

## INTRODUCTION

Many millions of dollars are spent annually on culverts, storm drains and subdrains, all vital to the protection of streets, highways and railroads. If inadequately sized, they can jeopardize the roadway and cause excessive property damage and loss of life. Over design means extravagance. Engineering can find an economical solution.

Topography, soil and climate are extremely variable, so drainage sites should be designed individually from reasonably adequate data for each particular site. In addition, the designer is advised to consult with those responsible for maintaining drainage structures in the area. One highway engineer comments:

"With the exception of the riding qualities of the traveled way, no other single item requires as much attention on the part of maintenance personnel as highway culverts. Many of the problems of culvert maintenance stem from the fact that designers in all too many instances consider that culverts will be required to transport only clear water. This is a condition hardly ever realized in practice, and in many instances storm waters may be carrying as much as 50 percent detrimental material. A rapid change in grade line at the culvert entrance can cause complete blockage of the culvert. This results in overflow across the highway and in some cases, especially where high fills are involved, the intense static pressure results in loss of the embankment."

## HYDRAULICS OF OPEN DRAINAGE CHANNELS

### General

Before designing culverts, storm sewers and other drainage structures, one should consider the design of ditches, gutters, chutes, median swales, and other channels leading to these structures. (See Figure 4.1).

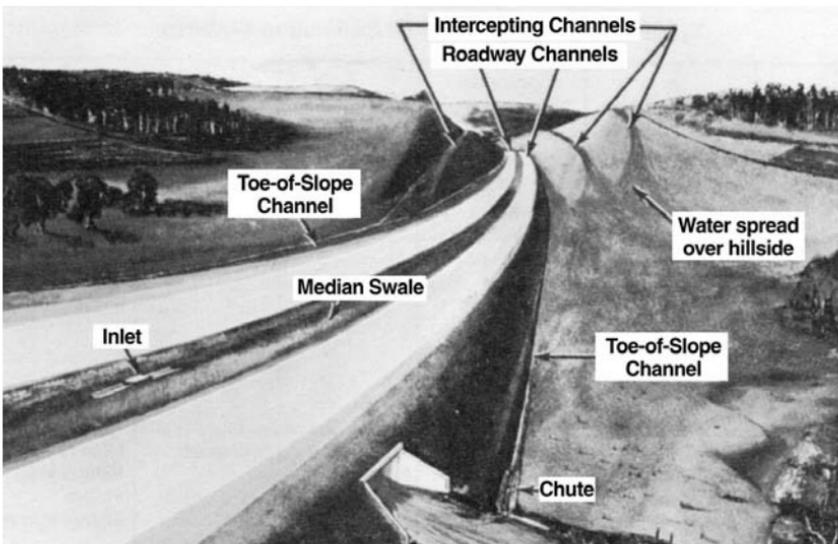


Figure 4.1 Types of roadside drainage channels.

The design engineer with needs beyond the scope of this handbook may refer to the CSPI publication, "Modern Sewer Design" and AISI "Design Charts for Open Channel Flow". These include numerous examples of calculations and references on all aspects of the subject.

Rainfall and runoff, once calculated, are followed by the design of suitable channels to handle the peak discharge with minimum erosion, maintenance and hazard to traffic.

The AASHTO publication "A Policy on Geometric Design of Highway and Streets" states: "The depth of channels should be sufficient to remove the water without saturation of the pavement subgrade. The depth of water that can be tolerated, particularly on flat channel slopes, depends upon the soil characteristics. In open country, channel side slopes of 5:1 or 6:1 are preferable in order to reduce snow drifts."

Systematic maintenance is recognized as essential to any drainage channel and therefore should be considered in the design of those channels.

### Chezy Equation

Chezy developed a basic hydraulic formula for determining the flow of water, particularly in open channels. It is as follows:

$$Q = AV$$

if:  $V = c \sqrt{RS}$

then:  $Q = Ac \sqrt{RS}$

where:  $Q =$  discharge,  $m^3/s$

$A =$  cross-sectional area of flow,  $m^2$

$V =$  mean velocity of flow,  $m/s$

$c =$  coefficient of roughness, depending upon the surface over which water is flowing,  $m^{1/2}/s$

$R =$  hydraulic radius,  $m$

$$= \frac{A}{WP}$$

$WP =$  wetted perimeter (length of wetted contact between water and its containing channel),  $m$

$S =$  slope, or grade,  $m/m$

This fundamental formula is the basis of most capacity formulas.

### Manning's Equation

Manning's equation, published in 1890, gives the value of  $c$  in the Chezy formula as:

$$c = \frac{R^{1/6}}{n}$$

where:  $n =$  coefficient of roughness (see Tables 4.1 and 4.2)

**Table 4.1** Manning's  $n$  for constructed channels

Type of channel and description	$n$
1. LINED OR BUILT-UP	
A. Concrete - Trowel Finish . . . . .	0.013
B. Concrete - Float Finish . . . . .	0.015
C. Concrete - Unfinished . . . . .	0.017
D. Gunite - Good Section . . . . .	0.019
E. Gravel Bottom with sides of:	
1) Formed Concrete . . . . .	0.020
2) Random Stone in Mortar . . . . .	0.023
3) Dry Rubble or Rip Rap . . . . .	0.033
2. EXCAVATED OR DREDGED - EARTH	
A. Straight and Uniform	
1) Clean, Recently Completed . . . . .	0.018
2) Clean, After Weathering . . . . .	0.022
3) Gravel, Uniform Section, Clean . . . . .	0.025
4) With Short Grass, Few Weeds . . . . .	0.027
B. Winding and Sluggish	
1) No Vegetation . . . . .	0.025
2) Grass, Some Weeds . . . . .	0.030
3) Dense Weeds, Deep Channels . . . . .	0.035
4) Earth Bottom and Rubble Sides . . . . .	0.030
5) Stony Bottom and Weedy Banks . . . . .	0.035
6) Cobble Bottom and Clean Sides . . . . .	0.040
3. CHANNELS NOT MAINTAINED, WEEDS & BRUSH UNCUT	
A. Dense Weeds, High as Flow Depth . . . . .	0.080
B. Clean Bottom, Brush on Sides . . . . .	0.050
C. Same, Highest Stage of Flow . . . . .	0.070
D. Dense Brush, High Stage . . . . .	0.100

**Table 4.2** Manning's  $n$  for natural stream channels  
Surface width at flood stage less than 30 m

1. Fairly regular section:	
a. Some grass and weeds, little or no brush . . . . .	0.030–0.035
b. Dense growth of weeds, depth of flow materially greater than weed height . . . . .	0.035–0.05
c. Some weeds, light brush on banks . . . . .	0.035–0.05
d. Some weeds, heavy brush on banks . . . . .	0.05–0.07
e. Some weeds, dense willows on banks . . . . .	0.06–0.08
f. For trees within channel, with branches submerged at high stage, increase all above values by . . . . .	0.01–0.02
2. Irregular sections, with pools, slight channel meander; increase values given above about . . . .	0.01–0.02
3. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stage:	
a. Bottom of gravel, cobbles, and few boulders . . . . .	0.04–0.05
b. Bottom of cobbles, with large boulders . . . . .	0.05–0.07

The complete Manning equation is:

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

Combining this with the Chezy Equation results in the equation:

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

In many calculations, it is convenient to group the channel cross section properties in one term called conveyance,  $K$ , so that:

$$K = \frac{AR^{2/3}}{n}$$

Then:

$$Q = KS^{1/2}$$

Uniform flow of clean water in a straight unobstructed channel would be a simple problem but is rarely attained. Manning's formula gives reliable results if the channel cross section, roughness, and slope are fairly constant over a sufficient distance to establish uniform flow.

### The Use of Charts and Tables

While design charts for open-channel flow reduce computational effort, they cannot replace engineering judgment and a knowledge of the hydraulics of open-channel flow and flow through conduits with a free water surface.

Design charts contain the channel properties (area and hydraulic radius) of many channel sections and tables of velocity for various combinations of slope and hydraulic radius. Their use is explained in the following examples.

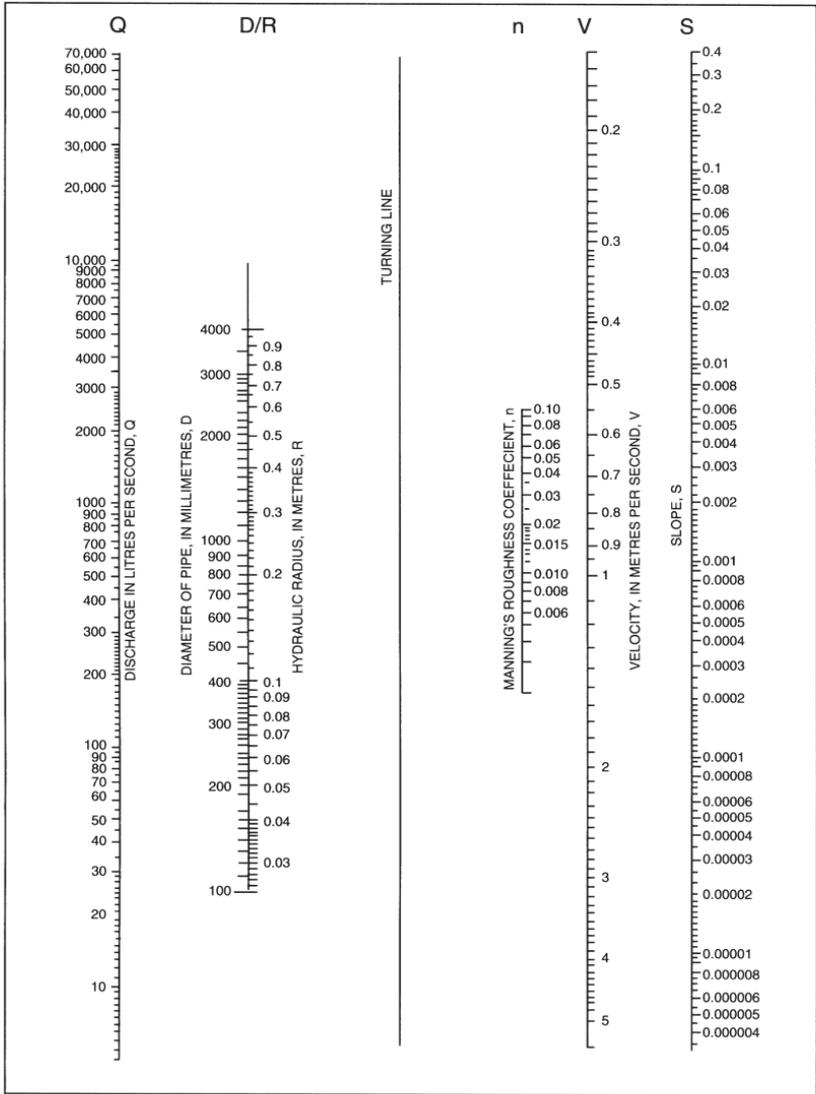
#### Example 1

*Given:* A trapezoidal channel of straight alignment and uniform cross section in earth with a bottom width of 0.6 m, side slopes at 1:1, a channel slope of 0.003 m/m, and a normal depth of water of 0.3 m.

*Find:* Velocity and discharge.

*Solution:*

1. Based on Table 4.1, for an excavated channel in ordinary earth,  $n$  is taken as 0.022.
2. Cross-sectional area,  $A$ , is  $0.27 \text{ m}^2$  [ $0.3 * (0.6 + 1 * 0.3)$ ].
3. Wetted perimeter,  $WP$ , is  $1.449 \text{ m}$  [ $0.6 + 2 * 0.3 * (1^2+1)^{1/2}$ ].
4. Hydraulic radius,  $R$ , is  $0.186 \text{ m}$  [ $0.27 / 1.449$ ].
5. Using the nomograph in Figure 4.2, lay a straight edge between the outer scales at the values of  $S = 0.003$  and  $n = 0.02$ . Mark where the straight edge intersects the turning line.
6. Place the straight edge to line up the point on the turning line and the hydraulic radius of  $0.186 \text{ m}$ .
7. Read the velocity,  $V$ , of  $0.80 \text{ m/s}$  on the velocity scale.
8. Discharge,  $Q$ , is  $0.216 \text{ m}^3/\text{s}$  [ $0.27 * 0.89$ ].



Alignment chart for energy loss in pipes, for Manning's formula.  
 Note: Use chart for flow computations,  $H_L = S$

**Figure 4.2** Nomograph for solution of Manning's equation.

Figure 4.3 provides the means to calculate a trapezoidal channel capacity for a specific bottom width, channel slope, side slope, n value and a variety of flow depths. For a given drainage project, these variables are known or determined using known site parameters through trial and error. The flow rate, Q, can then be calculated.

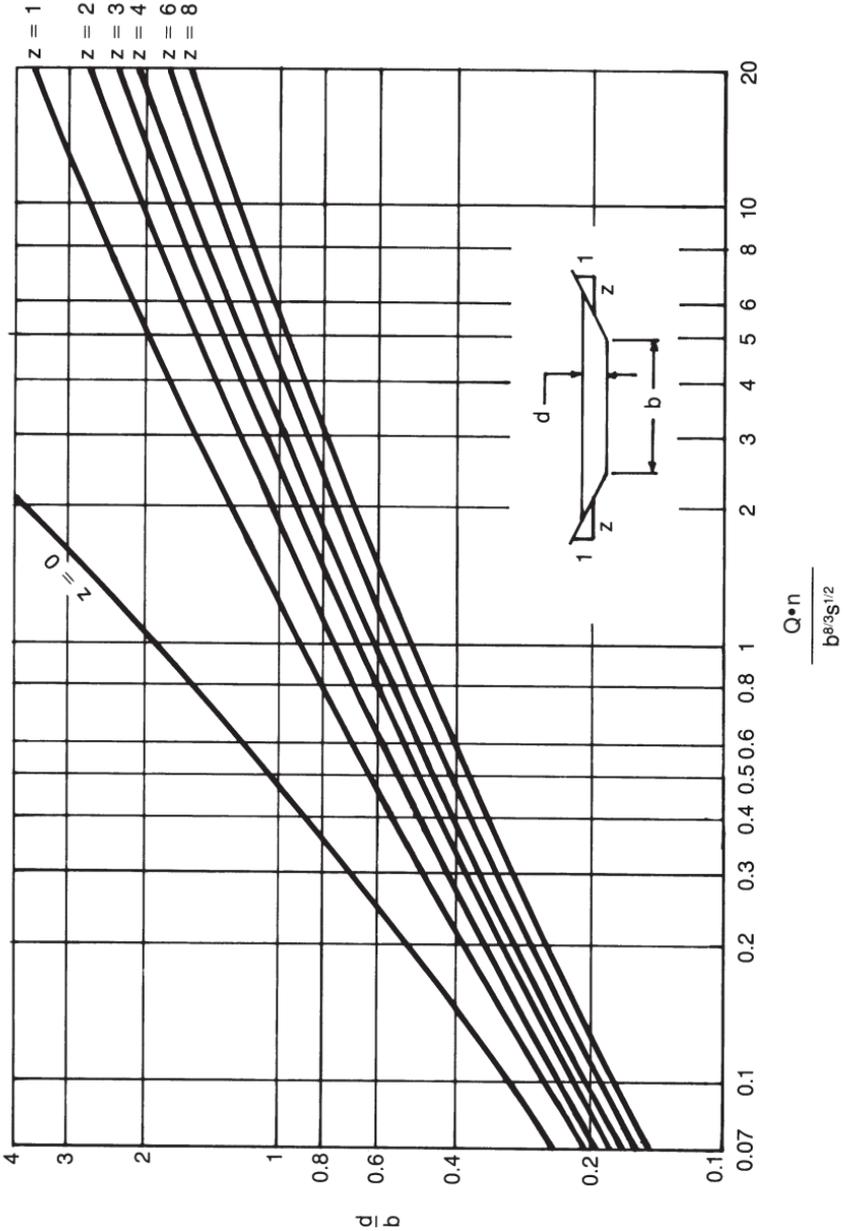


Figure 4.3 Capacity of trapezoidal channel.

**Example 2**

*Given:* Bottom width,  $b = 6.1$  m  
 Side slopes @ 2:1, so  $z = 2$   
 Roughness coefficient,  $n = 0.030$  (from Table 4.2 for grass and weeds, no brush)  
 Channel slope,  $S = 0.002$  m/m

Depth to width ratio,  $\frac{d}{b} = 0.6$  (flood stage depth)

*Find:* Depth of flow,  $d$ , and flow rate,  $Q$ .

*Solution:*

Depth,  $d = 0.6 (6.1) = 3.66$  m

From Figure 4.3:  $\frac{Q \cdot n}{b^{8/3} S^{1/2}} = 0.62$

So:  $\frac{Q (0.030)}{(6.1)^{8/3} (0.002)^{1/2}} = 0.62$

And:  $Q = 114.8 \text{ m}^3/\text{s}$

If the resulting design is not satisfactory, the channel parameters are adjusted and the design calculations are repeated.

**Safe Velocities**

The ideal situation is one where the velocity will cause neither silt deposition nor erosion. For the design of a channel, the approximate grade can be determined from a topographic map, from the plan profiles, or from both.

To prevent the deposition of sediment, the minimum gradient for earth and grass-lined channels should be about 0.5 percent and that for smooth paved channels about 0.35 percent.

Convenient guidelines for permissible velocities are provided in Tables 4.3 and 4.4. More comprehensive design data may be found in the U.S. FHWA's HEC 15 (Design of Stable Channels with Flexible Linings).

**Channel Protection**

Corrugated steel flumes or chutes (and pipe spillways) are favored solutions for channel protection especially in wet, unstable or frost susceptible soils. They should be anchored to prevent undue shifting. This will also protect against buoyancy and uplift, which can occur especially when empty. Cutoff walls or collars are used to prevent undermining.

If the mean velocity exceeds the permissible velocity for the particular type of soil, the channel should be protected from erosion. Grass linings are valuable where grass growth can be supported. Ditch bottoms may be sodded or seeded with the aid of temporary quick growing grasses, mulches, or erosion control blankets. Grass may also be used in combination with other, more rigid types of linings, where the grass is on the upper bank slopes and the rigid lining is on the channel bottom. Linings may consist of stone which is dumped, hand placed or grouted, preferably laid on a filter blanket of gravel or crushed stone and a geotextile.

**Table 4.3** Comparison of limiting water velocities and tractive force values for the design of stable channels (straight channels after aging; channel depth = 1m)

Material	n	For Clear Water		Water Transporting Colloidal Silts	
		Velocity, m/s	Tractive Force, Pa	Velocity, m/s	Tractive Force, Pa
Fine sand colloidal	0.020	0.46	1.29	0.76	3.59
Sandy loam noncolloidal	0.020	0.53	1.77	0.78	3.59
Silt loam noncolloidal	0.020	0.61	2.30	0.91	5.27
Alluvial silts noncolloidal	0.020	0.61	2.30	1.07	7.18
Ordinary firm loam	0.020	0.76	3.59	1.07	7.18
Volcanic ash	0.020	0.76	3.59	1.07	7.18
Stiff clay very colloidal	0.025	1.14	12.45	1.52	22.02
Alluvial silts colloidal	0.025	1.14	12.45	1.52	22.02
Shales and hardpans	0.025	1.83	32.08	1.83	32.08
Fine gravel	0.020	0.76	3.59	1.52	15.32
Graded loam to cobbles when non-colloidal	0.030	1.14	18.19	1.52	31.60
Graded silts to cobbles when colloidal	0.030	1.22	20.59	1.68	38.30
Coarse gravel non-colloidal	0.025	1.22	14.36	1.83	32.08
Cobbles and shingles	0.035	1.52	43.57	1.68	52.67

**Table 4.4** Maximum permissible velocities in vegetal-lined channels<sup>d</sup>

Cover Average, Uniform Stand, Well Maintained	Slope Range	Permissible Velocity <sup>a</sup>		
		Erosion Resistant Soils	Easily Eroded Soils	
		Percent	m/s	m/s
Bermudagrass	0-5		2.44	1.83
	5-10		2.13	1.52
	over 10		1.83	1.22
Buffalograss Kentucky bluegrass Smooth brome Blue grama	0-5		2.13	1.52
	5-10		1.83	1.22
	over 10		1.52	0.91
Grass mixture <sup>b</sup>	0-5		1.52	1.22
	5-10		1.22	0.91
Lespedeza sericea Weeping lovegrass Yellow bluestem Kudzu Alfalfa Crabgrass	0-5		1.07	0.76
Common lespedeza <sup>b</sup> Sudangrass <sup>b</sup>	0-5 <sup>c</sup>		1.07	0.76

<sup>a</sup> From "Handbook of Channel Design for Soil and Water Conservation," Soil Conservation Service SCS-TP-61, Revised June 1954

<sup>b</sup> Annuals-used on mild slopes or as temporary protection until permanent covers are established.

<sup>c</sup> Use on slopes steeper than 5 percent is not recommended.

<sup>d</sup> Data for this table is a composite of data from several reference sources.

Asphalt and concrete lined channels are used for steep erodible channels.

Ditch checks are an effective means of decreasing the velocity and thereby the erodability of the soil.

High velocities, where water discharges from a channel, must be considered and provisions must be made to dissipate the excess energy.

## HYDRAULICS OF CULVERTS

### Introduction

Culvert design has not yet reached the stage where two or more individuals will always arrive at the same answer, or where actual service performance matches the designer's expectation. The engineer's interpretation of field data and hydrology is often influenced by personal judgement, based on experience in a given locality. However, hydrology and hydraulic research are closing the gap to move the art of designing a culvert closer to becoming a science.

Up to this point, the design procedure has consisted of (1) collecting field data, (2) compiling facts about the roadway, and (3) making a reasonable estimate of flood discharge. The next step is to design an economical corrugated steel structure to handle the flow (including debris) with minimum damage to the slope or culvert barrel. Treatment of the inlet and outlet ends of the structure must also be considered.



