mitigate for the potential adverse loading impacts of foundations on a storm drain, a minimum of five feet of additional easement width shall be required for storm drains which are located in areas directly adjacent to a building. Easements with the potential for structures on both sides of the easement will require ten feet of additional width.

4.1.6.6 Easements for Special Land Uses

Easements located on commercial property, industrial property, private streets, private driveways, apartment complexes, or condominium complexes that require vehicular access for storm drain maintenance shall be a minimum of 20 feet wide and the entire width of the easement shall be paved.

4.1.6.7 Easements Adjacent to Slopes, Building, or Retaining Walls

Any storm drain easement adjacent to slopes, buildings, or retaining walls shall require a submittal showing design calculations performed by a Registered Civil or Structural Engineer to show that there will be no adverse loading on the storm drain and that the limits of the trenching operations for storm drain repair or replacement will be outside the area of influence of the slopes, buildings, or retaining walls.

4.1.6.8 Easement Vacation

When storm drains in easements are abandoned, the associated easement shall be vacated provided the requirements of Municipal Code section 125.1040 are met.

4.1.6.9 Standard Access Roads

Access roads must be provided to all storm drain appurtenances (manholes, cleanout, outfalls, etc.). The access road shall be a minimum 15 feet wide, with a maximum grade not to exceed four (4) percent. The access easement and road must provide a minimum 200-foot transition prior to meeting the maximum grade of no greater than four (4) percent. The access easement and road must be widened to 20 feet at least 30 feet from storm drain structure (e.g., manhole, cleanout, outfall, etc.). Where access roads are not for the exclusive use of storm drain maintenance vehicles, the road shall be designed to maintain pedestrian and/or vehicular access (as applicable) during the storm drain repair and maintenance operations and shall be a minimum of 24 feet in width.

When only one point of access is proposed to the storm drain structure(s), then a turning radius of at least 30 feet must be provided such that the maintenance vehicle can properly execute a three-point turn around. If gated access is required, a minimum 15 feet wide gate will be required.

4.1.6.10 Access Roads in Open Space

This section is for open space locations other than canyon and environmentally sensitive lands such as designated or dedicated property managed by the City. For City-owned open space, access roads in open space shall have a cross slope of 1% or less. The roadbed section shall be centered over the storm drain where possible. The improved roadbed shall be a minimum 15 feet wide and shall be designed for H-20 loading and per City of San Diego Drawing SDG-113, Schedule "J", or equivalent. The maximum grade not to exceed four (4) percent. The roadbed shall be topped with an engineered geotextile filter fabric with a minimum 4 inch decomposed granite surfacing and soil bonding agent. No asphalt products shall be used in open space roads. In open space areas, an equivalent section of recycled, Class II base, free of oil-based products, may be used in lieu of Cement Treated Base (CTB). Consideration should be given to environmental impacts of required access roads so that the impacts of the roads can be addressed in the environmental document during the early stages of design. Access roads should be located in geotechnically stable areas, away from environmental sensitive areas, and a minimum of two feet above the 100-year flood plain, based on the ultimate development of the drainage basin.

4.1.7 Water-Tight Joints

Water-tight joints in a storm drain system shall be specified in prescribed locations and situations:

1. Where the HGL will exceed the inside crown (soffit) of the pipe by more than 5 feet for more than 40 feet of pipe length for the design storm.
2. Where pipe grade is twenty percent (20%) or greater.
3. Where the pre-project geologic investigation (i.e., soils report) indicates that groundwater levels might exceed the pipe invert elevation.

4.1.8 Slope Drains

A slope drain is defined as a conduit constructed on a grade of 5:1 (20%) or steeper that does not fall within a traveled way.

Slope drain outfalls shall extend to the nearest well-defined natural drainage channel that can adequately convey the discharge. Downstream conditions shall be investigated to verify the appropriateness of the point of discharge. This may require offsite storm drains or channel stabilization measures.

If extending the outfall to a well-defined low point would result in substantial cost increase, the outfall may be placed on a slope face if the calculated velocity does not exceed the Maximum Permissible Average Velocity found in Table 7–1 (for unlined channels only) for any section of the stream or channel from the outlet to one of the following downstream limits:

1. At least one reach downstream of the first grade control point (either an engineered grad control structure or surface such as a culvert or bedrock exposure that resists erosion thus acting as a grade control structure);
2. Tidal backwater/lentic (still water) waterbody;
3. Confluence point with an equal or larger order tributary area; or
4. A two-fold increase in drainage area

Velocity may be calculated as channel flow or shallow concentrated flow, whichever more accurately represents the downstream conditions. For shallow concentrated flow, use the Equation 4–1.

Equation 4-1. Velocity Calculation for Shallow Concentrated Flow

$$V=\frac{1.49}{n}R^{2/3}S^{1/2}$$

where:

R = 0.4 ft

n = 0.05

The simplified formula is presented in Equation 4–2.

Equation 4-2. Simplified Velocity Calculation for Shallow Concentrated Flow

$$V=16.13S^{1/2}$$

When an existing storm drain outlet needs to be replaced or upgraded, the requirements for the proposed outfall location shall be met to the extent feasible given existing physical and land ownership constraints. The financial impacts of increased infrastructure and land acquisition required to meet newer standards are to be considered when public funds are involved.

Additional requirements specific to slope drains are as follows:

1. Slope drains may be permanent installations or temporary drains for a future extension of a permanent installation, above or below ground.
2. Any slope drain that would be conspicuous or placed in landscaped areas shall be concealed by burial or other means.
3. Slope drains shall have a 100-year service life.
4. All slope drains shall have positive water-tight joint connections.
5. Adequate anchorage or cutoff walls shall be installed at intervals up to a maximum of thirty feet (30') for all conduit pipe placed on or within slopes 3:1 or steeper. Metal or fiberglass cutoff walls are not permitted.
6. Cutoff walls shall be installed at intervals up to a maximum of thirty feet (30'; horizontally) for all pipes placed in slopes where there is the possibility of erosion of the pipe trench on the slope.
7. All aboveground slope drains must have special approval from the City and shall be temporary.
8. Reinforced masonry or reinforced cast in place concrete cutoff walls is required.
9. Cleanouts for slope drains are to be half benched. Refer to Section 4.2.7.3.

4.1.9 Pipe Design Life

Storm drains shall have a minimum design life of at least 60 years except as noted below.

The design life for storm drains shall be 100 years when:

1. The height of cover is in excess of fifteen feet (15');
2. The conduit is or may be located under a structure or overhang of a structure permitted with an Encroachment Maintenance and Removal Agreement (EMRA);
3. The conduit is located within the traveled way of a four-lane collector, major, or prime arterial street;
4. The conduit is adjacent to or between structures that are located horizontally a distance equal to or less than the vertical pipe cover depth from the structure to the centerline of the pipe;
5. Any storm drain is under a pressure head; or
6. Slope drains.

The selection of pipe material shall consider factors such as strength of the conduit under maximum or minimum cover, bedding and backfill conditions, anticipated loading, length of sections, ease of installation, corrosive action of surrounding soils, expected deflection, and cost of maintenance. Where field conditions indicate the use of one pipe material in preference to others (for instance, corrosive soil conditions, presence of a groundwater table, or a seawater outfall), the reasons shall be clearly presented in the plans and specifications.

In cases where a storm drain is expected to carry a large amount of debris or abrasive sediment material, it shall have measures to provide sufficient design life for the facility. The pipe material will dictate the type and degree of protection required. When protection is warranted, the invert of the pipe (i.e., the lower 90 degrees of the pipe) shall be protected on all straight-aways and the invert and walls (i.e., the lower 180 degrees of the pipe) shall be protected on all curves.

Additional conduit thickness shall be considered sacrificial and shall not be included in a structural analysis.

4.1.10 Storm Drain Plans

Storm drain plans shall provide a minimum amount of information regarding storm drain design and construction, including **all** of the following:

1. Plan and profile for all public storm drains showing all cleanouts, inlets, and catch basins with their respective invert elevations, rim elevations, type, and station; and
2. Stationing, which shall increase in the up-grade direction from the lower end of the storm drain; and
3. Hydraulic Grade Line (HGL) of the flow within the pipe, including hydraulic jumps; and
4. Design flow and velocity (50-year, or 100-year, as appropriate); and
5. Pipe design load rating or equivalent information (depending on pipe material, this might include pipe gauge or wall thickness); and
6. Flow and velocity at the outfall of the pipe; and
7. Flow capacity of the pipe (Q_{pipe}); and
8. Length, material, and diameter of all storm drains; and
9. Property lines, right-of-way limits, street names and widths, finished grade; and
10. Conflicting underground utilities; and
11. Drawing numbers for related easements and existing structures; and
12. Delineation of the drainage basin for the storm drain that includes area calculation.

4.2. Hydraulic Design of Storm Drains

This section presents general procedures for hydraulic design and evaluation of storm drains.

4.2.1 Minimum Gradient

The minimum pipe gradient shall be 0.5 percent grade or the pipe shall have a minimum velocity of four feet per second (fps) with the pipe flowing one quarter full. Flatter grades may be approved where no other practical solution is available. Pipes shall be designed to flow full and free of pressure heads except for short runs where the grade changes and a small pressure head cannot be avoided. Where it is necessary to design for a pressure head in a system and it is approved by the City Engineer, pressure pipe with water-tight joints shall be used.

4.2.2 Basic Design Procedure

Storm drain capacity analysis shall account for changes in flow conditions (open channel versus pressure flow) in the hydraulic grade line (HGL) calculations. Figure 4–2 provides a definition sketch for storm drain hydraulic computations.

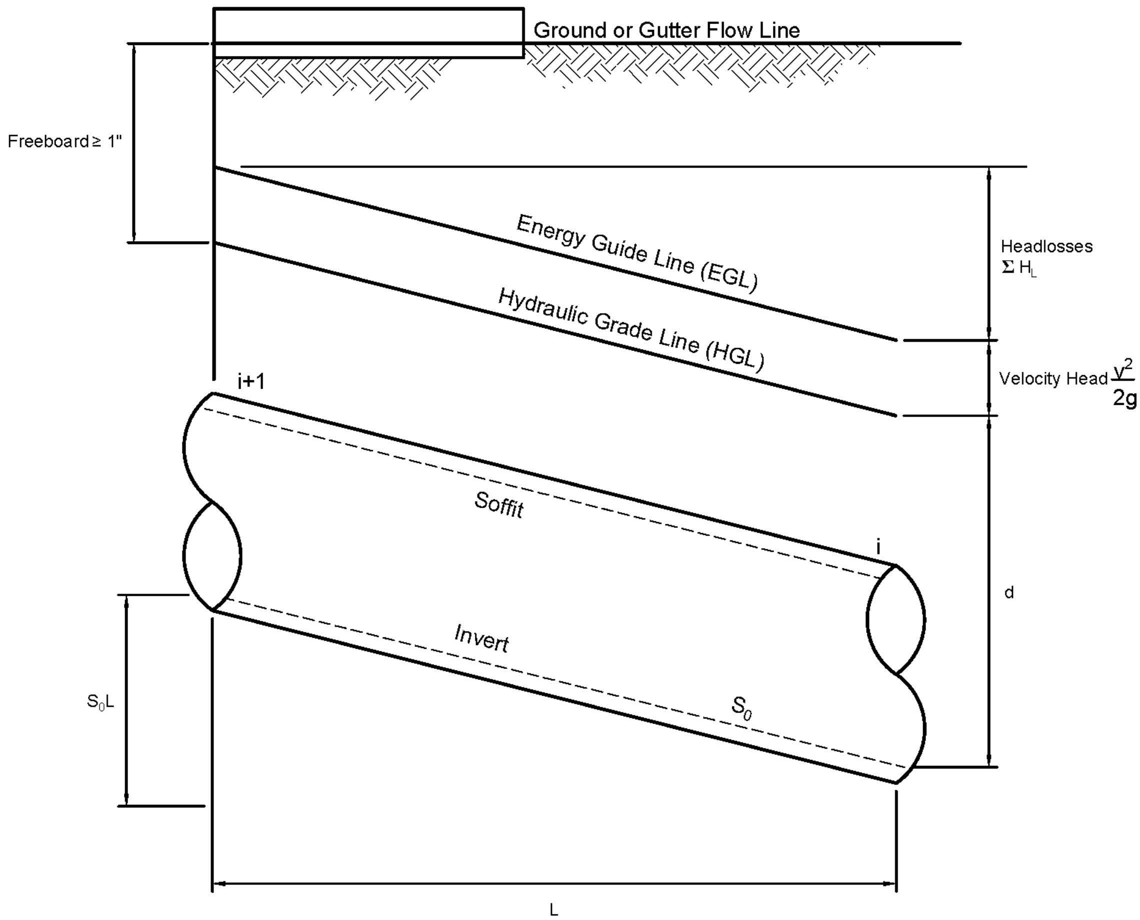


Figure 4-2. Definition Sketch for Storm Drain Hydraulic Calculations

The procedure for storm drain design proceeds as follows:

Step 1. Size storm drain system on a preliminary basis assuming uniform, steady flow conditions for the peak design discharge.

Step 2. Check the initial pipe sizes using the energy equation, accounting for all head losses.

Step 3. Adjust the pipe size and vertical alignment as necessary to provide minimum HGL freeboard.

4.2.3 Storm Drain Analysis – Uniform Flow

When a storm drain is not flowing full, it operates as an open channel and the hydraulic properties can be calculated using open channel techniques. The flow in a conduit operating as an open channel can be evaluated numerically using the Uniform Flow Equation (Equation 4-3).

Equation 4-3. Uniform Flow Equation

$$Q=\frac{1.49}{n}AR^{2/3}S_f^{1/2}$$

where:

Q	=	Discharge (ft^3/s)
n	=	Manning roughness coefficient (dimensionless)
A	=	cross-sectional area of the flow (ft^2)
R	=	hydraulic radius (ft)
S_f	=	friction slope, typically assumed to be equivalent to longitudinal slope of storm drain (S_o; ft/ft)

During full-flow conditions, the flow area and hydraulic radius for a circular pipe of diameter (D) can be simplified to the following relationships presented in Equation 4–4.

Equation 4-4. Simplified Flow Area and Hydraulic Radius for Circular Pipe

$$A=A_{full}=\frac{\pi D^2}{4}$$

$$R=R_{full}=\frac{D}{4}$$

Therefore, the minimum required diameter for a circular pipe (D_r) needed to convey a particular design flow (Q) can be calculated by Equation 4–5.

Equation 4-5. Pipe Diameter Calculation to Convey Design Flow

$$D_r=\left(\frac{2.16nQ}{\sqrt{S_0}}\right)^{3/8}$$

The pipe diameter is specified as the next standard pipe size larger than the minimum required (D_r). An analogous procedure can be followed for alternative conduit shapes. Figure 4–3 illustrates the hydraulic properties for circular pipes, assuming the Manning roughness coefficient does **not** vary with depth.

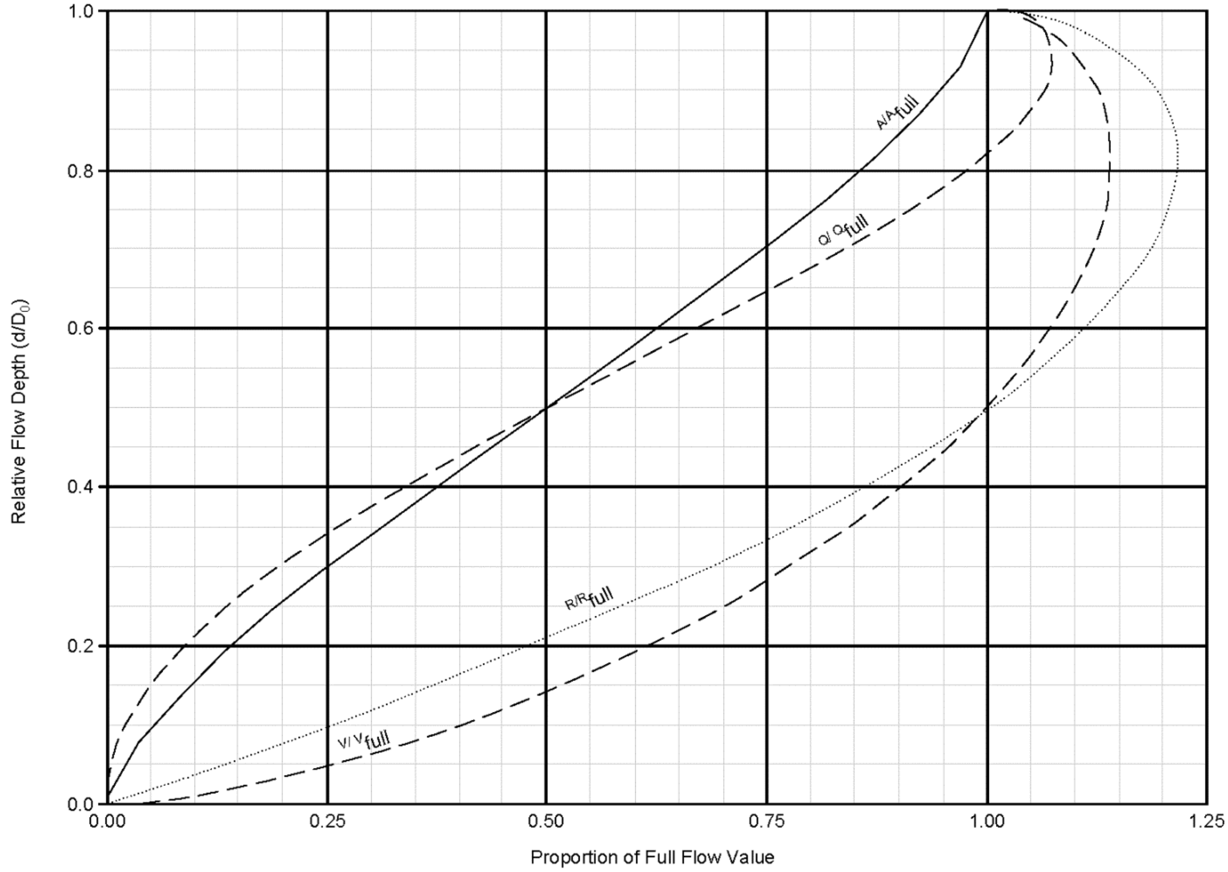


Figure 4-3. Hydraulic Properties of Circular Pipe

4.2.4 Storm Drain Analysis – Pressure Flow

When a storm drain is flowing under a pressure flow condition, the friction slope (S_f) and longitudinal slope of the storm drain (S_o) may not be equivalent. Therefore, the energy and hydraulic grade lines cannot be calculated using the Uniform Flow Equation. The capacity calculations generally proceed upstream from the storm drain outlet, accounting for all energy losses through each pipe run and drainage structure. These losses are added to the EGL and accumulate to the upstream end of the storm drain. The HGL is then determined by subtracting the velocity head (Hv) from the EGL at each change in the EGL slope.

4.2.5 Storm Drain Analysis - HGL Calculations

The designer shall check the available energy at all junctions and transitions to determine whether the flow in the storm drain will be pressurized due to backwater effects, even when the design flow is less than the full flow capacity of the storm drain.

To calculate the Energy Grade Line (EGL) for a storm drain system, divide the system into "runs" of pipe between structures (clean-outs, inlets, or other structures). The slope of the pipe shall be constant within each run. Starting with the downstream control elevation (EGL_i) for the most downstream run of pipe, first calculate the friction losses and bend losses through the pipe and then the losses across the upstream drainage structure. The EGL at the upstream end of the run (EGL_{i+1}) will be the sum of the downstream control elevation, friction losses, and structure losses, and will be the downstream control elevation for the next run of pipe (Equation 4–6).

Equation 4-6. EGL Calculation for Storm Drain System

$$EGL_{i+1} = EGL_i + (\Sigma H_L)_{PIPE} + (\Sigma H_L)_{STRUCTURES}$$

Refer to Figure 4–2 for an illustration of the components used in the energy grade line and head loss calculations. The hydraulic grade line (HGL) is then calculated by subtracting the velocity head ($v^2/2g$) from the energy grade line (Equation 4–7).

Equation 4-7. HGL Calculation

$$HGL_i = EGL_i - \frac{v_i^2}{2g}$$

EGL elevations must always decrease in the downstream direction, and must always increase in the upstream direction. On the other hand, HGL elevations may increase or decrease at structure locations regardless of the direction considered. For instance, the HGL will increase in the downstream direction within a pipe when there is a hydraulic jump.

4.2.6 Downstream Control (Tailwater) Elevation

The hydraulic analysis of a storm drain system begins at the downstream outfall. The controlling water surface elevation at the point of discharge is commonly referred to as the tailwater. At the outfall, one of several conditions will be encountered:

1. another closed conduit;
2. outfall to a drainage channel, storage facility, reservoir, lake, or detention facility;
3. a free outfall; or
4. a tidally influenced outfall.

The tailwater elevation criteria described here are for the purpose of determining HGL and EGL elevations only; Chapter 9, “Energy Dissipation,” describes tailwater elevation criteria for energy dissipation calculations.

For free outfalls, the initial water surface elevation (tailwater) shall be assumed to be equivalent to the soffit elevation. For outfalls into other drainage facilities, a drainage channel, reservoir, or detention facility, the initial water surface elevation shall be set at the 100-year water surface elevation calculated for the channel or described on the appropriate Flood Insurance Rate Map (FIRM) at the location of the outfall. In cases where the storm drain outfall condition is tidally influenced, it is usually sufficient to use the historic high tide elevation as the tailwater elevation. In cases where storm surge is a concern or for other situations with unusual tailwater conditions, the appropriate design outfall tailwater elevation shall be chosen in consultation with the City.

4.2.7 Energy Loss Calculations

This Manual presents a standard energy-loss method for use in the hydraulic analysis of storm drain systems in which head loss is calculated as a proportion of velocity head ($v^2/2g$). The standard energy-loss method is appropriate for storm drain analysis in most typical cases. Though not discussed in detail in this Manual, energy loss calculation methods based on pressure-momentum theory are also acceptable and may be more appropriate in some cases (e.g., systems with steep gradients).

Numerous computer programs are available for computing energy loss and hydraulic grade lines, such as the Los Angeles County Flood Control District's Storm Drain Analysis Program PC/RD4412 (STORM) and a variety of proprietary computer software packages. The design engineer should be aware of the energy loss method used by a particular computer program and recognize the limitations of any software and/or energy loss method applied.

Minor pipe losses and structure losses need not be calculated in situations where minor losses will not significantly affect the performance of the drainage system. The design engineer's best judgment will determine whether to calculate minor losses in cases where the HGL is significantly below freeboard requirements, cases with low pipe velocities, or in cases where backwater does not affect other properties.

4.2.7.1 Pipe Losses – Friction

Friction Losses – Open Channel

For open channel conditions under sub-critical flow, the friction slope of a pipe (S_f) and the longitudinal slope of the storm drain (S_o) may be assumed equivalent. As a result, the energy grade line and hydraulic grade line may be parallel as the flow approaches normal depth (see Equation 4–8).

Equation 4-8. Head Loss Calculation for Sub-Critical Flow

$$H_f = S_f L \approx S_o L$$

where:

H_f = head loss due to pipe friction (ft)

S_o = longitudinal pipe slope (not the S_f) of pipe (ft/ft)

S_f = friction slope (not the S_o) of pipe (ft/ft)

L = length of pipe (ft)

Friction Losses – Pressure Flow

When the downstream control elevation is higher than the downstream crown elevation, the storm drain is surcharged. When a storm drain is flowing under such a pressure flow condition, the friction slope (S_f) and longitudinal slope of the storm drain (S_o) may not be equivalent. In this case, friction loss (H_f) is computed using the expression found in Equation 4 –9.

Equation 4-9. Friction Loss Equation

$$H_f = S_f L$$

where:

H_f	=	head loss due to pipe friction (ft)
S_f	=	friction slope (not the S_o) of pipe (ft/ft)
L	=	length of pipe (ft)

The friction slope for a pipe under full-flow conditions can be derived from Manning's equation (see Equation 4–10).

Equation 4-10. Manning's Equation for Pipe Friction Slope

$$S_f = \left(\frac{Qn}{0.46D^{8/3}}\right)^2$$

where:

S_f	=	friction slope (ft/ft)
Q	=	discharge (ft^3/s)
n	=	Manning roughness coefficient
D	=	pipe diameter (ft)

4.2.7.2 Pipe Losses – Bend Losses

The bend loss coefficient (K_b) is primarily a function of the angle of curvature (Δ). The head loss for bends in excess of that caused by an equivalent length of straight pipe is expressed in Equation 4–11.

Equation 4-11. Head Loss Calculation for Bends

$$H_L = K_b \frac{v_2^2}{2g}$$

$$K_b = 0.0033\Delta$$

where:

Δ	=	angle of curvature (degrees)

This equation is most appropriate for bends with radii between eight and twenty times the diameter of storm drain (8D<R<20D). For tighter radius bends (R<8D), the design engineer shall multiply the value of K_b determined in the equation by the quantity 1+D/R. Bend losses do not need to be calculated when the radius of curvature is more than 20 times the diameter of the storm drain (R>20*D*). Head loss due to pipe bends shall be applied at the exit of the bend (point of curvature). For abrupt angular changes in alignment, use the structure loss defined in Section 4.2.7.3.

Expansion Losses – Open Channel Condition

Equation 4–12 describes expansion head loss under open flow conditions. The expansion loss coefficient (K_e) depends on the suddenness of the expansion and the relative size of the inflow and outflow pipes.

Equation 4-12. Expansion Head Loss under Open Flow

$$H_L=K_e\left(\frac{v_1^2-v_2^2}{2g}\right)$$

where:

K_e	=	expansion loss coefficient (Table 4-3 below)
v_1, v_2	=	upstream and downstream flow velocity, respectively (ft/s)
g	=	gravitational acceleration (32.2 ft/s^2)

Table 4–3 presents expansion loss coefficients for storm drains under open-channel conditions.

Table 4-3. Expansion Loss Coefficients under Open Channel Conditions

Expansion Loss Coefficient, K_e		
Cone Angle (θ)	$D_2/D_1=3$	$D_2/D_1=1.5$
10°	0.17	0.17
20°	0.40	0.40
45°	0.86	1.06
60°	1.02	1.21
90°	1.06	1.14
120°	1.04	1.07
180°	1.00	1.00

Contraction Losses – Open Channel Conditions

Equation 4–13 describes contraction head loss under open flow conditions. Note that the velocity head term in the open-channel contraction loss equation is the downstream velocity head less the upstream velocity head, which is the opposite of the expansion head loss equation. For gradual contractions, the contraction coefficient is assumed to be half the expansion loss coefficient ($0.5K_e$; see Table 4–4).

Equation 4-13. Contraction Head Loss under Open Flow

$$H_L = K_c\left(\frac{v_2^2 - v_1^2}{2g}\right)$$

where:

K_c	=	contraction loss coefficient ($0.5K_e$ or Table 4-4)
v_1, v_2	=	upstream and downstream flow velocity, respectively (ft/s)
g	=	gravitational acceleration (32.2 ft/s²)

Table 4–4 provides values for sudden contraction loss coefficients under open channel conditions.

Table 4-4. Contraction Loss Coefficients under Open Channel Conditions

D_2/D_1	Contraction Loss Coefficient, K_c
approaching 0	0.5
0.4	0.4
0.6	0.3
0.8	0.1
1.0	0.0

Expansion Losses – Pressure Flow

Expansion of the flow area in a storm drain under submerged conditions will result in a shearing action between the incoming high velocity jet and the surrounding conduit boundary. As a result, eddy currents and turbulence dissipate much of the kinetic energy. The head loss is expressed using Equation 4 –14.

Equation 4-14. Expansion Head Loss under Submerged Conditions

$$H_L = K_E\frac{v_1^2}{2g}$$

where…

v_1	=	upstream flow velocity (ft/s)
K_E	=	expansion loss coefficient, pressure flow (see Table 4-5 and Table 4-6)

The value of the expansion loss coefficient (K_E) varies from approximately 1.0 for a sudden expansion to 0.2 for a well-designed expansion transition. Table 4–5 and Table 4–6 present loss coefficients for pressure flow conditions for sudden and gradual expansions, respectively.

Table 4-5. Expansion Loss Coefficient (K_E) for Sudden Enlargement under Pressure Flow Conditions

	Velocity V_1 (ft/s)											
D_2/D_1	2.00	3.00	4.00	5.00	6.00	7.00	8.00	10.00	12.00	15.00	20.00	30.00	40.00
1.20	0.11	0.10	0.10	0.10	0.10	0.10	0.10	0.09	0.09	0.09	0.09	0.09	0.08
1.40	0.26	0.26	0.25	0.24	0.24	0.24	0.24	0.23	0.23	0.22	0.22	0.21	0.20
1.60	0.40	0.39	0.38	0.37	0.37	0.36	0.36	0.35	0.35	0.34	0.33	0.32	0.32
1.80	0.51	0.49	0.48	0.47	0.47	0.46	0.46	0.45	0.44	0.43	0.42	0.41	0.40
2.00	0.60	0.58	0.56	0.55	0.55	0.54	0.53	0.52	0.52	0.51	0.50	0.48	0.47
2.50	0.74	0.72	0.70	0.69	0.68	0.67	0.66	0.65	0.64	0.63	0.62	0.60	0.58
3.00	0.83	0.80	0.78	0.77	0.76	0.75	0.74	0.73	0.72	0.70	0.69	0.67	0.65
4.00	0.92	0.89	0.87	0.85	0.84	0.83	0.82	0.80	0.79	0.78	0.76	0.74	0.72
5.00	0.96	0.93	0.91	0.89	0.88	0.87	0.86	0.84	0.83	0.82	0.80	0.77	0.75
10.00	1.00	0.99	0.96	0.95	0.93	0.92	0.91	0.89	0.88	0.86	0.84	0.82	0.80
∞	1.00	1.00	0.98	0.96	0.95	0.94	0.93	0.91	0.90	0.88	0.86	0.83	0.81

Note: D_2/D_1 = ratio of diameter of larger pipe to smaller pipe; V_1 = velocity in smaller pipe (upstream of transition).

Table 4-6. : Expansion Loss Coefficient (K_E) for Gradual Enlargement under Pressure Flow Conditions

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

Note: D_2/D_1 = ratio of diameter of larger pipe to smaller pipe; Angle of cone is angle in degrees between the side of the tapering section.

Contraction Losses – Pressure Flow

Loss due to contraction of cross-sectional flow area in a storm drain under submerged condition is expressed in the form of Equation 4–15.

Equation 4-15. Contraction Loss for Cross Sectional Flow

$$H_L = K_c \frac{v_2^{\,2}}{2g}$$

where:

K_C	=	contraction loss coefficient (Table 4-7)
v_2	=	downstream flow velocity (ft/sec)

Contraction loss coefficient (K_C) varies from approximately 0.4 for large pipe size differences (greater than 10:1) to approximately 0.1 for minor pipe size differences. Table 4–7 presents contraction loss coefficients for pressure flow conditions.

Table 4-7. Contraction Loss Coefficient (K_C) for Sudden Contraction under Pressure

	Velocity V_1 (ft/s)												
D_2/D_1	2.0	3.0	4.0	5.0	6.0	7.0	8.0	10.0	12.0	15.0	20.0	30.0	40.0
1.1	0.03	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.05	0.05	0.06
1.2	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.08	0.08	0.08	0.09	0.10	0.11
1.4	0.17	0.17	0.17	0.17	0.17	0.17	0.17	0.18	0.18	0.18	0.18	0.19	0.20
1.6	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.25	0.25	0.24
1.8	0.34	0.34	0.34	0.34	0.34	0.34	0.33	0.33	0.32	0.32	0.32	0.29	0.27
2.0	0.38	0.38	0.37	0.37	0.37	0.37	0.36	0.36	0.35	0.34	0.33	0.31	0.29
2.2	0.40	0.40	0.40	0.39	0.39	0.39	0.39	0.38	0.37	0.37	0.35	0.33	0.30
2.5	0.42	0.42	0.42	0.41	0.41	0.41	0.40	0.40	0.39	0.38	0.37	0.34	0.31
3.0	0.44	0.44	0.44	0.43	0.43	0.43	0.42	0.42	0.41	0.40	0.39	0.36	0.33
4.0	0.47	0.46	0.46	0.46	0.45	0.45	0.45	0.44	0.43	0.42	0.41	0.37	0.34
5.0	0.48	0.48	0.47	0.47	0.47	0.46	0.46	0.45	0.45	0.44	0.42	0.38	0.35
10.0	0.49	0.48	0.48	0.48	0.48	0.47	0.47	0.46	0.46	0.45	0.43	0.40	0.36
∞	0.49	0.49	0.48	0.48	0.48	0.47	0.47	0.47	0.46	0.45	0.44	0.41	0.38

Note: D_2/D_1 = ratio of diameter of larger pipe to smaller pipe; V_1 = velocity in smaller pipe (downstream of transition)

4.2.7.3 Structures Losses – Inlets, Junctions, and Clean-Outs

Significant head losses can occur at a clean-out structure, whether or not one or more lateral storm drains confluence with the main line storm drain. Head losses in a clean-out can be due to incoming and outgoing pipe size, angle of confluence, relative discharges, and design details of the cleanout itself.

Inlets, Junctions, and Cleanouts - Simplified Method

Because of the difficulty in evaluating hydraulic losses at junctions due to the many complex conditions of pipe size, geometry of the junction, and flow combinations, it can sometimes be impractical to perform detailed head loss calculations for drainage structures. This section presents a simplified method for calculating head loss calculations at drainage structures. This simplified method may be used for the design of storm drain systems with low velocity head and minimal hydraulic constraints (i.e., systems with ample HGL freeboard). This method is also useful for developing preliminary design estimate for head loss in more complex drainage systems.

The head loss for a particular structure can be estimated using Equation 4–16.

Equation 4-16. Structure Head Loss Calculation

$$H_L = K\frac{v_o^2}{2g}$$

where:

H_L	=	head loss at drainage structure (ft)
K	=	simplified structure head loss coefficient (ft)
v_o	=	outflow velocity (ft/s)
g	=	gravitational acceleration (32.2 ft/s^2)

Table 4–8 presents values for the simplified head loss coefficient (K) across a drainage structure. This head loss can be used to estimate the drop between pipe crowns needed to offset energy losses at the structure, thus helping to avoid a submerged flow condition.

Table 4-8. Simplified Structure Head Loss Coefficient, K

Structure Configuration	Simplified Head Loss Coefficient, K
Inlet – Straight Run	0.50
Inlet – Angled Through 90 60 45 22.5	 1.50 1.25 1.10 0.70
Clean-Out – Straight Run	0.15
Clean-Out – Angled Through 90 60 45 22.5	 1.00 0.85 0.75 0.45

Inlets, Junctions, and Clean-Out – Composite Method

For the design and evaluation of storm drain systems with high velocity head or having significant hydraulic constraints, it may be necessary to complete more detailed calculations to account for the various factors contributing to head loss. The head loss at clean-out or other drainage structures is expressed as an initial or basic loss ($K_o v^2/2g$) modified by several correction factors as presented in Equation 4–17.

Equation 4-17. Head Loss at Clean-Out Structures Calculation

$$H_L = C_D C_Q C_P C_B \left(K_O \frac{v_2^2}{2g} \right)$$

where:

H_L	=	structure head loss (ft)
C_D	=	relative flow depth correction factor
C_Q	=	relative flow correction factor
C_P	=	plunging flow correction factor
C_B	=	benching correction factor
K_O	=	initial or basic loss coefficient
v_2	=	outflow velocity (ft/s)
g	=	gravitational acceleration (32.2 ft/s^2)

Basic Structure Loss Coefficient (K_O)

The initial or basic loss at a cleanout structure is defined in Equation 4–18.

Equation 4-18. Basic Structure Loss Calculation

$$K_O = 0.1\left(\frac{b}{D_O}\right)(1-\sin\theta) + 1.4\left(\frac{b}{D_O}\right)^{0.15}\sin\theta$$

where:

K_o	=	initial or basic loss coefficient
b	=	drainage structure diameter or equivalent diameter (ft)
D_o	=	outflow pipe diameter (ft)
θ	=	deflection angle

This basic equation is valid only when the water level in the receiving inlet or cleanout is above the invert of the incoming pipe. In cases where this is not true, the structure losses are assumed to be zero. For non-circular drainage structures, the equivalent structure diameter is defined as the diameter of a circular structure having the equivalent area of the actual non-circular one. Table 4–9 and Figure 4–4 present basic head loss for standard clean-outs.

Table 4-9. Equivalent Diameters for San Diego Regional Standard Clean-Outs

SD-RSD Standard Clean-Out	Length (ft)	Width (ft)	Area (ft²)	Equivalent Diameter (ft)
A-4	4	4	16	4.5
A-5	5	4	20	5.0
A-6	6	4	24	5.5
A-7	7	4	28	6.0
A-8	8	4	32	6.4

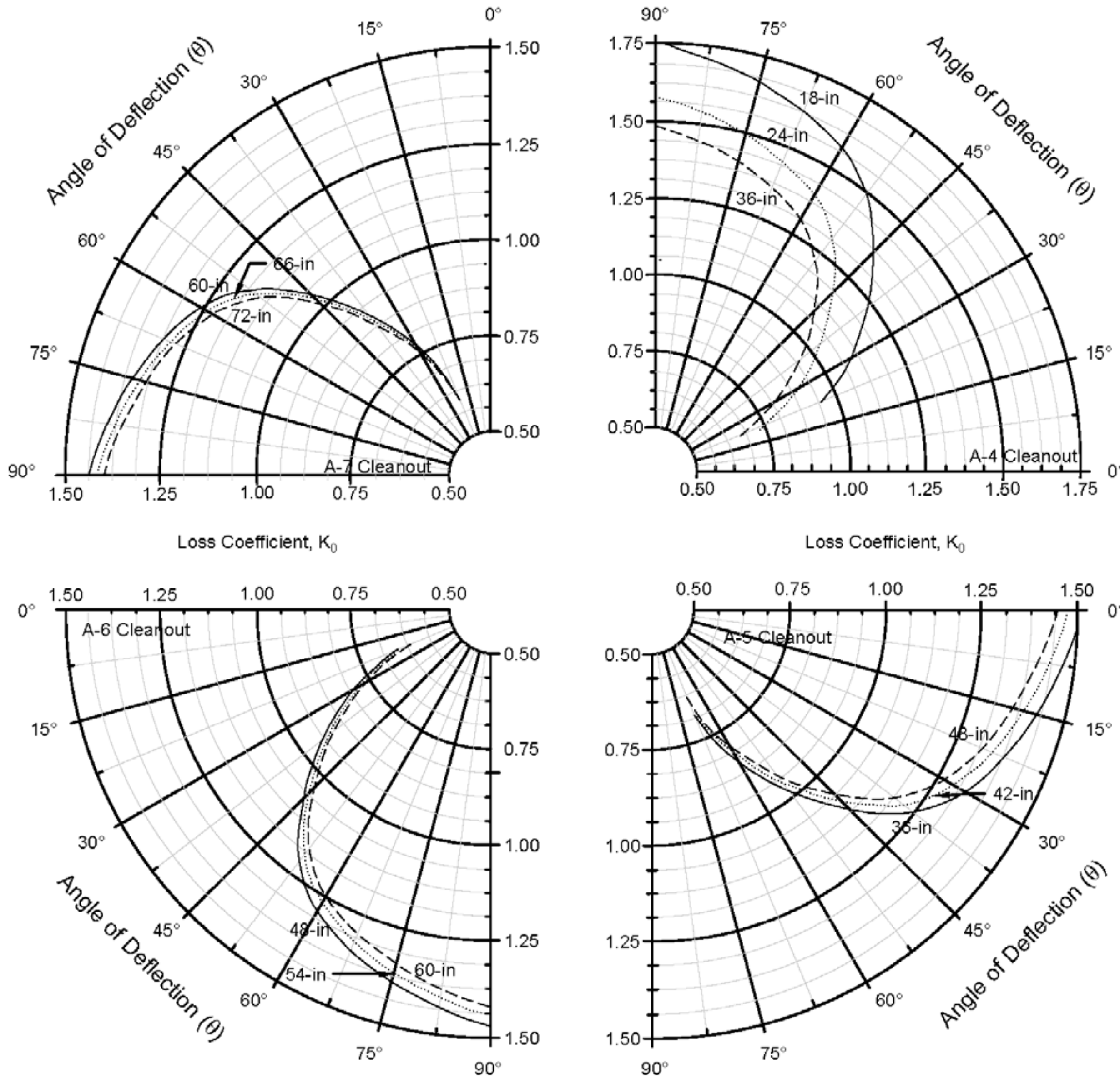


Figure 4-4. Basic Structure Head Loss Coefficients for San Diego Regional Standard Cleanouts

Relative Pipe Diameter and Flow Depth Correction Factor (C_D)

Equation 4–19 describes the correction factor that accounts for the relative pipe diameter and flow depth within a drainage structure. The relative flow depth correction factor depends on the depth of flow within the structure, which, in this case, is measured relative to the crown of the outlet pipe. When the flow depth in the structure above the crown of the outlet pipe (d_{out}-D_o) is much higher relative to the outlet pipe diameter (D_o; i.e., there is submerged flow or a high-pressure condition), the correction factor is based on the relative diameters of the inflow and outflow pipes. In cases where the relative flow depth is lower than, or not significantly larger than, the diameter of the outlet pipe, the correction factor is a function of the flow depth relative to the outlet pipe diameter.

For practical purposes, the correction factor for relative pipe diameter and flow depth need not be greater than C_D=3.0.

Equation 4-19. Correction Factor Calculation

$$C_D=\begin{cases} 0.5\left(\frac{d_{out}}{D_O}\right)^{0.6}, & (d_{out})/D_O<3.2 \\ \left(\frac{D_o}{D_i}\right)^{3}, & (d_{out})/D_O\geq 3.2 \end{cases}$$

where:

C_D	=	relative flow depth correction factor
d_{out}	=	depth of flow in clean out, measured as the difference between the HGL and the upstream invert of the outlet pipe (ft)
D_o	=	outflow pipe diameter (ft)
D_i	=	inflow pipe diameter (ft)

Relative Flow Correction Factor (C_Q)

A correction factor (Equation 4-20) can be applied when a lateral inflow to a cleanout where the incoming flow is greater than 10 percent of the main line flow. When the incoming lateral flow is less than 10%, this head loss equation is invalid and C_Q=1.0.

Equation 4-20. Relative Correction Factor Calculation

$$C_Q=(1-2\sin\theta)\left(1-\frac{Q_i}{Q_o}\right)^{0.75}+1$$

where:

C_Q	=	relative flow correction factor
Q_i	=	inflow to structure (ft^3/s)
Q_o	=	outflow from structure (ft^3/s)
θ	=	deflection angle

Plunging Flow Correction Factor (C_P)

When water falls into a structure, either from an inlet or from a smaller pipe significantly above the invert of the structure, a plunging flow correction factor (Equation 4-21) should be applied to the basic structure head loss.

Equation 4-21. Plunging Flow Correction Factor Calculation

$$C_P=1+0.2\left(\frac{h_p}{D_o}\right)\left(\frac{h_p-d_{out}}{D_o}\right)$$

where:

C_P	=	plunging flow correction factor
h_p	=	plunge height above centerline of outflow pipe, measured as the difference in elevation between the highest incoming pipe invert and the centerline of the outlet pipe (ft)
d_{out}	=	depth of flow in clean out, measured as the difference between the HGL and upstream invert of the outlet pipe (ft)
D_o	=	diameter of outlet pipe (ft)

Benching Correction Factor (C_B)

Benching the invert of a drainage structure can reduce head loss by directing flow through the structure. The benching correction factor depends on the type of benching that is specified and whether flow in the structure is submerged. Figure 4–5 illustrates invert benching for a drainage structure. Table 4–10 presents benching correction factor for submerged and unsubmerged flow for various benching configurations; for intermediate conditions ($1.0<d_{out}/D_o<3.2$), the designer may interpolate linearly between the tabulated values.

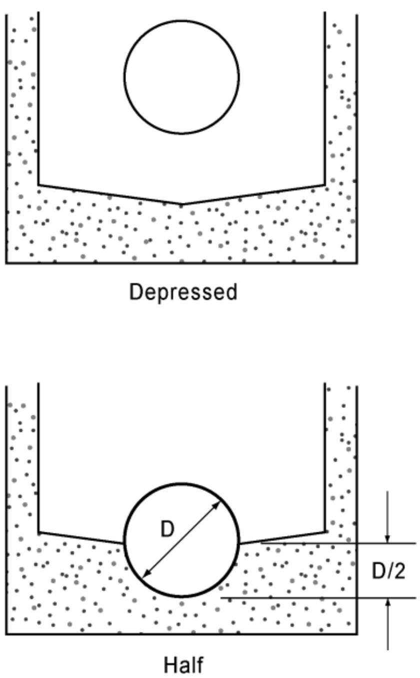


Figure 4-5. Invert Benching Configurations

Table 4-10. Benching Correction Factors (C_B)

Benching Type	Submerged Flow $d_{out}/D_o \geq 3.2$	Unsubmerged Flow $d_{out}/D_o \leq 1$
Flat or Depressed Floor	1.00	1.00
Half Bench	0.95	0.15
Full Bench	0.75	0.07

4.2.7.4 Entrance Losses

When runoff enters a storm drain system from open channels or other locations other than street inlets, an energy loss occurs at the entrance in the form of a contraction loss. The head loss at storm drain entrances is expressed in Equation 4 –22.

Equation 4-22. Storm Drain Entrance Loss Calculation

$$H_L = K_E \frac{v_2^2}{2g}$$

The entrance loss coefficient (K_E) for storm drain systems is the same as the entrance loss coefficient used for the entrance loss calculation for culverts. Table 4–11 provides a list of recommended entrance loss coefficients.

Table 4-11. Entrance Loss Coefficients for Storm Drains and Culverts
(Outlet Control, Full or Partly Full)

Type of Structure and Design of Entrance	Coefficient K_E (defined in Eq. 4-14)
Concrete Pipe	
Projecting from Fill, Socket End (Groove-End)	0.2
Projecting from Fill, Square-Cut End	0.5
Headwall or Headwall and Wingwalls Socket End of Pipe (Groove-End) Square-Edge	 0.2 0.5
Rounded (Radius = D/12)	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
Corrugated Metal Pipe or Pipe-Arch	
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)	0.5
Square-Edged on 3 Edges Rounded on 3 edges to Radius of D/12 or B/12, or Beveled Edges on 3 sides	0.2
Wingwalls, at 30° to 75° to Barrel Square-Edged at Crown Crown Edge Rounded to Radius of D/12 or Beveled Top Edge	 0.4 0.2
Wingwall at 10° to 25° to Barrel Square-Edged at Crown	0.5
Wingwalls Parallel (Extension of Sides) Square-Edged at Crown Side- or Slope-Tapered Inlet	 0.7 0.2

* "End sections conforming to fill slope," made of either metal or concrete, are the sections commonly available from manufacturers. From limited hydraulic tests they are equivalent in operation to a headwall in both inlet and outlet control. Some end sections, incorporating a closed taper in their design have a superior hydraulic performance. These latter sections can be designed using the information given for the beveled inlet.

4.2.7.5 Outlet Losses

When the storm drain system discharges into open channels, additional losses occur at the outlet in the form of expansion losses. These losses are due to several reasons. For most storm drain outlets, the flow velocity in the storm drain is greater than the allowable or actual flow velocity in the downstream channel. Therefore, energy-dissipating facilities are used to remove excess energy from the storm drain flow (see Chapter 8). In addition, the alignment of the storm drain at the outlet may not be the same as the downstream channel. Therefore, energy is lost in changing the flow direction between the storm drain to the downstream channel. The head loss at storm drain outlets is expressed in Equation 4–23.

Equation 4-23. Storm Drain Outlet Head Loss Calculation

$$H_L = K_{OUT}\frac{v_1^2}{2g}$$

An outlet loss coefficient (K_{OUT}) of 1.0 shall be used for all storm drain outlets.

4.2.7.6 Outlet Protection

Storm drain outlets shall be constructed with erosion protection when they discharge to unlined channels or natural drainage courses. Chapter 8 of this Manual discusses the requirements for outlet protection.

Chapter 5

Culverts and Low Water Crossings

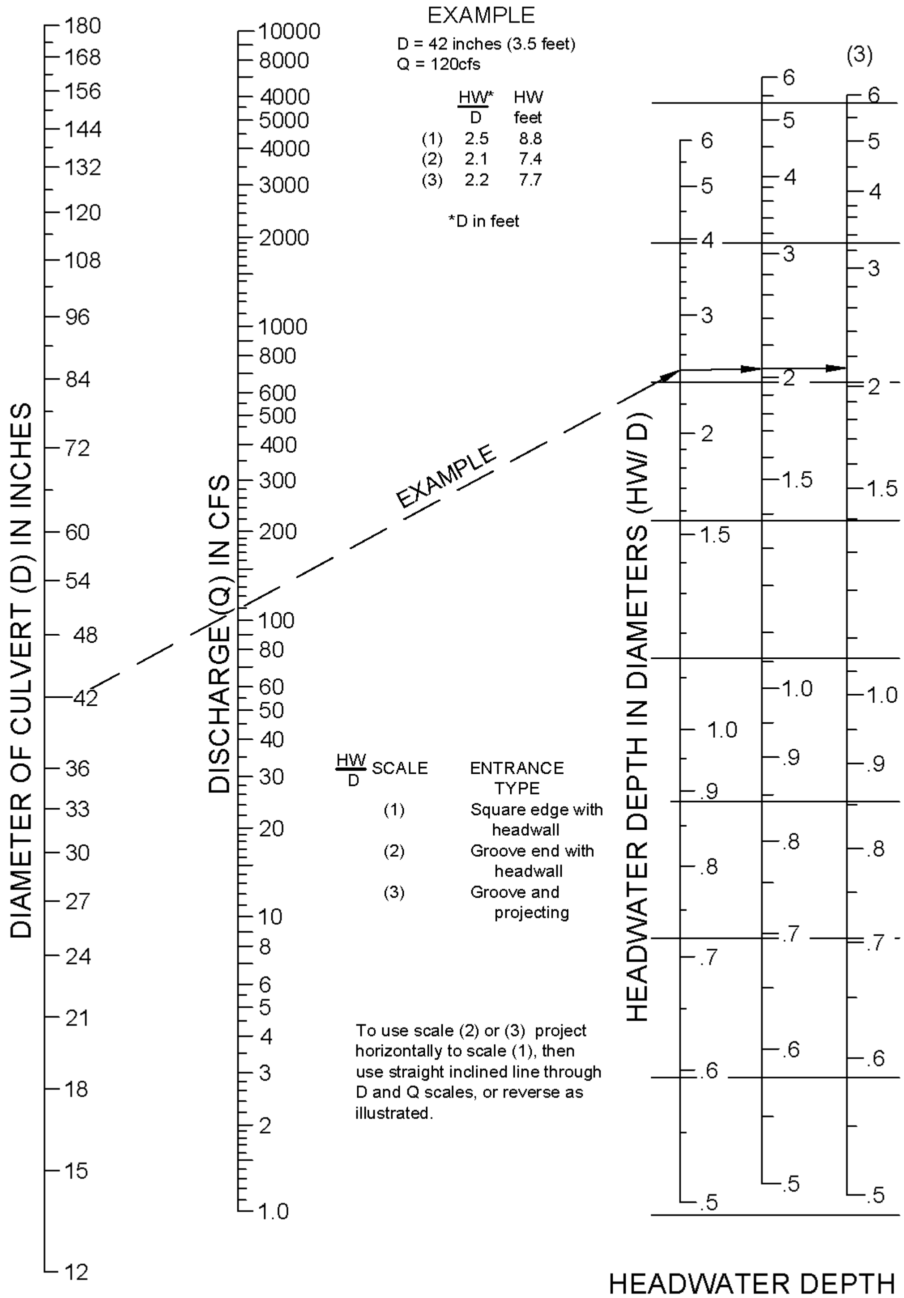
Culverts are hydraulically short conduits typically used to convey surface water through a highway or railroad embankment, or other type of obstruction. Culverts are usually designed to take advantage of submergence in order to increase the capacity of the conduit. This Manual provides only a basic level of information on culvert design criteria and design procedures. The Federal Highway Administration's Hydraulic Design of Highway Culverts (Hydraulic Design Series No. 5, 2001) provides further information on culvert design.

5.1. General Design Criteria

Culvert design shall follow the same principles of storm drain design with the following exceptions.

5.1.1 Hydraulic Criteria

Culverts shall be designed to convey the peak 100-year design flow for public roads. When outlet velocities exceed permissible velocities for the outlet channel, suitable outlet protection (e.g., energy dissipation or channel lining) shall be provided (see Figures 5–1 and 5–2 for inlet and outlet control nomographs).



HEADWATER SCALES 2 & 3
REVISED MAY 1964

HEADWATER DEPTH FOR
CONCRETE PIPE CULVERTS
WITH INLET CONTROL

BUREAU OF PUBLIC ROADS
JAN. 1963

Figure 5-1. Sample Inlet Control Nomograph

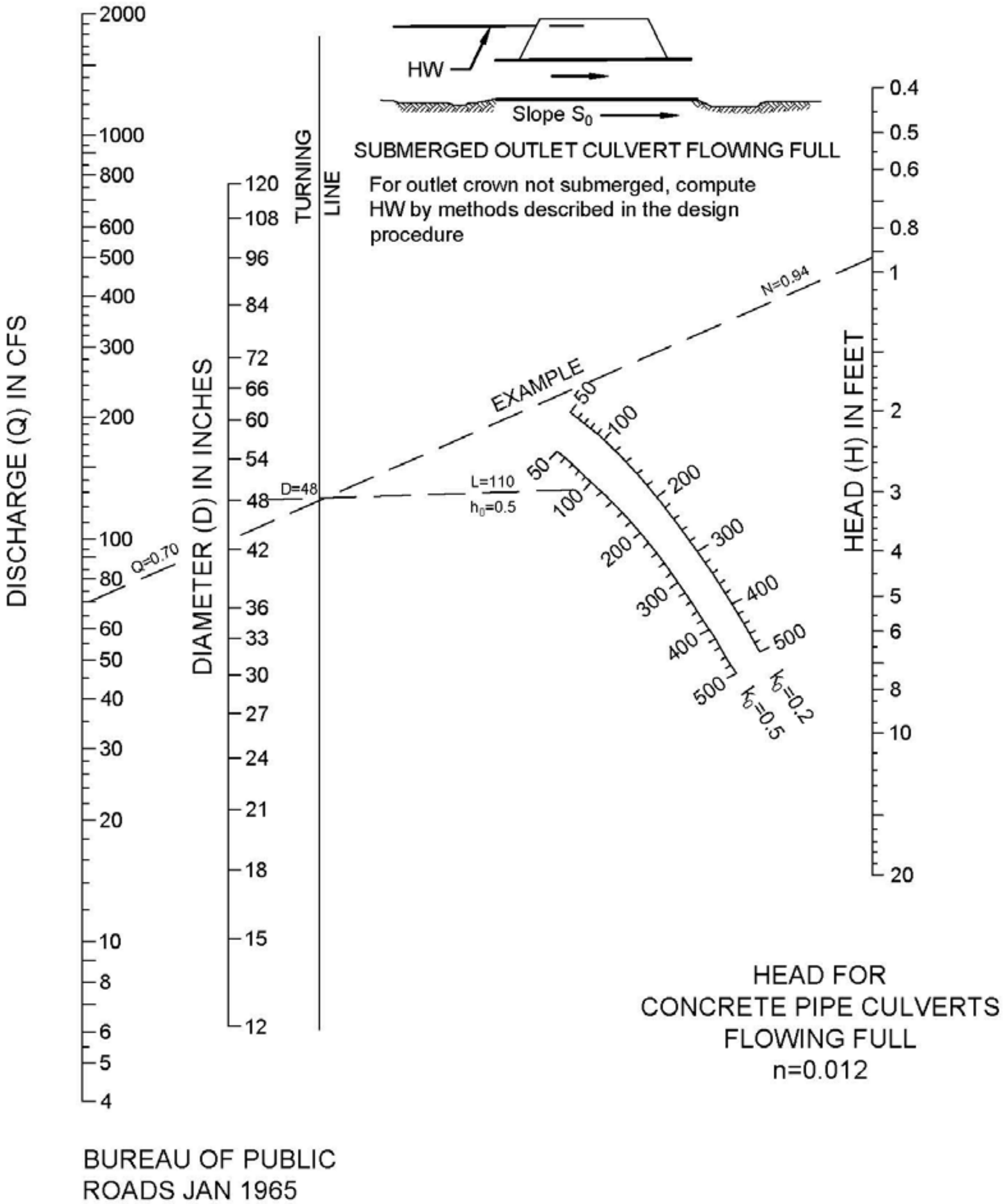


Figure 5-2. Sample Outlet Control Nomograph

For culvert facilities within the public right-of-way, the minimum culvert size shall be an 18-inch diameter round pipe or a 4-foot (4') tall box culvert. This Manual generally references its culvert design criteria and procedures to circular and rectangular cross-sections. These criteria and procedures can be adapted to equivalent cross-sections (e.g., arches and other non-circular or non-rectangular shapes) with due care. Culverts with non-standard cross-section shapes must also adhere to the design criteria in this Manual. Multiple barrel culverts are acceptable, so long as each barrel meets minimum gradient and velocity criteria.

Appendix C provides a table of recommended Manning Roughness coefficients for culverts and Appendix E provides nomographs for several culvert categories.

Culvert headwater elevations shall maintain a freeboard of at least one foot below the roadway crest and the finished floors of structures within the zone influenced by the culvert headwater. When a culvert crossing increases the existing limits of flooding, the project owner shall obtain appropriate documentation from affected property owners as required by the City. Figure 5–3 provides a definition sketch for a typical culvert installation. A culvert headwall or other slope protection is required when the headwater elevation exceeds the top of the culvert conduit.

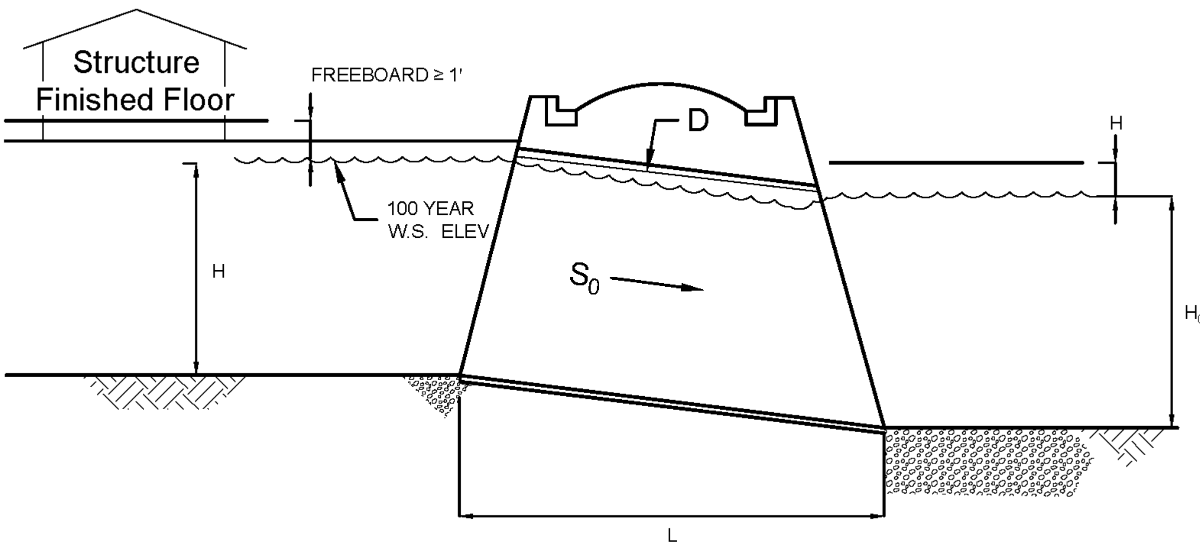


Figure 5-3. Definition Sketch for Culverts

Culverts shall follow the alignment and grade of the natural channel whenever possible. In cases where the barrel cannot be aligned with the channel flow line, the angle of flow approaching the inlet shall be less than 90 degrees and the additional head loss due to approach angle shall be accounted for at the entrance of the culvert (see Table 4–11).

5.1.1.1 Low-Water Crossings/ "Arizona Crossings"

Low-water crossings (also known as dip crossings, at-grade crossings, or "Arizona crossings") are subject to City approval.

Chapter 6

Site Drainage

Site drainage refers to any storm water conveyance system used for the transport of runoff generated from a City-owned parcel rather than from the public right-of-way. Drains for the sole purpose of transporting building drainage are not considered Site Drainage and are not subject to these requirements.

6.1. General Design Criteria

Site drainage shall comply with the rest of this Manual with the following exceptions:

1. Storm drains used for site drainage may use 8” or 12” diameter pipe if these sizes have the capacity to convey discharge from the Design Storm frequency established in Section 2.2.
2. The number of inlets and pipes shall be minimized by using the surface slope and cross gutters to direct flow.
3. For 8” diameter pipes, a modified sewer lateral cleanout may be used. The maximum distance between these types of cleanouts is 200 ft.

THIS PAGE INTENTIONALLY LEFT BLANK FOR DOUBLE-SIDED PRINTING

Chapter 7

Open Channels

This chapter presents technical standards and design criteria for the evaluation and design of natural and engineered open channels. It also discusses design standards for various channel types that might be encountered or used in the City of San Diego. This Manual provides only the minimum hydraulic standards for channel evaluation and design.

This Manual discusses open-channel design procedures in the context of channels with rectangular and trapezoidal cross-sections. The criteria and methods can be adapted to other cross-section shapes (e.g., parabolic or composite shapes) with due care.

The ultimate responsibility for a functional channel design lies with the engineer in charge of the design. The execution of this responsibility may require additional analyses and stricter design standards than the minimum standards presented in this Manual.

Lifetime maintenance is critical to maintain the hydraulic capacity and intended function of an open channel facility. Special attention shall be taken to provide appropriate safety measures (e.g., gates, guardrails, and/or fencing) and maintenance access for open channels.

7.1. Types of Open Channels

Open channels can be categorized as either natural or engineered. Natural channels include all watercourses that are carved and shaped naturally. Engineered channels are those constructed by human efforts. Open channels can be separated into six different types, and these types may be combined:

1. Natural or Designed Alluvial-Bed Channels
2. Grass-Lined Channels
3. Wetland-Bottom Channels
4. Riprap-Lined Channels
5. Concrete-Lined Channels
6. Other Channel Linings

7.1.1 Natural and Designed Alluvial-Bed Channels

In general, a natural channel continually changes its position and shape within an alluvial valley because of hydraulic forces acting on its bed and banks. When feasible, natural channels shall be kept undisturbed and new development shall be placed at least one foot (1 ft) above 100-year water surface elevations.

A designed alluvial bed channel is a conveyance system designed and constructed to mimic the characteristic of a natural alluvial channel. Designed alluvial bed channels are allowed to change position and shape within channel banks and most often involve a composite cross-section with low-flow channel and floodplain components.

7.1.2 Grass-Lined Channels

Grass-lined channels (see Figure 7–1) are engineered channels with grass or other short-stemmed vegetation lining its bottom and sides. Grass-lined channels are often considered the most desirable “rigid” engineered channels from an aesthetic viewpoint. The channel storage, lower velocities, and the sociological benefits of grass-lined channels often create significant advantages over other types of engineered channels. The grass cover can stabilize the channel side slopes, minimize erosion of the channel surface, and control the movement of soil particles along the channel bottom. Low-flow channels within the grass-lined channel may need to be concrete or rock-lined to minimize erosion and maintenance. Section 7.4 provides specific design criteria for grass-lined channels.

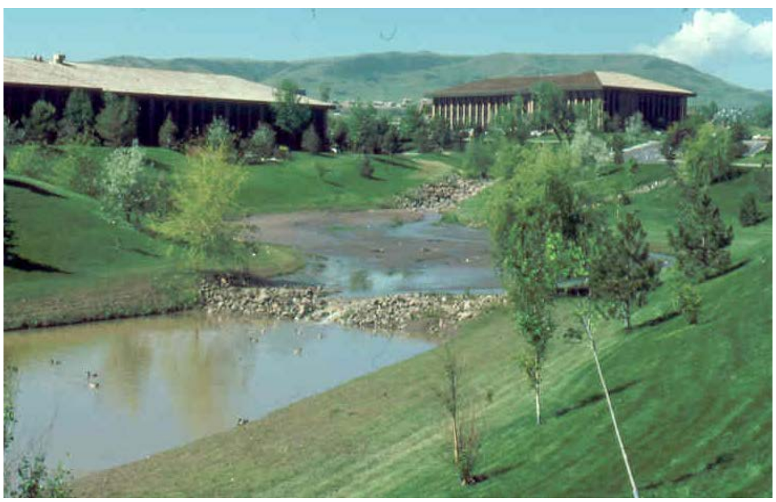


Figure 7-1. Example of a Grass-Lined Channel

7.1.3 Wetland-Bottom Channels

Wetland-bottom channels (see Figure 7–2) utilize vegetative components and other natural materials in combination with structural measures to construct natural-like channels that are stable and resistant to erosion. Wetland-bottom channels provide channel storage, slower velocities, and various multiple use benefits. Wetland bottom channels encourage the development of wetlands or certain types of riparian vegetation in the channel bottom. The potential benefits associated with a wetland bottom channel include habitat for aquatic, terrestrial, and avian wildlife and water quality enhancement as the base flows move through the marshy vegetation. The potential benefits of wetland-bottom channels should be measured against additional land requirements, maintenance requirements, and potential vector problems. Section 7.5 provides specific design criteria for wetland-bottom channels.

Figure 7-2. Example of a Wetland-Bottom Channel

7.1.4 Riprap- or Cobble-Lined Channels

Riprap-lined channels (see Figure 7–3) are channels in which riprap is used for lining the channel banks and bottom. Riprap is a popular choice for erosion protection because the initial installation costs are often less than alternative methods for preventing erosion. However, the design engineer must be mindful that there are additional costs associated with riprap erosion protection since riprap installations require periodic inspection and maintenance. Riprap-lined channels may be permitted in areas of existing development where existing right-of-way limitations preclude the use of vegetated channels. Riprap lining might be appropriate in cases where:

1. major flows such as the 100-year flood are found to produce channel velocities in excess of allowable non-eroding values,
2. channel side slopes must be steeper than 3H:1V,
3. low flow channels are used, and
4. rapid changes in channel geometry occur such as channel bends and transitions.

Figure 7-3. Example of Riprap-Lined Channel

Section 7.6 provides specific design criteria for riprap-lined channels.

7.1.5 Concrete-Lined Channels

Concrete-lined channels (see Figure 7–4) are rectangular or trapezoidal channels in which reinforced concrete is used for lining the channel banks and/or bottom. Concrete-lined channels may be permitted only where existing right-of-way restrictions preclude the use of other channel types. These will be approved on a case-by-case basis. Section 7.7 provides specific design criteria for concrete-lined channels.

Figure 7-4. Example of Concrete-Lined Channel

7.1.6 Other Types of Channel Lining

There are various engineered channel liners available to the design engineer. These include gabion boxes and mattresses, cable-stayed articulated concrete block mattresses (ACB), interlocked concrete blocks, cobble stone, concrete revetment mats formed by injecting concrete into doublelayer fabric forms, and various types of synthetic fiber liners. Section 7.8 discusses design criteria for engineered channel liners.

