CHAPTER four

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:

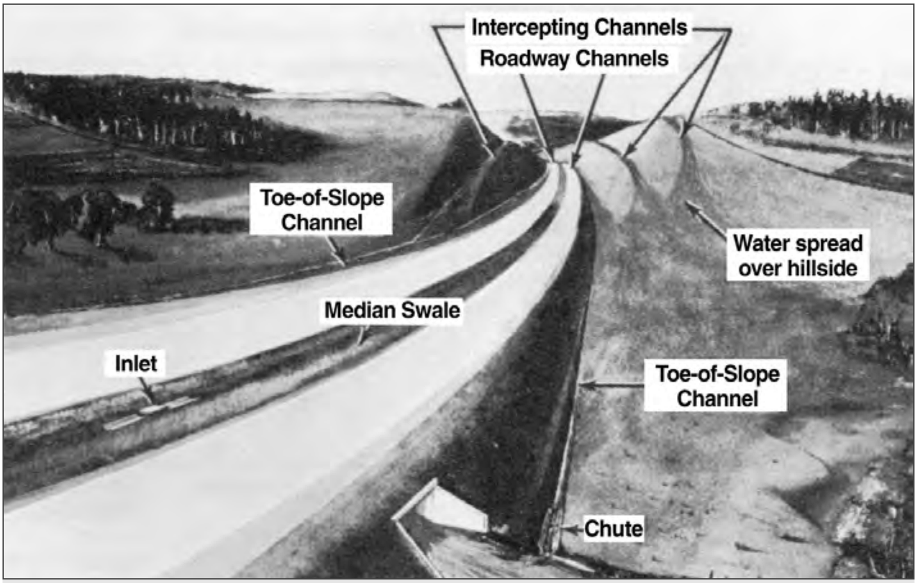
> With the exception of the riding qualities of the traveled way, no other single item requires as much attention on the part of maintenance personnel as highway culverts. Many of the problems of culvert maintenance stem from the fact that designers in all too many instances consider that culverts will 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 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).

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 Highways 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. Therefore maintenance should be considered in the design of all channels.



■ **Figure 4.1** Types of roadside drainage channels.

Chezy Equation

Chezy developed a basic hydraulic relationship 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, ft^3/s
- A = cross-sectional area of flow, ft^2
- V = mean velocity of flow, ft/s
- c = coefficient of roughness, depending upon the surface over which water is flowing, $ft^{1/2}/s$
- R = hydraulic radius, ft $= \dfrac{A}{WP}$
- WP = wetted perimeter (length of wetted contact between water and its containing channel), ft
- S = slope, or grade, ft/ft

This fundamental equation is the basis of most capacity formulations.

Manning's Equation

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

$$c = 1.486 \frac{R^{1/6}}{n}$$

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

Table 4.1

Manning's *n* for constructed channels

Types 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 100 ft.

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:

$$V = 1.486 \frac{R^{2/3}S^{1/2}}{n}$$

Combining this with the Chezy Equation results in the equation:

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

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

$$K = 1.486 \frac{AR^{2/3}}{n}$$

then:

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

Uniform flow of clean water in a straight unobstructed channel would be an ideal condition, but it is rarely found. Manning's equation 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

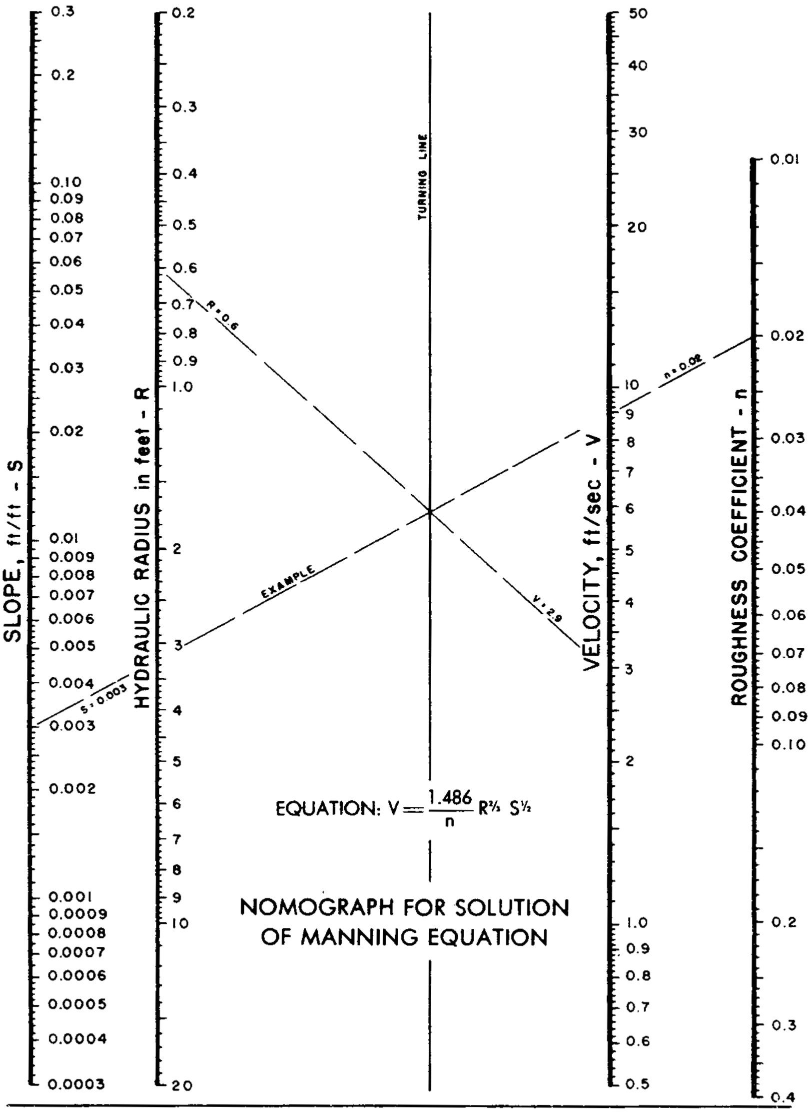
Given: A trapezoidal channel of straight alignment and uniform cross section in earth with a bottom width of 2 feet, side slopes at 1:1, a channel slope of 0.003 ft/ft, and a normal depth of water of 1 foot.

Find: Velocity and discharge.

Solution:

1. Based on Table 4.1, for an excavated channel in ordinary earth, n is taken as 0.02.
2. Cross-sectional area, A, is 3 ft^2 [1 * (2 + 1 * 1)].
3. Wetted perimeter, WP, is 4.83 ft [2 + 2 *(1 * $(1^2+1^2)^{1/2}$)].
4. Hydraulic radius, R, is 0.62 ft [3 / 4.83].
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 0.62 ft.
7. Read the velocity, V, of 2.9 ft/s on the velocity scale.
8. Discharge, Q, is 8.7 ft^3/s[3 * 2.9].

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 either known or determined using known site parameters through trial and error. The flow rate, Q, can then be calculated.



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

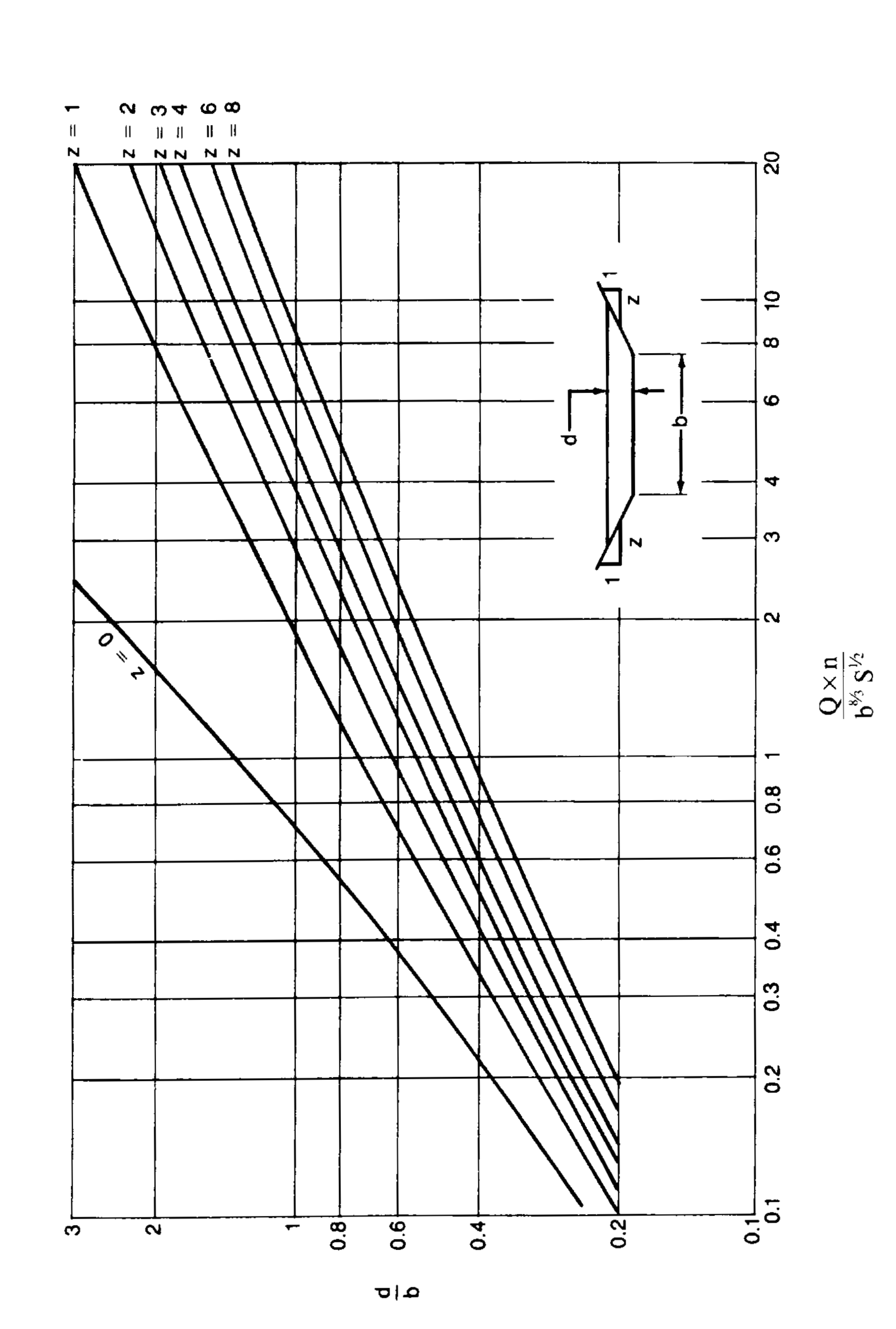


Figure 4.3 Capacity of trapezoidal channel.

Example 2

Given: Bottom width, b = 20 ft
Side slopes @ 2:1, z = 2
Roughness coefficient, n = 0.030
(from Table 4.2 for grass and weeds, no brush)
Channel slope, S = 0.002 ft/ft
Depth to bottom width ratio, $^d/_b$ = 0.6 (flood stage depth)

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

Solution:

1. Depth, d = 12 ft [0.6 * 20]
2. From Figure 4.3:

$$\frac{Q \bullet n}{b^{8/3}\ S^{1/2}} = 0.92$$

3. So:

$$\frac{Q\ (0.030)}{20^{8/3}\ (0.002)^{1/2}} = 0.92$$

4. And: $Q = 4042\ ft^3/s$

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 HEC 15, Design of Stable Channels with Flexible Linings, U.S. Federal Highway Administration (FHWA).

Table 4.3

Comparison of water velocity limits and tractive force values for the design of stable channels

Material	*n*	For Clear Water Velocity ft/sec	For Clear Water Tractive* Force lb/ft^2	Water Transporting Colloidal Silts Velocity ft/sec	Water Transporting Colloidal Silts Tractive* Force lb/ft^2
Fine sand colloidal	0.020	1.50	0.027	2.50	0.075
Sandy loam noncolloidal	0.020	1.75	0.037	2.50	0.075
Silt loam noncolloidal	0.020	2.00	0.048	3.00	0.11
Alluvial silts noncolloidal	0.020	2.00	0.048	3.50	0.15
Ordinary firm loam	0.020	2.50	0.075	3.50	0.15
Volcanic ash	0.020	2.50	0.075	3.50	0.15
Stiff clay very colloidal	0.025	3.75	0.26	5.00	0.46
Alluvial silts colloidal	0.025	3.75	0.26	5.00	0.46
Shales and hardpans	0.025	6.00	0.67	6.00	0.67
Fine gravel	0.020	2.50	0.075	5.00	0.32
Graded loam to cobbles when non-colloidal	0.030	3.75	0.38	5.00	0.66
Graded silts to cobbles when colloidal	0.030	4.00	0.43	5.50	0.80
Coarse gravel noncolloidal	0.025	4.00	0.30	6.00	0.67
Cobbles and shingles	0.035	5.00	0.91	5.50	1.10

* Tractive force or shear is the force which the water exerts on the periphery of a channel due to the motion of the water. The tractive values shown were computed from velocities given by S. Fortier and Fred C. Scobey and the values of *n* shown.

The tractive force values are valid for the given materials regardless of depth. For depths greater than 3 ft, higher velocities can be allowed and still have the same tractive force.

From *U.S. Bureau of Reclamation*, Report No. Hyd-352, 1952, 60 pp.

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 the pipe is 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 4.4

Maximum permissible velocities in vegetal-lined channels

Cover Average, Uniform Stand, Well Maintained	Slope Range Percent	Permissible Velocity[a] Erosion Resistant Soils ft/sec	Permissible Velocity[a] Easily Eroded Soils ft/sec
Bermudagrass	0 - 5 5 - 10 over 10	8 7 6	6 5 4
Buffalograss Kentucky bluegrass Smooth brome Blue grama	0 - 5 5 - 10 over 10	7 6 5	5 4 3
Grass mixture[b]	5 - 10	5 4	4 3
Lespedeza sericea Weeping lovegrass Yellow bluestem Alfalfa Crabgrass	0 - 5	3.5	2.5
Common lespedeza[b] Sudangrass[b]	0 - 5[c]	3.5	2.5

a From "Engineering Field Manual" USDA - Soil Conservation Service, 1979, (now Natural Resource Conservation Service).
b Annuals—used on mild slopes or as temporary protection until permanent covers are established .
c Use on slopes steeper than 5 percent is not recommended.

Asphalt and concrete lined channels are used for steep erodible channels. Ditch checks are an effective means of decreasing the velocity and thereby the erodibility of the soil. High velocities, where water discharges from a channel, must be considered and provisions 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 culvert design 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.

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.

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 Department of Transportation uses an excellent method of accomplishing this, which has worked well for many years. Other states and agencies 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 such as those published by the U.S. FHWA. It provides a usable rationale for determining a minimum required waterway area. This design method is highly recommended and is followed here in conjunction with FHWA charts.

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.