Improving hydraulic capacity (inlet control) with special features.

### What Makes a Good Culvert?

An ASCE Task Force on Hydraulics of Culverts offers the following recommendations for "Attributes of a Good Highway Culvert":

1. The culvert, appurtenant entrance and outlet structures should properly take care of water, bed-load, and floating debris at all stages of flow.
2. It should cause no unnecessary or excessive property damage.
3. Normally, it should provide for transportation of material without detrimental change in flow pattern above and below the structure.

4. It should be designed so that future channel and highway improvement can be made without too much loss or difficulty.
5. It should be designed to function properly after fill has caused settlement.
6. It should not cause objectionable stagnant pools in which mosquitoes may breed.
7. It should be designed to accommodate increased runoff occasioned by anticipated land development.
8. It should be economical to build, hydraulically adequate to handle design discharge, structurally durable and easy to maintain.
9. It should be designed to avoid excessive ponding at the entrance which may cause property damage, accumulation of drift, culvert clogging, saturation of fills, or detrimental upstream deposits of debris.
10. Entrance structures should be designed to screen out material which will not pass through the culvert, reduce entrance losses to a minimum, make use of the velocity of approach in so far as practicable, and by use of transitions and increased slopes, as necessary, facilitate channel flow entering the culvert.
11. The design of the culvert outlet should be effective in re-establishing tolerable non-erosive channel flow within the right-of-way or within a reasonably short distance below the culvert.



CSP structure ready for backfill placement and headwalls.

12. The outlet should be designed to resist undermining and washout.
13. Energy dissipaters, if used, should be simple, easy to build, economical and reasonably self-cleaning during periods of easy flow.

### Design Method

The culvert design process should strive for a balanced result. Pure fluid mechanics should be combined with practical considerations to help assure satisfactory performance under actual field conditions. This includes due consideration of prospective maintenance and the handling of debris.

The California Division of Highways uses an excellent method of accomplishing this; one that has worked well for many years. Other jurisdictions have used similar approaches. California culvert design practice establishes the following:

Criteria for Balanced Design:

The culvert shall be designed to discharge

- a) a 10 year flood without static head at the entrance, and
- b) a 100 year flood utilizing the available head at the entrance.

This approach lends itself well to most modern design processes and computer programs. It provides a usable rationale for determining a minimum required waterway area.

The permissible height of water at the inlet controls hydraulic design. This should be determined and specified for each site based on the following considerations:

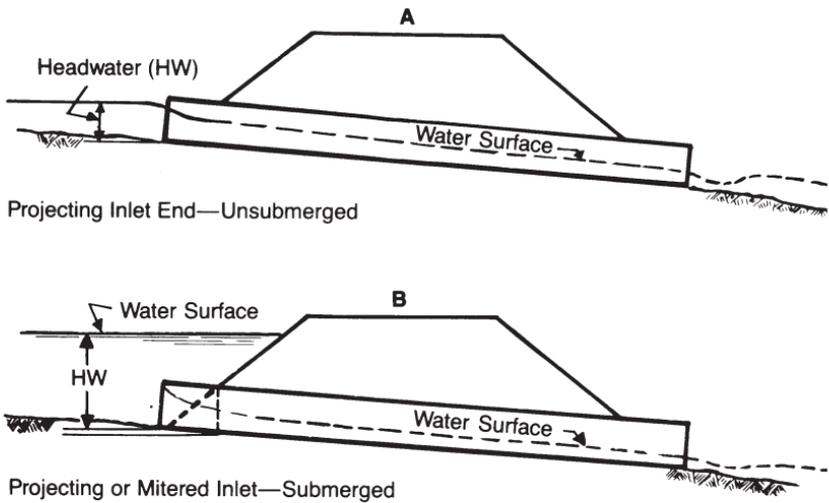
1. Risk of overtopping the embankment and the resulting risk to human life.
2. Potential damage to the roadway, due to saturation of the embankment, and pavement disruption due to freeze-thaw.
3. Traffic interruptions.
4. Damage to adjacent or upstream property, or to the channel or flood plain environment.
5. Intolerable discharge velocities, which can result in scour and erosion.
6. Deposition of bed load and/or clogging by debris on recession of flow.

### Flow Conditions and Definitions

Culverts considered here are circular pipes and pipe-arches with a uniform barrel cross-section throughout.

There are two major types of culvert flow conditions:

**Inlet Control** – A culvert flowing in inlet control is characterized by shallow, high velocity flow categorized as supercritical. Inlet control flow occurs when the culvert barrel is capable of conveying more flow than the inlet will accept. The control section is near the inlet, and the downstream pipe and flow have no impact on the amount of flow through the pipe. Under inlet control, the factors of primary importance are (1) the cross-sectional area of the barrel, (2) the inlet configuration or geometry, and (3) the headwater elevation or the amount of ponding upstream of the inlet (see Figure 4.4). The barrel slope also influences the flow under inlet control, but the effect is small and it can be ignored.



**Figure 4.4** Inlet control flow regimes.

Outlet Control – A culvert flowing in outlet control is characterized by relatively deep, lower velocity flow categorized as subcritical. Outlet control flow occurs when the culvert barrel is not capable of conveying as much flow as the inlet opening will accept. The control section is at the outlet of the culvert. In addition to the factors considered for inlet control, factors that must be considered for outlet control include (1) the tailwater elevation in the outlet channel, (2) the barrel slope, (3) the barrel roughness, and (4) the length of the barrel (see Figure 4.5).

### Hydraulics of Culverts in Inlet Control

Inlet control means that the discharge capacity is controlled at the entrance by the headwater depth, cross-sectional area and type of inlet edge. The roughness, length, and outlet conditions are not factors in determining the culvert capacity.

Sketches A and B in Figure 4.4 show unsubmerged and submerged projecting inlets. Inlet control performance is classified by these two regions (unsubmerged flow and submerged flow) as well as a transition region between them.

Entrance loss depends upon the geometry of the inlet edge and is expressed as a fraction of the velocity head. Research with models and prototype testing have resulted in coefficients for various types of inlets, as shown in Table 4.5 and Figure 4.6.

**Table 4.5** Entrance loss coefficients for corrugated steel pipe or pipe-arch

Inlet End of Culvert	Entrance Type	Coefficient, $k_g$
Projecting from fill (no headwall)	1	0.9
Mitered (bevelled) to conform to fill slope	2	0.7
Headwall or headwall and wingwalls square-edge	3	0.5
End-Section conforming to fill slope	4	0.5
Headwall rounded edge	5	0.2
Bevelled Ring	6	0.25

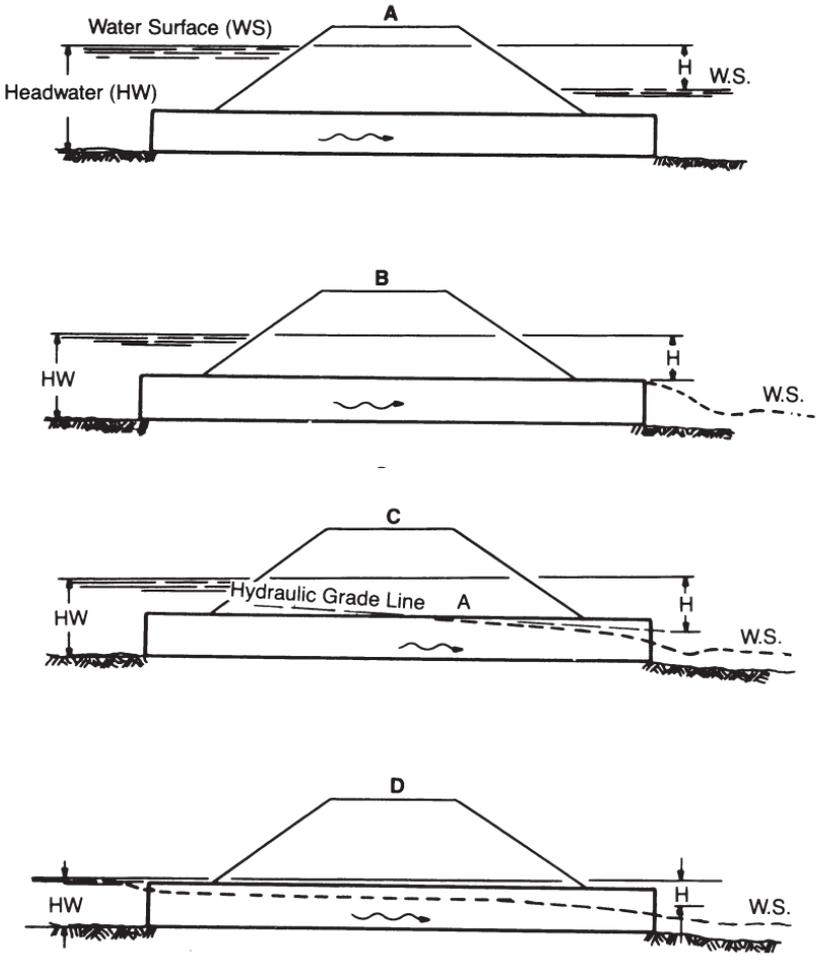
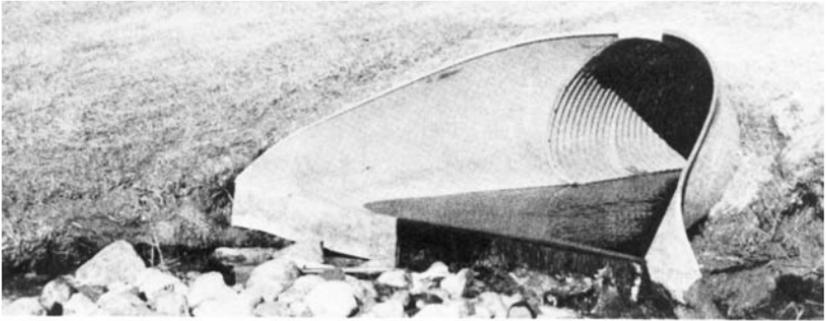
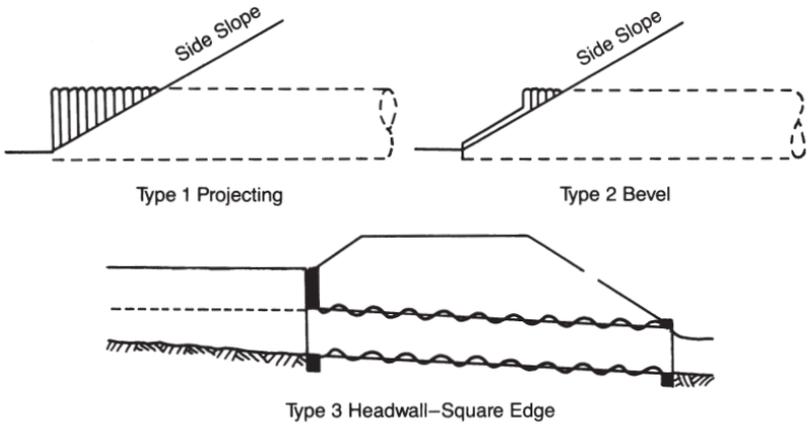
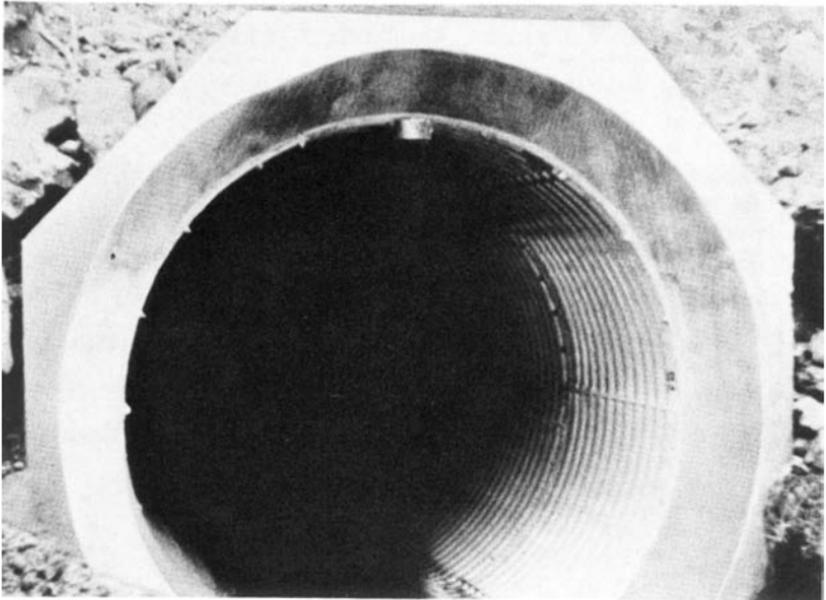


Figure 4.5 Outlet control flow regimes.



Type 4 End Section



Type 6 Bevelled Ring

Figure 4.6 Typical inlet entrances.

The model testing and prototype measurements also provide information used to develop equations for unsubmerged and submerged inlet control flow. The transition zone is poorly defined, but it is approximated by plotting the two flow equations and connecting them with a line which is tangent to both curves. These plots, done for a variety of structure sizes, are the basis for constructing the design nomographs included in this handbook.

In the nomographs, the headwater depth, HW, is the vertical distance from the culvert invert (bottom) at the entrance to the energy grade line of the headwater pool. It, therefore, includes the approach velocity head. The velocity head tends to be relatively small and is often neglected. The resulting headwater depth is therefore conservative and the actual headwater depth would be slightly less than the calculated value. If a more accurate headwater depth is required, the approach velocity head should be subtracted from the headwater depth determined using the nomographs.

### Hydraulics of Culverts in Outlet Control

Outlet control means that the discharge capacity is controlled at the outlet by the tailwater depth or critical depth, and it is influenced by such factors as the slope, wall roughness and length of the culvert. The following energy balance equation contains the variables that influence the flow through culverts flowing under outlet control:

$$L \cdot S_o + HW + \frac{V_1^2}{2g} = h_o + H + \frac{V_2^2}{2g}$$

where: L = length of culvert, m  
 $S_o$  = slope of barrel, m/m  
 HW = headwater depth, m  
 $V_1$  = approach velocity, m/s  
 $g$  = gravitational constant = 9.806 m/s<sup>2</sup>  
 $h_o$  = outlet datum, m  
 H = head, m  
 $V_2$  = downstream velocity, m/s

The headwater depth, HW, is the vertical distance from the culvert invert at the entrance (where the entrance is that point in the pipe where there is the first full cross-section) to the surface of the headwater pool.

As discussed under inlet control hydraulics, the water surface and energy grade line are usually assumed to coincide at the entrance (the approach velocity head is ignored). The same can be said for the downstream velocity head. That being the case, the approach velocity head and downstream velocity head terms in the above equation would be dropped and the equation would take the form below. Note that this equation has been organized to provide the resulting headwater depth.

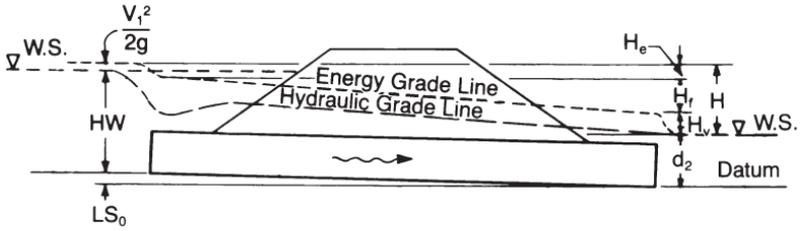
$$HW = h_o + H - L \cdot S_o$$

The head, or energy (Figures 4.5 through 4.9), required to pass a given quantity of water through a culvert flowing in outlet control, is made up of a (1) entrance loss, (2) friction loss, and (3) exit loss.

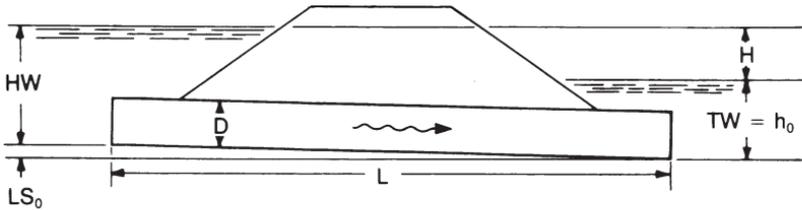
This energy is expressed in equation form as:

$$H = H_e + H_f + H_o$$

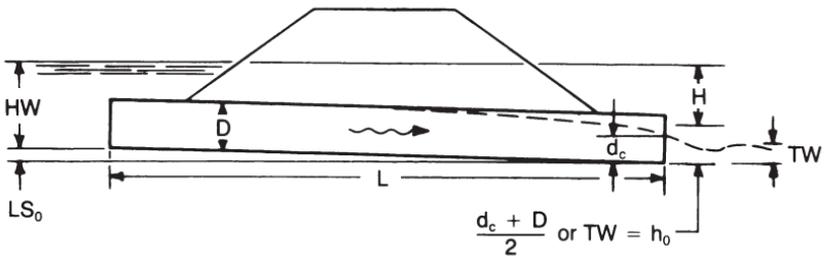
where:  $H_e$  = entrance loss, m  
 $H_f$  = friction loss, m  
 $H_o$  = exit loss, m



**Figure 4.7** Difference between energy grade line and hydraulic grade line.



**Figure 4.8** Relationship of headwater to high tailwater.



**Figure 4.9** Relationship of headwater to low tailwater.

The hydraulic slope, or hydraulic grade line, sometimes called the pressure line, is defined by the elevations to which water would rise in small vertical pipes attached to the culvert wall along its length (see Figure 4.7). For full flow, the energy grade line and hydraulic grade line are parallel over the length of the barrel except in the vicinity of the inlet where the flow contracts and re-expands. The difference between the energy grade line and hydraulic grade line is the velocity head. It turns out that the velocity head is a common variable in the expressions for entrance, friction and exit loss.

The velocity head is expressed by the following equation:

$$H_v = \frac{V^2}{2g}$$

- where:  $H_v$  = velocity head, m  
 $V$  = mean velocity of flow in the barrel, m/s =  $\frac{Q}{A}$   
 $Q$  = design discharge, m<sup>3</sup>/s  
 $A$  = cross sectional area of the culvert, m<sup>2</sup>

The entrance loss depends upon the geometry of the inlet. This loss is expressed as an entrance loss coefficient multiplied by the velocity head, or:

$$H_e = k_e \frac{V^2}{2g}$$

where:  $k_e$  = entrance loss coefficient (Table 4.5)

The friction loss is the energy required to overcome the roughness of the culvert barrel and is expressed by the following equation:

$$H_f = \left( \frac{2gn^2L}{R^{1.33}} \right) \frac{V^2}{2g}$$

where:  $n$  = Manning's friction factor (see Tables 4.6 and 4.7)

$R$  = hydraulic radius,  $m = \frac{A}{WP}$

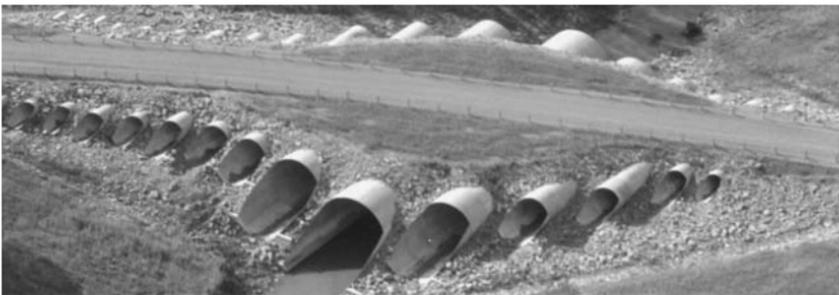
$WP$  = wetted perimeter,  $m$

**Table 4.6** Values of Manning's  $n$  for corrugated and spiral rib steel pipe

	Annular 68 x 13 mm	Helical								
		38 x 6.5 mm		68 x 13 mm						1400 & Larger
		200	250	300	400	500	600	900	1200	
Unpaved	0.024	0.012	0.011	0.013	0.014	0.015	0.016	0.018	0.020	0.021
25 % Paved	0.021						0.014	0.017	0.020	0.019
Fully Paved	0.012						0.012	0.012	0.012	0.012
	Annular 76 x 25 mm All Diameters	Helical-76 x 25 mm							2200 & Larger	
		1200	1400	1600	1800	2000				
		Unpaved	0.027	0.023	0.023	0.024	0.025	0.026	0.027	
25 % Paved	0.023	0.020	0.020	0.021	0.022	0.022	0.023			
Fully Paved	0.012	0.012	0.012	0.012	0.012	0.012	0.012			
	Annular 125 x 25 mm All Diameters	Helical-125 x 25 mm				2000 & Larger				
		1400	1600	1800						
		Unpaved	0.025	0.022	0.023	0.024	0.025			
25 % Paved	0.022	0.019	0.020	0.021	0.022					
Fully Paved	0.012	0.012	0.012	0.012	0.012					

Spiral Rib Pipe - all diameters Manning's  $n = .013$

Note: \*\* When helically corrugated steel pipe is used for air conduction, the Darcy-Weisbach Formula with other values of  $F$  (or  $n$ ) is used.



Combination stream crossing and voids to reduce dead load on foundation soils. (Ontario Ministry of Transportation project.)

**Table 4.7** Values of Manning's  $n$  for 152 mm x 51 mm corrugation structural plate pipe

	Diameters			
	1500 mm	2120 mm	3050 mm	4610 mm
Plain-unpaved	0.033	0.032	0.030	0.028
25 % Paved	0.028	0.027	0.026	0.024

The exit loss depends on the change in velocity at the outlet of the culvert. For a sudden expansion, the exit loss is expressed as:

$$H_o = 1.0 \left[ \frac{V^2}{2g} - \frac{V_2^2}{2g} \right]$$

As discussed previously, the downstream velocity head is usually neglected, in which case the above equation becomes the equation for the velocity head:

$$H_o = H_v = \frac{V^2}{2g}$$

Substituting in the equation for head we get (for full flow):

$$H = \left\{ k_e + \frac{2gn^2L}{R^{1.33}} + 1 \right\} \frac{V^2}{2g}$$

Nomographs have been developed and can be used for solving this equation. Note that these nomographs provide the head, whereas the inlet control nomographs provide the headwater depth. The head is then used to calculate the headwater depth by solving the preceding equation for HW (including the terms of  $h_o$  and  $L \cdot S_o$ ).