7.1.7 Selection of Channel Type

This Manual does not have preference for or prejudice against any particular channel-lining system as long as it is properly evaluated on its merits. Each type of channel must be evaluated for its longevity, integrity, maintenance requirements and costs, and general suitability for community needs, among other factors. Selection of a channel type that is most appropriate for the conditions that exist at a project site shall be based on a multi-disciplinary evaluation that may include hydraulic, structural, environmental, sociological, maintenance, economic, and regulatory factors.

7.2. General Design Criteria

This Section presents general hydraulic design standards that are applicable to all improved channels. The specific requirements for a particular type of channel may be stricter than the general design criteria outlined in this Section.

7.2.1 Hydraulic Capacity

All new open channels shall be designed, at a minimum, to safely confine and convey the runoff from the 100-year design event. The City of San Diego prefers that flows that can be conveyed in a 48-inch diameter pipe or smaller be conveyed within an underground conduit rather than an open channel.

7.2.2 Manning Roughness Coefficient

Appendix C provides recommended values for the Manning roughness coefficient for various channel and overbank types and conditions. Manning roughness coefficients for riprap channels shall be computed based on the criteria outlined in Section 7.6.2.

7.2.3 Uniform Flow

Open channel drainage systems shall be designed assuming uniform flow conditions. Section 7.9.1 presents the uniform flow equation and methods for calculating uniform flow.

7.2.4 Vertical and Horizontal Alignment

Open channels shall have a minimum longitudinal gradient of 0.5 percent whenever practical. Flatter grades may be approved with prior consultation with the City. Open channels with grades flatter than 0.5 percent shall have provisions for the drainage of nuisance low flows.

Horizontal alignment changes of two degrees or less may be accomplished without the use of a circular curve for subcritical flow designs (FR <1.0, see 7.9.3). Curves must be used for supercritical flow designs (FR >1.0), no matter the degree of change in horizontal alignment. Curved channel alignments shall have superelevated banks in accordance with Section 7.9.4.

Spiral transition curves shall be used upstream and downstream of curves for supercritical channel designs with reverse curves or horizontal alignments with consecutive circular curves. Spiral curves may also be used to reduce required superelevation allowances and cross-wave disturbances.

7.2.5 Maximum Permissible Velocity

The design of open channels shall be governed by maximum permissible velocity. This design method assumes that a given channel section will remain stable up to a maximum permissible velocity, provided that the channel is designed in accordance with the standards presented in this Manual. Table 7–1 presents the maximum permissible velocities for several types of natural, improved, unlined, and lined channels.

Table 7-1. Maximum Permissible Velocities for Lined and Unlined Channels

Material or Lining	Maximum Permissible Average Velocity* (ft/sec)
Natural and Improved Unlined Channels	
Fine Sand, Colloidal	1.50
Sandy Loam, Noncolloidal	1.75
Silt Loam, Noncolloidal	2.00
Alluvial Silts, Noncolloidal	2.00
Ordinary Firm Loam	2.50
Volcanic Ash	2.50
Stiff Clay, Very Colloidal	3.75
Alluvial Silts, Colloidal	3.75
Shales and Hardpans	6.00
Fine Gravel	2.50
Graded Loam to Cobbles when Noncolloidal	3.75
Graded Silts to Cobbles when Colloidal	4.00
Coarse Gravel, Noncolloidal	4.00
Cobbles and Shingles	5.00
Sandy Silt	2.00
Silty Clay	2.50
Clay	6.00
Poor Sedimentary Rock	10.0
Fully-Lined Channels	
Unreinforced Vegetation	5.0
Reinforced Turf	10.0
Loose Riprap	per Table 7-3
Grouted Riprap	25.0
Gabions	15.0
Soil Cement	15.0
Concrete	35.0

*Maximum permissible velocity listed here is basic guideline; higher design velocities may be used, provided appropriate technical documentation from manufacturer.

Regardless of these maximum permissible velocities, the channel section shall be designed to remain stable at the final design discharge and velocity. The design flow may not always yield the highest flow velocity. Therefore, best practice is to confirm channel section stability during events smaller than the design flow. This may be accomplished by evaluating flows of specific more frequent storm events (e.g., 10-year, 2-year, etc.) or testing successive fractions (e.g., one-half, one-quarter, and further if necessary) of the design flow. Only calculations for the full design flow are required to be submitted for review.

Additional geotechnical and geomorphological investigation and analyses may be required for natural channels or improved unlined channels to verify that the channel will remain stable based on the maximum design velocities.

7.2.6 Subcritical and Supercritical Flow

Flow can be classified as critical, subcritical, or supercritical according to the level of energy in the flow. This energy is commonly expressed in terms of a Froude Number (FR) and critical depth (d_c). Section 7.9.2 discusses the characteristics of critical flow and describes methods for determining Froude Number and critical depth. All channel design submittals shall include the calculated Froude Number (FR) and critical depth (d_c) for each unique reach of channel to identify the flow state and verify compliance with these criteria.

Flow at or near the critical state (FR=1.0 or $d=d_c$) is unstable. As a result, minor factors such as channel debris have the potential to cause severe and acute changes in flow depth. Channels shall be designed to convey their design flow following the flow energy limitations described in Table 7–2. When necessary to convey flows at or near critical state (0.80< FR <1.20), flow instabilities may be accommodated by providing additional freeboard.

Table 7-2. Limitations on Flow Energy for Rectangular and Trapezoidal Channels

Design Flow Condition	Froude Number
Subcritical	FR <0.80
Supercritical	FR >1.20

In rare cases, the specific energy relationship of a cross-section might result in a situation where flows less than the design flow may have a greater depth than the depth calculated for the design flow. The design engineer shall check supercritical channel designs to evaluate whether the channel will maintain freeboard requirements (Section 7.2.7) during flows less than the design flow (see suggested method in Section 7.2.5).

7.2.7 Freeboard

In the context of this Manual, freeboard is the additional height of a flood control facility (e.g., channel, levee, or embankment) measured above the design water surface elevation. All channel linings shall extend to the design freeboard height. In this way, the freeboard will provide a factor of safety when designing open channels. Freeboard shall be calculated using the maximum Manning roughness coefficient expected during the lifetime of the channel. Unless other information justifies a lower roughness value, the design engineer may assume the maximum lifetime channel roughness to be n=0.150.

Open channel facilities conveying a design flow of less than 10 cfs shall have a minimum freeboard of 0.5 feet. Open-channel facilities conveying a design flow of 10 cfs or more shall have a minimum freeboard of 1.0 foot, with allowances for velocity, super-elevation, standing waves, and/or other water surface disturbances such as slug flow. Sections 7.9.3 and 7.9.4 provide design methods for calculating these allowances. Equations 7–1 and 7–2 describe the minimum design freeboard for subcritical and supercritical flow designs, respectively.

Equation 7-1. Minimum Design Freeboard for Subcritical Flow Designs

$$(h_{fr})_{SUBCRITICAL} = \max \begin{cases} 1.0 \\ 0.5 + \dfrac{v^2}{2g} + \dfrac{Cv^2T_W}{rg} + \Delta y \end{cases}$$

where:

h_{fr}	=	minimum required freeboard (ft)
v	=	flow velocity (ft/s)
g	=	gravitational acceleration (32.2 ft/s²)
$\dfrac{Cv^2T_W}{rg}$	=	superelevation allowance (ft), see Section 7.9.5
Δy	=	allowances for other hydraulic phenomenon (ft), (e.g., standing waves, slug flow – see Section 7.9.4.3)

Equation 7-2. Minimum Design Freeboard for Supercritical Flow Designs

$$(h_{fr})_{SUPERCRITICAL} = 1.0 + 0.025vd^{1/3} + \frac{Cv^2T_W}{rg} + \Delta y$$

where:

h_{fr}	=	minimum required freeboard (ft)
v	=	flow velocity (ft/s)
g	=	gravitational acceleration (32.2 ft/s²)
$\dfrac{Cv^2T_W}{rg}$	=	superelevation allowance (ft), see Section 7.9.5
Δy	=	allowances for other hydraulic phenomenon (ft), (e.g., standing waves, slug flow – see Section 7.9.4.3)

Superelevation allowance is a function of flow velocity, channel geometry, and channel alignment. Applying transition curves to the alignment may reduce the required superelevation allowance. Section 7.9.5 discusses the calculations of superelevation allowance in more detail. The superelevation allowance shall be applied to both banks of the channel. The superelevation allowance shall be applied to channel bends in the in the following manner:

1. Begin at a point five times the characteristic wavelength of the design flow (5LW), measured from the downstream tangent point of the curve, with no superelevation allowance.
2. Taper uniformly to the full superelevation allowance at a point three times the characteristic wavelength of the design flow (3LW), measured from the downstream tangent point of the curve.
3. Maintain the full superelevation allowance through the curve.
4. Continue the top of bank elevation level from the upstream tangent point of the curve to its intersection with the normal top of bank.

Figure 7–5 illustrates freeboard superelevation allowance.

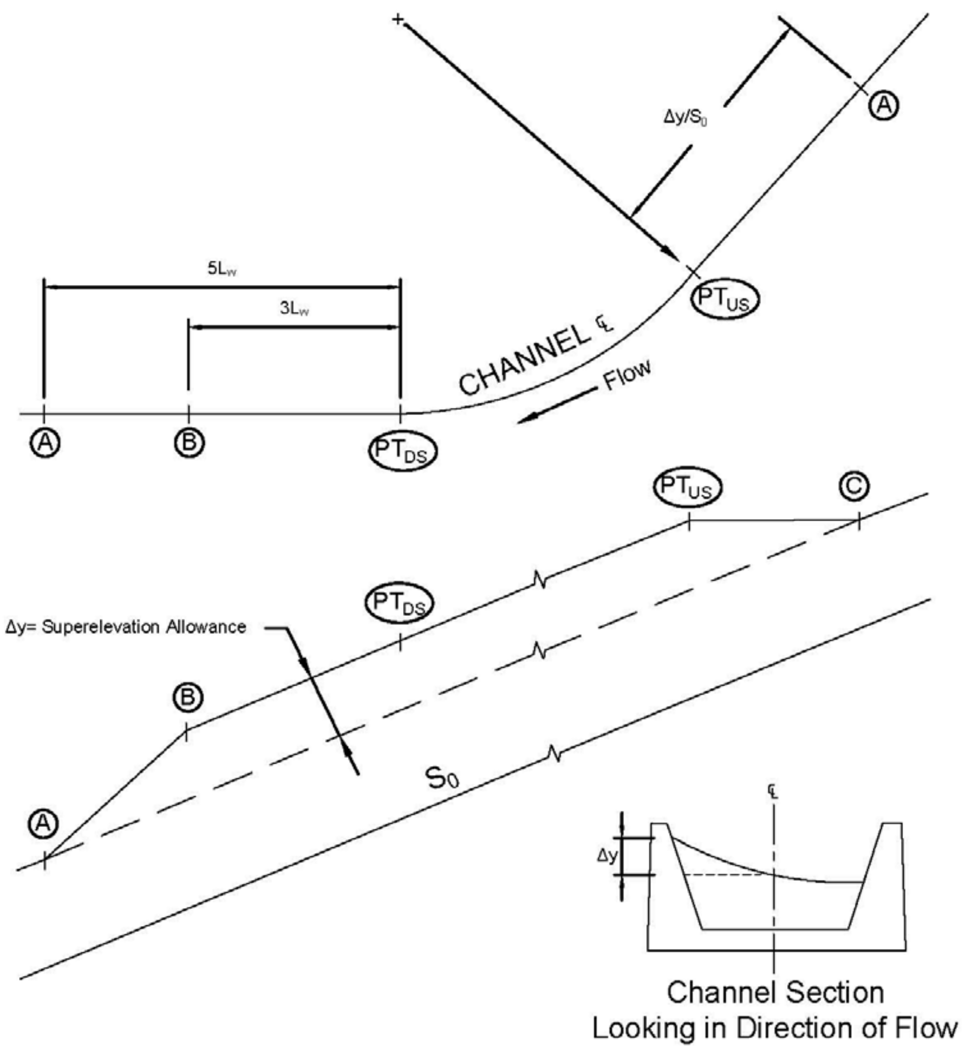


Figure 7-5. Layout for Freeboard Superelevation Allowance

Equation 7–3 and Equation 7–4 describe the characteristic wavelengths for subcritical and supercritical flow, respectively.

Equation 7-3. Characteristic Wavelengths for Subcritical Flow

$$(L_W)_{SUBCRITICAL} = 2T_W$$

where:

L_W	=	characteristic wavelength (ft)
T_W	=	top width of water surface (ft)

Equation 7-4. Characteristic Wavelengths for Supercritical Flow

$$(L_W)_{SUPERCRITICAL} = 2T_W\sqrt{F_R^{\,2}-1}$$

where:

L_W	=	characteristic wavelength (ft)
T_W	=	top width of water surface (ft)
FR	=	Froude Number (no dimension)

The freeboard under the lowest chord of bridge deck (i.e., the soffit elevation) shall be a minimum of 2 feet during the 100-year design event. In cases where the bridge has been designed to withstand hydraulic forces of floodwaters and impact from large floating debris, the water surface elevation upstream of the bridge shall maintain a freeboard of at least one foot below the roadway crest and the finished floors of structures within the zone influenced by the bridge headwater. When a bridge crossing increases the existing limits of flooding, the project owner shall obtain appropriate documentation from affected property owners as required by the City.

This Manual only describes the City of San Diego's minimum freeboard requirements for open channel design. Major drainage ways involving road crossings or other types of crossings, streams that the FEMA has mapped as Special Flood Hazard Areas, or facilities that interface with Caltrans facilities might have significantly different freeboard requirements.

7.2.8 Flow Transition

Channel transitions occur in open channel design whenever there is a change in channel slope or shape and at junctions with other open channels or storm drain. Properly designed flow transitions mimic the expansion or contraction of natural flow boundaries as best as possible, as well as minimize surface disturbances from cross-waves and turbulence. Drop structures and hydraulic jumps are special transitions where excess energy is dissipated by design. Transitions in open channels are generally designed for either subcritical or supercritical flow transitions.

Hydraulic jumps shall be designed to take place only within energy dissipation or drop structures and not within an erodible channel. Subcritical transitions shall satisfy the minimum transition lengths described in Section 7.9.3. Supercritical transitions shall satisfy the minimum transition lengths described in Section 7.9.4. Special transitions such as drop structures and hydraulic jumps shall satisfy the specifications described in Section 7.11.

7.2.9 Access and Safety

7.2.9.1 Access

Any easement encompassing a channel shall be wide enough to provide for the channel structure and adequate maintenance access. Easements shall be placed on one side of lot or ownership lines in new developments and in existing development where conditions permit.

1. The minimum width of any channel easement shall be the top width of channel plus 4 feet on each side of the channel.
2. Channels with a top width of less than 40 feet require a minimum 15-foot wide service road parallel to one side of the channel and a 4-foot wide access on the opposite side.
3. Channels 40 feet or more in top width require a minimum 15-foot wide service road on both sides of the channel.

Service roads parallel to a channel facility may be omitted when the lack of a service road is not considered detrimental to the maintenance and integrity of the channel. The following are examples of circumstances where service roads parallel to the channel facility may be omitted:

1. Channels with a bottom width of 8 feet or less, with a maximum design flow depth of 2 feet.
2. Channels with a bottom width of more than 8 feet and a maximum design flow depth of more than 2 feet may omit access roads parallel to the channel when suitable exit-entry ramps are provided at street crossings and at other locations to facilitate travel of maintenance vehicles in the channel bottom. At a minimum, one access ramp must be provided at each end of the channel.
3. Vehicular access and loading points from the access road parallel to the channel shall be provided at intervals of 300 feet or less. Access easements must be at least 15 feet wide and have a maximum grade of five percent (5%). Access and loading points must be level and not super elevated. At the end of every access road, a minimum 30-foot wide by 30-foot wide area shall be provided to facilitate a three-point turn around. No utilities will be allowed within the channel or access and loading points. Access ramps shall be in the down-gradient direction. Access ramps designed for personnel access shall have a maximum slope of five percent (5%).

In addition to access paths, staging areas must also be reserved when constructing a new channel. For channels between 4 and 20 feet in bottom width, a staging area of 1000 square feet must be reserved (minimum dimension for each side is 20 feet). For channels greater than 20 feet in bottom width, a staging area of 2000 square feet is required (minimum dimension for each side is 30 feet).

New channels must have a gated concrete access ramp from the public right-of-way into the channel bottom. The ramp must be 15 feet wide and with a maximum grade of four percent (4%).

No hardscape or landscape will be permitted over any dedicated access easement and or along the top longitudinal distance of the top of channel access roads and loading points.

Bridges Over Channels

All proposals for bridges over channels must be submitted to and approved by the channel asset owner. To provide access to all necessary equipment, minimum clearance height from the bottom of bridge to finish grade must be at least 20 feet, columns must not be within 20 feet from of one another, and columns must be at least 10-feet from the toe of slope. Bridges must be structurally designed so they do not load the channel banks in any way.

7.2.9.2 Safety

Specific safety requirements shall be determined on a case-by-case basis in consultation with the City. At a minimum, guardrails or other approved traffic barriers as described in the relevant Traffic Manual shall be provided when a channel is located next to traffic.

Fencing or access barriers, as required by the City, is required for channels abutting residential developments, schools, parks, and pedestrian walkways as follows:

1. Fencing is required for all concrete Concrete-lined or rip rapped channels where the design frequency storm produces a velocity that exceeds 5 feet per second and 2 feet in depth or a combination thereof for a factor of ten (10) within five feet of the water's edge during design flow conditions. This requirement does not apply to brow ditches.
2. Fencing or access barriers are required for all unlined Alluvial-bed, grass-lined, and wetland-bottom channels with side slopes steeper than 4H:1V where the design frequency storm produces a velocity that exceeds 5 feet per second and 2 feet in depth or a combination thereof for a factor of ten (10) within five feet of the water's edge during design flow conditions.

Gates shall be provided for maintenance and emergency access at regular intervals, with 20-foot wide gates placed 1,000 feet on center and 4-foot gates placed 500 feet on center or portion thereof. Fencing or access barriers shall be located a minimum of 6 inches inside the easement boundary lines unless otherwise approved.

7.2.10 Environmental Permitting

Open channel facilities are often located within or adjacent to sensitive environmental areas. The design engineer must investigate which permits might be necessary from various Agencies, including but not limited to: U.S. Army Corps of Engineers (e.g., Section 404 Wetland Permit), U.S. Department of Fish and Wildlife, California Department of Fish and Game (e.g., Section 1600 Permit), California State Water Resource Control Board and Regional Water Quality Control Board (e.g., Section 401 Water Quality Certification), and California Coastal Commission. It is important that the final permits and/or permit conditions allow for the future and perpetual maintenance of a channel facility without the necessity of returning to a permitting Agency for regular maintenance activities.

7.2.11 Maintenance

Where failure of an open-channel facility might cause flooding of a public road or structure, the facility shall have an operation and maintenance plan. These operation and maintenance plans shall specify regular inspection and maintenance at specific time intervals (e.g., annually before the wet season) and/or maintenance "indicators" when maintenance will be triggered (e.g., vegetation more than 6 inches in height). Operation and maintenance plans shall ensure that vegetation is removed or maintained on a regular basis to maintain the function of the facility.

Flood control channels require lifetime maintenance. The project owner and design engineer shall consult the City for determination of which maintenance mechanism is required for a particular project. At a minimum, privately owned and maintained open channel facilities shall have a recorded easement agreement with a covenant binding on successors or other mechanism acceptable to the City.

7.3. Design Criteria: Stabilization of Existing Channels

Open channels are important drainage elements that contribute to the image and livability in an urban environment. The areas around open channels may have multiple uses that integrate trails, open space corridors, and wildlife habitat.

7.3.1 Existing Channels and Channel Stabilization

When designing a project that affects an existing drainage course, the design engineer shall evaluate the potential project effects on velocity, flow variation, width, depth, change in slope, scour, deposition, vegetation growth, effects to tributary streams, and other effects due to permanent structures within the channel (e.g., bridge piers) or other project conditions (e.g., changes to peak discharge). The project design shall evaluate the long-term effects within the project boundaries as well as upstream and downstream of the project, and channel alterations or channel stabilization shall be incorporated into the project when deemed necessary. Engineering judgment, the development process, and the guidelines of the governing Agency will determine the requirements for the stabilization of drainage channels.

Natural channels shall remain in their natural state whenever practicable. New development shall be set back from natural floodplains when practicable, and placed at least one foot above 100-year water surface elevations. Because some natural channels tend to migrate horizontally, the required setback distance shall be determined through appropriate hydraulic and scour analyses that evaluate the stability, flow velocity, and materials of the subject drainage way. All modifications within the 100-year floodplain shall be done in accordance with currently adopted floodplain development and management regulations and guidelines of the governing Agency.

The design engineer shall identify the capacity of a natural drainage way relative to a 100-year design event and delineate areas inundated by the 100-year design event, and assess the relative channel stability in both plan and profile.

When the stability analysis demonstrates that either bank erosion outside of the designated flow path (as defined by flowage easement and/or right-of-way) or channel degradation is likely to occur, then an analysis of the magnitude and extent of the erosion may be necessary. In such a condition, the design engineer shall confer with the governing Agency to determine:

1. Additional analysis necessary (if any) to estimate the potential extent of lateral and vertical channel movements;
2. The potential risk to the proposed development from channel degradation and/or bank failure;
3. Solutions and/or remedies available to mitigate the potential risk due to the channel instability.

7.3.2 Bank-Lined Channels

Bank-lined channels are a type of channel stabilization where the banks are lined but the channel bottom remains in a natural state with minimal regrading. Figure 7–6 illustrates an example of a bank-lined channel. Bank-lined channel designs attempt to minimize scour of the channel bottom at the bank lining interface as well as maintaining a stable natural channel. The bank lining shall extend below calculated scour depths at the lining interface and provide the minimum freeboard as outlined in Section 7.2.7

Figure 7-6. Example of Bank-Lined Channel

7.3.3 Bio-Engineered Channel Stabilization

Traditionally, "hard lined" channel stabilization techniques (e.g., riprap, gabion, concrete, etc.) have been used to stabilize erosion problem areas. Bioengineering integrates structural, biological, and ecological principles to construct living structures (plant communities) for erosion, sediment, and flood control purposes. In many instances, bio-engineered channel stabilization measures can be safely utilized in place of "hard lined" measures and these methods are encouraged whenever practical. Successful application of bio-engineered stabilization measures depends on accurate diagnosis of the causes of channel stability problems, rather than just treating visible problem areas. The reference section of this Manual provides useful resources on bioengineered solutions for natural channel stabilization.

7.4. Design Criteria: Grass-Lined Channels

This section presents minimum design criteria for grass-lined channels. The design engineer is responsible for confirming that a channel design meets these criteria, the general open-channel criteria outlined in Section 7.2 and any special considerations for a particular design situation.

7.4.1 Longitudinal Channel Slopes

Grass-lined channel slopes are dictated by maximum permissible velocity requirements. Where the natural topography is steeper than desirable, drop structures may be utilized to maintain design velocities. Grass-lined channels shall have a minimum longitudinal gradient of 0.5 percent whenever practical (see Section 7.2.4).

7.4.2 Roughness Coefficient

Appendix C (Table C–5) provides appropriate Manning roughness coefficients for grass-lined channels. The Manning roughness coefficient used in evaluating channel capacity shall assume a mature channel (i.e., substantial vegetation with minimal maintenance). For evaluating channel slope and permissible velocity, the Manning roughness coefficient shall assume a freshly mowed condition.

7.4.3 Low Flow and Trickle Channels

Waterways that are normally dry prior to urbanization will often have a continuous flow after urbanization because of lawn irrigation return flows from both overland and from ground water in-flow. Since continuous flow over grass will destroy a grass stand and may cause the channel profile to degrade, a **trickle channel** is required on all urban grass-lined channels. Usually, concrete trickle channels are preferred because of their ease of maintenance. Other types of trickle channels, such as rock-lined trickle channels, are acceptable if they are properly designed. Trickle channels are not appropriate for grass-lined channels intended for water quality treatment.

Trickle channels may not be practical on larger streams and rivers, or in channels located on sandy soils. In these cases, a **low flow channel** may be a more appropriate choice.

7.4.3.1 Trickle Channels

Trickle channels are recommended for grass-lined channels with a 100-year design flow less than or equal to 200 cfs. The trickle channel capacity shall convey 5 percent of the 100-year design discharge or 5 cfs, whichever is greater. There is no freeboard requirement for trickle-channels. The flow capacity of the main channel shall be determined without considering the flow capacity of the trickle channel. Care shall be taken to ensure that low flows enter the trickle channel without flow paralleling the trickle channel or bypassing the inlets.

Trickle channels are not typically required for swales and other grass-lined channels conveying a 100-year peak runoff of 20 cfs or less. For these smaller channels, the design engineer shall evaluate the factors such as drainage slope, flow velocity, soil type, and upstream impervious area and specify a trickle channel when needed based on their engineering judgment.

Concrete Trickle Channel

Concrete trickle channels (see Figure 7–7) can help prevent erosion, silting, and excessive plant growth. Concrete trickle channels are not appropriate for wetland-bottom channels (see Section 7.5) or swales intended for water quality treatment. The concrete trickle channel shall have a minimum depth of 6 inches. A Manning roughness coefficient value of n=0.015 will be used to design the concrete trickle channel. At a minimum, concrete trickle channels shall be 6 inches thick with #4 reinforcement 12 inches on-center in each direction.

Figure 7-7. Example of Concrete Trickle Channel

Rock-Lined Trickle Channel

Rock-lined trickle channels shall have a minimum depth of 12 inches with the Manning roughness coefficient determined as described in Section 7.6.17. The minimum stone size for rock-lined trickle channels shall be 6 inches.

7.4.3.2 Low-Flow Channels

Low-flow channels are used to contain relatively frequent flows within a recognizable channel section. Low-flow channels are recommended for channels with a 100-year flow greater than 200 cfs. At a minimum, they shall have the capacity to convey the 2-year flow event with no freeboard. The overall flow capacity of the channel shall include the capacity of the low flow channel.

Low-flow channels shall have a minimum depth of 12 inches. The side slopes of the low-flow channel shall be 2.5H:1V to 3H:1V whenever practical. The main channel depth limitations (Section 7.4.5) do not apply to the low-flow channel area of the overall channel cross-section.

7.4.4 Bottom Width

The selection of the overall channel bottom width shall consider factors such as possible wetland mitigation requirements, constructability, channel stability and maintenance, multi-use purpose, and width of the low-flow channel.

7.4.5 Freeboard and Flow Depth

Swales and grass-lined channels conveying a 100-year flow less than or equal to 10 cfs shall have a minimum freeboard of 6 inches. Grass-lined channels conveying larger discharges shall meet the minimum freeboard requirements outlined in Section 7.2.7

The recommended design depth of flow for a grass-lined channel (outside the low flow channel area) is 5 feet for a 100-year flow of 1,500 cfs or less whenever practical. Excessive depths shall also be avoided in channels with greater design flows to the maximum extent practicable. Section 7.2.9 discusses access and safety for open channels, including thresholds for flow depth and velocity.

7.4.6 Side Slopes

Side slopes of a grass-lined channel shall be not be steeper than 3H:1V.

7.4.7 Grass Lining

Satisfactory performance of a grass-lined channel depends on constructing the channel with the proper shape and preparing the area in a manner that provides conditions favorable to vegetative growth. Between the time of seeding and the actual establishment of the grass, the channel is unprotected and subject to considerable damage unless interim erosion protection is provided. Jute, plastic, paper mesh, or hay mulch may be used to protect the waterway until the vegetation becomes established.

The grass lining for channels may be seeded or sodded with a grass species that is adapted to the local climate and will flourish with minimal irrigation. Channel vegetation is usually established by seeding. In the more critical sections of some channels, it may be desirable to provide immediate protection by transplanting a complete sod cover. All seeding, planting, and sodding shall conform to local landscape recommendations.

7.4.8 Horizontal Channel Alignment and Bend Protection

The potential for erosion increases along the outside bank of a channel bend due to the acceleration of flow velocities on the outside part of the bend. Thus, it is often necessary to provide erosion protection in natural or grass-lined channels that otherwise would not need protection.

The minimum radius for channels with a 100-year runoff of 20 cfs or less shall be 25 feet. For channels carrying larger flows, horizontal channels alignment shall be limited based on the presence of erosion protection.

No channel bend protection is required along bends where the radius is greater than two times the top width of the 100-year water surface or the channel is constructed in erosion-resistant soils. Channels without bend protection are not allowed to have a curvature with a radius of less than two times the 100-year flow top width or less than 100 feet, whichever is greater.

Channel bends built in areas with erosive soil conditions shall always have erosion protection. When erosion protection is provided, channels are allowed to have minimum radius equivalent to 1.2 times the 100-year flow top width, but in no case shall the radius of curvature be less than 50 feet.

Erosion protection shall extend downstream from the end of the bend a distance that is equal to the length of the bend measured along the channel centerline.

7.4.9 Maintenance

Grass-lined channels shall be maintained to ensure that vegetation is removed or maintained on a regular basis to maintain the function of the facility. The project owner shall ensure that appropriate mechanisms are in place to provide maintenance for the lifetime of the facility.

7.5. Design Criteria: Wetland Bottom Channel

This section presents minimum design criteria for wetland-bottom channels. The design engineer is responsible for confirming that a channel design meets these criteria, the general open-channel criteria outlined in Section 7.2, and any special considerations for a particular design situation.

When designing a wetland-bottom channel, the design engineer must consider both the interim ("new channel") condition and ultimate ("mature channel") condition. For the interim condition, the channel shall maintain non-erosive velocities under the design flow (Section 7.5.1). The design engineer shall evaluate the channel conveyance capacity under ultimate conditions (Section 7.5.2).

7.5.1 Longitudinal Channel Slope

The design engineer shall establish a longitudinal channel slope that maintains non-erosive velocities during the interim condition (i.e., the "new channel" condition), assuming minimal or immature wetland vegetation in the channel bottom. Table 7–1 provides guidelines for maximum permissible velocity. Wetland-bottom channels shall maintain a minimum longitudinal slope of 0.5 percent whenever practicable (see Section 7.2.4).

The design engineer may increase the maximum permissible velocity when temporary erosion control measures are properly installed and maintained during the interim condition. The design engineer may also employ temporary grade control structures to reduce the effective slope of the channel during the interim condition. The Froude Number for wetland-bottom portions of a channel during the interim condition shall not exceed FR =0.7. Where topography is steeper than desirable, permanent drop structures may be used to maintain design velocities.

7.5.2 Roughness Coefficients

Appendix C (Table C–5) provides recommended values for the Manning roughness coefficient for various channel types and overbank conditions. As discussed in Section 7.5.1, a Manning roughness coefficient assuming new channel condition shall be used to determine the longitudinal channel slope. A Manning roughness coefficient representing fully vegetated growth on the channel bottom (i.e., the "mature channel" condition) shall be used to determine channel capacity and evaluate freeboard requirements. Unless other information justifies a different roughness value, the design engineer shall assume a mature channel roughness of n=0.150 (typical of dense riparian vegetation).

7.5.3 Low-Flow and Trickle Channels

Concrete trickle channels are not permitted in wetland bottom channels. Low-flow channels may be used when the 100-year flow exceeds 1,000 cfs. Low-flow channel design shall be as discussed in Section 7.4.3.2.

7.5.4 Bottom Width

The selection of the over-all channel bottom width shall consider factors such as ultimate conveyance requirements, constructability, channel stability, and maintenance.

7.5.5 Freeboard and Flow Depth

Wetland-bottom channels shall meet the minimum freeboard requirements outlined in Section 7.2.7. Whenever practical, excessive depths and velocities shall be avoided for public safety considerations (see Section 7.2.9).

7.5.6 Side Slopes

Side slopes of wetland-bottom channels shall not be steeper than 3H:1V whenever practical. When the side slopes of a wetland-bottom channel are grass-lined, refer to the guidelines provided in Section 7.4.

7.5.7 Horizontal Channel Alignment and Bend Protection

Channel bends shall be designed according to the criteria provided in Section 7.4.8.

7.5.8 Maintenance

Wetland-bottom channels require a maintenance and operation plan, along with appropriate easements and mechanisms for assuring the perpetual maintenance of the facility. The project owner shall ensure that appropriate mechanisms are in place to provide maintenance for the lifetime of the facility.

7.6. Design Criteria: Riprap-Lined Channels

This section presents minimum design criteria for rock riprap-lined channels. Riprap-lined transitions and bends in otherwise non-riprap channels are also considered riprap-lined channels and shall be designed in accordance with the design standards outlined in this section. The design engineer is responsible for confirming that a channel design meets these criteria, the general open-channel criteria outlined in Section 7.2, and any special considerations for a particular design situation.

7.6.1 Longitudinal Channel Slope

The longitudinal slope of riprap-lined channels shall be dictated by maximum permissible velocity requirements. Table 7–3 summarizes the maximum permissible velocity for standard riprap gradations. Where topography is steeper than desirable, drop structures may be used to maintain design velocities (see Section 7.11).

Table 7-3. Channel Bottom Riprap Protection

Design Velocity (ft/s)	Rock Gradation
6–10	No. 2 Backing
10–12	¼ ton
12–14	½ ton
14–16	1 ton
16–18	2 ton
> 18	Special Design

7.6.2 Roughness Coefficients

The Manning roughness coefficient (n) for hydraulic computations shall be estimated for loose rock riprap using the Manning-Strickler equation (Equation 7-5). Equation 7–5 (Chang, 1992) does not apply to grouted rock riprap or to very shallow flow. Table 7–4 provides Manning roughness coefficients for standard rock riprap classifications based on the Manning-Strickler method.

Equation 7-5. Manning-Strickler Equation

$$n=0.0395{d_{50}}^{1/6}$$

where:

n = Manning roughness coefficient (dimensionless)

d_{50} = median stone diameter (feet)

Table 7-4. Manning Roughness Coefficients for Standard Rock Riprap Classifications

Rock Gradation (1)	Median Stone Weight (W_{50}) (3)	Median Stone Diameter (d_{50}) (4)	Manning *n* (Ungrouted) (5)
No. 3 Backing	5 lb	0.4 ft	0.034
No. 2 Backing	25 lb	0.7 ft	0.037
No. 1 Backing (2)	75 lb	1.0 ft	0.039
Light	200 lb	1.3 ft	0.041
¼ Ton	500 lb	1.8 ft	0.044
½ Ton	1000 lb	2.3 ft	0.045
1 Ton	2000 lb	2.9 ft	0.047
2 Ton	4000 lb	3.6 ft	0.049

(1) Except for 2 ton rock, classification is based upon Caltrans Method B Placement, which allows dumping of the rock and spreading by mechanical equipment. Local surface irregularities shall not vary from the planned grade by more than 1 foot, measured perpendicular to the slope. Two-ton rock requires special placement, see Caltrans (2002) or Greenbook for more information.

(2) No. 1 Backing has same gradation as Facing Riprap.

(3) per Caltrans (2002).

(4) Assumes specific weight of 165 lb/ft^3. The designer shall take care to apply a unit weight that is applicable to the type of riprap specified for the project, and adjust their calculations when necessary.

(5) Based on Manning-Strickler relationship (Chang, 1988).

Where hydraulic radius is less than or equal to two times the maximum rock size, the roughness coefficient will be greater than indicated by Equation 7-5. In these cases, the design engineer shall use the method outlined in Section 7.6.17 to calculate the roughness of the channel. Appendix C (Table C-1) provides recommended Manning roughness coefficient (n) for grouted riprap applications.

7.6.3 Low Flow and Trickle Channels

Riprap-lined channels conveying a 100-year peak runoff of 20 cfs or less do not require trickle channels. The design engineer shall evaluate the factors such as drainage slope, flow velocity, soil type, and upstream impervious area and specify a trickle channel when needed based on their engineering judgment. Low-flow channels shall be designed in accordance with Section 7.4.3.2.

7.6.4 Bottom Width

The selection of the over-all channel bottom width shall consider factors such as ultimate conveyance requirements, constructability, channel stability, and maintenance.

7.6.5 Freeboard and Flow Depth

Riprap-lined channels shall meet the minimum freeboard requirements outlined in Section 7.2.7. Excessive depths and high velocities shall be avoided whenever practicable to maintain public safety. Section 7.2.9 discusses access and safety for open channels, including thresholds for flow depth and velocity.

7.6.6 Side Slopes

The side slopes of riprap-lined channel shall not ordinarily be steeper than 2H:1V except in cases where an embankment stability analysis can justify a steeper side slope. The stability analysis should be completed in consultation with a soils engineer and consider such factors such as:

1. soil characteristics
2. groundwater and river conditions
3. special construction methods and designs (e.g., hand-placed stone keyed well into the bank)
4. shear forces of flow
5. angle of repose of the riprap, and
6. rapid water-level recession.

7.6.7 Horizontal Channel Alignment

Horizontal channel alignment shall be carefully coordinated with the riprap size and configuration (i.e., fully lined or bank-lined) and checked to ensure adequate erosion protection at the toe of the channel bank to account for variations in flow velocity through curves.

7.6.8 Rock Riprap Material

Rock used for riprap shall be hard, durable, angular in shape, and free from cracks, overburden, shale and organic matter. Neither the breadth nor thickness of a single stone shall be less than one-third of its length; rounded stone shall be avoided. Rock having a minimum specific gravity of 2.65 is preferred. Construction debris (e.g., broken concrete or asphalt) is typically not appropriate for use as riprap. Table 7-5 summarizes common rock riprap gradations from Caltrans and the APWA Southern California Greenbook. Section 200 of the Greenbook and Section 72 of the Caltrans Standard Specification provide more detailed specifications for rock riprap material.

7.6.9 Rock Riprap Stone Weight and Gradation

This section discusses the selection of rock riprap gradation for open channels with non-turbulent flow (e.g., not immediately downstream of stilling basins) and with longitudinal slopes of less than 2 percent. Section 7.6.17 provides guidance for riprap design for steep channels.

7.6.9.1 Rock Gradation

Caltrans has developed several standard rock gradations for riprap slope protection of stream banks and shores. Table 7–5 summarizes common rock riprap gradations from Caltrans and the APWA Southern California Greenbook. Other standard riprap specifications, such as from the Federal Highways Administration (FHWA) or Corps of Engineers, are also acceptable for facility design in the City of San Diego when appropriately applied.

Table 7-5. Common Riprap Gradations
(Caltrans and APWA Southern California Greenbook)

Riprap Gradation									
Stone Weight (1)	2 Ton (2)	1 Ton	½ Ton	¼ Ton (3)	375-lb (3)	Light(3)	No. 1 Backing (3), (4)	No. 2 Backing	No. 3 Backing
4 ton	0-5								
2 Ton	50-100	0-5							
1 Ton	95-100	50-100	0-5						
1000 lb		--	50-100	0-5					
700 lb					0-10				
500 lb		95-100	--	50-100	10-50	0-5			
200 lb			95-100	--	85-100	50-100	0-5		
75 lb				90-100	95-100	--	50-100	0-5	
25 lb				95-100		90-100	90-100	25-75	0-5
5 lb						95-100	--	90-100	25-75
2.2 lb							95-100		
1 lb									90-100

(1) Except for 2-ton rock, classification is based upon Caltrans Method B Placement, which allows dumping of the rock and spreading by mechanical equipment.

(2) Two-ton rock requires special placement; see Caltrans or APWA Southern California Greenbook for more information.

(3) 375-lb Class Rock is from APWA Southern California Greenbook.

(4) No. 1 Backing has same gradation as Facing Riprap.

7.6.9.2 Minimum Stone Weight

Riprap channel design involves an iterative process between calculated roughness, stone stability, and channel geometry to achieve an economical and practical combination of channel factors and stone gradation. The design criteria outlined here and in following sections reference general APWA Southern California Greenbook and Caltrans standards. Other riprap-lined channel design methods such as the Corps of Engineers (EM 1110-2 1601) are also acceptable for facility design in the City of San Diego when appropriately applied.

For riprap installed on a channel bottom, rock gradation shall be based upon channel velocity as described in Table 7–3.

Caltrans' Highway Design Manual (Section 870) presents a simplified method for determining the minimum rock weight for the top (outside) layer of riprap bank protection. Equation 7–6 provides the minimum stone weight that will resist the forces of flowing water and remain stable on the slope of a riverbank. The rock riprap shall extend up the side slopes to an elevation of the design water surface plus the calculated freeboard and superelevation. Figure 7–8 presents a nomograph for selection of minimum stone weight based on Equation 7–6. Figure 7-9 presents the definition sketch for riprap bank protection sizing.

Equation 7-6. Minimum Stone Weight Calculation

$$W_{min} = \frac{0.00002 V_A^{\,6} SG}{(SG-1)^3 \sin^3(\beta-\alpha)}$$

where:

W_{MIN}	=	theoretical minimum rock weight (lbs.)
V_A	=	bank velocity (ft/s), $V_A = KV_M$
V_M	=	mean channel velocity (ft/s)
K	=	coefficient (K=0.67 for parallel flow; K=1.33 for impinging flow)
SG	=	specific gravity of rock riprap (no dimension)
β	=	70 degrees (characteristic of randomly placed rubble)
α	=	outside slope face angle with horizontal

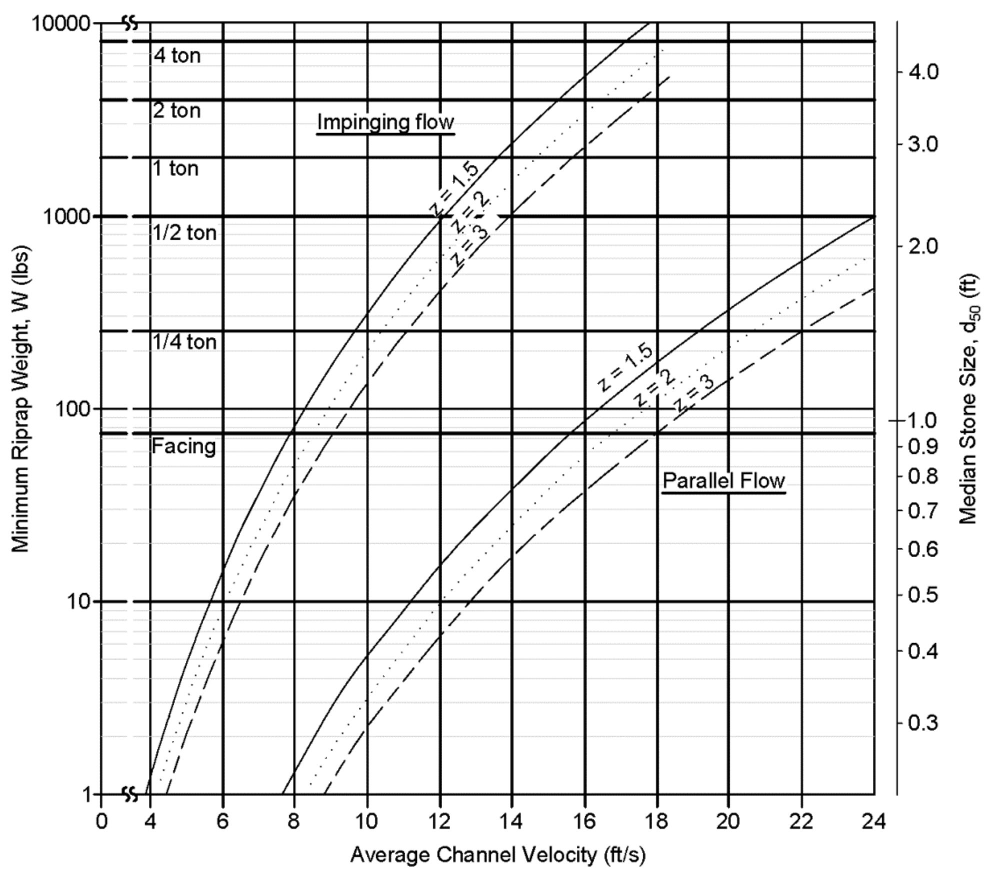


Figure 7-8. Minimum Stone Weight for Riprap Channel Sideslopes

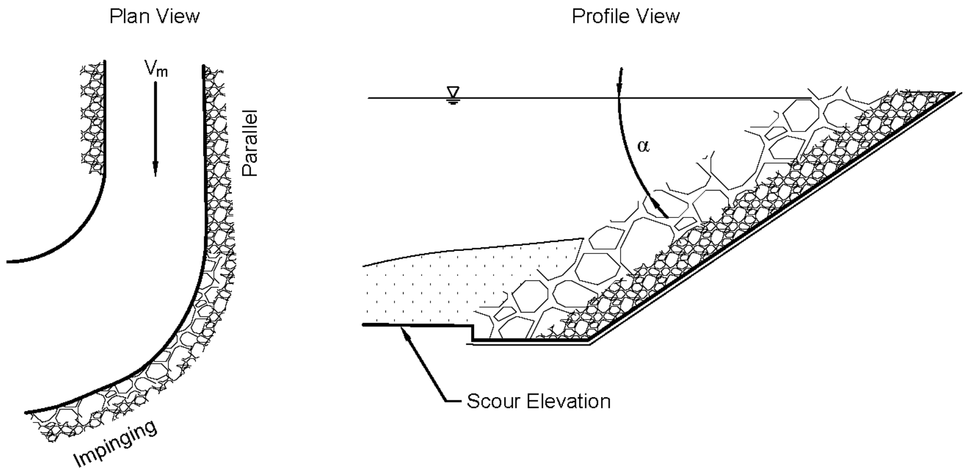


Figure 7-9. Definition Sketch for Riprap Bank Protection Sizing

7.6.10 Riprap Thickness

Riprap layers must be thick enough to ensure mutual support and interlock between individual stones in each layer. The minimum riprap shall not be less than the diameter of the largest stone (d_{100}) or less than 1.5 times the median stone diameter ($1.5d_{50}$). Table 7–6 summarizes the recommended minimum layer thickness for standard Caltrans gradations. When riprap is placed underwater, the riprap thickness shall be increased by at least 50 percent. The total thickness of a riprap installation is the sum of individual layer thicknesses (see Section 7.6.11).

Table 7-6. Minimum Riprap Layer Thickness

Placement Method A	
Rock Gradation	**Minimum Layer Thickness (ft)**
8 ton	8.50
4 ton	6.80
2 ton	5.40
1 ton	4.30
½ ton	3.40
Placement Method B	
Rock Gradation	**Minimum Layer Thickness (ft)**
1 ton	5.40
½ ton	4.30
¼ ton	3.30
Light	2.50
Facing	1.80
Backing No. 1	1.80
Backing No. 2	1.25
Backing No. 3	0.75

Note: Minimum layer thickness for Placement Method A is $1.50d_{50}$ and $1.875d_{50}$ for Placement Method B. These thickness calculations assume a specific weight of 165 lb/ft^3. The designer shall take care to apply a unit weight that is applicable to the type of riprap specified for the project, and adjust their calculations when necessary.

7.6.11 Bedding Requirements

The long-term stability of riprap erosion protection is strongly influenced by proper bedding conditions. Properly designed bedding provides a buffer of intermediate-sized material between the channel bed and the riprap to prevent movement of soil particles through the voids in the riprap. Three types of bedding are in common use: generic single-layer granular bedding, multiple-layer granular bedding, and filter fabric.

Standard riprap installations include an outside layer, one or more inner layers, a backing layer, and filter fabric. Section 7.6.9 describes the determination of gradation of the outside rock layer. The composition of the inner layer(s), backing layer, and filter fabric are design to be progressively smaller to prevent migration of material through voids of the layers. Table 7–7 summarizes the appropriate layers for standard riprap installations. Alternate designs for riprap bedding are acceptable when accompanied by appropriate design calculations.

Table 7-7. California Layered Rock Slope Protection

Outer Layer	Inner Layer	Backing	Fabric
8 ton	2 ton over ½ ton	1	B
8 ton	1 ton over ¼ ton	1 or 2	B
4 ton	½ ton	1	B
4 ton	1 ton over ¼ ton	1 or 2	B
2 ton	½ ton	1	B
2 ton	¼ ton	1 or 2	B
1 ton	LIGHT	NONE	B
1 ton	¼ ton	1 or 2	B
½ ton	NONE	1 or 2	B
¼ ton	NONE	1 or 2	A
Light	NONE	NONE	A
Facing (Backing)	NONE	NONE	A

Note: Minimum permittivity for all filter fabrics is of 0.5 s^{-1}; see Caltrans Standard Specifications for exact definitions of Type A and Type B filter fabrics.

Filter fabric can provide adequate bedding for channel linings along uniform mild sloping channels where leaching forces are primarily perpendicular to the fabric. The design engineer shall use care in specifying using filter fabrics and shall note appropriate construction methods on their plans and specifications. Some of the design considerations and limitations of filter fabric include:

1. Filter fabric shall only replace the bottom layer in a multi-layer granular bedding design.
2. Due care shall be exercised during construction. Construction specifications shall prohibit direct dumping of riprap rock on the filter fabric. Granular bedding shall be placed on top of the filter fabric as a cushion whenever practicable.
3. Due care shall be exercised when specifying filter fabric where seepage forces may run parallel with the fabric and cause piping along the bottom surface. In such situations, the fabric shall be folded vertically downward (similar to a cutoff wall) at regular intervals along the installation, particularly at the entrance and exit of the channel reach.
4. Filter fabric shall be overlapped a minimum of 12 inches at roll edges with upstream fabric being placed on top of downstream fabric at the lap.

7.6.12 Toe Protection

Where only the channel sides are to be lined, additional riprap is needed to provide for long-term stability of the lining. In all cases, the toe of the riprap blanket shall extend a minimum of 3 feet below the proposed channel bed and the thickness of the blanket below the proposed channel bed shall be increased to a minimum of three times the median stone size ($3d_{50}$). If the velocity in the channel exceeds the permissible velocity requirements of the soil comprising the channel bottom, a scour analysis shall be performed to determine if the toe requires additional protection. Riprap toe protection shall extend an additional 3 feet or two times median stone size ($2d_{50}$) below the calculated scour depth, whichever is deeper.

Total scour depth is comprised of three components:

1. Long-term aggradation and degradation of the river bed,
2. General scour due to contractions or other general scour phenomenon, and
3. Local scour at a structure.

An extensive discussion of geomorphic analysis procedures is beyond the scope of this Manual. Equation 7–7 (HEC-11, 2001) may be used to estimate the probable maximum depth of scour in straight channels and channels having mild bends. Because the low point in the cross section may eventually move adjacent to the riprap (even if this is not the case in the existing condition), the scour depth (d_S) shall be measured from the lowest elevation in the cross section.

Equation 7-7. Probable Maximum Depth of Scour in Straight Channels and Channels with Mild Bends

$$d_S = \begin{cases} 12 & d_{50} < 0.005 \text{ft} \\ 6.5 d_{50}^{-0.11} & d_{50} > 0.005 \text{ft} \end{cases}$$

where:

d_S	=	estimated probable maximum depth of scour (ft)
d_{50}	=	median diameter of bed material (ft)

The depth of scour predicted by Equation 7–7 must be added to the magnitude of predicted degradation and local scour (if any) to arrive at the total required toe depth.

7.6.13 Channel Bend Protection

Riprap size shall be increased by one gradation through bends unless calculations can demonstrate the stability of the straight-channel riprap gradation through the bend. For channels conveying 200 cfs or more, the minimum radius for a riprap-lined bend shall be 1.2 times the top width of design flow and in no case be less than 50 feet. Riprap protection shall extend downstream from the end of the bend a distance that is equal to the length of the bend measured along the channel centerline.

7.6.14 Transition Protection

Turbulent eddies near rapid changes in channel geometry (e.g., transitions and bridges) amplify scour potential. At these locations, the riprap lining thickness shall be increased by one gradation unless calculations can demonstrate stability of the smaller gradation through the transition section. Protection shall extend upstream from the transition entrance at least 5 feet and extend downstream from the transition exit at least 10 feet. Section 7.9 contains further discussion of transitions.

7.6.15 End Treatment and Special Conditions

Upstream and downstream ends of riprap-lined channels require particular attention from the design engineer. The end treatment can be accomplished by constructing a concrete cut-off wall, a riprap-filled trench, or thickening the riprap layer for a sufficient distance upstream and/or downstream. The Corps of Engineers' Design of Flood Control Channels (EM-1110-2-1603) provides specific guidance on end treatments for riprap channels. When failure of the riprap lining could seriously affect the health and safety of the public, the design engineer may consider constructing intermediate transverse cutoff walls at regular intervals to help preserve the integrity of the riprap channel lining.

7.6.16 Concrete-Grouted Riprap

Concrete-grouted riprap may be used when the calculated size of loose riprap is unreasonable or the installation of loose riprap is impractical. Grouted riprap requires less routine maintenance by reducing silt and trash accumulation and is particularly useful for lining low-flow channels and steep banks. Exposing the tops of individual stones and cleaning excess grout from the projecting rock with a wet broom prior to curing provides the appropriate channel roughness.

The rock used for concrete-grouted riprap is slightly different from the standard gradation of rock riprap in that the smaller rock is reduced to allow greater penetration by the grout. Table 7–8 summarizes Caltrans rock riprap specifications for grouted applications. Grouting of rock gradations larger than 1 ton is typically not recommended.

Proper composition and placement of grouting is vital to the performance of the lining. Concrete used for rock grout shall meet all standards and shall be installed in accordance with procedures outlined in Greenbook Section 300-11.

Table 7-8. Concrete-Grouted Riprap Gradations

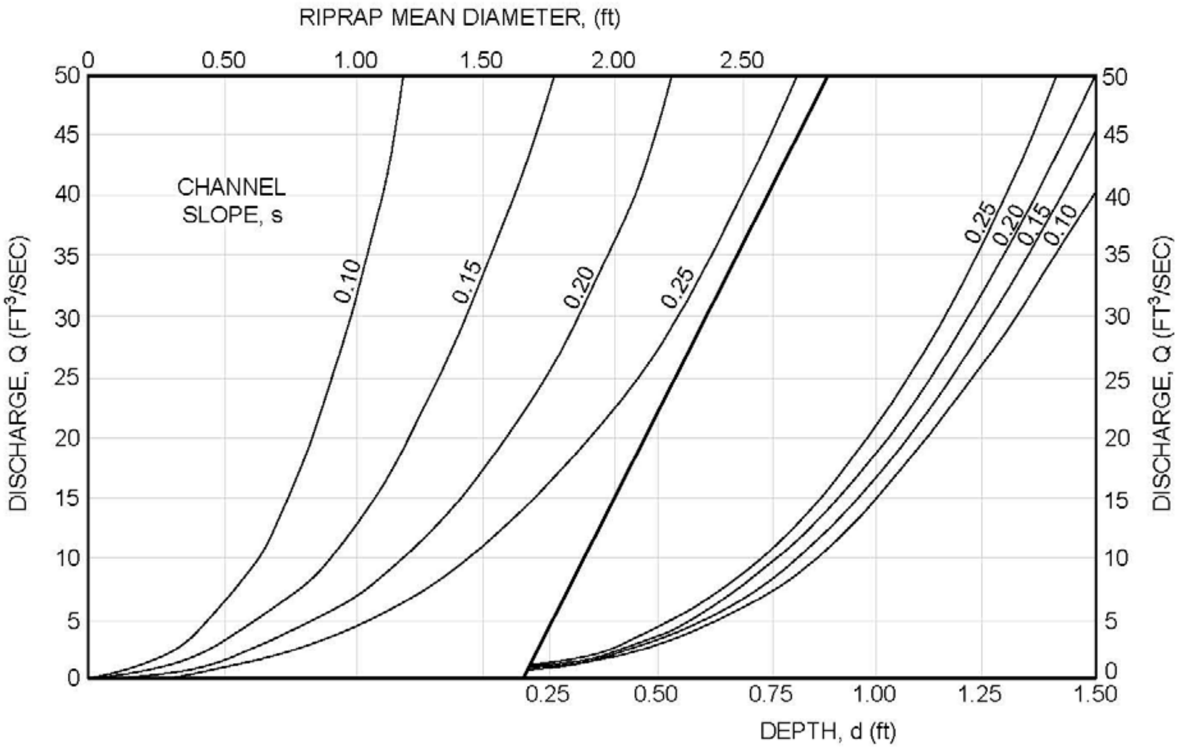
Rock Mass	Percentage Larger than Class				
	½ Ton	¼ Ton	Light	Facing	Cobble
1 Ton	0-5	-	-	-	-
½ Ton (1000 lb)	50-100	0-5	-	-	-
¼ Ton (500 lb)	-	50-100	0-5	-	-
200 lb	90-100	-	50-100	0-5	-
75 lb	-	90-100	90-100	50-100	0-5
25 lb	-	-	-	90-100	95-100
Minimum Grout Penetration (inches)	18	14	10	8	6

7.6.17 Riprap on Steep Longitudinal Slopes

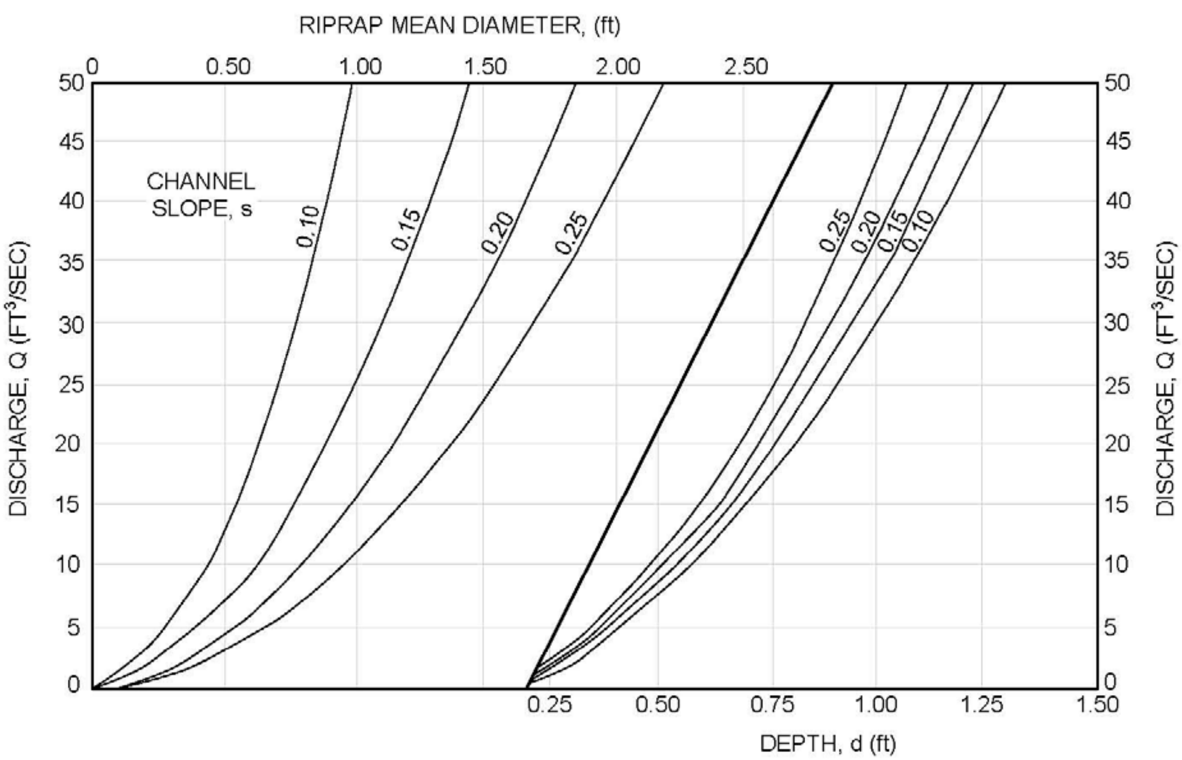
The Federal Highway Administration (FHWA HEC-15, 1988) provides a graphically-based method to design rock riprap-lined channels on steep slopes (i.e., those designed for supercritical flow). This procedure shall also be used for rock riprap lined channels whose depth of flow is equal to or less than d_{50}.

7.6.17.1 Rock Size

Figures 7-10 and 7-11 provide design curves that simplify riprap design for steep channels by median riprap size (d_{50}) for a given flow, channel slope, and channel width. The design curves were developed for channels with 3H:1V side slopes and bottom widths of 0 feet, 2 feet, 4 feet, 6 feet, and 8 feet. When the channel slope is not provided by one of the design curves, linear interpolation is used to determine the riprap size. This is done by extending a horizontal line at the given flow through the curves with slopes bracketing the design slope. A curve at the design slope is then estimated by visual interpolation. The design median stone size (d_{50}) is chosen at the point that the flow intercepts the estimated design curve. Linear interpolation can also be used to estimate the d_{50} size for bottom widths other than those supplied in the figures. For practical engineering purposes, the d_{50} size specified for the design shall be translated into standard riprap gradation.

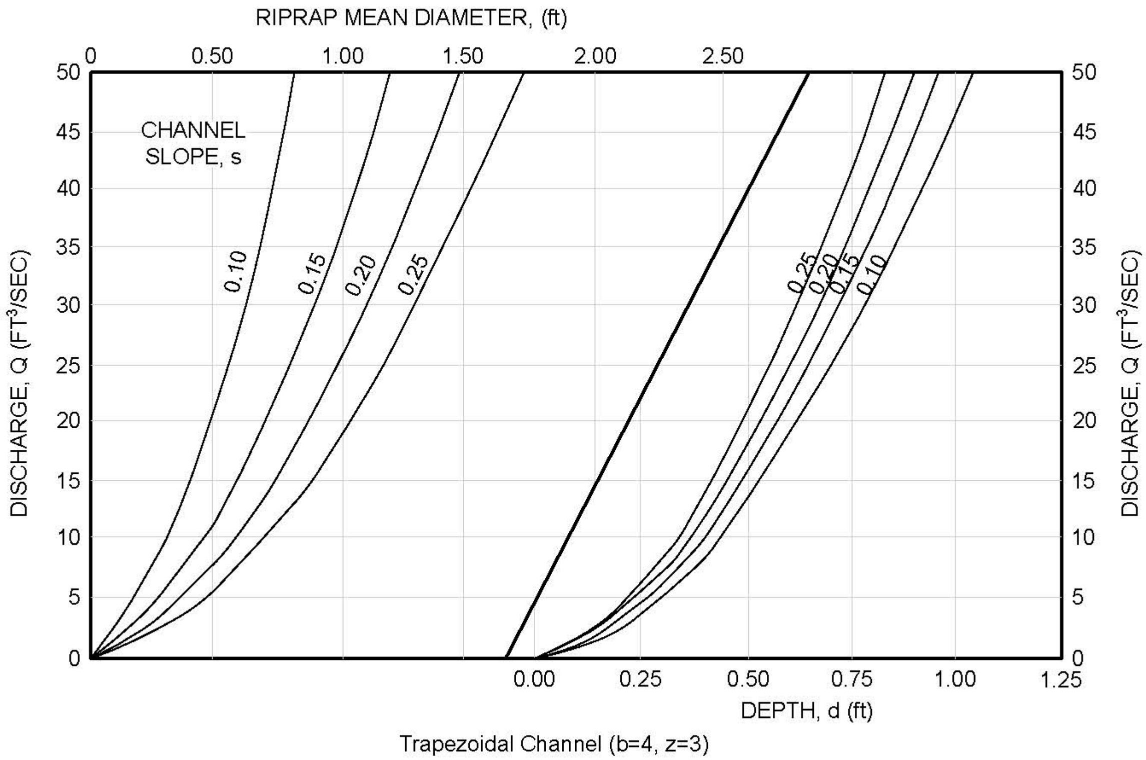


Triangular Channel (b=0, z=3)



Trapezoidal Channel (b=2, z=3)

Figure 7-10. Design Nomographs for Riprap on Steep Channels (part 1 of 2)



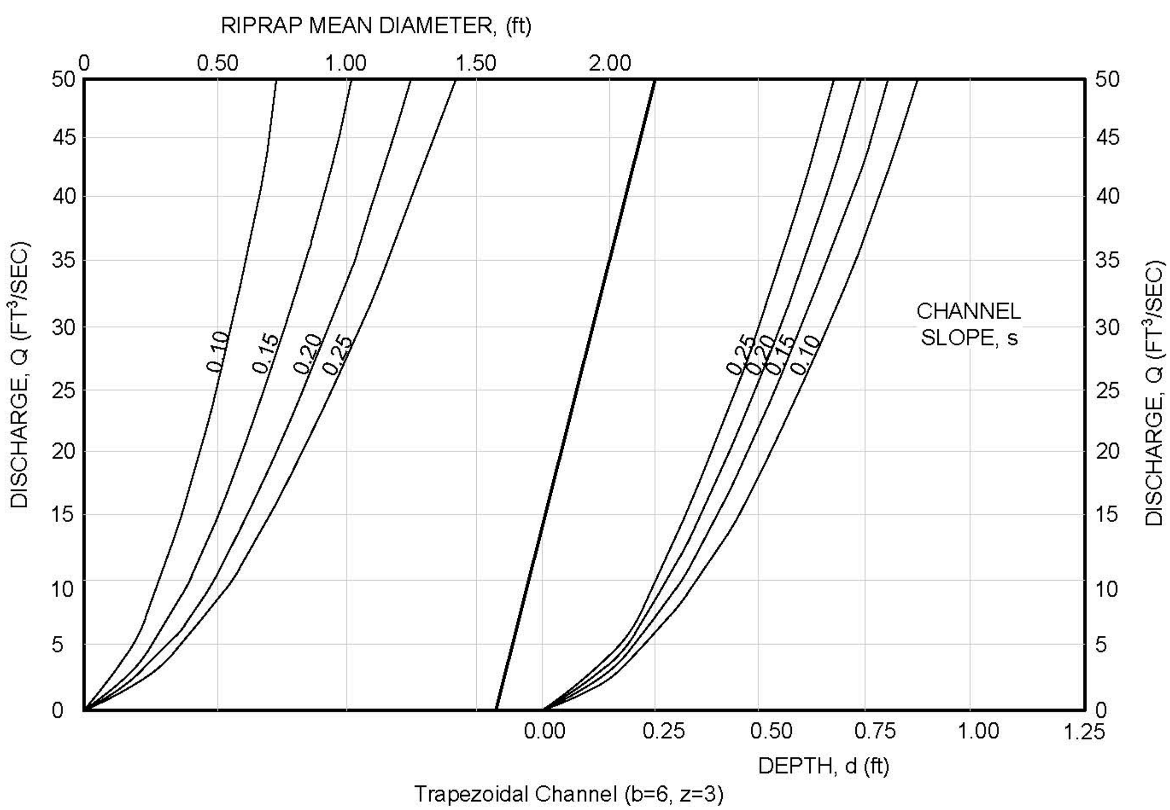


Figure 7-11. Design Nomographs for Riprap on Steep Channels (part 2 of 2)

7.6.17.2 Riprap Thickness for Steep Longitudinal Slopes

For riprap linings on steep slopes, the topmost riprap layer shall have a thickness of at least 1.25 times the median rock size (1.25d_{50}). The maximum resistance to the erosive forces of flowing water occurs when all rock is contained within the riprap layer thickness. Oversize rocks that protrude above the riprap layer reduce channel capacity and reduce riprap stability.

7.6.17.3 Riprap Placement on Steep Slopes

On steep slopes, riprap shall be placed using Caltrans Placement Method A; it shall never be placed by dropping it down the slope in a chute or pushing it down with a bulldozer.

7.6.17.4 Bedding Requirements on Steep Slopes

Either a granular bedding material or filter fabric may be used on steep slopes.