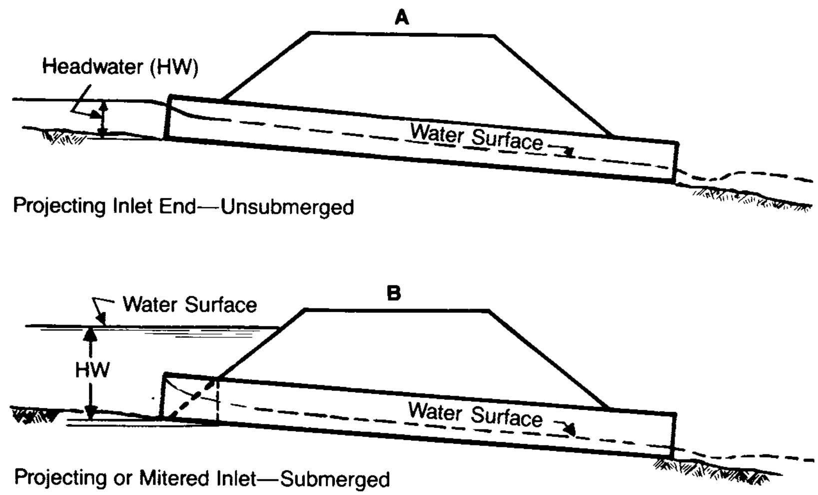
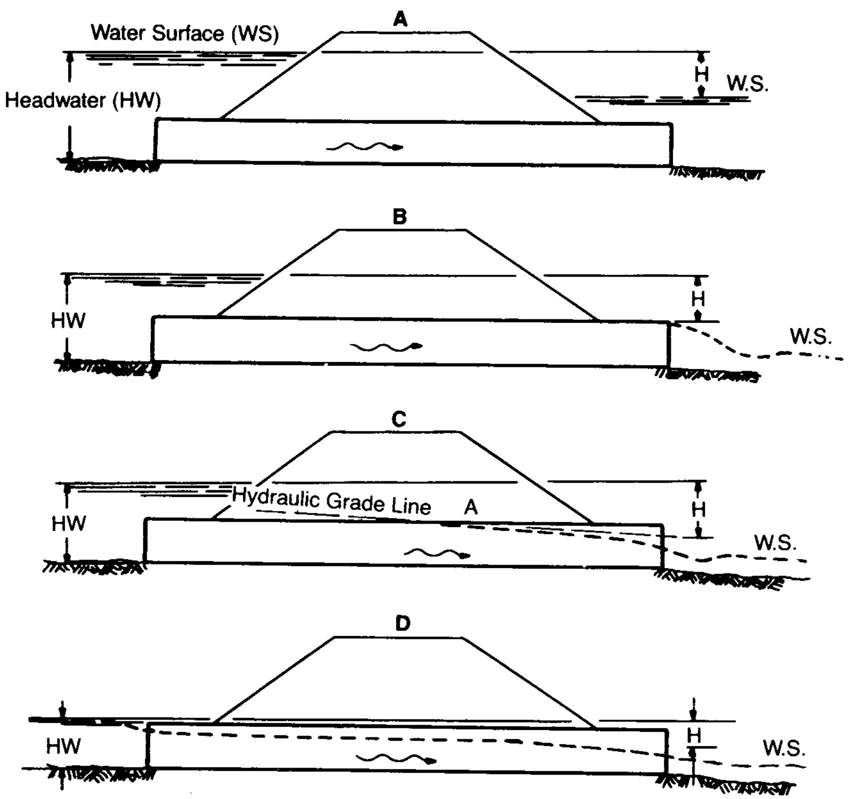


Figure 4.4 Inlet control flow regimes.

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).



■ **Figure 4.5** Outlet control flow regimes.

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 respectively. Inlet control performance is classified by these two regimes (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 pipes and pipe arches

Inlet End of Culvert	Entrance Type	Coefficient, k_e
Projecting from fill (no headwall)	1	0.9
Mitered (beveled) 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
Beveled Ring	6	0.25

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 design manual.

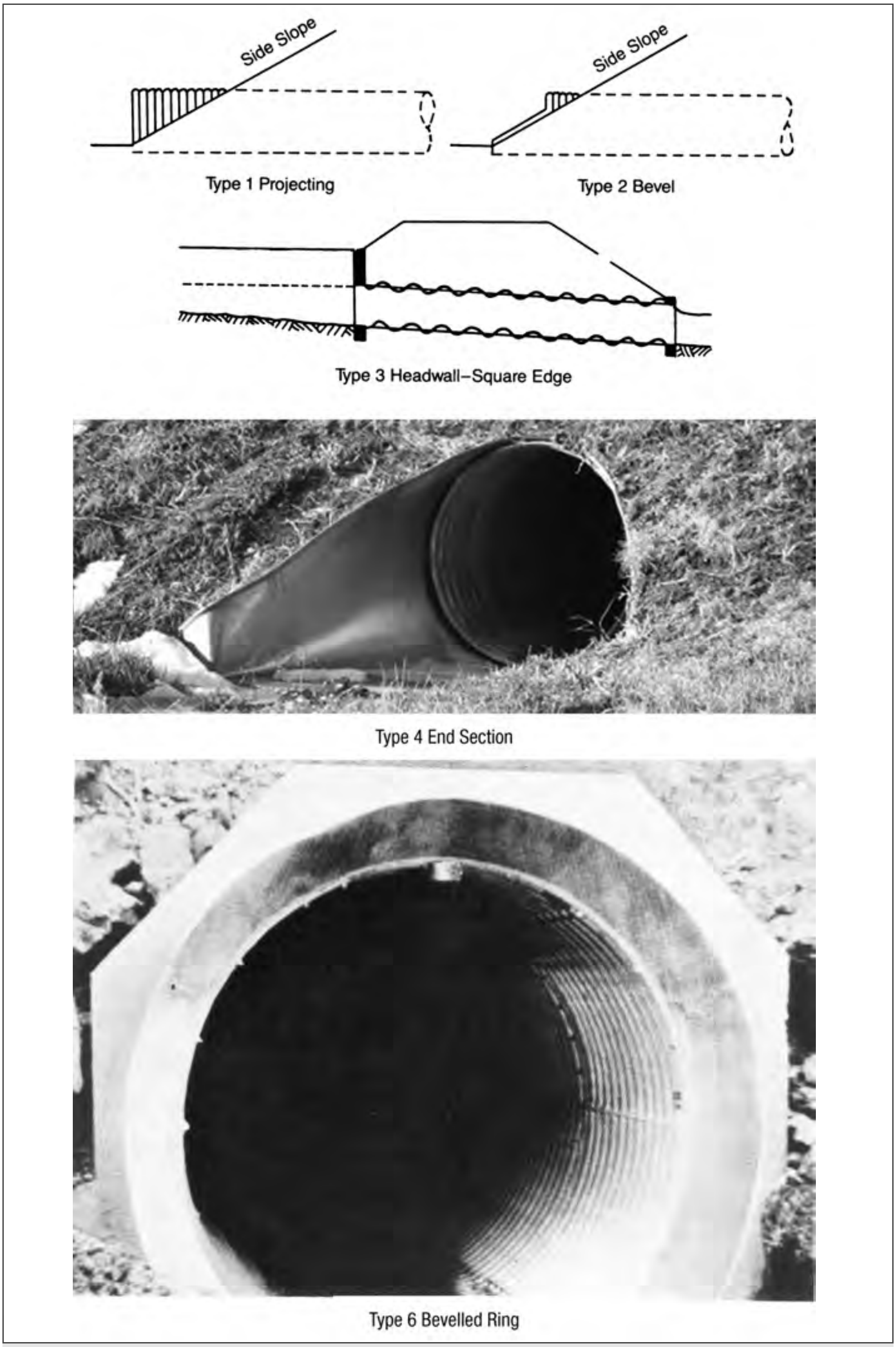
In the nomographs, the headwater depth (HW) is the vertical distance from the culvert invert (bottom) at the entrance to the energy grade line of the headwater pool. It therefore includes the approach velocity head. The velocity head tends to be relatively small and is often neglected. The resulting 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:

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

where:
- L = length of culvert, ft
- S_o = slope of barrel, ft/ft
- HW = headwater depth, ft
- V_1 = approach velocity, ft/s
- g = gravitational constant = 32.2 ft/s^2
- h_o = outlet datum, ft
- H = head, ft
- V_2 = downstream velocity, ft/s



■ **Figure 4.6** Typical entrance types.

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.

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

The head, or energy (Figures 4.7 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.

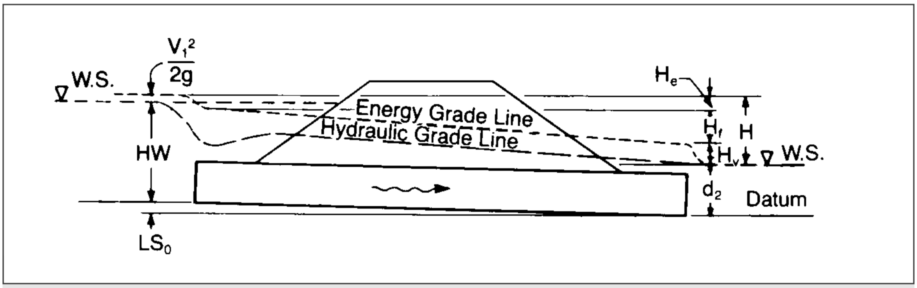


Figure 4.7 Definition of terms in energy balance equation.

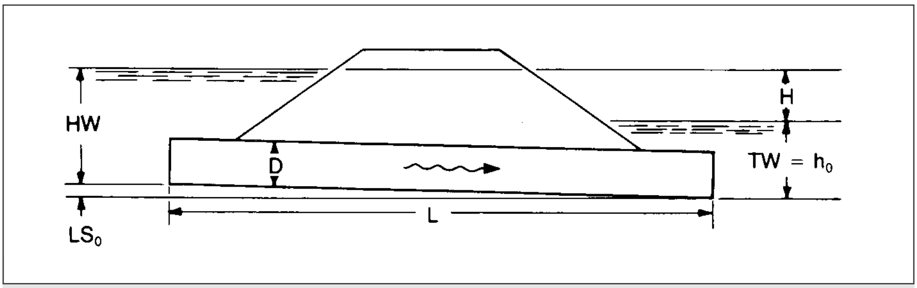


Figure 4.8 Terms of the energy balance equation related to a high tailwater condition.

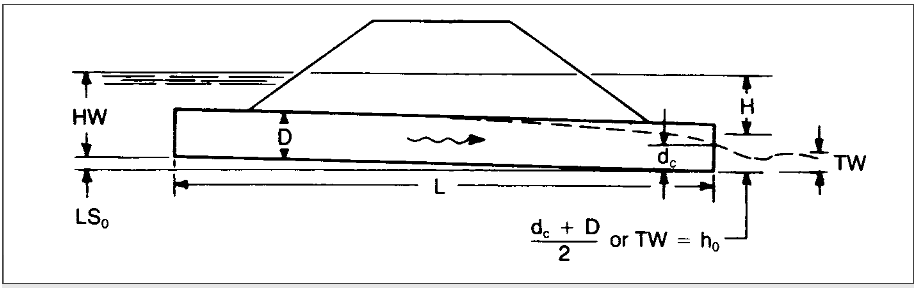


Figure 4.9 Terms of the energy balance equation related to a low tailwater condition.

This energy is expressed in equation form as:

$$H = H_e + H_f + H_o$$

where: H_e = entrance loss, ft
H_f = friction loss, ft
H_o = exit loss, ft

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, ft
V = mean velocity of flow in the barrel, ft/s = Q / A
Q = design discharge, ft^3/s
A = cross sectional area of the culvert, ft^2

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:

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

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:

$$H_f = \left\{ \frac{29 n^2 L}{R^{1.33}} \right\} \frac{V^2}{2g}$$

where:
- n = Manning's friction factor (see Tables 4.6 and 4.7)
- R = hydraulic radius, ft = A / WP
- WP = wetted perimeter, ft

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):

$$H_o = \left\{ k_e + \frac{29 n^2 L}{R^{1.33}} + 1 \right\} \frac{V^2}{2g}$$

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 $L \bullet S_o$).

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 either the critical depth or the tailwater depth, whichever is greater (Figure 4.9).

Table 4.6

Values of coefficient of roughness (Manning's n) for standard corrugated steel pipes

		2-2/3 x 1/2	Helical Corrugation, Pitch x Rise (in.)										
		Annular	1-1/2 x 1/4		2-2/3 x 1/2								
Flowing	Finish	Corrugation	Diameter (in.)										
		All Dia.	8	10	12	15	18	24	30	36	42	48	≥ 54
Full	Unpaved	0.024	0.012	0.014	0.011	0.012	0.013	0.015	0.017	0.018	0.019	0.020	0.021
Full	25% paved	0.021						0.014	0.016	0.017	0.018	0.020	0.019
Part Full	Unpaved	0.027			0.012	0.013	0.015	0.017	0.019	0.020	0.021	0.022	0.023
		All	Pipe Arch Span x Rise (in.)										
		Pipe Arches				17 x 13	21 x 15	28 x 20	35 x 24	42 x 29	49 x 33	57 x 38	≥ 64 x 43
Full	Unpaved	0.026				0.013	0.014	0.016	0.018	0.019	0.020	0.021	0.022
Part Full	Unpaved	0.029				0.018	0.016	0.021	0.023	0.024	0.025	0.025	0.026
		3 x 1	Helical Corrugation, Pitch x Rise (in.)										
		Annular	3 x 1										
		Corrugation	Diameter (in.)										
		All Dia.				36	42	48	54	60	66	72	≥ 78
Full	Unpaved	0.027				0.022	0.022	0.023	0.023	0.024	0.025	0.026	0.027
Full	25% Paved	0.023				0.019	0.019	0.020	0.020	0.021	0.022	0.022	0.023
		5 x 1	Helical Corrugation, Pitch x Rise (in.)										
		Annular	5 x 1										
		Corrugation	Diameter (in.)										
		All Dia.						48	54	60	66	72	≥ 78
Full	Unpaved	0.025						0.022	0.022	0.023	0.024	0.024	0.025
Full	25% Paved	0.022						0.019	0.019	0.020	0.021	0.021	0.022
			All Diameters										
Smooth Interior Pipe (1)			0.012										

Note (1): Includes fully paved, concrete lined. ribbed pipe and double wall pipe.

Table 4.7

Values of coefficient of roughness (Manning's n) for structural plate pipe, 6 in. x 2 in. corrugations

Corrugations 6 x 2 in.	Diameters			
	5 ft	7 ft	10 ft	15ft
Plain – unpaved	0.033	0.032	0.030	0.028
25% Paved	0.028	0.027	0.026	0.024

The backwater calculations can be tedious and time consuming. Approximation 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 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:

$$S_n = \frac{H_f}{L} = \left\{\frac{29n^2}{R^{1.33}}\right\}\frac{V^2}{2g}$$

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 method approximation 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 by used with this method of approximation. 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 5 x 1 inch corrugations have been based on tests conducted using 6 x 2 inch 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 ranged from 0.0328 for 5 foot diameter pipe to 0.0302 for 15 foot diameter pipe.

In 1968, the first full-scale measurements were made on a 1500 foot long 14 foot 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 10 foot 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 is considered one in which the approach establishes a minimum opening required to pass, for example, a 10-year flood with no ponding. In this example, 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 then 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 4.17. Round pipes, pipe arches, and arches are included, as indicated.

These figures are first used to determine the pipe size required so 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 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.