This equation was developed for the full flow condition, which is as shown in Figure 4.5 A. It is also applicable to the flow condition shown in Figure 4.5 B.

Backwater calculations are required for the partly full flow conditions shown in Figure 4.5 C and D. These calculations begin at the downstream water surface and proceed upstream to the entrance of the culvert and the headwater surface. The downstream water surface is based on the greater of the critical depth or the tailwater depth.

The backwater calculations can be tedious and time consuming. Approximate methods have therefore been developed for the analysis of partly full flow conditions. Backwater calculations have shown that a downstream extension of the full flow hydraulic grade line, for the flow condition shown in Figure 4.5 C, intersects the plane of the culvert outlet cross section at a point half way between the critical depth and the top of the culvert. This is more easily envisioned as shown in Figure 4.9. It is possible, then, to begin the hydraulic grade line at that datum point and extend the straight, full flow hydraulic grade line to the inlet of the culvert. The slope of the hydraulic grade line is the full flow friction slope:

$$S_n = \frac{H_f}{L} = \left\{ \frac{2gn^2}{R^{1.33}} \right\} \frac{V^2}{2g}$$

If the tailwater elevation exceeds the datum point described above, the tailwater depth is used instead as the downstream starting point for the full flow hydraulic grade line.

The headwater depth is calculated by adding the inlet losses to the elevation of the hydraulic grade line at the inlet.

This approximate method works best when the culvert is flowing full for at least part of its length, as shown in Figure 4.5 C. If the culvert is flowing partly full for its whole length, as shown in Figure 4.5 D, the results become increasingly inaccurate as the flow depth decreases. The results are usually acceptable down to a headwater depth of about three quarters of the structure rise. For lower headwater depths, backwater calculations are required.

The outlet control nomographs can be used with the approximate method. In this case, the head is added to the datum point elevation to obtain the headwater depth. This method also works best when the culvert is flowing full for part of its length, and the results are not as accurate for a culvert flowing partly full.

### Research on Values of $n$ for Helically Corrugated Steel Pipe

Tests conducted on helically corrugated steel pipe, both round and pipe arch flowing full and part full, demonstrate a lower coefficient of roughness compared to annularly corrugated steel pipe. The roughness coefficient is a function of the corrugation helix angle (angle subtended between corrugation direction and centerline of the corrugated steel pipe), which determines the helically corrugated pipe diameter. A small helix angle associated with small diameter pipe, correlates to a lower roughness coefficient. Similarly, as the helix angle increases with diameter, the roughness coefficient increases, approaching the value associated with annularly corrugated pipe.

Values for 125 x 25 mm corrugations have been based on tests conducted using 152 x 25 mm and subsequently modified for the shorter pitch. Most published values of the coefficient of roughness,  $n$ , are based on experimental work conducted under controlled laboratory conditions using clear or clean water. The test pipe lines are straight with smooth joints. However, design values should take into account the actual construction and service conditions which can vary greatly for different drainage materials. Also, as noted on preceding pages, culvert or storm drain capacity under inlet control flow conditions is not affected by the roughness of pipe material.

### Field Studies on Structural Plate Pipe

Model studies by the U.S. Corps of Engineers, and analyses of the results by the U.S. Federal Highway Administration, have been the basis for friction factors of structural plate pipe for many years. These values, originally shown in the 1967 edition of this Handbook, ranged from 0.0328 for 1500 mm diameter pipe to 0.0302 for 4610 mm pipe.

In 1968, the first full-scale measurements were made on a 457 m long 4300 mm diameter structural plate pipe line in Lake Michigan. These measurements indicated a lower friction factor than those derived from the model studies. As a result, the recommended values of Manning's  $n$  for structural plate pipe of 3050 mm diameter and larger have been modified as shown in Table 4.7. The values for the smaller diameters remain as they were.

## HYDRAULIC COMPUTATIONS

A balanced design approach establishes a minimum opening required to pass a 10 year flood with no ponding.

The 10 year discharge is established from hydrology data. The pipe size required to carry this flow, with no head at the entrance ( $HW/D = 1.0$ ), is determined from nomographs. The designer uses the 10 year discharge to determine the pipe size required for inlet control and for outlet control, and uses whichever is greater. This is typically the minimum required opening size for the culvert.

### Inlet Control

The headwater,  $HW$ , for a given pipe flowing under inlet control can be determined from Figures 4.10 through 4.16. Note that these figures are for arches as well as round pipes and pipe-arches.

These figures are first used to determine the pipe size required so that there is no head at the entrance under a 10 year flood condition. Once a pipe size is chosen, the designer also checks that pipe to determine whether outlet control will govern (as described below), and makes pipe size adjustments accordingly.

The designer then uses the selected pipe size to determine the headwater (for specific entrance conditions) for the 100 year flood discharge under inlet control. If this amount of headwater is acceptable, the chosen size is satisfactory for the full 100 year design discharge under inlet control. If the resulting headwater is too high, a larger size must be selected based on the maximum permissible headwater.

The values from the nomographs give the headwater in terms of a number of pipe rises ( $HW/D$ ). The following formula is used to calculate the headwater depth:

$$HW_i = \frac{HW}{D} \cdot D$$

where:  $HW_i$  = headwater depth under inlet control, m

$\frac{HW}{D}$  = headwater depth in number of pipe rises, from nomograph, m/m

$D$  = diameter of pipe, or rise of arch or pipe-arch, m

### Outlet Control

Figures 4.17 through 4.24 are used, with the pipe size selected for inlet control, to determine the head loss,  $H$ . The head loss is then used in the following equation to determine the headwater depth under outlet control. If the depth computed for outlet control is greater than the depth determined for inlet control, then outlet conditions govern the flow conditions of the culvert and the higher headwater depth applies.

$$HW_o = h_o + H - L \cdot S_o$$

where:  $HW_o$  = headwater depth under outlet control, m

$h_o$  = outlet datum, m; the greater of the tailwater depth,  $TW$ , or  $\frac{(d_c + D)}{2}$

$H$  = head, from nomograph, m

$L$  = length of culvert barrel, m

$S_o$  = slope of culvert barrel, m/m

$TW$  = depth of flow in channel at culvert outlet, m

$d_c$  = critical depth, from Figures 4.25 through 4.28, m

$D$  = diameter of pipe, or rise of arch or pipe-arch, m

Wall roughness factors (Manning’s n), on which the nomographs are based, are stated on each figure. In order to use the nomographs for other values of n, an adjusted value for length, L', is calculated using the formula below. This value is then used on the length scale of the nomograph, rather than the actual culvert length.

$$L' = L \cdot \left(\frac{n'}{n}\right)^2$$

- where L' = adjusted length for use in nomographs, m
- L = actual length, m
- n' = actual value of Manning’s n
- n = value of Manning’s n on which nomograph is based

Values of Manning’s n for standard corrugated steel pipe, which were reported in Table 4.6, are shown for convenience in Table 4.8, together with the corresponding length adjustment factors,  $\left(\frac{n'}{n}\right)^2$ .

**Table 4.8** Length adjustment factors for corrugated steel pipe

Pipe Diameter or span, mm	Roughness Factor n for Helical Corr.*	Length Adjustment Factor $\left(\frac{n'}{n}\right)^2$
300	0.013	0.29
600	0.016	0.44
900	0.018	0.56
1200	0.020	0.70
1400 & Larger	0.021	0.77

\* Other values of roughness, n, are applicable to paved pipe, lined pipe, pipe with 76 x 25 and 125 x 25 corrugations, and spiral rib pipe. See Table 4.6.

Values of Manning’s n for structural plate corrugated steel pipe, which were determined in the 1968 full-scale field measurements and which were reported in Table 4.7, are shown for convenience in Table 4.9, together with the corresponding length adjustment factors,  $\left(\frac{n'}{n}\right)^2$ .

**Table 4.9** Length adjustment factors for 152 mm x 51 mm corrugation structural plate pipe

Pipe Diameter, mm	Roughness Factor		Length Adjustment Factor $\left(\frac{n'}{n}\right)^2$
	Curves based on n =	Actual n' =	
1500	0.0328	0.033	1.0
2120	0.0320	0.032	1.0
3050	0.0311	0.030	0.93
4920	0.0302	0.028	0.86
Pipe-arch Size, mm	Roughness Factor		Length Adjustment Factor $\left(\frac{n'}{n}\right)^2$
	Curves based on n =	Actual n' =	
2060 x 1520	0.0327	0.033	1.0
2590 x 1880	0.0321	0.032	1.0
3400 x 2010	0.0315	0.030	0.91
5050 x 3330	0.0306	0.028	0.84

An appropriate entrance loss curve is used based on the desired entrance condition. Typical values of the entrance loss coefficient,  $k_e$ , for a variety of inlet configurations, are in Table 4.5.

If outlet control governs the capacity of the culvert and the headwater exceeds the maximum allowable value, a larger size pipe can be selected so that an acceptable headwater depth results. In such a case, corrugated steel structures with lower roughness coefficients should be considered. See Table 4.6 for alternatives. A smaller size of paved pipe, a helical pipe or a spiral rib pipe may be satisfactory.

Entrance conditions should also be considered. It may be economical to use a more efficient entrance than originally considered if a pipe size difference results. This can be easily investigated by checking the pipe capacity using other entrance loss coefficient curves.

### Improved Inlets

Culvert capacity may be increased through the use of special inlet designs. The U.S. Federal Highway Administration (FHWA) has developed design methods for these types of structures. While these designs increase the flow, their use has been limited as a result of their cost and the level of knowledge of designers.

### Hydraulic Nomographs

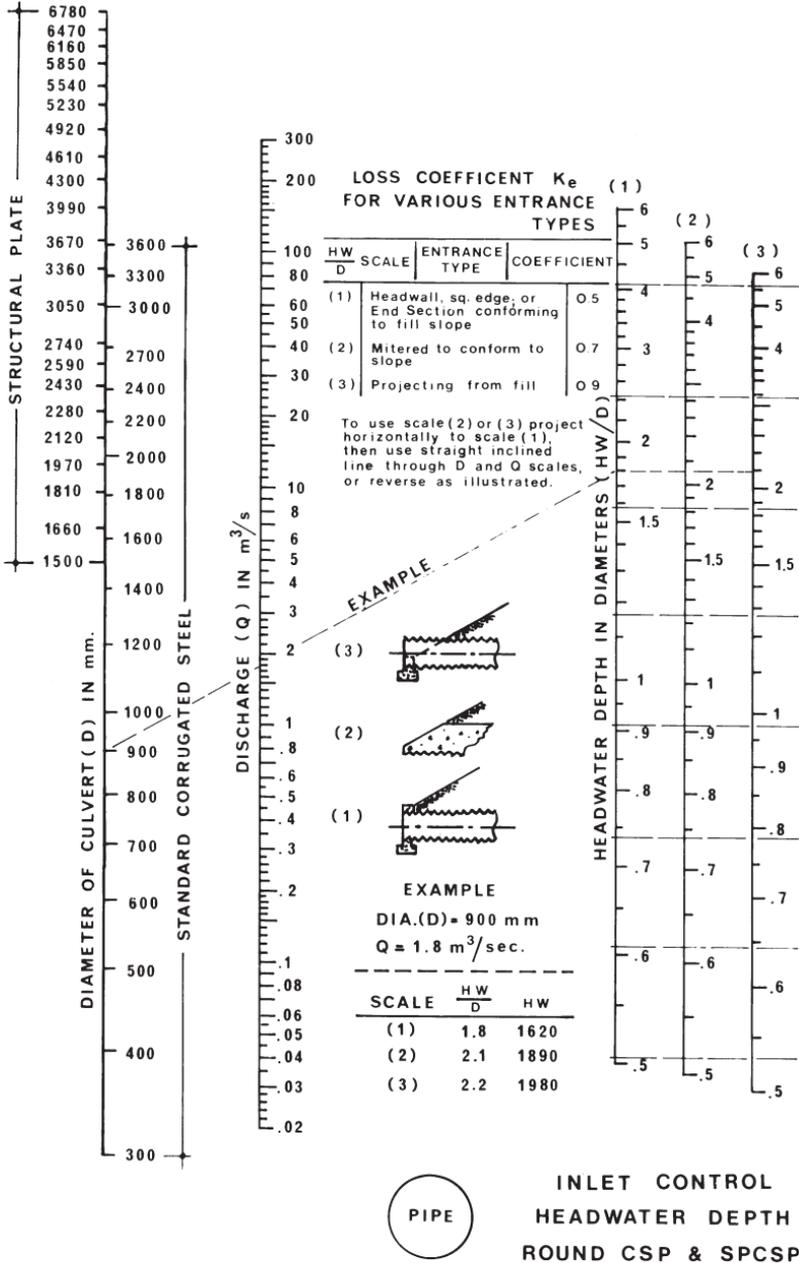
The inlet and outlet control design nomographs which appear in this handbook (Figures 4.10 through 4.24) were reproduced from nomographs developed and published by the FHWA. A certain degree of error is introduced into the design process due to the fact that the construction of nomographs involves graphical fitting techniques resulting in scales which do not exactly match equation results. All of the nomographs used in this handbook have a precision which is better than  $\pm 10$  percent of the equation value in terms of headwater depth (inlet control) or head loss (outlet control). This degree of precision is usually acceptable, especially when considering the degree of accuracy of the hydrologic data. If a structure size is not shown on a particular nomograph, accuracy is not drastically affected when a user interpolates between known points.

### Hydraulic Programs

Numerous computer programs now exist to aid in the design and analysis of highway culverts. These programs possess distinct advantages over traditional hand calculation methods. The increased accuracy of programmed solutions represents a major benefit over the inaccuracies inherent in the construction and use of tables and nomographs. In addition, programmed solutions are less time consuming. This feature allows the designer to compare alternative sizes and inlet configurations very rapidly so that the final culvert selection can be based on economics. Interactive capabilities in some programs can be utilized to change certain input parameters or constraints and analyze their effects on the final design. Familiarity with culvert hydraulics and the traditional analytical methods provides a solid basis for designers to take advantage of the speed, accuracy, and increased capabilities available in culvert hydraulics programs.

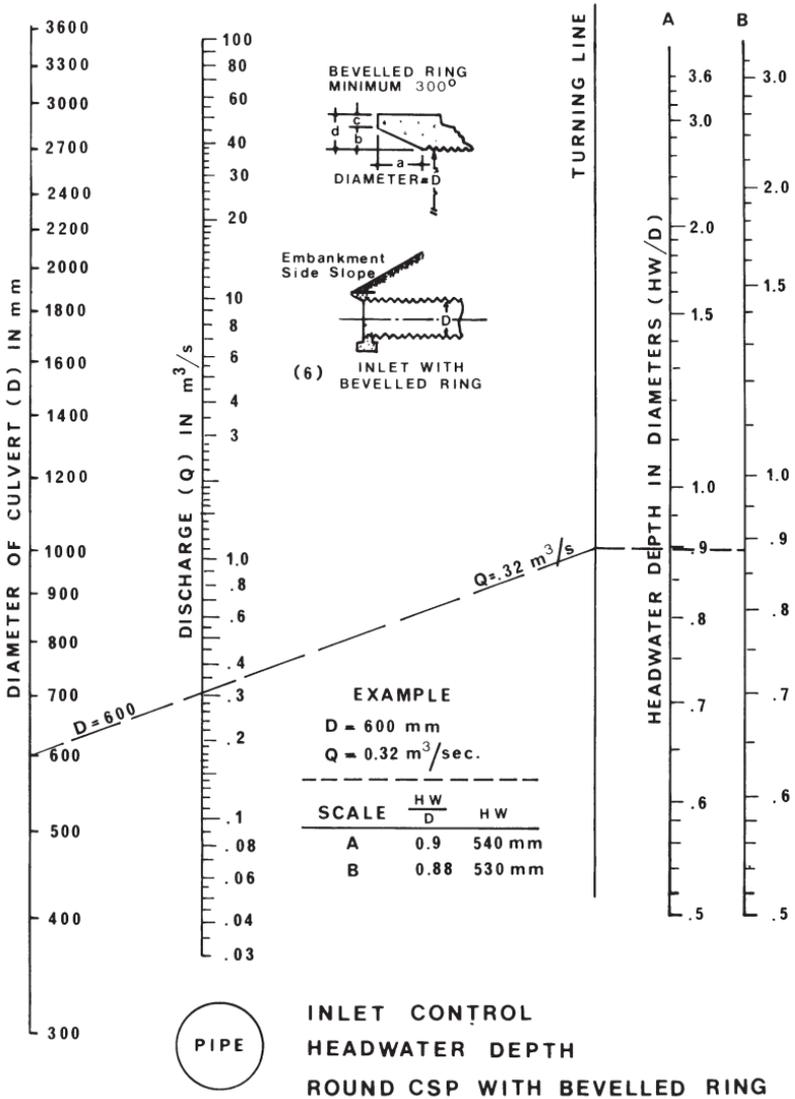
Most programs analyze the performance of a given culvert, although some are capable of design. Generally, the desired result of either type of program is to obtain a culvert design which satisfies hydrologic needs and site conditions by considering both inlet and outlet control. Results usually include the barrel size, inlet dimensions, headwater depth, outlet velocity, and other hydraulic data. Some programs are capable of analyzing side-tapered and slope-tapered inlets. The analysis or design of the barrel size can be for one barrel only or for multiple barrels.

Some programs may contain features such as backwater calculations, performance curves, hydrologic routines, and capabilities for routing based on upstream storage considerations.



**Figure 4.10** Headwater depth for round corrugated steel pipe and structural plate corrugated steel pipe under inlet control.

$\frac{b}{D}$	$\frac{a}{D}$	$\frac{c}{D}$	$\frac{d}{D}$	ENTRANCE TYPE
0.042	0.063	0.042	0.083	A
0.083	0.125	0.042	0.125	B