7.7. Design Criteria: Concrete-Lined Channels

This section presents minimum design criteria for concrete-lined channels. The design engineer is responsible for confirming that a channel design meets these criteria, the general open-channel criteria outlined in Section 7.2, and any special considerations for a particular design situation.

7.7.1 Longitudinal Channel Slope

Concrete-lined channels have the ability to accommodate supercritical flow conditions and thus can be constructed on almost any naturally occurring slope.

7.7.2 Roughness Coefficients

Appendix C provides Manning roughness coefficient for concrete-lined channels.

7.7.3 Channel Bottom Cross-Slope

The bottom of the concrete channel shall be constructed with a defined low flow channel or shall be adequately sloped to confine the low flows to the middle or one side of the channel cross-section as described in San Diego Regional Standard Drawings SDD-107 and SDD-108.

7.7.4 Bottom Width

There are no bottom width requirements for concrete-lined channels.

7.7.5 Freeboard

Concrete-lined channels conveying a 100-year flow less than or equal to 10 cfs shall have a minimum freeboard of 6 inches. Concrete-lined channels conveying more than 10 cfs shall meet the minimum freeboard requirements outlined in Section 7.2.7. There are no flow depth requirements for concrete-lined channels.

7.7.6 Concrete Lining Section

7.7.6.1 Thickness

In cases where a concrete channel is expected to carry a large amount of debris or abrasive sediment material at high velocities, it shall have a thickened lining section or other measures to provide sufficient design life for the facility. Concrete lining shall have a minimum thickness of 6 inches for flow velocities less than 30 fps and a minimum thickness of 8 inches for flow velocities of 30 fps and greater.

7.7.6.2 Concrete Detailing

Concrete channels shall be appropriately reinforced and jointed per Regional Standard Drawing No. SDD-107 or SDD-108.

7.7.7 Safety

Concrete-lined channels shall provide appropriate safety measures as described in Section 7.2.9 of this Manual and San Diego Regional Standard Drawings No. SDD-107 or SDD-108 to the satisfaction of the City.

7.7.8 Special Consideration for Supercritical Flow

The design engineer shall give special consideration to supercritical flow and its potential effects when designing, specifying, and inspecting concrete channels. Care shall be taken to prevent excessive waves that may extend down the entire length of the channel (see Section 7.9.4.3). The design engineer shall consider the possibility of hydraulic jumps forming in the channel.

7.8. Design Criteria: Other Channel Linings

Other channel linings include all channel linings that are not discussed in the previous sections. These include composite-lined channels, where two or more different lining materials are used. They also include gabions, soil cement linings, synthetic fabric, cobble, and geotextile linings, preformed block linings, reinforced soil linings, and floodwalls (vertical walls constructed on both sides of an existing floodplain). For those linings not discussed in this Manual, supporting documentation will be required to support the use of the desired lining. Some of the items that should be addressed include:

1. Structural integrity of the proposed lining
2. Interfacing between different linings
3. The maximum velocity under which the lining will remain stable
4. Potential erosion and scour problems
5. Access for operations and maintenance
6. Long term durability of the product

7. Ease of repair of damaged section
8. Past case history (if available) of the lining system in similar applications
9. Potential groundwater mitigation issues (e.g., weepholes, underdrains, etc.)

These linings will be allowed on a case-by-case basis. The City may reject the proposed lining system in the interests of operation, maintenance, or the protection of public safety.

7.9. Design Procedures: General Open-Channel Flow

An open channel is a conduit in which water flows with a free surface (non-pressurized flow). The hydraulics of an open channel can encompass many different flow conditions from steady state, uniform flow to unsteady, and rapidly varying flow. The calculations for uniform and gradually varying flow are relatively straightforward and are based upon similar assumptions (e.g., parallel streamlines). In contrast, rapidly varying flow computations (e.g., hydraulic jumps and flow over spillways) can be very complex and the solutions are generally empirical in nature. This section presents the basic equations and computational procedures for uniform, gradually varying and rapidly varying flow.

7.9.1 Uniform Flow Computation

Open-channel flow is uniform if the depth of flow is the same at every section of the channel. For a given channel geometry, roughness, discharge and slope, there is only one possible depth for maintaining uniform flow. This depth is referred to as the “normal depth.” For uniform flow within a prismatic channel (i.e., uniform cross section), the water surface will be parallel to the channel bottom. While uniform flow rarely occurs in nature and is difficult to achieve in a laboratory, a uniform-flow approximation is generally adequate for planning and design purposes.

The computation of uniform flow and normal depth shall be based upon the Manning or Uniform Flow Equation (Equation 7-8).

Equation 7-8. Uniform Flow Equation

$$Q=\frac{1.49}{n}A^{5/3}P^{-2/3}\sqrt{S}=\frac{1.49}{n}AR^{2/3}\sqrt{S}$$

where:

Q	=	Discharge (ft³/s)
n	=	Manning roughness coefficient
A	=	area (ft²)
P	=	wetted perimeter (ft)
R	=	hydraulic radius, R = A/P (ft)
S	=	slope of the energy grade line (ft/ft)

For prismatic channels, the energy grade line (EGL), hydraulic grade line (HGL), and the bottom can be assumed parallel for uniform, normal depth flow conditions.

The variables dependent on channel cross-section geometry (e.g., area and hydraulic radius) can be lumped together as the conveyance (K) of the channel. This simplifies the Uniform Flow Equation (Equation 7-9) to the following expression:

Equation 7-9. Simplified Uniform Flow Equation

$$Q=K\sqrt{S}$$

Figure 7–12 presents equations for calculating many of the parameters required for hydraulic analysis of different channel sections. Appendix C provides a list of Manning roughness coefficient values for many types of conditions that may occur in the San Diego Region. The Uniform Flow Equation and its constituent parameters are readily computed using hand-held calculators and personal computers.

Section	Area	Wet Perimeter	Hydraulic Radius	Top Width	Hydraulic Depth	Critical Section Factor
	(A)	(P)	(R)	(T)	(D)	$(Z_c = AD^{0.5})$
Rectangle	by	$b + 2y$	$\frac{by}{b+2y}$	b	y	$by^{3/2}$
Trapezoid	$(b + zy)y$	$b + 2y\sqrt{1+z^2}$	$\frac{(b+zy)y}{b+2y\sqrt{1+z^2}}$	$b + 2zy$	$\frac{(b+zy)y}{b+2zy}$	$\frac{[(b+zy)y]^{3/2}}{\sqrt{b+2zy}}$
Triangle	zy^2	$2y\sqrt{1+z^2}$	$\frac{zy}{2\sqrt{1+z^2}}$	$2zy$	$\frac{y}{2}$	$\frac{\sqrt{2}}{2}zy^{5/2}$
Circle[1]	$\frac{1}{8}(\theta - \sin\theta)d_0^2$	$\frac{1}{2}\theta d_0$	$\frac{1}{4}\left[1 - \frac{\sin\theta}{\theta}\right]d_0$	$\left[\sin\frac{\theta}{2}\right]d_0$ or $2\sqrt{y(d_0 - y)}$	$\frac{1}{8}\left[\frac{\theta - \sin\theta}{\sin\frac{\theta}{2}}\right]d_0$	$\frac{\sqrt{2}(\theta - \sin\theta)^{3/2}}{32\left(\sin\frac{\theta}{2}\right)^{1/2}}d_0^{5/2}$

[1] θ describes the angle that includes the chord corresponding to the water surface, measured in radians; $\theta = 2\arcsin\left[\frac{2y}{d_0} - 1\right] + \pi$

Figure 7-12. Cross-Section Hydraulic Elements

7.9.1.1 Gradually Varying Flow

The most common occurrence of gradually varying flow in storm drainage is the backwater created by culverts, storm drain inlets, or channel constrictions. For these conditions, the flow depth will be greater than normal depth in the channel and the water surface profile (a.k.a. "backwater curve") is computed using either the direct-step or standard step method. Many computer programs are available for the calculation of gradually varied flow. The most general and widely used programs are the U.S. Army Corps of Engineers' HEC-2 Water Surface Profiles and HEC-RAS River Analysis System and the Los Angeles County Flood Control District's Water Surface Pressure Gradient (WSPG). The design engineer may use these programs or proprietary computer software to compute water surface profiles for channel and floodplain analyses.

Direct-Step Method

The Direct-Step Method is best suited to the analysis of open prismatic channels. Water surface profiles in simple prismatic channels can be computed manually. Chow (1959) presents the basic method for applying the direct-step analysis. The Direct-Step Method is also available in many hand-held and personal computer programs.

Standard-Step Method

The Standard-Step Method is required for the analysis of irregular or non-uniform cross-sections. Because the Standard Step Method involves a more tedious iterative process, this Manual recommends that design engineers use computer programs such as HEC-RAS to accomplish these calculations.

7.9.1.2 Rapidly Varying Flow

Rapidly varying flow is characterized by very pronounced curvature of the water surface profile. The change in water surface profile may become so abrupt as to result in a state of high turbulence. Calculation methods for gradually varying flow (e.g., direct-step and standard-step methods) do not apply for rapidly varying flow. There are mathematical solutions to some specific cases of rapidly varying flow, but the solutions to most rapidly varying flow problems rely on empirical data.

The most common occurrence of rapidly varying flow in storm drainage applications involves weirs, orifices, hydraulic jumps, non-prismatic channel sections (transitions, culverts and bridges), and non-linear channel alignments (bends). Each of these flow conditions requires detailed calculations to properly identify the flow capacities and depths of flow in the given section. The design engineer must be cognizant of the design requirements for rapidly varying flow conditions and shall include all necessary calculations as part of the design submittal documents. This Manual refers the design engineer to the hydraulic references in Section 7.11 for the proper calculation methods to use in the design of drainage facilities with rapidly varying flow conditions.

7.9.2 Design Procedures: Critical Flow

The critical state of uniform flow through a channel is characterized by several important conditions regarding the relationship between the flow, specific energy, and slope of a particular hydraulic cross-section (Figure 7–13).

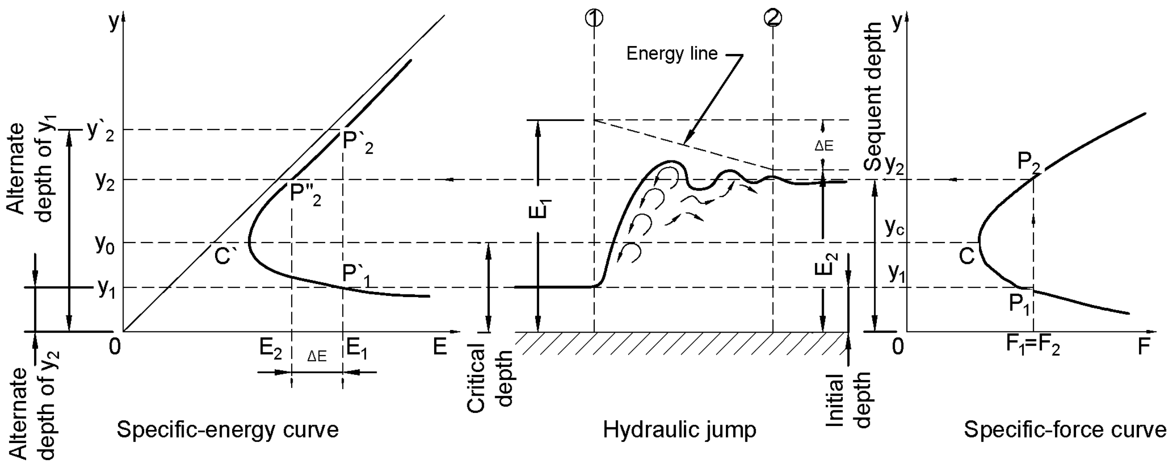


Figure 7-13. Specific Energy Curve

Critical state is characterized by the following conditions:

1. The specific energy ($E=y+v^2/2g$) is a minimum for a given discharge (Q).
2. The discharge (Q) is a maximum for a given specific energy (E).
3. The specific force is a minimum for a given discharge (Q).
4. The velocity head ($v^2/2g$) is equal to half the hydraulic depth (D/2) in a channel of small slope.
5. The Froude Number is equal to FR =1.0.

Typically, channels should not be designed to flow at or near critical state (0.80<FR<1.20, see Section 7.2.6). If the critical state of uniform flow exists throughout an entire reach, the channel flow is critical and the channel slope is at critical slope (S_C). A slope less than S_C will cause subcritical flow. A slope greater than S_C will cause supercritical flow.

The criteria of minimum specific energy for critical flow of the Froude Number (FR) is defined in Equation 7–10.

Equation 7-10. Froude Number Definition

$$FR=\frac{v}{\sqrt{gD}}$$

where:

FR	=	Froude Number (dimensionless)
v	=	velocity (ft/s)
g	=	gravitational acceleration (ft/s^2)
A	=	channel flow area (ft^2)
D	=	hydraulic depth, D=A/T (ft)
T	=	top width of flow area (ft)

The critical depth in a given trapezoidal channel section with a known discharge can be determined using the following method:

Step 1. Compute the section factor (Equation 7-11) for critical flow computation (*Z*).

Equation 7-11. Critical Flow Computation

$$Z_C=A\sqrt{D}=\frac{Q}{\sqrt{g}}$$

where:

Z_C	=	section factor for critical flow computation
A	=	channel flow area (ft^2)
D	=	hydraulic depth, D=A/T (ft)
T	=	top width of flow area (ft)
Q	=	Discharge (ft^3/s)
g	=	gravitational acceleration (32.2 ft/s^2)

Step 2. Determine the critical depth in the channel (d_c) from Figure 7–14, using appropriate values for the section factor for critical flow computation (Z_C), the channel bottom width, (b), and the channel side slope (z).

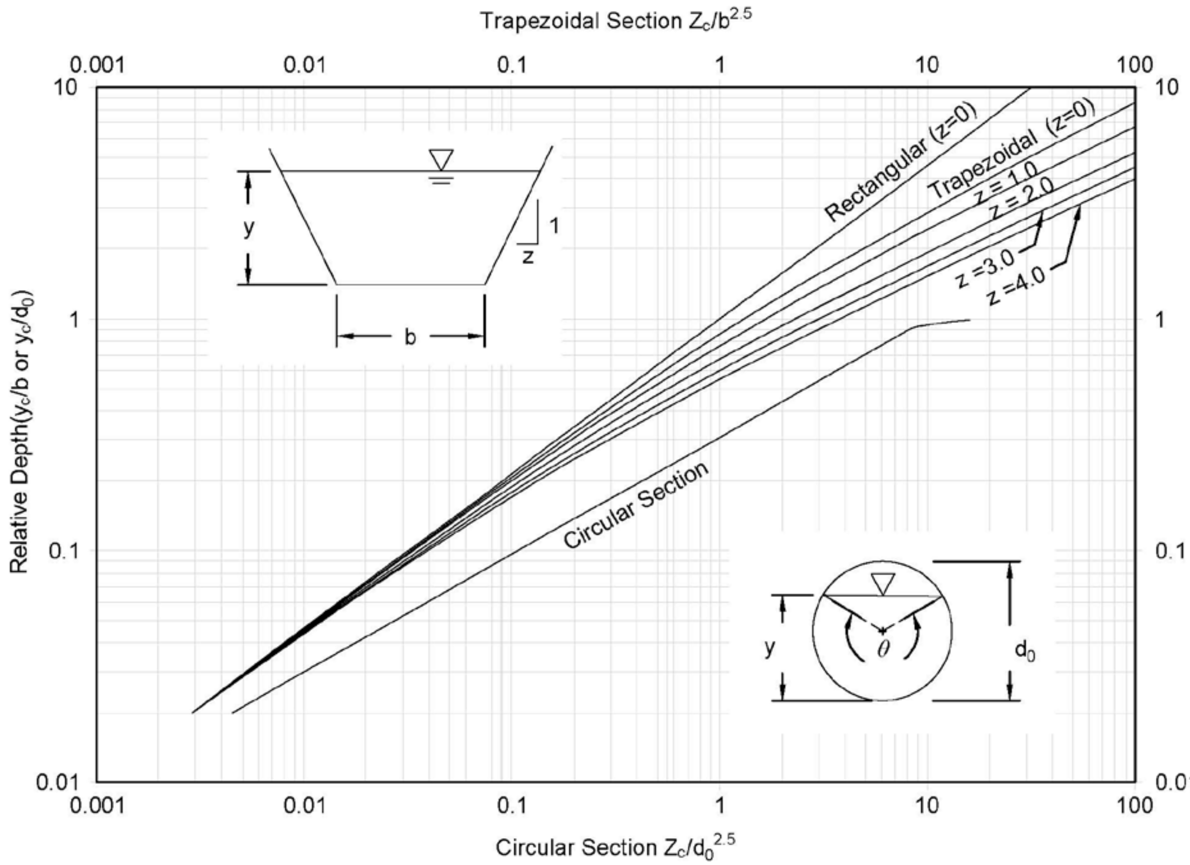


Figure 7-14. Critical Depth Nomograph

7.9.3 Design Procedures: Subcritical Flow

All open channels shall be designed with the limits as stated in Section 7.2 through Section 7.8. The following design procedures shall be used when the design runoff in the channel is flowing in a subcritical condition (FR <1.0).

7.9.3.1 Transitions – Subcritical Flow

Subcritical transitions occur when transitioning one subcritical channel section to another subcritical channel section (expansion or contraction) or when a subcritical channel section is steepened to create a super critical flow condition downstream (e.g., a sloping spillway entrance). Figure 7–15 presents several typical subcritical transition sections. The warped transition section, although most efficient, should only be used in extreme cases where minimum loss of energy is required since the section is very difficult and costly to construct. Conversely, the square-ended transition should only be used when either a straight-line transition or a cylinder-quadrant transition cannot be used due to topographic constraints or utility conflicts.

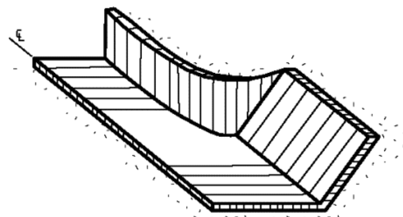
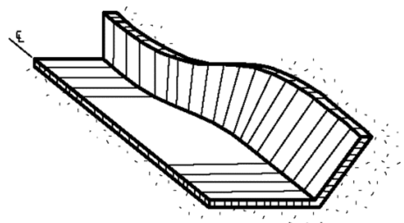
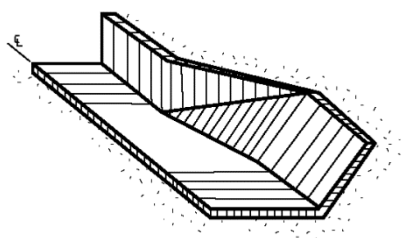
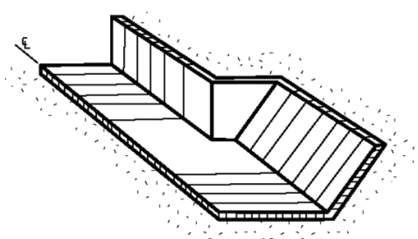
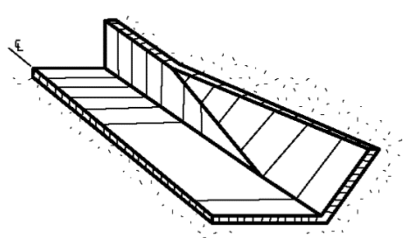
Transition Type	Expansion Coefficient	Contraction Coefficient
 Cylindrical Quadrant	$K_{te} = 0.20$	$K_{tc} = 0.15$
 Warped	$K_{te} = 0.20$	$K_{tc} = 0.10$
 Wedge	$K_{te} = 0.50$	$K_{tc} = 0.30$
 Square End	$K_{te} = 0.75$	$K_{tc} = 0.30$
 Straight-Line	$K_{te} = 0.50$	$K_{tc} = 0.30$

Figure 7-15. Typical Subcritical Transition Sections and Loss Coefficients

Subcritical Transitions - Contractions

The energy loss created by a contracting section may be calculated using Equation 7–12.

Equation 7-12. Energy Loss Calculation for Contracting Transition Section

$$H_t=K_{tc}\left(\frac{V_2^{\ 2}}{2g}-\frac{V_1^{\ 2}}{2g}\right)$$

where:

H_t	=	energy loss (ft)
K_{tc}	=	contraction transition coefficient
v_1	=	upstream velocity (ft/s)
v_2	=	downstream velocity (ft/s)
g	=	gravitational acceleration (32.2 ft/s^2)

Figure 7–15 presents contraction loss coefficient (K_{tc}) values for the typical open-channel transition sections.

Subcritical Transitions - Expansions

The energy loss created by an expanding transition section may be calculated using Equation 7–13.

Equation 7-13. Energy Loss Calculation for Expanding Transition Section

$$H_t=K_{te}\left(\frac{V_1^{\ 2}}{2g}-\frac{V_2^{\ 2}}{2g}\right)$$

Figure 7–15 presents expansion loss coefficients (K_{te}) values for typical open-channel transition sections.

Subcritical Transition - Length

The length of the transition section shall be long enough to keep the streamlines smooth and nearly parallel throughout the expanding (contracting) section. Experimental data and performance of existing structures have been used to estimate the minimum transition length necessary to maintain the stated flow conditions. Based on this information, the minimum length of the transition section shall be determined using Equation 7-14.

Equation 7-14. Minimum Length Calculation for Transition Section

$$L_t \geq 0.5L_c(\Delta T_W)$$

where:

L_t	=	minimum transition length (feet)
L_C	=	length coefficient (dimensionless)
ΔT_W	=	difference in the top width of the normal water surface upstream and downstream of the transition (feet)

Table 7–9 summarizes the transition length coefficients for subcritical flow conditions. These transition length guidelines are not applicable to cylinder-quadrant or square-ended transitions. For flow approach velocities of 12 feet per second or less, the transition length coefficient (L_C) shall be 4.5. This represents a 4.5L:1W expansion or contraction, or about a 12.5-degree divergence from the channel centerline. For flow approach velocities of more than 12 feet per second, the transition length coefficient (L_C) shall be 10. This represents a 10L:1W expansion or contraction, or about a 5.75-degree divergence from the channel centerline.

Table 7-9. Transition Length Coefficients for Subcritical Open Channels

Flow Approach Velocity (v) (ft/s)	Transition Length Coefficient (L_C)
≤ 12	4.5
> 12	10

7.9.4 Design Procedures: Supercritical Flow

All supercritical channels shall be designed within the limits as stated in Sections 7.2 through 7.8. The following design procedures shall be used when channels are designed to flow in a supercritical condition (FR>1.0).

Supercritical flow can become unstable in response to relatively minor disturbances to the channel cross section; even small obstructions can sometimes cause a hydraulic jump. Good design practice is to test supercritical flow stability during events smaller than the design flow by evaluating flows of specific more frequent storm events (e.g., 10-year, 2-year, etc.) or by testing successive fractions (e.g., one-half, one-quarter, and further if necessary) of the design flow. However, only calculations for the full design flow are required to be submitted for review.

7.9.4.1 Transitions – Supercritical Flow

The design of supercritical flow transitions is more complicated than subcritical transition design due to the potential damaging effects of the oblique jump created by the transition. The oblique jump results in cross-waves and higher flow depths that can cause damage if not properly accounted for in the design. Supercritical transitions can be avoided by designing a hydraulic jump, which must also be carefully designed to assure the jump will remain where the jump is designed to occur. Hydraulic jumps shall be designed to take place only within hardened parts of the channel, such as energy dissipation or drop structures, and not within erodible portions of the channel. Chapter 8 discusses energy dissipation devices.

Supercritical Transitions - Contractions

Figure 7–16 presents an example of a supercritical contracting transition with upstream flow contracted from width b_1 to b_3 and a wall diffraction angle of θ. The oblique jump occurs at the points A and B where the diffraction angles start. Wave fronts generated by the oblique jumps on both sides propagate toward the centerline with a wave angle $\beta 1$. Since the flow pattern is symmetric, the centerline acts as if there was a solid wall that causes a subsequent oblique jump and generates a backward wave front toward the wall with another angle β2. These continuous oblique jumps result in turbulent fluctuations in the water surface.

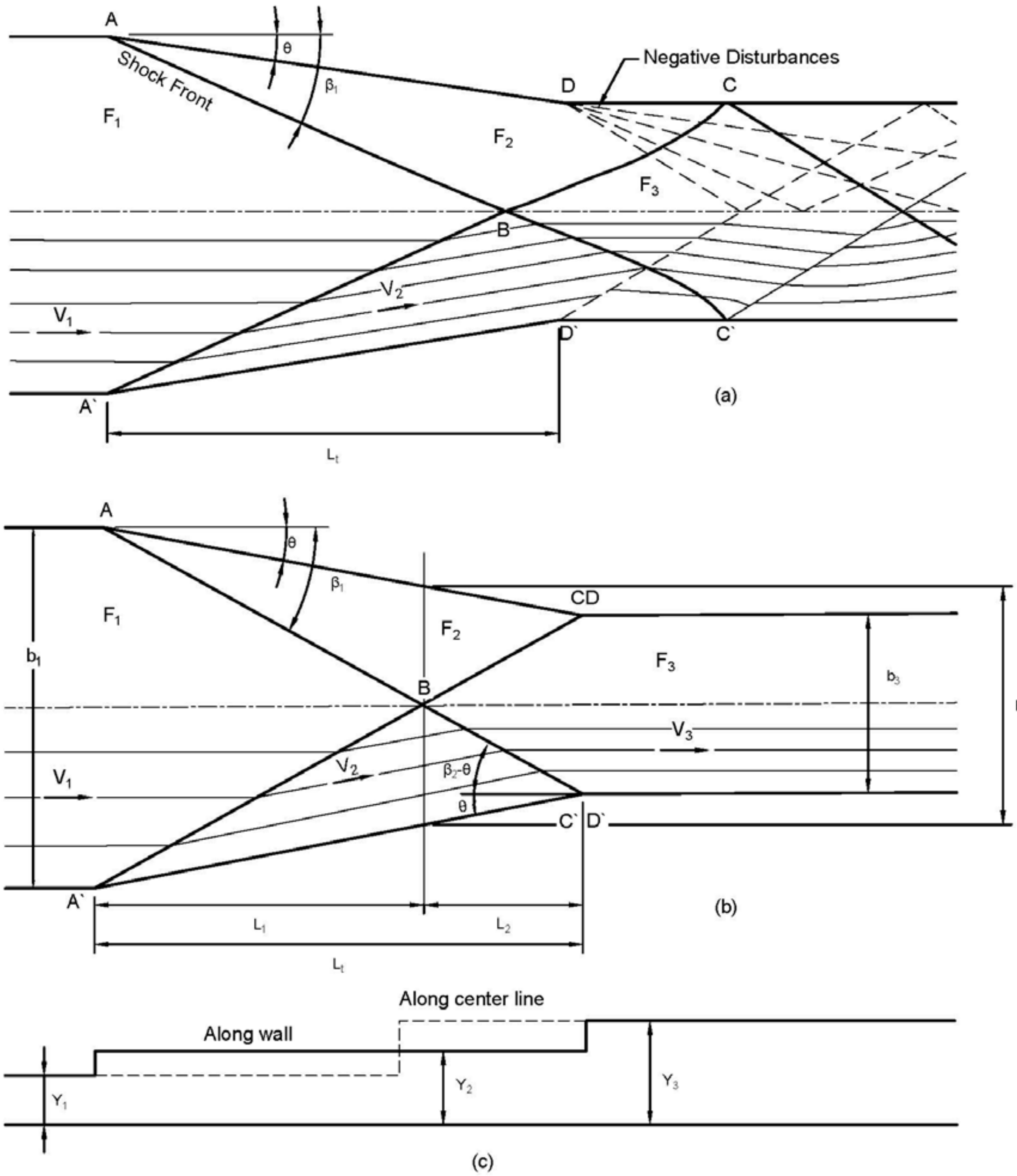


Figure 7-16. Supercritical Contraction Angle Definitions

To minimize the turbulence, the first two wave fronts are designed to meet at the center and then end at the exit of the contraction. Using the contraction geometry, the length of the transition shall be calculated by using Equation 7-15.

Equation 7-15. Length of Transition Calculation for Contractions

$$L_t = \frac{b_1 - b_3}{2\tan\theta}$$

where:

L_t	=	transition length (ft)
b_1	=	upstream top width of flow (ft)
b_3	=	downstream top width of flow (ft)
θ	=	wall angle as related to the channel centerline (degrees)

Using the continuity principle found in Equation 7-16.

Equation 7-16. Length of Transition Calculation with Continuity Principle

$$\frac{b_1}{b_3} = \left(\frac{y_3}{y_1}\right)^{3/2} \left(\frac{FR_3}{FR_1}\right)$$

where:

y_1	=	upstream depth of flow (ft)
y_3	=	downstream depth of flow (ft)
FR_1	=	upstream Froude Number
FR_3	=	downstream Froude Number

Also, by the continuity and momentum principles, the following relationship between the Froude Number, wave angle, and wall angle is found with Equation 7-17.

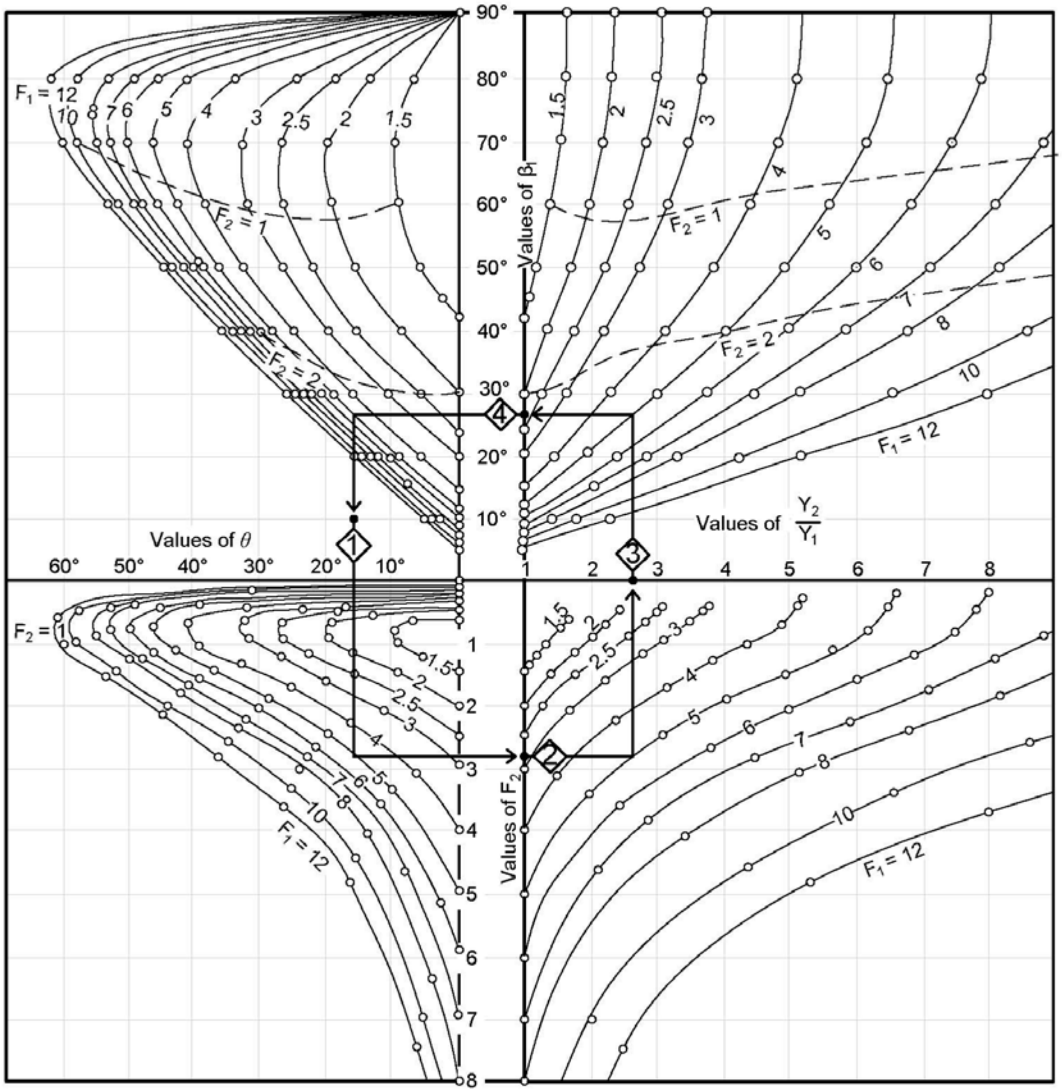
Equation 7-17. Calculation for Relationship of Froude Number, Wave Angle, and Wall Angle

$$\tan\theta = \frac{\tan\beta_1\left[\left(1+8FR_1^{\,2}\sin^2\beta_1\right)^{1/2} - 3\right]}{2\tan^2\beta_1 + \left(1+8FR_1^{\,2}\sin^2\beta_1\right)^{1/2} - 1}$$

where:

β_1	=	initial wave angle (degrees)

By trial and error, this design procedure can be used to determine the transition length and wall angle. Figure 7–17 offers a faster solution than trial and error using Equations 7–16 and 7–17. Figure 7–17 can also be used to determine the wave angle (β) or may be used with the equations to determine the required downstream depth or width parameter if a certain transition length is desired or required.



Example: For F_1 = 5.0, and θ = 15°, (1) read F_2 = 2.8; (2) read $\frac{y2}{y1}$ = 2.75; (3) read β_1 = 27°; (4) read θ = 15° (check).

Figure 7-17. Design Nomograph for Supercritical Contraction Transition Length and Wave Angle

To minimize the length of the transition section, the ratio of downstream and upstream flow depths should generally be greater than two and less than three ($2.0<y_3/y_1<3.0$). The downstream Froude Number should generally be greater than 1.7 ($FR_3>1.7$) to help avoid undulating hydraulic jumps downstream. For further discussion on oblique jumps and supercritical contractions, refer to Chow (1959).

Supercritical Transitions - Expansions

A properly designed expansion transition expands the flow boundaries at approximately the same rate as the natural flow expansion. Based on experimental and analytical data results, the minimum length of a supercritical expansion shall be determined by Equation 7-18.

Equation 7-18. Minimum Length of Supercritical Expansion Calculation

$$L_t \geq 1.5(\Delta W)FR_1$$

where:

L_t	=	Minimum transition length (feet)
ΔW	=	Difference in the top width of the normal water surface upstream and downstream of the transition
FR_1	=	Upstream Froude Number

7.9.4.2 Transition Curves

A transition curve may be used to reduce the required amount of freeboard or radius of curvature in a rectangular channel. The length of the transition curve measured along the channel centerline shall be determined by Equation 7-19.

Equation 7-19. Length of Transition Curve along Channel Centerline Calculation

$$L_c = 2D = 0.32\frac{WV}{\sqrt{y}}$$

where:

L_c	=	length of transition curve (ft)
D	=	distance from the start of curve to point of first maximum superelevation (ft). Typically $D=3L_W$; see description of how to apply superelevation allowance in Section 7.2.7
W	=	top width of design water surface (ft)
V	=	mean design velocity (ft/s)
y	=	depth of design flow (ft)

The radius of the transition curves should be twice the radius of the main bend. Transition curves should be located both upstream and downstream of the main bend.

7.9.4.3 Slug Flow and Roll Waves

Steep channels with significantly rapid flows (FR >2.0) are prone to developing pulsating flow profiles, often called slug flows or roll waves. These standing waves can cause flow to exceed freeboard limits and possible damage to the channel lining. The design engineer may resolve pulsating flow issues either by adjusting the channel slope to prevent the development of these waves or providing additional freeboard to account for the height of the standing waves.

Theoretically, slug flow will not occur when the Froude Number is less than two (FR <2.0). To avoid slug flow when the Froude Number is greater than 2.0, the channel slope shall be determined by Equation 7-20.

Equation 7-20. Channel Slope Calculation

$$S \leq \frac{12}{R_E}$$

where:

S	=	channel slope (ft/ft)
R_E	=	Reynolds Number, $R_E = \frac{uR}{\nu}$ (no dimensions)
u	=	mean design velocity (ft/s)
R	=	hydraulic radius (feet)
v	=	kinematic viscosity of water (ft^2/s)

More detailed discussion of pulsating flow is beyond the scope of this Manual. Several references, including Chow (1959) and Clark County (2000) provide further discussion of this topic. The Los Angeles County Flood Control District (1982) has developed nomographs for determining the appropriate freeboard allowance for roll wave height based on empirical research at the California Institute of Technology (Brock, 1967).

7.9.5 Design Procedures: Superelevation

Superelevation is the transverse rise in water surface that occurs around a channel bend, measured between the theoretical water surface at the centerline of a channel and the water surface elevation on the outside of the bend. Superelevation in bends shall be estimated from the following Equation 7-21.

Equation 7-21. Superelevation in Bends Estimate Calculation

$$\Delta y = \frac{CV^2 T_W}{rg}$$

where:

Δy	=	rise in water surface between design water surface at centerline of channel and outside water surface elevation (ft)
C	=	curvature coefficient (see Table 7-10)
r	=	radius of curvature at centerline of channel (ft)
T_W	=	top width at the design water surface at channel centerline (ft)
V	=	mean channel velocity (ft/s)
g	=	gravitational acceleration (ft/s^2)

The curvature coefficient (C) shall be 0.5 for subcritical flow conditions. For supercritical flow conditions, the curvature coefficient shall be 1.0 for all trapezoidal channels and for rectangular channels without transition curves and 0.5 for rectangular channels with transition curves. Table 7–

10 provides superelevation curvature coefficients for various flow regimes, cross-section shapes, and types of curves.

Bends in supercritical channels create cross-waves and superelevated flow in the bend section as well as further downstream from the bend. In order to minimize these disturbances, best design practice is to design the channel radius of curvature to limit the superelevation of the water surface to 2 feet or less. This can be accomplished by modifying Equation 7–21 to determine the allowable radius of curvature of a channel for a given superelevation value.

Table 7-10. Superelevation Curvature Coefficients

Flow Type	Cross Section	Type of Curve	Curvature C
Subcritical	Rectangular	No Transition	0.5
Subcritical	Trapezoidal	No Transition	0.5
Supercritical	Rectangular	No Transition	1.0
Supercritical	Trapezoidal	No Transition	1.0
Supercritical	Rectangular	with Spiral Transition	0.5
Supercritical	Trapezoidal	with Spiral Transition	1.0
Supercritical	Rectangular	with Spiral Banked Transition	0.5

7.10. Design Procedures: Alluvial (Movable-Bed) Channels

Alluvial-bed channels in the context of this Manual refers to channels with movable beds that operate in quasi-equilibrium with respect to longitudinal slope and cross-section. This Section outlines the basic procedures and concepts for the design of alluvial (movable-bed) channels. The procedures presented here are useful for small to moderate sized alluvial bed channels. Major channels warrant more thorough sediment transport analysis, usually involving computer modeling. Sediment transport analysis is not required for small ditches and swales. The design engineer is encouraged to explore the references provided in the reference section of this Manual for more comprehensive discussions of alluvial-bed channel design.

The procedures outlined in this Manual shall not be used for the design of channels on active alluvial fan formations.

7.10.1 Basic Design Procedure

The basic alluvial channel system consists of a composite cross-section with a low-flow channel and a flood-flow channel. The low flow channel section shall be designed to handle the channel forming discharge flow with no freeboard. In addition, the alluvial channel section design shall be checked for the 100-year flows to ensure that the system will be stable during and after major storm events. The following summarizes the recommended general alluvial channel design procedures:

Step 1. Identify the contributing watershed limits and determine the channel-forming discharge (see Section 7.10.2) and the 100-year design storm discharge rates for the design reach.

Step 2. Obtain pertinent information, including channel geometry, channel slope; channel resistance; and sediment size distribution (based on geotechnical analysis) for the upstream sediment supply reach (see Section 7.10.6). Calculate the hydraulic conditions based on the design discharges.

Step 3. After determining the applicability of the sediment transport equation (see Section 7.10.5), calculate the sediment supply from the upstream channel reach for the channel forming discharge. The calculated sediment supply is per unit width; the total sediment transport rate is obtained by multiplying the rate per unit width by the top width of the natural channel section.

Step 4. Determine the equilibrium slope (see Section 7.10.3) for the channel design reach under consideration using the upstream sediment supply rate. This usually requires a trial and error procedure. When the computed transport rate is equal to the upstream supply rate, the equilibrium slope for the design reach has been found.

Step 5. Based on the hydraulic conditions at equilibrium slope, estimate the largest particle size moving for armoring control check (see Section 7.10.7). Also, check the applicability of the equations used for the calculation by comparing hydraulic parameters with the range of parameters for the equations.

Step 6. If needed, design channel drop structures (see Section 7.11) and other necessary drainage structures to maintain the computed equilibrium channel slope for the design reach.

Step 7. Check the stability (horizontal and vertical) of the channel design reach (channel and overbanks) for the design storm events and provide channel protection measures where needed (see Section 7.6 and elsewhere).

7.10.2 Channel Forming Discharge

In natural settings, the shape and size of the defined channel portion of a drainage way is usually formed and controlled by the “channel forming” discharge. The channel forming discharge is often called the bank-full discharge. Research has concluded that the channel forming discharge rates most frequently range between 1-year and 5-year storm event (Leopold & Maddock, 1953; Wolman & Leopold, 1957; Dury, 1973; Pickup & Warner, 1976; Richards, 1982; Leopold, 1994). In the absence of specific studies or other geomorphologic evidence, the 2-year design storm flow shall be assumed as the channel-forming discharge for alluvial channel design.

7.10.3 Equilibrium Slope

The channel energy gradient slopes largely affect the resulting flow velocities, tractive forces, and sediment transport capacities of a given channel reach. A channel reach is considered to be at an equilibrium slope when the incoming sediment load (sediment supply) is equal to the outgoing sediment load (sediment transport). If the sediment transport capacity of a given channel reach is greater than sediment supply from the upstream reach, the channel reach will experience degradation (erosion and scour) because of flood flows picking up additional sediment particles from the channel bed and banks. If the sediment supply is greater than the sediment transport capacity, then the channel reach will experience aggradation as the flows drop off excess sediment particles. When a channel reach is in an equilibrium state, no substantial channel aggradation or degradation is expected. The equilibrium channel slope is typically determined based on the channel forming discharge rate.

7.10.4 Composite Manning Roughness Coefficient

Appendix C provides recommended values for the Manning roughness coefficient for various channel and overbank types and conditions. The composite Manning roughness coefficient is determined by Equation 7-22 (Chow, 1959).

Equation 7-22. Composite Manning Roughness Coefficient Calculation

$$n_c = \frac{(n_o{}^2 P_o + n_w{}^2 P_w)^{0.5}}{(P_o + P_w)}$$

where:

n_c	=	Manning roughness coefficient for the composite channel
n_o	=	Manning roughness coefficient for areas above the wetland area
n_w	=	Manning roughness coefficient for the wetland area
P_o	=	wetland perimeter of channel cross-section above the wetland area (feet)
P_w	=	wetland perimeter of the wetland channel bottom (feet)

7.10.5 Sediment Supply and Transport Analysis

A long reach of channel may be subjected to a general degradation or aggradation of the bed level over a long period of time. Anticipating degradation and aggradation accurately is important for determining design considerations such as adequate foundation depths.

Sediment routing analysis using a sediment routing model is the best method for estimating the general degradation and aggradation of a stream on a reach-by-reach basis. Examples of sediment routing computer models include the U.S. Army Corps of Engineers' HEC-6 Scour and Deposition in Rivers and Reservoirs, and proprietary models such as QUASED by Simons, Li & Associates; FLUVIAL-12 by Howard Chang; MIKE-21C by the Danish Hydraulic Institute; and ONETWOD by Y. H. Chen (FERC, 1992). However, less elaborate methods using rigid bed hydraulic and sediment transport calculations may be used to estimate the relative balance between sediment transport capacity and sediment supply between adjacent reaches. The design engineer shall determine the level of sediment transport analysis required for a particular alluvial channel design project in consultation with the City.

7.10.6 Upstream Sediment Supply

A major controlling factor when assessing channel response is the upstream sediment supply. Whether a channel degrades or aggrades depends on the balance between the incoming sediment supply and the transport capacity of the reach. This is especially true for channels where armoring does not occur.

Incoming sediment supply is very difficult to estimate. One practical way to estimate the incoming sediment supply is to select a supply reach. The supply reach must be close to its equilibrium condition. Usually, the sediment supply is determined from the following:

1. The sediment transport capacity at an upstream reach, using an appropriate maximum permissible velocity and estimated flow depth;
2. A natural channel reach upstream of the design reach that has not and will not be disturbed by human activities; or
3. An upstream channelized reach that has existed for many years and has not experienced a recent change in profile or cross section.

It is important to understand that the sediment supply system may be subjected to conditions that can drastically alter the sediment supply, such as urbanization and the construction of debris basins and detention ponds. In the long term, urbanization can reduce exposure to erosion and reduce sediment supply. Likewise, detention basins and debris basins will trap and prevent sediment from entering the stream system.

7.10.7 Erodible Sediment Size

The sediment transport equations presented here are based on the assumption that all the sediment sizes present in the channel bed can be moved by the flow. If this is not true, armoring will take place, and the equations will not be applicable. Similarly, these equations do not apply to conditions when the bed material is cohesive. The bed shear stress is given by two closely related equations (see Equation 7-23).

Equation 7-23. Bed Shear Stress Calculation

Calculation 1

$$\tau_0 = \gamma R S$$

Calculation 2

$$\tau_0 = (1/8)\rho f_0 V^2$$

where:

τ_0	=	shear stress (lb/ft²)
γ	=	specific weight of water (62.4 lb/ft³)
R	=	hydraulic radius (ft)
S	=	energy slope (ft/ft)
ρ	=	density of water (lb/ft³)
f_0	=	Darcy-Weisbach friction factor
V	=	mean flow velocity (ft/s)

Equation 7–23 is usually the simplest to utilize. The diameter of the largest particle moving is then determined by Equation 7–24.

Equation 7-24. Diameter of Largest Particle Moving Calculation

$$D = \tau_0 / (0.047 (S_S - 1)\gamma)$$

where:

D	=	diameter of the sediment (ft)
S_S	=	specific gravity of sediment
0.047	=	recommended value of Shields' parameter

7.10.8 Other Channel Scour Considerations

Additional channel erosion and scour conditions such as anti-dune trough depth, channel bend scour, channel contraction scour, and local scour at abutments, piers, etc. might have significant design implications for an alluvial channel. FHWA HEC-18, “Evaluating Scour at Bridges,” and several other references listed for Chapter 7 in the reference section of this Manual offer several references for more detailed discussion of these considerations.

7.11. Design Procedures: Channel Grade Control and Drop Structures

The design of stable open channels (rigid and alluvial) often requires the use of channel drop and/or grade control structures to control the longitudinal slope of channels to keep design velocities within the acceptable limits.

Channel grade control and drop structures presented in this Section shall only be used when the inflow channel condition is subcritical (FR <1.0). If the inflow channel condition is super critical (FR >1.0), an energy dissipator or stilling basin shall be used instead (see Chapter 8).

Channel grade control and drop structures may be constructed of many types of materials, including concrete, riprap, grouted riprap, gabions, sheet piles, or other materials. The selection of material and type of grade control depends in part on their hydraulic limitations (see Table 7–11 for typical hydraulic limitations), aesthetic considerations, and other site conditions such as presence of abrasive sediment bed load. This Section presents minimum design criteria and charts to aid in the design of sloping grouted boulder grade control structure. The reference section of this Manual provides several references for channel drop and energy dissipation design with detailed information available on other types of structures.

Table 7-11. Channel Drop Structures

Description	Upstream Flow Regime	Max. Drop Height (ft)	Max. Unit Discharge	Max. Inflow Velocity (ft/s)	Upstream Cross-Section
Sloping Riprap Drop Structure	Subcritical	10	35	7	Trapezoidal
Vertical Riprap Drop Structure	Subcritical	3	35	7	Trapezoidal
Straight Drop Structure	Subcritical	8	n/a	n/a	Rectangular
USBR Type IX Baffled Apron	Subcritical	n/a	60	n/a	Rectangular

The effectiveness of grade control and drop structures is dependent on many factors, including discharges, tailwater depths, and type of structures. The structures also must function over a wide range of discharges. Therefore, it is important to confirm performance during events smaller than the maximum design flow. This may be accomplished by evaluating flows of specific more frequent storm events (e.g., 10-year, 2-year, etc.), or testing successive fractions (e.g., one-half, one-quarter, and further if necessary) of the design flow. However, only calculations for the full design flow are required to be submitted for review.

7.11.1 Sloping Grouted Boulder Drop Structure

Sloping grouted boulder drop structures have gained popularity due to their design aesthetic and successful field application. Figure 7–18 illustrates a typical sloping grouted boulder drop structure. The sloping grouted boulder drop is designed to operate as a hydraulic jump dissipator, although some energy loss is incurred due to the roughness of the grouted rock slope. The quality of rock used and proper grouting procedure are very important to the structural integrity. The main design objectives are to maintain structural integrity and to contain erosive turbulence within the downstream basin.

Figure 7-18. Example of Sloping Grouted Boulder Drop Structure

Grouted boulder drops must be constructed of uniform size boulders grouted in place through the approach, sloping face, basin, and exit areas of the drop. Figure 7–19 illustrates the general configuration of the sloping grouted boulder drop structure, and Tables 7–12 and 7–13 summarize the design parameters for the drop structure.

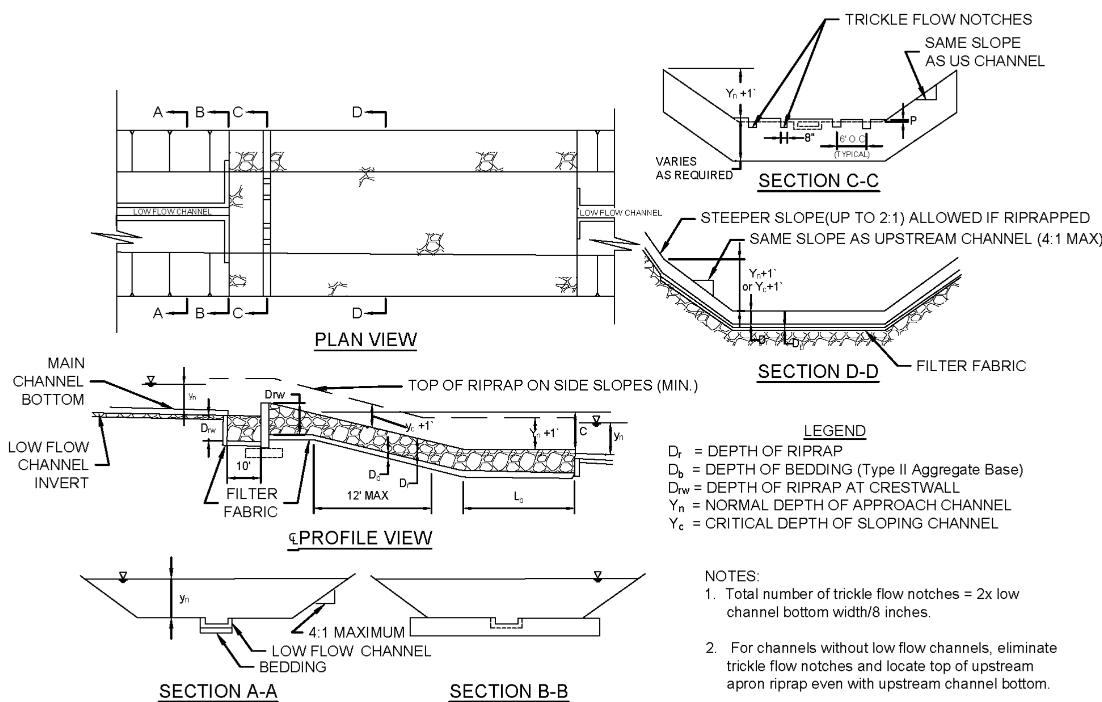


Figure 7-19. Grouted Riprap Drop Structure (for Illustration Only)

Table 7-12. Sloping Riprap Channel Drop Design Chart – Part 1

Max. Unit Discharge (cfs/ft)	Max. Allowable Chute Slope (S_o)		Downstream Apron Length (L_b) (ft)
	¼ Ton Riprap	Grouted Riprap	
0-15	7H:1V	4H:1V	15
15-20	8H:1V	5H:1V	20
20-25	10H:1V	6H:1V	20
25-30	12H:1V	7H:1V	25
30-35	13H:1V	8H:1V	25
> 35	Structure Not Allowed for Unit Discharges > 35 cfs/ft		
Incoming Velocity	**Riprap Apron Thickness (D_r)**		
	(ft)	**(ft)**	
V ≤ 5 ft/s	1.75	2.6	
V > 5 ft/s	2.0	3.0	
	Riprap Apron Thickness at Crest (D_{rw})		
	(ft)	**(ft)**	
	1.5 D_r	1.25 D_r	

Table 7-13. Sloping Riprap Channel Drop Design Chart – Part 2

Channel Bottom Width (b)	Crest Wall Elevation (P)		
	Y_N ≤ 4 ft	Y_N > 4 ft	
		V_N ≤ 5 ft	V_N > 5 ft/s
(ft)	(ft)	(ft)	(ft)
≤ 20	0.1	0.2	0.2
20-60	0.1	0.4	0.2
60-100	0.1	0.5	0.3
> 100	0.2	0.5	0.3

7.11.1.1 Approach Apron

The grouted boulder drop structure has a 10-foot trapezoidal riprap approach section immediately upstream of its crest. The approach apron is provided to protect against the increasing velocities and turbulence that result as the water approaches the sloping portion of the drop structure. The width of the approach apron and the side slopes should match the upstream channel, and the height of grouted boulder channel sides shall be equal to the depth of water in the upstream channel plus the required freeboard as described in Section 7.2.7.

A concrete cutoff wall shall be placed at the top of the slope and on the upstream side of the approach apron to reduce or eliminate seepage and piping through the structure (Figure 7–19). The depth of the cutoff wall shall be at least 1 foot and extend the full depth of the riprap layer. Depending on the soil type and hydraulic forces acting on the drop structure, the cutoff wall may need to be deeper to lengthen the seepage flow path.

7.11.1.2 Drop

The slope of the drop structure shall not be steeper than 4H:1V (Table 7–12). Slopes flatter than 4H:1V usually increase expense, but some improvement in appearance may be gained. The side slopes and bottom width of the drop shall be the same as the upstream channel. The grouted boulders shall extend up the side slopes a height of the tailwater depth plus freeboard as projected from the downstream channel or the critical depth plus 1 foot, whichever is greater.

7.11.1.3 Exit Apron

The exit apron is necessary to minimize any erosion that may occur due to secondary currents. The bottom width and side slopes of the exit apron shall be the same as the downstream channel. The apron sides shall extend to a height equal to the tailwater depth plus the required freeboard. Table 7–12 provides the length of the exit apron (L_b).

7.11.1.4 Drainage

Drop structure shall include appropriate structural analysis and analysis of geotechnical factors such as seepage. Weep drains should be considered for seepage and uplift control. A continuous manifold is preferred over a “point” system for weep drainage of a drop structure, as it provides more complete interception of subsurface drainage. Weep systems requires special attention during construction. The boulders can crush the pipes and alignment of the pipes between the boulders can be difficult. Flexible outlet pipes shall be used to allow alignment of the pipes around the boulders when necessary.

7.11.2 Drop Structures Used for Grade Control

The natural topographic slope of a project reach can often be too steep for a stable alluvial channel or particular engineered channel design. In these cases, the grouted sloping boulder drop structure and other drop structure designs can used as grade control structures to limit longitudinal slope of a channel.

The basic design procedure for grade control structures starts with the determination of a stable slope and configuration for the channel. For alluvial channels, the analysis should include discharges from the full floodplain flow to the dominant discharge. Section 7.9.5 explains the dominant or channel-forming discharge, and it is more fully explained in sediment transport texts such as Simons, Li and Associates (1982). The spacing of the grade control structures is based on the difference in slope between the natural topographic and projected stable slope.

7.11.3 Grade Control Sills Used for Grade Control

Control sills are another type of grade control structure that can be used to stabilize channels. Grade control sills can be constructed of concrete, or designed using materials such as gabion baskets or sheet piles. It is important that the sill extend below anticipated scour depths and far enough into adjacent channel banks to prevent flanking during high flow events. The top of the grade control sill should conform to the transverse channel cross-section profile when practicable.

The basic design procedure for grade control sills is to (1) determine a stable slope (Section 7.10.3), and then (2) determine spacing of the sills based on the difference in slope between the natural and projected stable slope. It is critical to take care to limit the vertical drop below grade control sills and provide adequate scour protection for the structure.

THIS PAGE INTENTIONALLY LEFT BLANK FOR DOUBLE-SIDED PRINTING

Chapter 8

Detention Basins for Flood Control

The development of a site often results in an increase in peak discharges and velocities of runoff from the property. Storm water detention facilities temporarily store storm runoff and release it in a controlled manner in order to reduce or eliminate flooding or other adverse effects downstream. The temporary storage of storm water can often decrease the cost of downstream conveyance facilities and provide water quality benefits.

Storm water facilities in the San Diego Region are typically designed to convey ultimate (post-developed) condition peak flows. Therefore, detention basins have historically not been a major component of regional flood management. Detention facilities shall only be specified when necessary and approved by the governing Agency.

This Chapter discusses general design criteria for storm water detention, standard features of detention facilities, and maintenance issues. This Chapter also provides guidance on detention routing analysis and hydraulic design of storm water detention facilities.

The discussion in this Manual focuses on storm water detention planning and design in the context of storm water conveyance and flood management (i.e., storm water quantity). For a more detailed discussion of storm water quality issues and design criteria, the design engineer is directed to other resources, including the City of San Diego Storm Water Standards Manual.

This Manual addresses storm water detention facilities, which only temporarily detain storm water. Other facilities commonly referred to as retention facilities, capture all the runoff from a watershed and have no outlet structures to release water downstream. While retention structures have many applications (e.g., water quality infiltration and irrigation), they are beyond the scope of this Manual.

Storm water detention facilities can be classified in many ways based on their design characteristics. The discussion in this Manual focuses on dry-pond, on-line storm water detention facilities. In many cases, site constraints or other project requirements may warrant consideration of alternative types of facilities. At a minimum, all detention facilities must meet the release rate criteria outlined in Section 7.1. This Manual's guidance on other detention facility features, routing calculations, and hydraulic design can be adapted and applied to these situations with due care, and specific design criteria developed in consultation with the governing Agency. As a reference, the following paragraphs describe several categories of detention facilities.

Dry Pond. Water remains in wet pond detention facilities during the dry periods between storm events. Wet pond outlet structures are elevated above the lowest elevation of the basin. Wet ponds are only feasible when inflows into the pond exceed the loss rates from the pond due to infiltration and evaporation. Wet ponds often have aesthetic and water quality advantages, but can experience problems such as odors, floating debris, and vectors such as mosquitoes when not properly maintained. Therefore, wet pond detention facilities are discouraged in the City of San Diego. Dry pond facilities drain completely between storm events, with outlets positioned at or below the lowest elevation of the basin.

Aboveground versus Belowground. Aboveground detention facilities typically consist of a depressed or excavated area, often with an earthen dam or embankment. Belowground facilities may be appropriate when there is not adequate surface area for aboveground detention. Underground facilities may have special hydraulic and maintenance requirements such as a forebay or pre-treatment of incoming flows, that must be considered carefully during the selection and design process.

8.1. Design Criteria

8.1.1 Protection Levels (Release Rate)

Detention basins may be required depending on site-specific conditions. Detention basin release rates must meet the requirements of local downstream drainage facilities. Examples of these conditions and requirements include:

1. Downstream drainage facilities are insufficient to convey post-project peak design flows. The capacity of downstream conveyance systems shall be analyzed in accordance with this Manual and compared against the peak design runoff from the tributary watershed assuming fully improved conditions. If the downstream facilities are not improved, a detention basin may be required to reduce peak flows to the capacity of the existing downstream facility.
2. The local drainage master plan limits the maximum peak flow in a particular facility. A detention basin may be required to reduce peak flows to the level prescribed by the drainage master plan.

Flood-control detention facilities are not required when the project discharges to a master-planned regional flood control facility designed to accommodate increased developed-condition flows. The design engineer shall confirm with the governing Agency whether detention is required at a particular site. The magnitude of the peak flow and type of hydrograph shall be determined following methods outlined in Chapter 2.

The design engineer must also determine if there are other project-specific requirements based on the project's location in a particular watershed or applicable requirements outlined by other reviewing Agencies. For instance, the San Diego Resource Protection Ordinance requires the attenuation of particular storm frequencies on certain watersheds, and Agencies such as the California Coastal Commission, Regional Water Quality Control Board (Regional Board), and the California Department of Water Resources, Division of Safety of Dams (DSOD) might have specific requirements for projects within their jurisdiction.

The design engineer shall confirm that outflow from a detention basin shall not have a detrimental effect on downstream facilities. In some cases detention facilities, while reducing peak on-site flows, might increase the peak flow from the watershed as a whole by delaying the on-site peak discharge to coincide more closely with the peak from the larger watershed. Therefore, hydrologic and hydraulic analyses must extend downstream from the project site to determine the regional effect of a detention basin. The outflow hydrograph from a detention facility shall be routed (analyzed) through the downstream conveyance system to a reasonable point, typically to the next improved drainage facility.

8.1.2 Jurisdictional Dams

The State of California defines a dam as any artificial barrier, together with appurtenant works, that impounds or diverts water (California Water Code, Division 3, Section 6002-6003). The State Division of Safety of Dams (DSOD) does not regulate structures that are 6 feet or less in height, regardless of storage capacity, nor do they regulate structures that have a storage capacity of 15 acre-feet or less, regardless of height. However, the DSOD does regulate any structure that meets the following criteria:

1. Dams with the capacity to store over 15 acre-feet of water and 25 feet or more in height, measured from the natural bed of the stream or watercourse at the downstream toe of the barrier to the maximum possible water storage elevation. When the structure is not across a stream channel or watercourse, the height of the structure is measured from the lowest elevation of the outside limit of the barrier; or
2. Dams over 6 feet high with the capacity to impound 50 acre-feet or more of water.

Figure 8–1 provides an illustration of DSOD depth and volume regulatory thresholds that define jurisdictional dams. Structures meeting the definition of jurisdictional dam have additional design criteria and require approvals from the DSOD, including special construction monitoring. The design engineer must consult with the governing Agency and the DSOD when considering the construction of a detention facility that falls within DSOD jurisdictional limits.

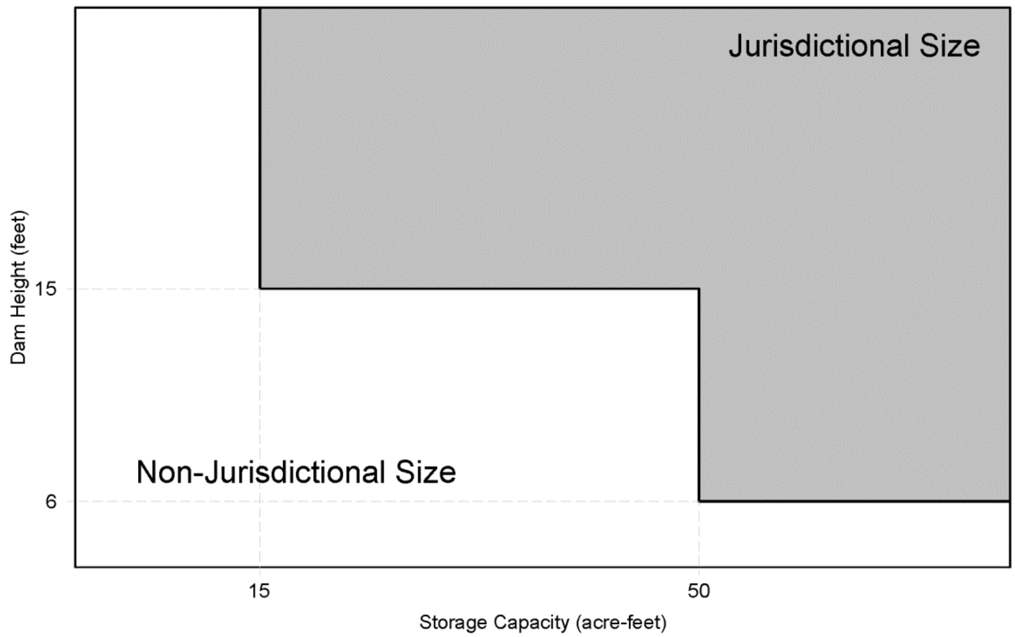


Figure 8-1. California DSOD Jurisdictional Dam Thresholds

8.1.3 Grading and Embankment Slopes

Dry-pond facilities shall have a low-flow ("trickle") channel to facilitate dry pond drainage between storm events. The base of all aboveground detention facilities shall have a minimum slope of one percent draining to the low-flow channel or the outlet. Forebays are recommended to capture and confine sediment and debris in one area of the detention facility.

Slopes of aboveground detention facilities shall have side slopes of 3H:1V or milder whenever practical. Detention basins with slopes steeper than 3H:1V or where the maximum design water depth exceeds 3 feet shall be fenced to control access. The top of the embankment shall have 1 foot of freeboard above the maximum water surface elevation when the emergency spillway is conveying the maximum design flow (the 100-year undetained flow, see Section 8.1.1). The embankment shall be appropriately armored with riprap or other slope protection when necessary.

Detention facilities and embankments often have particular geotechnical considerations and might require details such as impermeable clay or synthetic liners. Detailed design criteria for the geotechnical design of detention facility embankments are beyond the scope of this Manual. At a minimum, geotechnical investigations shall analyze the special conditions encountered at detention facilities, such as the potential for saturated soils and rapid drawdown. The design engineer may consult references such as the Design of Small Dams (U.S. Bureau of Reclamation, 1987) for further information.

8.1.4 Standard Features (Inlets, Outlets, and Emergency Spillways)

This section provides minimum design criteria for standard detention facility features such as inlets, outlets, and emergency spillways. Figure 8–2 presents an illustration of the standard features of an aboveground detention basin.