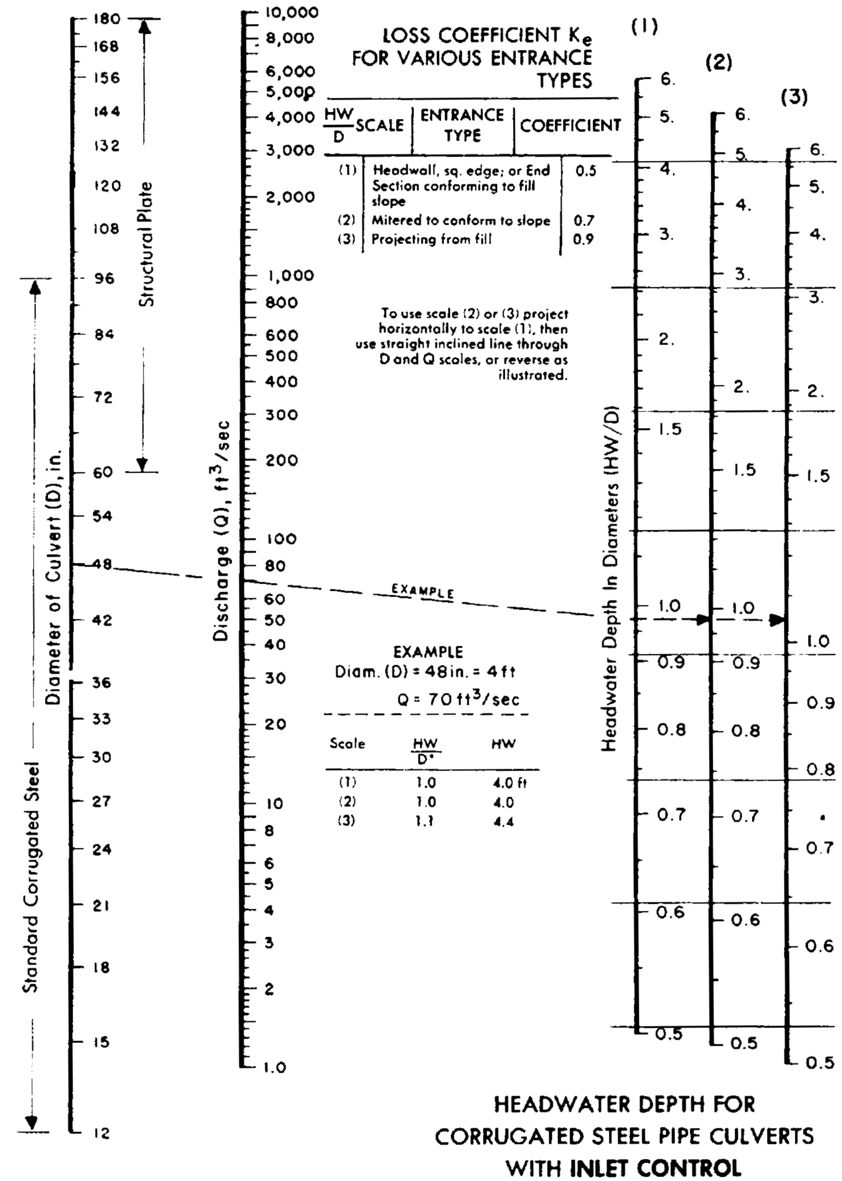
The values from the nomographs give the headwater in terms of a number of pipe rises (HW/D). The following equation is then used to calculate the headwater depth:

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

where: HW_i = headwater depth under inlet control, ft

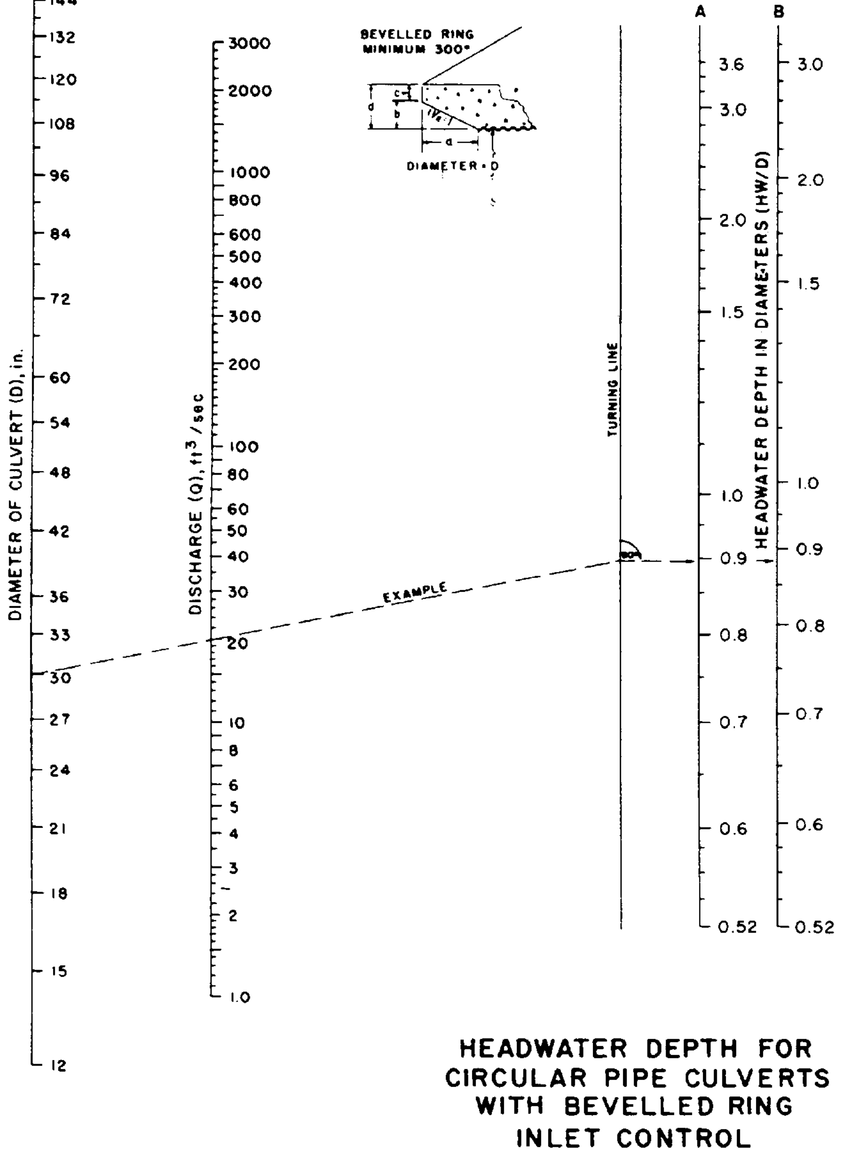
$\frac{HW}{D}$ = headwater depth in number of pipe rises, from nomograph, ft/ft

D = diameter of pipe, or rise of arch or pipe arch, ft

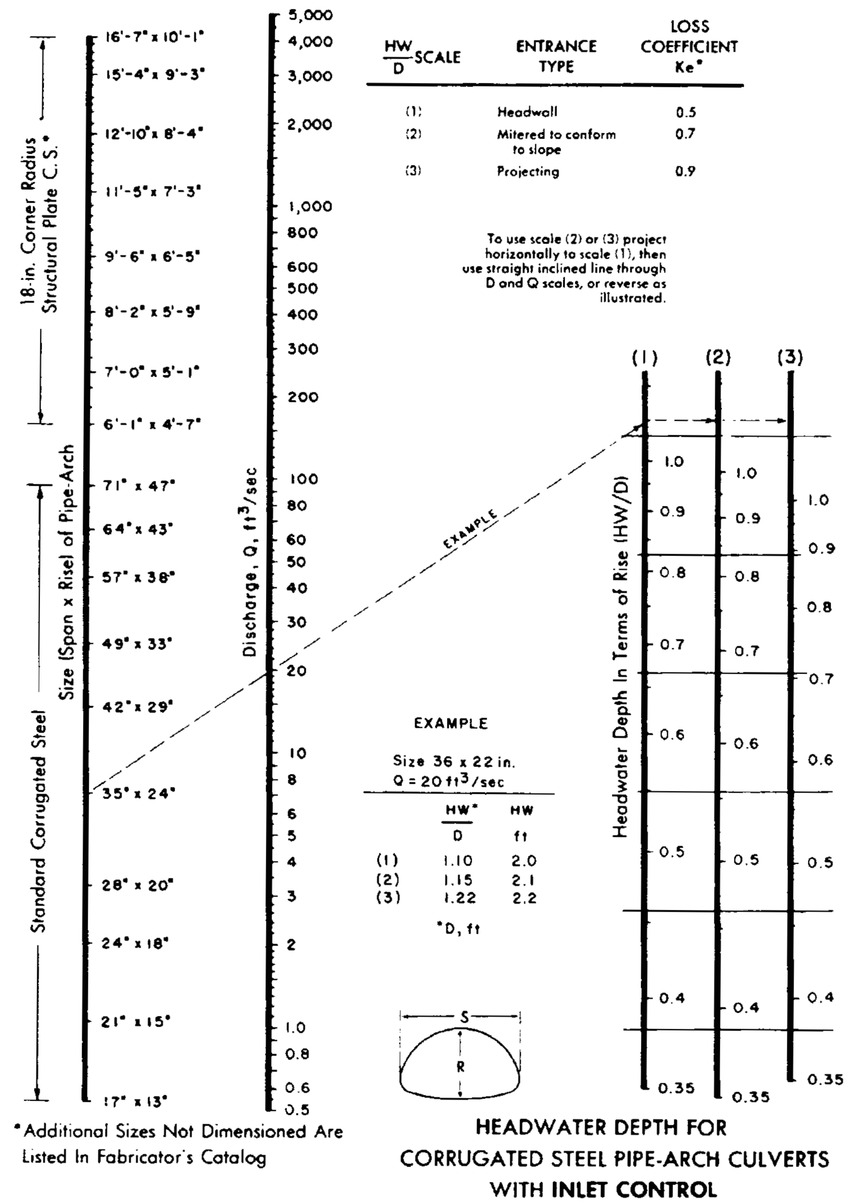


■ **Figure 4.10** Headwater depth for round corrugated steel pipes and structural plate corrugated steel pipes under inlet control.

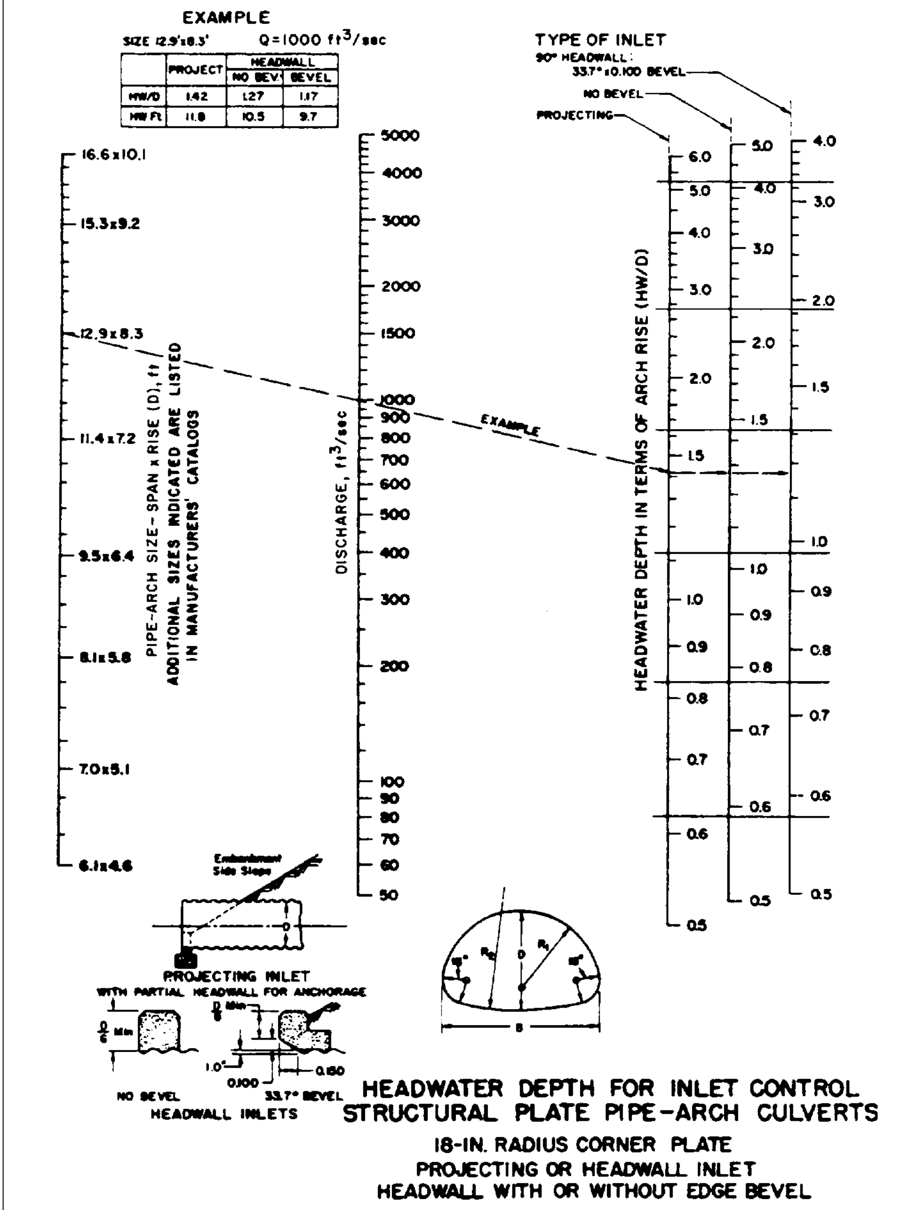
b/D	a/D	c/D	d/D	ENTRANCE TYPE
0.042	0.063	0.042	0.083	A
0.083	0.125	0.042	0.125	B



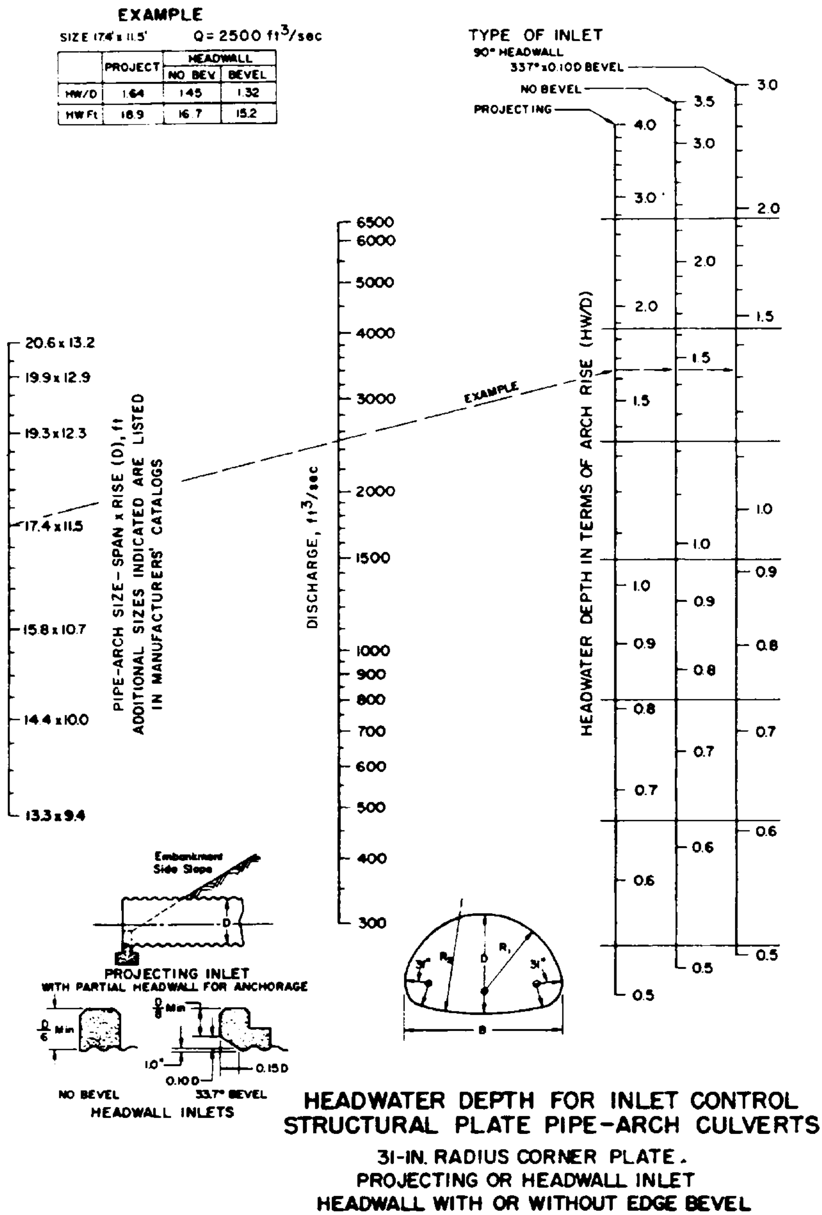
■ **Figure 4.11** Headwater depth for round corrugated steel pipes, with beveled ring headwall, under inlet control.