**Figure 4.11** Headwater depth for round corrugated steel pipe with bevelled ring under inlet control.

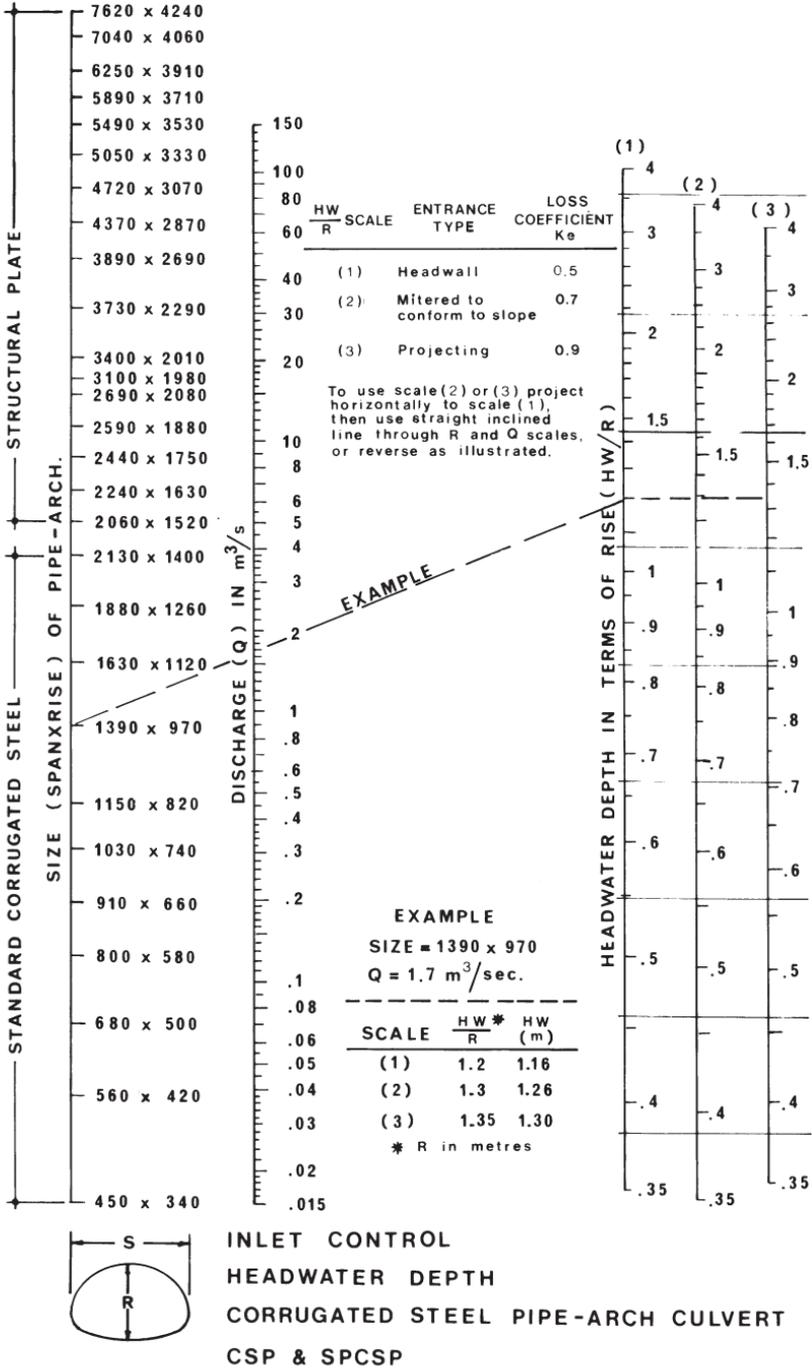
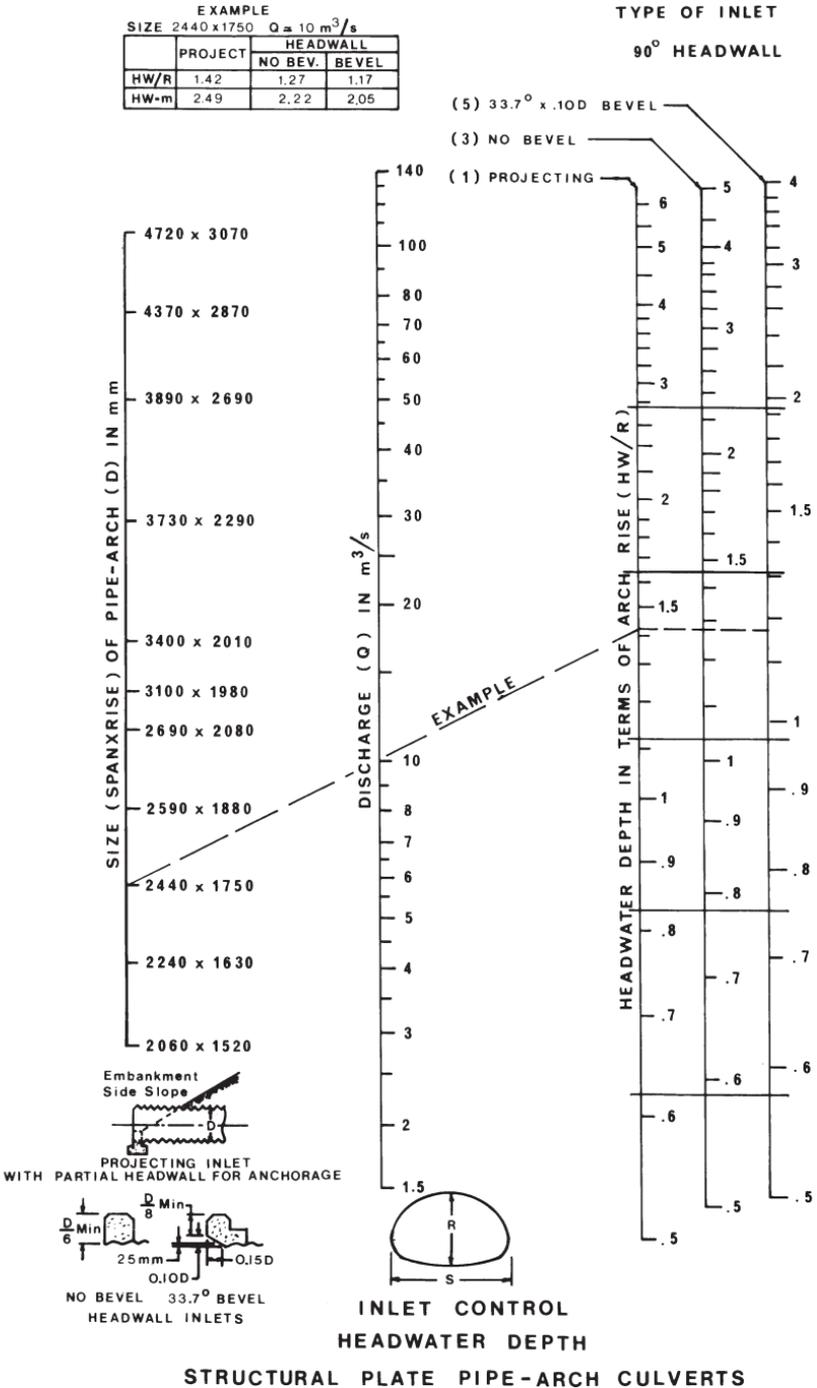
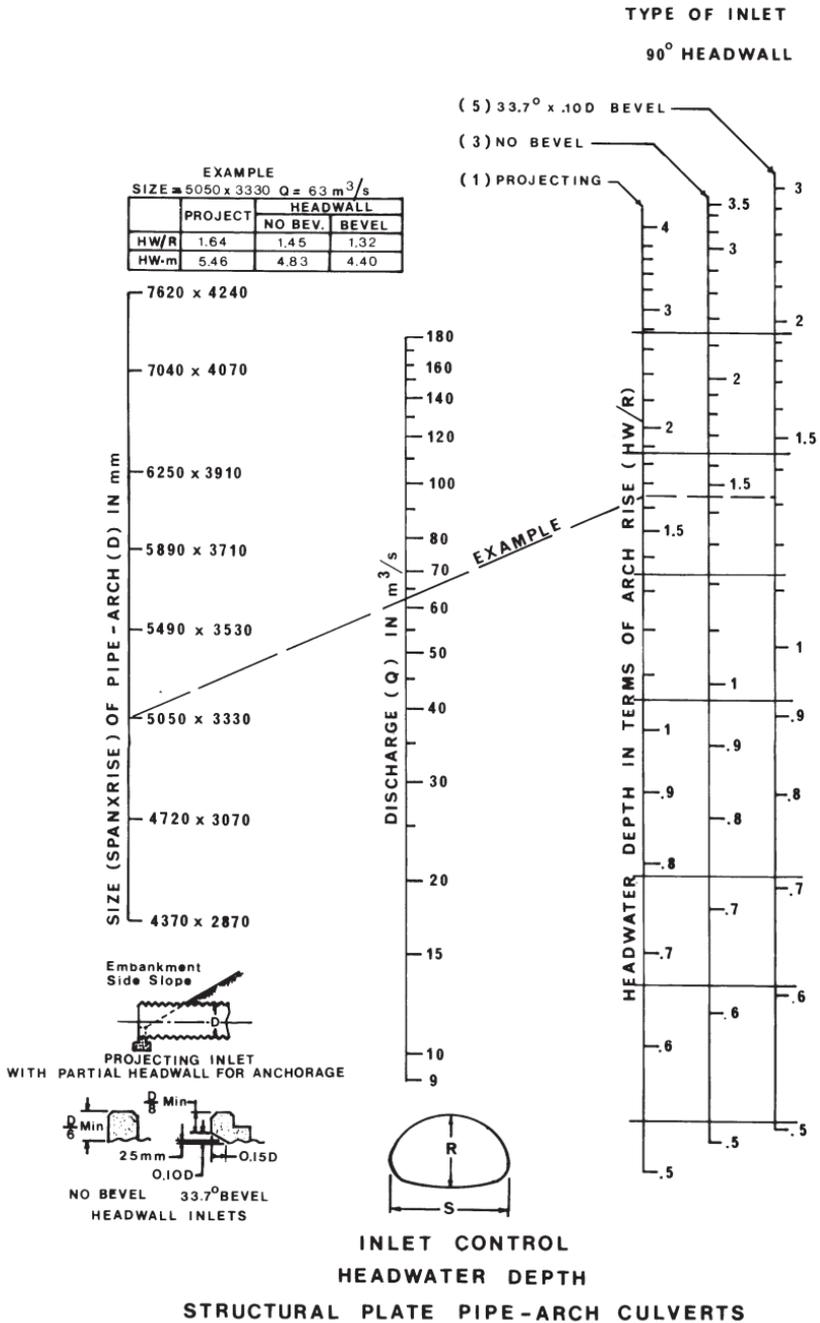


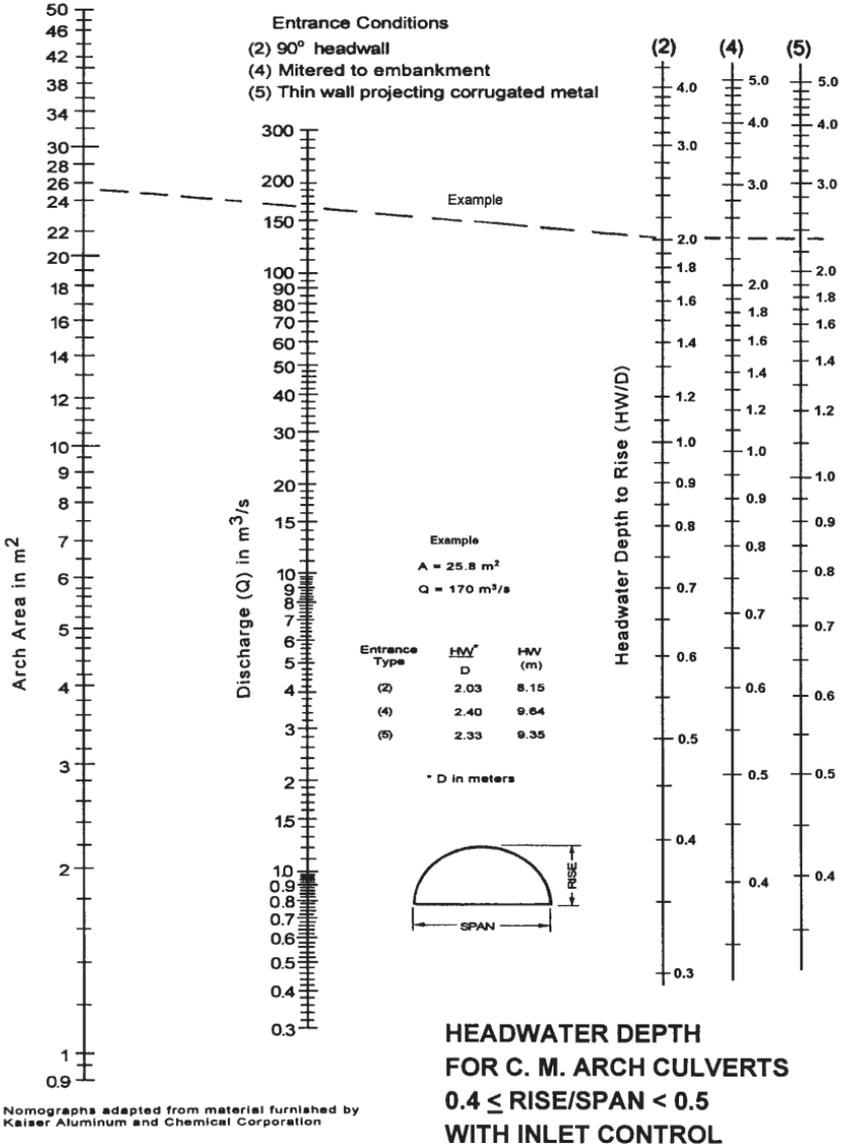
Figure 4.12 Headwater depth for corrugated steel and structural plate corrugated steel pipe-arch under inlet control.



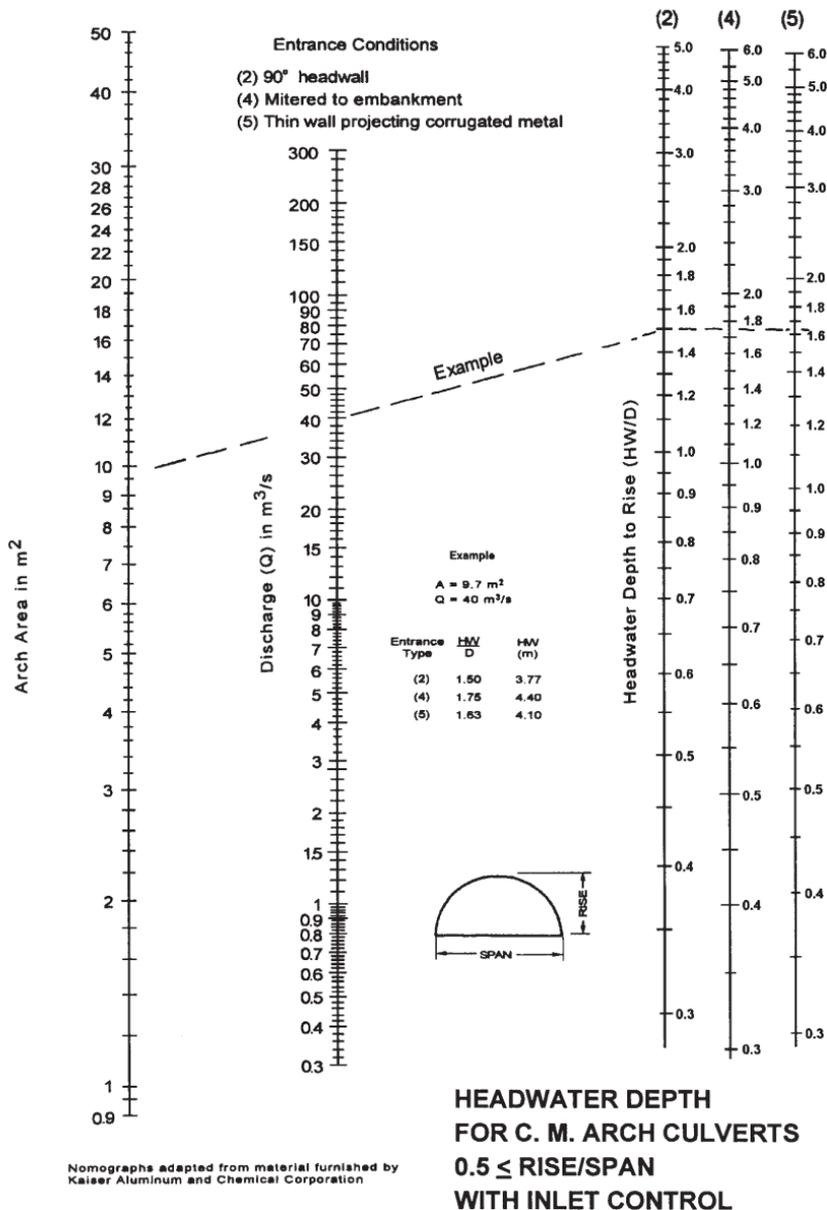
**Figure 4.13** Headwater depth for structural plate corrugated steel pipe-arch under inlet control (size range: up to 4720 mm x 3070 mm).



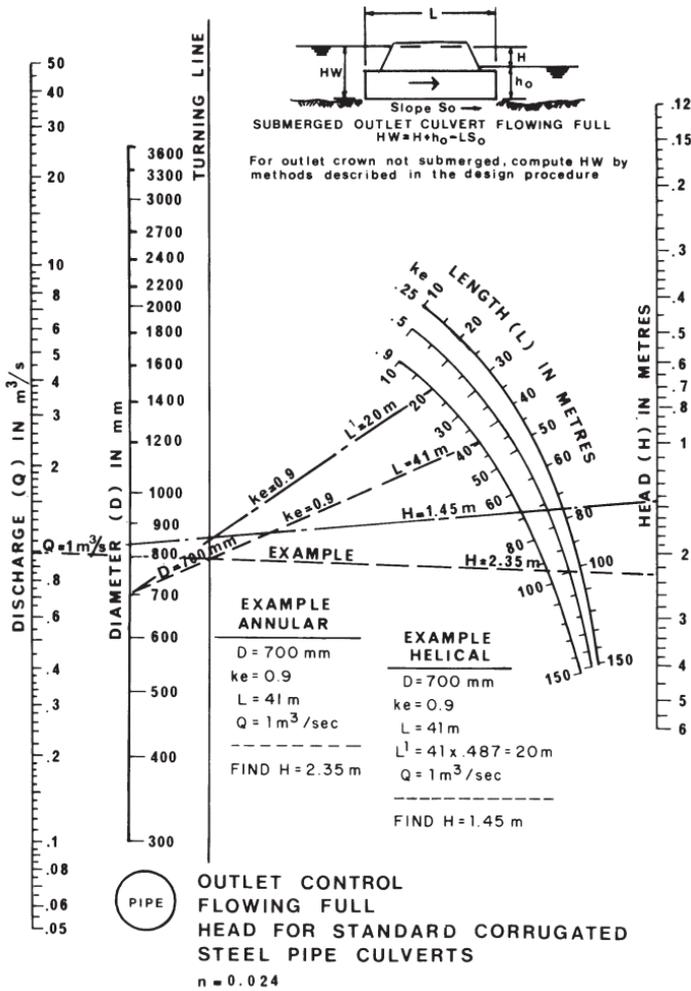
**Figure 4.14** Headwater depth for structural plate corrugated steel pipe-arch under inlet control (size range: 4370 mm x 2870 mm and over).



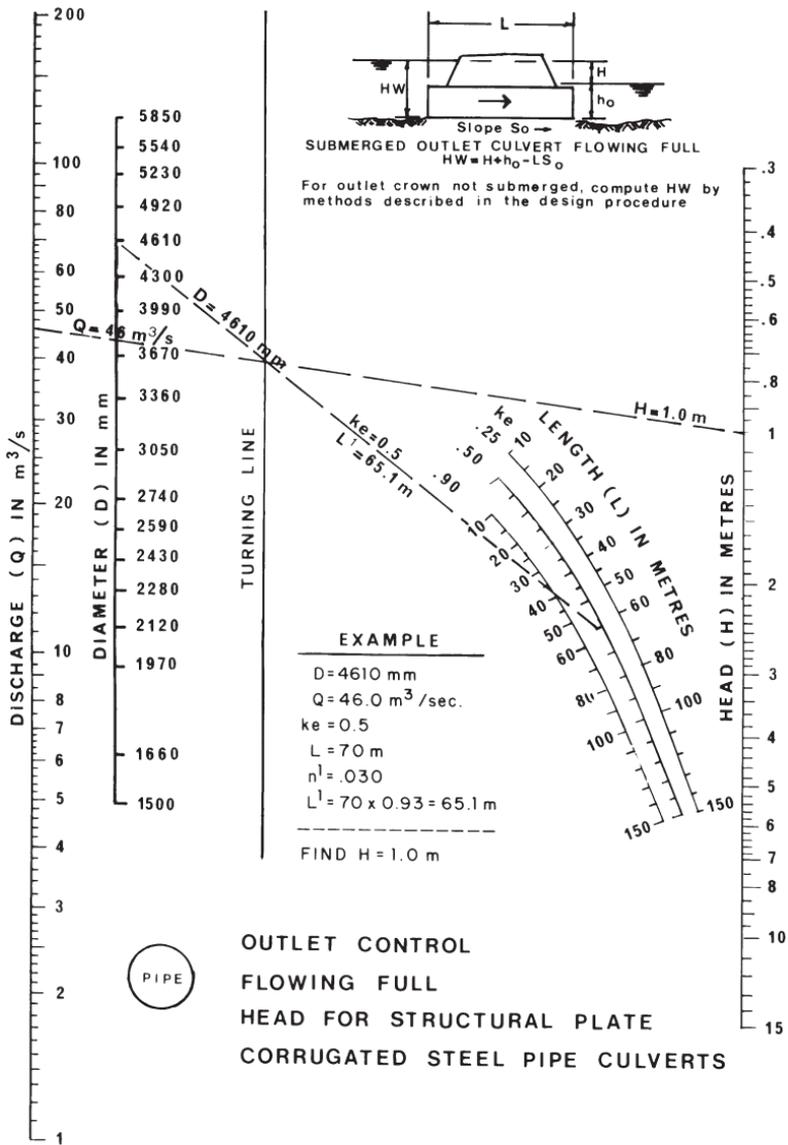
**Figure 4.15** Headwater depth for structural plate corrugated steel arch with  $0.4 \leq \text{rise}/\text{span} < 0.5$ , under inlet control.



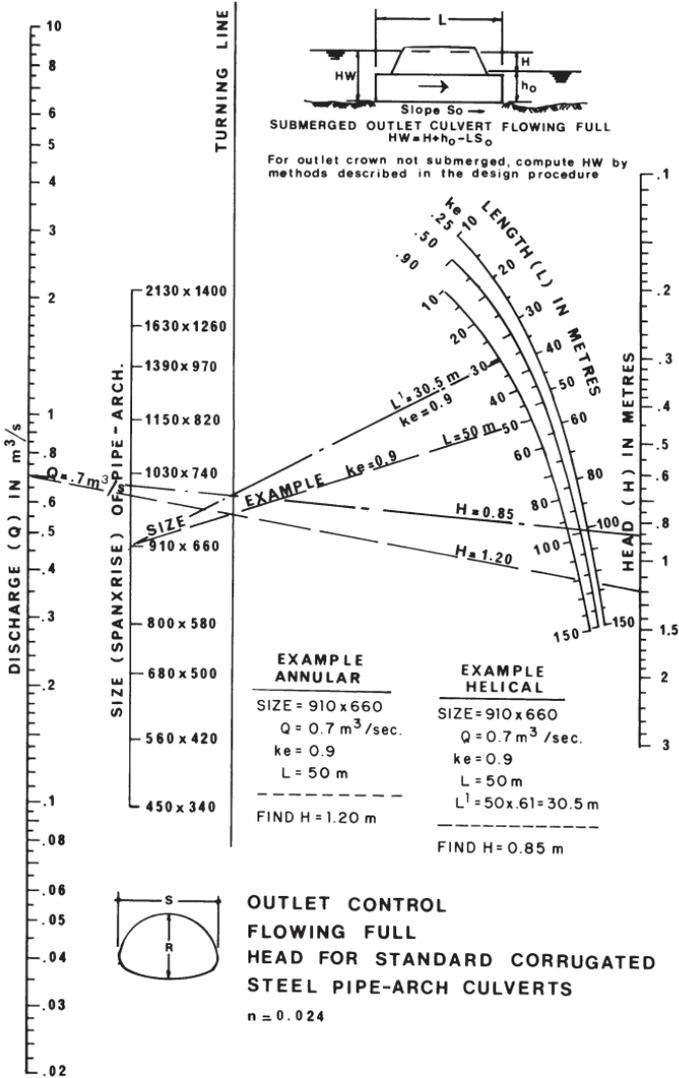
**Figure 4.16** Headwater depth for structural plate corrugated steel arch with  $0.5 \leq \text{rise}/\text{span}$ , under inlet control.