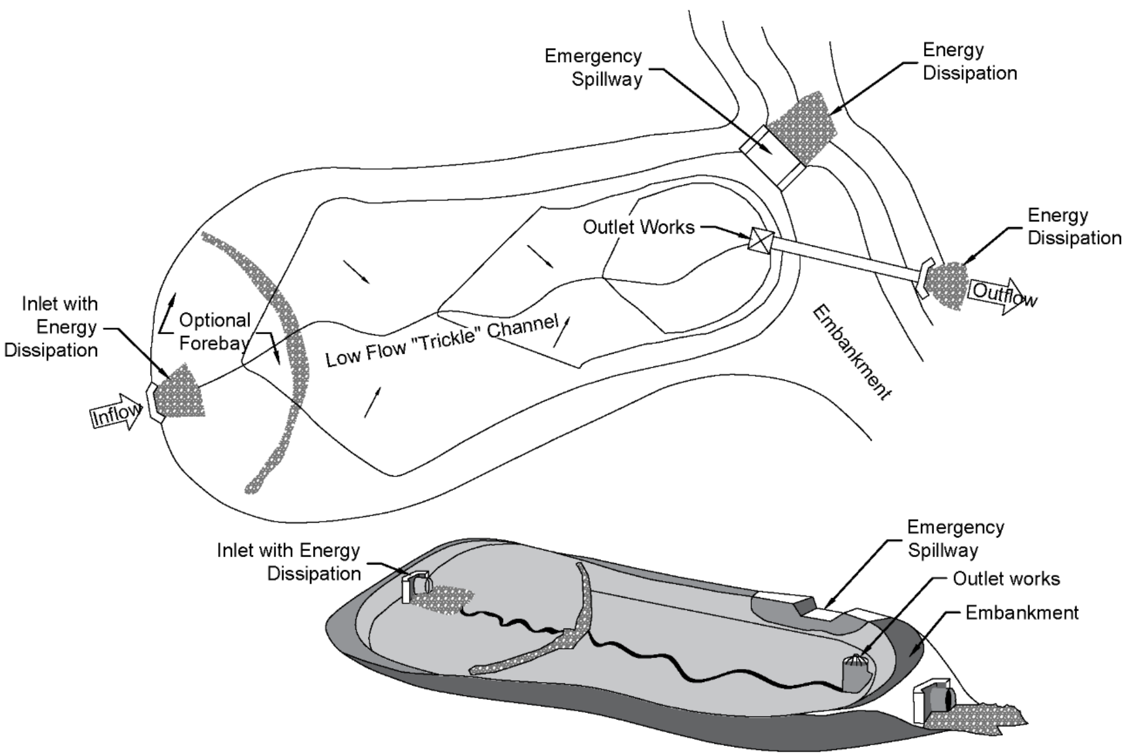


Figure 8-2. Plan and Section of Typical Flood Control Detention Basin

8.1.4.1 Inlet Structures

Aboveground detention facilities shall have adequate energy dissipation and/or erosion protection at the facility inlet to avoid damage as flow enters the facility. Chapter 9 provides a discussion of energy dissipation devices. Incorporating forebays and sediment traps at inflow points to larger basins can reduce the amount of sediment and debris to the main part of the facility, and are encouraged whenever practical.

8.1.4.2 Outlet Structures

Outlet structures must be carefully designed to ensure proper facility operation, to facilitate maintenance, and to maintain safety. Outlet structures for detention facility shall be designed to safely convey the design release rate as discussed in Section.8.1.1. Outlet structures shall completely drain the facility within 72 hours of the end of the storm event.

Outlet structures shall be constructed with no moving parts whenever practical. Collars or other anti-seepage devices shall be provided for all outlet structures conveying flow through a detention basin embankment. Appropriate energy dissipation shall be provided downstream of all detention facility outlet works (see Chapter 9).

Riser pipes and culvert outlet structures shall be equipped with debris racks, screens, or anti-vortex devices in order to help prevent clogging, and to prevent entry by unauthorized persons. These appurtenances shall be well secured but removable for the purposes of maintenance. Debris racks must not interfere with the hydraulic capacity of the outlet. Riser pipes or outlet structures with an inside dimension of 36 inches or larger and taller than 6 feet shall have ladder rungs or similar safety devices to facilitate access by maintenance personnel.

8.1.4.3 Emergency Spillways

Emergency spillways provide a safe means for conveying flows in excess of the maximum design capacity of the outlet works. Spillways shall be designed to pass flow from an "undetained" 100-year design event (i.e., the maximum 100-year peak flow that enters the basin) as defined in the San Diego County Hydrology Manual. Spillways shall be appropriately protected to prevent excessive damage to the structure or adjacent property during spill events. Spillways shall have appropriate downstream energy dissipation (see Chapter 9).

8.1.4.4 Other Features

Appropriate signage warning that areas are subject to flooding during storm events shall be provided for detention facilities designed for conjunctive recreational use. Detention facilities located near roadways shall have guardrails or other safety measures acceptable to the City.

8.1.5 Detention Facility Plans

The design engineer shall note the following on detention facility drawings: maximum design inflow and velocity; maximum total design outflow and velocity from the outlet works; maximum design storage volume and water surface elevation in the facility; and the maximum design flow, depth, and velocity over the emergency spillway. Plans for detention facilities shall include appropriate details for the facility inlet, outlet structures, energy dissipators, the emergency spillway, maintenance measures, and cross-sections of embankment fills. All detention facilities shall provide a debris/sediment depth gauge or other mechanism that will serve as a maintenance guide.

8.1.5.1 Operation and Maintenance Plan

All detention facilities shall have an operation and maintenance plan. These operation and maintenance plans shall specify regular inspection and maintenance at specific time intervals (e.g., annually before the wet season) and/or maintenance "indicators" when maintenance will be triggered (e.g., an accumulation of 6 inches of sediment and debris, or the basin does not drain within 72 hours). Operation and maintenance plans shall ensure that vegetation is removed or maintained on a regular basis to preserve the function of the facility.

8.1.5.2 Maintenance Access

Detention facilities shall be accessible to maintenance personnel and equipment for the removal of accumulated silt and debris, and the maintenance and repair of inlets, outlets, spillways, and embankments. Aboveground detention facilities greater than 3 feet in depth with more than 1 acre-foot of storage volume must provide access for maintenance equipment whenever practical. The detention facility design shall provide stable access to the base of the facility, outlet works, inlets, and any forebays. Maintenance access roads shall be designed to County of San Diego private road standards.

8.1.5.3 Easements and Maintenance Mechanisms

All detention basins require lifetime maintenance. The project owner and design engineer shall consult the governing Agency for determination of which maintenance mechanism is required for a particular project. At a minimum, privately owned and maintained detention facilities shall have a recorded easement agreement with a covenant binding on successors, or another mechanism acceptable to the governing Agency.

8.1.5.4 Environmental Permitting

Detention facilities are often located within or adjacent to sensitive environmental areas. The design engineer must investigate which permits might be necessary from various Agencies, including but not limited to:

1. U.S. Army Corps of Engineers (e.g., Section 404 Wetland Permit), U.S. Department of Fish and Wildlife,
2. California Department of Fish and Wildlife (e.g., Section 1600 Permit),
3. California State Water Resource Control Board and Regional Water Quality Control Board (e.g., Section 401 Water Quality Certification), and
4. California Coastal Commission.

It is important that the final permits and/or permit conditions allow for the future and perpetual maintenance of a detention facility without the necessity of returning to the permitting Agency.

8.1.6 Conjunctive Use of Detention Facilities

Conjunctive use means the use of a facility for two or more purposes. Because dry-pond detention facilities do not store water between storm events, it is often possible to propose conjunctive uses for detention facilities involving water quality treatment and active or passive recreation. Forebays are recommended to capture and confine sediment and debris in one area of the detention facility to enhance the possibilities for conjunctive use by reducing the scale of maintenance.

Conjunctive use of detention facilities for water quality treatment and flood management is acceptable and encouraged when it is desirable and feasible. When an aboveground detention facility is used for both water quality and flood control, the flood storage volume shall be provided in addition to the storage volume designated for water quality treatment. Design criteria for water quality facilities is beyond the scope of the Manual; the design engineer is referred to the City of San Diego Storm Water Standards Manual or the appropriate governing Agency's storm water quality manual for water quality aspects of detention basin design.

8.2. Design Procedure: Detention Routing Analysis

This section presents general procedures for the hydrologic (routing) analysis of detention basin performance. By following the analysis procedures outlined here, the design engineer can design a detention facility design that successfully meets the release rate criteria outlined in Section 8.1.

8.2.1 Basic Data

Storage routing and design calculations primarily depend upon three basic data:

1. the inflow hydrograph,
2. the stage-storage relationship, and
3. the stage-discharge relationship for the outlet structures.

8.2.1.1 Inflow Hydrograph

The inflow hydrograph to a detention facility shall be determined using the methods outlined in the San Diego County Hydrology Manual (June 2003). Figure 8–3 illustrates the relationship between inflow and outflow hydrographs when routed through a detention facility.

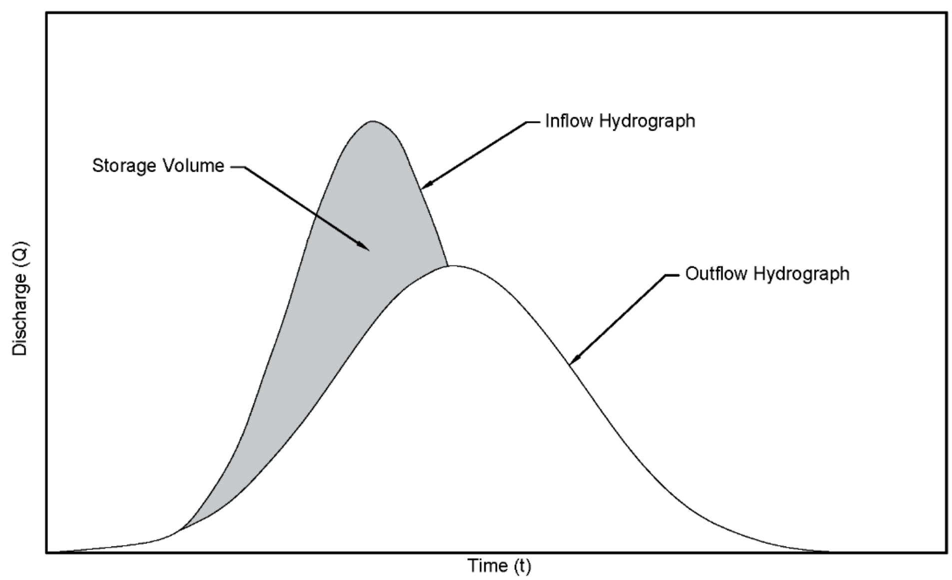


Figure 8-3. Example of Inflow Hydrograph and Outflow Hydrograph

8.2.1.2 Stage-Storage Curve

Stage-storage curves define the relationship between the depth of water (stage) and the storage (volume) available in the reservoir. Stage-storage curves are typically developed using topographic mapping and/or grading plans for the detention facility. Examples of equations that may be used to estimate the stage-storage curve may be determined by either an average-end area calculation (Equation 8–1) or as the volume of a conic frustum (Equation 8–2).

Equation 8-1. Average-End Area Calculation

$$V_{1,2}=\frac{(A_1+A_2)}{2}(h_2-h_1)$$

where:

$V_{1,2}$	=	storage volume between elevations h_1 and h_2 (ft^3)
A_1, A_2	=	surface area at elevations h_1 and h_2, respectively (ft^2)
h_1, h_2	=	lower and upper bounding elevations, respectively (ft)

Equation 8-2. Volume of Conic Frustum Calculation

$$V_{1,2}=\frac{1}{3}\left(A_1+A_2+\sqrt{A_1A_2}\right)(h_2-h_1)$$

where:

$V_{1,2}$	=	storage volume between elevations h_1 and h_2 (ft^3)
A_1, A_2	=	surface area at elevations h_1 and h_2, respectively (ft^2)
h_1, h_2	=	lower and upper bounding elevations, respectively (ft)

The stage-storage curve begins at the bottom of the storage basin or the maximum elevation of sediment or debris allowed in the operation and maintenance plan, whichever is greater. Volume reduction factors may be applied to account for vegetation and/or additional sediment and debris deposition within the detention facility when necessary.

8.2.1.3 Stage-Discharge Curve

Stage-discharge curves define a relationship between the depth of water in the detention facility and the outflow or release from its outlet structures. Section 8.3 describes the basic procedures for calculating discharges from outlet control structures. Figure 8–4 illustrates a typical stage-discharge curve.

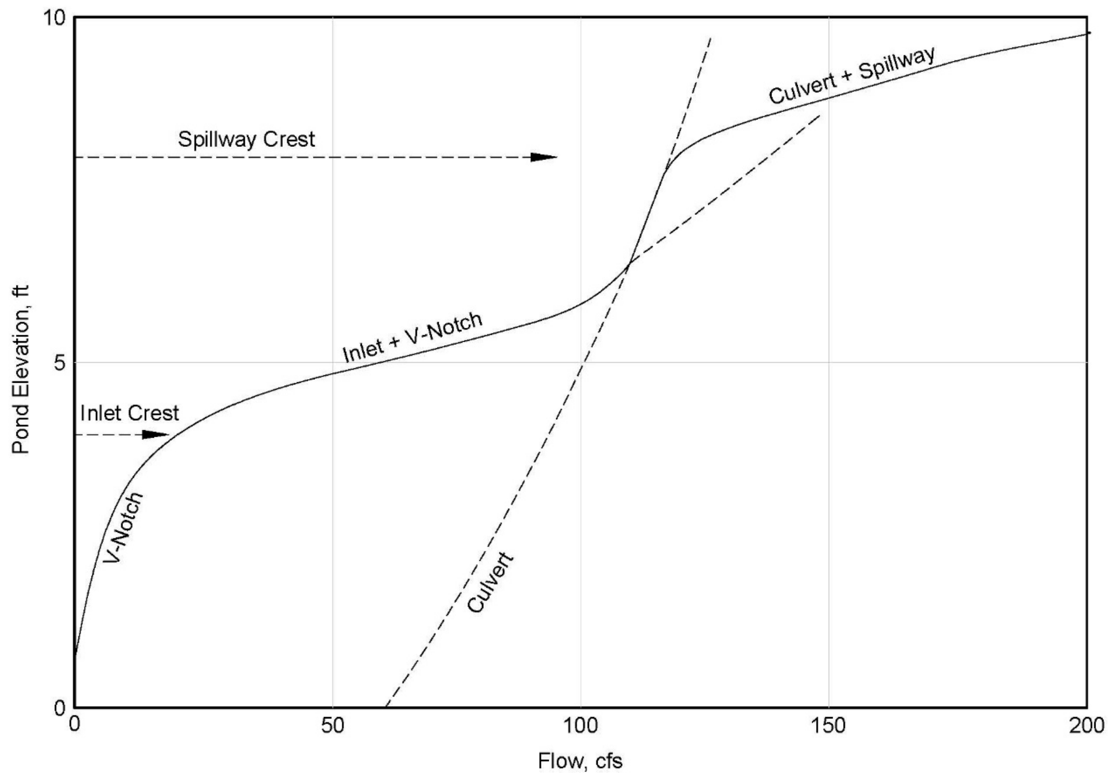


Figure 8-4. Example of Stage-Discharge Curve

8.2.2 Storage Routing Calculations

Routing is the process of analyzing flows entering and leaving a detention facility in order to determine the change of the water surface elevation within the facility over time. Storage routing calculations are typically performed using computer programs. The routing of flows through a detention facility is fundamentally based on conservation of mass (Inflow – Outflow=ΔStorage), approximated as a finite-difference found by Equation 8–3.

Equation 8-3. Conservation of Mass Calculation

$$\frac{S_{n+1}-S_n}{\Delta t}=\frac{I_n+I_{n+1}}{2}-\frac{O_n+O_{n+1}}{2}$$

where:

S_n, S_{n+1}	=	storage within a detention facility at a time step n and n+1, respectively (ft^3)
Δt	=	time interval (sec)
I_n, I_{n+1}	=	inflow rate at a time step n and n+1, respectively (ft^3/s)
O_n, O_{n+1}	=	outflow rate at a time step n and n+1, respectively (ft^3/s)

The most common method for performing routing analysis for a detention facility is the storage indication or modified Plus method. The storage indication method re-arranges the expression for mass conservation is presented in Equation 8-4.

Equation 8-4. Rearranged Expression for Mass Conservation

$$\left(\frac{2S_{n+1}}{\Delta t}+O_{n+1}\right)=\left(\frac{2S_n}{\Delta t}-O_n\right)+(I_n+I_{n+1})$$

The left-hand side of Equation 8–4 is usually called the storage indication number. The storage indication method facilitates the routing analysis of detention facilities, which can be accomplished by hand calculations or using computer programs such as the Corps of Engineers' HEC-1 Flood Hydrograph Package, HEC-HMS Hydrologic Modeling System, or proprietary software packages.

8.3. Design Procedure - Outlet Structures and Spillways

The type and configuration of outlet structures and emergency spillway establish the hydraulic performance of a detention facility. This section describes the basic methods available to define the performance curve for common types of detention facility outlets, including culverts, weirs, orifices, risers, perforated risers, and combination outlets.

8.3.1 Culverts

The hydraulic behavior of culverts is complex because different types of flows can occur depending upon the upstream/downstream conditions, discharge, and the barrel and inlet characteristics. Chapter 4 of this Manual presents guidance for the analysis of culvert flow.

8.3.2 Weirs

8.3.2.1 Sharp-Crested Weirs

Sharp-crested weirs have a relatively thin crest such that water will tend to develop a nappe as it flows over the crest (Figure 8–5).

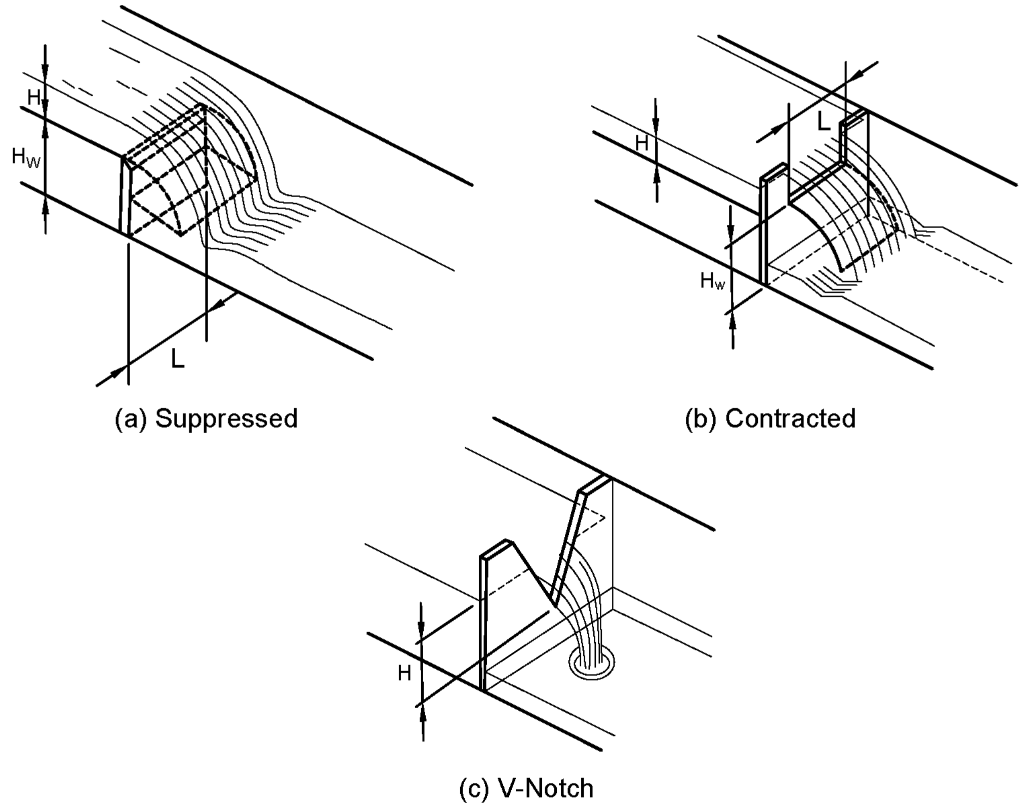


Figure 8-5. Sharp-Crested Weir Configurations

The capacity of a sharp-crested weir depends on the influence of end contractions. For a sharp-crested weir with no contractions, flow is calculated with Equation 8–5.

Equation 8-5. Sharp-Crested Weir Calculation

$$Q=C_{SCW}LH^{3/2}$$

where:

Q	=	flow over weir crest (ft^3/s)
C_{SCW}	=	sharp-crested weir coefficient
L	=	length of weir crest (ft)
H	=	head above of weir crest, excluding velocity head (ft)
H_W	=	height of the weir crest (ft)

The weir coefficient C_{SCW} varies with the ratio of hydraulic head above the weir and the height of the weir (H/H_W). For U.S. traditional units, the weir coefficient can be calculated using Equation 8–6.

Equation 8-6. Weir Coefficient Calculation—Traditional Units

$$C_{SCW}=3.27+0.4\frac{H}{H_W}$$

A sharp-crested weir with two end contractions can be analyzed using Equation 8-7.

Equation 8-7. Weir with Two-End Contractions Coefficient Calculation

$$Q=C_{SCW}(L-0.2H)H^{3/2}$$

When the tailwater behind a sharp-crested weir rises above the weir crest elevation, the submerged condition will reduce the discharge over the weir. The equation for a submerged sharp-crested weir is presented in Equation 8-8.

Equation 8-8. Submerged Sharp-Crested Weir Coefficient Calculation

$$\frac{Q_S}{Q}=\left[1-\left(\frac{H_2}{H_1}\right)^{3/2}\right]^{0.385}$$

where:

Q_S	=	flow over submerged sharp-crested weir (ft³/s)
Q	=	flow over sharp-crested weir under un-submerged conditions with same upstream headwater (ft³/s)
H_1, H_2	=	head above of weir crest upstream of crest and downstream of crest, respectively, excluding velocity head (ft)

8.3.2.2 V-Notch Weirs

V-notch weirs are a particular type of sharp-crested weir with a triangular cross-section. The discharge through a v-notch weir can be calculated from Equation 8-9.

Equation 8-9. V-Notch Weir Calculation

$$Q=2.5\tan\left(\frac{\theta}{2}\right)H^{5/2}$$

where:

Q	=	flow over weir crest (ft³/s)
θ	=	angle of v-notch (degrees)
L	=	length of weir crest (ft)
H	=	depth of water above apex of v-notch (ft)

8.3.2.3 Broad-Crested Weirs

In most cases, spillways traversing the top of an embankment are best modeled as broad-crested weirs. The equation for evaluating flow over a broad-crested weir is found in Equation 8-10.

Equation 8-10. Broad-Crested Weir Calculation

$$Q=C_{BCW}LH^{3/2}$$

where:

Q	=	discharge over the weir (ft^3/s)
C_{BCW}	=	broad-crested weir discharge coefficient
L	=	length of weir crest (ft)
H	=	head above of weir crest, excluding velocity head (ft)

The water surface elevation is measured at least 2.5 times the head above the weir elevation (2.5H) upstream of the weir crest when determining the broad-crested weir coefficients. Table 8–1 provides broad-crested weir coefficients based on the effective head over the weir and the breadth of the weir.

Table 8-1. Broad-Crested Weir Coefficient (C_{BCW}) as Function of Effective Head over Weir and Breadth of Weir

Measured Head (H)(1)	Weir Crest Breadth, b (ft)										
(ft)	0.50	0.75	1.0	1.5	2.0	2.5	3.0	4.0	5.0	10.0	15.0
0.2	2.80	2.75	2.69	2.62	2.54	2.48	2.44	2.38	2.34	2.49	2.68
0.4	2.92	2.80	2.72	2.64	2.61	2.60	2.58	2.54	2.50	2.56	2.70
0.6	3.08	2.89	2.75	2.64	2.61	2.60	2.68	2.69	2.70	2.70	2.70
0.8	3.30	3.04	2.85	2.68	2.60	2.60	2.67	2.68	2.68	2.69	2.64
1.0	3.32	3.14	2.98	2.75	2.66	2.64	2.65	2.67	2.68	2.68	2.63
1.2	3.32	3.20	3.08	2.86	2.70	2.65	2.64	2.67	2.66	2.69	2.64
1.4	3.32	3.26	3.20	2.92	2.77	2.68	2.64	2.65	2.65	2.67	2.64
1.6	3.32	3.29	3.28	3.07	2.89	2.75	2.68	2.66	2.65	2.64	2.63
1.8	3.32	3.32	3.31	3.07	2.88	2.74	2.68	2.66	2.65	2.64	2.63
2.0	3.32	3.31	3.30	3.03	2.85	2.76	2.27	2.68	2.65	2.64	2.63
2.5	3.32	3.32	3.31	3.28	3.07	2.89	2.81	2.72	2.67	2.64	2.63
3.0	3.32	3.32	3.32	3.32	3.20	3.05	2.92	2.73	2.66	2.64	2.63
3.5	3.32	3.32	3.32	3.32	3.32	3.19	2.97	2.76	2.68	2.64	2.63
4.0	3.32	3.32	3.32	3.32	3.32	3.32	3.07	2.79	2.70	2.64	2.63
4.5	3.32	3.32	3.32	3.32	3.32	3.32	3.32	2.88	2.74	2.64	2.63
5.0	3.32	3.32	3.32	3.32	3.32	3.32	3.32	3.07	2.79	2.64	2.63
5.5	3.32	3.32	3.32	3.32	3.32	3.32	3.32	3.32	2.88	2.64	2.63

(1) H measured at least 2.5H upstream of weir.

A typical roadway crossing can be modeled as a broad-crested weir with a weir coefficient C_{BCW}=2.6; for other applications, a broad-crested weir coefficient of C_{BCW}=3.0 is usually appropriate.

When using the broad-crested weir model to evaluate the capacity of a spillway with a rectangular or trapezoidal cross-section, the length of the weir crest is set as the base width of the spillway channel (b). The velocity of flow over the crest spillway is calculated as it passes through critical depth at the control section using Equation 8-11.

Equation 8-11. Velocity of Flow Calculation - Crest Spillway

$$v_c = 3.18\left(\frac{Q}{b}\right)^{1/3}$$

where:

v_c	=	critical velocity (ft/s)
Q	=	discharge over the weir (ft^3/s)
b	=	length of weir crest (ft)

8.3.3 Orifices

A vertical orifice is a circular or rectangular opening, often located in a headwall or the sidewall of a riser structure. Figure 8–6 illustrates typical orifice configurations. The discharge depends on the submergence of the orifice, effective height of the water above the center of the opening, and the size, shape, and edge treatment of the orifice.

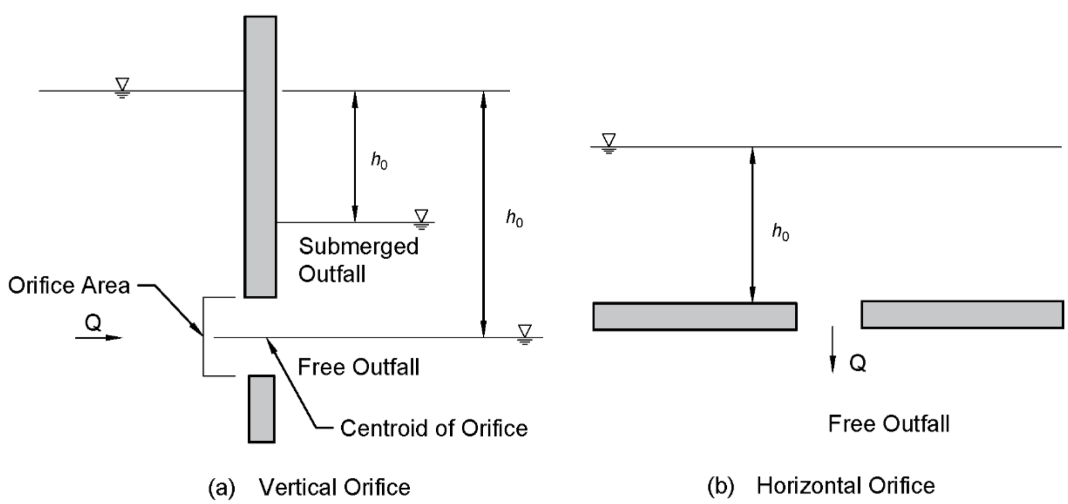


Figure 8-6. Typical Orifice Configurations

For a single submerged orifice, the discharge can be determined using the standard orifice equation (Equation 8-12).

Equation 8-12: Single Submerged Orifice Calculation

$$Q = C_O A_O \sqrt{2g(H_o)}$$

where:

Q	=	orifice flow discharge (ft^3/s)
C_O	=	orifice discharge coefficient
A_O	=	cross-sectional area of flow through the orifice (ft^2)
g	=	gravitational acceleration (32.2 ft/s^2)
H_O	=	effective head above orifice (ft)

When the orifice is unsubmerged, the effective head H_O is measured from the centerline of the orifice to the upstream water surface elevation. For submerged orifices, the effective head is the difference in elevation of the upstream and downstream water surfaces.

When the orifice has sharp, clean edges (e.g., the material is thinner than the orifice diameter), an orifice discharge coefficient (C_O) of 0.6 is appropriate. For sharp, ragged edged orifices, such as those produced by cutting openings in corrugated pipe with an acetylene torch, a value of C_O=0.40 should be used. The orifice coefficient should also be adjusted when the diameter of the orifice approaches the thickness of the orifice plate. Table 8–2 summarizes orifice discharge coefficient for different edge conditions. Pipes smaller than 1 foot in diameter may be analyzed as submerged orifices, as long as there is adequate headwater (H_O/D>1.5). Pipes larger than 1 foot in diameter are more appropriately analyzed as culverts (see Chapter 5). Flow through multiple orifices may be computed by summing the flow through the individual orifices.

Table 8-2. Orifice Coefficient for Different Edge Conditions

Edge Conditions	Orifice Coefficient, C_O
Sharp, Clean Edge (t<d)	0.60
Sharp, Ragged Edge (t<d)	0.40
Thick, Squared Edge (t>d)	0.80
Thick, Rounded Edge (t>d)	0.92

Note: t is the thickness of orifice plate; d is the diameter of orifice

8.3.4 Riser Structures

Riser structure is a general term for structures having inlet openings that are parallel to the water surface in the detention facility. Riser structures with circular cross-section are often called standpipes, and rectangular riser structures are often called inlet boxes. Figure 8–7 illustrates the hydraulic behavior of a typical riser structure.

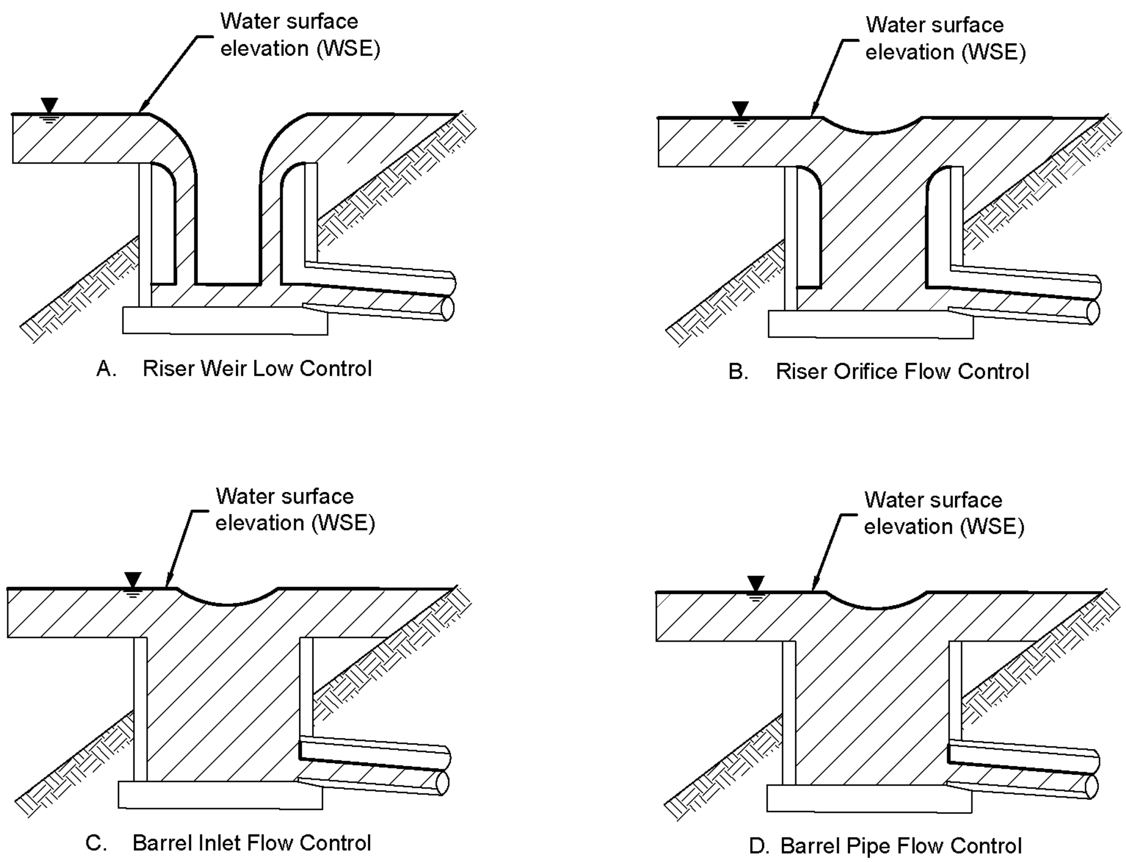


Figure 8-7. Hydraulic Control through a Typical Riser Structure

The hydraulic behavior of flow through a riser structure changes and must be analyzed differently depending on the stage in the basin. Flow through a riser structure generally proceeds through four phases: (1) riser weir flow control, (2) riser orifice flow control, (3) barrel inlet flow control, and (4) barrel pipe flow control.

The USBR Design of Small Dams (1987) discusses the hydraulics of riser structures in more detail, and includes design nomographs (Figure 8–8) that may be used in the design of riser structures.

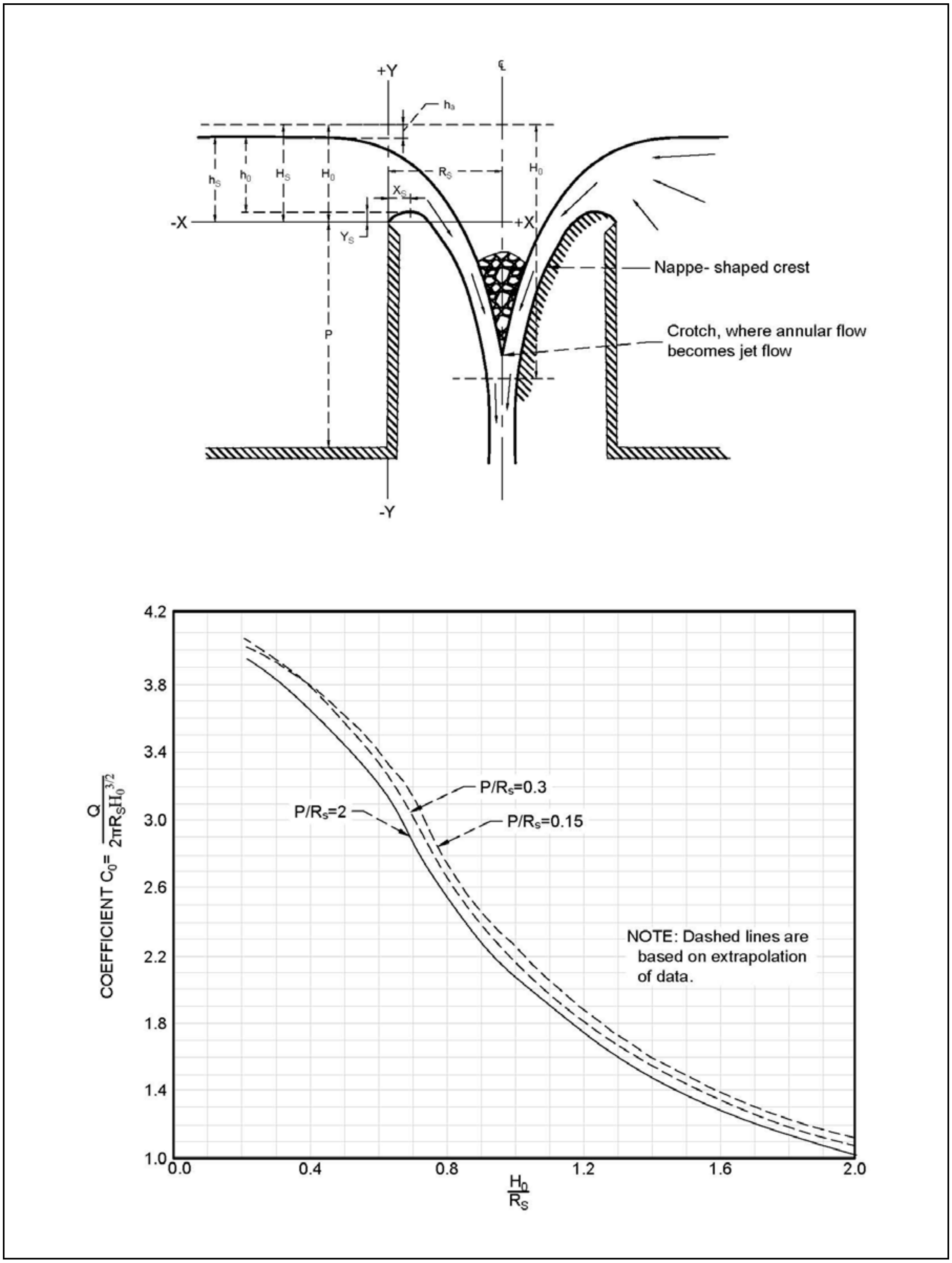


Figure 8-8. Riser Structure Design Nomograph

When the water surface reaches the top edge of the riser, flow will typically begin to pass through the structure in the manner of a sharp-crested weir (for sharp-crested weir equation and coefficients, see Section 8.3.2.1), with a crest length equivalent to the perimeter of the riser structure.

As the depth of water increases and submerges the top of the riser, the flow will transition to an orifice-type flow. This horizontal orifice flow depends upon the area of the top of the riser structure, and can be computed using Equation 8-13.

Equation 8-13. Horizontal Orifice Flow Calculation

$$Q=C_{HO}A_O\sqrt{2g(h-h_R)}$$

where:

Q	=	flow through the orifice (ft^3/s)
C_{HO}	=	horizontal orifice coefficient
A_O	=	area of the orifice (ft^2)
g	=	gravitational acceleration (32.2 ft/s^2)
h	=	elevation of water above the orifice (ft)
h_R	=	elevation of the crest of the riser orifice (ft)

The transition zone between weir and orifice flow for riser structures is not well defined. Though the transition from weir flow to orifice flow is gradual, it is commonly assumed to occur at a discrete water surface elevation (h_T) to simplify the analysis. The transition water surface elevation ($h=h_T$) (see Equation 8-14) is found by calculating the point at which the weir equation and orifice equation yield the same discharge.

Equation 8-14. Transition Water Surface Elevation Calculation

$$h_T=h_C+\frac{C_{HO}A_O}{C_{SCW}L}$$

Thus, the weir equation is used for calculating flow through a riser structure for water surface elevations $h \leq h_T$ and the orifice equation for water surface elevations $h>h_T$.

As the water surface elevation rises further, the control can change to barrel inlet flow control and/or barrel pipe flow control. General best practice is to ensure that the outlet barrel has greater capacity than the riser structure under design conditions.

8.3.5 Perforated Risers

Perforated risers are a special case of orifice flow that can be used to obtain extended detention times. As such, they are often useful in water-quality treatment applications. Holes are normally spaced a minimum of three to four orifice diameters (center to center) apart, limiting the number of holes such that they do not compromise the overall integrity of the riser.

Assuming the riser is constructed of a relatively thin material, the perforations will operate as orifices. Therefore, the discharge through the orifices on the perforated risers is equivalent to the summation of the flow through individual orifices in the riser. The design engineer shall use care when specifying perforated risers, since they are often subject to clogging, and measures to reduce such clogging such as gravel jackets and/or wire mesh also have implications for the maintenance of the perforated riser.

8.3.6 Combination Outlets

Combinations of culverts, weirs, orifices, and riser structures can provide multiple-stage outlet control for different control volumes and storm frequencies. These combination outlets may have independent outlet controls, but often outlet structures will share a common outlet. Combination outlets require composite stage-discharge curves based on the hydraulic performance curves of the component outlet structures. The total discharge from the outlets will generally be the summation of its individual outlets, constrained by the capacity of common outlet conduits and possibly tailwater conditions. Figure 8–4 illustrates a typical stage-discharge curve for detention facility outlet works.

8.3.7 Trash Racks and Debris Control

Trash racks and debris control structures can be crucial to the successful operation of a detention facility, especially if outlet works. Head losses from trash racks can generally be neglected when the clear-opening area of the trash rack is at least four times the clear-opening area of the inlet. Chapter 9 of this Manual provides additional guidance for trash racks and debris control.

8.3.8 Buoyancy

Buoyant forces create uplift forces that can damage detention basin riser structures. Outlet structures shall be anchored properly such that they will withstand buoyant forces. The design engineer shall consider resistance to buoyant forces both for the anchoring of the system as a whole and for major connecting components (e.g., band couplings) of the outlet structure. The design condition shall assume maximum design water surface elevation in the basin with no water inside the outlet structure.

Chapter 9

Energy Dissipation

The development of a site often results in modifications to drainage characteristics, including increases in peak discharges and velocities of runoff from the property. Storm drain discharges into unlined channels or natural watercourses have the potential to cause erosion. This Chapter focuses on riprap aprons, which are the most common type of energy dissipator, but also summarizes several other methods for energy dissipation. The designer is encouraged to investigate reference sources for further discussion of energy dissipators.

9.1. General Design Criteria

Energy dissipation is required when a project increases the exit velocity and turbulence at a conduit outlet above the existing (pre-project) condition. However, if the velocity does not have the potential to be erosive, energy dissipation is not required. Energy dissipation may also be required when a project proposes to concentrate surface runoff into discrete discharge points (e.g., concentrating sheet flow and discharging into a stream via a down-drain, etc.). Energy dissipation shall reduce the velocity to non-erosive levels where practical, and shall be designed to handle the same design event as the storm drain, culvert, or other facility immediately upstream.

9.2. Hydraulic Design

Selection of the most appropriate energy dissipator for use on a project site requires the consideration of a number of factors. In addition to the hydraulic parameters, the designer needs to consider safety factors, the potential for vandalism, economic feasibility, and other factors. For instance, the design engineer might specify a larger stone size or grouted riprap to prevent stones from being carried away by vandals. When energy dissipators are used on a project, the designer shall submit design specifics with their drainage report.

Many structural devices can be used to provide energy dissipation. This Manual discusses some of the more common types that are applicable to a wide range of situations. The designer shall carefully consider the project requirements prior to using any of the following devices. If the project requires an energy dissipator that is not listed by this Manual, the designer shall contact the City prior to proceeding with design. This will enable the City and designer to coordinate design, submittal, inspection, and maintenance requirements.

9.3. Riprap Aprons

Riprap apron energy dissipation shall conform to San Diego regional design standards (Standard Drawing No. SDD-104).

9.3.1 Concrete Energy Dissipator (Impact Basin) (SDRSD No. SDD-105)

Design standards for the impact basin depicted on San Diego Regional Standard Drawing No. SDD-105 are based on the USBR Type VI Basin. The original USBR basin has been modified to allow drainage of the basin during dry periods, which enhances the usefulness of the basin in urban environments. The width of SDD-105 is based on discharge from the storm drain or culvert; this width must be specified on drawings.

Figure 9–1 (FHWA HEC-14, 1983) provides a nomograph that may be used to estimate the energy loss through an impact basin.

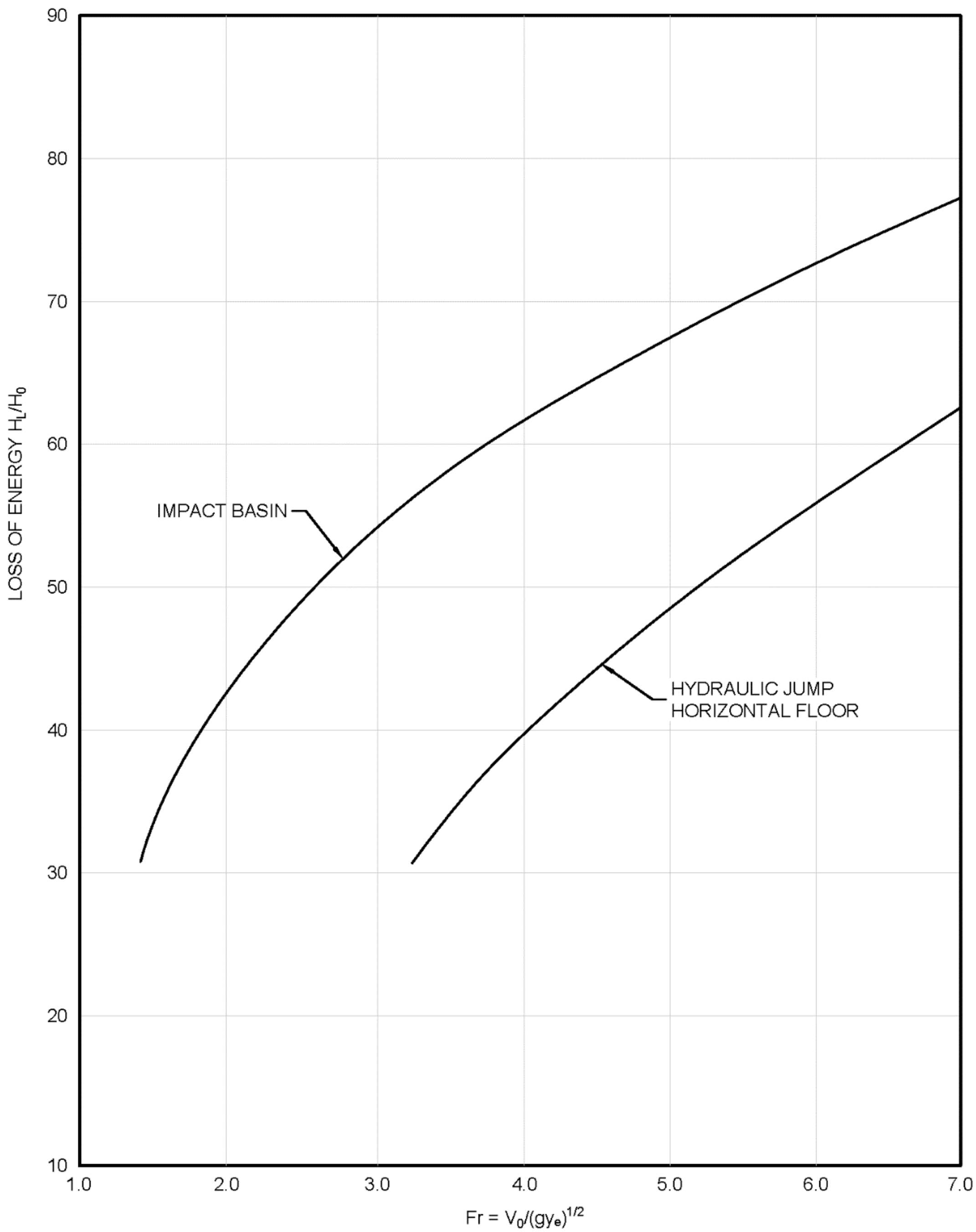


Figure 9-1. Impact Basin Energy Loss Nomograph

This energy loss can then be used to estimate the flow velocity exiting the impact basin. The energy loss through the impact basin is a function of the Froude Number of the flow entering the impact basin, calculated in this using Equation 9–1.

Equation 9-1. Energy Loss through Impact Basin

Calculation 1

$$FR = v_o / \sqrt{g y_e}$$

Calculation 2

$$y_e = \sqrt{A/2}$$

Calculation 3

$$H_O = y_e + \frac{v_o^2}{2g}$$

Calculation 4

$$H_B = H_O (1 - H_L / H_O)$$

where:

FR	=	Froude Number of flow entering the impact basin
v_O	=	velocity of flow entering the dissipator (ft/s)
g	=	gravitational acceleration (32.2 ft/s^2)
y_e	=	equivalent depth of flow entering the dissipator (ft)
A	=	area of flow entering the dissipator (ft^2)
H_O	=	kinetic energy of flow entering the dissipator (ft)
H_B	=	kinetic energy of flow leaving the dissipator (ft)
H_L	=	kinetic energy loss in the dissipator (ft)
H_L/H_O	=	Loss of energy from Figure 9-1. Convert percent from y-axis to fraction for use in Calculation 4

Chapter 10

Debris Barriers and Basins

The use of trash racks and other barriers to protect culverts and underground pipes should be carefully considered on a case-by-case basis. Properly used debris barriers/basins prevent clogging and associated flooding; improperly used, they may curtail the conveyance capacity of a culvert or channel and increase flood risk.

10.1. General Design Criteria

The purpose of debris basins and barriers is to reduce the potential for debris clogging channels, pipes, and culverts. When flows are likely to convey rock or other debris in sufficient size and volume to block or obstruct a culvert, some form of debris barrier or basin shall be used where necessary.

Debris basins and other debris facilities require on-going maintenance to assure proper function. All debris facilities shall have an operation and maintenance plan that specifies regular inspection and maintenance at specific time intervals and/or maintenance "indicators" when maintenance activity will be triggered. Operation and maintenance plans shall ensure that vegetation and debris are removed or maintained on a regular basis to maintain the function of the facility.

The project owner and design engineer shall consult with the City for determination of the appropriate maintenance mechanism required for a particular project. At a minimum, privately owned and maintained debris facilities shall have a recorded easement agreement with a covenant binding on successors or other mechanism acceptable to the City. Typically, the easement will cover the debris basin or an area around the debris barrier sufficient to provide adequate access and maintenance.

10.2. Hydraulic Design of Debris Basins and Barriers

Several types of structures might be used to reduce the effect of debris on culverts and pipes. The first step in selecting the appropriate debris control device is to categorize the type of debris inflow. The U.S. Department of Transportation (USDOT, 1971) has developed the following classification of debris:

1. **Light floating debris:** small limbs or sticks, orchard prunings, tules, and refuse
2. **Medium floating debris:** limbs or large sticks
3. **Large floating debris:** logs or trees
4. **Flowing debris:** heterogeneous fluid mass or clay, silt, sand, gravel, rock, refuse, or sticks

5. **Fine detritus:** fairly uniform bedload of silt, sand, gravel more or less devoid of floating debris, tending to deposit upon diminution of velocity
6. **Coarse detritus:** coarse gravel or rock fragments carried as channel bedload at flood stage

10.2.1 Debris Racks

Debris racks provide a physical barrier across the upstream face of channels or culverts. Debris racks vary greatly in size and materials. Many factors influence the design of debris racks, including: (1) the size and type of debris, (2) the size of culvert or structure being protected, (3) the amount of flow, and (4) flooding issues.

Debris racks shall typically have an open area equivalent to approximately four times the flow area of the conduit or channel they are protecting to maintain flow conveyance and reduce head loss. The height of the rack shall typically extend above the expected depth of flow under the design storm. The design engineer shall consider the use of sloped racks to reduce the risk of pinning debris where applicable. Debris racks shall be well-secured but removable for the purposes of maintenance. The California Standard Specifications for Public Works Construction or "Greenbook" Standard Plan No. 361-0 offers details and specifications for a typical trash rack. Maintenance access to the debris rack is critical and must be provided. Access can take the form of an easement, public trail, or road. Finally, the design engineer shall specify debris racks only when special circumstances warrant their use.

10.2.2 Debris Posts

A debris post is a structural system of posts placed upstream of a culvert entrance causing debris to deposit before entering the culvert. The design engineer shall specify debris posts only when special circumstances warrant their use.

The posts shall be a minimum of 4 inches in diameter and are usually constructed of metal embedded in a concrete base. Posts are typically spaced at 1/3 of the culvert diameter to a maximum of 24 inches, and are placed upstream of the culvert entrance a distance of twice the culvert diameter where practicable.

The posts shall be designed assuming the fence to be 100 percent effective in blocking the flow; the fence will therefore act as a submerged sharp-crested weir with a height equivalent to the height of the debris posts. The design engineer must check that the water elevation spilling over the top of the weir will continue to flow towards the culvert and not flood the surrounding area.

Debris posts shall be embedded to a depth adequate to resist hydrostatic pressures and help prevent failures due to scour. San Diego Regional Standard Drawing No. D-82 provides a standard detail for debris fence with an embedment depth that is appropriate for most typical applications. The Los Angeles Department of Public Works (1979) provides an equation for embedment depth that may be used when necessary due to special conditions such as unusually high-anticipated debris loads. The design engineer shall compare the embedment depth to the potential local scour depth near the debris posts and culvert entrance, and specify an appropriate embedment depth based on these values.

Chapter 11

Shallow Sub-Surface Groundwater Drainage

Shallow subsurface groundwater is underground water that is close enough to the surface so that its movement may affect pavement and structures. It is considered detrimental if it increases the risk of slope failures, negatively affects the support of pavement or structures, or results in nuisance flow associated with seepage. This chapter provides design criteria for both mitigating existing problems and for prevention of new problems triggered by land alteration.

11.1. Policy and Regulatory Considerations

If improvements that mitigate existing seepage and shallow subsurface groundwater problems will have a benefit to both the City and private property owner, then eligibility for cooperative financing according to Council Policy 800-4 may be considered.

If improvements direct seepage or shallow subsurface groundwater to the City's storm water conveyance system or a protected receiving water body, compliance with permits issued under the National Pollutant Discharge Elimination Systems (NPDES) Program must be verified prior to approval of the improvements.

11.2. Investigation Requirements

Preliminary geotechnical investigation reports (i.e., soils reports, geological reports, etc.) submitted for grading and/or building permits shall address groundwater and the potential for detrimental effects related to subsurface water or daylight water seepage. Preliminary geotechnical investigation reports shall contain:

1. Identification of streams, ponds, springs, or seeps and their relationship to site topography and geology;
2. Identification of detention/retention basins and storm water BMPs on the site;
3. Sufficient subsurface exploration to evaluate and adequately characterize groundwater conditions;
4. Identification of seasonal high groundwater levels (i.e., "water table");
5. Description of onsite soil types and permeability characteristic of the onsite earth materials;
6. Potential for differential permeability pathways (i.e., preferential permeability pathways) for subsurface water flow; and
7. Recommendations to mitigate potential detrimental effects related to subsurface water.

All final as-graded or as-built geotechnical reports shall describe the geologic and groundwater conditions encountered during construction. The report shall identify any groundwater issues including seepage and describe the measures implemented to mitigate the detrimental effects related to subsurface water build up or daylight water seepage.

Refer to the City's "Guidelines for Geotechnical Reports" for additional information (http://www.sandiego.gov/development-services/pdf/industry/geoguidelines.pdf).

11.3. Design Criteria

Engineered subsurface drainage systems shall be included in grading plans when the geotechnical analysis determines that the potential exists for detrimental seepage or shallow subsurface groundwater flow (see Figures 11–1 and 11–2). The system may also be permitted to address an existing problem.

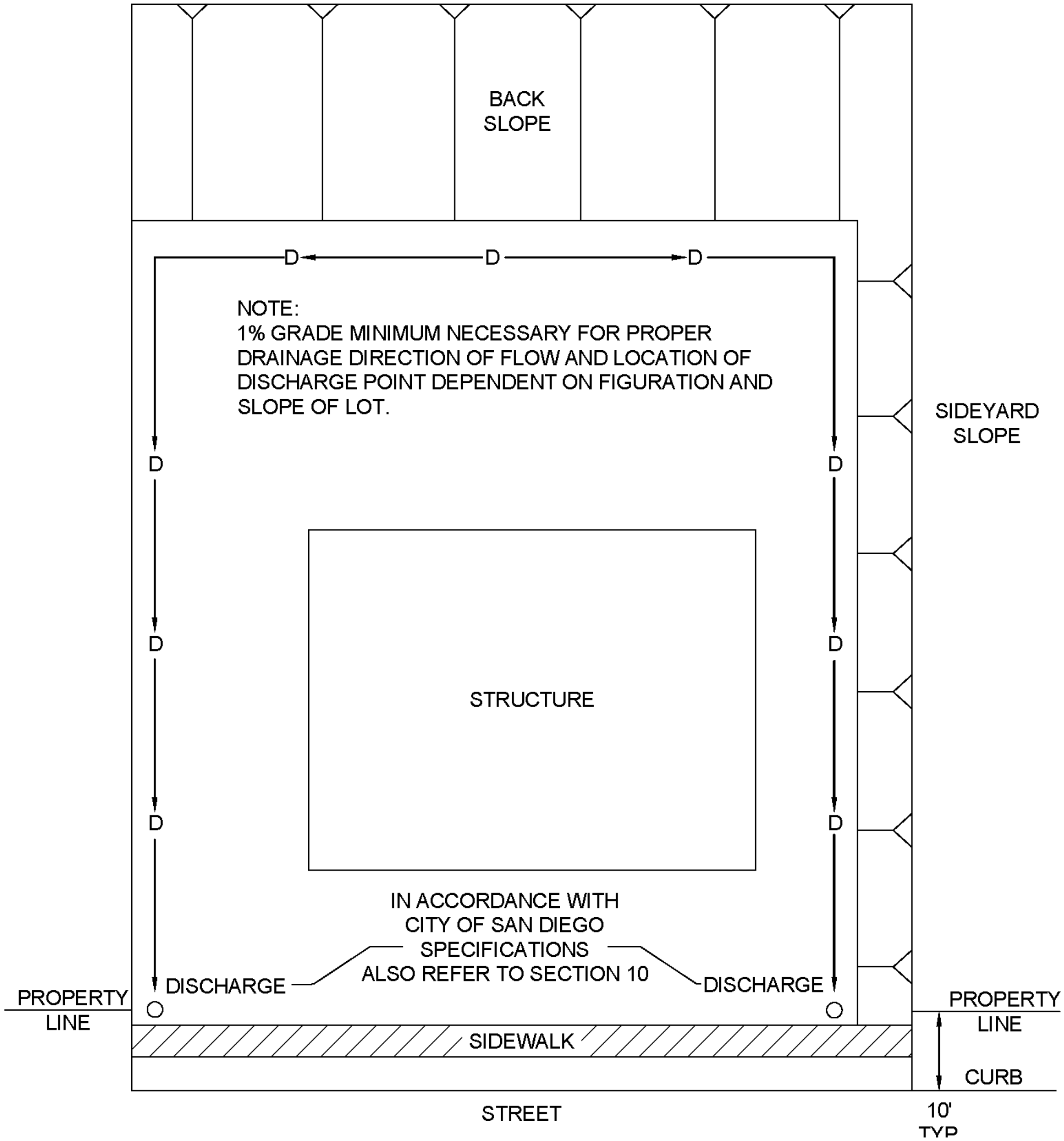
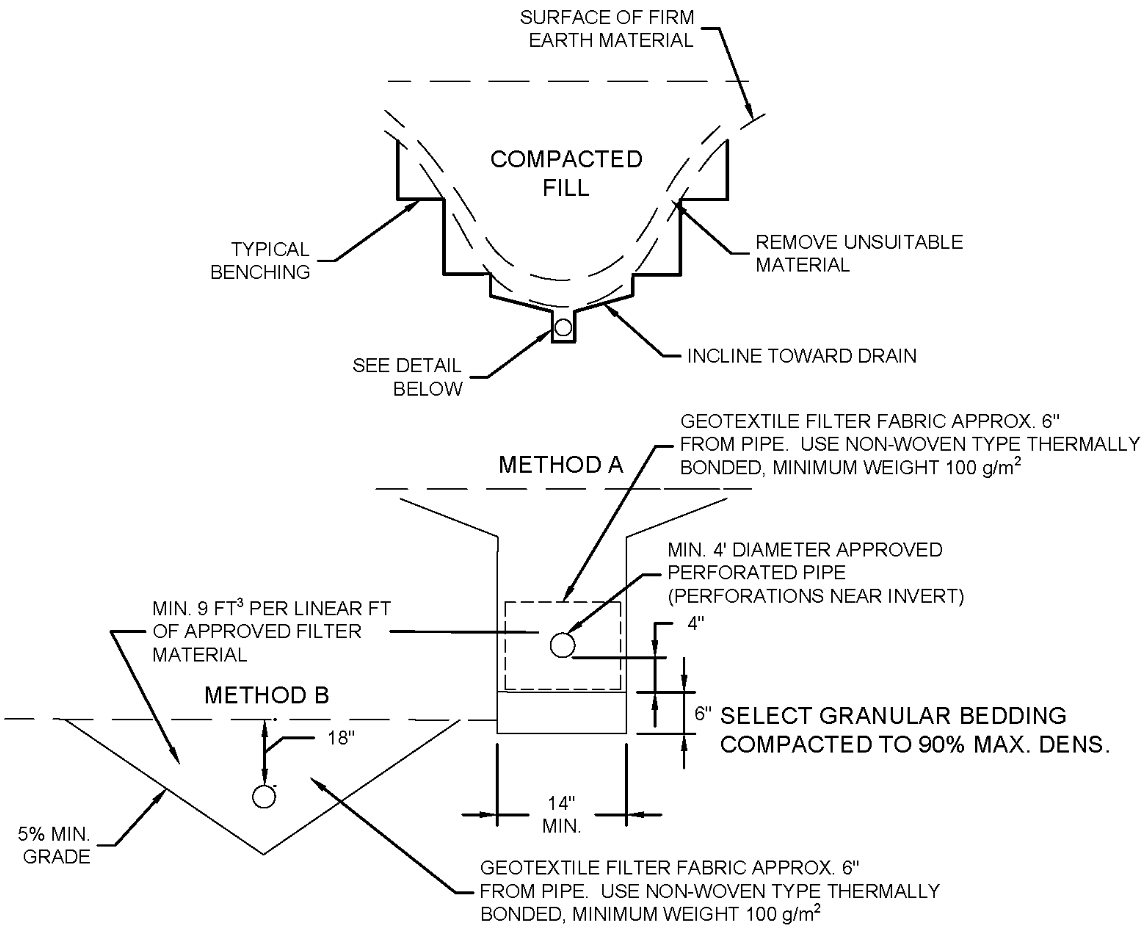


Figure 11-1. Plan View of Typical Subdrain Layout



DETAIL

FILTER MATERIAL TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUAL:

SIEVE SIZE	PERCENTAGE PASSING
1"	100
$\frac{3}{4}$"	90-100
$\frac{3}{8}$"	40-100
No. 4	25-40
No. 8	18-33
No. 30	5-15
No. 50	0-7
No. 200	0-3

Figure 11-2. Typical Subdrain Cross-Section Details

11.3.1 All Conveyance Systems

The following criteria shall be met for all conveyance systems:

1. The minimum size for subsurface drains shall be four-inch (4”) diameter pipe unless otherwise specified by the project’s Geotechnical Consultant.
2. Surface drainage will not be permitted to discharge into a subsurface drain.
3. Location and spacing of cleanouts shall be as specified by the project’s Geotechnical Consultant or as required by the City Engineer.
4. The minimum allowable grade shall be 0.5 percent. If conditions require flatter grades, approval of the plan by the City Engineer is required.
5. Subdrain discharge points must be clearly identified on the grading plans. If subdrains outlet offsite, a letter of permission and easement for maintenance from the adjacent property owner must be secured by the permittee as required by the City Engineer.
6. A detail for a subdrain pipe headwall must be shown on the grading plans if subdrains do not connect directly to an existing subdrain or storm drain structure.
7. Subdrains (existing or proposed) must be shown on the improvement plans and profiles.
8. The precise location of subdrains shall be accurately located and shown on the Final As-Built Grading and Improvement Plans.

11.3.2 Conveyance Systems in the Right-of-Way and on City-Owned Parcels

If a subsurface drainage system is needed within the public right-of-way or on a City-owned parcel, or if a private system connects to a City storm water conveyance system, the following additional criteria apply:

1. The discharge from a subsurface drain to a City storm water conveyance system will be permitted only if the subsurface drain conveys gravity flow.
2. Subsurface drains are not permitted to discharge to the gutter.
3. The minimum pipe size of a seepage collector line shall be eight inches (8”).
4. Subsurface drainage that is pumped to a City storm water conveyance system may require a NPDES permit from the Regional Water Quality Control Board, San Diego Region.

If the nearest City storm drain leads to an existing low-flow diversion structure, the City may, at the City’s discretion, allow a subsurface drain to be connected to the sanitary sewer instead of the underground storm drain provided appropriate measures are taken to measure flow and to prevent excesses inflow to the sewer during rain events.

THIS PAGE INTENTIONALLY LEFT BLANK FOR DOUBLE-SIDED PRINTING

Chapter 12

Green Infrastructure

12.1. Introduction

Green Infrastructure (GI) refers to storm water management systems that mimic nature by soaking up and storing water for water quality treatment and flow control purposes. GI has previously been referred to as Low Impact Development (LID) or Permanent Storm Water Best Management Practices (BMPs).

GI design is used to capture and treat pollutants in storm water runoff before those pollutants reach our waterways downstream. GI is built to meet the requirements in the San Diego Municipal Storm Water Permit (Permit), the City's Storm Water Standards (SWS), and to meet other special regulations such as various Total Maximum Daily Load (TMDL) reductions under the Clean Water Act and Areas of Special Biological Significance (ASBS) regulations under the California Oceans Plan.

Designers are referred to the SWS and, in particular, the Storm Water Applicability Checklist to determine whether GI is required for each project. Projects deemed to be Standard Development Projects or Exempt Projects per the Permit may still be required to build GI to meet the City's more stringent standards. All Priority Development Projects are required to build GI to capture, retain, or treat the runoff volume from an 85th percentile storm, or the flow-based equivalent as specified by the Permit, and may be required to build additional GI to meet the City's standards. Some Priority Development Projects are also subject to the hydromodification requirements per the Permit.

12.2. General Design Criteria

All projects required to build GI that will be owned and maintained by the City must meet the design criteria presented in this chapter.

Volume-based GI shall use the Water Quality Volume equation and flow-based GI shall use the Water Quality Flow equation presented in Equation 12-1.

Equation 12-1: Water Quality Volume and Water Quality Flow

$$\text{Water Quality Volume (WQV)} = CPA$$
and
$$\text{Water Quality Flow (WQF)} = CIA/43{,}200$$

where:

WQV	=	Water Quality Volume expressed in ft^3
WQF	=	Water Quality Flow expressed in cfs
C	=	runoff coefficient (use Table A-1 from Appendix A)
P	=	the rainfall depth (in feet) for the required storm size
I	=	the rainfall intensity (in in/hr) assumed to be 0.2 in/hr
A	=	watershed area (in ft^2) draining to the GI

The dimensions of the GI are to be minimized while still meeting the sizing requirements. Dimensions may be rounded up to make a more pleasing shape on the surface or underground for aesthetics or maintenance reasons. Within the same Priority Development Project, GI in one drainage area may be used to make up for GI requirements in other drainage areas that cannot meet the sizing criteria due to site constraints. Otherwise, GI is not to be oversized.

Plans for GI projects shall provide the following information in a chart for each drainage area:

1. Calculated Q (discharge) or V (volume) of the runoff for the design storm, depending on whether the GI is flow-based or volume-based; and
2. Q or V that the GI design can handle; and
3. Q_{50} or Q_{100} for the drainage area of the GI, depending on which is required for the acreage; and
4. The flow rates for the upper and lower ends of the geomorphically significant flow range plus the required sizes of orifices and other flow controls (unless the GI is exempt from hydromodification requirements); and
5. Q_{bypass} of the GI.

The following information shall also be provided on the plans:

1. Dimensions of the GI which are sufficient to verify that the sizing requirement has been met using post-construction measurements; and
2. Maximum designed flow and water surface elevation at maximum design flow condition; and
3. Slope measurements for all sides of the GI, expressed as a ratio such as 5H:1V.

As a check, Q_{bypass} of the GI shall convey flows equal to or larger than Q_{50} or Q_{100} for the drainage area for the case that the GI ever becomes completely clogged.

Unless prohibited by geotechnical conditions, infiltration GI shall be used. This minimizes underground infrastructure and maintenance of that infrastructure.

Unless constrained by another project condition, GI should be located immediately upstream of a storm drain inlet or catch basin in order to capture as much runoff from the drainage area as possible.

12.3. Infiltration

Design calculations for infiltration GI must demonstrate that the ponded water will drain down completely within 48 hours. Additionally, the slope at the bottom of infiltration GI should be as close to 0% as possible to maintain the large contact area as the runoff infiltrates. The feasibility of infiltration GI should be considered based on site specific conditions determined through a preliminary geologic/geotechnical investigation prepared in accordance with the City's Guidelines for Geotechnical Reports. Appendix F of the guidelines provides information on site evaluations to determine feasibility of infiltration GI and some of the conditions that are not suitable for infiltration GI. In addition, the appendix provides a list of potential impacts of infiltration GI which must be considered.