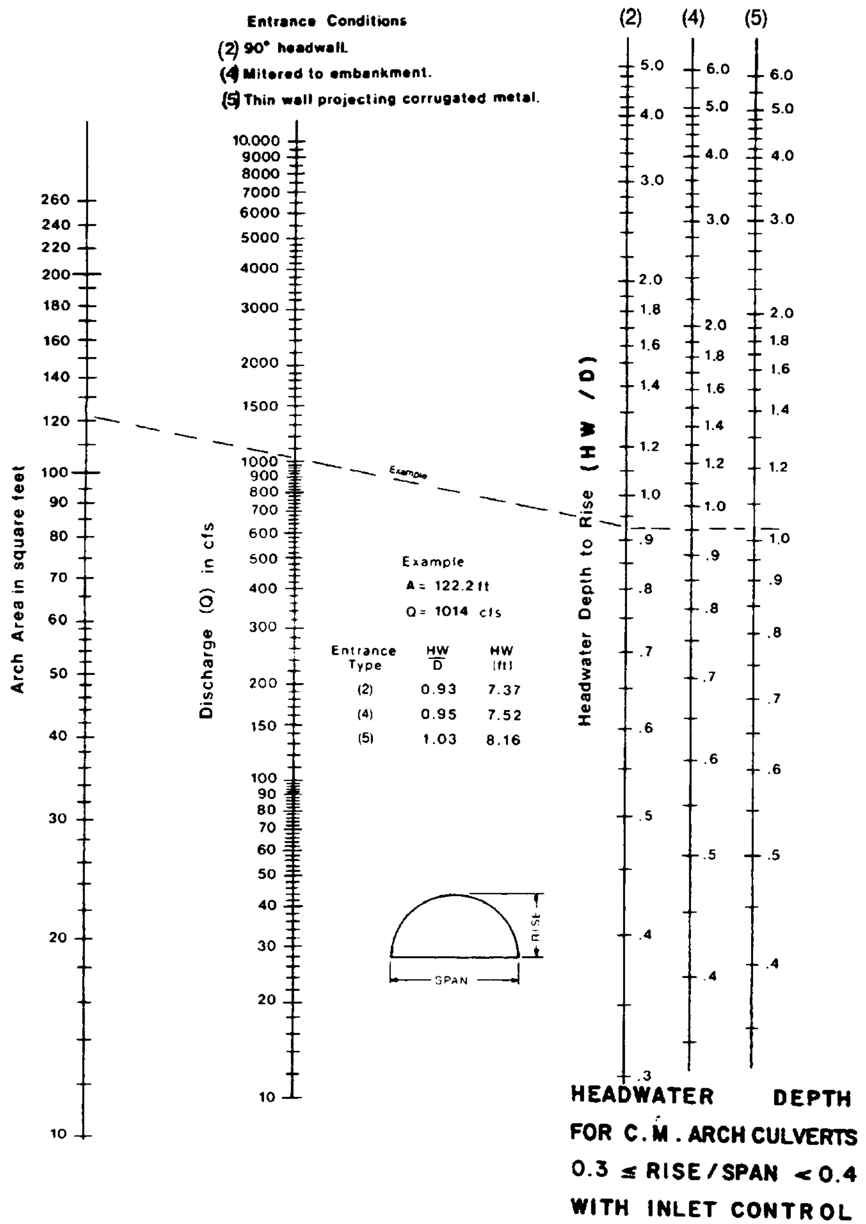
■ **Figure 4.12** Headwater depth for corrugated steel and structural plate corrugated steel pipe arches under inlet control.



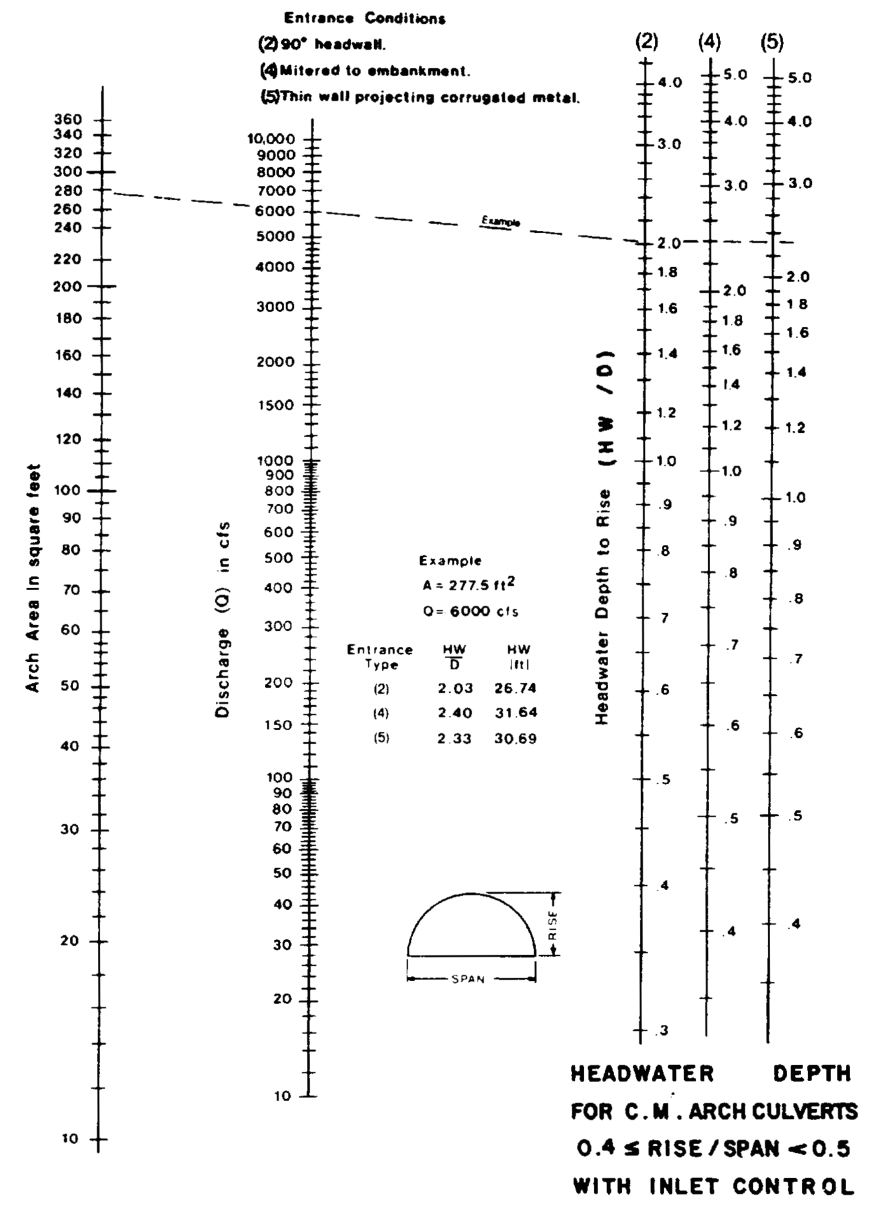
■ **Figure 4.13** Headwater depth for structural plate corrugated steel pipe arches, with 18-in. radius corner plate, under inlet control.



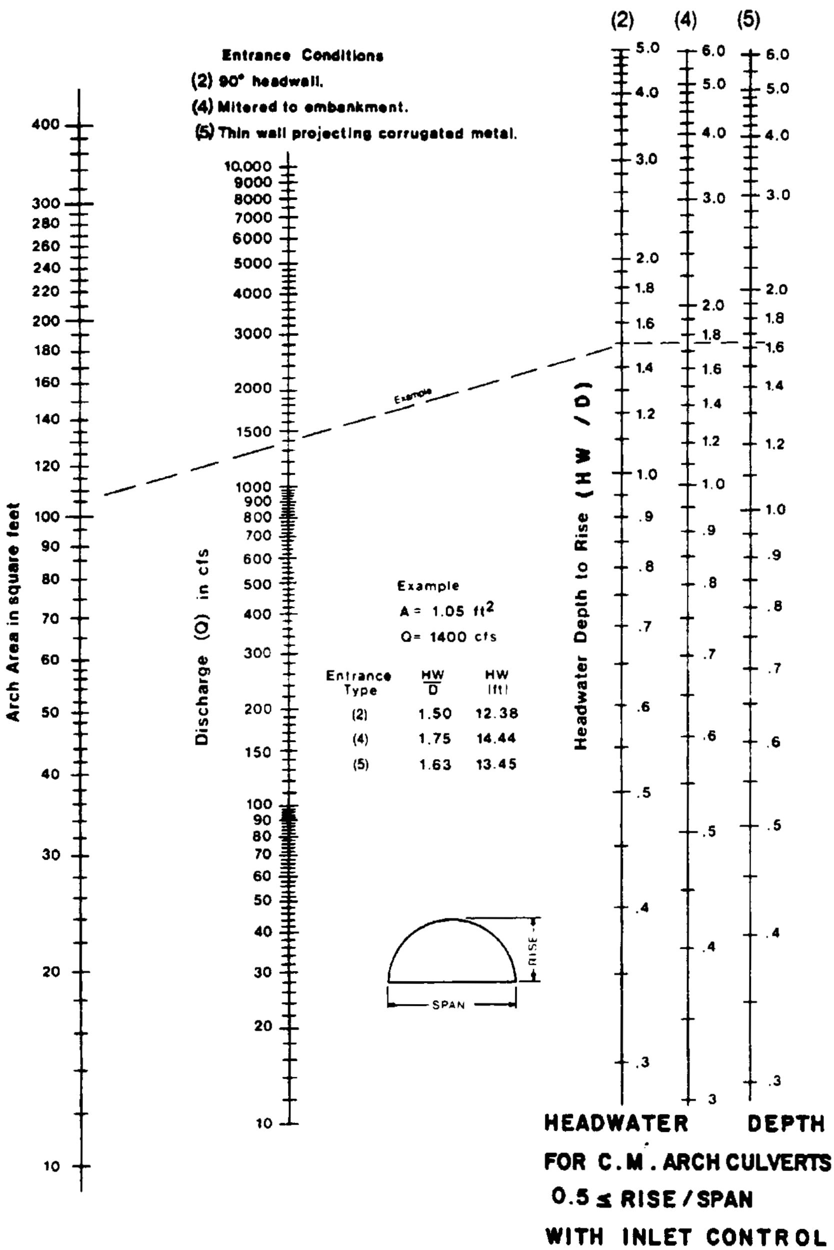
■ **Figure 4.14** Headwater depth for structural plate corrugated steel pipe arches, with 31-in. radius corner plate, under inlet control.



■ **Figure 4.15** Headwater depth for structural plate corrugated steel arches, with 0.3 <= rise/span < 0.4, under inlet control.



■ **Figure 4.16** Headwater depth for structural plate corrugated steel arches, with 0.4 <= rise/span < 0.5, under inlet control.



■ **Figure 4.17** Headwater depth for structural plate corrugated steel arches, with 0.5 <= rise/span, under inlet control.

Outlet Control

Figures 4.18 through 4.27 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.

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

where: HW_o = headwater depth under outlet control, ft
h_o = outlet datum, ft; the greater of the tailwater depth, TW, or $\frac{(d_c + D)}{2}$
H = head, from nomograph, ft
L = length of culvert barrel, ft
S_o = slope of culvert barrel, ft/ft
TW = depth of flow in channel at culvert outlet, ft
d_c = critical depth, from Figures 4.28 through 4.31, ft
D = diameter of pipe, or rise of arch or pipe arch, ft

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 equation below. This value is then used on the length scale of the nomograph, rather than the actual culvert length.

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

where L' = adjusted length for use in nomographs, ft
L = actual length, ft
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 listed 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 pipes

Pipe Diameter, in.	Roughness Factor n′ for Helical Corr.	Length Adjustment Factor $\left(\frac{n'}{n}\right)^2$
12	0.011	0.21
24	0.016	0.44
36	0.019	0.61
48	0.020	0.70

Values of Manning's n for structural plate corrugated steel pipe, which were determined in the 1968 full-scale field measurements and were listed 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

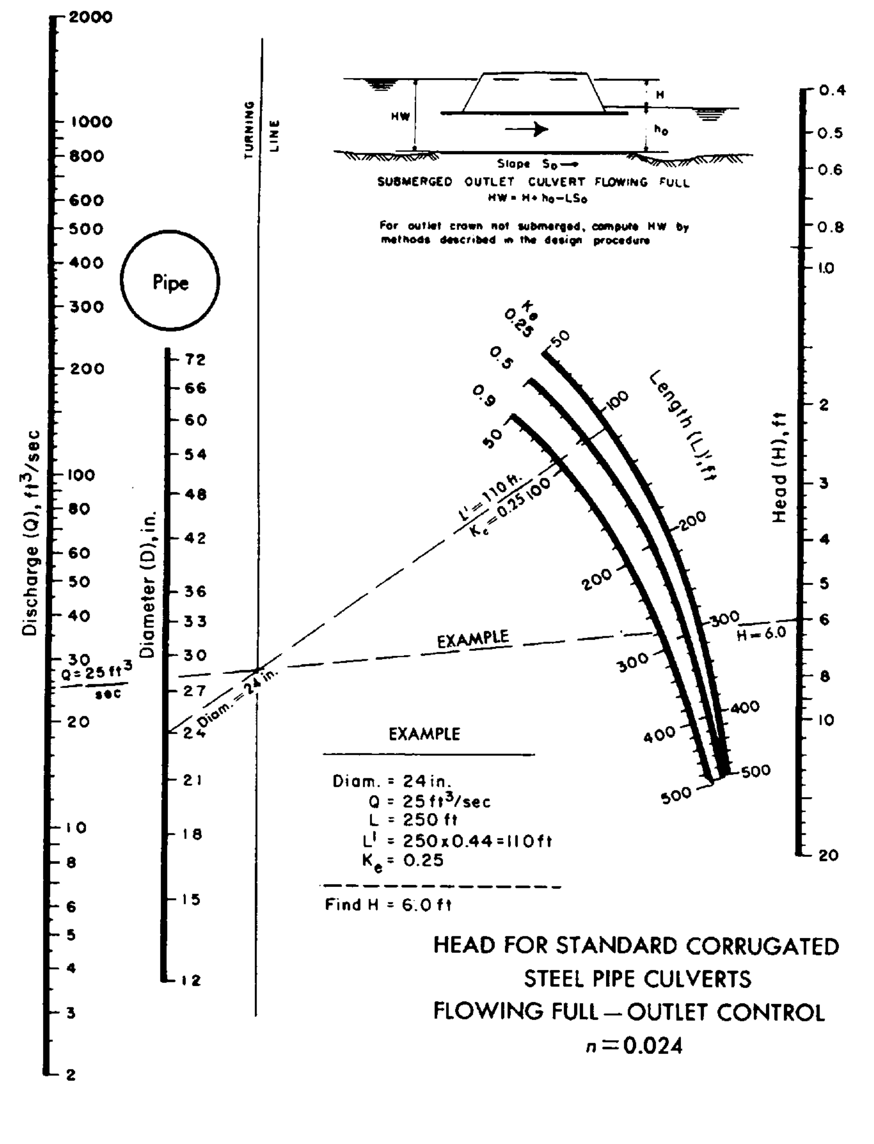
Length adjustment factors for 6 x 2 in. corrugation structural plate pipe