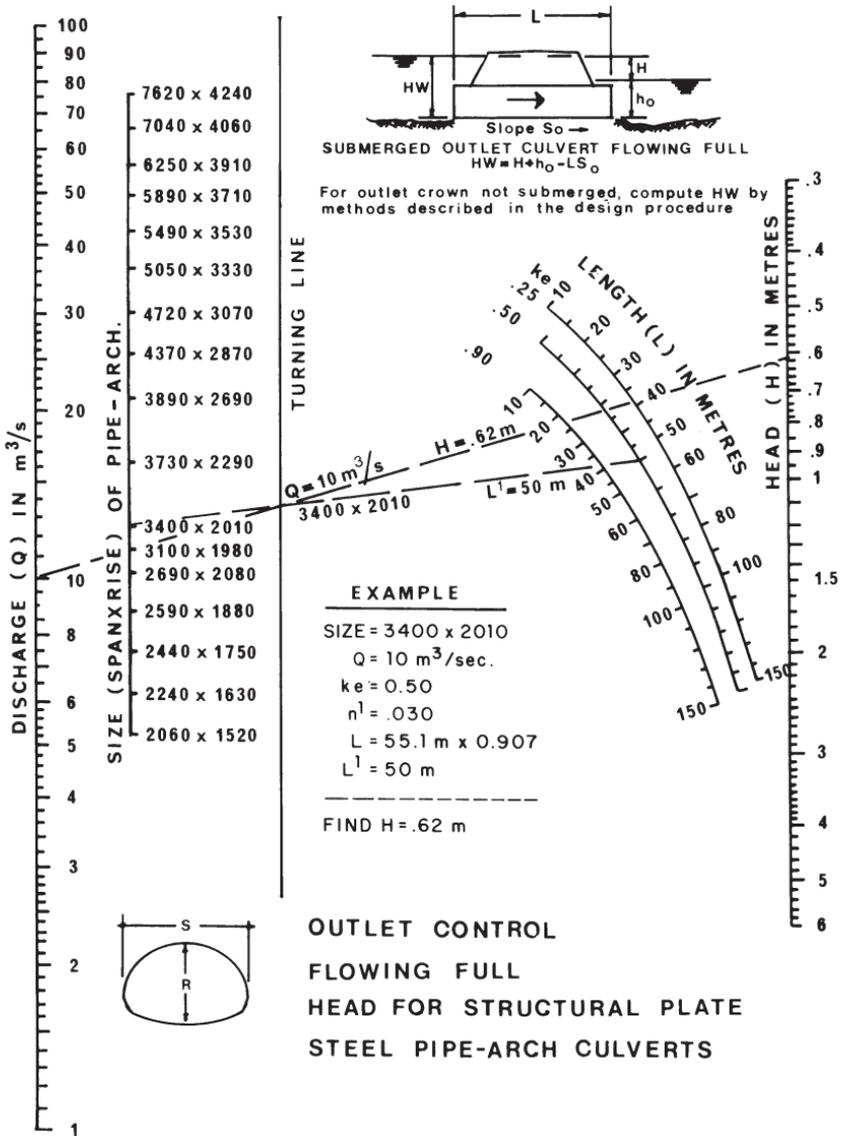
**Figure 4.17** Head for round corrugated steel pipe flowing full under outlet control.



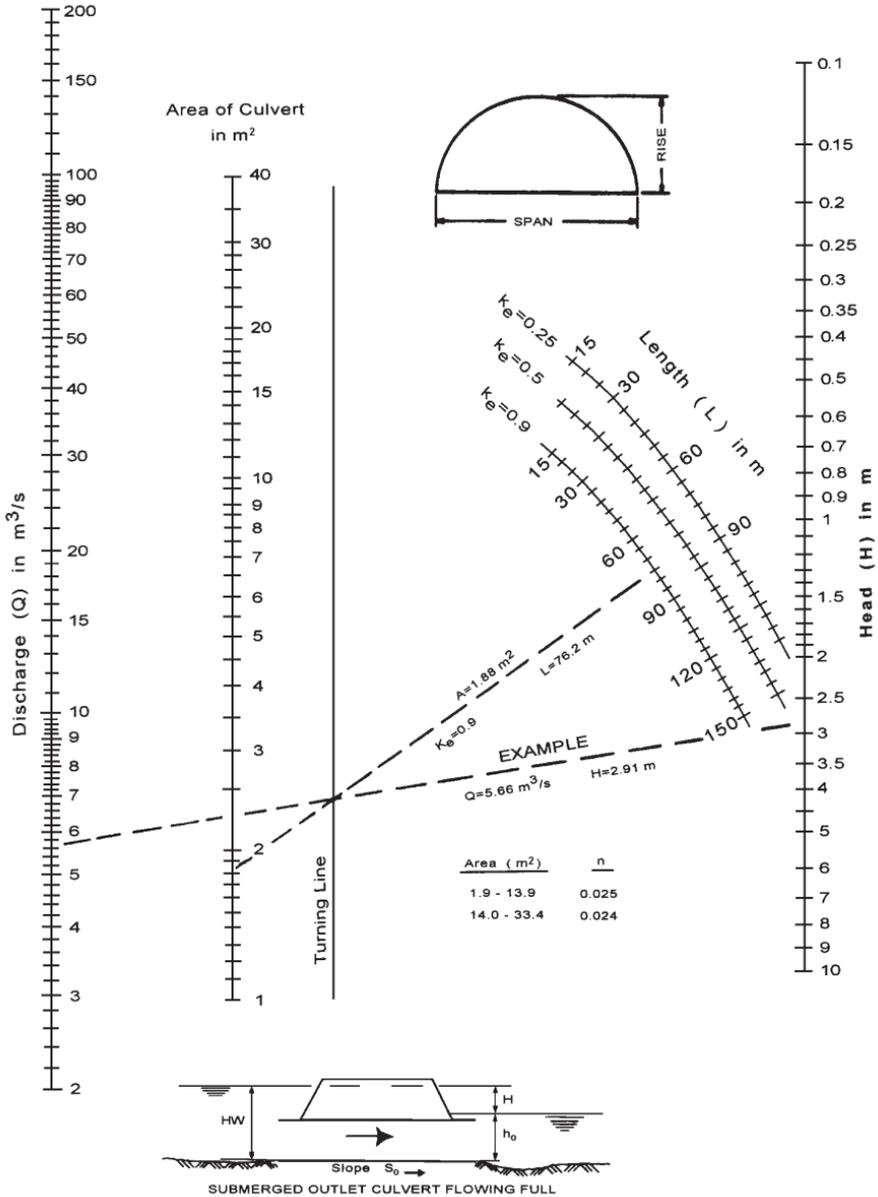
**Figure 4.18** Head for round structural plate corrugated steel pipe flowing full under outlet control.



**Figure 4.19** Head for corrugated steel pipe-arch flowing full under outlet control.



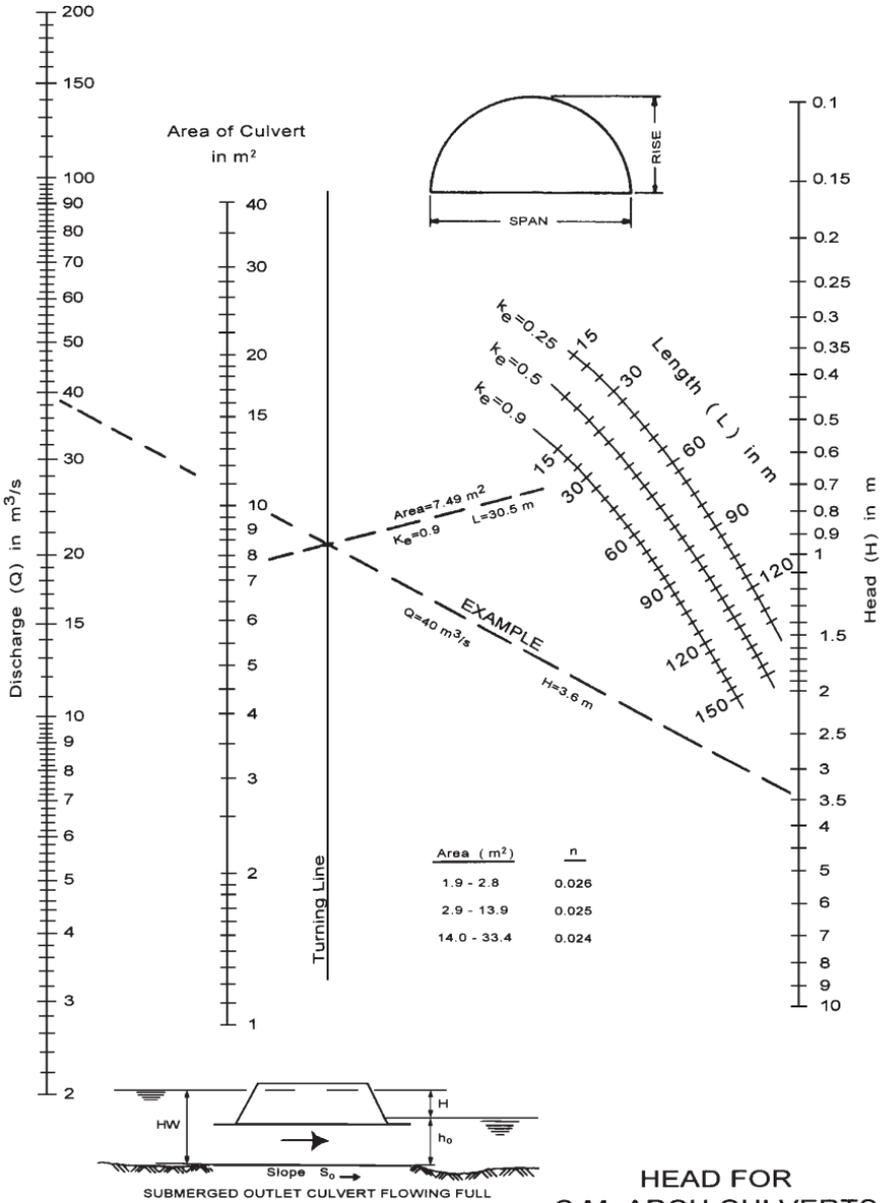
**Figure 4.20** Head for structural plate corrugated steel pipe-arch flowing full under outlet control.



**HEAD FOR  
C.M. ARCH CULVERTS  
FLOWING FULL  
CONCRETE BOTTOM  
0.4 ≤ RISE/SPAN < 0.5**

Adapted from material furnished by Kaiser Aluminum and Chemical Corporation

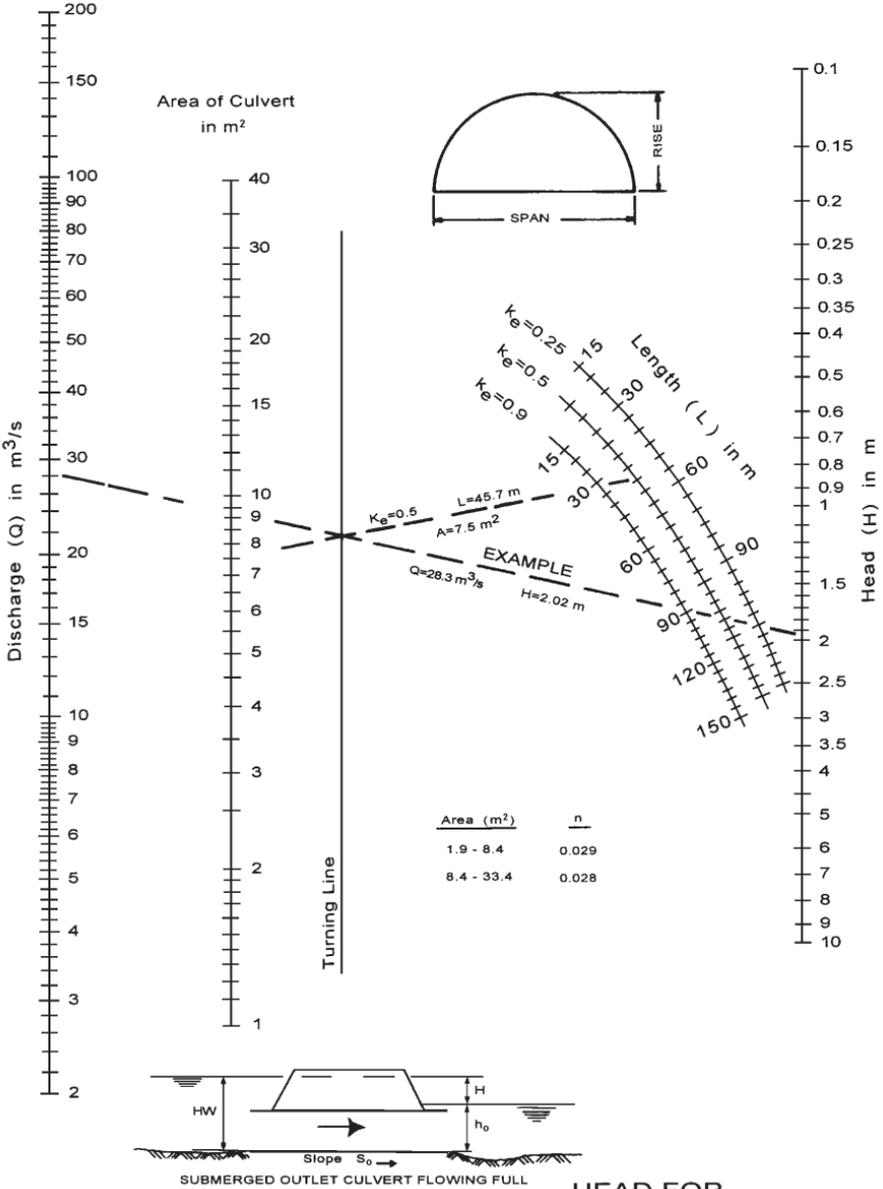
**Figure 4.21** Head for structural plate corrugated steel arch with concrete bottom and  $0.4 \leq \text{rise/span} < 0.5$ , flowing full under outlet control.



**HEAD FOR  
C.M. ARCH CULVERTS  
FLOWING FULL  
CONCRETE BOTTOM  
0.5 < RISE/SPAN**

Adapted from material furnished by  
Kaiser Aluminum and Chemical Corporation

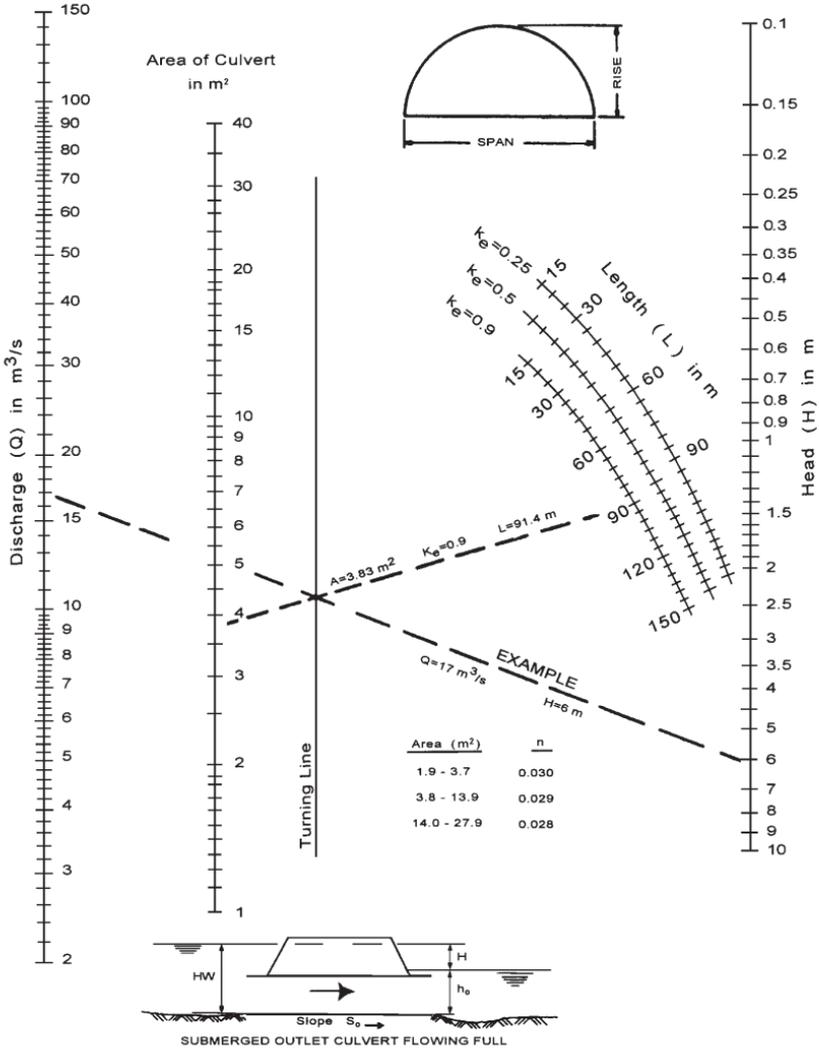
**Figure 4.22** Head for structural plate corrugated steel arch with concrete bottom and  $0.5 \leq \text{rise/span}$ , flowing full under outlet control.



**HEAD FOR  
C.M. ARCH CULVERTS  
FLOWING FULL  
EARTH BOTTOM ( $n_b = 0.022$ )  
 $0.4 \leq \text{RISE}/\text{SPAN} < 0.5$**

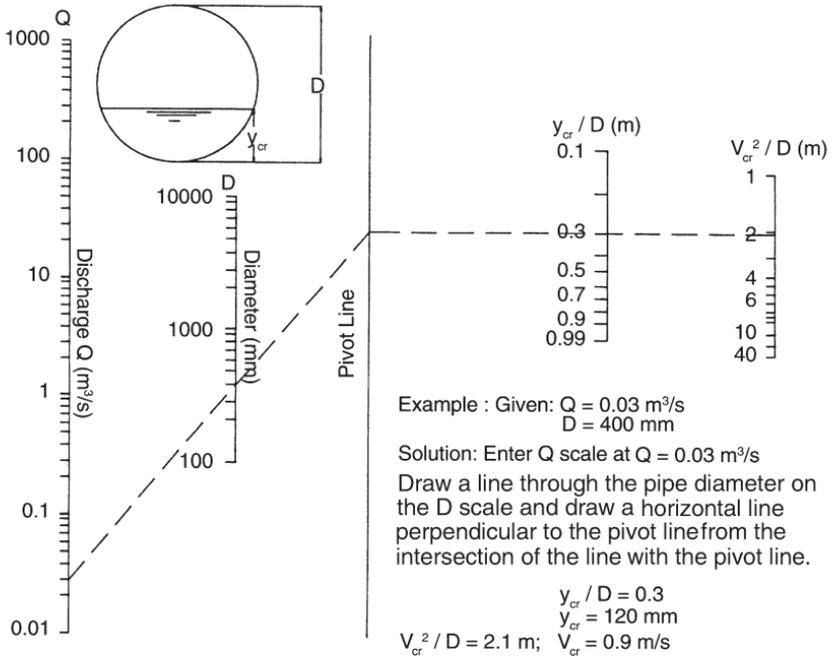
Adapted from material furnished by  
Kaiser Aluminum and Chemical Corporation

**Figure 4.23** Head for structural plate corrugated steel arch with earth bottom and  $0.4 \leq \text{Rise}/\text{Span} < 0.5$ , flowing full under outlet control.



Adapted from material furnished by  
 Kaiser Aluminum and Chemical Corporation

**Figure 4.24** Head for structural plate corrugated steel arch with earth bottom and  $0.5 \leq \text{Rise}/\text{Span}$ , flowing full under outlet control.