Geologic/geotechnical site evaluation and testing are critical for proper design and function of infiltration GI. Where the site evaluation indicates potential feasibility of infiltration GI, field investigation will be necessary to support recommended design infiltration rates.

If lateral subsurface water flow from the infiltration GI could impact adjacent improvements or result in soil piping or in daylight seepage, cut off walls or partial liners (with or without an underdrain) could be considered (see Section 12.18).

12.4. Filtration

Underdrains are used in filtration GI to empty the captured storm water runoff. Underdrains are to be a minimum of 8" in diameter and must be capable of handling the flow through the GI. Minimum slope is 0.5%.

Minimum perforation pattern is to be two rows of holes, 90 degrees apart. The size and distance of perforations shall be sufficient for a calculated drawdown time of 48 hour or less using a factor of safety of 4 to account for blockage by the drainage rock. Underdrains are to be placed with holes down, each row of holes 45 degrees off center.

Type A cleanouts are to be used at every vertical and horizontal bend as access points with one exception. Access points for 8" underdrains may use a modified sewer lateral cleanout and are to be spaced no farther than 200 ft apart. No more than two of these cleanouts shall be used in a row; the third cleanout in the line must be a Type A cleanout.

Once underdrains pass the most downstream GI, the underdrain becomes a standard storm drain and must comply with the standards in Chapter 4.

Filtration GI must have an impermeable liner between the GI and the native soil when geotechnical analyses determine that infiltration into the surrounding soil will create an undesirable condition.

Consideration could be given to placing a geotextile fabric between grade and an impermeable liner to protect and cushion the impermeable liner from puncture.

12.5. Porous pavement

Porous pavement is any pavement that allows runoff to pass through the pavement for the purposes of storm water capture and treatment. Runoff may pass through the pavement itself or it may pass through the spaces between sections of the pavement.

There are three major categories of porous pavement: porous asphalt, porous concrete, and porous pavers.

12.5.1 Color

Porous concrete shall be a natural concrete color (gray). Porous pavers on City parcels shall be a color that coordinates with the color of the building or other pavement. Porous pavement in the right-of-way shall be a color approved by the asset owner.

12.5.2 Location

Porous pavement shall not be located where driving speed is expected to exceed 15 miles per hour. Additionally, it shall not be located where it will be exposed to large amounts of sediment from pedestrian or vehicle tracking or from sediment laden flows. Examples of prohibited areas include areas adjacent to sand beaches and areas downstream of erosive slopes.

Porous pavement in alleys shall be located at the low point of the cross-section (in a typical alley section, this would be the centerline) and shall be no more than five feet (5') wide.

To prevent runoff from bypassing the GI in locations other than alleys, the downstream portion of the porous pavement shall be directly adjacent to a:

1. Standard curb; or
2. Landscaped area that is part of the GI; or
3. Porous gutter; or
4. Non-porous gutter that leads to a bioretention area before it leads to the storm water conveyance system.

The location of porous pavement is partially determined by the method of maintenance. Porous concrete must be maintained by a street sweeper. Porous pavers can be maintained by street sweepers or by power washing, depending on the product.

12.5.2.1 Maintenance by a Street Sweeper

Porous pavement needing maintenance by a street sweeper shall not be used in any area that cannot be reached by a standard street sweeper, which may include sidewalks and medians. When porous pavement is used on a City parcel, the location must be accessible for a street sweeper even if the area is off-limits to other vehicles. Bollards, gates, and other barriers used to prevent other vehicle access must be removable for sweeper access.

Street sweeper dimensions will impact where porous pavement may be located. Street sweepers are 12 ft wide and have a turning radius of 22 ft.

Porous pavement needing a street sweeper shall meet the following criteria:

1. Driving surfaces shall be a minimum of 14 ft wide; and
2. The distance between the driving surface and any overhanging structures shall be a minimum of 12 ft.; and
3. A minimum of 12 inches of clear space (no walls, buildings, or other structures) shall be given on either side of the driving surface to allow the sweeper space to maneuver; and
4. A turnaround area with a minimum of a 22 ft radius shall be provided unless the sweeping area is linear. In the case of a linear path, access must be provided at both ends of the path.

The following is a list of common objects that impede sweeper access. Check plans to ensure minimum distances around these objects are maintained: wheel stops, signs, traffic delineators, curbs, and bollards.

12.5.2.2 Maintenance by Power Washing

Porous pavement maintained by power washing shall be located so that all water from the power washer can be collected and retained or used on site and before reaching a storm drain inlet or catch basin. Water from the power washer that is captured by the GI is considered to be retained on site.

12.5.3 Curbs

A curb shall be installed around the perimeter of all porous pavement to differentiate between porous and non-porous surfaces except where the porous pavement is adjacent to a landscaped area. A zero height curb shall be used where vehicles are expected to drive over the curb; otherwise, a 6” curb shall be used.

12.5.4 Storage

Storage beneath the porous pavement is required for volume-based GI and storage may be necessary for flow-based GI if the discharge through and out of the porous pavement is slower than the discharge in to the GI. The discharge through the porous pavement shall be equal to or faster than the discharge for the design storm for each drainage area.

Storage of the design storm above the surface (ponded water) is prohibited.

12.6. Landscaping

Existing trees in good condition shall be preserved whenever possible. So as not to disturb the root system, consult an arborist or landscape architect when proposing GI within the limits established by the future canopy of the mature tree. Each tree proposed in lined GI shall be provided a minimum of 120 cubic feet of soil.

All proposed landscaping shall follow the Landscaping Standards in the Land Development Code. Proposed landscaping shall be divided in to two categories: (1) that which is required for the function of the GI and (2) that which is aesthetic. In order to qualify for category 1, the landscaping must be included in the GI sizing calculations. Otherwise, the landscaping falls in to category 2. Only category 1 landscaping shall be shown on the civil engineering drawings. Category 2 landscaping shall be shown in the landscaping plans. The inclusion of category 2 landscaping is at the discretion of the asset owning Department, usually the Park & Recreation Department.

12.7. Irrigation

Installing new, permanent irrigation using non-potable water or using existing irrigation with potable or non-potable water is permitted. Otherwise, permanent irrigation shall not be permitted.

Temporary irrigation, whether by water truck or by above ground pipelines, is permitted until new landscaping is established

12.8. Soil Media

The depth of bioretention media to be used in GI varies based on the purpose. The depth of media used in filtration projects is based on the pollutant targeted. The minimum depth is 18". To specifically target phosphorous, media depth shall be no less than 24". To target nitrogen, media depth shall be no less than 30". There is no minimum depth for infiltration projects as long as the total volume of the media is large enough to hold the runoff from the prescribed storm. Landscaped GI requires soil media to be used. Shrubs and trees require soil media to be at least 36" deep.

12.9. Overflows

GI overflows must be capable of handling the 50-year or 100-year storm, whichever is required for the drainage area. Overflow routes may bypass the GI completely or may be located inside the GI to intercept high flows before passing through the media. When pipe is used to convey overflows away from GI, the pipe shall be considered a storm drain and must comply with the standards in Chapter 4.

12.10. Freeboard

Freeboard for GI is defined as the elevation difference between the top of the overflow and the top of the infiltration or filtration GI. Minimum freeboard shall be 2".

12.11. Conflicting Utilities

No utilities are allowed in the GI except for short runs of water services or sewer laterals that cross the GI. All water services and sewer laterals located inside the GI shall be encased in concrete that extends 3" beyond the outside wall of the pipe in all directions.

12.12. Directing Gutter Flow to GI

When GI is in the parkway, curb cuts shall be used to direct flow from the gutter in to the GI. Curb cuts shall be the full height of the curb. When gutter flow must pass under a sidewalk to reach the GI, a curb-face inlet shall be used.

There shall be sufficient elevation difference between the flow line at the curb cut or curb-face inlet and the high water line in the GI to prevent any backwater influence from reducing the discharge through the curb cut or curb-face inlet.

12.13. Underground Storage

A vault, rock media, or soil media may be used to hold runoff underground. Underground storage is nearly impossible to maintain properly more than a few feet from each cleanout unless it consists of one large vault; therefore, designers of underground storage that are not one large vault shall assume that no maintenance will occur and shall design the storage area accordingly. In particular, this means all underground storage shall have a pretreatment system or device that removes any trash, debris, and sediment (or other pollutants that has the ability to clog the underground storage) before the flow enters the storage area.

Pipes leading into underground storage basins either must drop the runoff in to the top of the basin or must connect to the side of the basin with the soffit of the pipe as high as possible in the basin. Any volume of the basin above the pipe soffit shall not be used in the calculation for basin volume.

12.14. Linear GI

Linear GI may be located between the sidewalk and private property, between the sidewalk and the existing curb, or in a popout in the parking lane. The sidewalk may be moved to accommodate GI on either side. If a popout is used, it is preferable to use multiple, small popouts in front of several properties rather than one large popout in front of only one property so that all of the lost parking is not in front of only one property.

The minimum width of linear GI is two feet (2'). Ponded water must drain in to the soil within 24 hours.

Longitudinal slope of the linear GI shall not exceed 2% unless check dams are added such that one continuous section does not exceed 2%. Longitudinal flow velocity shall not exceed 1 ft/sec in mulched swales or 3 ft/sec in grassy swales unless check dams are added such that one continuous section does not exceed these parameters.

The surface of GI may be cobble, gravel, dirt, or mulch or may be landscaped. The discharge through the surface material shall be equal to or faster than the discharge of the design storm in to the GI.

For filtration GI, only one underdrain shall be used along the length of the project. The bottom and sides of the GI shall be sloped to a low point to allow the one underdrain to capture and empty the entire cross-section.

12.15. Non-Linear GI

Non-linear GI may collect runoff from surface runoff or via incoming pipes. When pipes are used to convey runoff in to a GI, the high water line of the GI shall be calculated at the invert of the incoming pipe (or, if there are two or more incoming pipes, the invert of the lowest pipe) whenever possible. If slope or pipe cover requirements do not allow this, the high water line may be calculated at the soffit of the incoming pipe but the joints of the incoming pipe shall be watertight. Ponded water must drain in to the soil within 24 hours.

The surface of GI may be cobble, gravel, dirt, or mulch or may be landscaped. The discharge through the surface material shall be equal to or faster than the discharge of the design storm in to the GI.

Filtration GI shall use sloping sides and bottoms to minimize the number of underdrains used.

12.16. Accessibility (Federal ADA and California Regulations) and Safety Requirements

All GI that is not the same elevations as the adjacent surface must comply with the requirements below.

When GI is at least 6” higher than the adjacent surface, designers shall provide reasonable assurances that material (e.g., dirt, mulch, rock) will not fall or wash on to the adjacent surface.

GI which is less than 4” higher than the adjacent surface shall be a contrasting color to serve as a visual cue of the elevation change. A 6” high concrete curb may be used in place of a contrasting color.

For GI lower than the adjacent surface, one of the following shall be implemented:

1. Ensure the drop in to the GI is no more than four inches (4”); or
2. Provide twelve inches (12”) of buffer between the adjacent surface and the start of the slope in the GI. This buffer zone shall be firm, compacted, and stable but can be any material that fulfills the requirement. When used in conjunction with a buffer, the steepest allowable slope in to the GI is 3H:1V.; or
3. Ensure the slope from the adjacent surface is no steeper than 4H:1V; or
4. Provide a standard 6” high concrete curb between the adjacent surface and the GI.

If the criteria above do not apply, but the GI is adjacent to a parking space, wheel stops or other barriers shall be installed to prevent a vehicle from parking in the GI.

Existing sidewalks adjacent to GI shall be evaluated for accessibility compliance if the sidewalk will be impacted during the construction of the GI. Adding curbs around the GI that are attached to the sidewalk are an impact. Non-compliant sidewalks are required to be replaced in accordance with the most current and adopted City of San Diego standards.

Disabled Persons Parking Zones (blue curbs) shall be preserved or replaced. Refer to the most current Council Policy on Disabled Persons Parking Zones and contact Transportation Engineering Operations if the accessible parking spaces are proposed to be relocated or altered in any way.

Accessible diagonal or perpendicular parking spaces shall have dedicated accessible walkways across linear GI. Accessible parallel parking spaces shall have dedicated walkways across linear GI, which shall be the same length as the blue curb. Both walkways shall comply with accessibility regulations and standards regarding size and slope of walkways.

For any on-street accessible parking spaces in residential zones, mix-use, commercial, or industrial zones, one accessible walkway per parcel shall be provided for pedestrian access across linear GI. This walkway shall be aligned as closely as possible to the existing private walkway that connects to the property's main entrance. This walkway shall be located at least 48" from the driveway to separate the driving and pedestrian pathways.

Marked or metered parking spaces adjacent to GI shall be re-evaluated for accessibility compliance. If parking is prohibited adjacent to the GI, whether by red curbs or by signage, no walkways are required across the GI.

12.17. Filter Course

Graded aggregate choker material shall be installed as a filter course to separate bioretention soil media from the drainage rock reservoir layer. This ensures that no migration of sand or other fines occurs. The filter course consists of two layers of choking material increasing in particle size. The top layer of the filter course shall be constructed of thoroughly washed ASTM C33 fine aggregate sand material conforming to gradation limits contained in Table 12-1. The bottom layer of the filter course shall be constructed of thoroughly washed ASTM No. 8 aggregate material conforming to gradation limits contained in Table 12-2.

Table 12-1: Sand Gradation Limits

Sieve Size	Percent Passing Sieves Choker Sand - ASTM C33
0.375 in	100
No. 4	95 – 100
No. 8	80 – 100
No. 16	50 – 85
No. 30	25 – 60
No. 50	5 – 30
No. 100	0 – 10
No. 200	0 – 3

Table 12-2: Crushed Rock and Stone Gradation Limits

Sieve Size	Percent Passing Sieves	
	AASHTO No. 57	ASTM No. 8
3 in	-	-
2.5 in	-	-
2 in	-	-
1.5 in	100	-
1 in	95 – 100	-
0.75 in	-	-
0.5 in	25 – 60	100
0.375 in	-	85 – 100
No. 4	10 max.	10 – 30
No. 8	5 max.	0 – 10
No. 16		0 – 5
No. 50		-

12.18. Hydraulic Restriction Layers

To prevent sideways movement of runoff underground, which may impact the foundation of nearby structures or the street, the following criteria shall be met:

1. Line the GI with an impermeable liner; or
2. Build impermeable side walls or use root barriers the height of the GI; or
3. Build impermeable side walls or use root barriers to the height required by the geotechnical engineer; or
4. Maintain a minimum distance from any structures as determined by the geotechnical engineer; or
5. Maintain a minimum distance of 10 feet from any structure in question.

12.19. Encroachments

In general, infrastructure shall not be allowed on top of underground GI for maintenance and replacement reasons. Buildings or other structures are prohibited. Fences and paved surfaces (asphalt or concrete) shall be avoided whenever possible.

12.20. Proprietary Products

Pre-manufactured, proprietary products may be used for non-priority development projects in place of GI if they meet the requirements in this chapter and are on the approved materials list.

12.21. Operations and Maintenance Plans

No later than 60% design shall the AMD receive an Operations and Maintenance Plan. At minimum, the plan shall include a recommended inspection and cleaning interval with cleaning methodology that includes how to remove accumulated sediment and trash.

THIS PAGE INTENTIONALLY LEFT BLANK FOR DOUBLE-SIDED PRINTING

References

Chapter 3: Hydrology

American Public Works Association. (1981). Urban Storm Water Management. Special Report No. 49.

American Society of Civil Engineers. (1992). Design and Construction of Urban Stormwater Management Systems. ASCE Manual of Practice No. 77/WEF Manual of Practice No. FD-20. New York.

Orange County (California) Environmental Management Agency. (January 1996). Orange County Local Drainage Manual.

U.S. Department of Transportation, Federal Highway Administration. (August 2001). Urban Drainage Design Manual, 2nd edition. Hydraulic Engineering Circular No. 22. FHWA-NHI-01-021.

U.S. Department of Transportation, Federal Highway Administration. (September 2001). Hydraulic Design of Highway Culverts, Hydraulic Design Series No. 5, 2nd edition. FHWA-NHI-01-020.

Chapter 4: Storm Drains

American Public Works Association. (1981) Urban Storm Water Management. Special Report No. 49.

American Society of Civil Engineers. (1992). Design and Construction of Urban Stormwater Management Systems. ASCE Manual of Practice No. 77/WEF Manual of Practice No. FD-20. New York.

City of Los Angeles, Bureau of Engineering, Storm Drain Design Division. (1968). Hydraulic Analysis of Junctions. Office Standard No. 115.

Los Angeles Flood Control District. (January 1988). User Manual, Storm Drain Analysis Program PC/RD4412 (STORM).

U.S. Department of Transportation, Federal Highway Administration. (August 2001). Urban Drainage Design Manual, 2nd edition. Hydraulic Engineering Circular No. 22. FHWA-NHI-01-021.

U.S. Department of Transportation, Federal Highway Administration. (September 2001). Hydraulic Design of Highway Culverts, Hydraulic Design Series No. 5, 2nd edition. FHWA-NHI-01-020.

Chapter 5: Culverts and Low Water Crossings

U.S. Department of Transportation, Federal Highway Administration. (May 1987). HY-8 Microcomputer Program Applications Guide. FHWA-ED-87-101.

U.S. Department of Transportation, Federal Highway Administration. (September 2001). Hydraulic Design of Highway Culverts. Hydraulic Design Series No. 5, 2nd edition. FHWA-NHI-01-020.

Chapter 7: Open Channels

Brater, Ernest, King, Horace, Lindell, James, & Wei, C. (1976). Handbook of Hydraulics. McGraw-Hill Professional. 6th edition.

Brock, Richard R. (July 1967). Development of Roll Waves in Open Channels. California Institute of Technology Report No. KH-R-16.

REFERENCES

California Department of Transportation (Caltrans). (July 2002). Standard Specifications for Construction of Local Streets and Roads. Sacramento, CA.

California Department of Transportation (Caltrans). (May 2001). Highway Design Manual. Sacramento, CA.

California Department of Transportation (Caltrans). (October 2000). California Bank and Shore Rock Slope Protection Design. FHWA-CA-TL-95-10/Caltrans Study No. 990TL03. Sacramento, CA.

Chang, Howard. (1992). Fluvial Processes in River Engineering. Reprint ed. Krieger Publishing: Malabar, Florida.

Chow, V. T. (1959). Open Channel Hydraulics. McGraw-Hill.

Clark County Regional Flood Control District. (August 1999). Hydrologic Criteria and Drainage Design Manual.

Lane, E. W. (1957.) A study of the shape of channels formed by natural streams flossing in erodible material: M.R.D. Sediment Series No. 9, US. Army Engineer Division, Missouri River, Corps of Engineers, Omaha, NE.

Los Angeles County Flood Control District. (April 1979). User Manual, Water Surface Pressure Gradient Hydraulic Analysis Computer Program F0515P. Los Angeles, CA.

Los Angeles County Flood Control District. (March 1982). Hydraulic Design Manual. Los Angeles, California.

Peterka, A. J. (1978). Hydraulic Design of Stilling Basins and Energy Dissipators, U. S. Department of the Interior, Bureau of Reclamation EM 25. Denver, CO.

Public Works Standards, Inc. (2003). Standard Specifications for Public Works Construction, 13th edition. BNi Publications: Anaheim, CA.

Ritter, Dale F. (1986). Process Geomorphology. Wm C. Brown Publishers, Dubuque, IA.

Rosgen, D. (1996). Applied River Morphology. Woldland Hydrology. Pagosa Springs, CO.

Simons, Li & Associates. (1982). Engineering Analysis of Fluvial Systems. Simons, Li & Associates, Fort Collins, Colorado.

The Federal Interagency Stream Restoration Working Group (FISRWG). (2001). Stream Corridor Restoration: Principles, Processes, and Practices.

U. S. Department of Transportation, Federal Highway Administration. (May 2001.) Evaluating Scour at Bridges. Hydraulic Engineering Circular No. 18 (HEC-18). FHWA NHI 01-001. Washington, D. C.

U.S. Army Corps of Engineers. (1993). HEC-6 Scour and Deposition in Rivers and Reservoirs, User's Manual. Hydrologic Engineering Center: Davis, CA.

U.S. Army Corps of Engineers. (June 1994). Hydraulic Design of Flood Control Channels. EM 1110-2-1601. Washington, D.C.

U.S. Army Corps of Engineers. (October 1993). River Hydraulics. EM 1110-2-1416.

U.S. Army Corps of Engineers. (May 1991). HEC-2 Water Surface Profiles. User Manual, Version 4.6.2.

U.S. Army Corps of Engineers. (May 2005). HEC-RAS River Analysis System. User Manual, Version 3.1.3.

U.S. Department of Agriculture Forest Service. (2002). A Soil Bioengineering Guide for Streambank and Lakeshore Stabilization.

U.S. Department of the Interior, Bureau of Reclamation. (1987). Design of Small Dams. Washington, D.C.

Chapter 8: Detention Basins for Flood Control

Internet Resources

California Coastal Commission, http://www.coastal.ca.gov/

California Department of Fish and Game, http://www.dfg.ca.gov/

California Department of Water Resources, Division of Safety of Dams (DSOD), http://damsafety.water.ca.gov/

California State Water Resource Control Board, http://www.swrcb.ca.gov/

Regional Water Quality Control Board, San Diego Region, http://www.waterboards.ca.gov/sandiego/

U.S. Army Corps of Engineers, http://www.usace.army.mil/

U.S. Department of Fish and Wildlife, http://www.fws.gov/

General References

American Society of Civil Engineers. (1993). Design and Construction of Urban Stormwater Management Systems. ASCE Manual of Practice No. 77.

Debo, Thomas N., and Andrew J. Reese. (2002). Municipal Storm Water Management, 2nd edition. CRC Press: New York, NY.

U.S. Department of Transportation, Federal Highways Administration (FHWA). (1996). Urban Drainage Design Manual. Hydraulic Engineering Circular No. 22.

Haestad Methods and S. Rocky Durrans. (2003). Stormwater Conveyance Modeling and Design, First Edition. Waterbury, Connecticut: Haestad Press.

U.S. Department of the Interior, Bureau of Reclamation. (1987). Design of Small Dams, 3rd edition.

Virginia Department of Conservation and Recreation. (1999). Virginia Stormwater Management Handbook.

Chapter 9: Energy Dissipation

U.S. Department of Transportation, Federal Highway Administration (FHWA). (1983). Hydraulic Design of Energy Dissipators for Culverts and Channels. Hydraulic Engineering Circular No. 14. FHWA Publication No. EPD-86-110. Washington, D.C.

U.S. Army Corps of Engineers. (July 1991). Hydraulic Design of Flood Control Channels. Engineer Manual EM 1110-2-1601. Washington, D.C.

U.S. Department of Interior, Bureau of Reclamation. (1978). Design of Small Canal Structures. Denver, CO.

Chapter 10: Debris Barriers and Basins

American Public Works Association and Associated General Contractors of Southern California (APWA-AGC). (2000). California Standard Specifications for Public Works Construction. 12th edition.

Los Angeles County Department of Public Works. (June 1993). Sedimentation Manual.

Los Angeles County Flood Control District. (October 1979). Debris Dams and Basins Design Manual.

U.S. Army Corps of Engineers. (2000). Los Angeles District Method for Prediction of Debris Yield.

U.S. Department of Transportation, Federal Highways Administration. (1971). Debris Control Structures. Hydraulic Engineering Circular No. 9. FHWA Publication No. EPD-86-106. Washington, D.C.

Appendix C: Manning Roughness Coefficients

Barfuss, S., and J. P. Tullis. (1994). Friction factor test on high density polyethylene pipe. Hydraulics Report No. 208. Utah Water Research Laboratory, Utah State University: Logan, UT.

Barnes, H.H. (1964). Roughness Characteristics of Natural Channels. U.S.G.S. Water Supply Paper No. 1849.

Chow, V. T. (1959). Open Channel Hydraulics. New York: McGraw-Hill.

Federal Highway Administration. (1988). Design of Roadside Channels with Flexible Linings. Hydraulic Engineering Circular No. 15 (HEC-15). U.S. Department of Transportation, Washington D.C.

Federal Highway Administration. (1996). Urban Drainage Design Manual. Hydraulic Engineering Circular No. 22 (HEC-22). U.S. Department of Transportation, Washington D.C.

Federal Emergency Management Agency. (2002). Flood Insurance Study for San Diego County. Community No. 06073.

Phillips, J.V., and Ingersoll, T.I. (1997). Comparison of verified roughness coefficients for gravel-bed stream in central Arizona with other areas of the western United States in Floodplain Management. Proceedings of 21st Annual Conference of the Association State Floodplain Managers. Little Rock, AK. pages 154-158.

Thomsen, B.W., and H.W. Hjalmarson. (1991). Estimated Manning's roughness coefficients for stream channels and flood plains in Maricopa County, Arizona. Phoenix, AZ: Flood Control District of Maricopa County.

Appendix D: RCP Design Criteria and Specifications

American Association of State Highway and Transportation Officials (AASHTO) (1973). Standard Specification for Highway Bridges, 12th edition.

Ameron (1973). Loads on Underground Conduits. Engineering Library 1-2.

Spangler, M. G., and R. L. Handy. (1973). Soil Engineering. 3rd edition. Intext Educational Publishers.

Appendix E: Culvert Design Monographs

Department of Transportation, Federal Highway Administration. (September 2001). Hydraulic Design of Highway Culverts. Hydraulic Design Series No. 5, 2nd edition. Publication No. FHWA-NHI-01-020.

THIS PAGE INTENTIONALLY LEFT BLANK FOR DOUBLE-SIDED PRINTING

APPENDICES

THIS PAGE INTENTIONALLY LEFT BLANK FOR DOUBLE-SIDED PRINTING

Appendix A

Rational Method and Modified Rational Method

A.1. Rational Method (RM)

The Rational Method (RM) is a mathematical formula used to determine the maximum runoff rate from a given rainfall. It has particular application in urban storm drainage where it is used to estimate peak runoff rates from small urban and rural watersheds for the design of storm drains and drainage structures. The RM is recommended for analyzing the runoff response from drainage areas for watersheds less than 0.5 square miles. It should not be used in instances where there is a junction of independent drainage systems or for drainage areas greater than approximately 0.5 square mile in size. In these instances, the Modified Rational Method (MRM) should be used for junctions of independent drainage systems in watersheds up to approximately 1 square mile in size (see Section A.2); or the NRCS Hydrologic Method should be used for watersheds greater than approximately 1 square mile in size (see Appendix B).

A.1.1. Rational Method Formula

The RM formula estimates the peak rate of runoff at any location in a watershed as a function of the drainage area (A), runoff coefficient (C), and rainfall intensity (I) for a duration equal to the time of concentration (T_c), which is the time required for water to flow from the most remote point of the basin to the location being analyzed. The RM formula is expressed in Equation A-1.

Equation A-1. RM Formula Expression

$$Q = C\ I\ A$$

where:

Q	=	peak discharge, in cubic feet per second (cfs)
C	=	runoff coefficient expressed as that percentage of rainfall which becomes surface runoff (no units); Refer to Appendix A.1.2
I	=	average rainfall intensity for a storm duration equal to the time of concetrnatation (T_c) of the contributing draiange area, in inches per hour; Refer to Appendix A.1.3 and Appendix A.1.4
A	=	drainage area contributing to the design location, in acres

Combining the units for the expression CIA yields:

$$\left(\frac{1\,\text{acre}\times\text{inch}}{\text{hour}}\right)\left(\frac{43{,}560\,\text{ft}^2}{\text{acre}}\right)\left(\frac{1\,\text{foot}}{12\,\text{inches}}\right)\left(\frac{1\,\text{hour}}{3{,}600\,\text{seconds}}\right)\Rightarrow 1.008\,\text{cfs}$$

For practical purposes, the unit conversion coefficient difference of 0.8% can be ignored.

The RM formula is based on the assumption that for constant rainfall intensity, the peak discharge rate at a point will occur when the raindrop that falls at the most upstream point in the tributary drainage basin arrives at the point of interest.

Unlike the MRM (discussed in Appendix A.2) or the NRCS hydrologic method (discussed in Appendix B), the RM does not create hydrographs and therefore does not add separate subarea hydrographs at collection points. Instead, the RM develops peak discharges in the main line by increasing the T_c as flow travels downstream.

Characteristics of, or assumptions inherent to, the RM are listed below:

1. The discharge resulting from any I is maximum when the I lasts as long as or longer than the T_c.
2. The storm frequency of peak discharges is the same as that of I for the given T_c.
3. The fraction of rainfall that becomes runoff (or the runoff coefficient, C) is independent of I or precipitation zone number (PZN) condition (PZN Condition is discussed in the NRCS method).
4. The peak rate of runoff is the only information produced by using the RM.

A.1.2. Runoff Coefficient

The runoff coefficients are based on land use (see Table A–1). Soil type "D" is used throughout the City of San Diego for storm drain conveyance design. An appropriate runoff coefficient (C) for each type of land use in the subarea should be selected from this table and multiplied by the percentage of the total area (A) included in that class. The sum of the products for all land uses is the weighted runoff coefficient (Σ[CA]). Good engineering judgment should be used when applying the values presented in Table A–1, as adjustments to these values may be appropriate based on site-specific characteristics.

Table A-1. Runoff Coefficients for Rational Method

Land Use	Runoff Coefficient (C)
	Soil Type [1]
Residential:	
Single Family	0.55
Multi-Units	0.70
Mobile Homes	0.65
Rural (lots greater than ½ acre)	0.45
Commercial [2]	
80% Impervious	0.85
Industrial [2]	
90% Impervious	0.95

Note:
[1] Type D soil to be used for all areas.
[2] Where actual conditions deviate significantly from the tabulated imperviousness values of 80% or 90%, the values given for coefficient C, may be revised by multiplying 80% or 90% by the ratio of actual imperviousness to the tabulated imperviousness. However, in case shall the final coefficient be less than 0.50. For example: Consider commercial property on D soil.

Actual imperviousness = 50%
Tabulated imperviousness = 80%
Revised C = (50/80) x 0.85 = 0.53

The values in Table A–1 are typical for urban areas. However, if the basin contains rural or agricultural land use, parks, golf courses, or other types of nonurban land use that are expected to be permanent, the appropriate value should be selected based upon the soil and cover and approved by the City.

A.1.3. Rainfall Intensity

The rainfall intensity (I) is the rainfall in inches per hour (in/hr.) for a duration equal to the T_C for a selected storm frequency. Once a particular storm frequency has been selected for design and a T_C calculated for the drainage area, the rainfall intensity can be determined from the Intensity-Duration-Frequency Design Chart (Figure A-1).

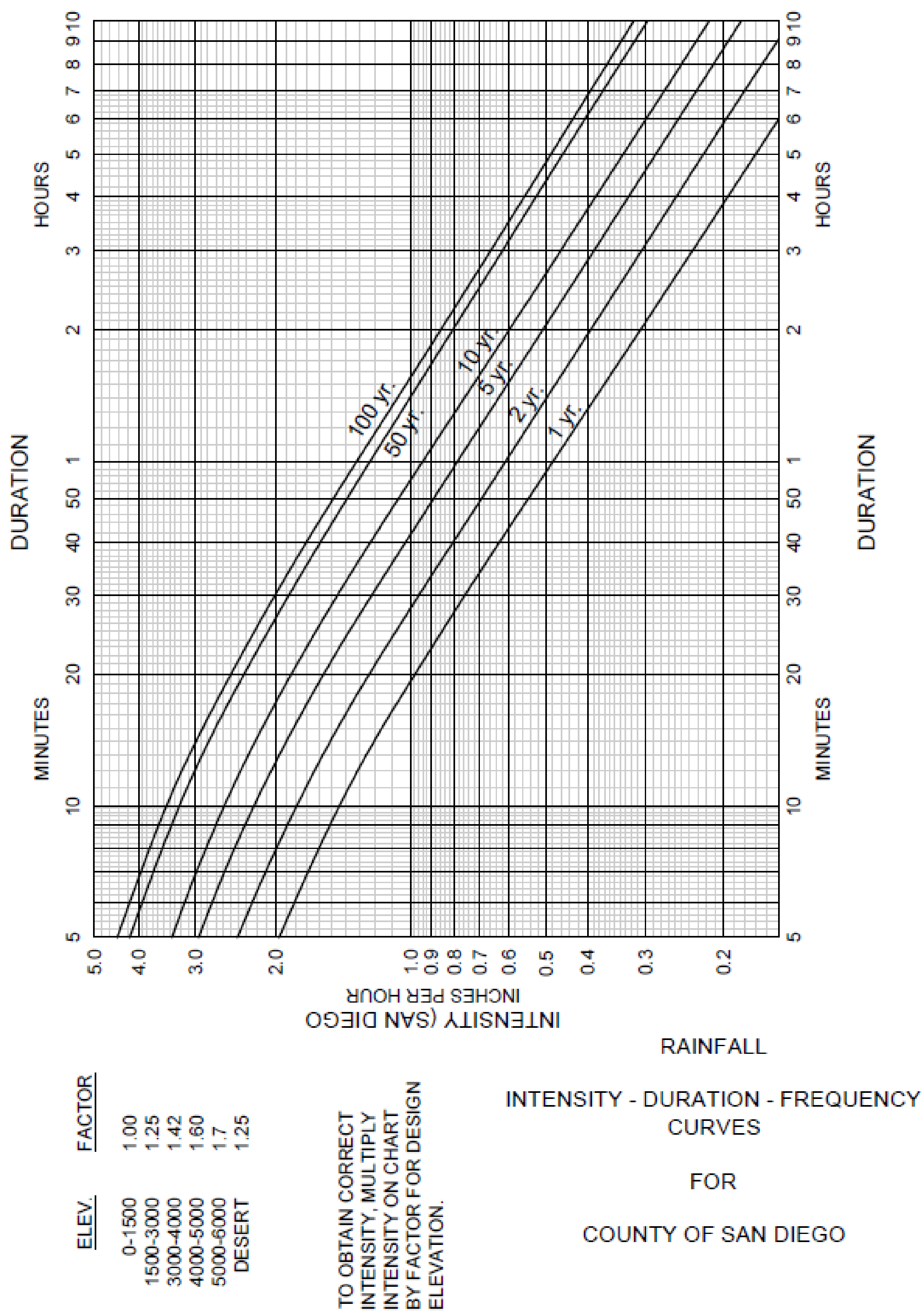


Figure A-1. Intensity-Duration-Frequency Design Chart

A.1.4. Time of Concentration

The Time of Concentration (T_c) is the time required for runoff to flow from the most remote part of the watershed to the outlet point under consideration.

Methods of calculation differ for natural watersheds (non-urbanized) and for urban drainage systems. Also, when designing storm drain systems, the designer must consider the possibility that an existing natural watershed may become urbanized during the useful life of the storm drain system. Future land uses must be used for Tc and runoff calculations, and can be determined from the Community Plans.

a. Natural watersheds: Obtain Tc from Figures A.2 and A.3

b. Urban drainage systems: In the case of urban drainage systems, the time of concentration at any point within the drainage area is given by:

   $T_c = T_i + T_t$ where

   T_i is the inlet time or the time required for the storm water to flow to the first inlet in the system. It is the sum of time in overland flow across lots and in the street gutter.

   T_t is the travel time or the time required for the storm water to flow in the storm drain from the most upstream inlet to the point in question.

   Travel Time, T_t is computed by dividing the length of storm drain by the computed flow velocity. Since the velocity normally changes at each inlet because of changes in flow rate or slope, total travel time must be computed as the sum of the travel times for each section of the storm drain.

   The overland flow component of inlet time, T_i, may be estimated by the use of the chart shown in Figure A-4. Use Figure A-5 to estimate time of travel for street gutter flow.

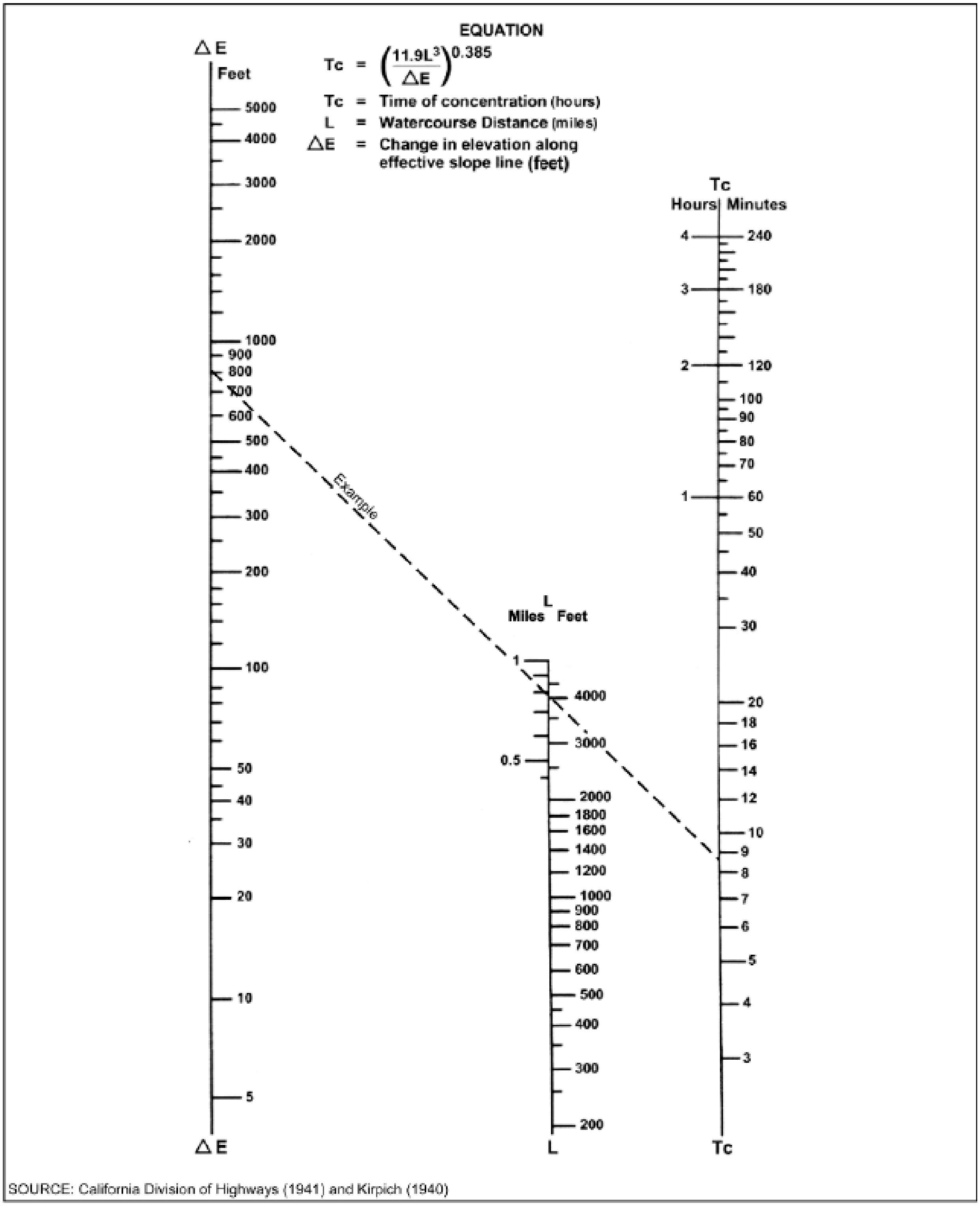
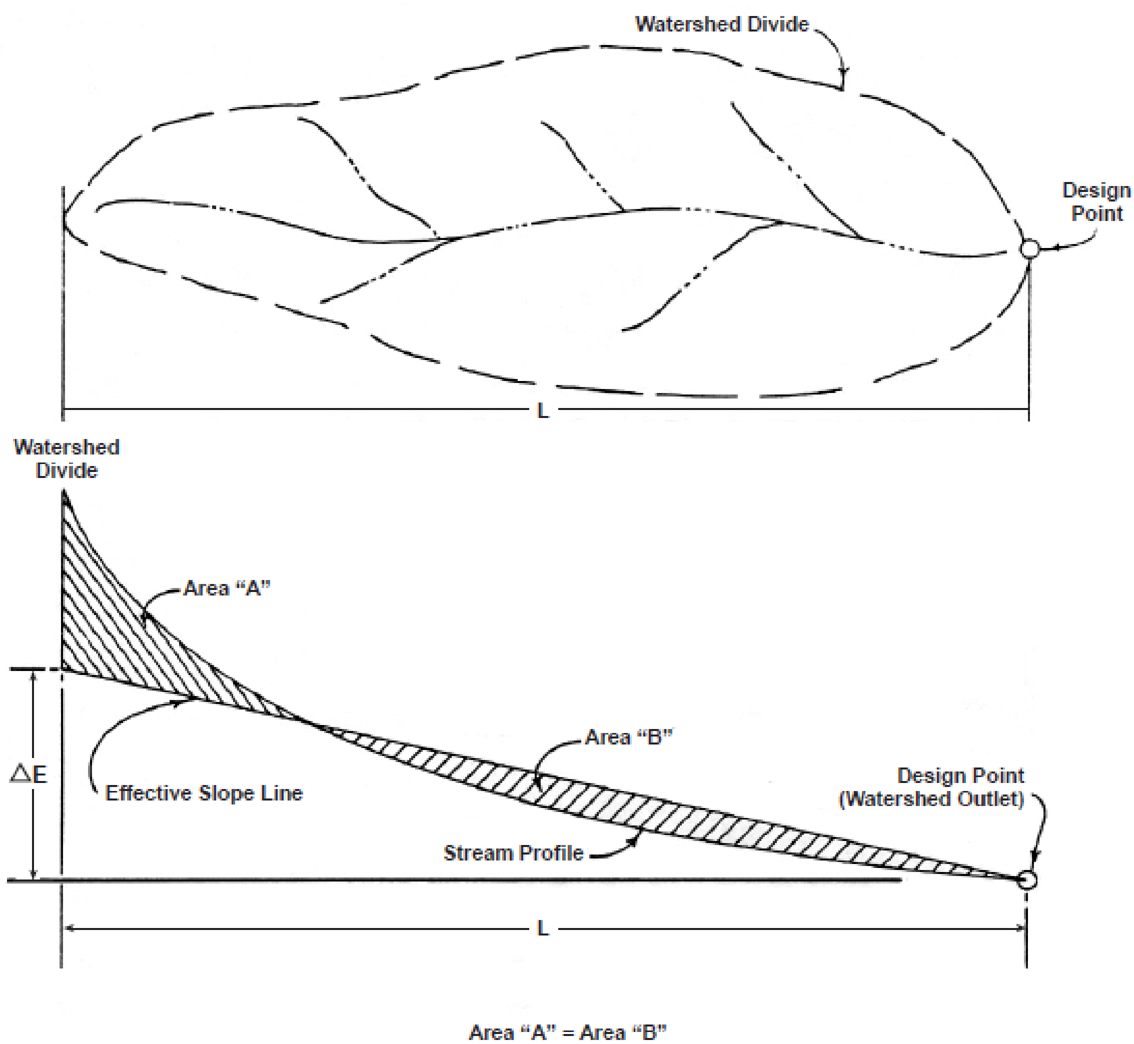


Figure A-2. Nomograph for Determination of T_c for Natural Watersheds

<u>**Note**</u>: Add ten minutes to the computed time of concentration from Figure A-2.



SOURCE: California Division of Highways (1941) and Kirpich (1940)

Figure A-3. Computation of Effective Slope for Natural Watersheds

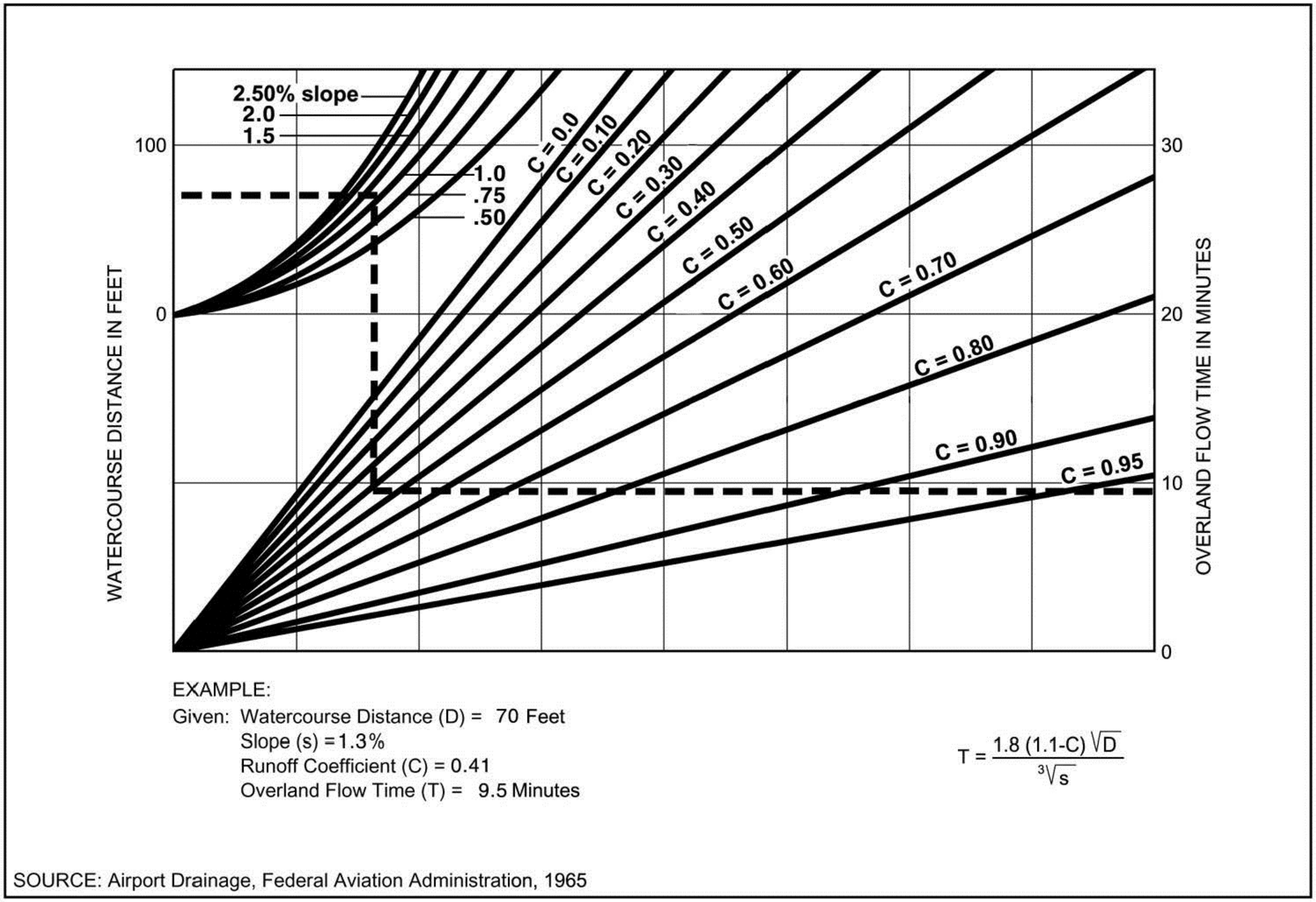


Figure A-4. Rational Formula – Overland Time of Flow Nomograph

<u>Note</u>: Use formula for watercourse distances in excess of 100 feet.

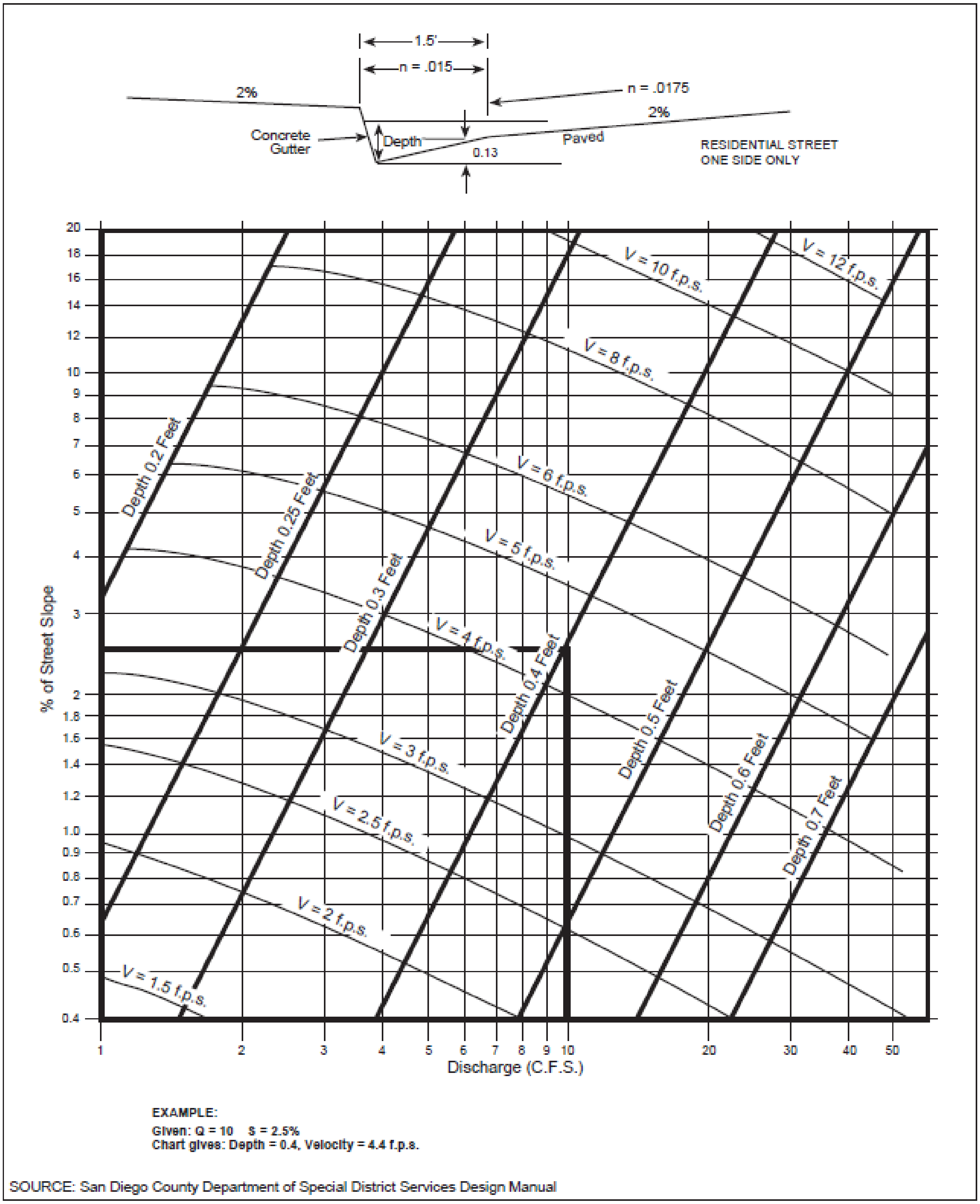


Figure A-5. Gutter and Roadway Discharge – Velocity Chart

THIS PAGE INTENTIONALLY LEFT BLANK FOR DOUBLE-SIDED PRINTING

A.2. Modified Rational Method (MRM; for Junction Analysis)

The purpose of this section is to describe the steps necessary to develop a hydrology report for a small watershed using the MRM. It is necessary to use the MRM if the watershed contains junctions of independent drainage systems. The process is based on the design manuals of the City/County of San Diego. The general process description for using this method is described below.

The engineer should only use the MRM for drainage areas up to approximately 1 square mile in size. If the watershed will significantly exceed 1 square mile then the NRCS method described in Appendix B should be used.

A.2.1. Modified Rational Method General Process Description

The general process for the MRM differs from the RM only when a junction of independent drainage systems is reached. The peak Q, T_c, and I for each of the independent drainage systems at the point of the junction are calculated by the RM. The independent drainage systems are then combined using the MRM procedure described below. The peak Q, T_c, and I for each of the independent drainage systems at the point of the junction must be calculated prior to using the MRM procedure to combine the independent drainage systems, as these values will be used for the MRM calculations. After the independent drainage systems have been combined, RM calculations are continued to the next point of interest.

A.2.2. Procedure for Combining Independent Drainage Systems at a Junction

1. Calculate the peak Q, T_c, and I for each of the independent drainage systems at the point of the junction. These values will be used for the MRM calculations.
2. At the junction of two or more independent drainage systems, the respective peak flows are combined to obtain the maximum flow out of the junction at T_c. Based on the approximation that total runoff increases directly in proportion to time, a general equation may be written to determine the maximum Q and its corresponding T_c using the peak Q, T_c, and I for each of the independent drainage systems at the point immediately before the junction. The general equation requires that contributing Qs be numbered in order of increasing T_c.
3. Let Q_1, T_1, and I_1 correspond to the tributary area with the shortest T_c. Likewise, let Q_2, T_2, and I_2 correspond to the tributary area with the next longer $T_{c,}$ Q_3, T_3, and I_3 correspond to the tributary area with the next longer T_c, and so on. When only two independent drainage systems are combined, leave Q_3, T_3, and I_3 out of the equation. Combine the independent drainage systems using the junction equation (see Equation A-2).

Equation A-2. Junction Equation

$$T_1 < T_2 < T_3$$

$$Q_{T1} = Q_1 + \frac{T_1}{T_2} Q_2 + \frac{T_1}{T_3} Q_3$$

$$Q_{T2} = Q_2 + \frac{I_2}{I_1} Q_1 + \frac{T_2}{T_3} Q_3$$

$$Q_{T3} = Q_3 + \frac{I_3}{I_1} Q_1 + \frac{I_3}{I_2} Q_2$$

4. Calculate Q_{T1}, Q_{T2}, and Q_{T3}. Select the largest Q and use the T_c associated with that Q for further calculations (see the three Notes for options). If the largest calculated Q's are equal (e.g., $Q_{T1} = Q_{T2} > Q_{T3}$), use the shorter of the T_cs associated with that Q.
5. This equation may be expanded for a junction of more than three independent drainage systems using the same concept. The concept is that when Q from a selected subarea (e.g., Q_2) is combined with Q from another subarea with a shorter T_c (e.g., Q_1), the Q from the subarea with the shorter T_c is reduced by the ratio of the I's (I_2/I_1); and when Q from a selected subarea (e.g., Q_2) is combined with Q from another subarea with a longer T_c (e.g., Q_3), the Q from the subarea with the longer T_c is reduced by the ratio of the T_cs (T_2/T_3).

The following notes should be considered:

Note #1: At a junction of two independent drainage systems that have the same T_c, the tributary flows may be added to obtain the Q_p.

$$Q_p = Q_1 + Q_2 \text{ ; when } T_1 = T_2\text{; and } T_c = T_1 = T_2$$

This can be verified by using the junction equation above. Let Q_3, T_3, and $I_3 = 0$. When T_1 and T_2 are the same, I_1 and I_2 are also the same, and T_1/T_2 and $I_2/I_1 = 1$. T_1/T_2 and I_2/I_1 are cancelled from the equations. At this point, $Q_{T1} = Q_{T2} = Q_1 + Q_2$.

Note #2: In the upstream part of a watershed, a conservative computation is acceptable. When the times of concentration are relatively close in magnitude (within 10%), use the shorter T_c for the intensity and the equation $Q = \Sigma(CA)I$.

Appendix B

Soil Conservation Service: NRCS Hydrologic Method

The Soil Conservation Service (SCS) (now called the Natural Resources Conservation Service [NRCS]) hydrologic method (NRCS hydrologic method) requires basic data similar to the RM: drainage area, a "runoff curve number" (CN) describing the proportion of rainfall that runs off, time to peak (T_p), the elapsed time from the beginning of unit effective rainfall to the peak flow for the point of concentration, and total rainfall (P). The NRCS approach, however, is more sophisticated in that it also considers the time distribution of the rainfall, the initial rainfall losses to interception and depression storage, and an infiltration rate that decreases during the course of a storm. Results of the NRCS approach are more detailed, in the form of a runoff hydrograph. Details of the methodology can be found in the NRCS National Engineering Handbook (NEH), Section 4 (NEH-4) (USDA, 1985). The NRCS hydrologic method should be used for study areas approximately 1 square mile and greater in size.

B.1. Procedure for Calculation of Runoff Curve Number (CN)

1. Locate basin on 1:2000 scale USGS topographic map(s).
2. Using a ½-inch or 1-inch grid (1/2-inch for areas less than 5 square miles) on a translucent overlay sheet, trace the basin boundary and other significant information from the topographic maps.
3. Locate basin on 1:2000 scale SCS hydrologic ground cover and soil group maps at the offices of the Department of Sanitation and Flood Control.
4. Overlay the grid sheet onto the ground cover and soil group maps; for each map record appropriate group cover (OB, NC, DL, etc.) and soil group (A, B, B, or D) at each grid intersection within the basin.
5. For each combination of ground cover/soil group (OB/A, NC/B, NC/D, etc.) count and record the number of grid intersections where that combination occurs.
6. Compute the total number of grid intersections within the basin. For a 1-inch grid, each intersection represents 1 square inch on the maps, and the total area of the basin is found by scale conversion; for ½-inch grid, each intersection is ¼ square inch. Compute the total area of the basin.
7. By field inspection, determine the hydrologic conditions which exist in the basin for each type of ground cover.

8. For each ground cover/soil group combination compute the fraction of the total area represented by that combination by the ratio of the number grid intersections counted in Step 5 to the total number of grid intersections counted in Step 6.
9. For each ground cover/soil group/hydrologic condition combination, select the appropriate runoff curve number for antecedent moisture condition 2 (CN_2). Refer to **Table B-1**.
10. Compute the partial CN_2 for each ground cover/soil group combination by the product of area fraction of each combination from Step 8 and the selected CN_2's from Step 9.
11. Sum the partial CN_2's from Step 10 to obtain the CN_2 for the entire basin.
12. For future land uses modify existing ground cover designations and use same procedures.
13. If stream bed is alluvial fill with deep group "A" soils (sand and gravels), the CN adjustment procedure should be considered.

Table B-1. Runoff Curve Numbers for Hydrologic Soil-Cover Complexes (CN); AMC 2; I_a = 0.2S

Cover			Hydrologic Soil Groups			
Land Use	Treatment or Practice	Hydrologic Condition	A	B	C	D
Water Surfaces (During Floods)			97	98	99	99
Urban						
Commercial-industrial			89	90	91	92
High density residential			75	82	88	90
Medium density residential			73	80	86	88
Low density residential			70	78	84	87
Barren			78	86	91	93
Fallow	Straight row		76	85	90	90
Vineyards	Disked		76	85	90	92
	Annual grass or legume cover	Poor	65	78	85	89
		Fair	50	69	79	84
		Good	38	61	74	80
Roads	Hard surface		74	84	90	92
	Dirt		72	82	87	89
Row Crops	Straight row	Poor	72	81	88	91
		Good	67	78	85	89
	Contoured	Poor	70	79	84	88
		Good	65	75	82	86

Cover			Hydrologic Soil Groups			
Land Use	Treatment or Practice	Hydrologic Condition	A	B	C	D
Narrowleaf chaparral		Poor	71	82	88	91
		Fair	55	72	81	86
Perennial grass		Poor	67	79	86	89
		Fair	50	69	79	84
		Good	38	61	74	80
Annual grass		Poor	67	78	86	89
		Fair	50	69	79	84
		Good	38	61	74	80
Close-seeded legumes or rotated pasture	Straight row	Poor	66	77	85	89
		Good	58	72	81	85
	Contoured	Poor	64	75	83	85
		Good	55	69	78	83
Small grain	Straight row	Poor	65	76	84	88
		Good	63	75	83	87
	Contoured	Poor	63	74	82	85
		Good	61	73	81	84
Meadow		Poor	63	77	85	88
		Fair	51	70	80	84
		Good	30	58	72	78
Open brush		Poor	62	76	84	88
		Fair	46	66	77	83
		Good	41	63	75	81
Farmsteads			59	74	82	86
Irrigated pasture		Poor	58	74	83	87
		Fair	44	65	77	82
		Good	33	58	72	79
Turf		Poor	58	74	83	87
		Fair	44	65	77	82
		Good	33	58	72	79

Cover			Hydrologic Soil Groups			
Land Use	Treatment or Practice	Hydrologic Condition	A	B	C	D
Woodland-grass		Poor	57	73	82	86
		Fair	44	65	77	82
		Good	33	58	72	79
Orchards (evergreen)		Poor	57	73	82	86
		Fair	44	65	77	82
		Good	33	58	72	79
Broadleaf chaparral		Poor	53	70	80	85
		Fair	40	63	75	81
		Good	31	57	71	78
Woods (woodland)		Poor	45	66	77	83
		Fair	36	60	73	79
		Good	28	55	70	77

B.2. Procedure for Calculation of Lag Time and Time to Peak

1. Locate basin on 1:2000 scale USGS topographic map(s).
2. Compute:
    a. Drainage area, A, square miles.
    b. Length of longest watercourse, L in miles.
    c. L_c, length along longest watercourse in miles, measured upstream to point opposite center of area.
3. Compute overall slope, S:
    a. E_h = elevation of most remote point on watercourse, in feet.
    b. E_1 = elevation at outlet, in feet.
    c. $S = [E_h – E_1]/L$, in feet/mile.
4. By field inspection select basin n factor, the average of the Manning's n values of the watercourse and tributaries
5. Compute Lag time using **Equation B-1**.

EquationB-1. Empirical formula expressing Lag time

$$Lag=24\bar{n}\left(\frac{L\times L_c}{s^{0.5}}\right)^m$$

where:

L	=	Length to longest watercourse (miles)
Lc	=	Length along longest watercourse, measured upstream to a point opposite the watershed centroid (miles)
s	=	Overall slope of drainage area between the headwaters and the collection point (feet per mile)
m	=	Constant determined by regional flood reconstitution studies (0.38 for San Diego County)
$\bar{n}$	=	Average of the Manning's n values of the watercourse and its tributaries

6. If no unit hydrograph has been developed for the basin based on recorded rainfall- runoff data, use NRCS dimensionless unit hydrograph for which the time to peak shall be estimated using **Equation B-2**.

Equation B-2. Relationship of Time to Peak to Lag time

$$Lag=1.16T_p$$

Or

$$T_p=0.0862 \text{ x } Lag$$

B.3. Procedure for using Peak Flow Charts to Compute Peak Flow

1. Determine design recurrence interval (frequency).
2. Determine the precipitation zone number (PZN) of the center of the basin on the map of **Figure B-1**.
3. Determine the antecedent moisture condition (AMC) from **Table B-3** for the appropriate PZN. Interpolation, if necessary, is linear.
4. Adjust basin CN_2 for AMC calculated in Step 3. CN_1 and CN_3 values are given in **Table B-2** and interpolation is linear.
5. From the precipitation maps, select the 6 and 24-hour precipitation amounts for the design frequency for the center of the basin.
6. If the basin area is greater than 10 square miles from **Figure B-4**, determine the area rainfall reduction ratio and reduce the point precipitation amounts from Step 5 by that ratio.
7. Determine the time to peak (T_p) and check if it's in the range covered by the Peak Flow Charts.

8. Select from the Peak Flow Charts times to peak (T_{pa} and T_{pb}) which bracket the computed basin T_p, if no chart is available for the exact basin T_p.
   For the adjusted CN from Step 4 and the precipitation amounts from Steps 5 and 6. Select flows Q_{a6} and Q_{b6}.
9. Using the formulas below, interpolate Q'_6 between Q_{a6} and Q_{b6}, and Q'_{24} between Qa_{24} and Q_{b24}.

$$Q'_6 = Q_{b6} + (Q_{a6} - Q_{b6}) \frac{T_{pb} - T_p}{T_{pb} - T_{pa}}$$

$$Q'_{24} = Q_{b24} + (Q_{a24} - Q_{b24}) \frac{T_{pb} - T_p}{T_{pb} - T_{pa}}$$

10. Select the greater of Q'_6 and Q'_{24}.
11. Compute the ratio of the actual area of the basin to 10 square miles (R_a).
12. Compute the peak flow from the basin by the product of the basin area ratio from Step 11 and the flow from Step 10.
13. Record a summary of the computations:
    a. The computed peak flow from Step 12.
    b. The design frequency from Step 1.
    c. The duration of the controlling storm (6-hour or 24-hour) from Step 10.