Pipe Diameter, ft	Roughness Factor		Length Adjustment Factor $\left(\frac{n'}{n}\right)^2$
	Curves Based on n =	Actual n' =	
5	0.0328	0.033	1.0
7	0.0320	0.032	1.0
10	0.0311	0.030	0.93
15	0.0302	0.028	0.86
Pipe Arch Size ft	**Roughness Factor**		**Length Adjustment Factor $\left(\frac{n'}{n}\right)^2$**
	Curves Based on n	**Actual n'**	
6.1 x 4.6	0.0327	0.0327	1.0
8.1 x 5.8	0.0321	0.032	1.0
11.4 x 7.2	0.0315	0.030	0.907
16.6 x 10.1	0.0306	0.028	0.837

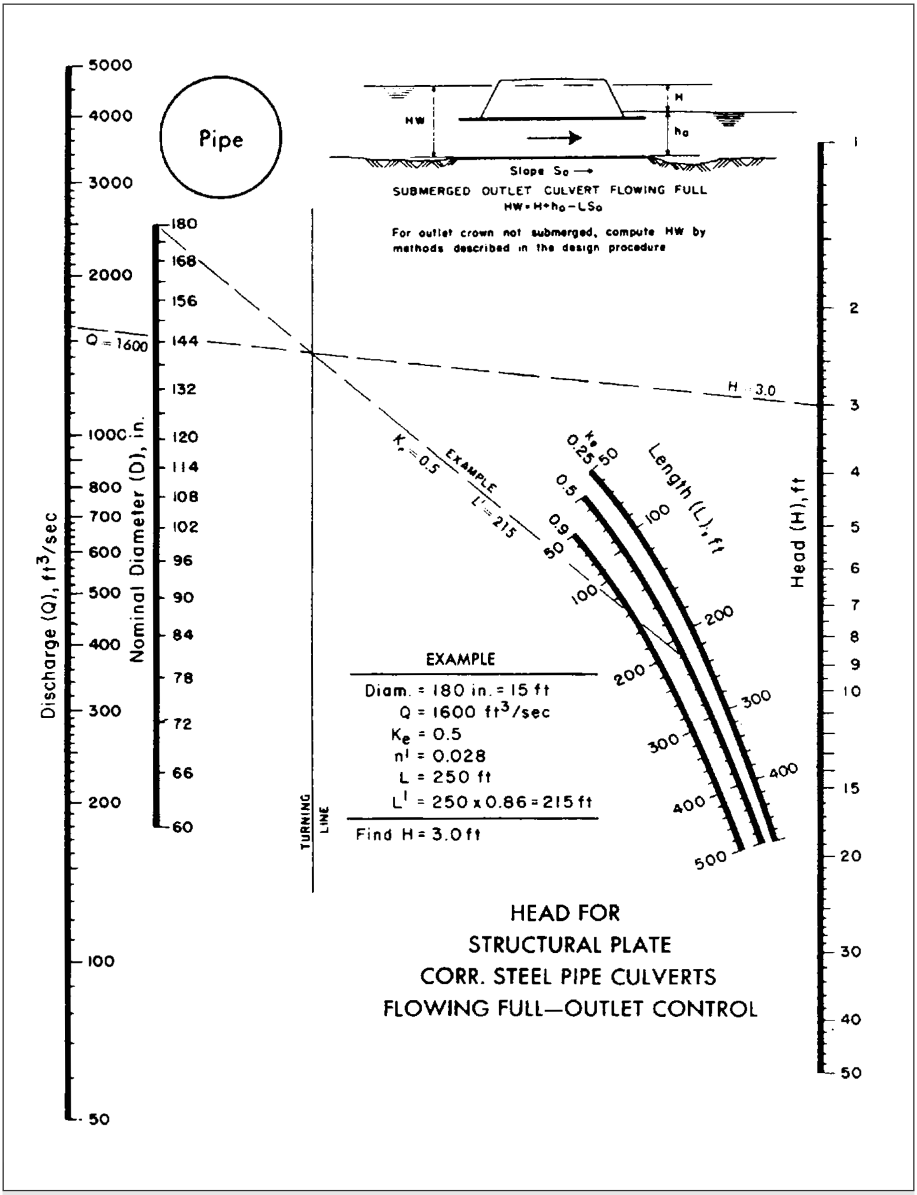
An appropriate entrance loss curve is used based on the desired entrance condition. Typical values of the entrance loss coefficient, k_e, for a variety of inlet configurations, are listed in Table 4.5.

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 ribbed pipe may be satisfactory.

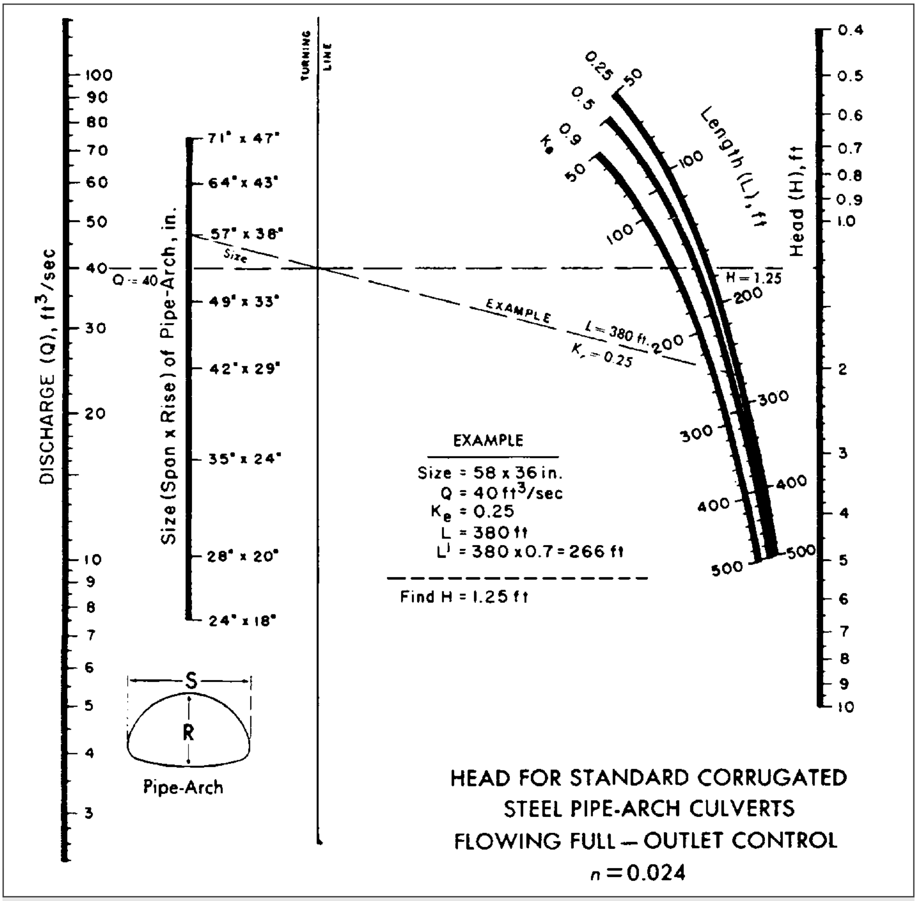
Entrance conditions should also be considered. It may be economical to use a more efficient entrance than originally considered if a pipe size difference results. This can be easily investigated by checking the pipe capacity using other entrance loss coefficient curves.



■ **Figure 4.18** Head for round corrugated steel pipes flowing full under outlet control.

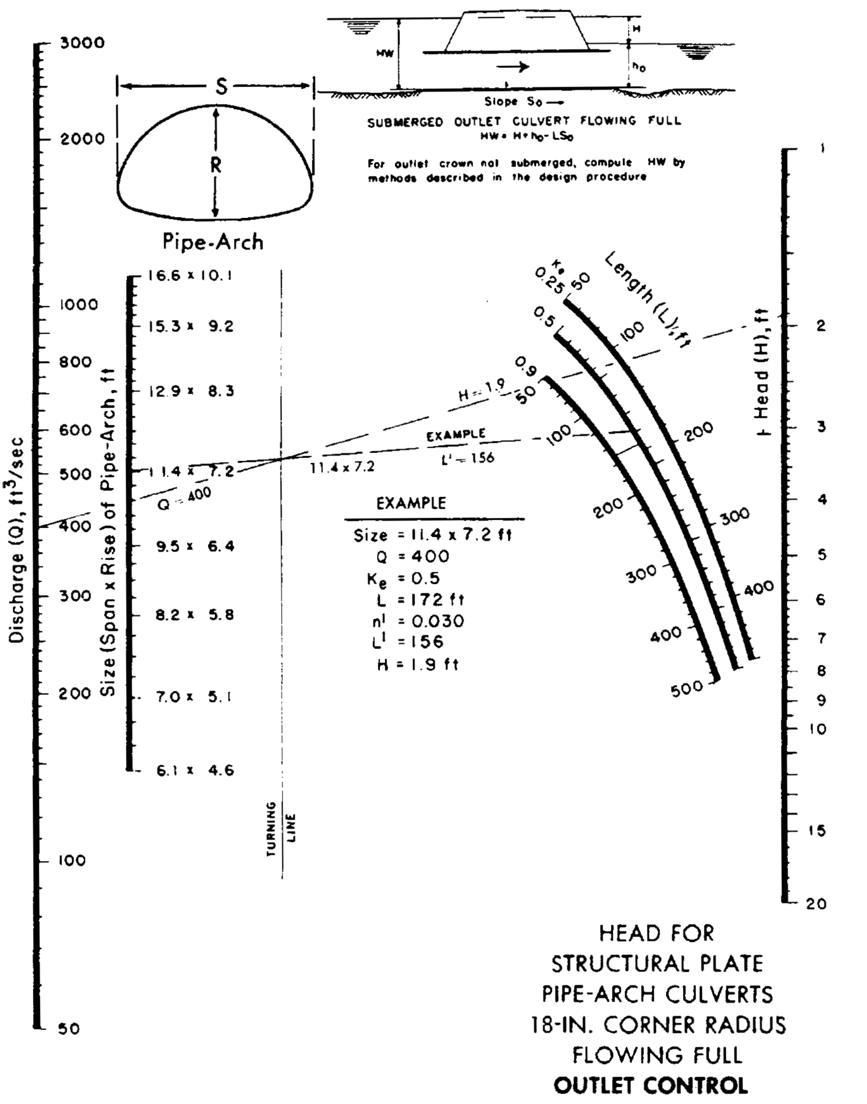


■ **Figure 4.19** Head for round structural plate corrugated steel pipes flowing full under outlet control.

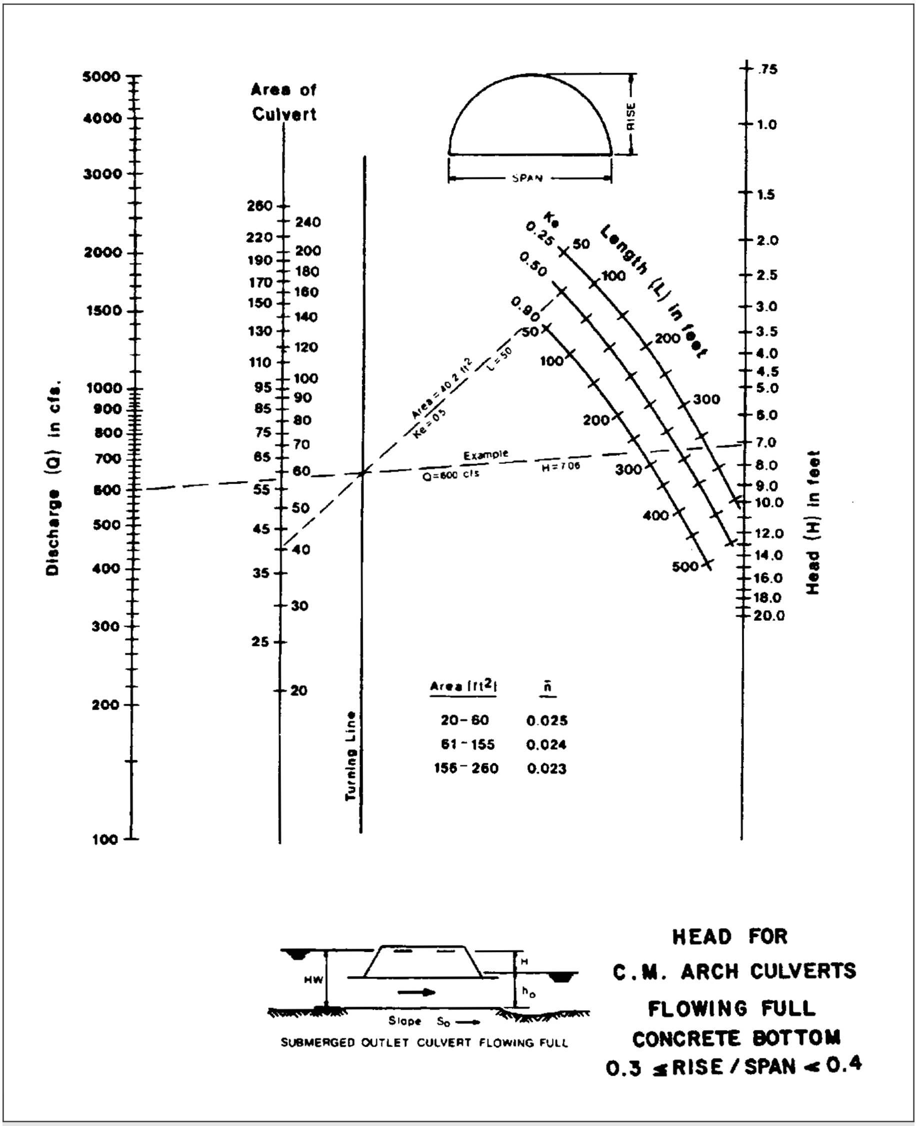


■ **Figure 4.20** Head for corrugated steel pipe arches flowing full under outlet control.

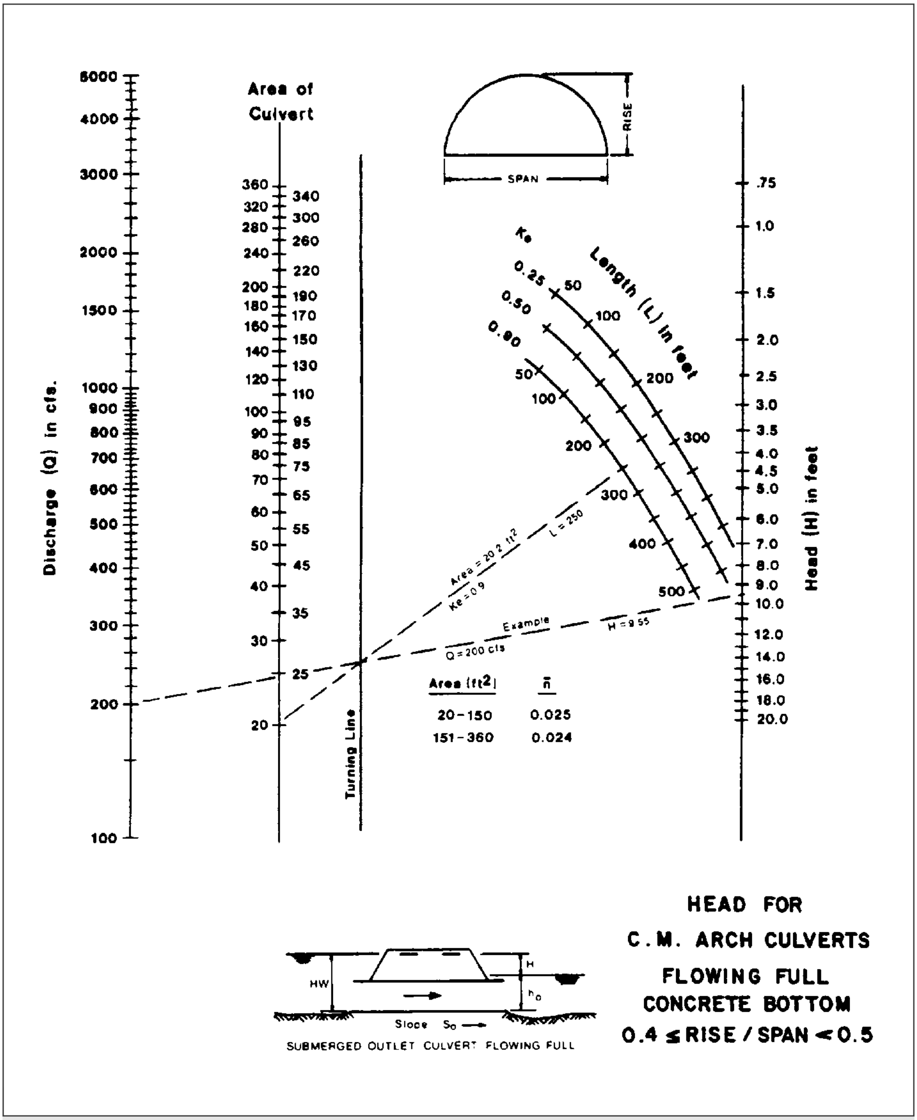
■ Structural plate pipe arch for an irrigation ditch crossing.