**Figure 4.25** Critical depth for round corrugated steel and structural plate corrugated steel pipe.

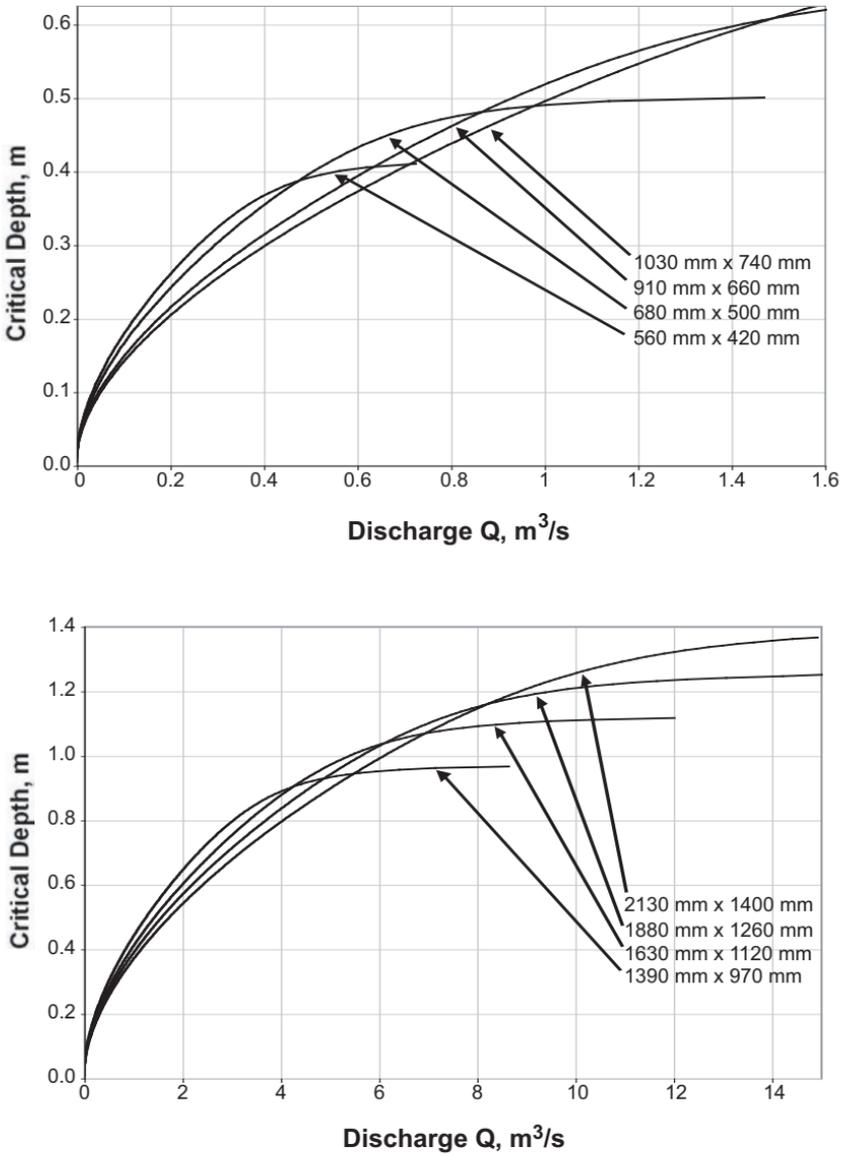
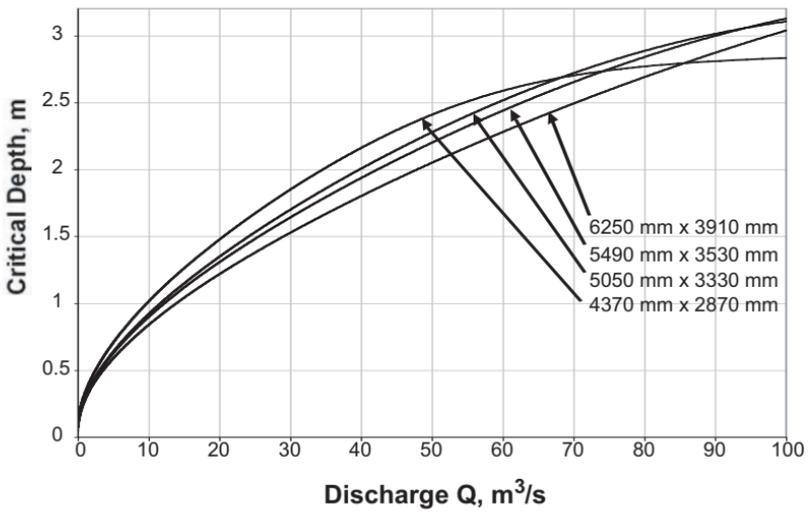
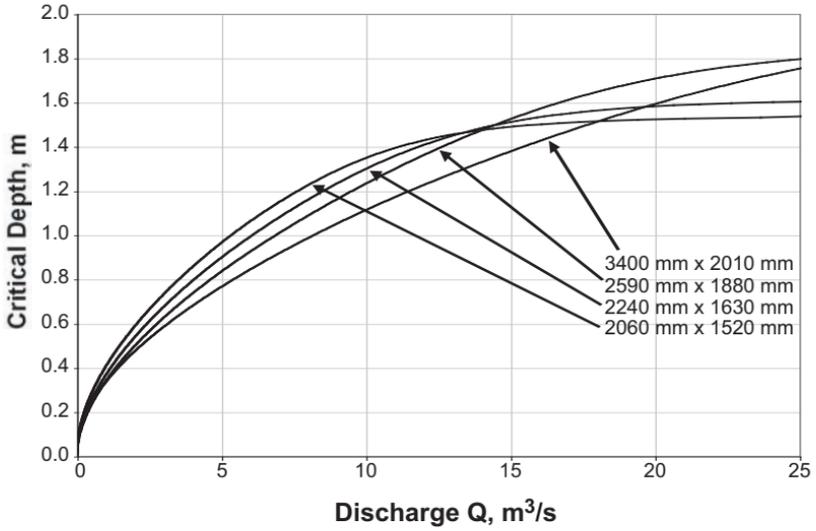
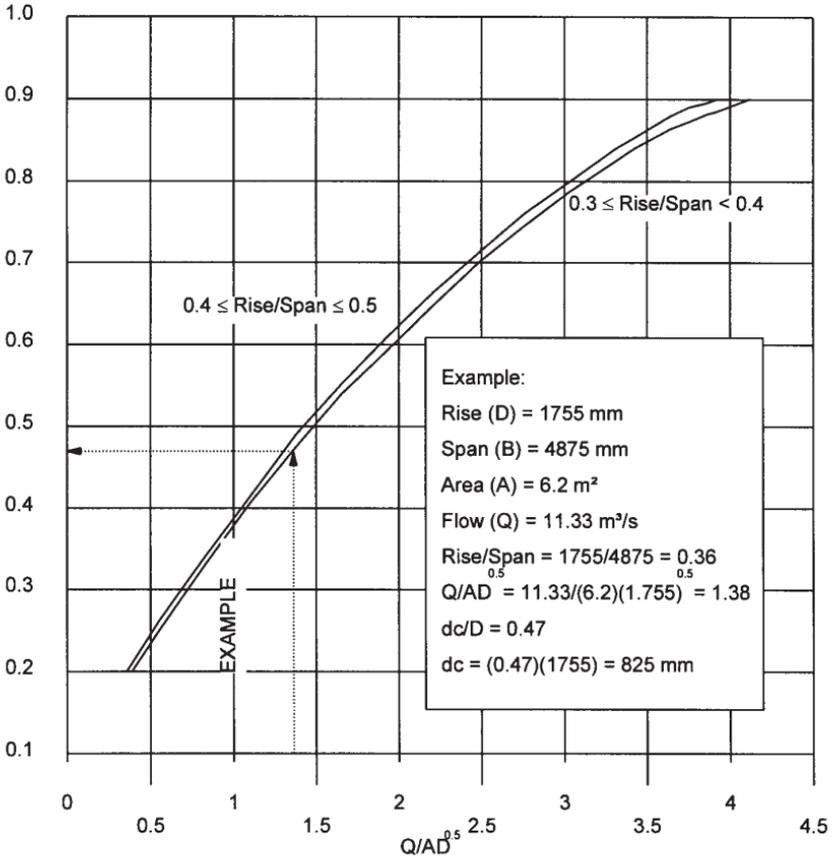


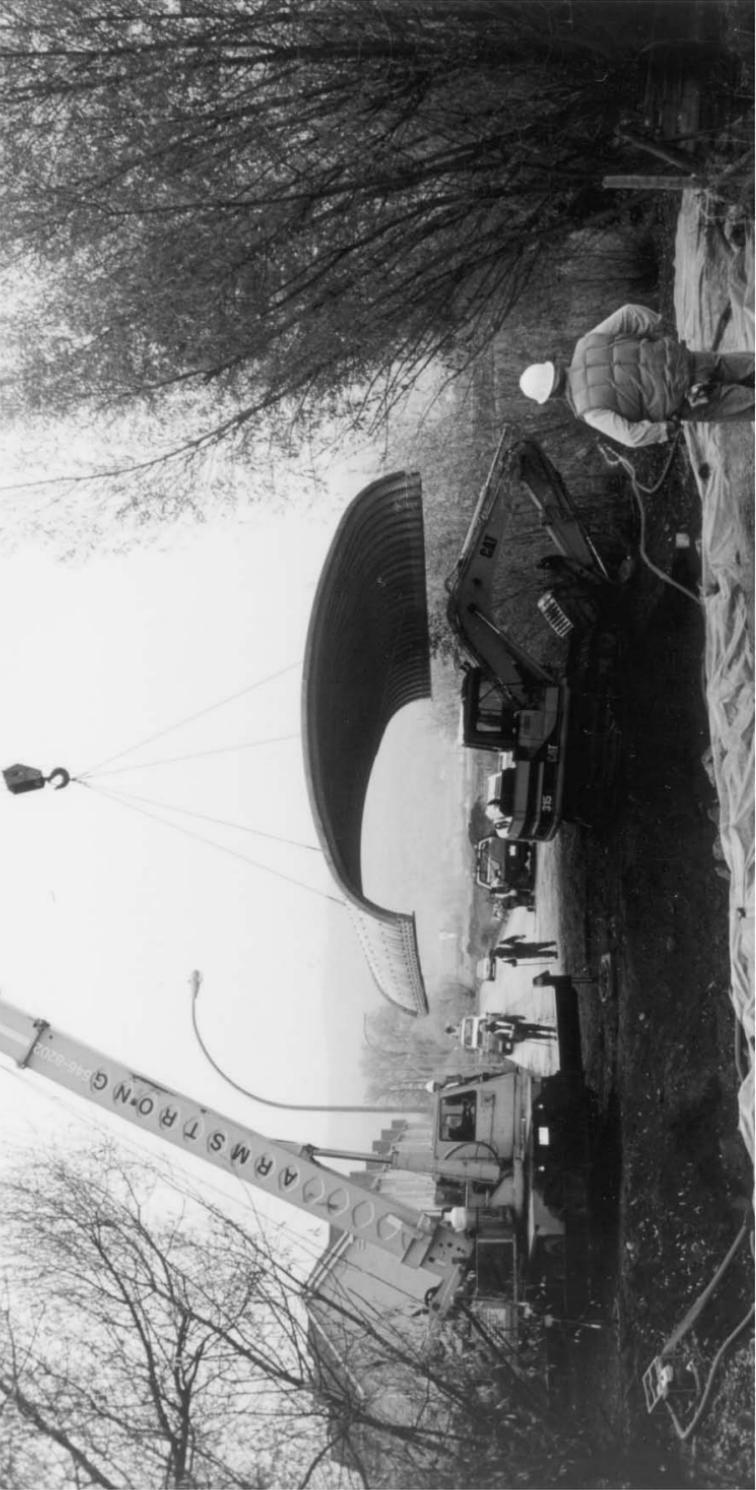
Figure 4.26 Critical depth for corrugated steel pipe-arch.



**Figure 4.27** Critical depth for structural plate corrugated steel pipe-arch.



**Figure 4.28** Critical depth for structural plate corrugated steel arch.



Long span structure under construction.

## HYDRAULICS OF LONG SPAN STRUCTURES

### Introduction

Standard procedures are presented here to determine the headwater depth resulting from a given flow through a long span structure under both inlet and outlet control conditions. The most common long span hydraulic shapes are the horizontal ellipse, the low profile arch, and the high profile arch. Useful hydraulic data pertaining to these shapes are presented in tabular and graphic form. Basic hydraulic formulas, flow conditions and definitions have been given previously. However, long span hydraulics include factors which are not considered in the earlier calculations.

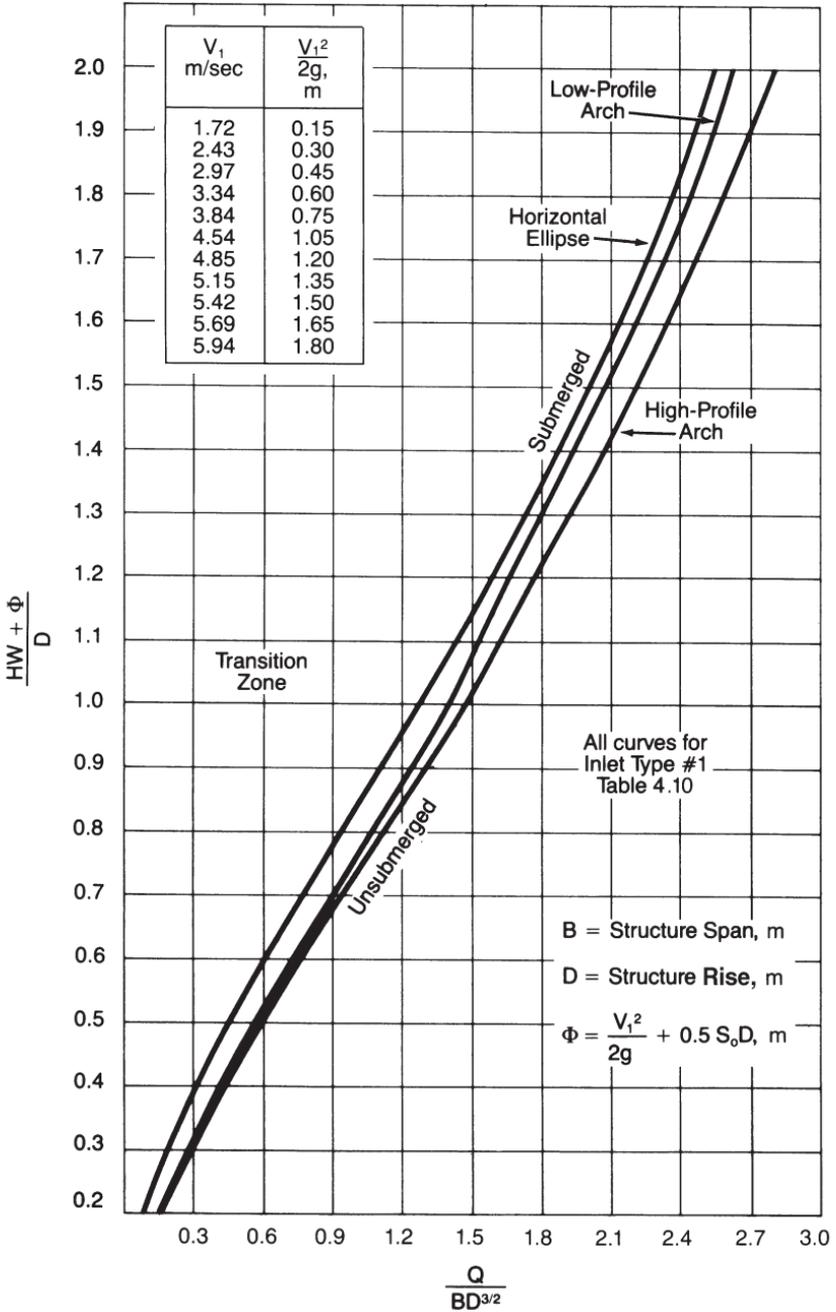
### Design

Long span structures are often small bridges which span the flood channel. This type of structure ordinarily permits little or no ponding at the inlet. Maximum headwater is usually below the top of the structure. In other words, there is usually some freeboard between the water surface and the top of the structure. This condition is quite different from the ordinary culvert which normally presents a small opening in an embankment crossing a larger flood channel.

The typical long span hydraulic conditions just described maintain effective approach velocity. The following long span hydraulic design procedure considers this approach velocity. The formulas and coefficients taken from the U.S. Federal Highway Administration (FHWA) methodology have been modified to include the approach velocity. In this discussion, headwater, HW, refers to the water surface and not to the energy grade line. This is different than the FHWA procedures, where HW refers to the energy grade line which corresponds to  $HW + \Phi$  in this discussion.

### Design Chart

Inlet control is expected to govern in most long spans. Figure 4.29 allows the designer to conveniently calculate the headwater depth for three standard shapes having the most typical inlet condition. This figure is a plot of the two design equations below (for unsubmerged and submerged inlets), and is based on an inlet that is either a square end with a headwall or a step-beveled end with a concrete collar (Type 1 in Table 4.10). The accuracy of the curves is within the degree to which the graph can be read. Using the design discharge and the structure span and rise, the curve for the structure desired gives the ratio of the headwater depth, approach velocity head and slope correction to the structure rise. The headwater depth is determined by subtracting the velocity head and slope correction from the product of the ratio and the structure rise. Figure 4.29 also includes a table of velocity heads for a variety of approach velocities.



**Figure 4.29** Headwater depth for long span corrugated steel structures under inlet control.

**Table 4.10** Inlet loss coefficients for long spans

Inlet Type	Enhanced Coefficients					$\left\{ \frac{Q}{AD^{1/2}} \right\}$	
	$k_d$	$k_p$	$k$	$j$	$k_e$	Unsubmerged Maximum	Submerged Minimum
1	0.1243	0.69	0.0272	2.0	0.5	1.82	2.10
2	0.0984	0.74	0.0079	2.5	0.2	1.82	2.32

- Notes: 1) Type 1 inlet is square end with headwall or step-beveled end with concrete collar.  
 2) Type 2 inlet is square or step-beveled end with mitered edge on headwall.  
 3) Special improved inlet or outlet configurations can reduce headwater depths.  
 4) Coefficients  $k$  and  $k_d$  are not dimensionless.

## Design Calculations

### Inlet Control

The equations for calculating headwater depth for long span structures under inlet control are:

For unsubmerged inlets:

$$HW = H_c + H_e - 0.5 S_o D - \frac{V_1^2}{2g}$$

For submerged inlets:

$$HW = k_d D \left\{ \frac{Q}{AD^{1/2}} \right\} + k_p D - 0.5 S_o D - \frac{V_1^2}{2g}$$

- where:  $HW$  = headwater depth from the invert to the water surface, m  
 $H_c$  = critical head, m  
 $H_e$  = increment of head above the critical head, m  
 $S_o$  = slope of the structure, m/m  
 $D$  = rise of the structure, m  
 $V_1$  = approach velocity, m/s  
 $g$  = gravitational constant, 9.806 m/s<sup>2</sup>  
 $k_d, k_p$  = coefficients based on inlet type (Table 4.10)  
 $Q$  = design discharge, m<sup>3</sup>/s  
 $A$  = full cross-sectional end area of the structure, m<sup>2</sup>

To determine if the flow condition is submerged or unsubmerged, the value of  $\left\{ \frac{Q}{AD^{1/2}} \right\}$  is calculated and reference is made to Table 4.10. If the flow is in the transition zone between unsubmerged and submerged, a reasonable approximation can be made by using both equations and interpolating based on where the value occurs relative to the limits in the table. When a performance curve is plotted, such as in Figure 4.29, the transition zone is filled in manually.

The critical head is equal to the critical depth in the structure at design flow plus the velocity head at that flow:

$$H_c = d_c + \frac{V_c^2}{2g}$$

- where:  $d_c$  = critical depth, m  
 $V_c$  = critical velocity, m/s

The critical depth can be interpolated from Tables 4.11 through 4.13. Using the design discharge, the critical depth, as a decimal fraction of the structure rise, is estimated by interpolating between known discharges for a number of set critical depth decimal fractions.

**Table 4.11** Hydraulic data for structural plate horizontal ellipse

Span, mm	Rise, mm	Full Flow Data				Discharge – Q (m <sup>3</sup> /s) When Critical Depth =					
		Area, m <sup>2</sup>	WP, m	R, m	AR <sup>2/3</sup> , m <sup>8/3</sup>	0.4D	0.5D	0.6D	0.7D	0.8D	0.9D
1630	1350	1.74	4.717	0.369	0.895	1.255	1.996	2.821	3.792	4.973	6.704
2130	1420	2.41	5.692	0.425	1.362	1.799	2.859	4.011	5.352	6.956	9.288
2900	1930	4.36	7.643	0.570	2.997	3.782	5.928	8.434	11.407	15.172	20.590
3200	2260	5.64	8.618	0.654	4.249	5.295	8.299	11.796	15.940	21.151	28.681
3680	2440	6.85	9.593	0.714	5.472	6.620	10.364	14.819	20.177	26.934	36.467
4420	2790	9.78	11.544	0.847	8.755	10.244	16.101	22.756	30.568	40.178	54.775
4953	3251	12.86	13.007	0.988	12.756	14.838	23.255	33.051	44.661	59.252	80.344
5156	3683	14.87	13.983	1.063	15.488	17.795	27.966	39.627	53.381	70.401	95.650
5715	3988	18.08	15.446	1.170	20.074	22.585	35.670	50.336	67.541	88.457	119.91
6230	3840	18.40	15.938	1.155	20.255	22.326	35.004	49.902	67.657	90.356	122.69
6680	3990	20.49	16.909	1.212	23.292	25.340	39.743	56.725	77.022	103.01	139.87
7010	4290	23.15	17.884	1.294	27.489	29.710	46.584	66.421	90.070	120.31	163.36
7470	4470	25.49	18.859	1.351	31.151	33.271	52.183	74.483	101.13	135.27	183.67
7950	5540	34.25	21.298	1.608	47.009	50.168	78.689	111.73	150.82	199.87	271.35
8280	5820	37.59	22.273	1.687	53.270	56.526	88.678	125.87	169.87	224.93	305.36
8970	6070	42.23	23.736	1.779	62.002	64.763	101.54	144.28	194.9	258.70	351.26
10110	6120	47.57	25.687	1.852	71.739	72.906	114.32	163.07	221.24	295.69	401.48
10640	6500	53.29	27.150	1.963	83.546	84.182	131.99	188.22	255.28	341.05	463.09
11250	7800	68.25	30.076	2.269	117.84	118.66	186.12	264.29	356.79	472.92	642.06
11790	8510	78.31	32.027	2.445	142.12	142.45	223.60	317.20	427.82	565.47	767.49

**Table 4.12** Hydraulic data for structural plate low profile arch

Span, mm	Rise, mm	Full Flow Data				Discharge – Q (m <sup>3</sup> /s) When Critical Depth =					
		Area, m <sup>2</sup>	WP, m	R, m	AR <sup>2/3</sup> , m <sup>8/3</sup>	0.4D	0.5D	0.6D	0.7D	0.8D	0.9D
6120	2290	11.18	14.608	0.765	9.351	12.490	18.699	31.558	40.739	51.808	67.973
5920	2080	9.75	13.901	0.701	7.693	9.964	15.085	26.384	34.024	43.237	56.703
6550	2360	12.39	15.544	0.797	10.650	14.606	21.714	35.771	46.139	58.642	76.911
6780	2410	13.01	16.013	0.812	11.323	15.722	23.305	37.985	48.974	62.228	81.600
7010	2440	13.64	16.481	0.827	12.017	16.878	24.953	40.270	51.901	65.928	86.438
7240	2490	14.29	16.949	0.843	12.752	18.074	26.659	42.628	54.919	69.743	91.426
7470	2540	14.94	17.418	0.858	13.489	19.311	28.422	45.060	58.03	73.675	96.565
7670	2570	15.62	17.886	0.873	14.267	20.588	30.245	47.565	61.234	77.724	101.85
7900	2620	16.30	18.354	0.888	15.058	21.906	32.127	50.144	64.532	81.891	107.30
8310	3280	22.04	20.130	1.094	23.400	33.827	49.090	66.715	95.236	121.28	159.27
8760	3350	23.74	21.067	1.127	25.709	37.437	54.205	80.649	104.26	132.72	174.24
9420	3480	26.39	22.472	1.174	29.368	43.197	62.369	91.840	118.61	150.89	198.01
9630	3680	28.69	23.179	1.237	33.060	48.759	70.211	94.988	132.35	168.42	221.06
9860	3730	29.64	23.647	1.253	34.449	50.928	73.281	106.62	137.73	175.24	229.99
10080	3780	30.61	24.116	1.269	35.878	53.145	76.420	110.91	143.23	182.21	239.10
10110	3610	29.15	23.874	1.221	33.300	49.377	71.137	103.81	133.95	170.3	223.39
10490	4040	34.09	25.288	1.348	41.599	61.795	88.615	119.53	164.48	209.26	274.62
10540	3680	31.06	24.814	1.251	36.061	53.736	77.326	112.23	144.74	183.93	241.22
11560	4780	44.30	28.325	1.564	59.688	89.104	127.07	170.48	230.88	293.77	385.53
10770	3730	32.03	25.282	1.266	37.484	55.988	80.526	116.58	150.30	190.96	250.41
11790	4800	45.51	28.793	1.580	61.737	92.194	131.43	176.31	238.49	303.43	398.19