Worksheet B-1. Peak Flow Computation Worksheet

(For use with Peak Flow Charts; Western Drainage Precipitation Zones 1.0 to 3.5 only)

Step	Peak Flow Computation Worksheet								
	Basin Name:								
	Geographic Location of center of basin:			Long:			Lat:		
1	Strom Frequency			year	Basin Area:		Sq. miles		
2	Precipitation Zone Number (Refer to Figure B-1) PZN =								
3	Antecedent Moisture Condition (Refer to Table B-3) =								
4	Runoff Curve Numbers (Refer to Table B-2) (interpolate basin CN for basin AMC)								
		CN_2		CN		CN_3			
5	Precipitation for Storm Duration:				6 hr. (P_6)		inches		
					24 hr. (P_{24})		inches		
6	Area Rainfall Reduction Ratio (R_r) (Refer to Figure				P_6 x R_r		inches		
	B-4; for areas greater than 10 square miles)				$P24$ x R_r		inches		
7	Determine Time to Peak (T_p)								
7a	Lag (Estimate using Equation B-1)						hours		
7b	Time to Peak (Estimate using Equation B-1)						hours		
8	Enter Peak Flow Charts with T_p, CN, P_6 and P_{24}; select two nearest T_p's available								
				6 hour storm			24 hour storm		
8a	T_{pa}		hour	Q_{a6}		cfs	Q_{a24}		cfs
8b	T_{pb}		hour	Q_{b6}		cfs	Q_{b24}		cfs
9a	$Q'_6 = Q_{b6} + (Q_{a6} - Q_{b6})\dfrac{T_{pb} - T_p}{T_{pb} - T_{pa}}$						cfs		
9b	$Q'_{24} = Q_{b24} + (Q_{a24} - Q_{b24})\dfrac{T_{pb} - T_p}{T_{pb} - T_{pa}}$						cfs		
10	Greater of Q'_6 and Q'_{24} for 10 sq. mile basin= max (9a, 9b)						cfs		
11	Basin Area Ratio (Ra) = basin area / 10 sq. miles						unitless		
12	Peak Flow (Q_p) = ($Q'_{6\text{ or }24}$) x (R_a) = Step 10 x Step 11						cfs		
13	**SUMMARY**								
	Peak Flow				cfs				
	Frequency				year				
	Duration				hour				

Table B-2. Curve Numbers (CN) and Constants for the Case I_a = 0.2S

Curve Number			S (inches)	Curve Starts Where P = (inches)	Curve Number			S (inches)	Curve Starts Where P = (inches)
AMC II	AMC I	AMC III			AMC II	AMC I	AMC III		
100	100	100	0	0	60	40	78	6.67	1.33
99	97	100	.101	.02	59	39	77	9.95	1.39
98	94	99	.204	.04	58	38	76	7.24	1.45
97	91	99	.309	.06	57	37	75	7.54	1.51
96	89	99	.417	.08	56	36	75	7.86	1.57
95	87	98	.526	.11	55	35	74	8.18	1.64
94	85	98	.638	.13	54	34	73	8.52	1.70
93	83	98	.753	.15	53	33	72	8.87	1.77
92	81	97	.870	.17	52	32	71	9.23	1.85
91	80	97	.989	.20	51	31	70	9.61	1.92
90	78	96	1.11	.22	50	31	70	10.0	2.00
89	76	96	1.24	.25	49	30	69	10.4	2.08
88	75	95	1.36	.27	48	29	68	10.8	2.16
87	73	95	1.49	.30	47	28	67	11.3	2.26
86	72	94	1.63	.33	46	27	66	11.7	2.34
85	70	94	1.76	.35	45	26	65	12.2	2.44
84	68	93	1.90	.38	44	25	64	12.7	2.54
83	67	93	2.05	.41	43	25	63	13.2	2.64
82	66	92	2.20	.44	42	24	62	13.8	2.76
81	64	92	2.34	.47	41	23	61	14.4	2.88
80	63	92	2.50	.50	40	22	60	15.0	3.00
79	62	91	2.66	.53	39	21	59	15.6	3.12
78	60	90	2.82	.56	38	21	58	16.3	3.26
77	59	89	2.99	.60	37	20	57	17.0	3.40
76	58	89	3.16	.63	36	19	56	17.8	3.56
75	57	88	3.33	.67	35	18	55	18.6	3.72
74	55	88	3.51	.70	34	18	54	19.4	3.88
73	54	87	3.70	.74	33	17	53	20.3	4.06
72	53	86	3.89	.78	32	16	52	21.2	4.24
71	52	86	4.08	.82	31	16	51	22.2	4.44
70	51	85	4.28	.86	30	15	50	23.3	4.66
69	50	84	4.49	.90					
68	48	84	4.70	.94	25	12	43	30.0	6.00
67	47	83	4.92	.98	20	9	37	40.0	8.00
66	46	82	5.15	1.03	15	6	30	56.7	11.34
65	45	82	5.38	1.08	10	4	22	90.0	18.00
64	44	81	5.62	1.12	5	2	13	190.0	38.0
63	43	80	5.87	1.17	0	0	0	infinity	infinity
62	42	79	6.13	1.23					
61	41	78	6.39	1.28					

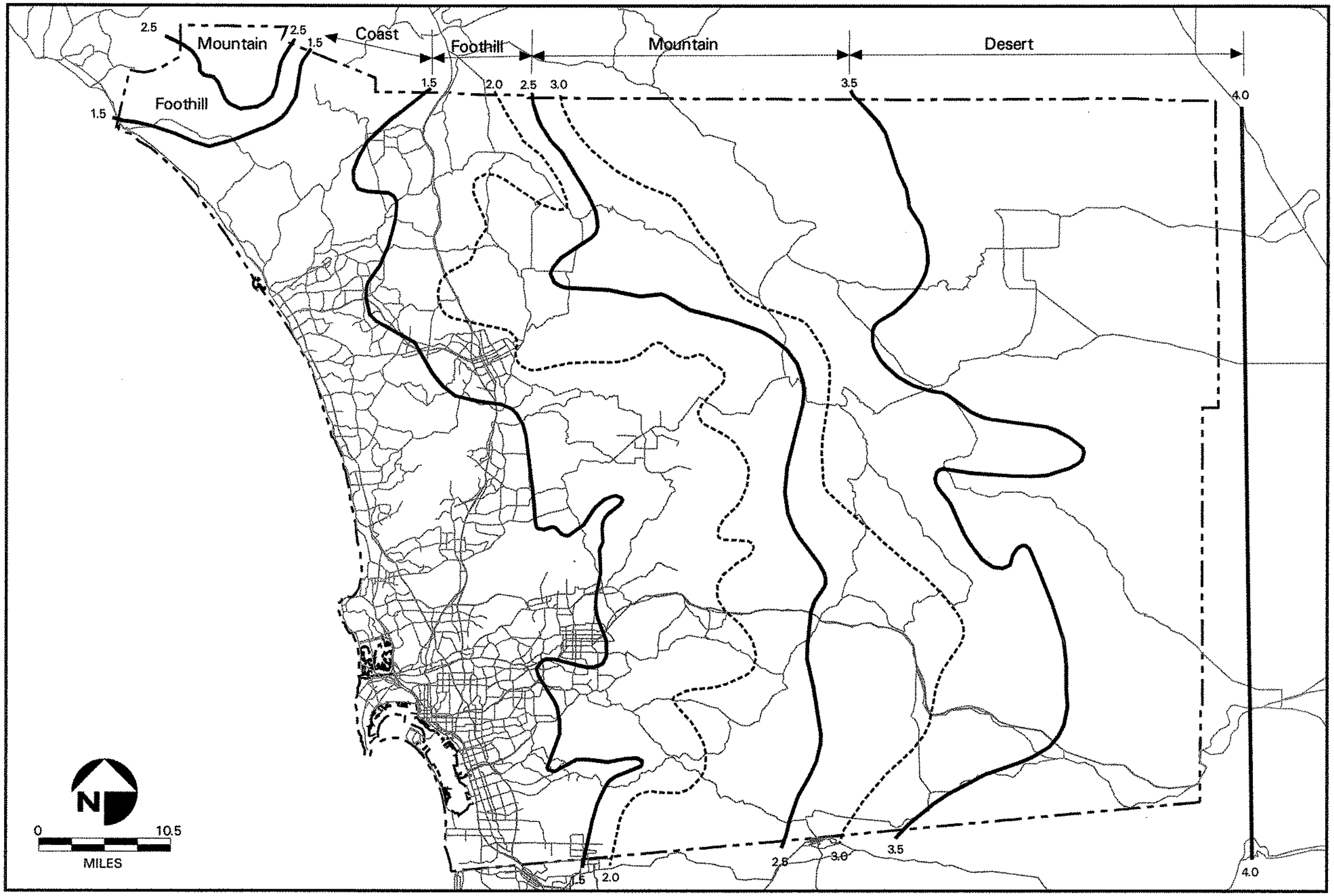


Figure B-1. Precipitation Zone Numbers (PZN).

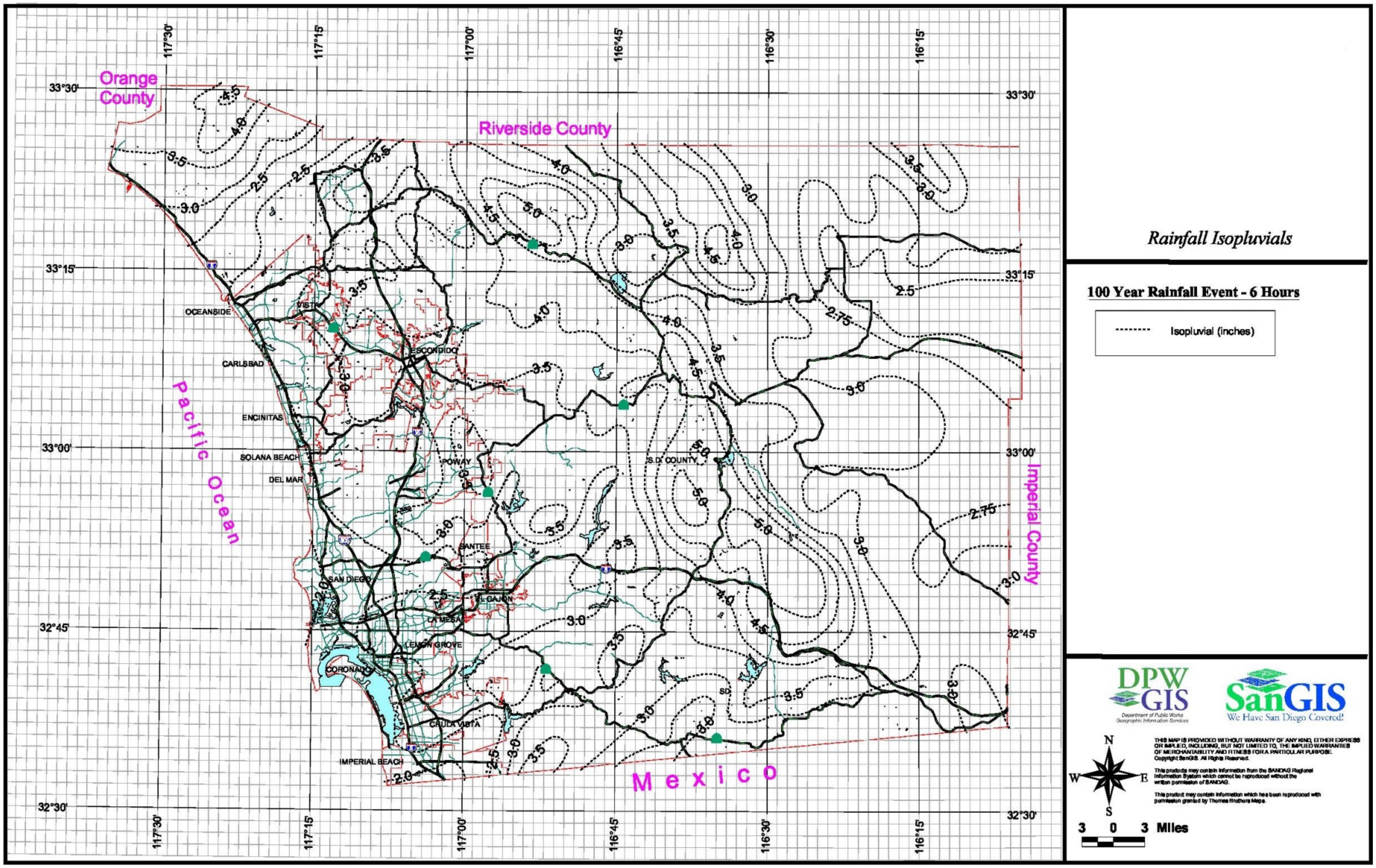


Figure B-2. 100-Year 6-Hour Isopluvials.

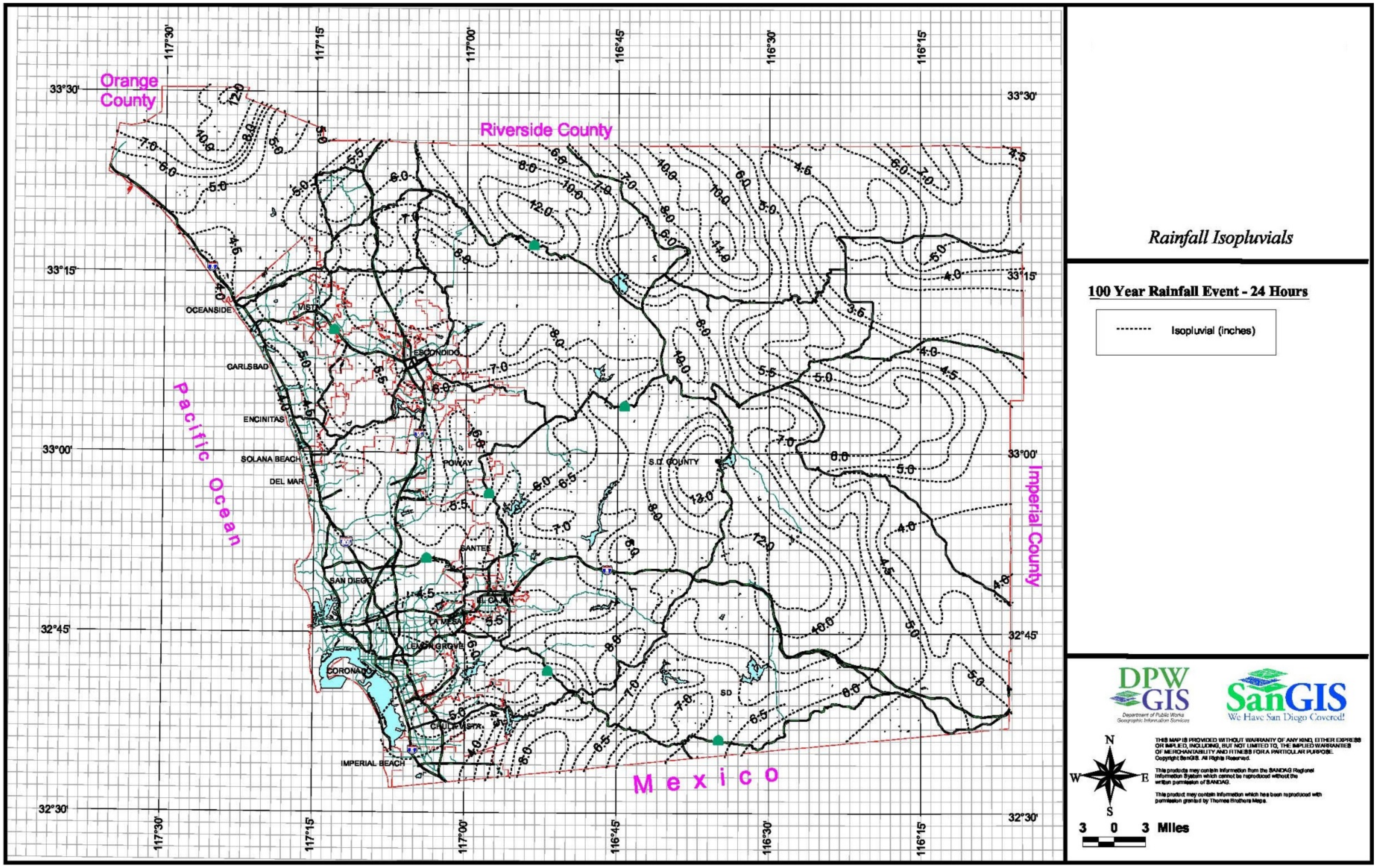


Figure B-3. 100-Year 24-Hour Isopluvials

B.4. Rainfall Distribution

The County is divided into two main drainages. The westerly drainage area (about two-thirds of the County) drains toward the Pacific Ocean. The remaining portion of the County drains easterly to the dessert.

B.4.1. Western Drainage

The Type B distribution supersedes Type 1 for 24-hour duration storms and is to be used in San Diego County. The effect of using Type B instead of Type 1 is to lower peak flows for smaller basins. The 6-hour Type B distribution (**Figure B-4**) controls for certain smaller basins (depending upon area, time to peak, CN, frequency, etc.), producing greater flows than derived from 24-hour Type B, and the larger peak flow should be used.

The antecedent moisture condition (AMC) to be used for flood flow computations is given in Table B-3 below:

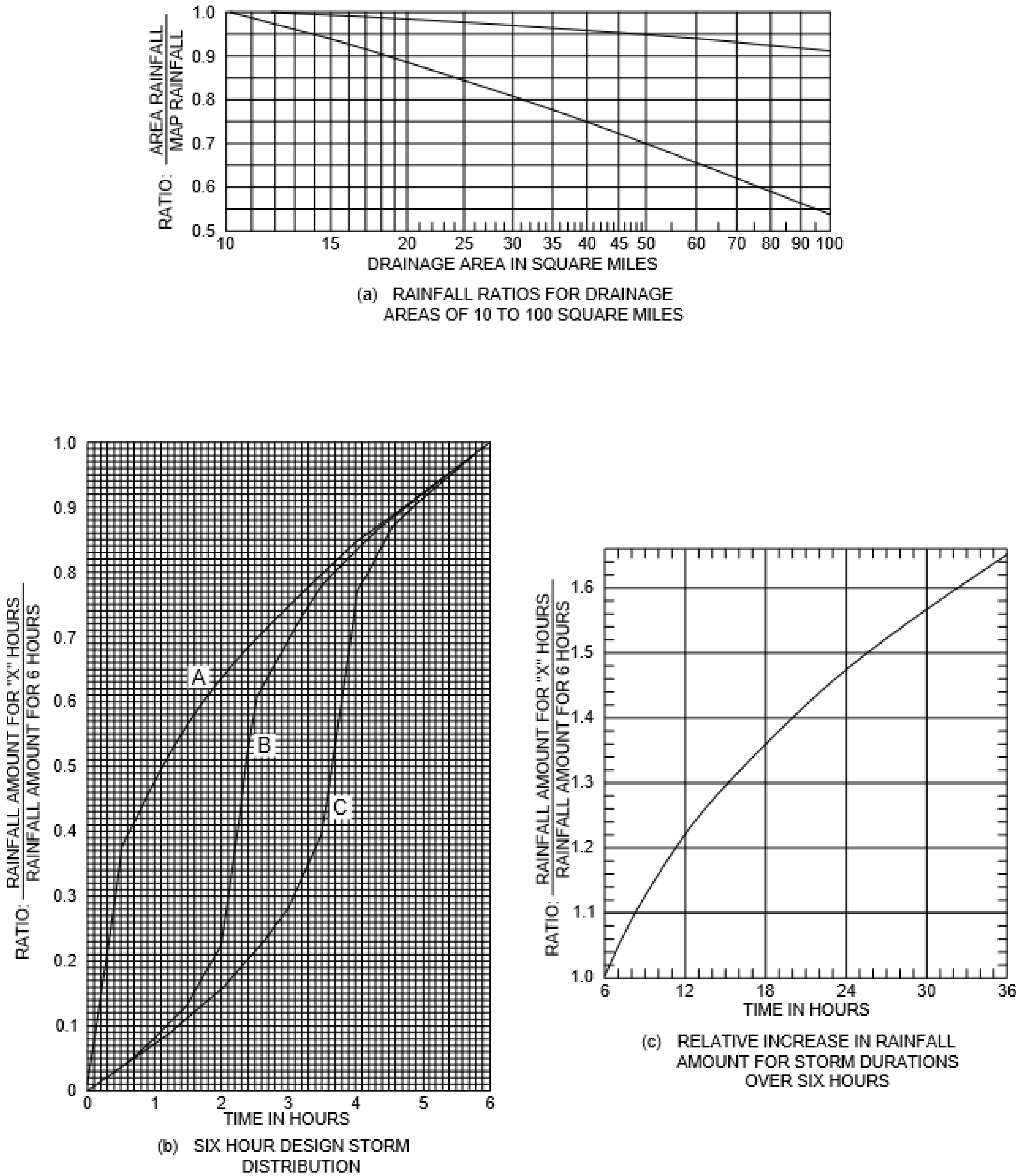
Table B-3. Antecedent Moisture Condition for Flood Flow Computations (San Diego County).

Storm Frequency	Coast (PZN = 1.0)	Foothills (PZN = 2.0)	Mountains (PZN = 3.0)	Desert (PZN = 4.0)
5 to 35 years	1.5	2.5	2.0	1.5
35 to 150 years	2.0	3.0	3.0	2.0

Notes: PZN is the precipitation zone number (see **Figure B-1**). The PZN adjustment factor represents the PZN Condition that the CN for the watershed should be adjusted to. The Pacific Coast Climate area reduction ratio given in **Figure B-4** should be used.

B.4.2. Eastern (Desert) Watershed

A 6-hour storm of Distribution C shall be used for flood flow computations. The Arid and Semi-arid Climate area reduction given in **Figure B-4** should be used.



REFERENCE:
U.S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION

Figure B-4. Hydrology Criteria for Design Storms

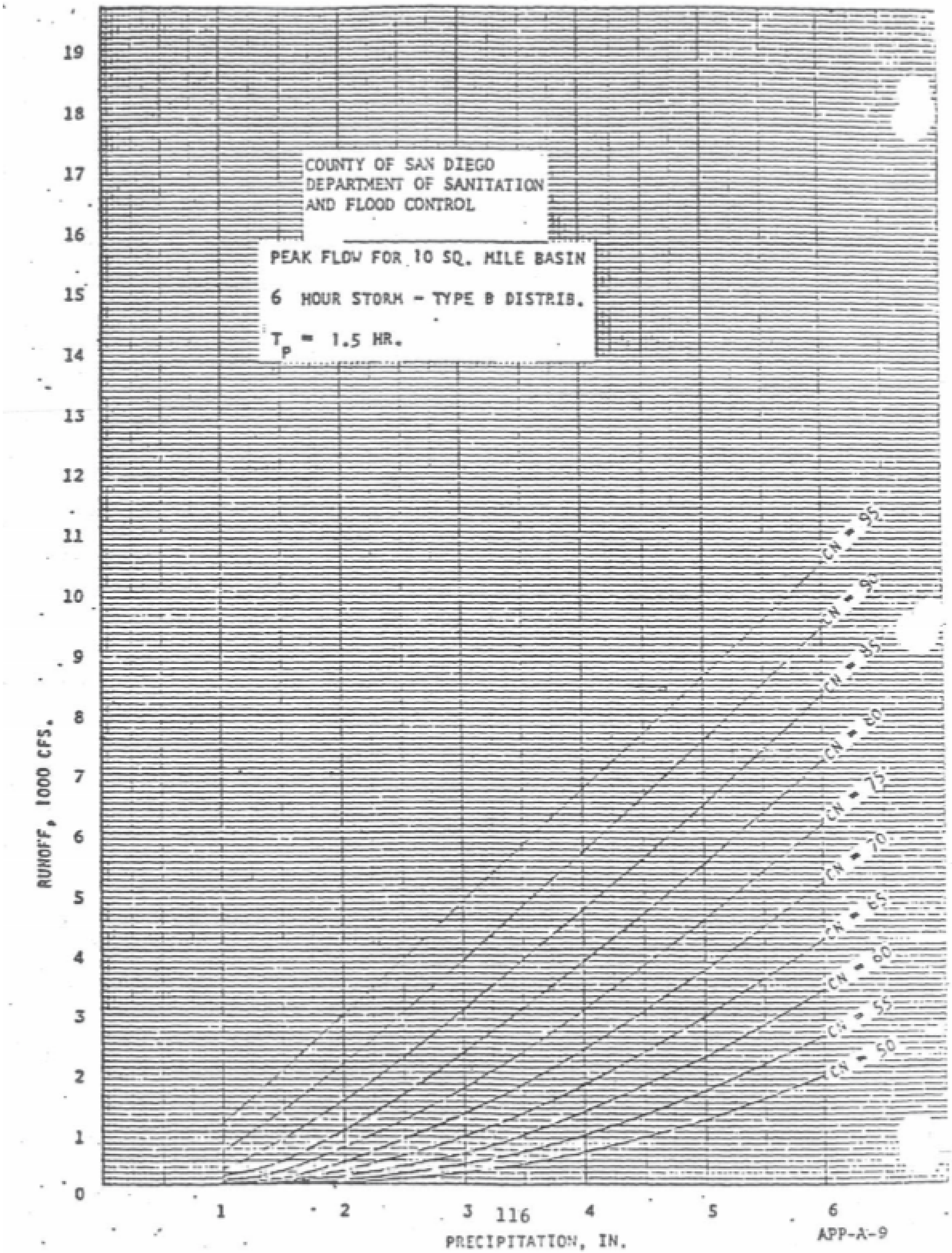


Figure B-5. 6-Hour Storm- Type B Distribution; T_p = 1.5 Hour

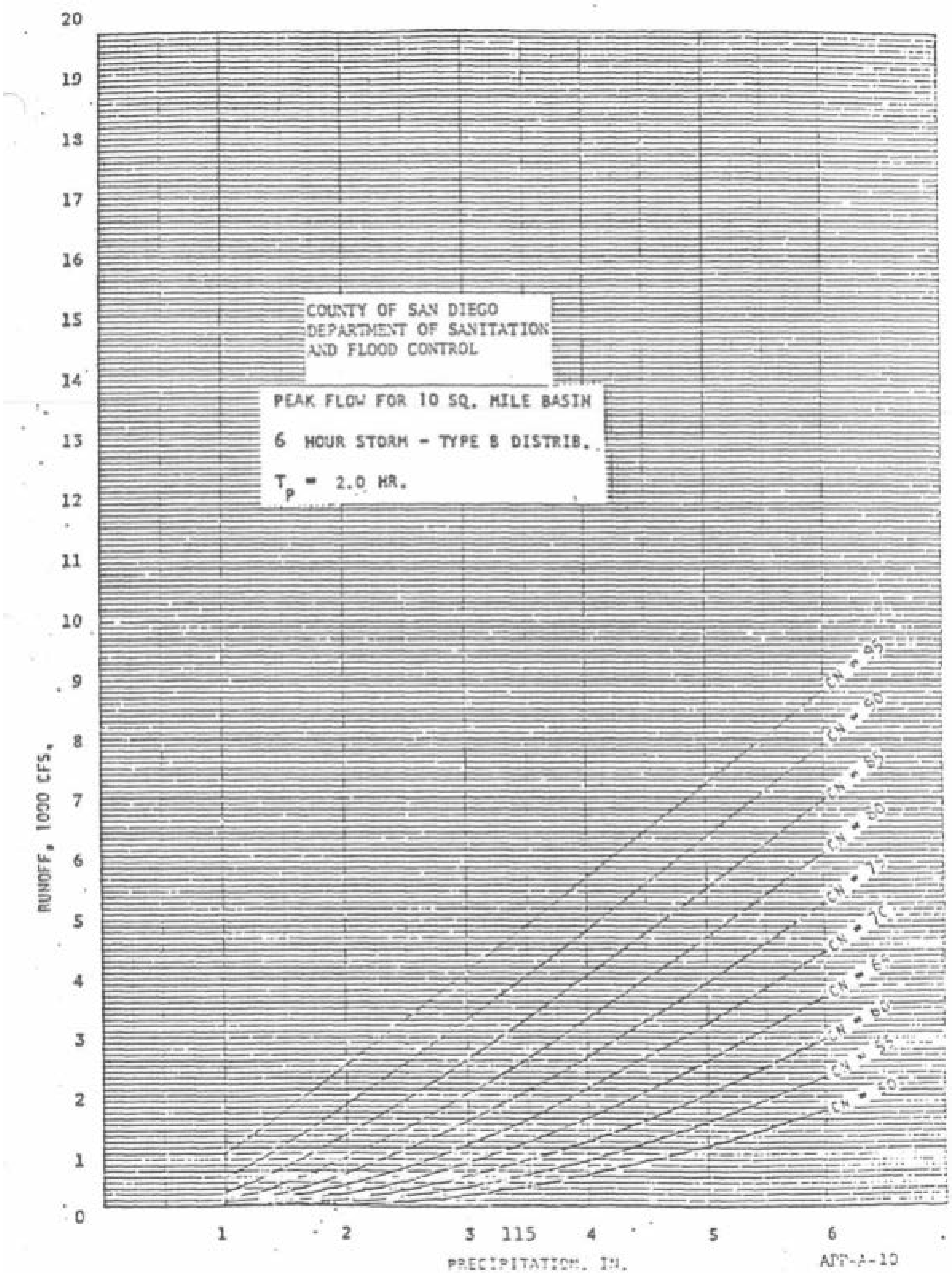


Figure B-6. 6-Hour Storm- Type B Distribution; T_p = 2.0 Hour

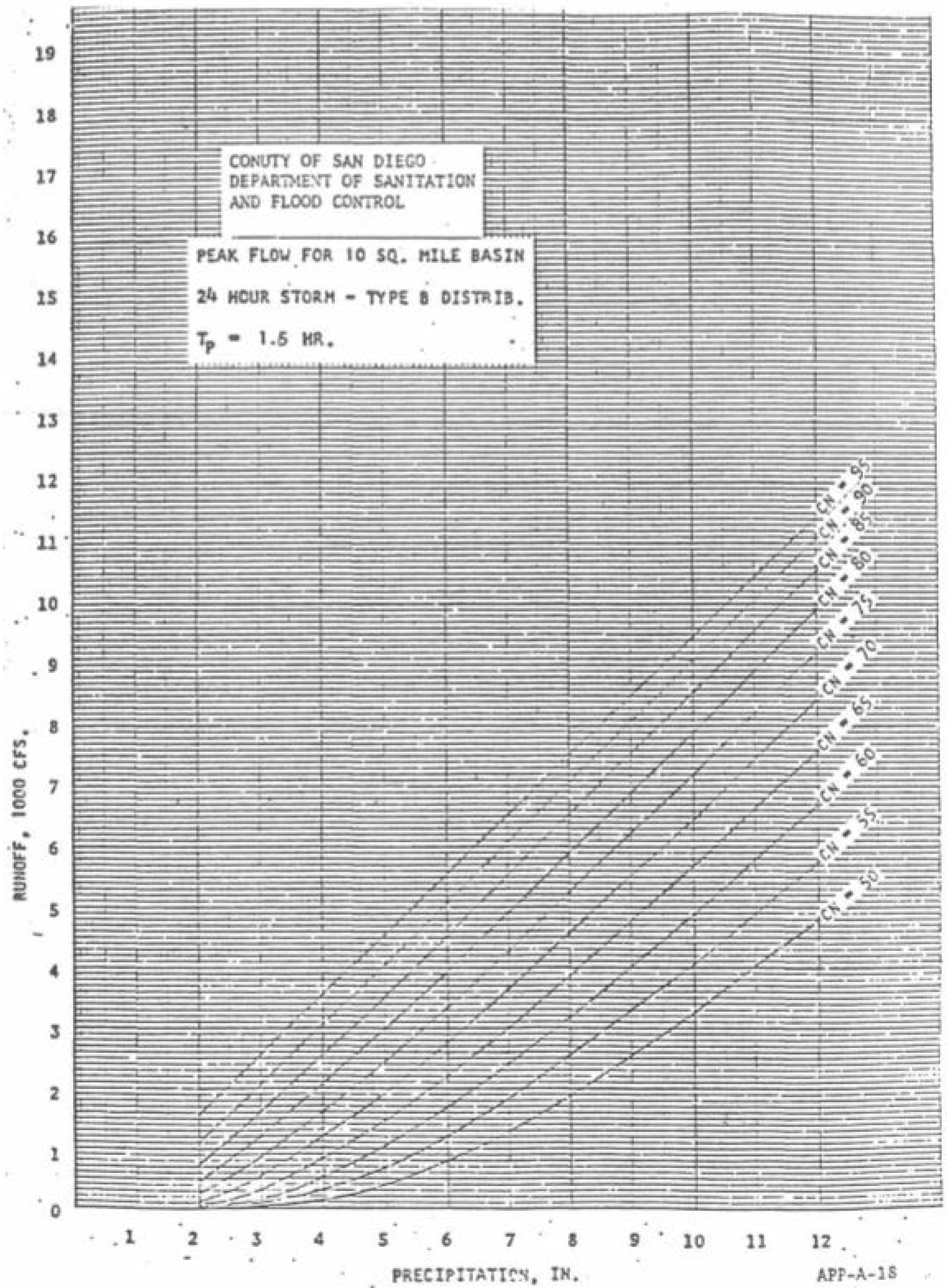


Figure B-7. 24-Hour Storm-Type B Distribution; Tp = 1.5 Hour

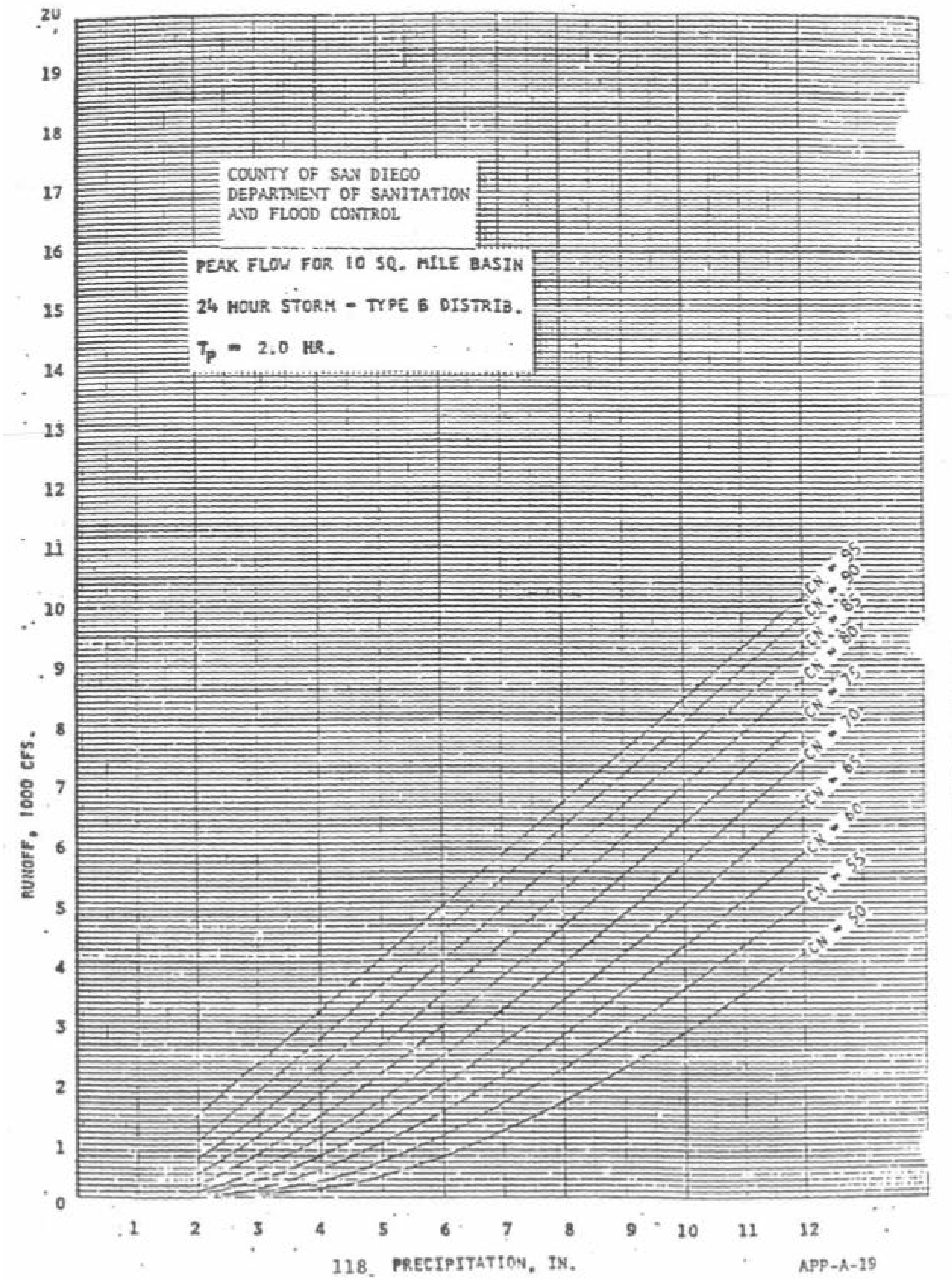


Figure B-8. 24-Hour Storm-Type B Distribution; Tp = 2.0 Hour

THIS PAGE INTENTIONALLY LEFT BLANK FOR DOUBLE-SIDED PRINTING

Appendix C

Manning Roughness Coefficients

The Manning roughness coefficient (n) is used to represent flow resistance in open-channel hydraulic computations. This Appendix offers a compilation of Manning roughness coefficients that may be used in the hydraulic design and evaluation of drainage facilities.

These values serve only as a basic guide. The procedure for selecting appropriate values for Manning roughness coefficient, especially in natural channel systems, is subjective and requires judgment and skill that is primarily developed through experience. For work where very accurate determination of water surface profile is necessary, the design engineer should consult the governing Agency to obtain data regarding roughness coefficient values applicable to specific streams. The design engineer may also examine Flood Insurance Study data, or one of several references for more specific information on determining roughness coefficient.

Table C-1. Average Manning Roughness Coefficients for Pavement and Gutters (1)

Material	Manning Roughness Coefficient (n)
Concrete Gutter (2)	0.015
Concrete Pavement	
Float Finish	0.014
Broom Finish	0.016
Concrete Gutter with Asphalt Pavement	
Smooth Finish	0.013
Rough Texture	0.015
Asphalt Pavement	
Smooth Finish	0.013
Rough Texture	0.016

Based on FHWA HEC-22.

(1) Based on materials and workmanship required by standard specifications.

(2) Increase roughness coefficient in gutters with mild slopes where sediment might accumulate by 0.020.

Table C-2. Average Manning Roughness Coefficients for Closed Conduits [1]

Conduit	Manning Roughness Coefficient (n)
Reinforced Concrete Pipe (RCP)	0.013
Corrugated Metal Pipe and Pipe Arch	
2-3/8 x ½ inch Corrugations	
Unlined	0.024
Half Lined	
Full Flow	0.018
d/D>=0.60	0.016
d/D<0.60	0.013
Fully Lined	0.013
3x1 inch Corrugations	0.027
6x2 inch Corrugations	0.032
Spiral Rib Pipe	0.013
Helically Wound Pipe	
18-inch	0.015
24-inch	0.017
30-inch	0.019
36-inch	0.021
42-inch	0.022
48-inch	0.023
Plastic Pipe (HPDE and PVC)	
Smooth	0.013
Corrugated	0.024
Vitrified Clay Pipe	0.014
Cast-Iron Pipe (Uncoated)	0.013
Steel Pipe	0.011
Brick	0.017
Cast-In-Place Concrete Pipe	
Rough Wood Forms	0.017
Smooth Wood or Steel Forms	0.014

[1] Based on materials and workmanship required by standard specifications.

Table C-3. Average Manning Roughness Coefficients for Small Open Channels Conveying Less than 50 cfs[1]

Lining Type	Design Flow Depth		
	0 – 0.5 ft	0.5 – 2.0 ft	> 2.0 ft
Concrete (Poured)	0.015	0.013	0.013
Air Blown Concrete	0.023	0.019	0.016
Grouted Riprap	0.040	0.030	0.028
Stone Masonry	0.042	0.032	0.030
Soil Cement	0.025	0.022	0.020
Bare Soil	0.023	0.020	0.020
Rock Cut	0.045	0.035	0.025
Rock Riprap	Based on Rock Size (See Chapter 7, Section 7.6.17)		

(1) Based on materials and workmanship required by standard specifications.

Table C-4. Average Manning Roughness Coefficients for Larger Open Channels

Channel	Manning Roughness Coefficient(n)
Unlined Channels Clay Loam Sand	 0.023 0.020
Lined Channels Grass Lined (well maintained) Grass Lined (not maintained)	 0.035 0.045
Wetland-Bottom Channels (New Channel)	0.023
Wetland-Bottom Channels (Mature Channel)	See Table A–5
Riprap-Lined Channels	See Chapter 7, Section 7.6.17
Concrete (Poured)	0.014
Air Blown Mortar (Gunite or Shotcrete)[1]	0.016
Asphaltic Concrete or Bituminous Plant Mix	0.018

(1) For air blown concrete, use n=0.012 (if troweled) and n=0.025 if purposely roughened.
Note: For channels with revetments or multiple lining types, use composite Manning roughness coefficient based on component lining materials.

Table C-5. Average Manning Roughness Coefficients for Natural Channels

Channel	Manning Roughness Coefficient (n)
Minor Streams (Surface Width at Flood Stage < 100 ft)	
Fairly Regular Section	
(A) Some Grass and Weeds, Little or No Brush	0.030
(B) Dense Growth of Weeds, Depth of Flow Materially Greater than Weed Height	0.040
(C) Some Weeds, Light Brush on Banks	0.040
(D) Some Weeds, Heavy Brush on Banks	0.060
(E) For Trees within Channel with Branches Submerged at High Stage, Increase all above values by:	0.015
Irregular Section, with Pools, Slight Channel Meander	
Channels (A) through (E) above, Increase all Values by:	0.015
Mountain Streams; No Vegetation in Channel, Banks Usually Steep, Trees and Brush along Banks Submerged at High Stage	
(A) Bottom, Gravel, Cobbles and Few Boulders	0.050
(B) Bottom, Cobbles with Large Boulders	0.060
Flood Plains (Adjacent to Natural Streams)	
Pasture, No Brush	
(A) Short Grass	0.030
(B) High Grass	0.040
Cultivated Areas	
(A) No Crop	0.040
(B) Mature Row Crops	0.040
(C) Mature Field Crops	0.050
Heavy Weeds, Scattered Brush	0.050
Light Brush and Trees	0.060
Medium-to-Dense Brush	0.090
Dense Willows	0.170
Cleared Land with Tree Stumps, 100-150 per Acre	0.060
Heavy Stand of Timber, Little Undergrowth	
(A) Flood Depth below Branches	0.110
(B) Flood Depth Reaches Branches	0.140

Appendix D

RCP Design Criteria and Specifications

D.1. General Criteria

D-load values given in the table indicate greater accuracy than warranted in field installation. Thus, when specifying, pipe should be classified in 50-D increment, for example. 800-D, 850-D.

D.2. Bedding

Table D-1 is based on installations with ordinary bedding (Spangler and Handy, 1973) and should not be used for other conditions, except as noted. D-loads given in the table are based on a load factor of 1.50. For classes of bedding with load factors other than 1.50, the corrected dead load may be obtained by multiplying the table's dead load by 1.50 and dividing by the desired dead load factor.

D.3. Backfill

Based on Marston's curve for saturated topsoil, when $K\mu$ =0.150 (Ameron, 1973). The table is conservative for sands, gravels, and cohesionless materials. The D-load should be recomputed for clay backfills, when $K\mu$ >0.150, using the correct coefficient. The table has been computed using materials with a unit weight of 110 pounds per cubic foot. For materials having a unit weight other than 110 pounds per cubic foot, the correct dead load can be calculated by multiplying the dead load shown in the table by the desired unit weight and dividing by 110.

D.4. Trench Width

D-loads given in the table are based on trench widths (at top of pipe) of pipe OD plus 16 inches for pipe diameters 33 inches or less; and pipe OD plus 24 inches for pipe diameters greater than 33 inches. Pipe ODs are based on wall thicknesses given in the dimensional data table for Wall A pipe through 96-inch diameter, and on wall thicknesses given in table for large diameter pipe with 102- and 108-inch diameters. Thicker wall designs may require a slightly higher D-load classification.

For earth covers of two to eight feet, the tabulated dead load D-loads approach the maximum loads that occur at the transition trench width. The difference in dead load for wider trench widths or the projecting conduit conditions may be a small value, and the pipe may safety withstand the increase. For assurance, it will be necessary to recompute the D-loads for any installation change at any depth of cover.

D.5. Safety Factor

A safety factor of 1.0 against the occurrence of the 0.01-inch crack is assumed in the calculations. If a factor different than 1.0 is desired, corrected D-loads can be obtained by multiplying loads shown in the table by the desired safety factor.

D.6. Live Load

Live load distribution is calculated from AASHTO HS-20 for truckloads (American Association of State Highway and Transportation Officials [AASHTO], 1973). For different wheel loadings, correct live loads can be obtained by multiplying live loads shown in the table by the desired maximum wheel load in kips and dividing by 16. This table is limited to AASHTO live load distributions (a square at backfill depth, H, whose sides equal 1.75H) for single truck loading with impact factors based on depth. A live load factor of 1.50, recommended in Iowa State College Bulletin 112 by Spangler for ordinary bedding or better, is used. For covers nine feet and greater, live loads are included in the indicated D-loads.

Table D-1. Maximum Height of Cover for RCP

COVER (ft)	RCP DIAMETER	12"	15"	18"	21"	24"	27"	30"	33"	36"	39"	42"	45"	48"	51"	54"	COVER (ft)
2.0	Dead Load	350	309	286	267	254	243	234	227	255	250	244	238	234	230	226	2.0
	Live Load	1393	1323	1306	1281	1262	1248	1236	1226	1219	1125	1044	975	914	860	812	
	Total Load	1743	1632	1592	1549	1516	1491	1471	1454	1474	1375	1289	1213	1148	1090	1039	
2.5	Dead Load	426	377	349	327	311	298	288	280	315	308	301	295	289	284	280	2.5
	Live Load	817	776	766	751	740	732	725	719	719	723	722	715	670	631	595	
	Total Load	1243	1153	1115	1079	1052	1031	1014	1000	1034	1031	1024	1010	960	915	876	
3.0	Dead Load	497	441	410	385	366	352	340	331	373	365	358	349	344	337	333	3.0
	Live Load	515	490	483	474	467	462	457	454	454	456	456	453	453	451	451	
	Total Load	1013	931	893	859	834	814	798	785	827	822	814	803	797	789	784	
4.0	Dead Load	629	560	523	493	470	453	439	427	485	475	466	456	448	441	435	4.0
	Live Load	312	296	293	287	283	280	277	275	275	276	276	274	274	273	273	
	Total Load	942	857	816	780	753	733	716	703	760	752	742	730	723	714	709	
5.0	Dead Load	747	668	626	592	566	546	531	518	590	580	569	557	549	539	533	5.0
	Live Load	220	209	206	202	199	197	195	194	193	194	194	193	193	192	192	
	Total Load	968	878	832	794	766	744	726	712	784	775	763	751	742	732	726	
6.0	Dead Load	853	765	720	682	655	633	616	603	690	679	667	654	644	634	627	6.0
	Live Load	164	155	153	150	148	147	145	144	144	145	145	144	144	143	143	
	Total Load	1017	921	874	833	803	780	762	747	835	824	812	798	789	778	771	
7.0	Dead Load	947	853	805	766	736	714	696	682	785	773	760	746	736	725	717	7.0
	Live Load	127	120	119	117	115	113	112	112	111	112	112	111	111	111	111	
	Total Load	1074	974	924	883	852	828	809	794	897	886	872	858	848	836	829	
8.0	Dead Load	1031	932	883	842	812	789	771	756	875	863	849	834	824	812	804	8.0
	Live Load	101	96	95	93	92	91	90	89	89	89	89	89	89	88	88	
	Total Load	1132	1029	978	935	904	880	861	846	964	953	939	923	913	901	893	
9.0	Total Load	1194	1087	1036	993	961	937	918	903	1037	1025	1011	995	985	972	964	9.0
10.0	Total Load	1250	1141	1090	1047	1015	992	973	959	1108	1096	1082	1065	1055	1042	1034	10.0
11.0	Total Load	1301	1191	1141	1098	1067	1043	1026	1012	1177	1165	1151	1134	1123	1110	1103	11.0
12.0	Total Load	1347	1236	1187	1145	1115	1092	1075	1062	1242	1231	1217	1200	1190	1177	1170	12.0
14.0	Total Load	1426	1315	1269	1229	1201	1181	1166	1155	1365	1356	1343	1326	1317	1304	1297	14.0
16.0	Total Load	1490	1380	1338	1301	1276	1259	1247	1239	1477	1470	1458	1442	1434	1422	1417	16.0
18.0	Total Load	1541	1433	1396	1363	1341	1327	1318	1312	1578	1574	1564	1549	1543	1532	1528	18.0
20.0	Total Load	1582	1477	1445	1415	1397	1386	1380	1378	1670	1669	1661	1647	1643	1633	1631	20.0
24.0	Total Load	1642	1542	1519	1496	1485	1482	1483	1486	1828	1833	1830	1819	1820	1813	1816	24.0
28.0	Total Load	1679	1585	1570	1554	1550	1553	1560	1571	1955	1967	1969	1963	1969	1966	1973	28.0
32.0	Total Load	1703	1613	1605	1595	1597	1606	1619	1635	2058	2077	2085	2083	2094	2095	2107	32.0
36.0	Total Load	1718	1632	1629	1624	1631	1646	1664	1686	2141	2166	2180	2183	2198	2204	2220	36.0
40.0	Total Load	1727	1644	1645	1644	1656	1675	1698	1724	2208	2240	2258	2265	2286	2296	2317	40.0

Table D-2. Maximum Height of Cover for RCP (continued)

COVER (ft)	RCP DIAMETER	57"	60"	63"	66"	69"	72"	75"	78"	81"	84"	87"	90"	93"	96"	102"	108"	COVER (ft)
2.0	Dead Load	223	221	218	216	214	212	211	209	208	206	205	204	203	202	203	201	2.0
	Live Load	769	731	696	664	636	609	585	562	541	522	504	487	471	457	430	406	
	Total Load	993	952	915	881	850	822	796	772	749	729	710	692	675	659	633	607	
2.5	Dead Load	277	273	271	268	266	263	261	259	258	256	255	253	252	251	252	250	2.5
	Live Load	564	536	510	487	466	446	429	412	397	383	369	357	346	335	315	297	
	Total Load	841	810	781	756	732	710	690	672	655	639	625	611	598	586	568	548	
3.0	Dead Load	329	325	322	319	316	314	311	309	307	305	304	302	300	299	401	298	3.0
	Live Load	427	406	386	369	353	338	325	312	300	290	280	270	262	253	239	225	
	Total Load	756	731	709	688	669	652	636	622	606	596	584	573	563	553	540	524	
4.0	Dead Load	430	426	421	418	414	411	409	406	404	401	399	397	395	394	396	393	4.0
	Live Load	273	273	273	273	273	273	262	252	243	234	226	218	211	205	193	182	
	Total Load	704	699	695	691	688	685	671	658	647	636	626	616	607	599	589	576	
5.0	Dead Load	527	522	518	513	510	506	503	500	497	495	492	490	488	486	490	486	5.0
	Live Load	192	192	192	192	192	192	192	192	192	192	192	192	186	180	170	160	
	Total Load	720	715	710	706	702	699	696	693	690	687	685	683	675	667	660	647	
6.0	Dead Load	621	615	610	606	601	598	594	591	588	585	583	580	578	576	581	577	6.0
	Live Load	143	143	143	143	143	143	143	143	143	143	143	143	143	143	143	143	
	Total Load	765	759	753	749	745	741	738	735	732	729	726	724	722	719	727	720	
7.0	Dead Load	711	705	700	695	690	686	682	679	676	673	670	668	665	663	669	665	7.0
	Live Load	111	111	111	111	111	111	111	111	111	111	111	111	111	111	113	113	
	Total Load	822	816	811	806	802	796	794	790	787	784	782	779	777	775	782	778	
8.0	Dead Load	797	791	786	781	776	772	769	765	761	758	756	753	751	748	756	752	8.0
	Live Load	88	88	88	88	88	88	88	88	88	88	88	88	88	88	90	90	
	Total Load	886	880	874	869	865	861	857	853	850	847	845	842	840	837	846	842	
9.0	Total Load	957	951	945	940	936	932	928	924	921	918	915	913	911	908	918	914	9.0
10.0	Total Load	1027	1021	1016	1011	1006	1002	996	995	992	989	986	984	982	979	991	987	10.0
11.0	Total Load	1096	1090	1085	1080	1076	1072	1068	1065	1062	1059	1057	1054	1052	1050	1063	1059	11.0
12.0	Total Load	1163	1157	1152	1148	1144	1140	1137	1134	1131	1128	1126	1124	1122	1120	1135	1131	12.0
14.0	Total Load	1292	1287	1283	1279	1275	1272	1270	1267	1265	1263	1262	1260	1258	1257	1275	1272	14.0
16.0	Total Load	1412	1409	1405	1403	1400	1396	1397	1395	1394	1393	1392	1391	1390	1389	1411	1409	16.0
18.0	Total Load	1525	1523	1521	1519	1518	1517	1517	1516	1516	1516	1516	1516	1516	1516	1542	1542	18.0
20.0	Total Load	1630	1629	1629	1629	1629	1630	1631	1631	1632	1633	1634	1635	1636	1637	1668	1669	20.0
24.0	Total Load	1818	1821	1825	1828	1832	1835	1839	1842	1846	1850	1853	1857	1860	1863	1903	1908	24.0
28.0	Total Load	1980	1987	1994	2002	2009	2016	2032	2030	2037	2043	2050	2056	2062	2068	2118	2126	28.0
32.0	Total Load	2118	2130	2142	2153	2164	2175	2186	2196	2206	2218	2226	2235	2244	2253	2313	2326	32.0
36.0	Total Load	2237	2253	2269	2285	2300	2315	2329	2343	2357	2371	2383	2396	2408	2420	2490	2509	36.0
40.0	Total Load	2338	2359	2379	2399	2418	2437	2456	2474	2491	2508	2524	2540	2556	2571	2651	2675	40.0

Table D-3. Deflection and Curvature for Straight RCP

Pipe Size	Wall Thickness	Outside Diameter	Bell Depth	Maximum Joint Opening	Minimum Radius*	Maximum Deflections
(inch)	(inch)	(inch)	(inch)	(inch)	(feet)	(degrees)
12	2	16	11/16	5/8	206	2.23
15	2	19	1	3/4	204	2.27
18	2-1/4	22-1/2	15/16	3/4	242	1.92
21	2-3/8	25-3/4	1-5/8	3/4	277	1.67
24	2-1/2	29	1-5/8	3/4	311	1.48
27	2-5/8	32-1/4	1-5/8	3/4	346	1.33
30	2-3/4	35-1/2	1-3/4	3/4	381	1.22
33	2-7/8	38-3/4	1-3/4	3/4	417	1.12
36	3-1/8	42-1/4	1-3/4	3/4	454	1.02
39	3-1/2	46	1-7/8	1	370	1.25
42	3-3/4	49-1/2	1-7/8	1	398	1.17
45	3-7/8	52-3/4	1-7/8	1	424	1.08
48	4-1/8	56-1/4	2	1	453	1.02
51	4-1/4	59-1/2	2	1	478	0.97
54	4-1/2	63	2	1	507	0.92
57	4-3/4	66-3/5	2	1	535	0.87
60	5	70	2	1	563	0.82
63	5-1/4	73-1/2	2	1	591	0.78
66	5-1/2	77	2	1	619	0.75
69	5-3/4	80-1/2	2	1	647	0.72
72	6	84	2-5/8	1	675	0.68
75	6-1/4	87-1/2	2-5/8	1	704	0.65
78	6-1/2	91	2-5/8	1	731	0.62
81	6-3/4	94-1/2	2-5/8	1	760	0.58
84	7	98	2-3/4	1	788	0.57
87	7-1/4	101-1/2	3	1	816	0.55
90	7-1/2	105	3	1	844	0.53
93	7-3/4	108-1/2	3	1	872	0.52
96	8	112	3	1	900	0.50

* Radius based upon 8-foot RCP segments; scale proportionally for other segment lengths.

Table D-4. Radius of Curvature for Beveled RCP

Pipe Size(in)	Minimum Radius of Curvature (feet)			
	8-ft Lengths 5 Bevel One End	8-ft Lengths 5 Bevel Both Ends	4-ft Lengths 5 Bevel One End	4-ft Lengths 5 Bevel Both Ends
12	91.5	45.5	45.0	22.5
15	91.0	45.0	45.0	22.5
18	91.0	45.0	44.5	22.5
21	91.0	45.0	44.0	22.0
24	91.0	45.0	44.0	22.0
27	90.5	44.5	44.0	22.0
30	90.5	44.5	44.0	22.0
33	90.5	44.5	43.5	22.0
36	90.0	44.5	43.5	22.0
39	90.0	44.0	43.5	22.0
42	90.0	44.0	43.0	21.5
45	89.5	44.0	43.0	21.5
48	89.5	43.5	42.0	21.0
51	87.5	41.5	41.0	20.5
54	87.5	41.5	40.5	20.5
57	89.0	43.0	42.5	21.5
60	87.0	40.0	39.5	20.0
63	87.0	40.5	40.5	20.5
66	86.5	41.0	40.0	20.0
69	86.5	41.0	40.0	20.0
72	85.5	39.5	39.0	19.5
75	86.0	40.5	39.5	20.0
78	86.0	40.5	40.0	20.0
81	86.5	40.0	39.5	20.0

Table D-5. Anticipated Additional Service Life for Steel and Aluminum Pipes with Bituminous Coatings

Environment				Bituminous Coating	Bituminous Coating and Paved Invert	Asbestos Bonded Bituminous Coating and Paved Invert
Location	Probable Channel Slope	Probable Flow Velocity of Q_{10}	Channel Material[1]			
Valley	< 2%	< 5 ft/s	Abrasive	6	15	20
			Non-Abrasive	8	15	20
Foothill	~ 3%	5-7 ft/s	Abrasive	6	12	20
			Non-Abrasive	8	15	20
Mountains	> 4%	> 7 ft/s	Abrasive	0	5	8
			Non-Abrasive	2	10	20

Notes:

1. Channel Materials. If there is no existing culvert, it may be assumed that the channel is potentially abrasive to culvert if sand and/or rocks are present. Presence of silt, clay, or heavy vegetation may indicate a non-abrasive flow. For continuous flow, the years of invert protection can be expected to be one-half of that shown.

2. Any bituminous coating may add up to 25 years of service on the backfill side of the culvert.

THIS PAGE INTENTIONALLY LEFT BLANK FOR DOUBLE-SIDED PRINTING

Appendix E

Culvert Design Nomographs

This Appendix contains selected design nomographs from the Federal Highway Administration's Hydraulic Design Series No. 5, Hydraulic Design of Highway Culverts (2001). The design engineer should refer to HDS-5 for additional design nomographs and guidance on the application of these curves to culvert design.

The nomographs that follow are organized in the following categories:

1. **Circular Culvert Nomographs**: Figures E-1 through E-7 (7 figures)
2. **Rectangular Box Culvert Nomographs**: Figures E-8 through E-15 (8 figures)
3. **Corrugated Metal Box Culvert Nomographs**: Figures E-16 through E-28 (13 figures)
4. **Corrugated Metal Arch Culvert Nomographs**: Figures E-29 through E-38 (10 figures)

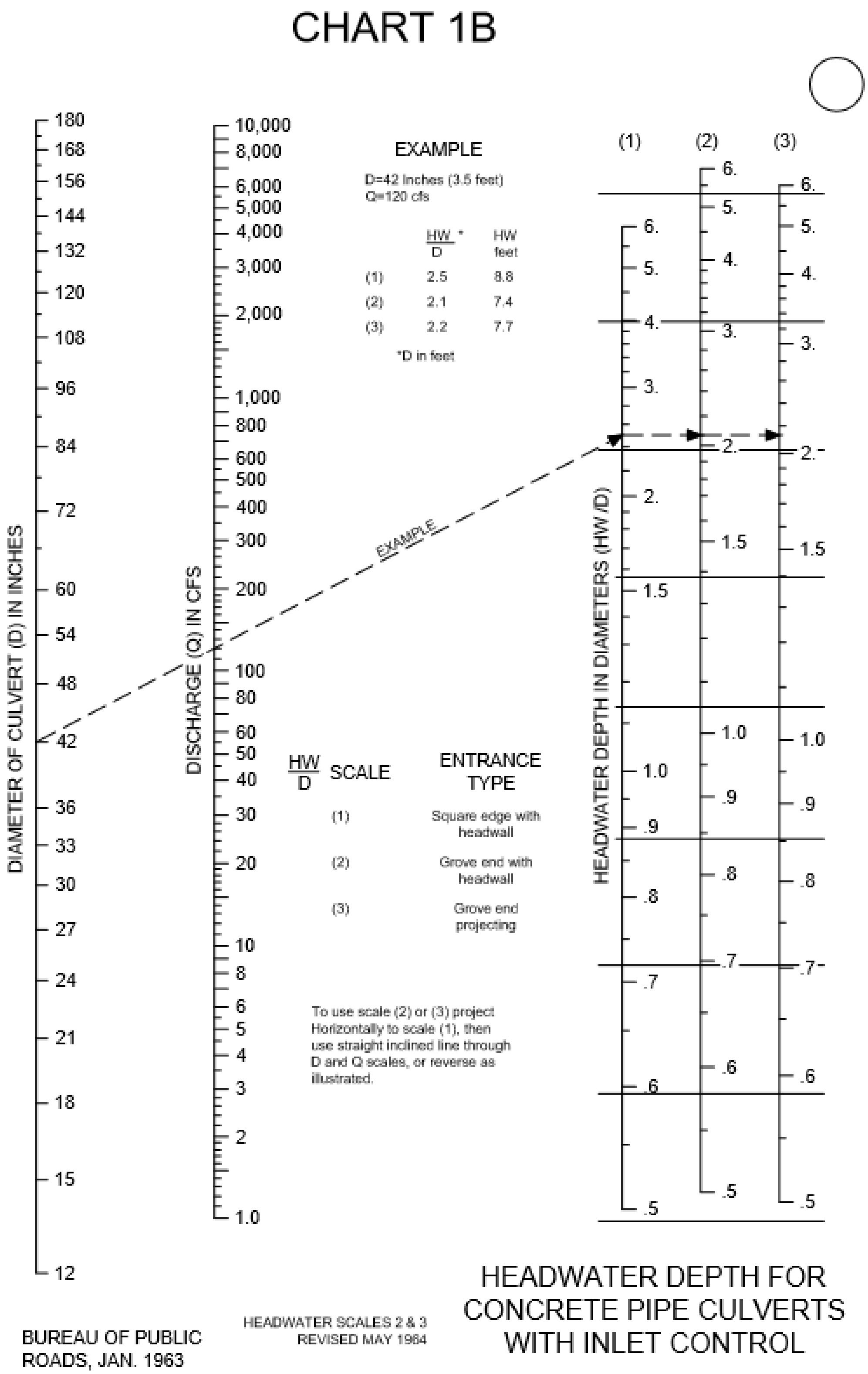


Figure E-1. Circular Culvert Nomograph (Chart 1B)

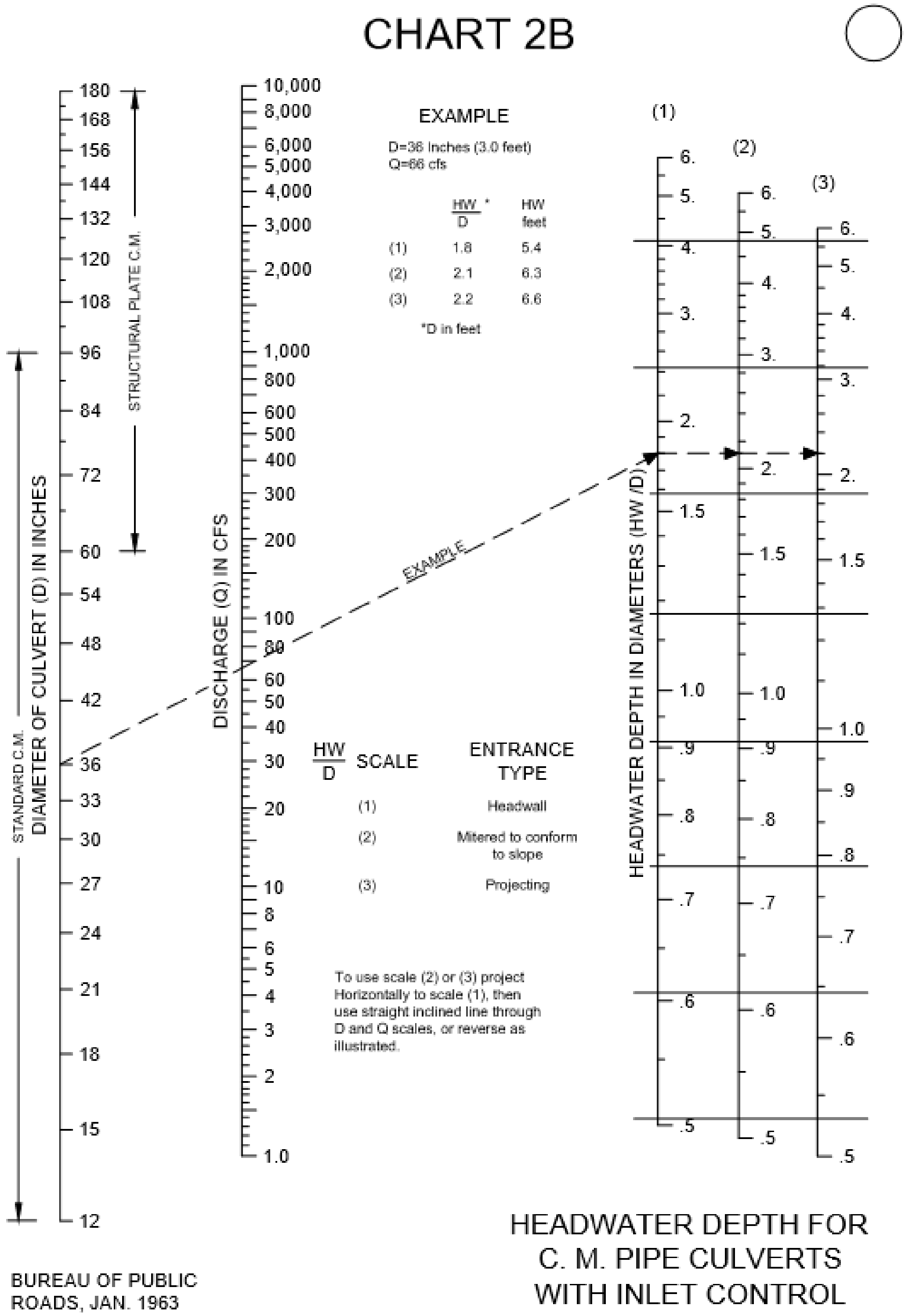


Figure E-2. Circular Culvert Nomograph (Chart 2B)

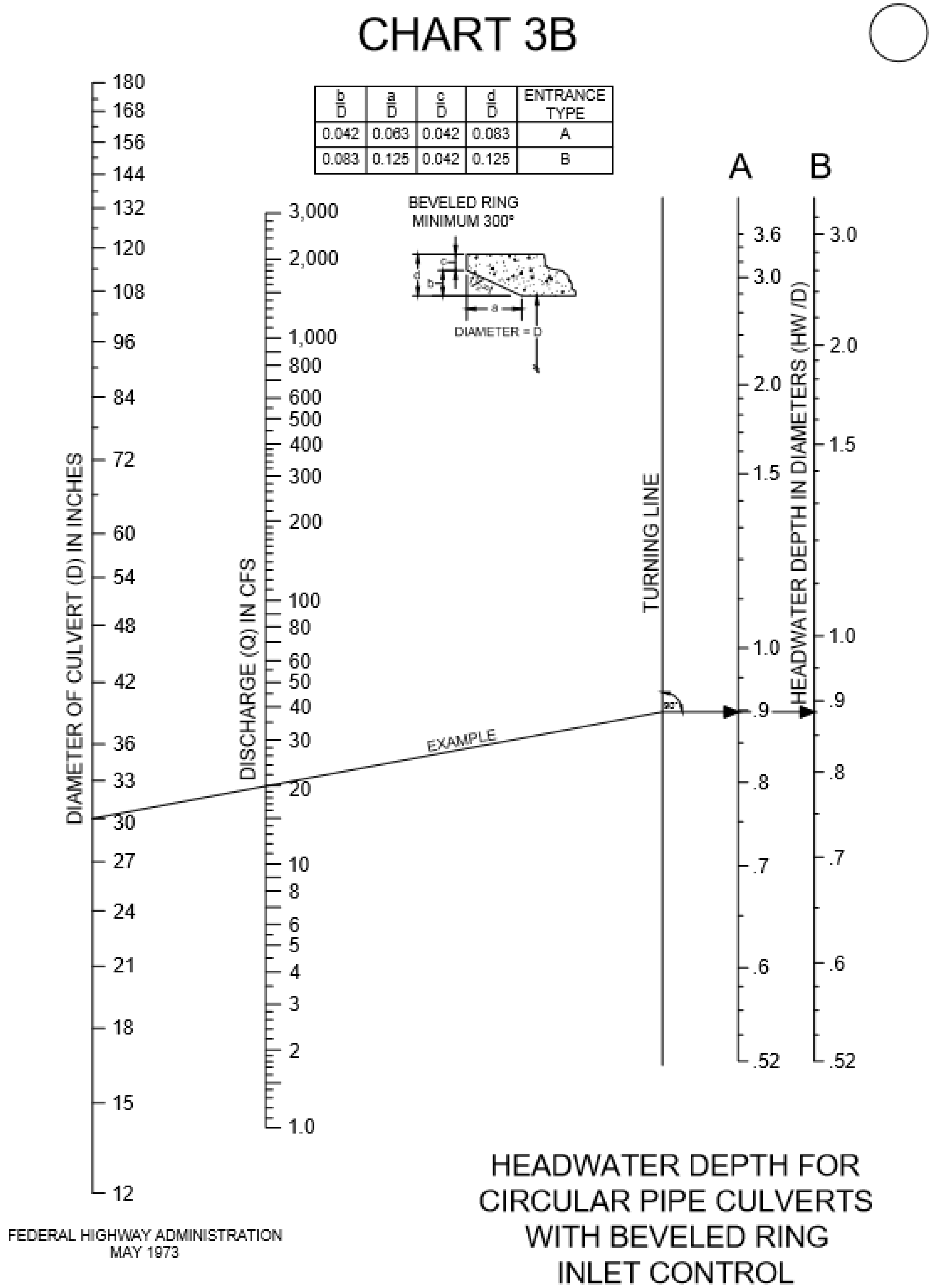


Figure E-3. Circular Culvert Nomograph (Chart 3B)