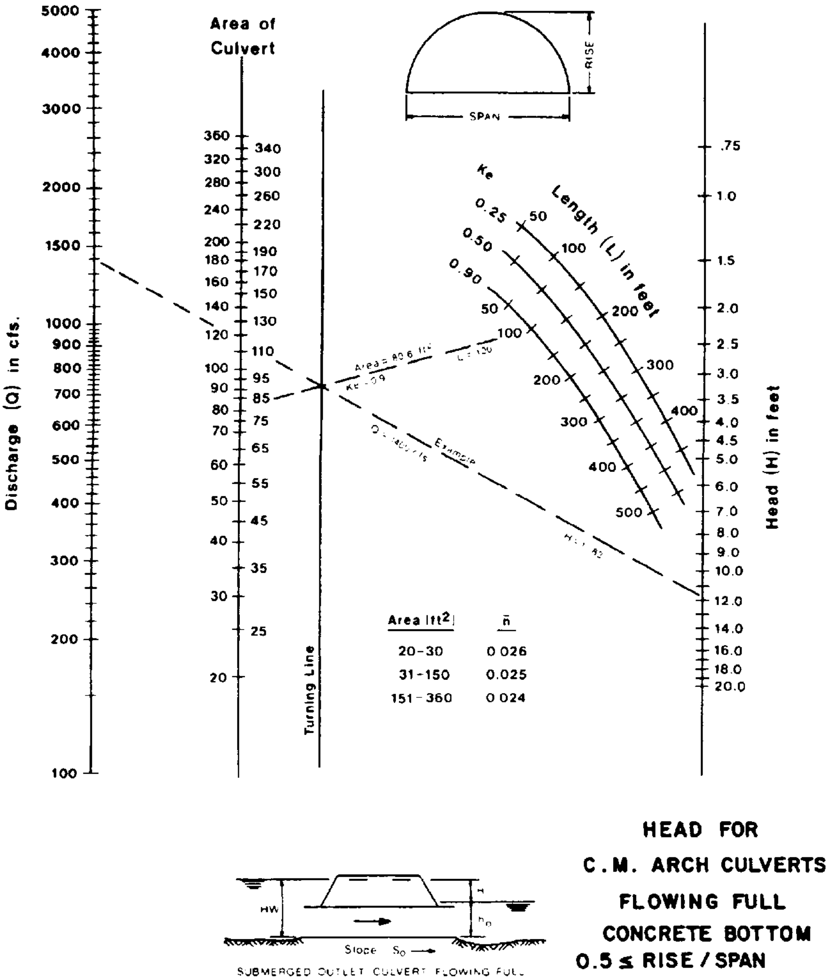
■ **Figure 4.21** Head for structural plate corrugated steel pipe arches with 18-in. corner radius, with submerged outlet and flowing full under outlet control. For 31-in. corner radius structures, use structure sizes on the size scale with equivalent end areas.



■ **Figure 4.22** Head for structural plate corrugated steel arches, with concrete bottom and 0.3 <= rise/span < 0.4, flowing full under outlet control.



■ **Figure 4.23** Head for structural plate corrugated steel arches, with concrete bottom and 0.4 <= rise/span < 0.5, flowing full under outlet control.



■ **Figure 4.24** Head for structural plate corrugated steel arches, with concrete bottom and 0.5 <= rise/span, flowing full under outlet control.

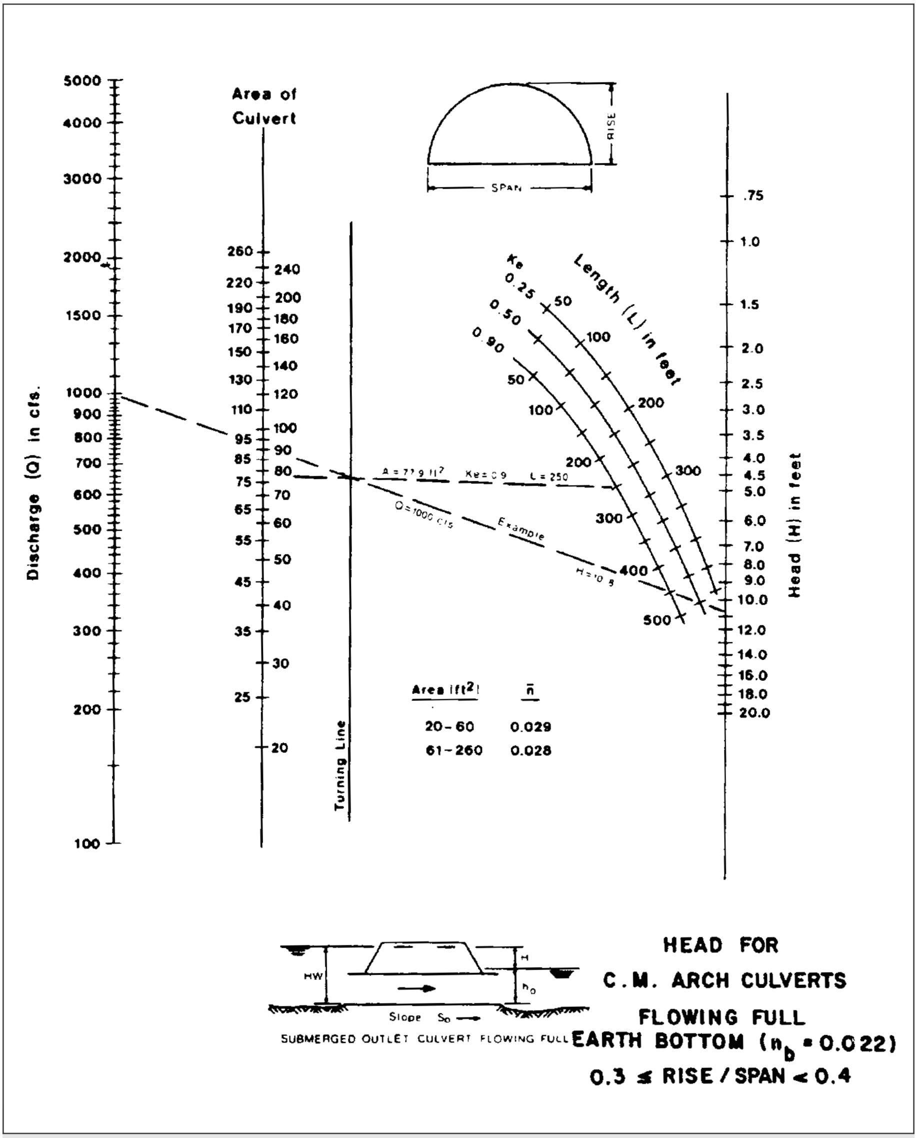
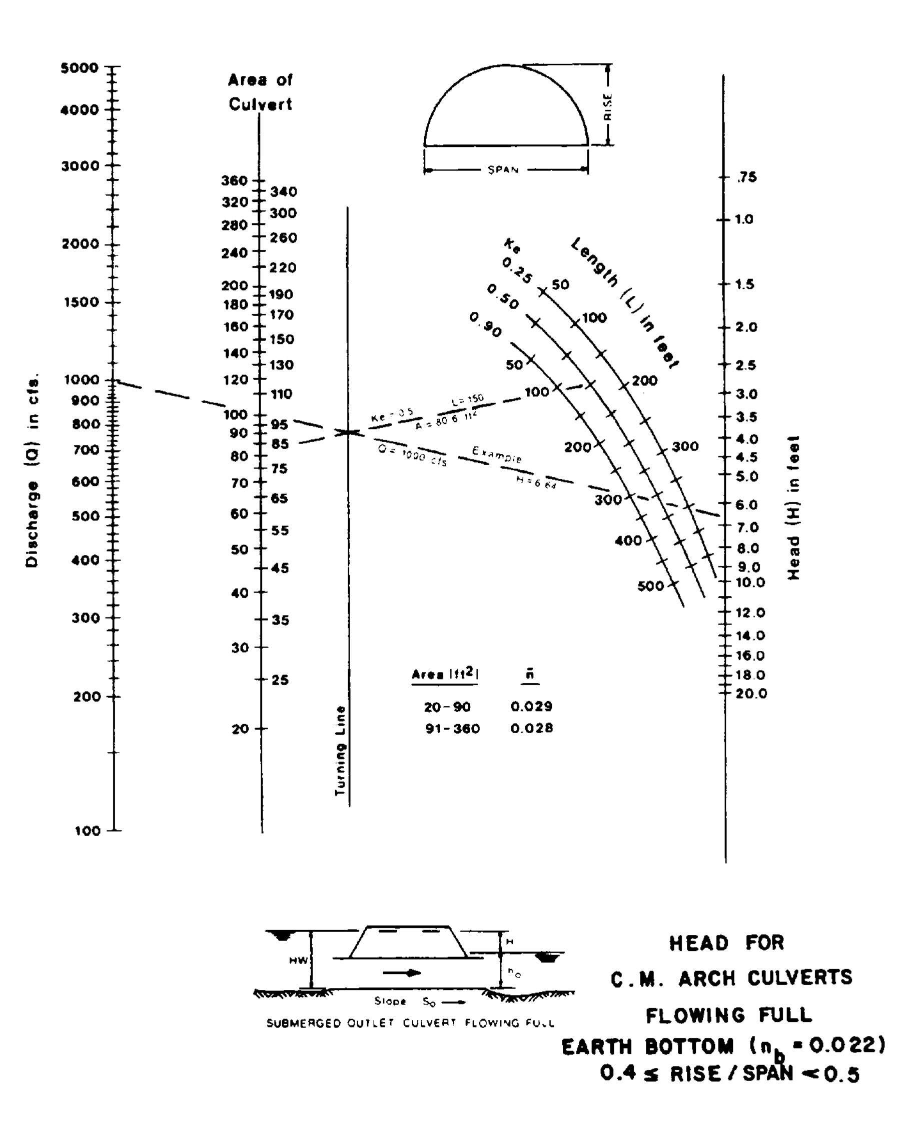


Figure 4.25 Head for structural plate corrugated steel arches, with earth bottom and 0.3 <= rise/span < 0.4, flowing full under outlet control.



■ **Figure 4.26** Head for structural plate corrugated steel arches, with earth bottom and 0.4 <= rise/span < 0.5, flowing full under outlet control.

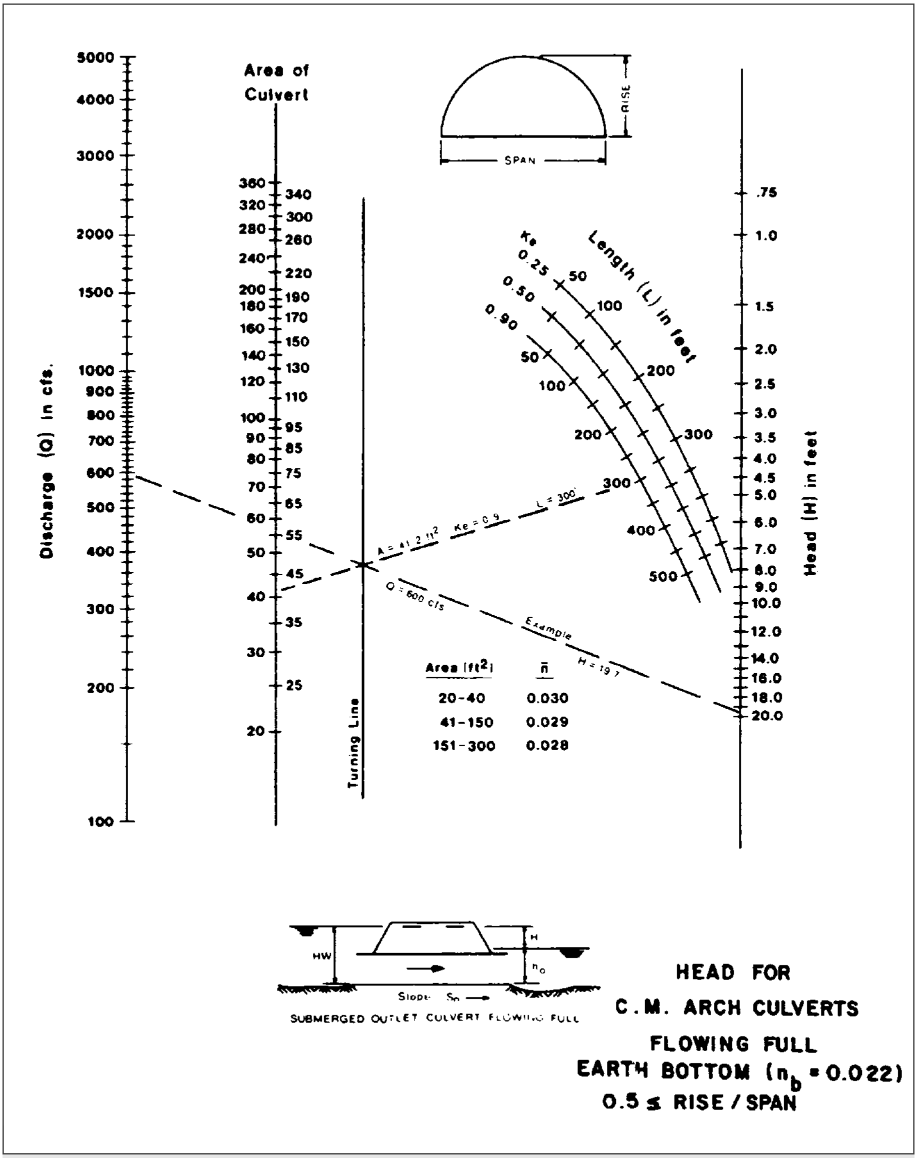


Figure 4.27 Head for structural plate corrugated steel arches, with earth bottom and 0.5 <= rise/span, flowing full under outlet control.