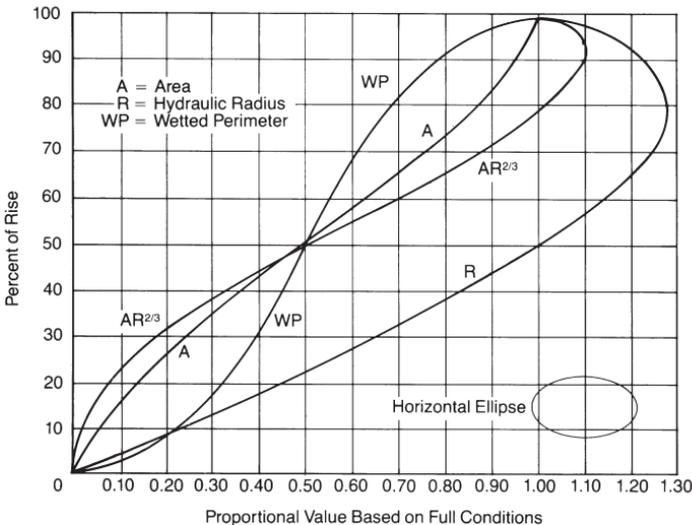
**Table 4.13** Hydraulic data for structural plate high profile arch

Span, mm	Rise, mm	Full Flow Data				Discharge - Q (m <sup>3</sup> /s) When Critical Depth =					
		Area, m <sup>2</sup>	WP, m	R, m	AR <sup>2/3</sup> , m <sup>8/3</sup>	0.4D	0.5D	0.6D	0.7D	0.8D	0.9D
6300	3680	20.34	17.558	1.158	22.429	26.746	37.644	44.668	56.453	69.591	84.429
6550	3560	20.46	17.878	1.144	22.379	24.665	34.666	40.150	50.502	61.991	74.810
6780	3610	21.36	18.359	1.163	23.622	26.004	36.533	42.028	52.888	64.935	78.382
7010	3660	22.28	18.839	1.182	24.907	27.389	38.465	43.973	55.360	67.99	82.093
7240	3680	23.21	19.318	1.201	26.224	28.821	40.463	45.987	57.920	71.155	85.945
7670	3740	25.09	20.273	1.238	28.927	31.828	44.661	50.224	63.311	77.829	94.086
7870	4655	32.98	22.230	1.484	42.908	41.100	57.882	76.634	86.947	105.88	126.90
8100	4650	34.17	22.718	1.504	44.854	42.896	60.383	79.911	90.075	109.73	131.54
8560	5020	38.74	24.118	1.606	53.128	47.769	67.26	89.035	112.93	122.79	146.64
8590	4630	35.51	23.524	1.509	46.717	41.377	58.155	76.855	97.333	103.06	123.00
9220	4920	40.28	25.135	1.602	55.148	52.619	73.941	97.696	107.09	130.73	156.93
9450	4970	41.53	25.615	1.621	57.308	54.719	76.871	101.54	110.78	135.29	162.46
9680	5260	45.25	26.537	1.705	64.580	58.103	81.670	107.93	136.70	144.89	173.35
9910	5280	46.58	27.017	1.724	66.971	60.327	84.774	112.01	141.83	149.68	179.15
10360	5380	49.28	27.976	1.761	71.864	64.938	91.210	120.46	131.08	159.63	191.21
10360	5830	54.58	28.864	1.891	83.463	67.355	94.744	125.30	158.81	195.08	203.30
11350	6910	69.09	32.061	2.155	115.26	121.36	171.01	205.53	259.30	319.27	386.82
10570	5440	50.65	28.454	1.780	74.392	67.326	94.545	124.84	135.25	164.80	197.49
10590	5870	56.07	29.346	1.910	86.315	69.778	98.125	129.74	164.39	176.33	209.47
11580	6930	70.85	32.554	2.176	118.97	124.78	175.78	210.35	265.38	326.70	395.74

The critical velocity is calculated by dividing the design discharge by the partial flow area corresponding to the critical depth. The partial flow area can be determined from Figures 4.30 through 4.32 using the critical depth as a percentage of the structure rise. The partial flow area is the product of the proportional value from the figure and the full cross sectional area of the structure. The critical velocity is then:

$$V_c = \frac{Q}{A_c}$$

where:  $A_c$  = partial flow area based on the critical depth, m<sup>2</sup>



**Figure 4.30** Hydraulic properties of long span horizontal ellipse.

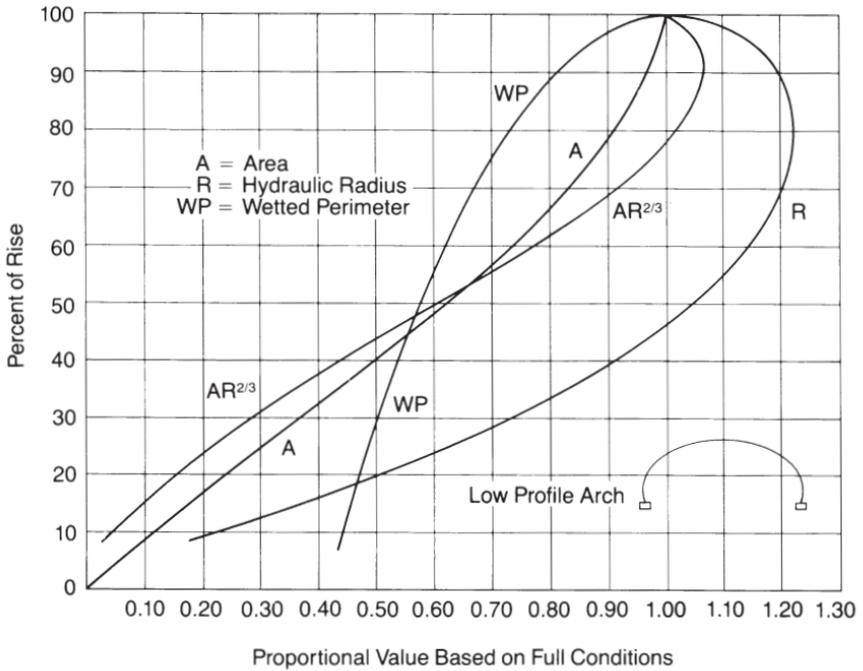


Figure 4.31 Hydraulic properties of long span low profile arch.

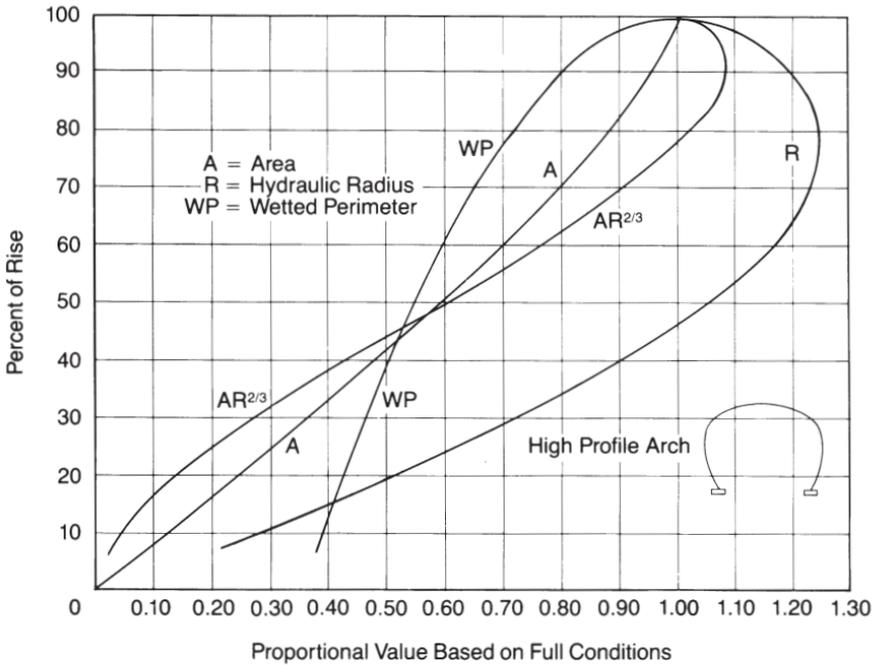


Figure 4.32 Hydraulic properties of long span high profile arch.

The accuracy of the critical depth may be checked using the basic formula for critical flow:

$$Q_c = \sqrt{\frac{gA_c^3}{T_c}}$$

where:  $T_c$  = width of the water surface for the critical depth case, m

For this calculation, detailed structure cross section geometry is required in order to calculate the water surface width when the water depth is the critical depth.

The increment of head above the critical head is:

$$H_c = k D \left\{ \frac{Q}{AD^{1/2}} \right\}^j$$

where:  $k, j$  = coefficients based on inlet type (Table 4.10)

### Outlet Control

#### *Free Water Surface*

The situation where a long span has a free water surface extending through its full or nearly full length, as shown in Figure 4.5 D (possibly the most common flow condition), exists when the headwater depth is less than:

$$D + (1 + k_e) \frac{V_c^2}{2g}$$

where:  $k_e$  = entrance loss coefficient based on inlet type (Table 4.10)

Under this condition, the headwater depth must be determined by a backwater analysis if accurate results are required. Datum points  $d_1$  and  $d_2$  are established upstream and downstream from the structure, beyond the influence of the entrance and outlet. The backwater analysis determines the water surface profile by starting at the downstream point and moving to the upstream point. The backwater analysis must consider channel geometry between the downstream point and the outlet end of the structure, outlet loss, changing geometry of flow within the structure, inlet loss, and conditions between the inlet end of the structure and the upstream point.

Long span hydraulic properties are provided in Tables 4.11 through 4.13 and Figures 4.30 through 4.32. Entrance loss coefficients are in Table 4.10. The exit loss for these types of structures is typically very small and is often assumed to be zero.

Backwater analyses are considered outside the scope of this handbook. There are references that provide guidance for this procedure. In particular, the FHWA's "Hydraulic Design of Highway Culverts" CDROM contains a discussion and example of the backwater analysis procedure.

#### *Full Flow*

When full flow or nearly full flow exists, the headwater depth is determined by the formula:

$$HW = (k_e + \frac{2gn^2L}{R^{4/3}} + 1) \frac{V^2}{2g} + h_o - L S_o - \frac{V_1^2}{2g}$$

- where: HW = headwater depth, m  
 $k_e$  = entrance loss coefficient (Table 4.10)  
 $g$  = gravitational constant = 9.806 m/s<sup>2</sup>  
 $n$  = Manning's friction factor (Table 4.7)  
 $L$  = length of long span, m  
 $R$  = hydraulic radius,  $m = \frac{A}{WP}$   
 $A$  = full cross sectional area of the long span, m<sup>2</sup>  
 $WP$  = perimeter of the long span, m  
 $V$  = velocity, m/s  
 $h_o$  = outlet datum, m  
 $S_o$  = slope of structure, m/m  
 $V_1$  = approach velocity, m/s

These conditions are as shown in Figure 4.5 A through C. They occur when the headwater depth is greater than:

$$D + (1 + k_e) \frac{V_c^2}{2g}$$

For arches or lined structures, a composite Manning's  $n$  value must be developed. A method described in an FHWA document is based on the assumption that the conveyance section can be broken down into a number of parts with associated wetted perimeters and Manning's  $n$  values. Each part of the conveyance section is then assumed to have a mean velocity equal to the mean velocity of the entire flow section. These assumptions lead to:

$$n = \left[ \frac{\sum_{i=1}^G (p_i n_i^{1.5})}{p} \right]^{0.67}$$

- where:  $n$  = weighted Manning's  $n$  value  
 $G$  = number of different roughnesses in the perimeter  
 $p_i$  = wetted perimeter influenced by material  $i$ , m  
 $n_i$  = Manning's  $n$  value for material  $i$   
 $p$  = total wetted perimeter, m

In the case of arches, the wetted perimeter used in hydraulic radius calculations includes that portion of the structure above the natural channel and the natural channel itself.

For flow conditions as shown in Figure 4.5 A and B, when the tailwater depth is equal to or greater than the structure rise:

$$h_o = TW$$

For flow conditions as shown in Figure 4.5 C, when the tailwater depth is less than the structure rise:

$$h_o = \frac{d_c + D}{2} \text{ or } TW \text{ (whichever is greater)}$$

The velocity,  $V$ , is determined by dividing the design discharge by the area, where the area is the full cross sectional area of the long span structure.

The remaining terms in the equation can be determined as previously discussed.

### Summary of Procedure

- Step 1. Collect all available information for the design. This includes the required design discharge, the structure length and slope, an allowable headwater elevation or depth, the average and maximum flood velocities in the channel, the proposed entrance type, and a desired structure shape.
- Step 2. Select an initial structure size. This may be an arbitrary choice, or estimated using a maximum allowable velocity. To estimate a structure size, the minimum structure end area is determined by dividing the design discharge by the maximum allowable velocity. Geometric constraints may also influence the choice of an initial structure size. An example of this is where a minimum structure span is required to bridge a channel.
- Step 3. Use Figure 4.29 and the design parameters to obtain a value for  $HW + \Phi$  and then the headwater depth,  $HW$ . When required, more accurate results can be achieved by using the inlet control formulas to calculate the headwater depth.
- Step 4. Check the calculated headwater depth against the allowable headwater depth. If the calculated headwater depth is greater than the allowable, select a larger structure and repeat Step 3. If the calculated headwater depth is less than the allowable, this is the resulting headwater depth for the structure selected under inlet control.
- Step 5. Calculate  $D + (1 + k_e) \frac{V_c^2}{2g}$
- If this value is greater than the allowable headwater depth, use the backwater curve method to determine the water surface profile through the structure and the headwater depth. If this value is equal to or less than the allowable headwater depth, the full flow formula should be used to determine the headwater depth. The resulting headwater depth is for the structure selected under outlet control.
- Step 6. Compare the inlet and outlet control headwater depths and use the larger. If the resulting headwater depth is greater than the allowable, a larger size or different shape structure should be chosen and the procedure repeated. If the headwater depth is significantly less than the allowable, a smaller size can be chosen and the procedure repeated in order to economize on the structure size.

## SPECIAL HYDRAULIC CONSIDERATIONS

In addition to flow hydraulics, the drainage designer must consider hydraulic forces and other hydraulic phenomena that may be factors in assuring the integrity of the culvert and embankment.

### Uplifting Forces

Uplifting forces on the inlet end of a culvert result from a variety of hydraulic factors that may act on the inlet during high flows. These may include; vortexes and eddy currents that cause scour, which in turn undermine the inlet and erode the culvert supporting embankment slope; debris blockage that accentuates the normal flow constriction, creating a larger trapped air space just inside the inlet, resulting in a significant buoyancy force that may lift the inlet; and sub-atmospheric pressures on the inside of the inlet, combined with flow forces or hydraulic pressures on the outside, that may cause the inward deflection of a skewed or beveled inlet, blocking flow and creating the potential for hydraulic uplift.

Buoyancy type failures can be prevented by structural anchorage of the culvert entrance. This anchorage should be extended into the embankment both below and to the sides of the pipe. Cut-end treatment of the culvert barrel in bevels or skews should have hook bolts embedded in some form of slope protection to protect against bending.

### Piping

Piping is a hydraulic phenomena resulting from the submersion of the inlet end of a culvert and high pore pressure in the embankment. Hydrostatic pressure at the inlet will cause the water to seek seepage paths along the outside of the culvert barrel or through the embankment. Piping is the term used to describe the carrying of fill material, usually fines, caused by seepage along the barrel wall. The movement of soil particles through the fill will usually result in voids in the fill. This process has the potential to cause failure of the culvert and/or the embankment. Culvert ends should be sealed where the backfill and embankment material is prone to piping.

### Weep Holes

These are perforations in the culvert barrel which are used to relieve pore pressure in the embankment. Generally, weep holes are not required in culvert design. For an installation involving prolonged ponding, there may be merit in considering a separate sub-drainage system to relieve pore pressure and control seepage in the embankment.

### Anti-Seepage Collars

Vertical cutoff walls may be installed around the culvert barrel at regular intervals to intercept and prevent seepage along the outer wall of the culvert. These may also be referred to as diaphragms. They are most often used in small earth fill dams or levees and are recommended when ponding is expected for an extended time. An example of this is when the highway fill is to be used as a detention dam or temporary reservoir. In such cases, earth fill dam design and construction practices should be considered.

### Single vs. Multiple Openings

A single culvert opening is, in general, the most satisfactory because of its greater ability to pass floating debris and driftwood. However, in many cases, the design

requires that the waterway be wide in order to get the water through quickly without ponding and flooding of the land upstream. In such cases, the solution may consist of using either an arch, a pipe-arch, or a battery of two or more openings. See Figure 4.33.

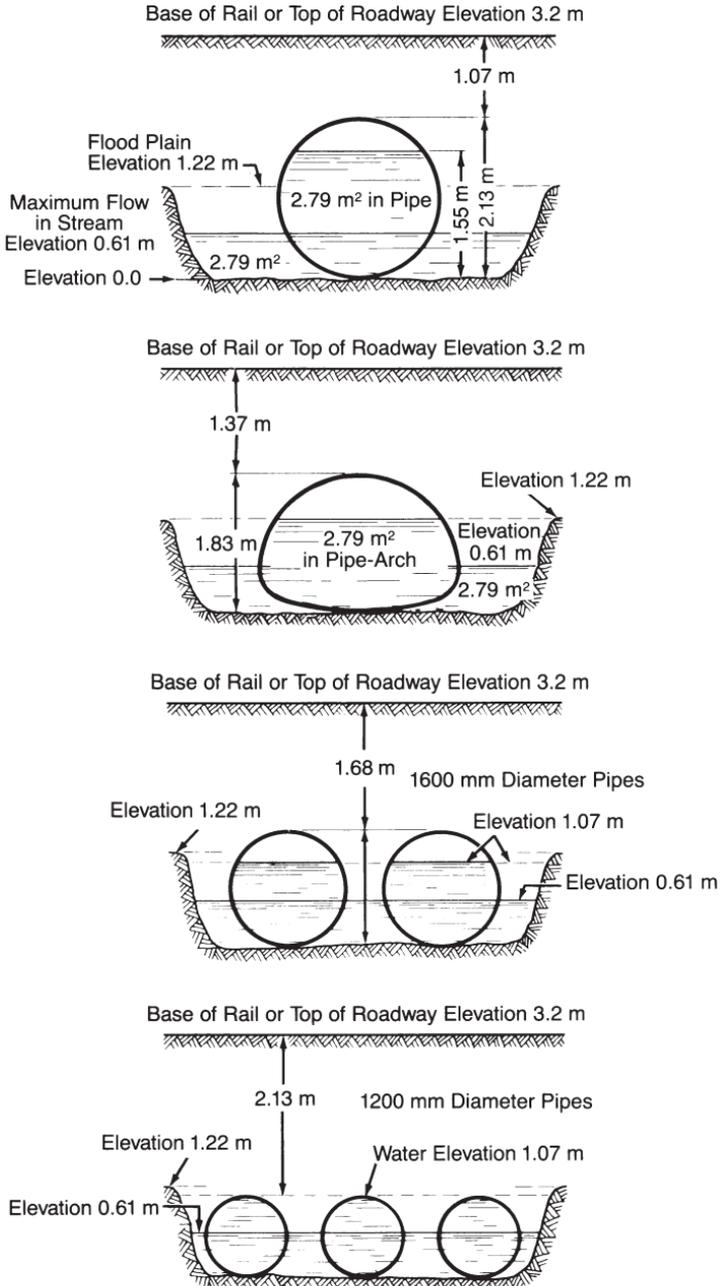


Figure 4.33 Culvert opening choices.

## HYDRAULICS OF SUBDRAINS

### Free Water

Ground water may be in the form of an underground reservoir or it may be flowing through a seam of pervious material. If it is flowing, it may be seeping or percolating through a seam between impervious strata, or be concentrated in the form of a spring.

Free water moves by gravity. It may consist of storm water seeping through cracks in the pavement or entering the ground along the edges of the road. It may be ground water percolating from a higher water-bearing stratum to a lower one, or from a water-bearing layer into the open as in the case of an excavation. A number of subdrainage applications are discussed in Chapter 1.

Water seeping through cracks in the pavement is especially noticeable in springtime and also visible shortly after rains when the remainder of the road has dried off. Passing traffic pumps some of this water, sometimes mixed with subgrade soil, up through the cracks or joints onto the road surface. This water is harmful because it may freeze on the surface and become an unexpected traffic hazard, and it can also destabilize the road subgrade. It can and should be removed in order to establish a stable subgrade and to prevent potential problems.