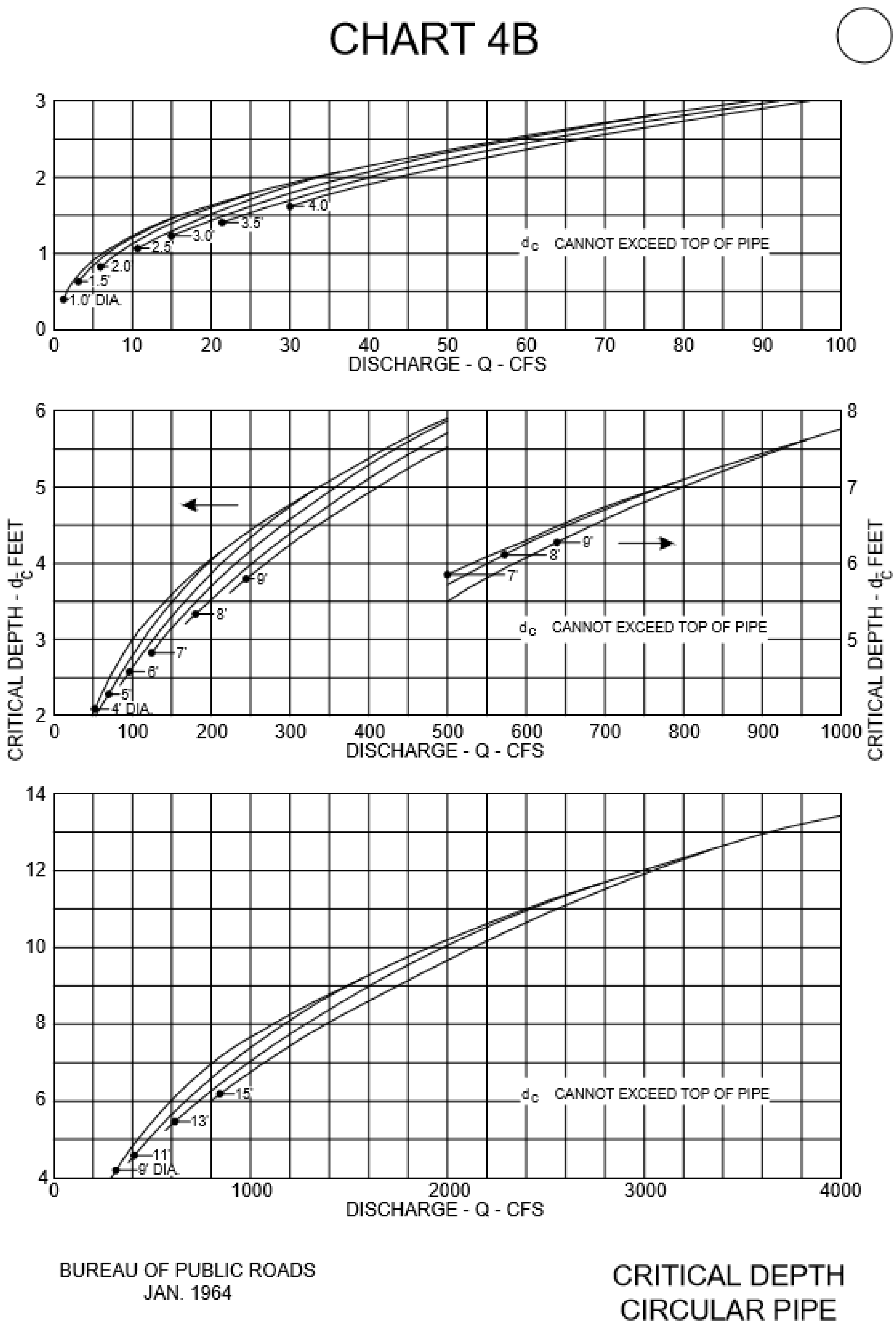


Figure E-4. Circular Culvert Nomograph (Chart 4B)

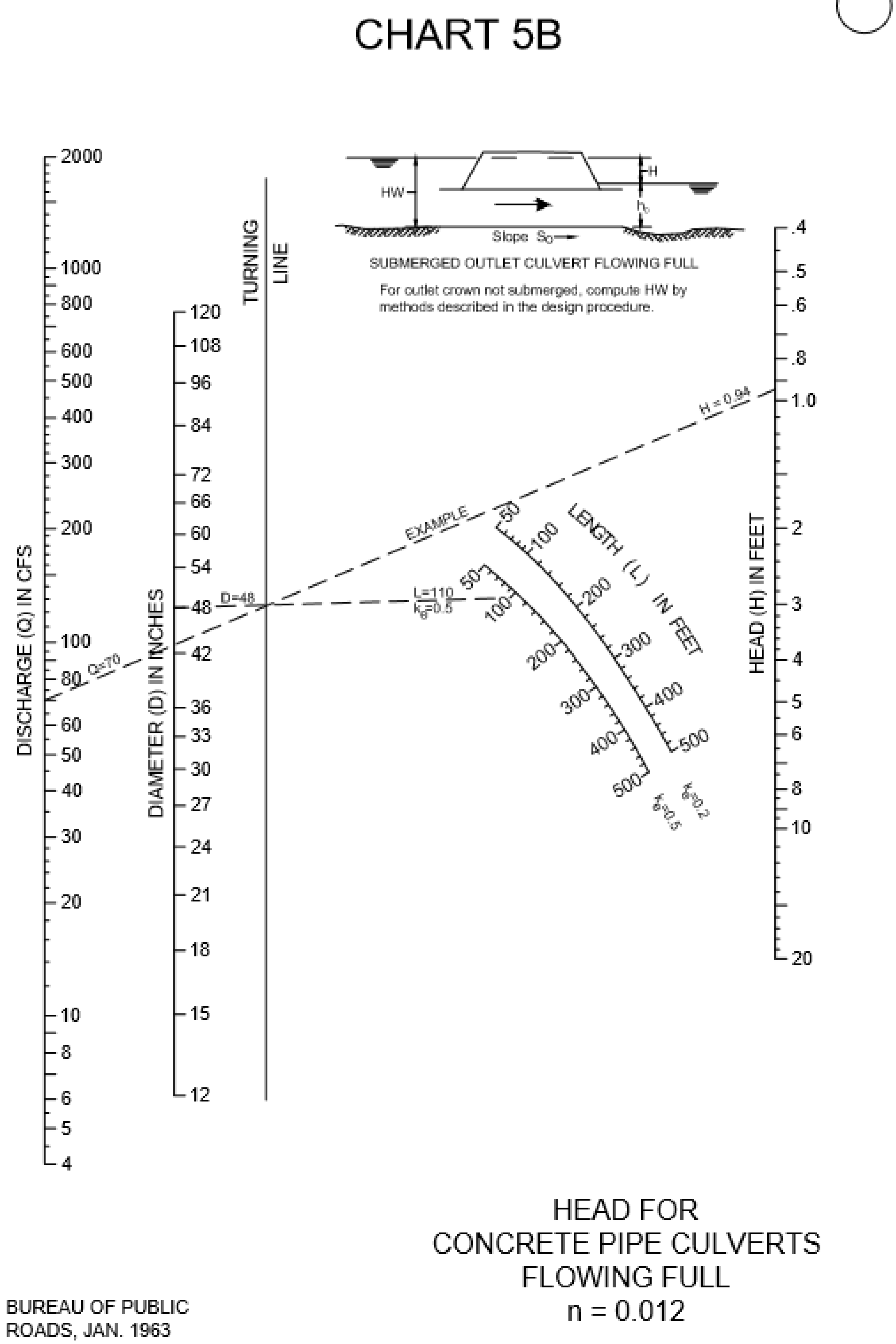


Figure E-5. Circular Culvert Nomograph (Chart 5B)

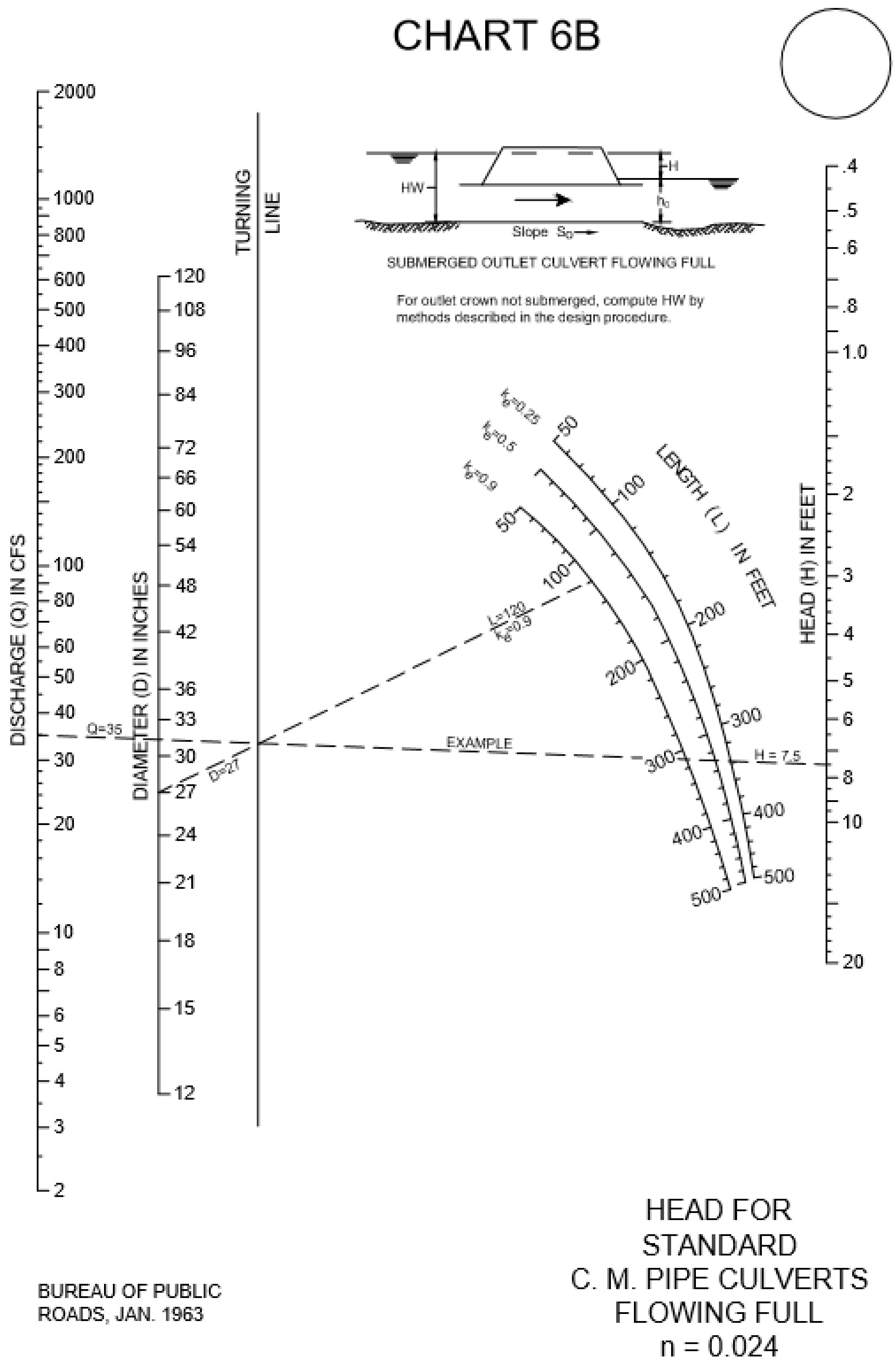


Figure E-6. Circular Culvert Nomograph (Chart 6B)

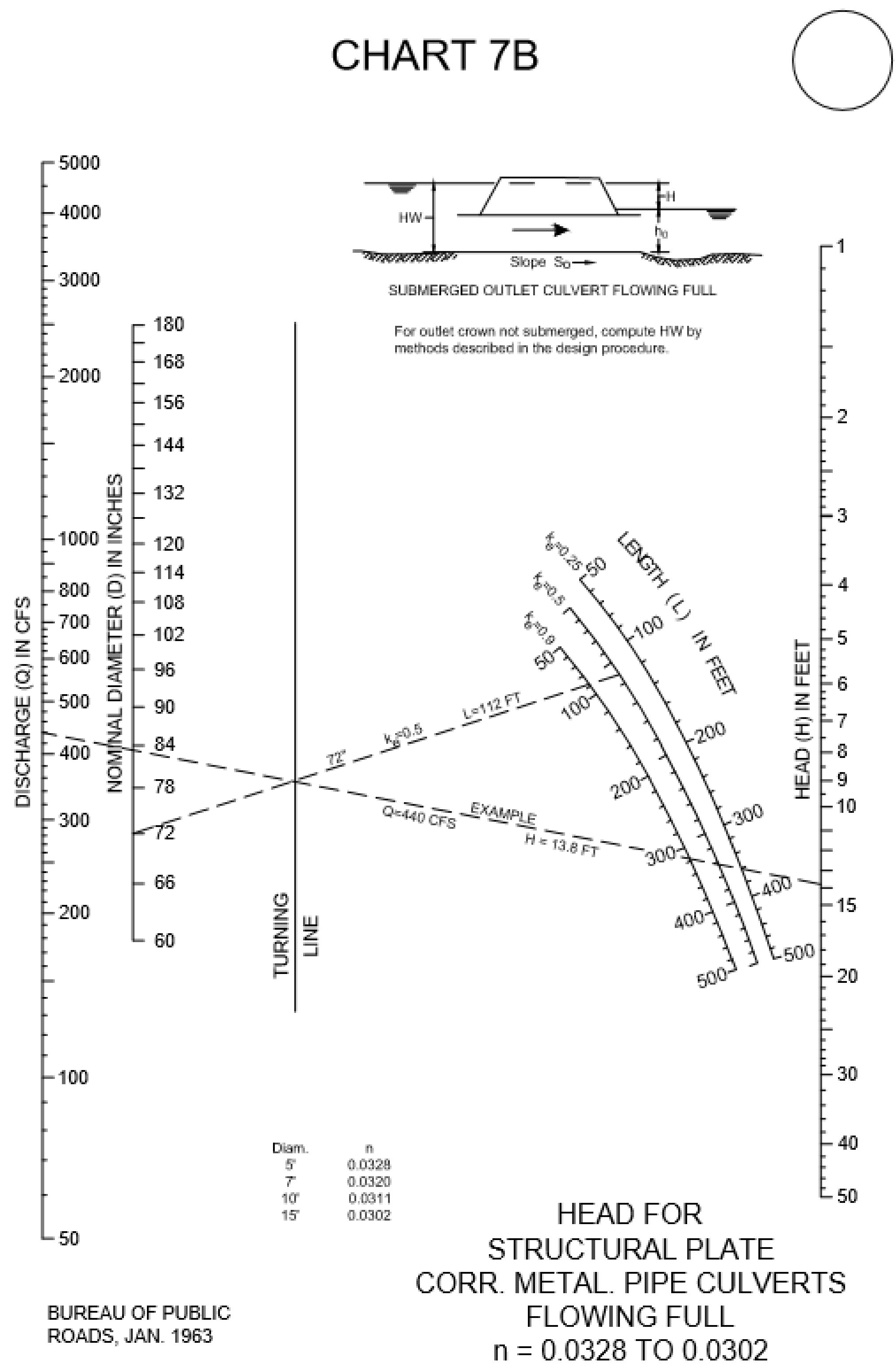


Figure E-7. Circular Culvert Nomograph (Chart 7B)

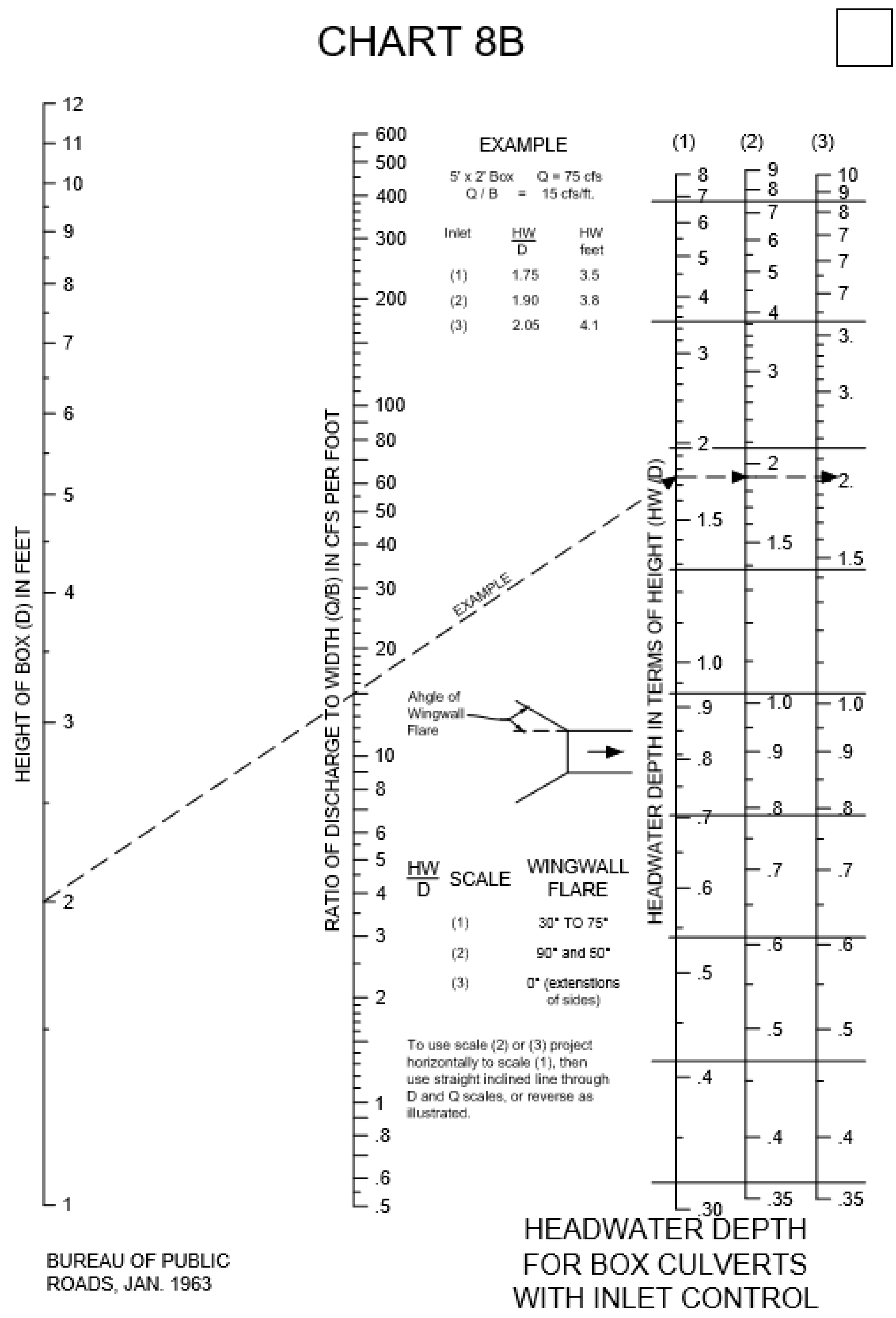


Figure E-8. Rectangular Box Culvert Nomograph (Chart 8B)

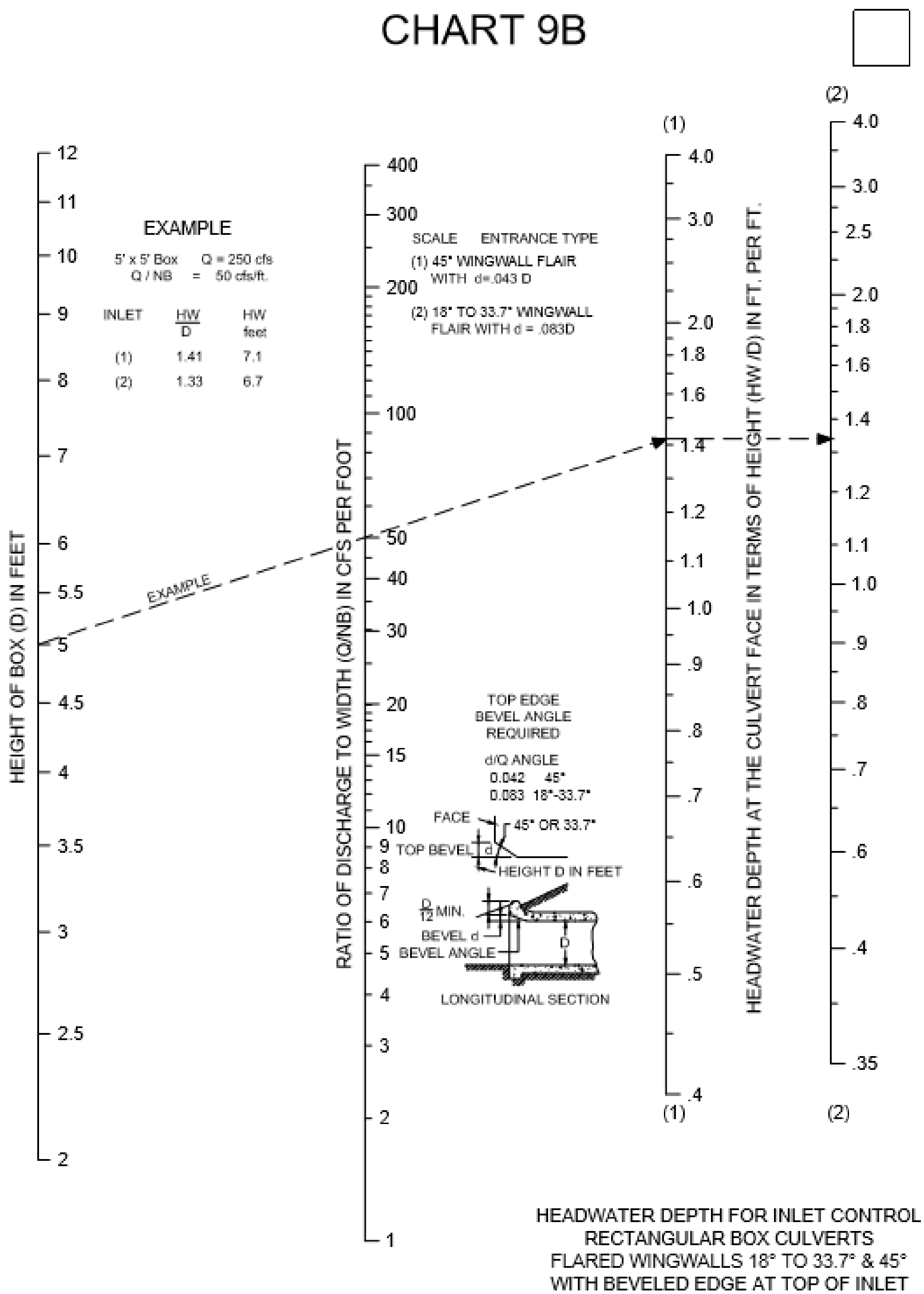


Figure E-9. Rectangular Box Culvert Nomograph (Chart 9B)

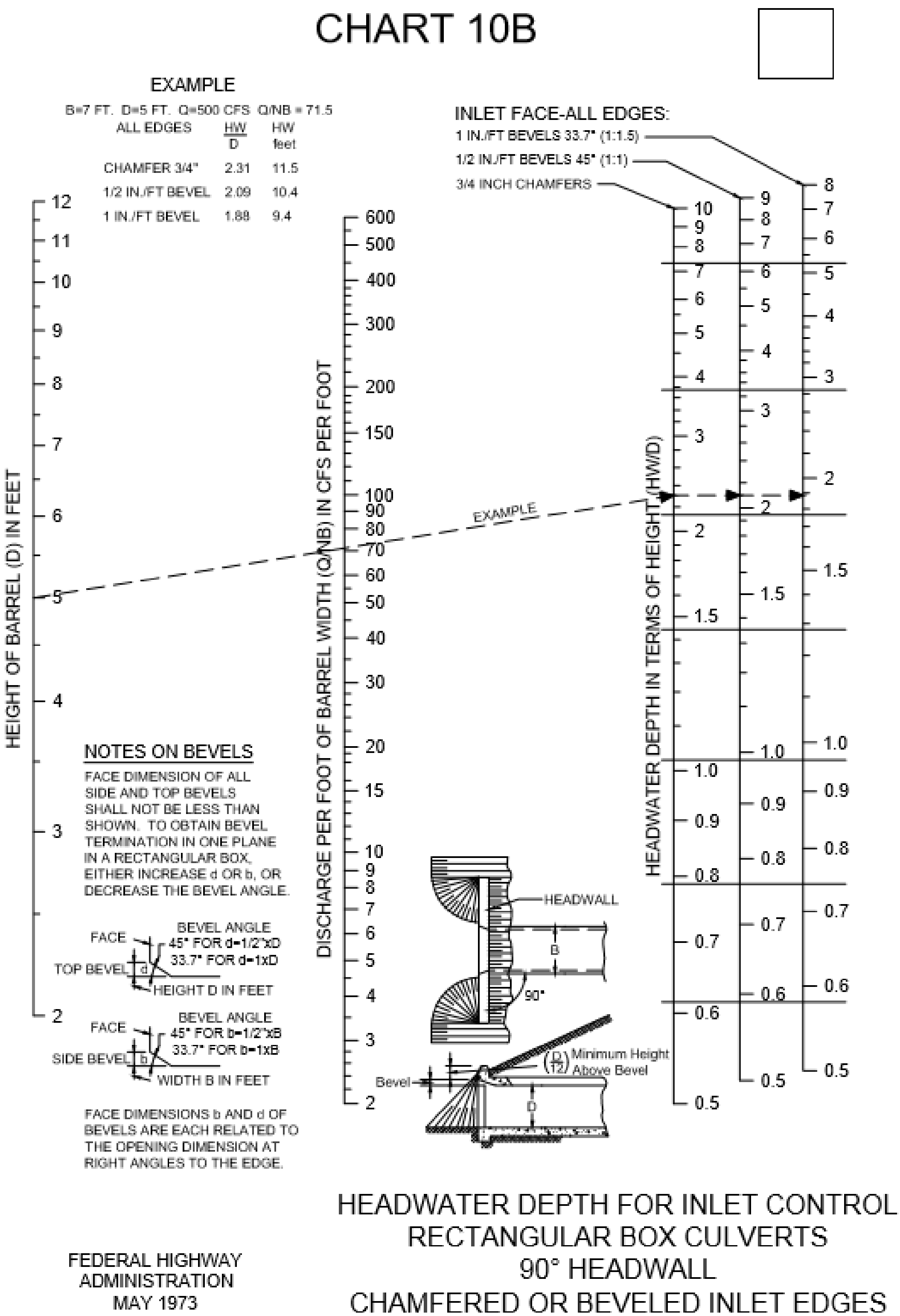


Figure E-10. Rectangular Box Culvert Nomograph (Chart 10B)

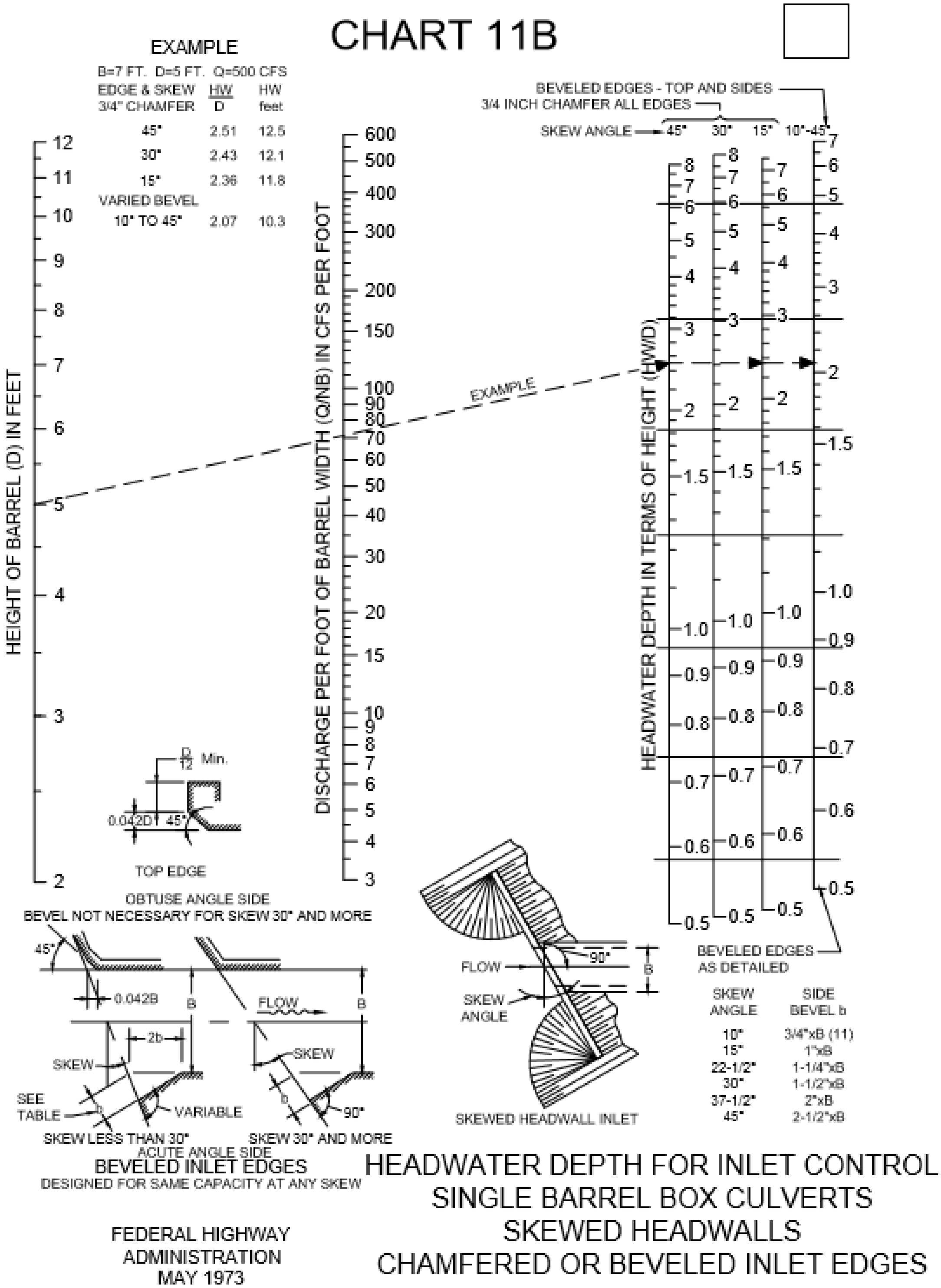


Figure E-11. Rectangular Box Culvert Nomograph (Chart 11B)

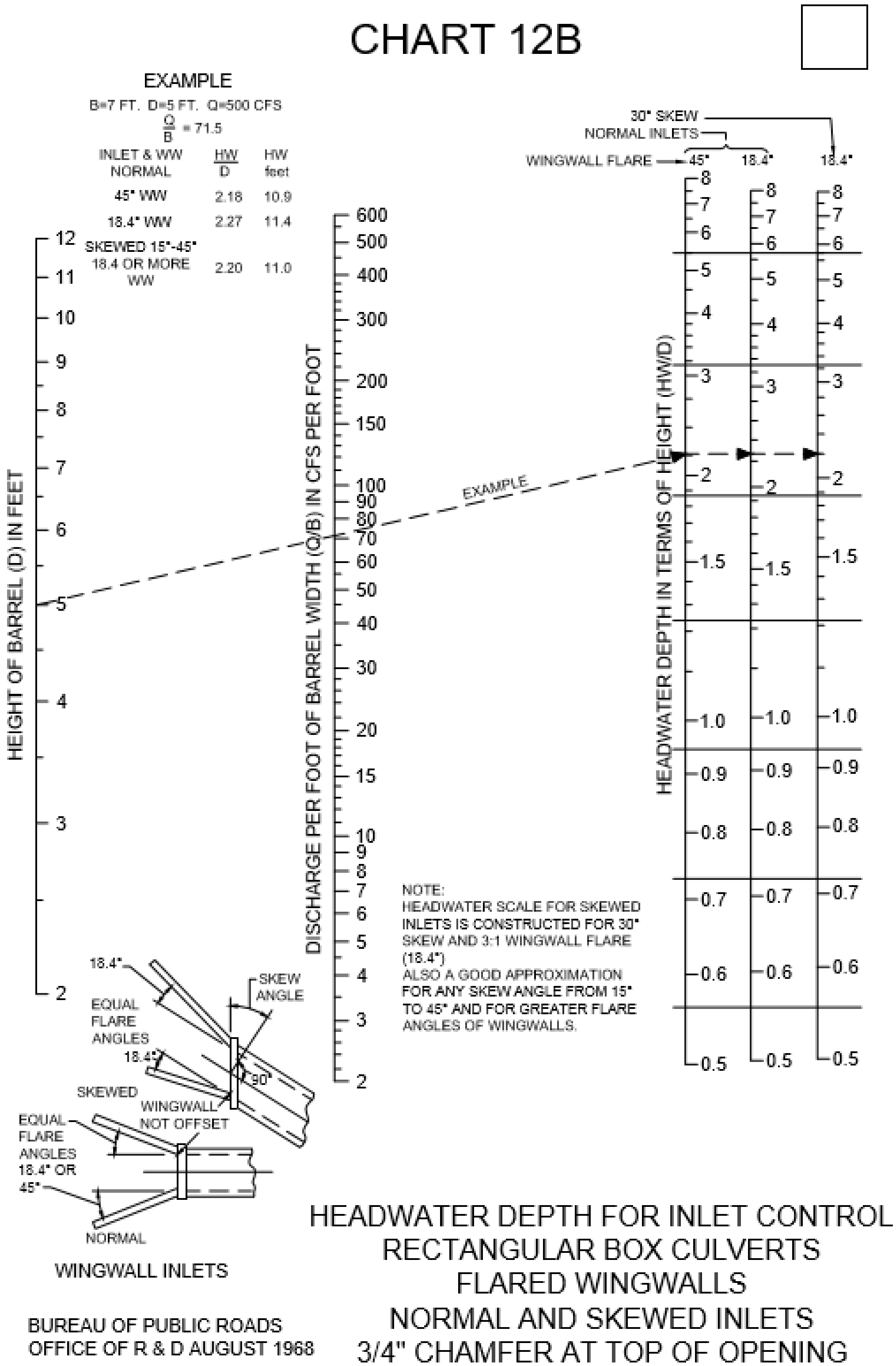


Figure E-12. Rectangular Box Culvert Nomograph (Chart 12B)

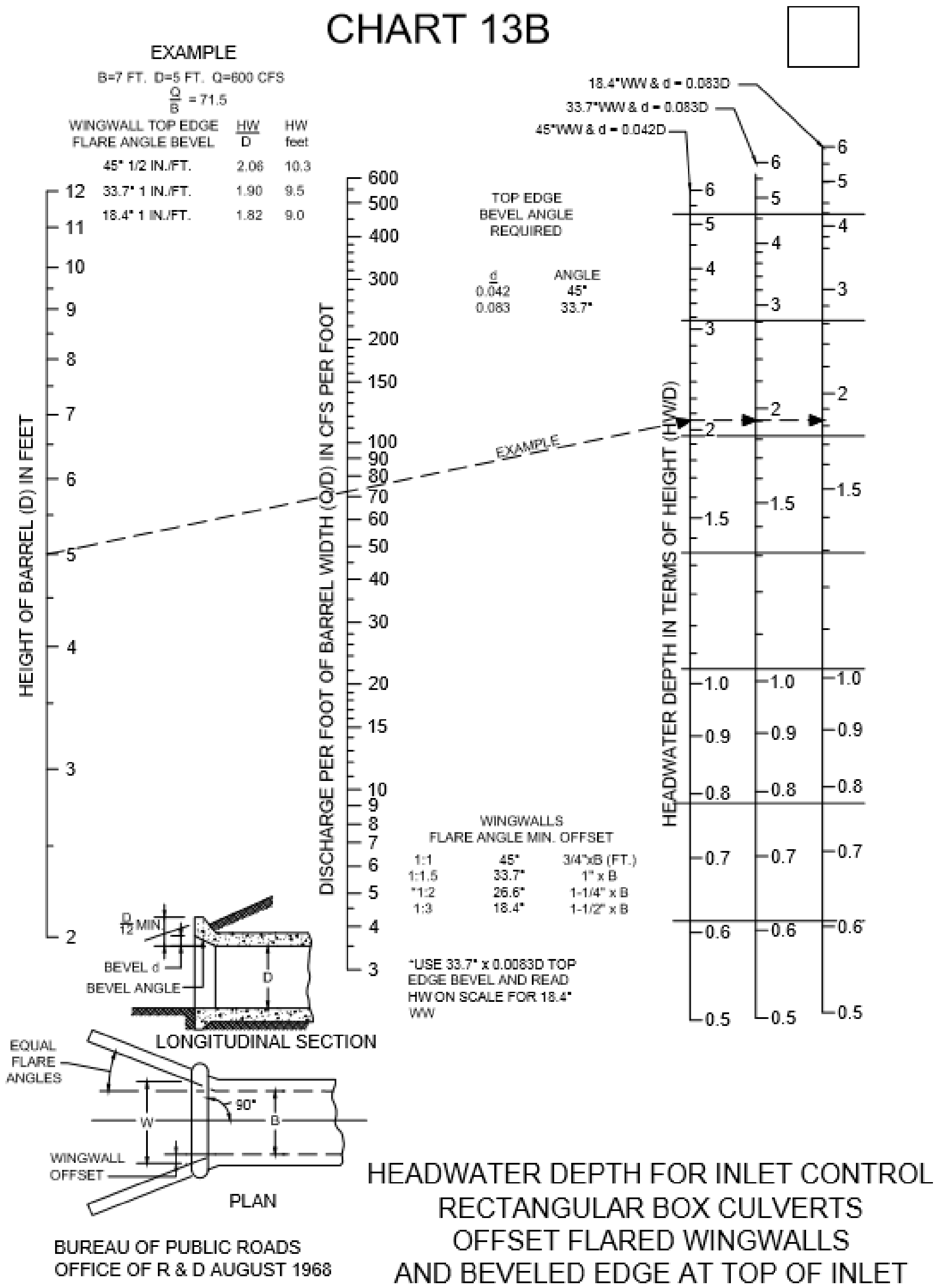


Figure E-13. Rectangular Box Culvert Nomograph (Chart 13B)

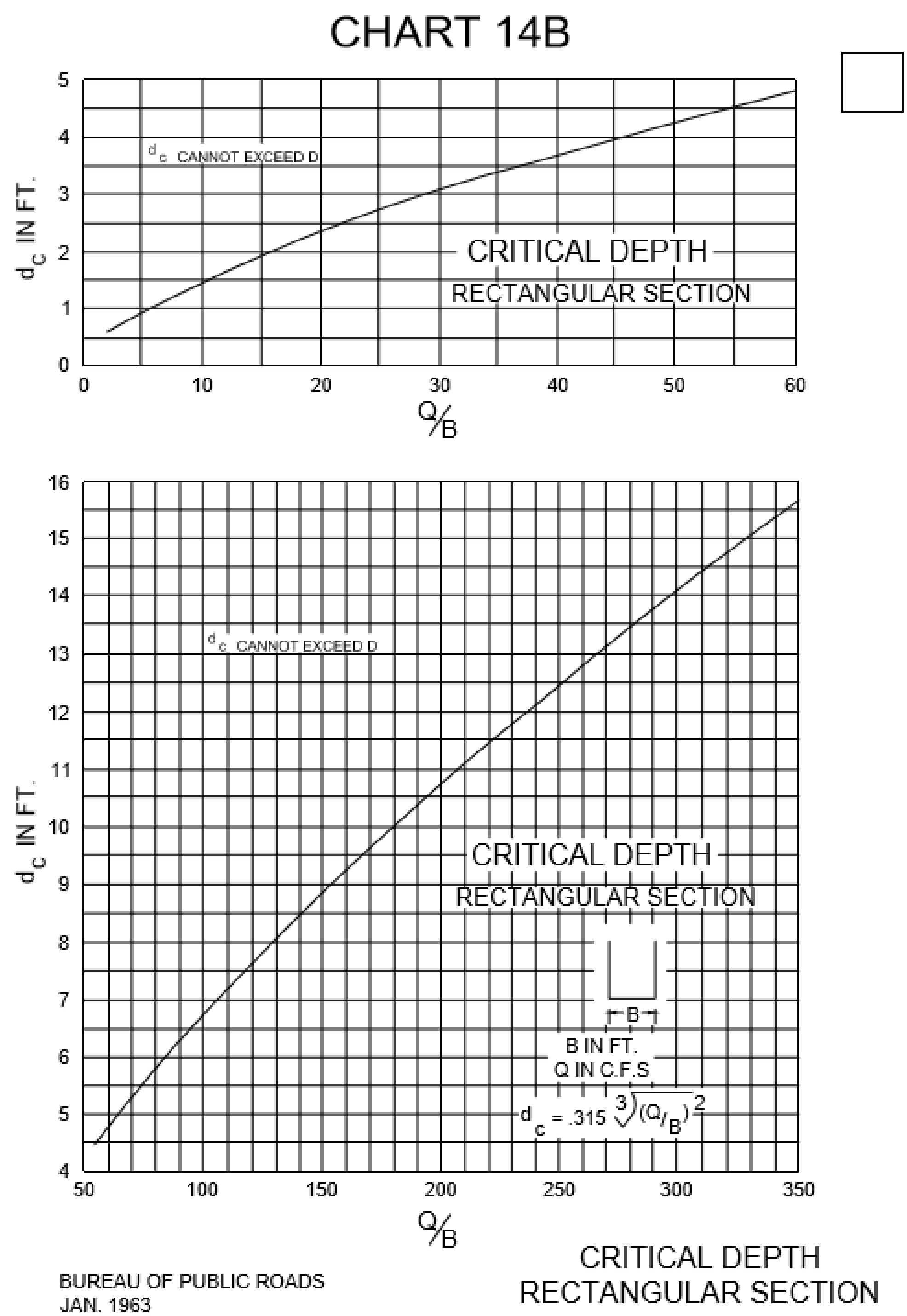


Figure E-14. Rectangular Box Culvert Nomograph (Chart 14B)

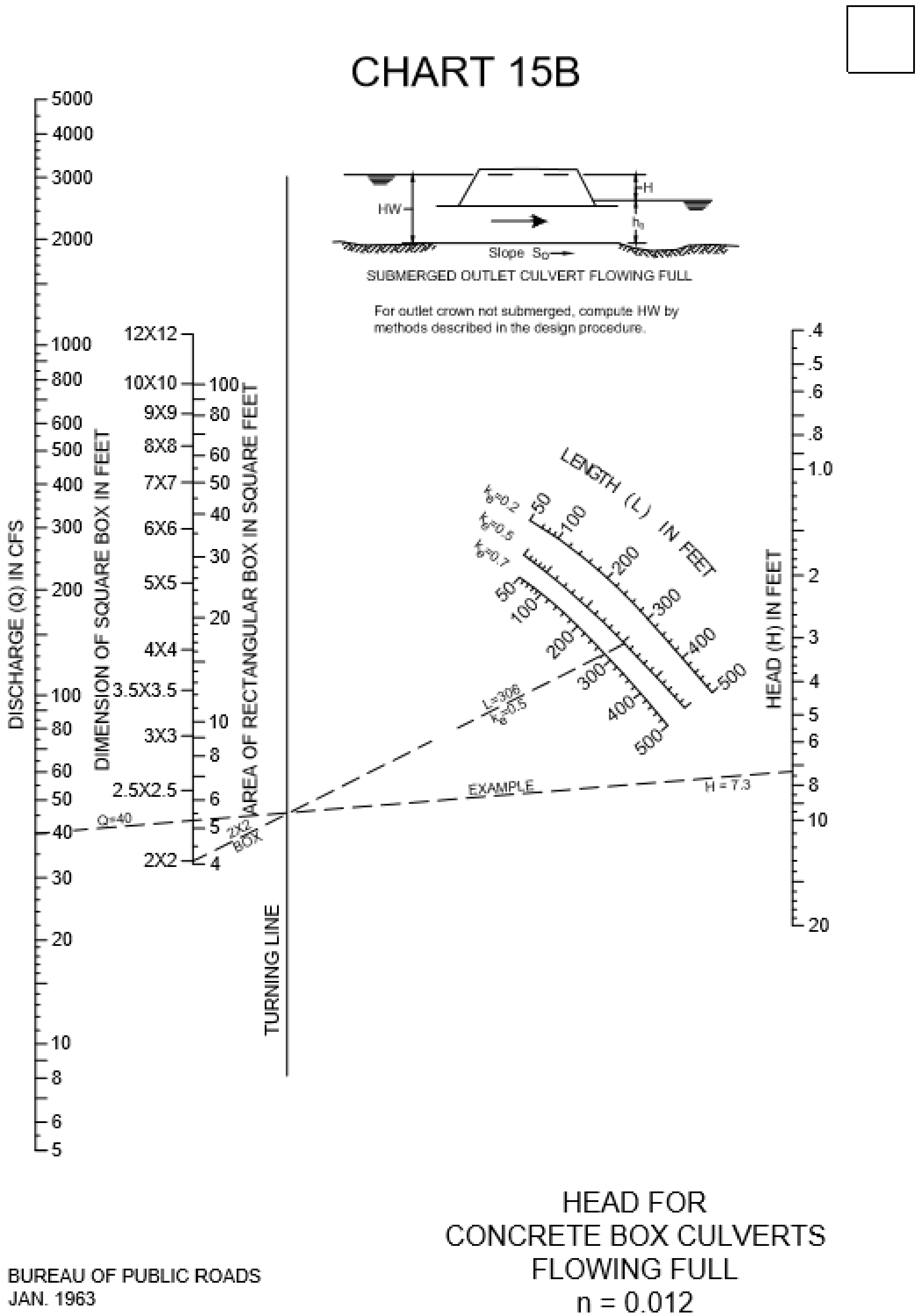


Figure E-15. Rectangular Box Culvert Nomograph (Chart 15B)

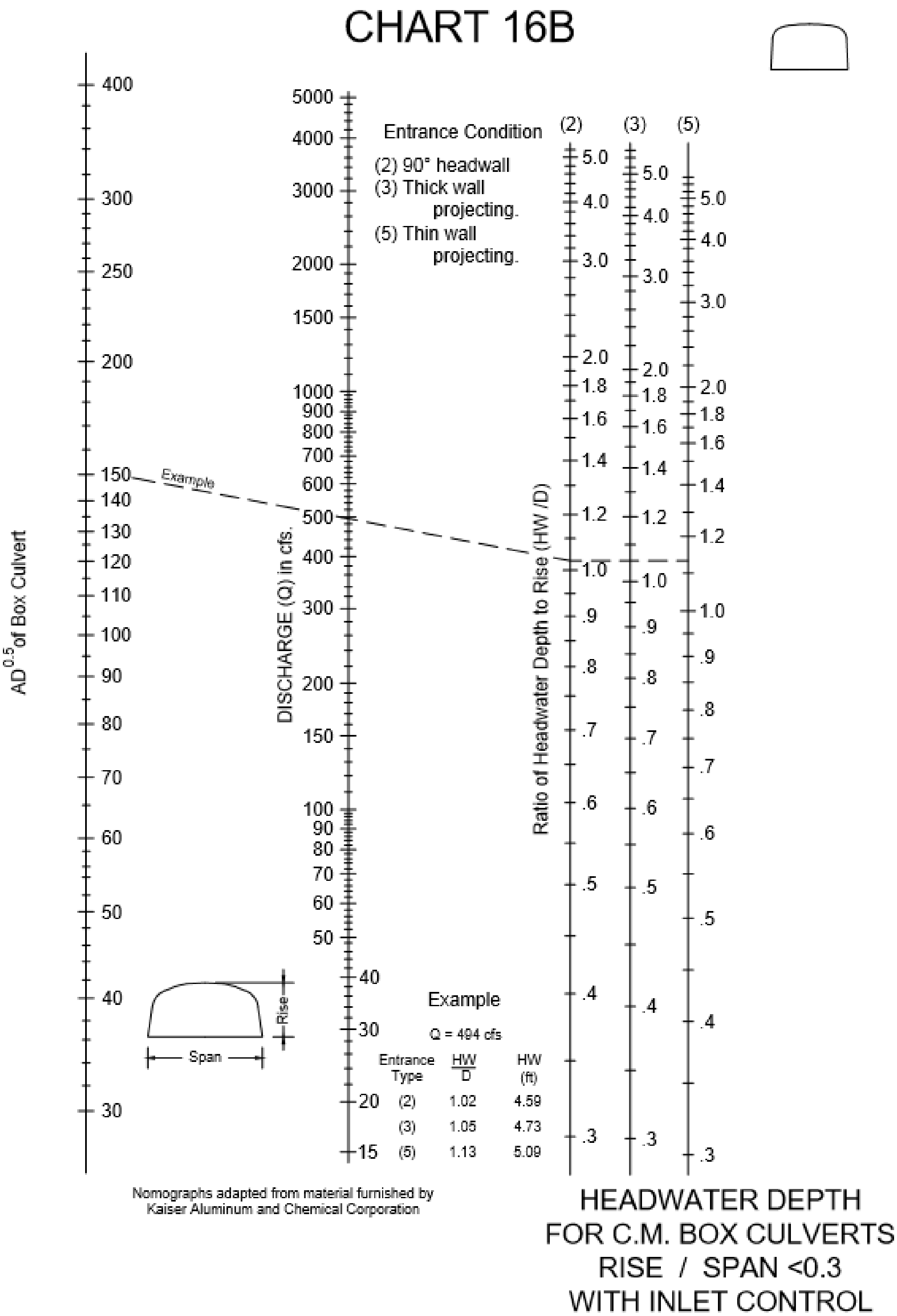


Figure E-16. Corrugated Metal Box Culvert Nomograph (Chart 16B)

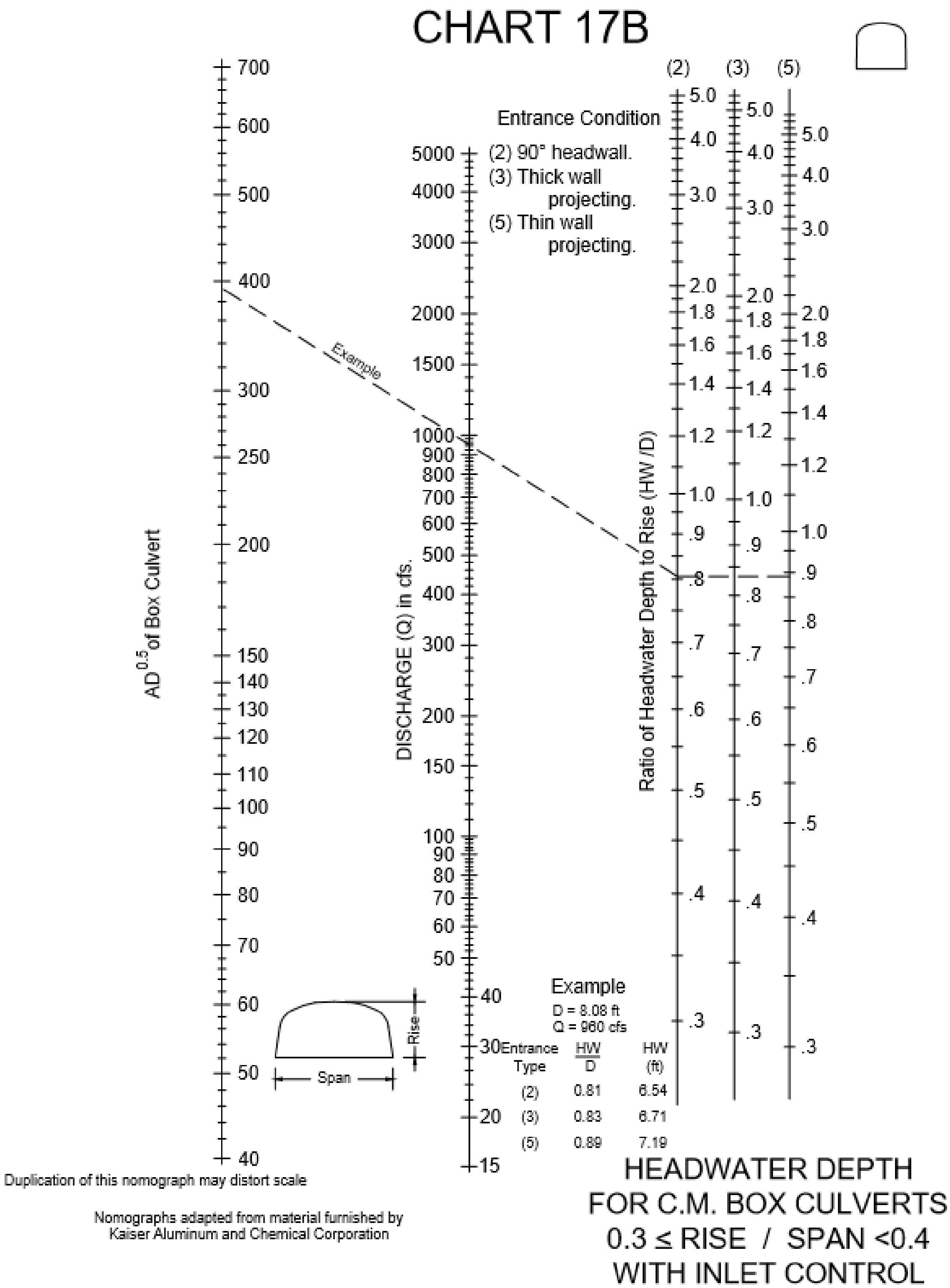


Figure E-17. Corrugated Metal Box Culvert Nomograph (Chart 17B)

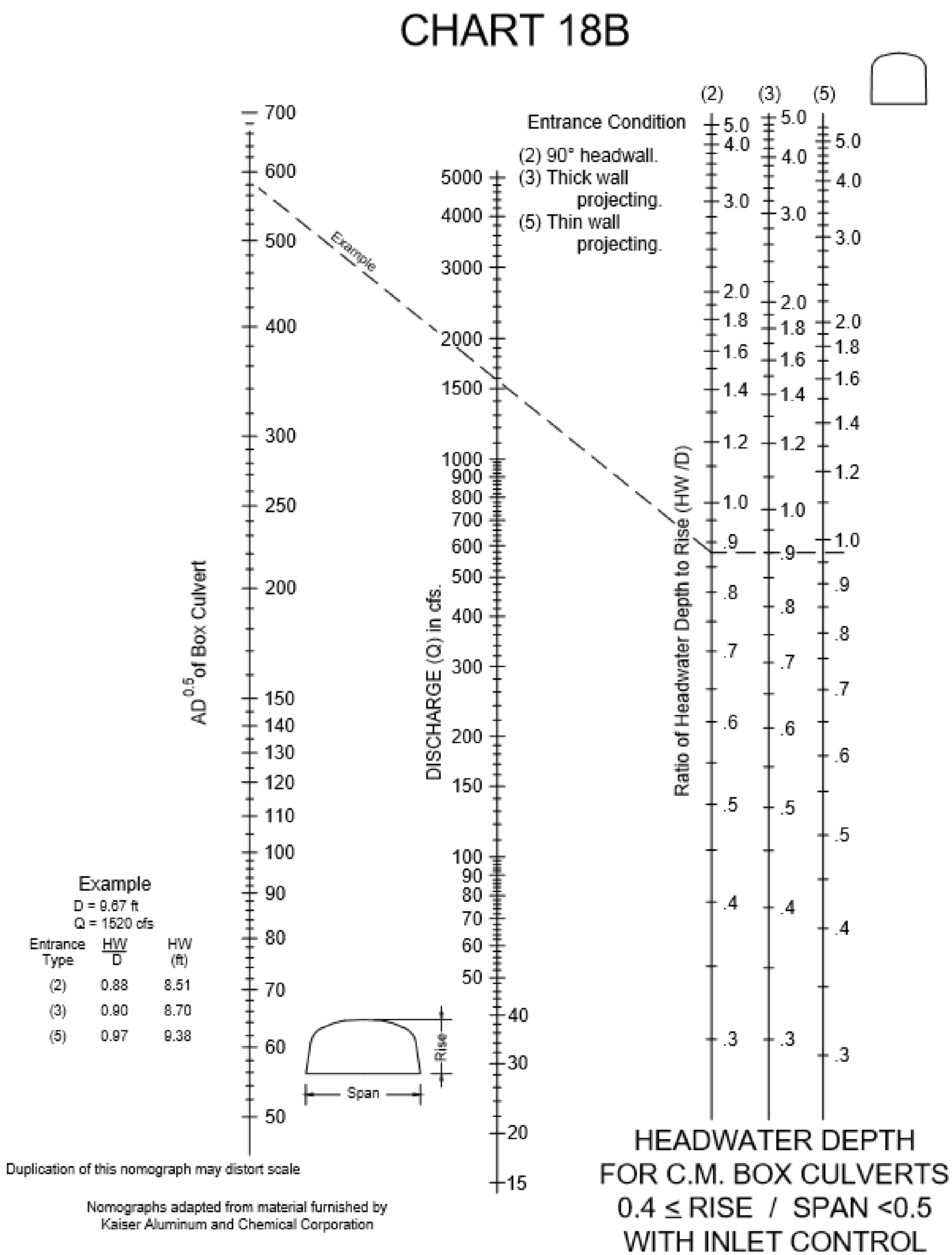


Figure E-18. Corrugated Metal Box Culvert Nomograph (Chart 18B)

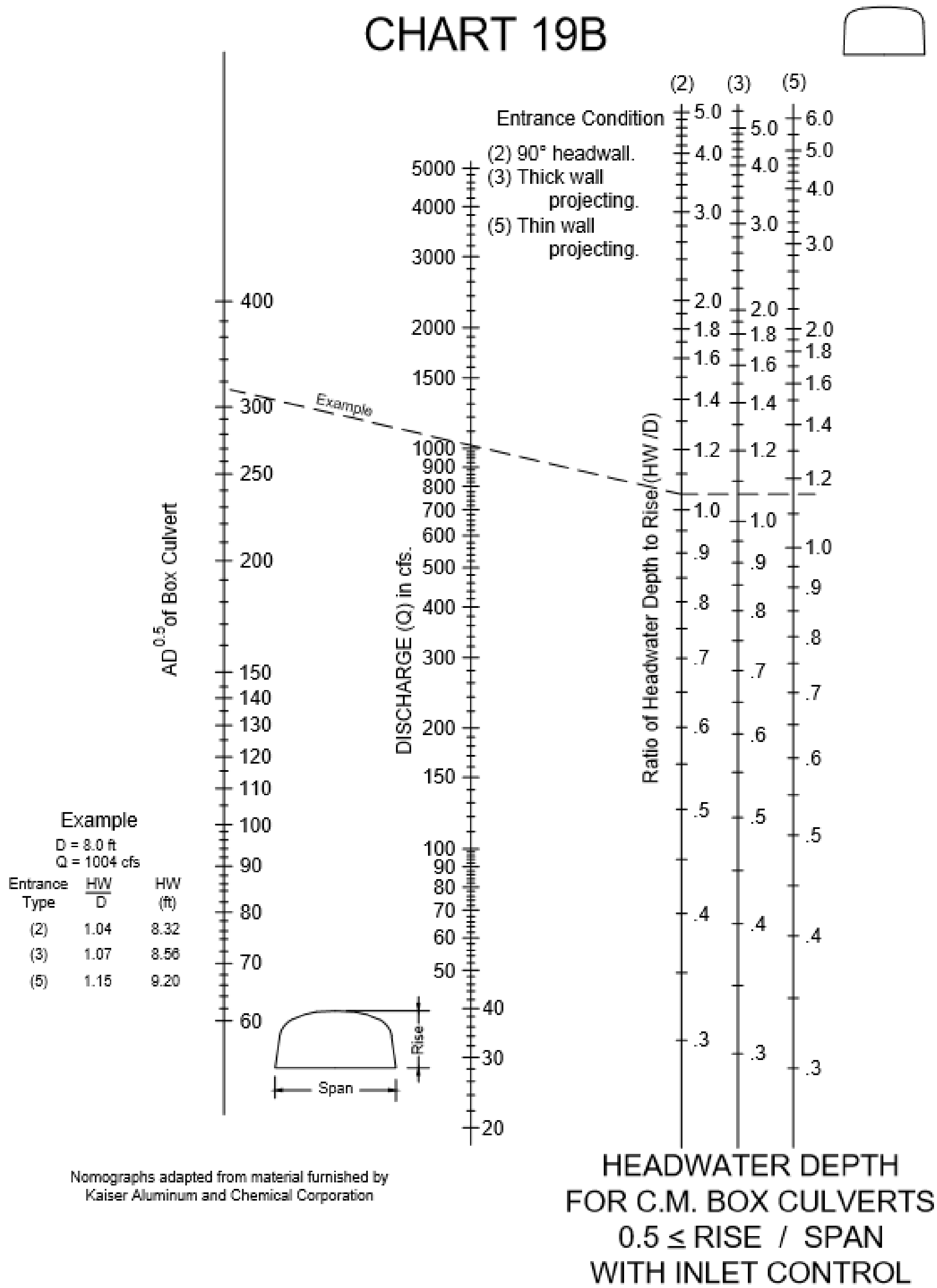


Figure E-19. Corrugated Metal Box Culvert Nomograph (Chart 19B)

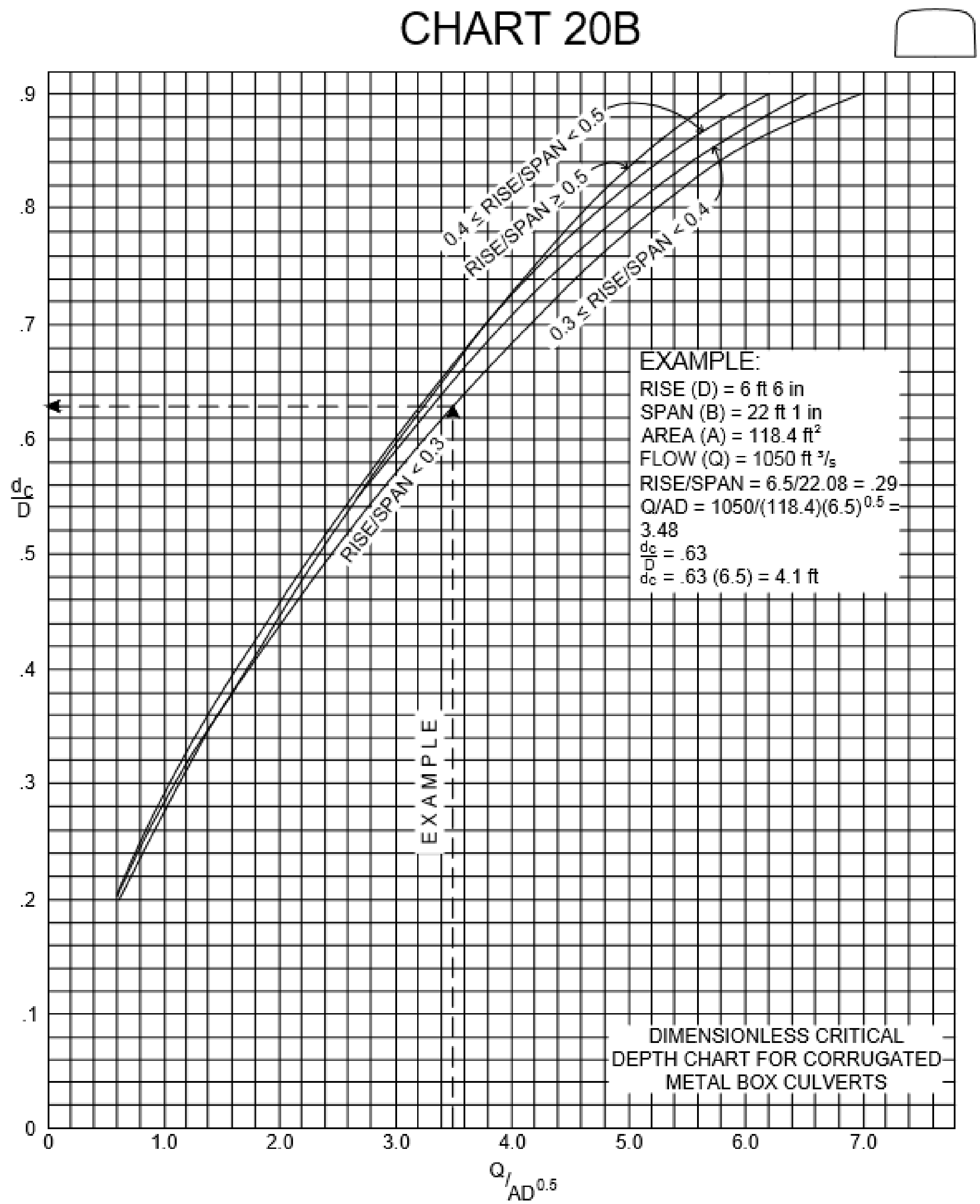


Figure E-20. Corrugated Metal Box Culvert Nomograph (Chart 20B)