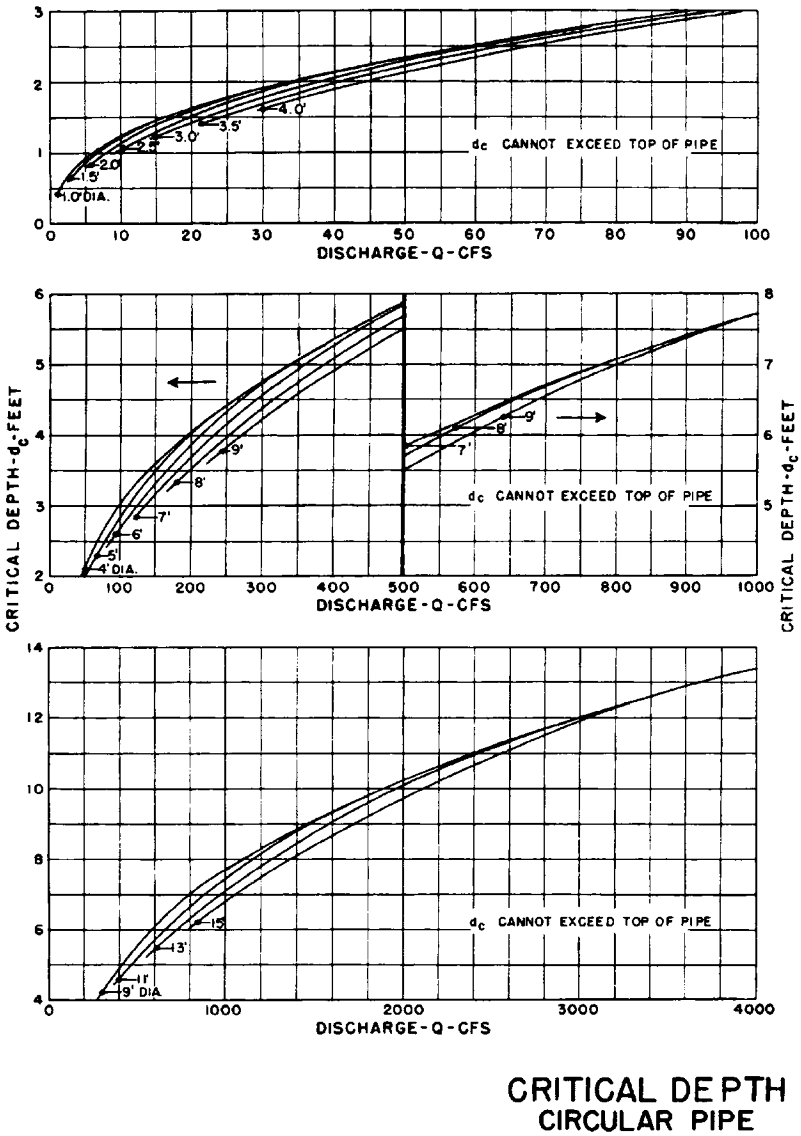
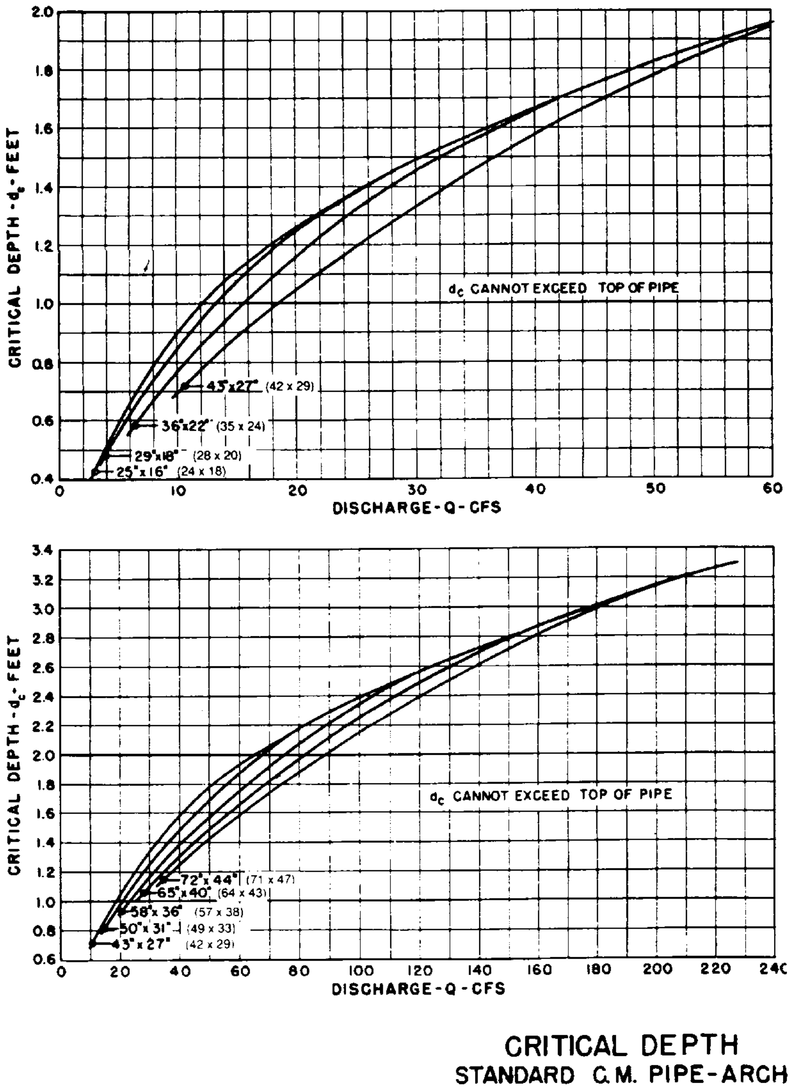
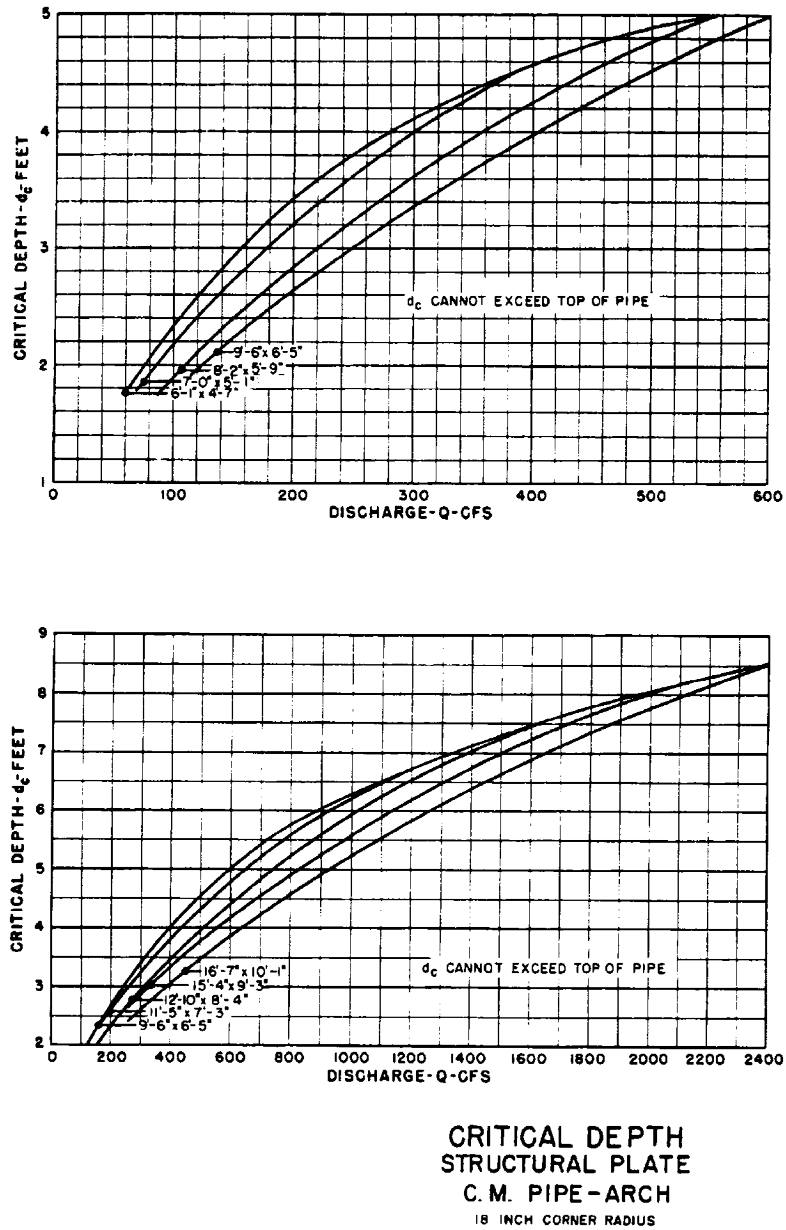


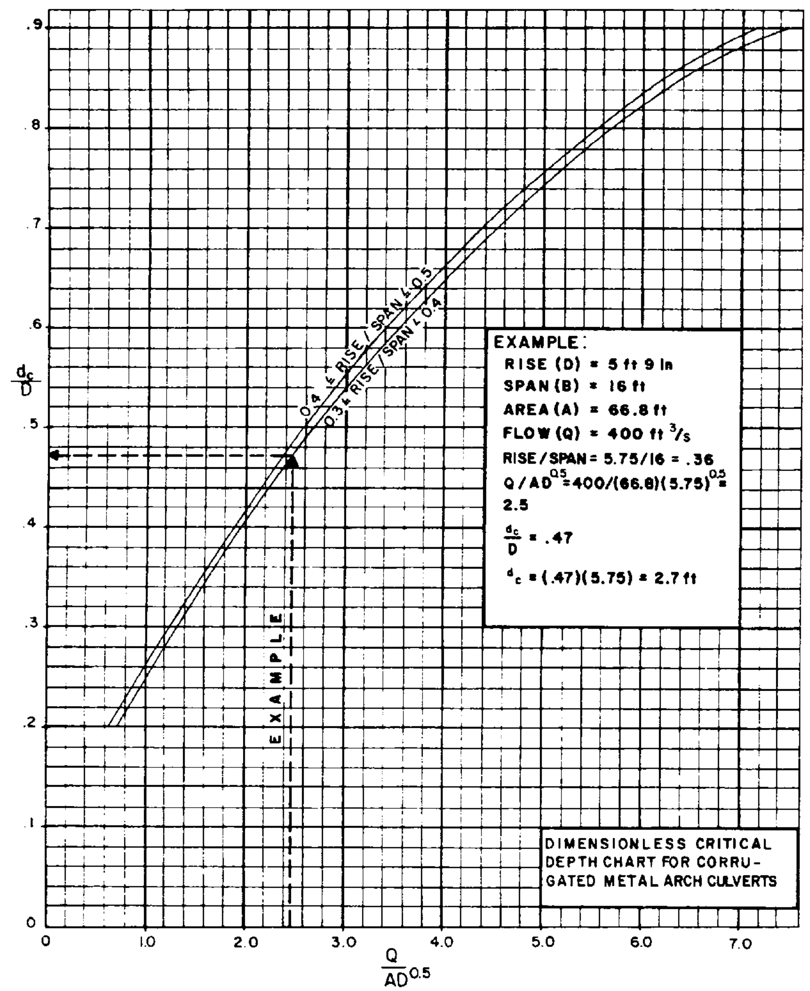
Figure 4.28 Critical depth for round corrugated steel and structural plate corrugated steel pipes.



■ **Figure 4.29** Critical depth for corrugated steel pipe arches.



■ **Figure 4.30** Critical depth for structural plate corrugated steel pipe arches.



■ **Figure 4.31** Critical depth for structural plate corrugated steel arches.

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 design manual (Figures 4.10 through 4.27) 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 design manual have a precision which is better that ±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.

Partly Full Flow

The pipe capacities derived from the above work are for pipes flowing full. Tables 4.10 through 4.14 provide full flow end areas and hydraulic radii for a variety of pipe shapes and sizes. Figures 4.32 through 4.34 provide the means to determine hydraulic section parameters for pipes flowing partly full.

The pipe arch shape is used when a low cover situation requires a pipe with less rise or when a larger flow area is desired for a given flow depth. Figure 4.35 shows a comparison, for an equivalent periphery round and pipe arch, of flow areas for a number of flow depths.

Table 4.10

Area and hydraulic radius for round pipe flowing full

Diameter in.	Area ft^2	Hydraulic Radius, ft	Diameter in.	Area ft^2	Hydraulic Radius, ft
12	0.8	0.250	156	132.7	3.250
15	1.2	0.312	162	143.1	3.375
18	1.8	0.375	168	153.9	3.500
21	2.4	0.437	174	165.1	3.625
24	3.1	0.500	180	176.7	3.750
30	4.9	0.625	186	188.7	3.875
36	7.1	0.750	192	201.1	4.000
42	9.6	0.875	198	213.8	4.125
48	12.6	1.000	204	227.0	4.250
54	15.9	1.125	210	240.5	4.375
60	19.6	1.250	216	254.5	4.500
66	23.8	1.375	222	268.8	4.625
72	28.1	1.500	228	283.5	4.750
78	33.2	1.625	234	298.6	4.875
84	38.5	1.750	240	314.2	5.000
90	44.2	1.875	246	330.1	5.125
96	50.3	2.000	252	346.4	5.250
102	56.8	2.125	258	363.1	5.375
108	63.6	2.250	264	380.1	5.500
114	70.9	2.375	270	397.6	5.625
120	78.5	2.500	276	415.5	5.750
126	86.6	2.625	282	433.7	5.875
132	95.0	2.750	288	452.4	6.000
138	103.9	2.875	294	471.4	6.125
144	113.1	3.000	300	490.9	6.250
150	122.7	3.125			

Table 4.11

Area and hydraulic radius for corrugated steel pipe arches flowing full

Corrugations 2 2/3 x 1/2 in.				Corrugations 3 x 1 in. and 5 x 1 in.			
Diameter in.	Pipe Arch Equivalent Size in.	Waterway Area ft^2	Hydraulic Radius $A/\pi D$ ft	Diameter in.	Pipe Arch Equivalent Size in.	Waterway Area ft^2	Hydraulic Radius $A/\pi D$ ft
15	17 x 13	1.1	0.280	54	60 x 46	15.6	1.104
18	21 x 15	1.6	0.340	60	66 x 51	19.3	1.230
21	24 x 18	2.2	0.400	66	73 x 55	23.2	1.343
24	28 x 20	2.9	0.462	72	81 x 59	27.4	1.454
30	35 x 24	4.5	0.573	78	87 x 63	32.1	1.573
36	42 x 29	6.5	0.690	84	95 x 67	37.0	1.683
42	49 x 33	8.9	0.810	90	103 x 71	42.4	1.800
48	57 x 38	11.6	0.924	96	112 x 75	48.0	1.911
54	64 x 43	14.7	1.040	102	117 x 79	54.2	2.031
60	71 x 47	18.1	1.153	108	128 x 83	60.5	2.141
66	77 x 52	21.9	1.268	114	137 x 87	67.4	2.259
72	83 x 57	26.0	1.380	120	142 x 91	74.5	2.373

Table 4.12

Area and hydraulic radius for structural plate pipe arches
(6x2 corrugation, 3N corner plates with 18 in. radius) flowing full

Dimensions, ft - in.		Waterway Area	Hydraulic Radius
Span	Rise	ft^2	ft
6-1	4-7	22	1.29
6-4	4-9	24	1.35
6-9	4-11	26	1.39
7-0	5-1	28	1.45
7-3	5-3	30	1.51
7-8	5-5	33	1.55
7-11	5-7	35	1.61
8-2	5-9	38	1.67
8-7	5-11	40	1.71
8-10	6-1	43	1.77
9-4	6-3	45	1.81
9-6	6-5	48	1.87
9-9	6-7	51	1.93
10-3	6-9	54	1.97
10-8	6-11	57	2.01
10-11	7-1	60	2.07
11-5	7-3	63	2.11
11-7	7-5	66	2.17
11-10	7-7	70	2.23
12-4	7-9	73	2.26
12-6	7-11	77	2.32
12-8	8-1	81	2.38
12-10	8-4	85	2.44
13-5	8-5	88	2.48
13-11	8-7	91	2.52
14-1	8-9	95	2.57
14-3	8-11	100	2.63
14-10	9-1	103	2.67
15-4	9-3	107	2.71
15-6	9-5	111	2.77
15-8	9-7	116	2.83
15-10	9-10	121	2.89
16-5	9-11	125	2.92
16-7	10-1	130	2.98