### Subsurface Runoff Computation

In general, the amount of available ground water is equivalent to the amount of water that soaks into the ground from the surface less the amount that is lost by evaporation and that is used by plants. The nature of the terrain and the catchment area size, shape and slopes, as well as the character and slopes of the substrata, are contributing factors to the amount of ground water available and the volume of subsurface runoff.

A practical way to determine the presence of ground water and the potential flow rate is to dig a trench or test pit. This is helpful especially where an intercepting drain is to be placed across a seepage zone to intercept the ground water and divert the flow, as shown in Figure 4.34.

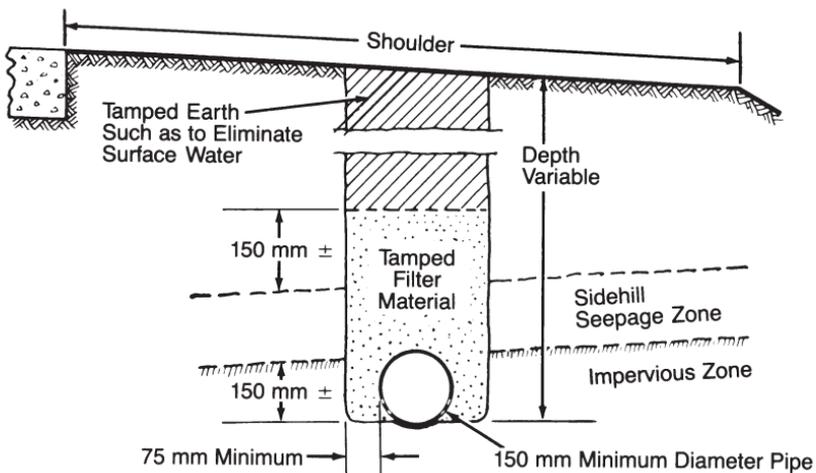


Figure 4.34 Intercepting drain.

Determining a correct size for subdrainage pipe requires an indirect approach. For problems other than those involving large flat areas, size determination becomes a matter of personal judgment and local experience. The following procedure applies to relatively flat areas.

The rate of runoff for average agricultural soils has been determined by agricultural engineering experiment stations to be about 10 mm in 24 hours. For areas of heavy rainfall or more pervious soils, this factor may be increased to 20 or 25 mm. The runoff expressed in mm per 24 hours is converted to  $\text{m}^3/\text{s}/\text{ha}$  for design discharge calculations. Table 4.14 provides a conversion table.

**Table 4.14** Constants for subsurface runoff for various soil permeability types  
Depth of water measured in 24 hours

Soil Permeability Type	Depth, mm	Quantity of Water per Lateral, $\text{m}^3/\text{s}/\text{ha} \times 10^{-4}$ , constant c
Slow to Moderate	2	2.32
Slow to Moderate	4	4.63
Slow to Moderate	6	6.94
Moderate	8	9.26
Moderate	10	11.57
Moderate	12	13.9
Moderate	14	16.2
Moderate to Fast	16	18.5
Moderate to Fast	18	20.8
Moderate to Fast	20	23.1
Moderate to Fast	22	25.5
Moderate to Fast	24	27.8
Moderate to Fast	26	30.1
Moderate to Fast	28	32.5
Moderate to Fast	30	34.7

The design discharge can be calculated using the following formula:

$$Q = CA$$

where:  $Q$  = discharge or required capacity,  $\text{m}^3/\text{s}$   
 $C$  = subsurface runoff factor,  $\text{m}^3/\text{s}/\text{ha}$   
 $A$  = area to be drained, ha

### Example

Assuming a drainage runoff rate of 10 mm in 24 hours (runoff factor,  $C = 11.57 \times 10^{-4}$ ) and laterals 180 m long spaced on 15 m centers, the following result is obtained:

$$Q = (11.57 \times 10^{-4}) \left\{ \frac{180(15)}{10^4} \right\} = 3.12 \times 10^{-4} \text{ m}^3/\text{s}$$

### Size of Pipe

The size of pipe can be determined using Manning's formula, or by the use of a nomograph. For standard subdrainage applications, approximately 150 m of 150 mm diameter perforated steel pipe may be used before increasing to the next size.

Where possible, a minimum slope of 0.15 percent should be used for subdrainage lines. It is often permissible to use an even flatter slope to achieve a free outlet, but the steeper slope provides a self-cleansing flow velocity.

## Geotextiles

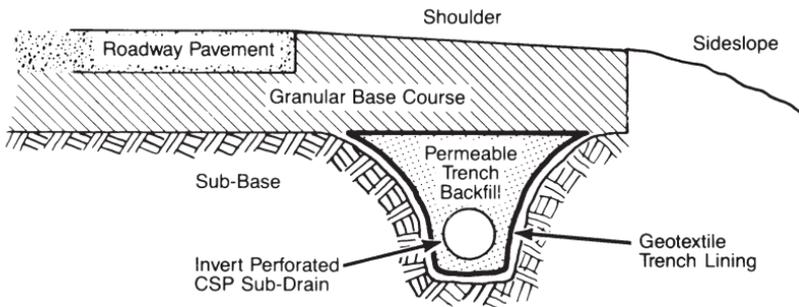
There has been a trend, in recent years, toward the use of geotextiles or filter cloths in lieu of graded aggregate filters. They are used as a filter, to allow the free flow of water into the subdrainage pipe network while preventing fine erodible soils or clogging fines from entering the system, and as a separator, to provide a barrier to soil migration between the surrounding trench wall material and permeable trench backfill.

The diminishing availability and increasing costs of good quality aggregates for graded filters, and the increasing availability and lower cost of geotextiles engineered for these types of applications, has provided the impetus to substitute filter fabrics. Their use also expedites construction and, in many cases, they are used with graded aggregate filters as added insurance against soil migration.

A wide range of filter fabrics are available in a variety of styles and materials. Geotextiles are available as either woven or non-woven products. The properties that are relevant to this application include; permeability, tensile strength, pore size, equivalent opening size (EOS), puncture strength, alkali or acid resistance, freeze-thaw resistance, burst strength, and ultra violet stability.

In the selection of a geotextile, it is important to recognize that the role of the product is as a separator or as a separator and filter. For separation and filtration, the major parameter used for the selection of a filter cloth is the Equivalent or Effective Opening Size (EOS). The choice of fabric EOS must take into consideration the grain size distribution and nature of the soil materials that it is to separate and the desired system permeability. Fabrics for separation and filtration usually have an EOS of between 150 and 200 mm.

A typical cross-section of a filter trench design utilizing filter fabric as a separator/filter is shown in Figure 4.35.



**Figure 4.35** Trench drain utilizing geotextile.

## HYDRAULICS OF STORM WATER INLETS

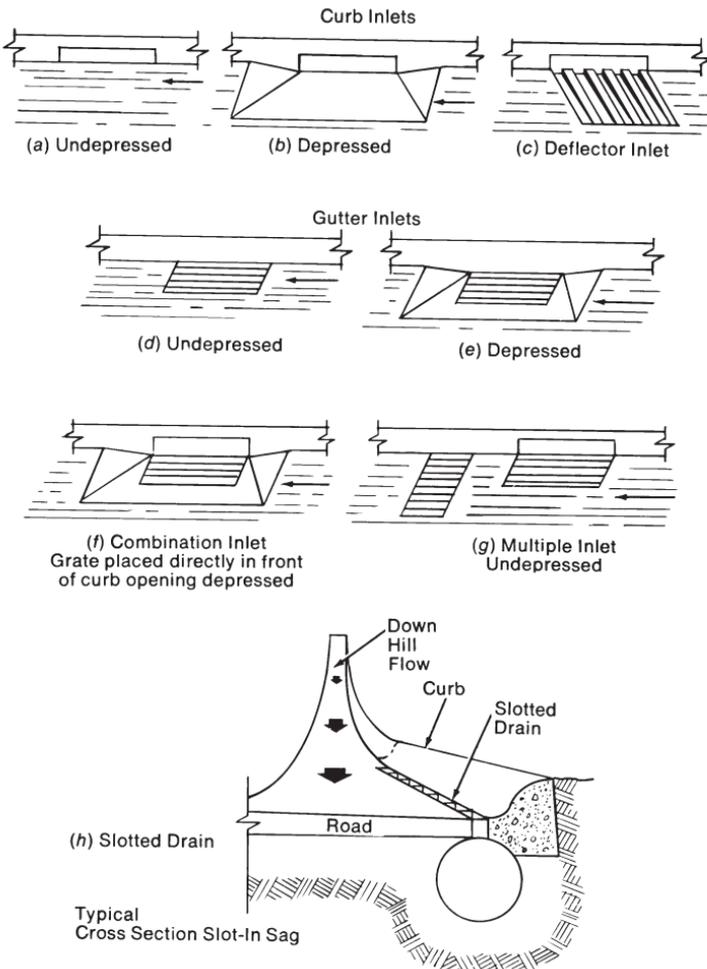
Storm water inlets are the means by which storm runoff enters the sewer system. Their design is often neglected, or it receives very little attention, during the design of storm drainage systems.

If inlets are unable to transmit the design inflow into the sewer system, then the system will not be utilized to its hydraulic design capacity. In some cases, though, it may be desirable to limit the inflow into the sewer system as a means of storm water management. In such cases it is imperative that more emphasis be placed on inlet design to assure that the type, location and capacity of the inlet will achieve the overall drainage requirements.



Slotted drain at work in Montreal, Quebec.

No single inlet type is best suited for all conditions. Many different types of inlets have been developed, as shown in Figure 4.36, based on practical experience and rules of thumb. The hydraulic capacities of some of these inlets is often unknown, resulting in erroneous capacity estimates.



**Figure 4.36** Stormwater inlets.

The hydraulic efficiency of inlets is a function of street grade and cross-slope, and inlet and gutter depression geometry. A steeper street cross-slope will increase the depth of flow in the gutter. Depressed gutters concentrate the flow at the inlets. The depth of flow in a gutter may be estimated from the nomograph in Figure 4.37.

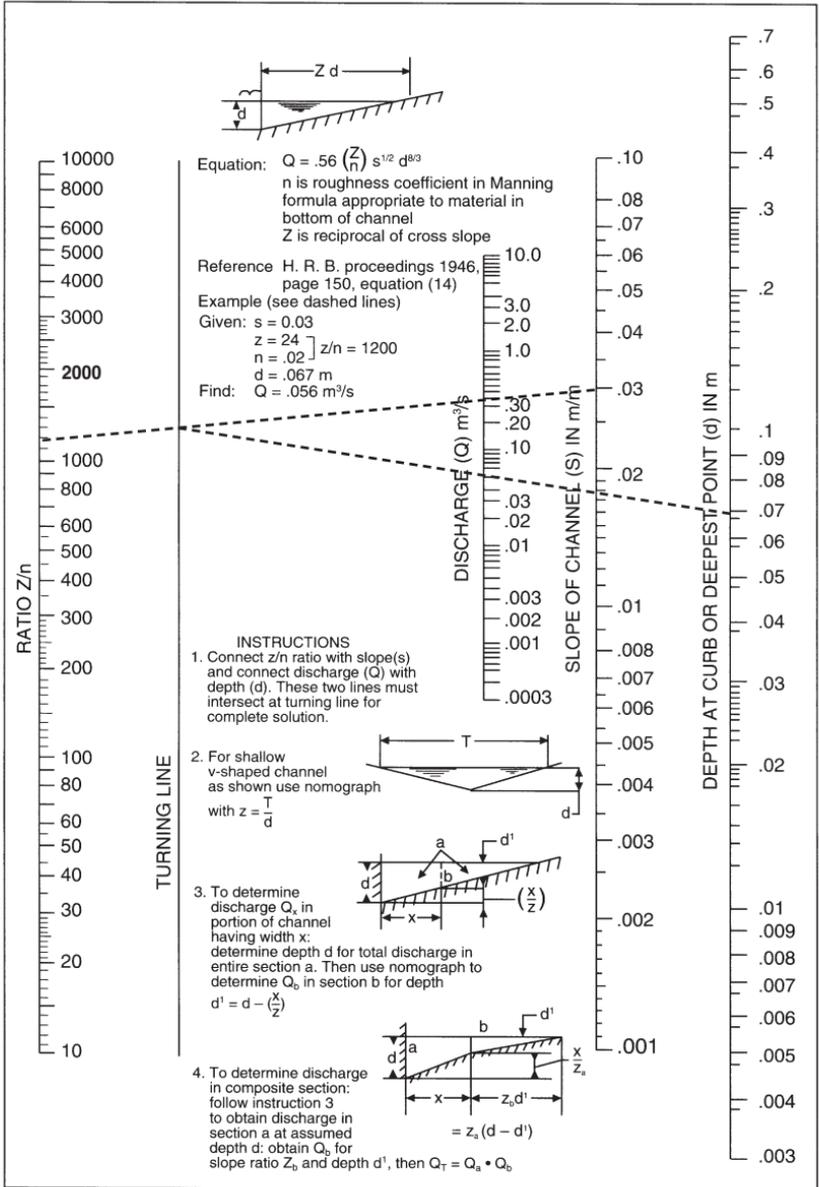


Figure 4.37 Nomograph for flow in triangular channels.

Research work on inlet capacities carried out by various agencies, institutions and municipalities has resulted in the development of empirical equations, hydraulic capacity charts and nomographs to help the designer with storm water inlet selection.

The inlet capacity of an undepressed curb inlet may be expressed by the equation:

$$Q = (4.82 \times 10^{-3}) d l \sqrt{gd}$$

where:  $Q$  = discharge into inlet,  $m^3/s$   
 $d$  = depth of flow in gutter,  $m$   
 $l$  = length of opening,  $m$   
 $g$  = gravitational constant,  $9.806 m/s^2$

If the gutter is of a wedge shape cross section with a street cross-slope of between 0.001 and 0.100  $m/m$ , the inlet capacity of an undepressed curb inlet may be expressed by the equation:

$$Q = 1.29 i^{0.579} l \left\{ \frac{Q_o}{\sqrt{s/n}} \right\}^{0.563}$$

where:  $i$  = cross-slope,  $m/m$   
 $Q_o$  = flow in the gutter,  $m^3/s$   
 $s$  = hydraulic gradient of the gutter (street grade),  $m/m$   
 $n$  = Manning's  $n$  of the gutter

Slotted drain inlets are typically located as spaced curb inlets on a grade (sloping roadway) to collect downhill flow, or located in a sag (low point). The necessary length of slot can be determined using Figures 4.37 through 4.39.

For a series of slotted drain curb inlets on a grade, each inlet will collect all or a major portion of the flow to it. The anticipated accumulated flow at points along the curb can be determined by the methods described above.

Once the initial upstream inlet flow is established, Figure 4.38 is used to determine the required length of slot to accommodate the total flow at the inlet.

The length of slot actually used may be less than required by Figure 4.38. Carryover is that portion of the flow that does not form part of the flow captured by the slotted drain. While some of the flow enters the drain, some flows past the drain to the next inlet. The efficiency of a slotted drain, required in order to consider carryover, is shown in Figure 4.39.

If carryover is permitted, the designer assumes an actual slot length,  $L_A$ , such that the ratio of the actual slot length to the length of slot required for no carryover ( $L_A/L_R$ ) is less than 1.0 but greater than 0.4. Standard slot lengths are 3 and 6  $m$ . Economics favor slotted drain inlets designed to allow carryover rather than for total flow interception. If carryover is allowed, there must be a feasible location to which the carryover may flow.

The slotted drain efficiency can also be calculated using the following equation:

$$E = 1 - 0.918 \left\{ 1 - \frac{L_A}{L_R} \right\}^{1.769}$$

where:  $E$  = efficiency, fraction  
 $L_A$  = actual slot length,  $m$   
 $L_R$  = slot length required for no carryover,  $m$

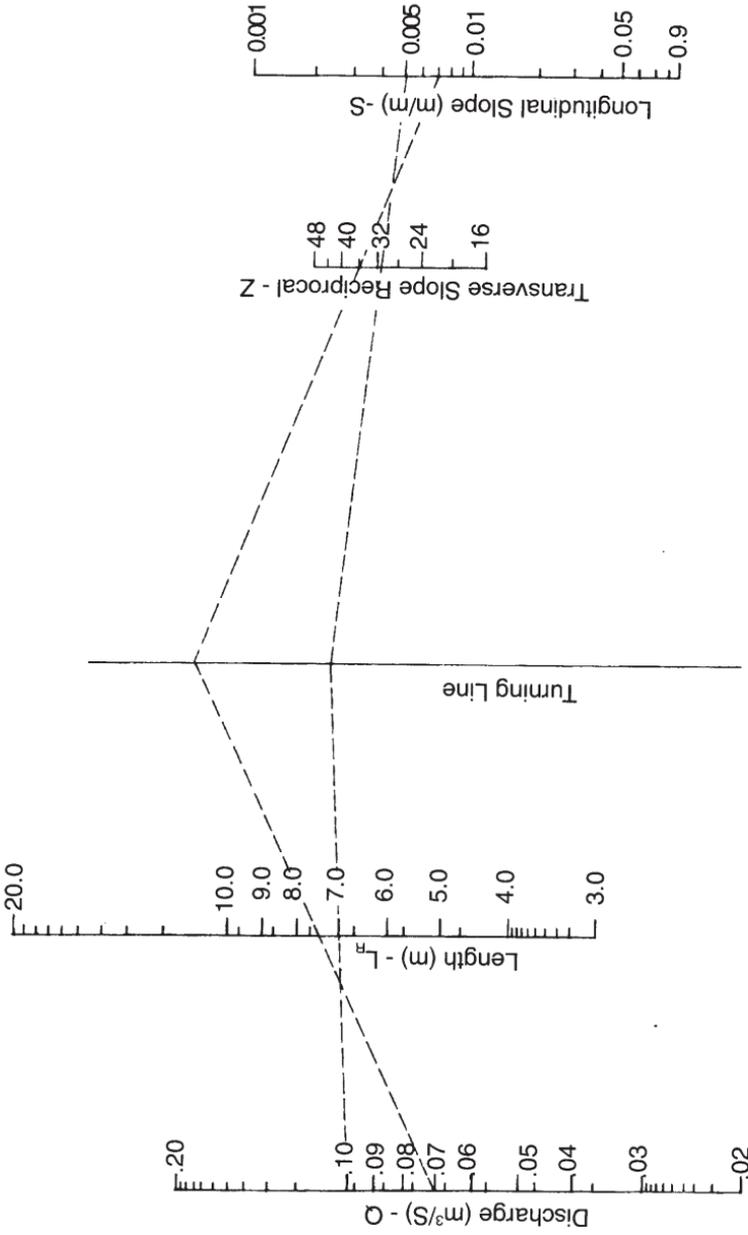


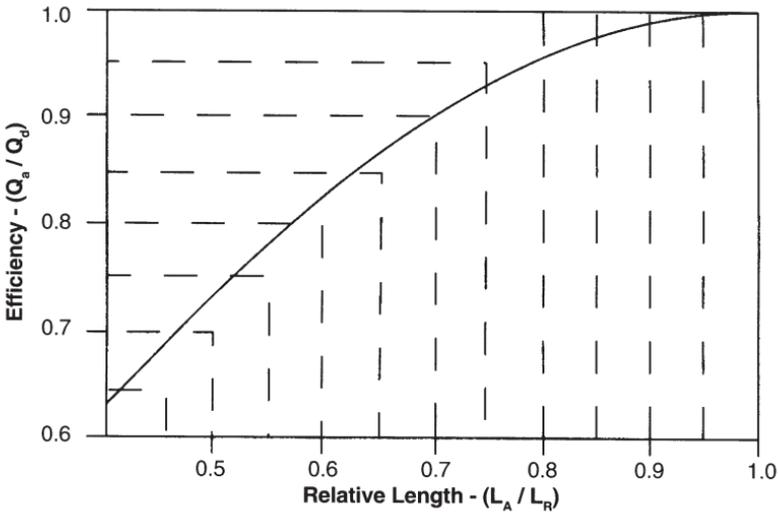
Figure 4.38 Slotted drain design nomograph.



Step-level end treatment.



Structural plate CSP with concrete end treatment.



Example: if 20% carryover ( $Q_a / Q_d = 80\%$ ) is allowed, then only 58% ( $L_A / L_R$ ) of the total slotted drain length is required resulting in a 42% savings in material and installation costs.

**Figure 4.39** Slotted drain carryover efficiency.

The amount of carryover can be calculated using the following equation:

$$CO = Q_d (1 - E)$$

where:  $CO$  = carryover flow,  $m^3/s$   
 $Q_d$  = total design flow,  $m^3/s$

Combining the above two equations results in the following equation for carryover flow:

$$CO = 0.918 Q_d \left\{ 1 - \frac{L_A}{L_R} \right\}^{1.769}$$

When slotted drain is used for sag inlets, the required slot length should be based on the orifice equation, which is:

$$Q_d = C A \sqrt{2gd}$$

where:  $C$  = orifice coefficient = 0.61  
 $A$  = open area of slot based on the width for which the hydraulic characteristics were measured (0.044 m),  $m^2 = L_R (0.044)$   
 $g$  = gravitational constant,  $9.806 m/s^2$   
 $d$  = maximum allowable depth of water in the gutter, m

Solving for the required slot length:

$$L_R = \frac{8.413 Q_d}{\sqrt{d}}$$

For a slotted drain in a sag at the end of a series of drains on a grade, the flow to the drain will include any carryover from the immediately adjacent drain up grade. Unlike a drain-on-grade situation, a slotted drain in a sag will produce significant ponding if its capacity will not accommodate the design flow. Therefore, the actual length of sag inlets should be at least 2 times the calculated required length.

Carryover is not usually permitted at level grade inlets. In that case, the actual slotted drain length must be at least the required length.



Attractive end finishes can be developed.

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