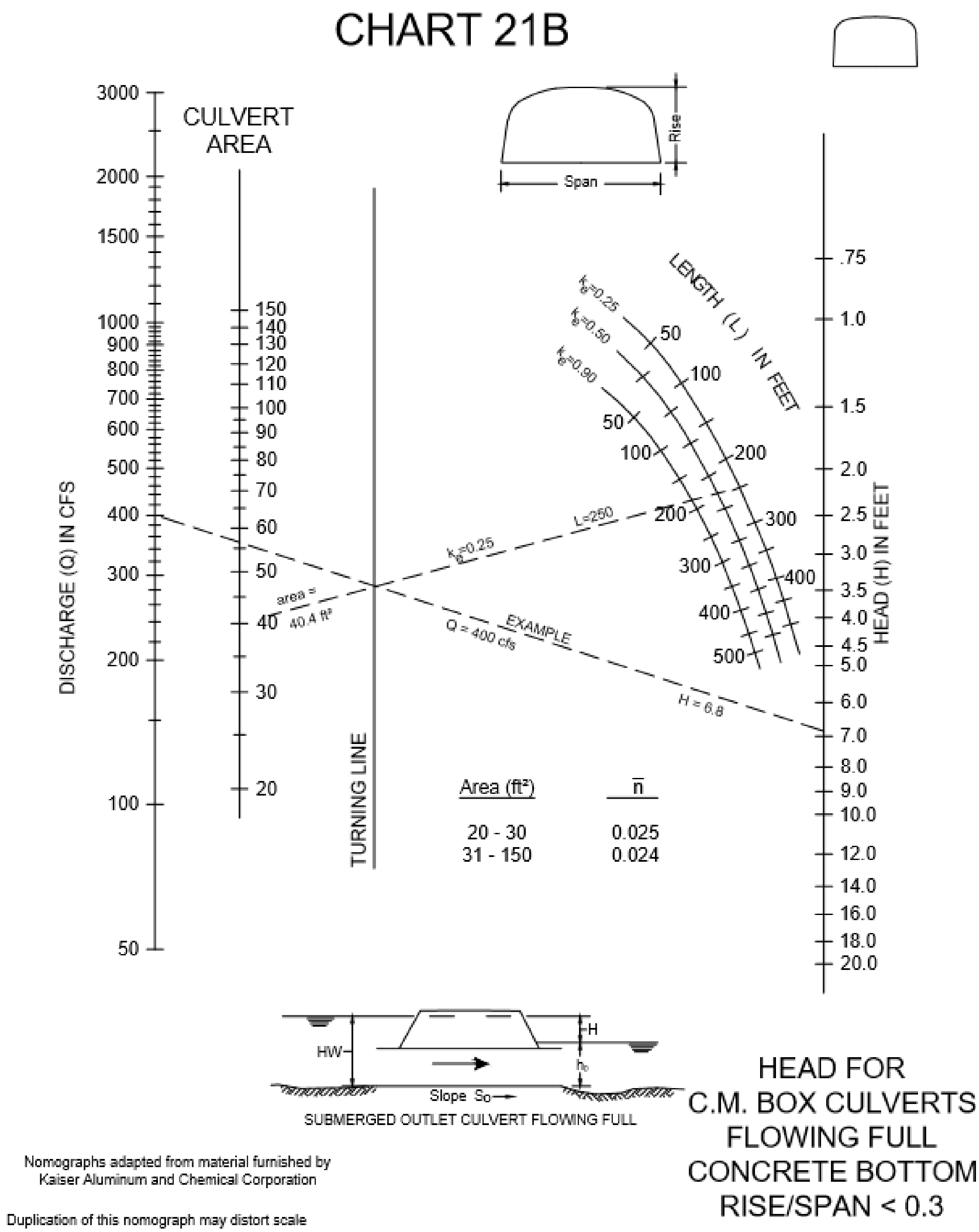


Figure E-21. Corrugated Metal Box Culvert Nomograph (Chart 21B)

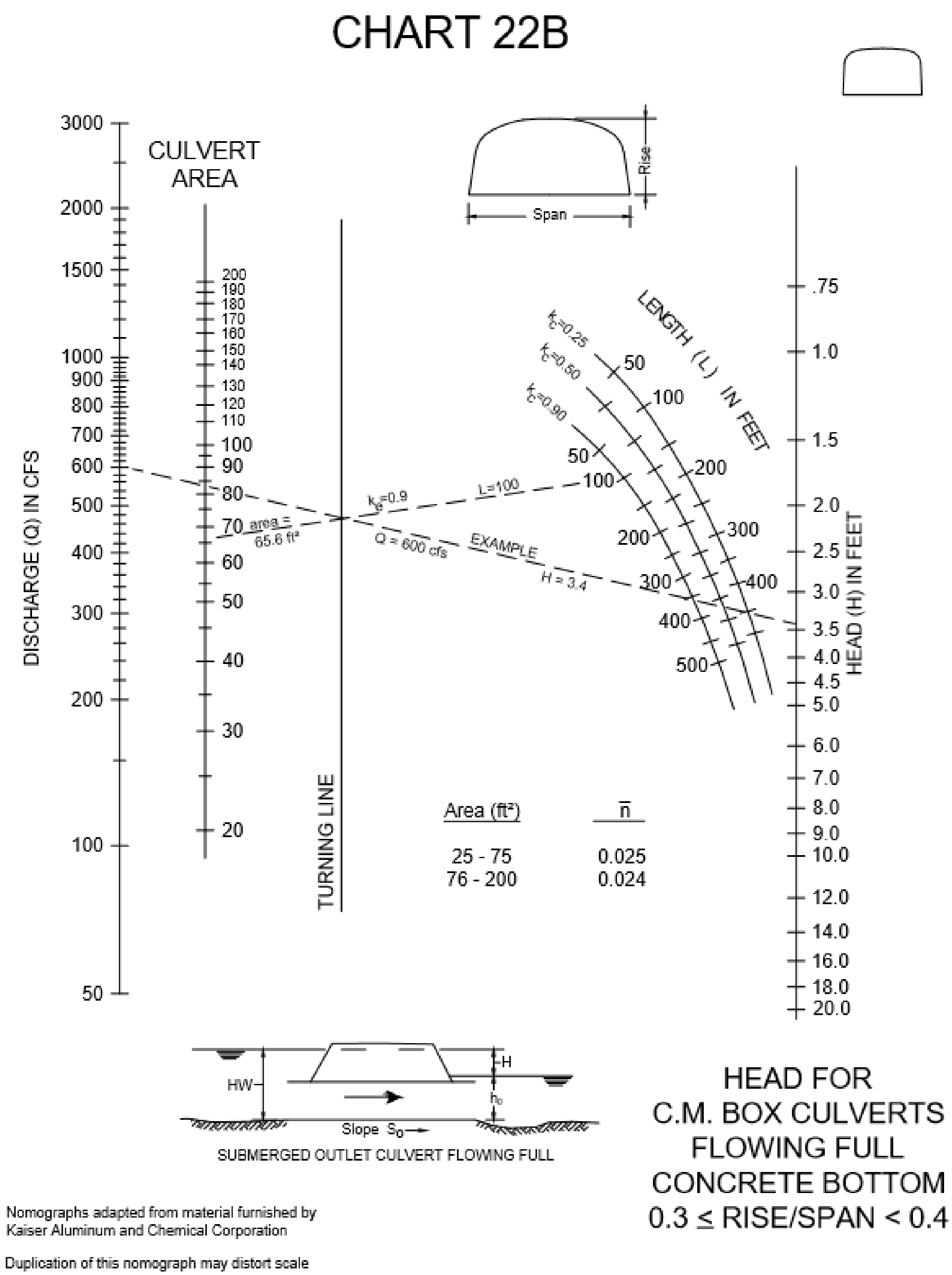


Figure E-22. Corrugated Metal Box Culvert Nomograph (Chart 22B)

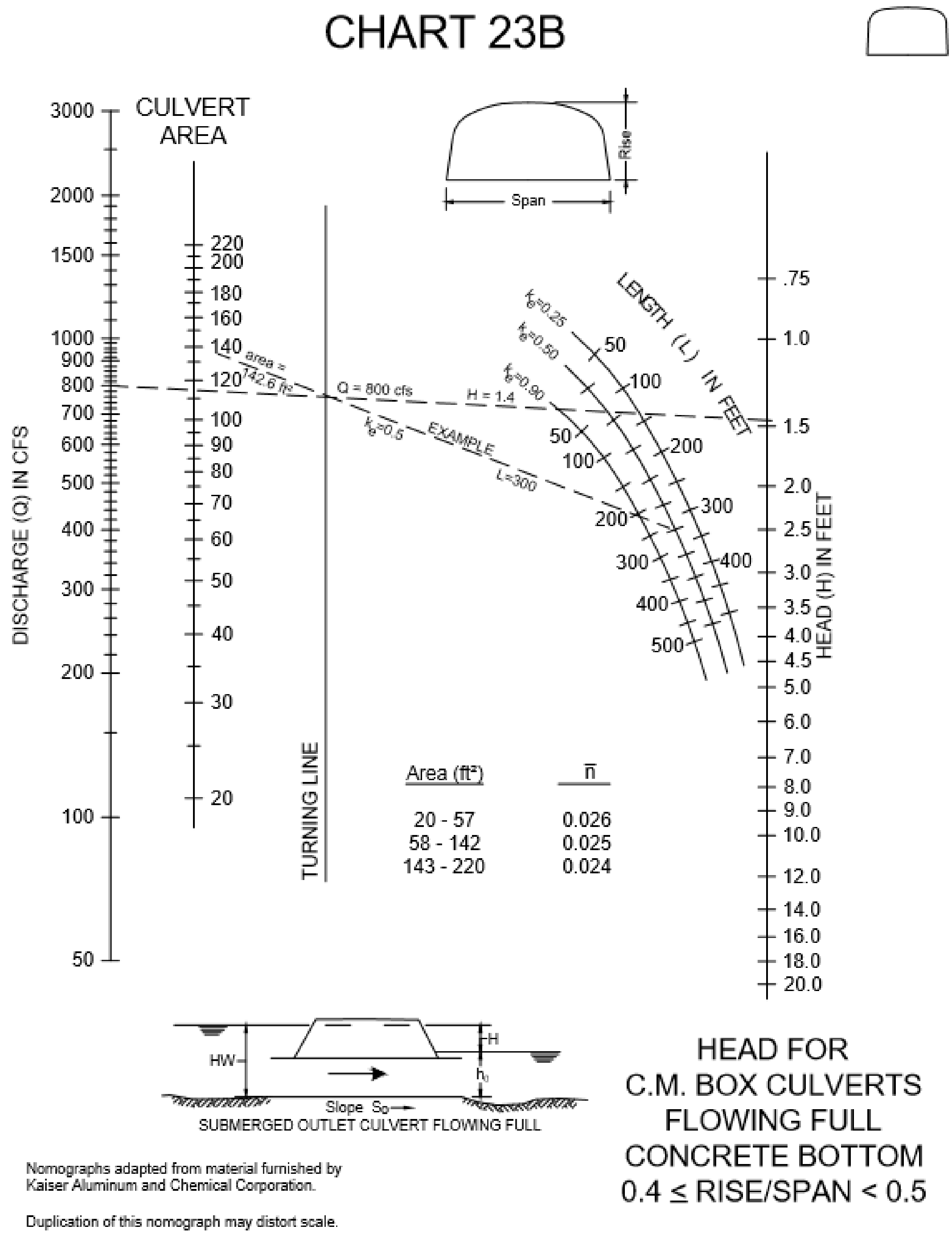


Figure E-23. Corrugated Metal Box Culvert Nomograph (Chart 23B)

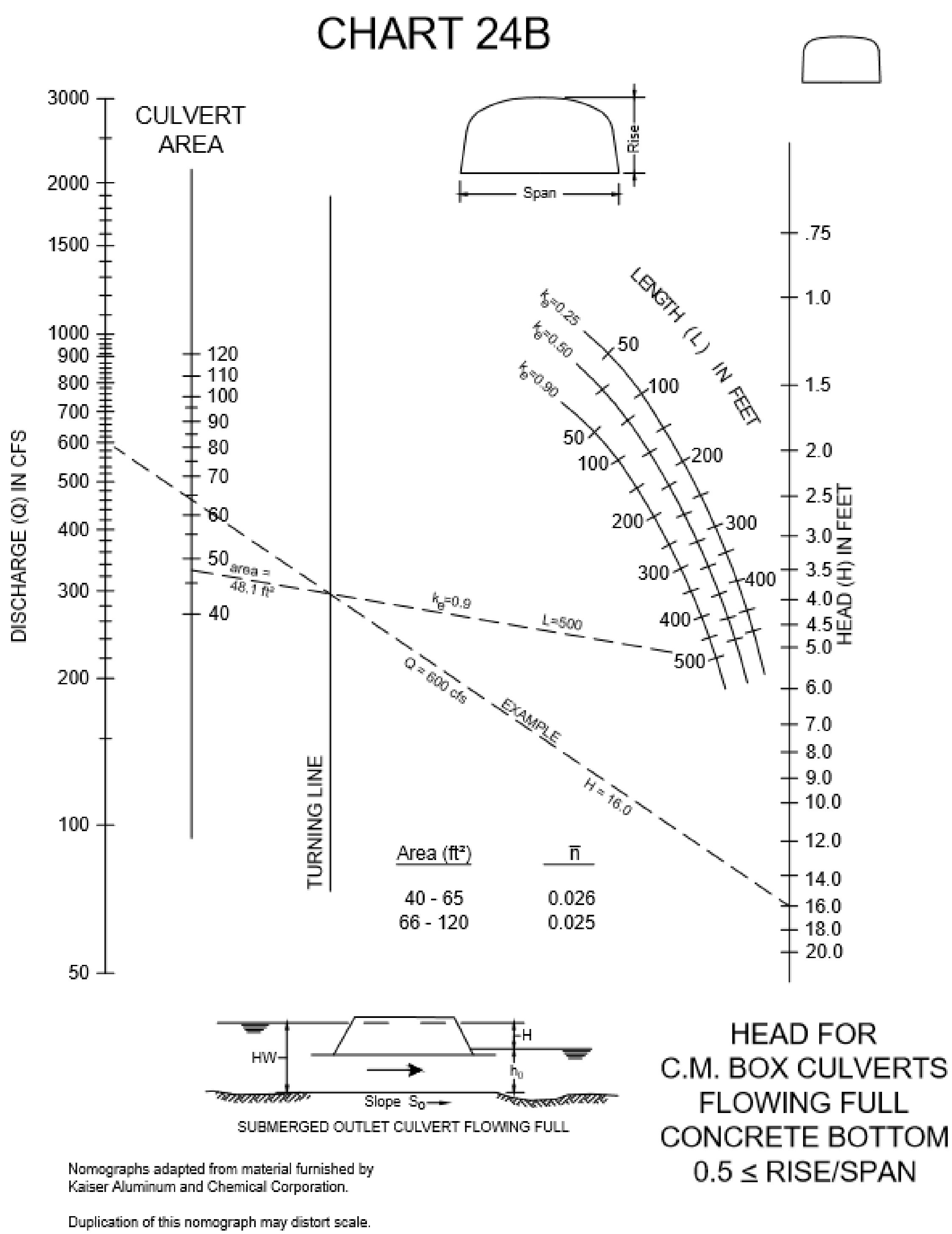
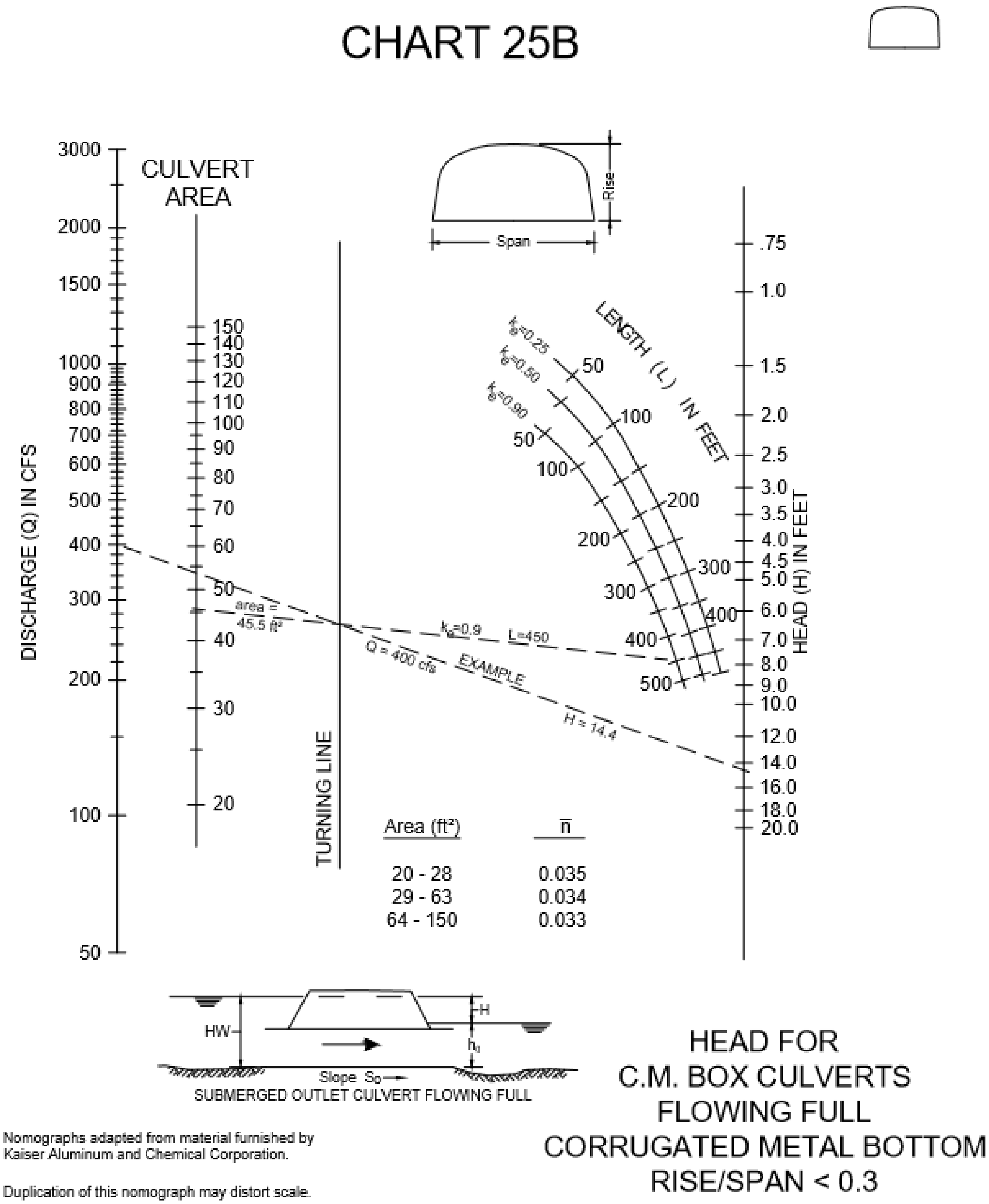


Figure E-24. Corrugated Metal Box Culvert Nomograph (Chart 24B)



Area (ft²)	n̄
20 - 28	0.035
29 - 63	0.034
64 - 150	0.033

Figure E-25. Corrugated Metal Box Culvert Nomograph (Chart 25B)

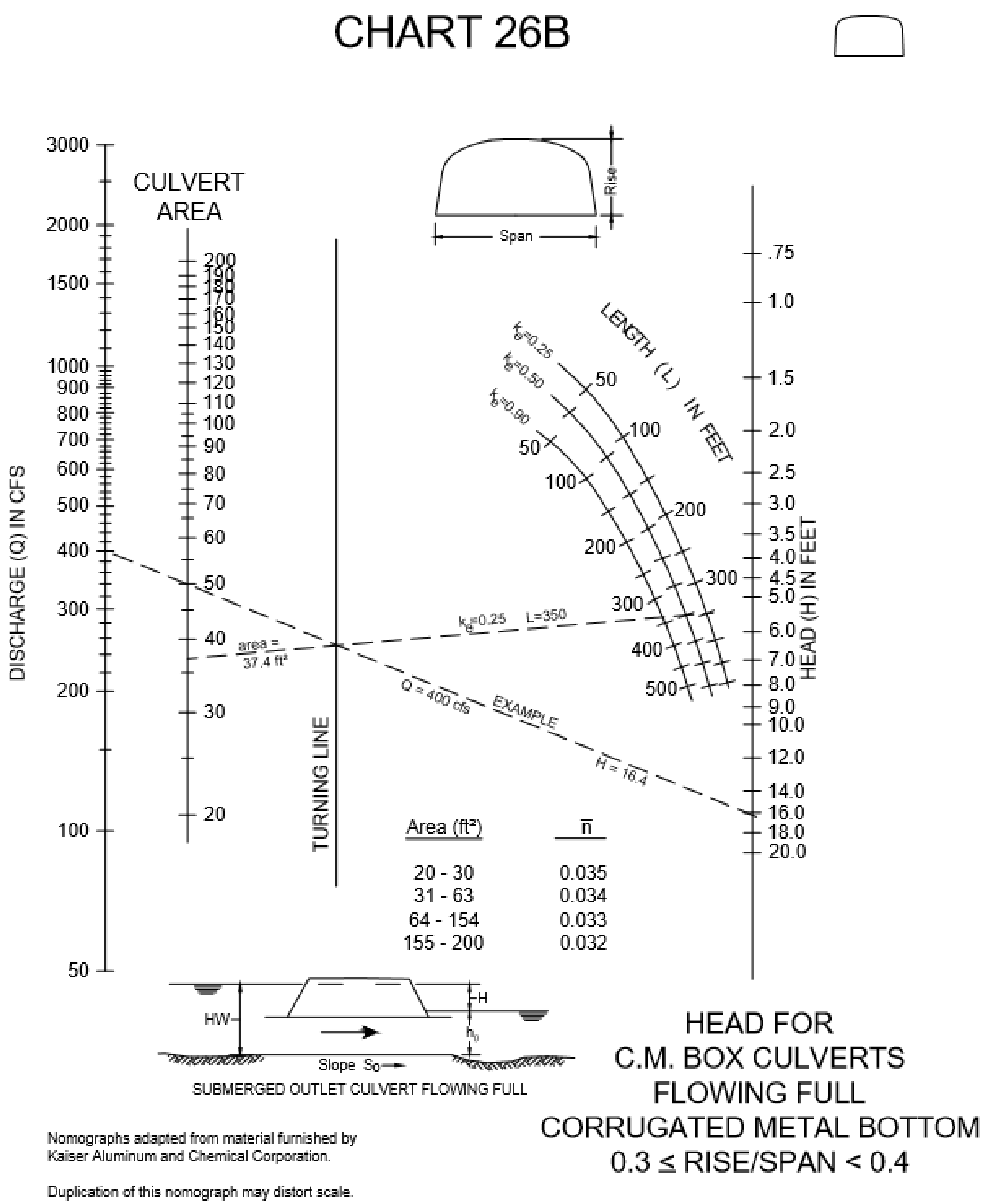


Figure E-26. Corrugated Metal Box Culvert Nomograph (Chart 26B)

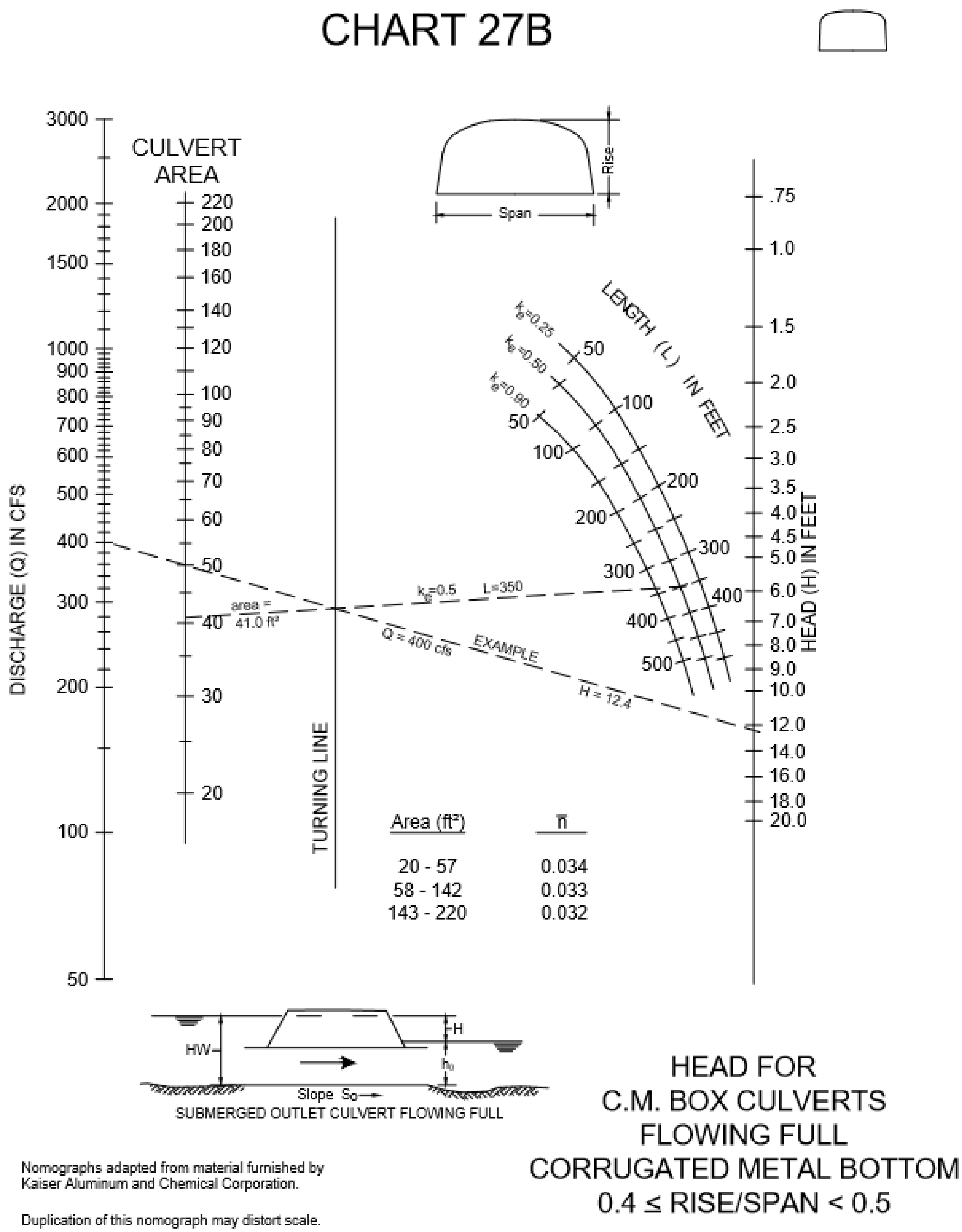


Figure E-27. Corrugated Metal Box Culvert Nomograph (Chart 27B)

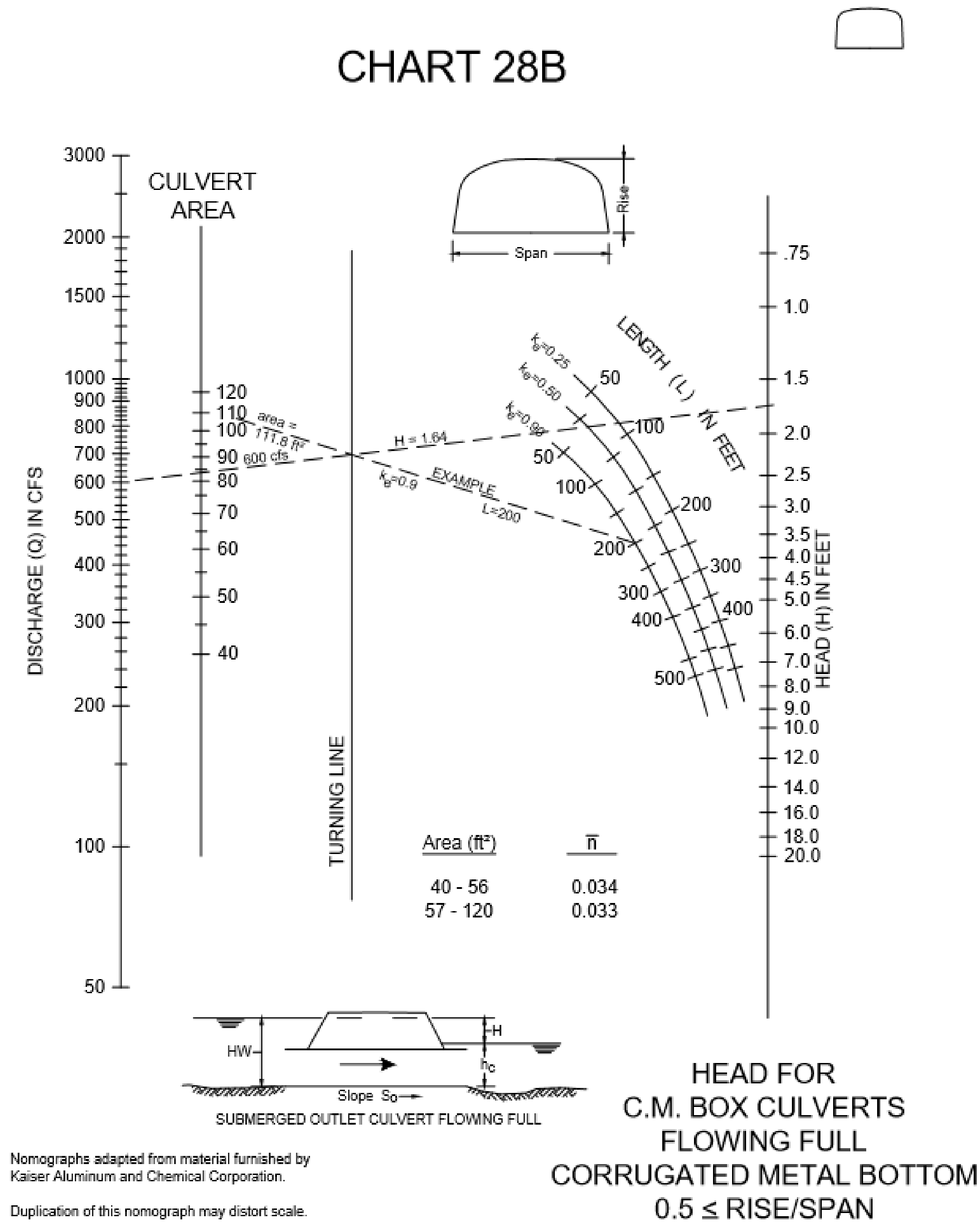


Figure E-28. Corrugated Metal Box Culvert Nomograph (Chart 28B)

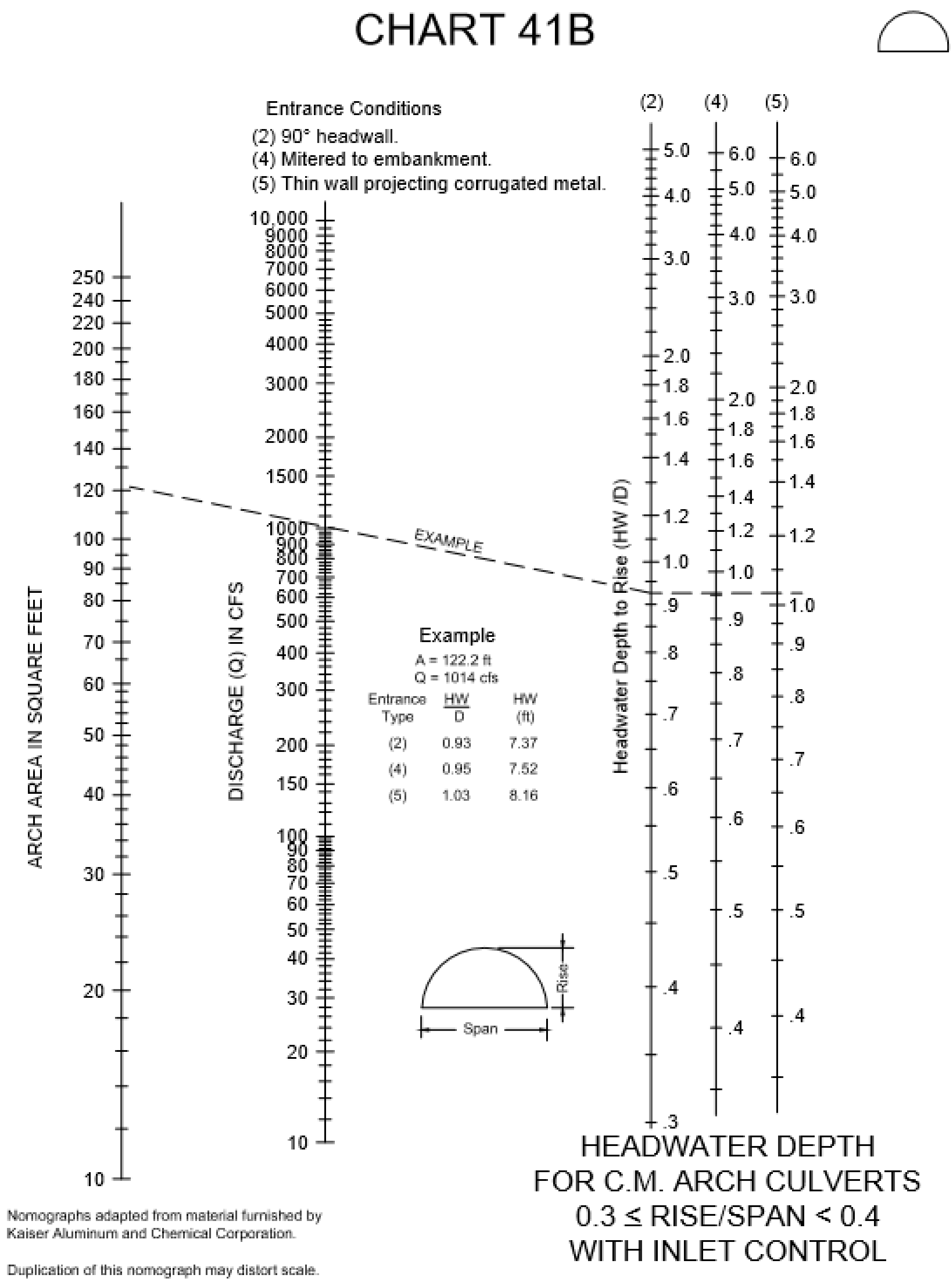


Figure E-29. Corrugated Metal Arch Culvert Nomograph (Chart 41B)

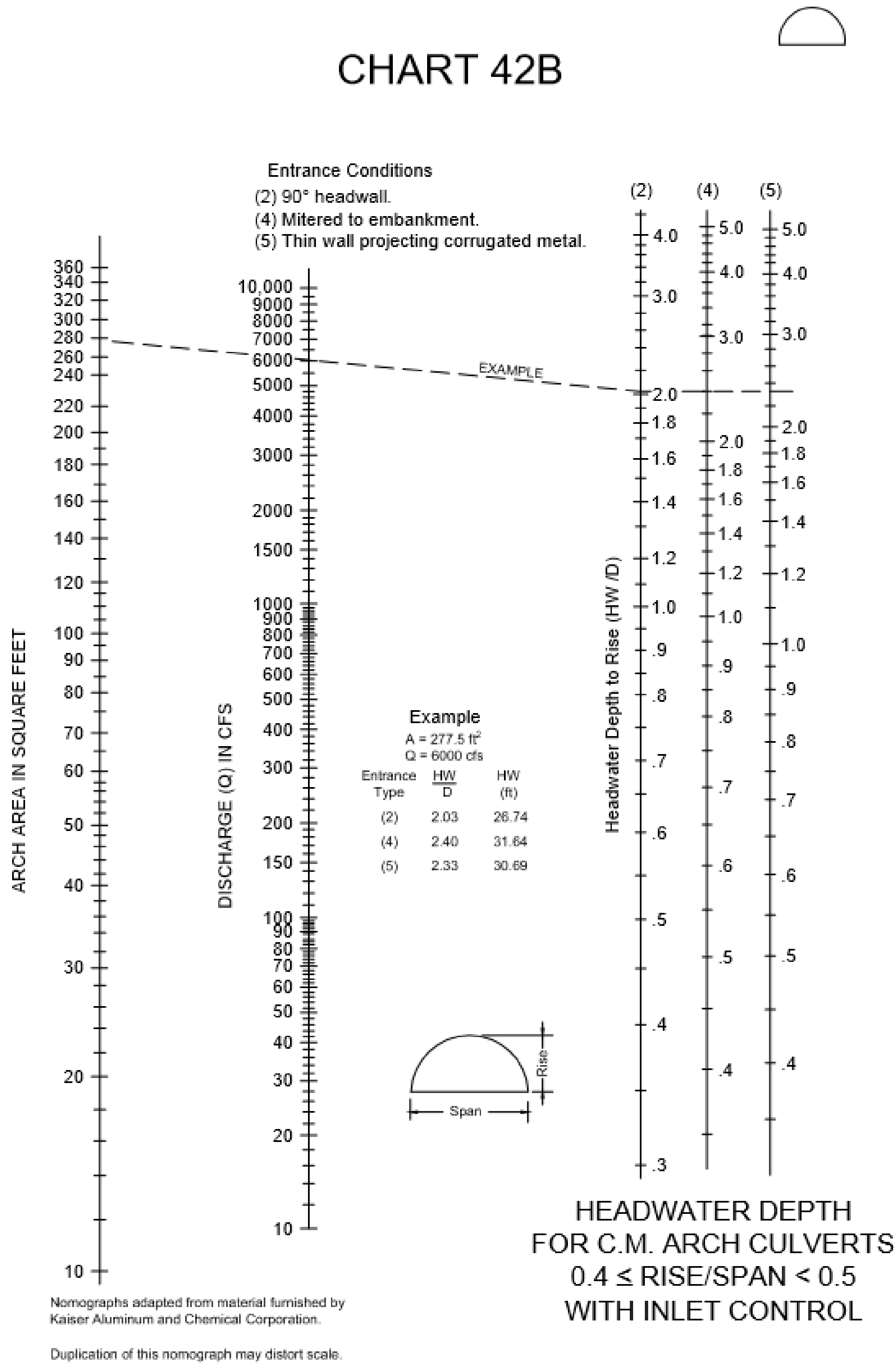


Figure E-30. Corrugated Metal Arch Culvert Nomograph (Chart 42B)

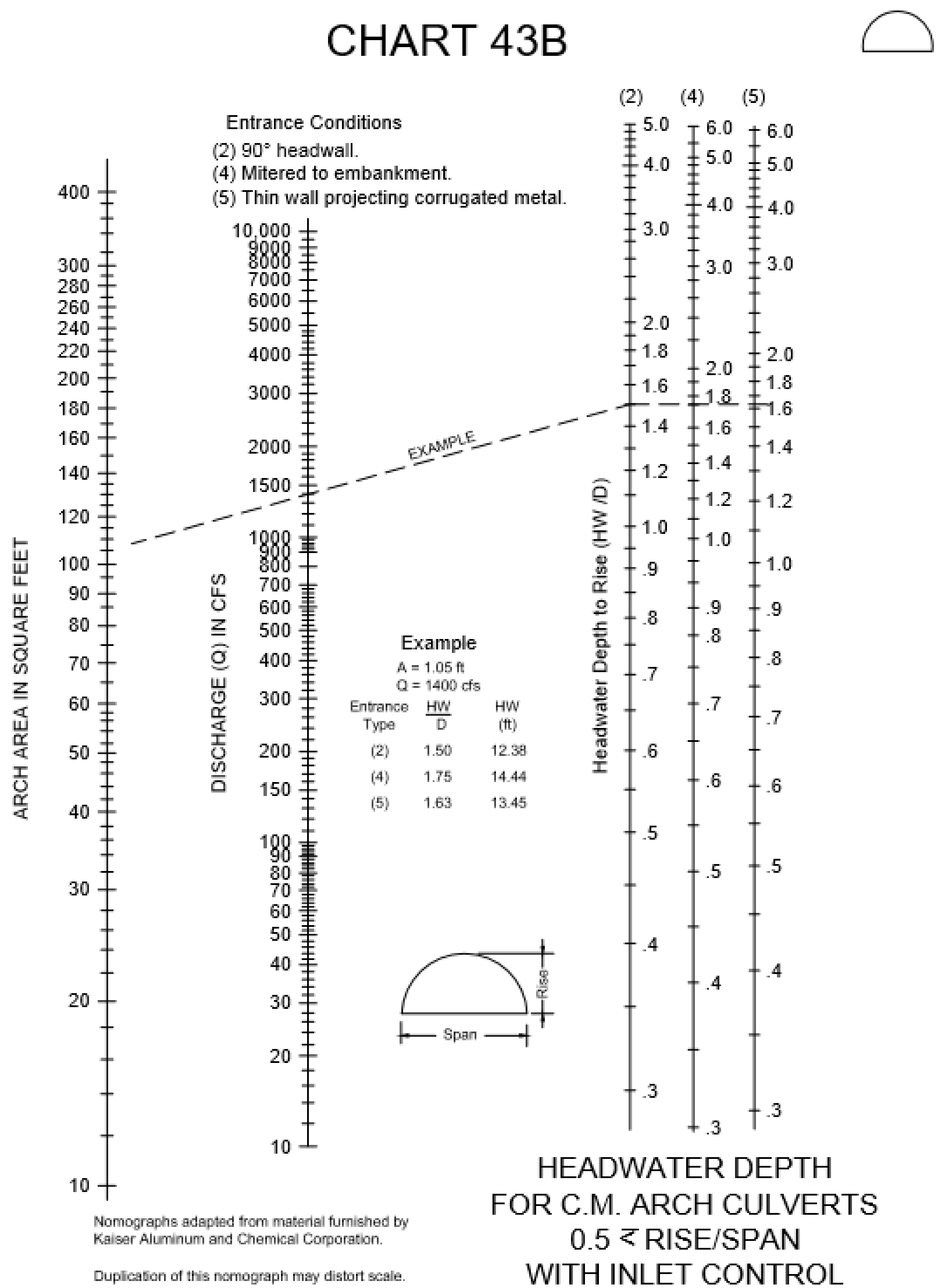


Figure E-31: Corrugated Metal Arch Culvert Nomograph (Chart 43B)

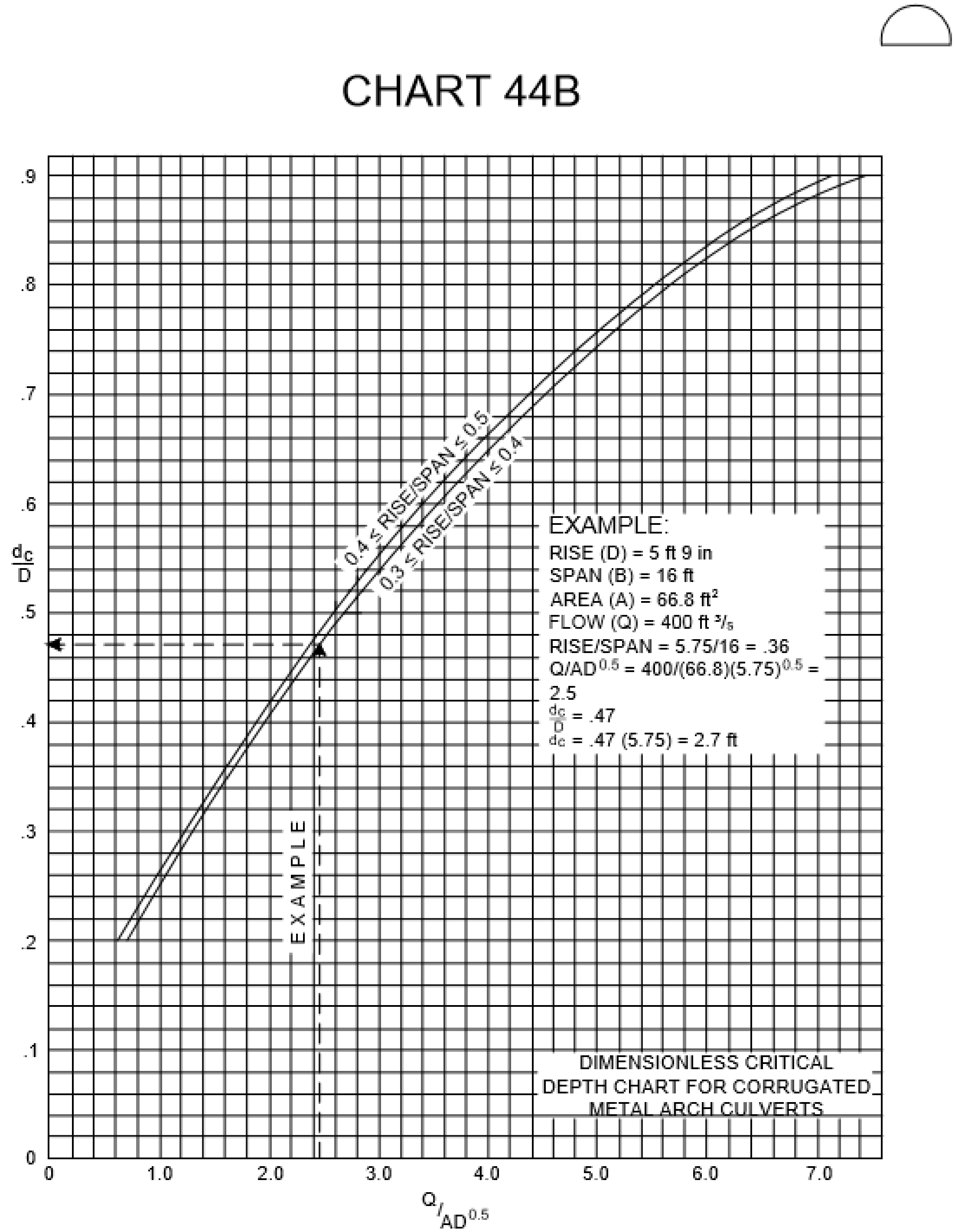


Figure E-32. Corrugated Metal Arch Culvert Nomograph (Chart 44B)

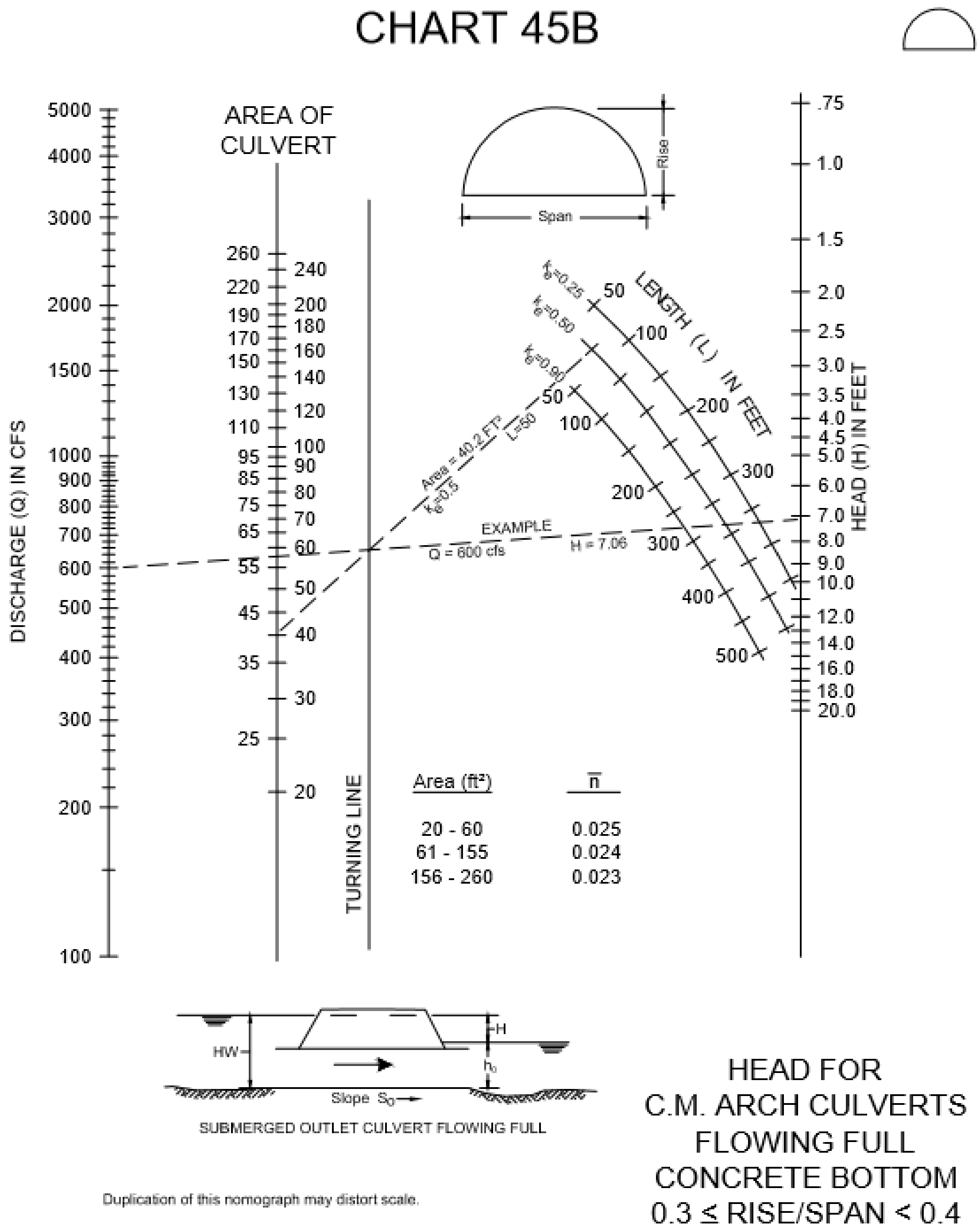


Figure E-33. Corrugated Metal Arch Culvert Nomograph (Chart 45B)

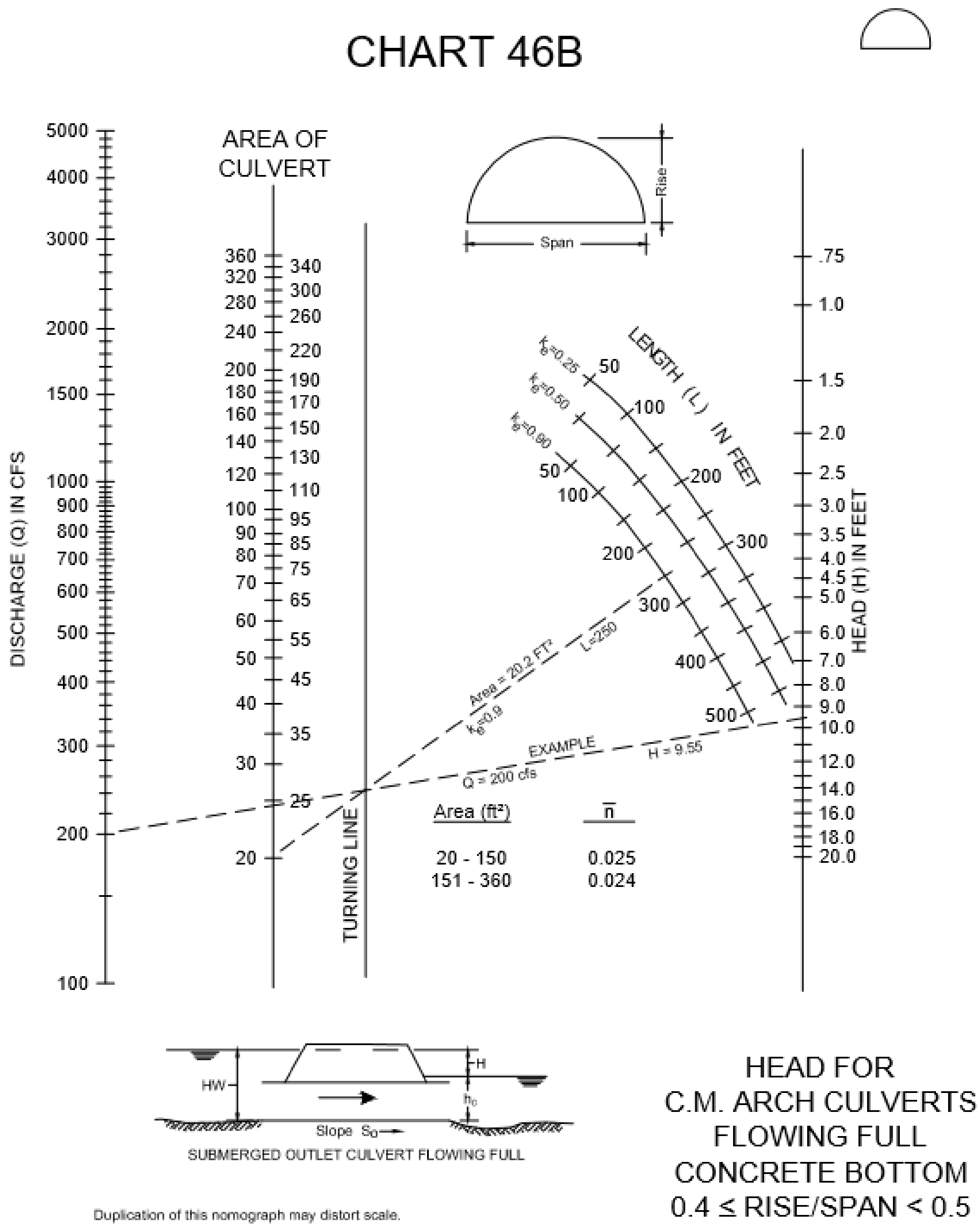


Figure E-34. Corrugated Metal Arch Culvert Nomograph (Chart 46B)

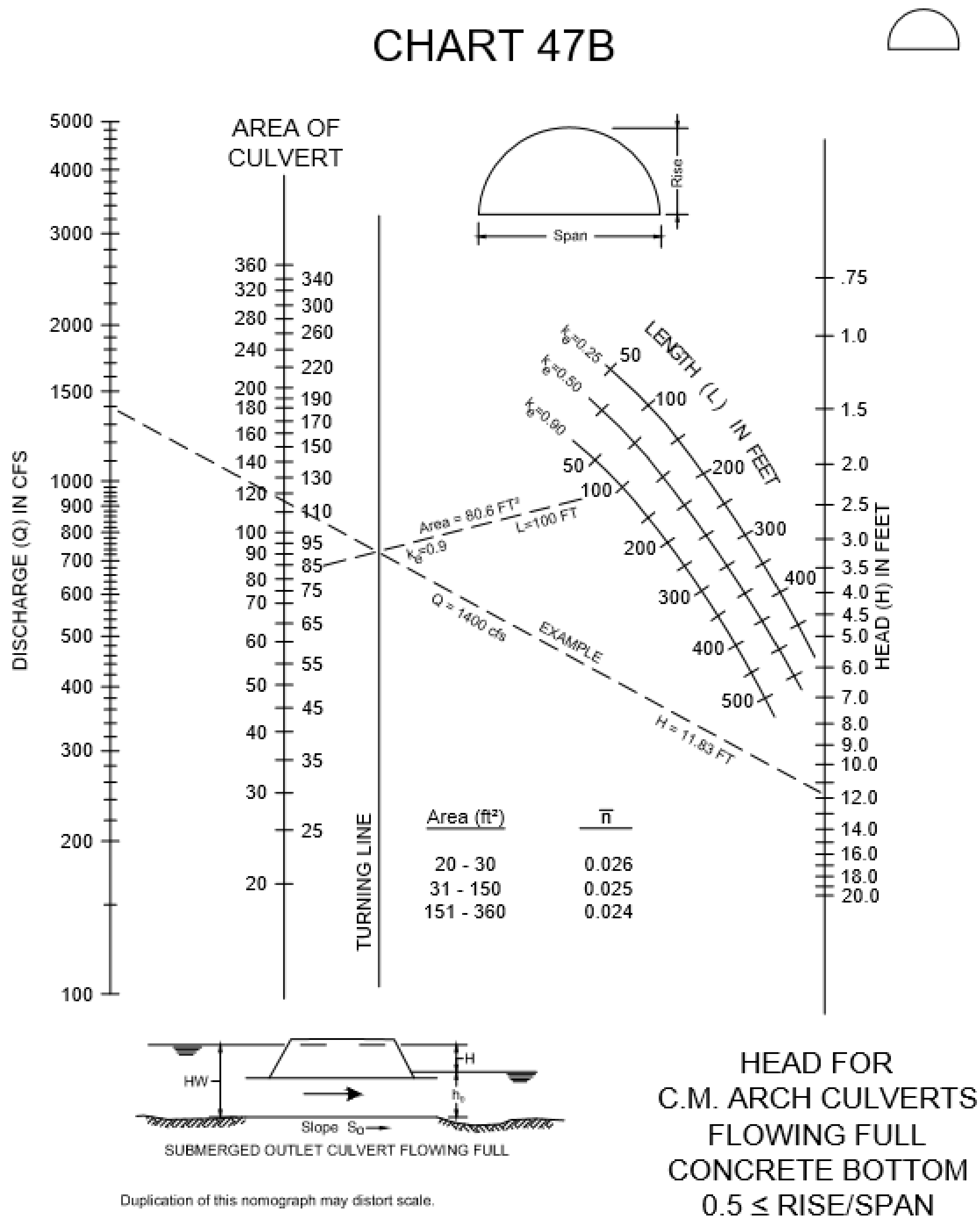


Figure E-35. Corrugated Metal Arch Culvert Nomograph (Chart 47B)

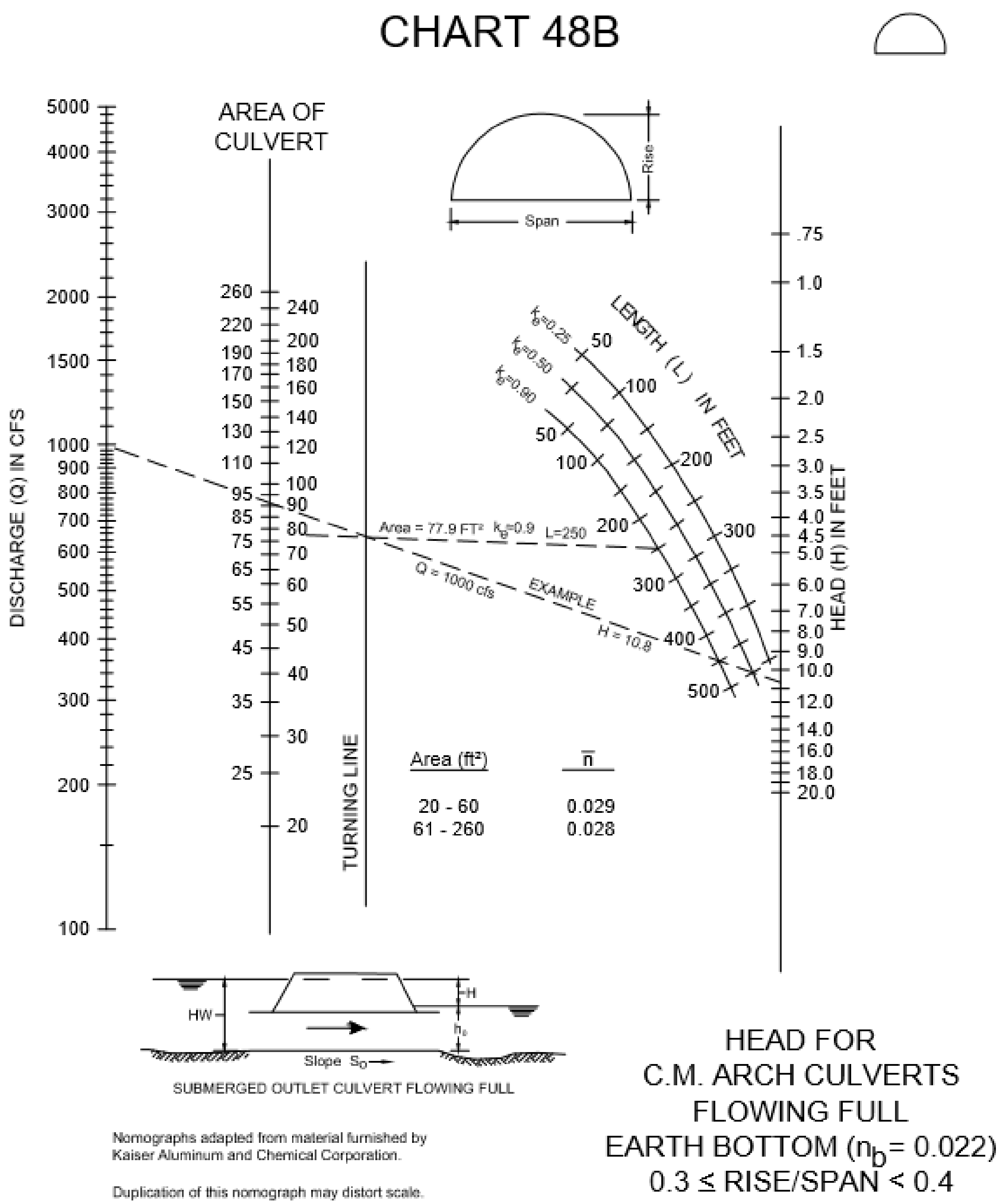


Figure E-36. Corrugated Metal Arch Culvert Nomograph (Chart 48B)

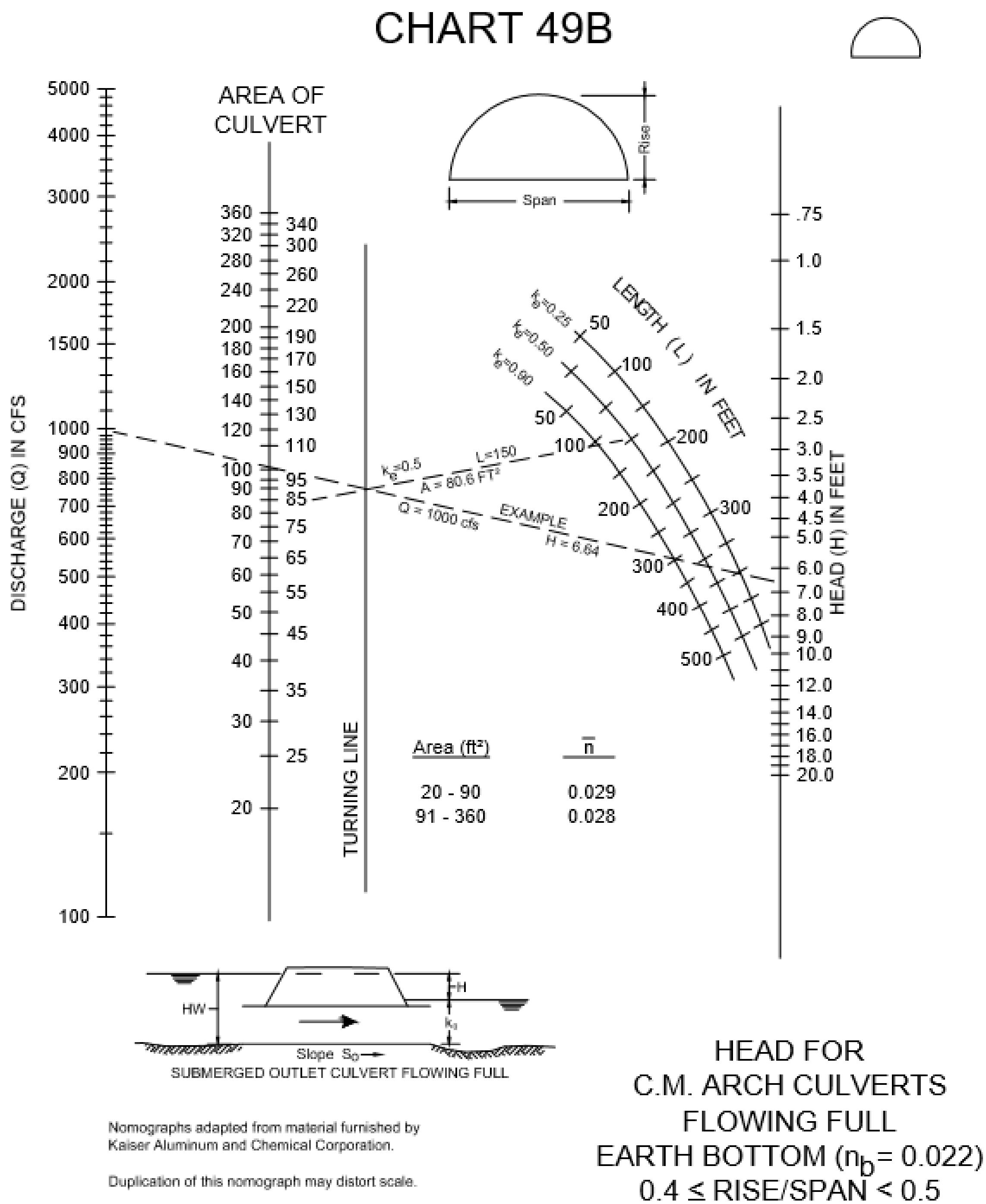


Figure E-37. Corrugated Metal Arch Culvert Nomograph (Chart 49B)

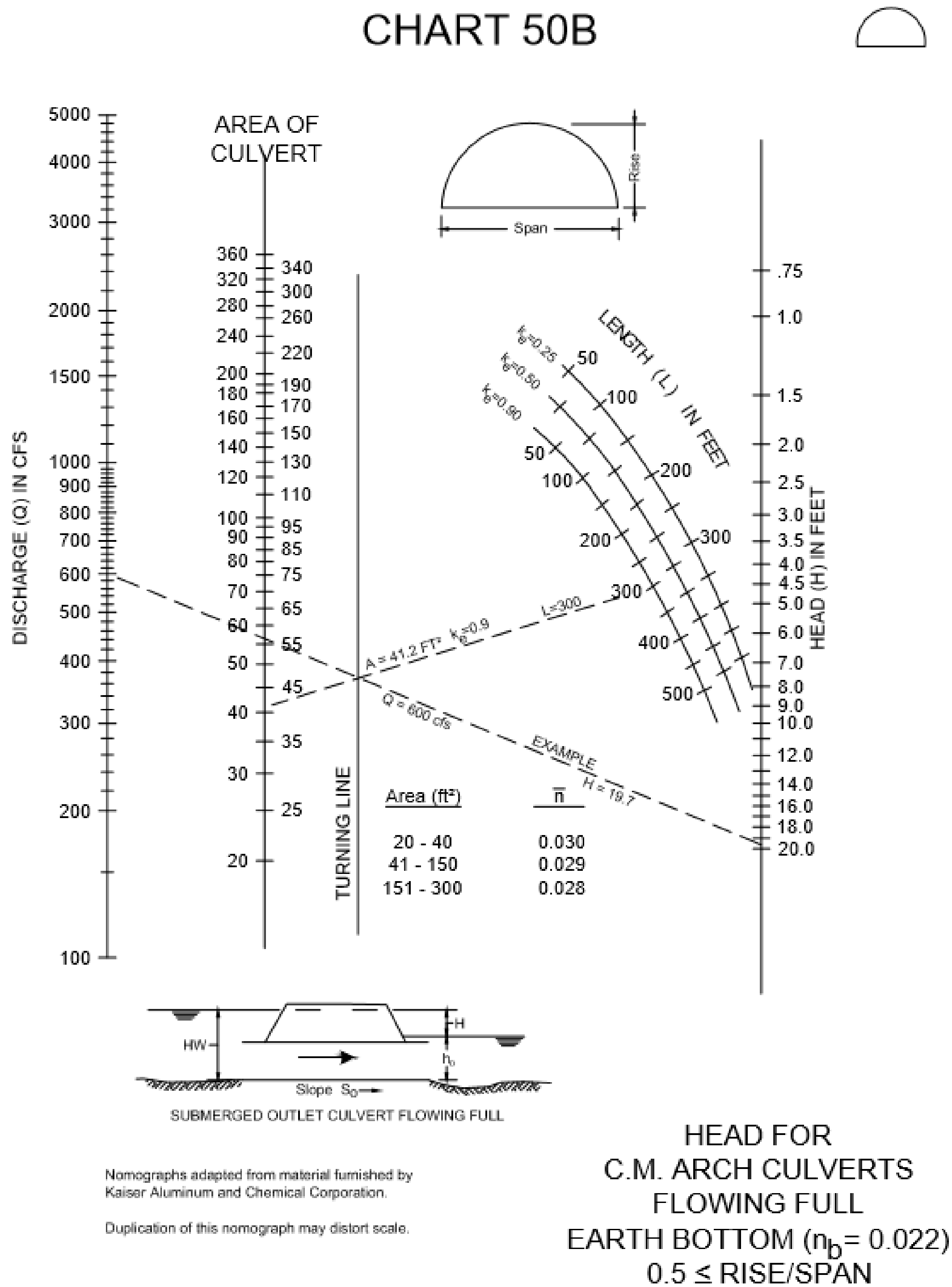


Figure E-38. Corrugated Metal Arch Culvert Nomograph (Chart 50B)

THIS PAGE INTENTIONALLY LEFT BLANK FOR DOUBLE-SIDED PRINTING