Table 4.13

Area and hydraulic radius for structural plate pipe arches
(6x2 corrugation, 5N corner plates with 31 in. radius) flowing full

Span ft - in.	Rise ft - in.	Area ft^2	Hydraulic Radius ft
13-3	9-4	97	2.68
13-6	9-6	102	2.74
14-0	9-8	105	2.78
14-2	9-10	109	2.83
14-5	10-0	114	2.90
14-11	10-2	118	2.94
1 5-4	10-4	123	2.98
15-7	10-6	127	3.04
15-10	10-8	132	3.10
16-3	10-10	137	3.14
16-6	1 1-0	142	3.20
1 7-0	1 1-2	146	3.24
17-2	1 1-4	151	3.30
17-5	1 1-6	157	3.36
17-11	11-8	161	3.40
18-1	11-10	167	3.45
18-7	12-0	172	3.50
18-9	12-2	177	3.56
19-3	12-4	182	3.59
19-6	12-6	188	3.65
19-8	12-8	194	3.71
19-11	12-10	200	3.77
20-5	13-0	205	3.81
20-7	13-2	211	3.87

Table 4.14

Area and hydraulic radius for structural plate arches flowing full

Dimensions[1]		Waterway Area ft^2	Wetted Perimeter ft	Hydraulic Radius ft
Span, ft	Rise, ft			
6.0	1-9-1/2	8	13.2	0.606
	2-3-1/2	10	14.0	0.714
	3-2	15	15.6	0.962
7.0	2-4	12	15.8	0.759
	2-10	15	16.6	0.904
	3-8	20	18.2	1.099
8.0	2-11	17	18.4	0.924
	3-4	20	19.2	1.042
	4-2	26	20.8	1.250
9.0	2-11	19	20.2	0.941
	3-10-1/2	27	21.8	1.239
	4-8-1/2	34	23.4	1.453

Table 4.14 continued

Area and hydraulic radius for structural plate arches flowing full

Dimensions[1]		Waterway Area ft²	Wetted Perimeter ft	Hydraulic Radius ft
Span, ft	Rise, ft			
10.0	3-5-1/2	26	22.8	1.140
	4-5	34	24.4	1.393
	5-3	41	26.0	1.577
11.0	3-6	28	24.6	1.138
	4-5-1/2	37	26.2	1.412
	5-9	50	28.6	1.748
12.0	4-0-1/2	35	27.2	1.287
	5-0	45	28.8	1.563
	6-3	59	31.2	1.891
13.0	4-1	38	29.0	1.310
	5-1	49	30.6	1.601
	6-9	70	33.8	2.071
14.0	4-7-1/2	47	31.6	1.487
	5-7	58	33.2	1.747
	7-3	80	36.4	2.198
15.0	4-7-1/2	50	33.4	1.497
	5-8	62	35.0	1.771
	6-7	75	36.6	2.049
	7-9	92	39.0	2.359
16.0	5-2	60	36.0	1.667
	7-1	86	39.2	2.194
	8-3	1 05	41.6	2.524
17.0	5-2-1/2	63	37.8	1.667
	7-2	92	41.0	2.244
	8-10	119	44.2	2.692
18.0	5-9	74	40.4	1.832
	7-8	104	43.6	2.385
	8-11	125	46.8	2.671
19.0	6-4	87	43.0	2.023
	8-2	118	46.2	2.554
	9-5-1/2	140	49.4	2.834
20.0	6-4	91	45.6	1.996
	8-3-1/2	124	48.0	2.521
	10-0	157	51.2	3.066
21.0	6-11	104	47.4	2.194
	8-10	140	50.6	2.767
	10-6	172	53.8	3.197
22.0	7-11	128	48.7	2.628
	8-11	1 46	52.4	2.786
	11-0	190	56.4	3.369
23.0	8-0	134	52.6	2.548
	9-10	170	55.8	3.047
	11 -6	207	59.0	3.508
24.0	8-6	149	55.2	2.699
	10-4	188	58.4	3.219
	12-0	226	61.6	3.669
25.0	8-6-1/2	155	57.0	2.719
	10-10-1/2	207	61.0	3.393
	12-6	245	64.2	3.816

1 Dimensions are to inside crests and are subject to manufacturing tolerances.

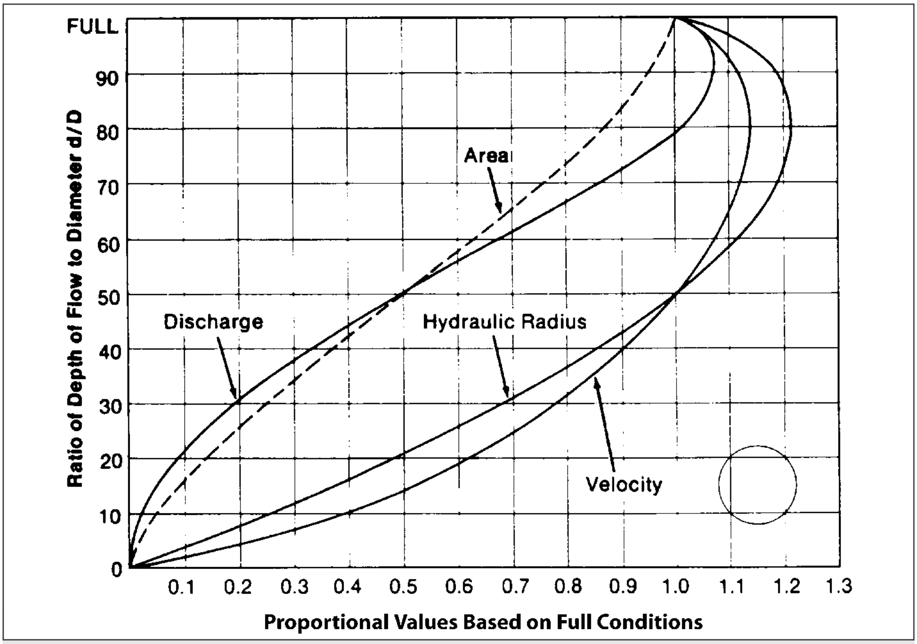


Figure 4.32 Hydraulic section parameters for circular corrugated steel and structural plate pipes.

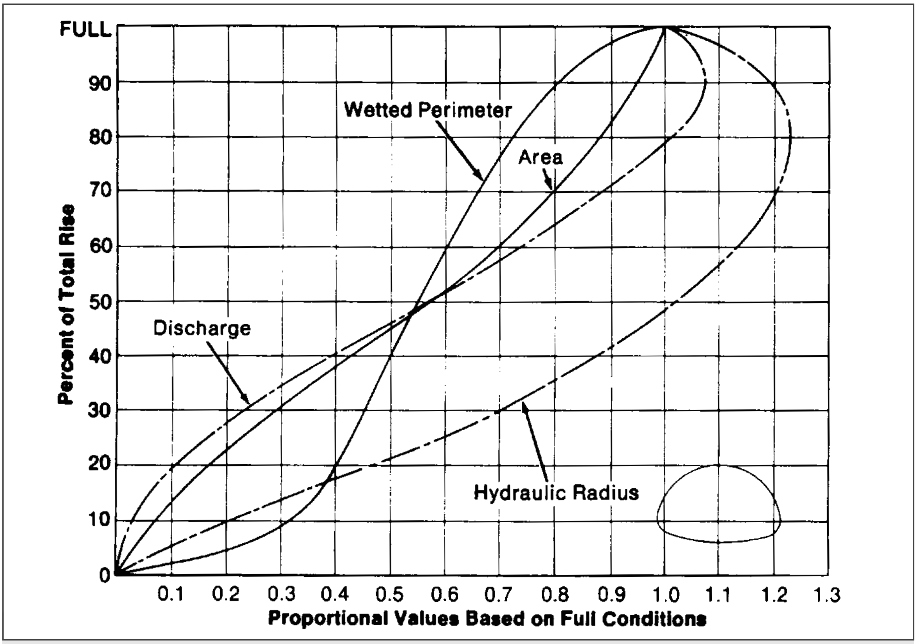
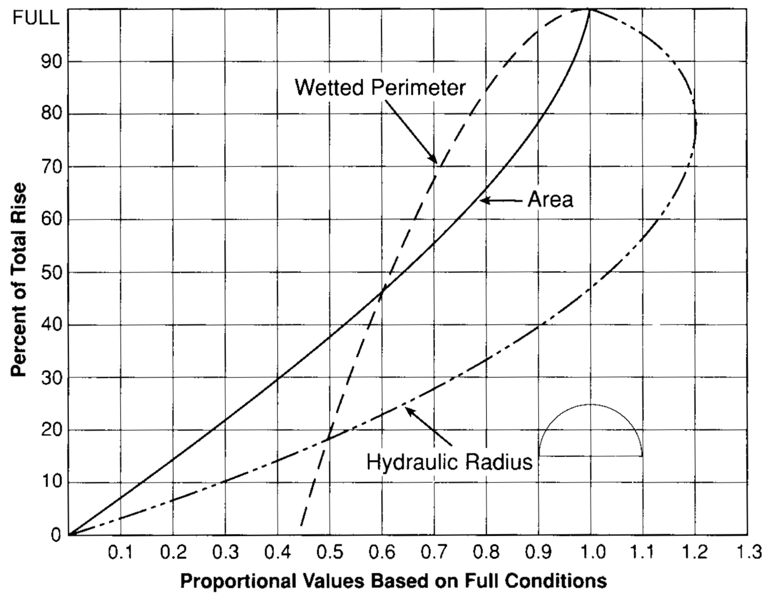
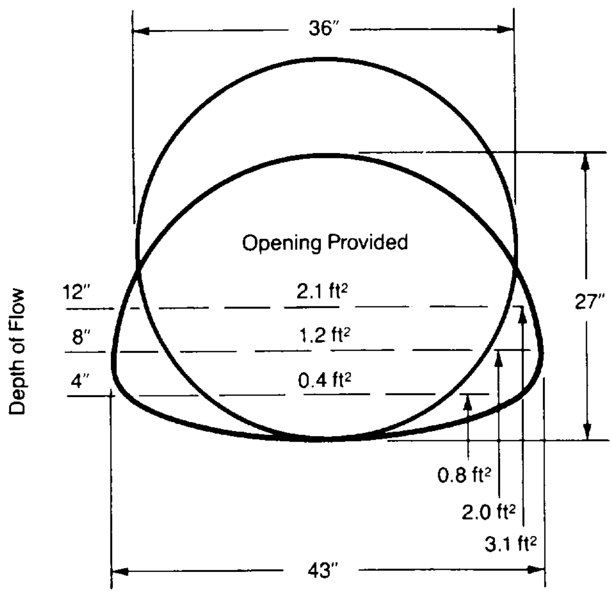


Figure 4.33 Hydraulic section parameters for corrugated steel and structural plate pipe arches.



■ **Figure 4.34** Hydraulic section parameters for structural plate arches.



■ **Figure 4.35** Comparison of waterway cross-sectional areas, at a constant depth of flow, in pipe and pipe arch shapes.

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.

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 equations, 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 that 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 from the FHWA procedures, where HW refers to the energy grade line, which corresponds to HW + Φ in this discussion.

Design Chart

Inlet control is expected to govern in most long spans. Figure 4.36 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 4.15). 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 4.36 also includes a table of velocity heads for a variety of approach velocities.

Table 4.15

Entrance loss coefficients for long spans

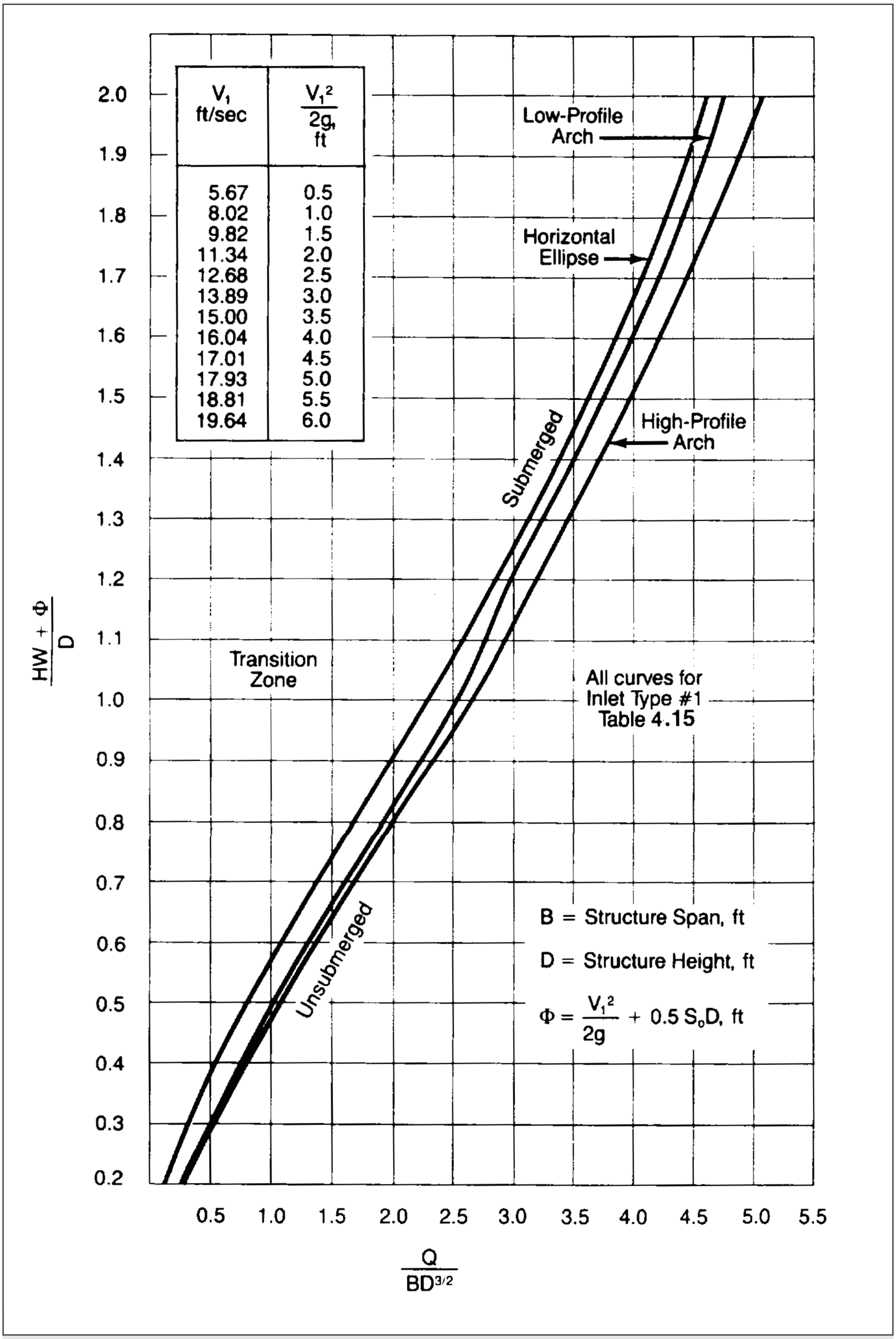
Type Inlet	Entrance Coefficients					$\frac{Q}{AD^{1/2}}$	
	k_d	k_p	k	j	k_e	Unsubmerged Maximum	Submerged Mnimum
1	0.0379	0.69	0.0083	2.0	0.5	3.3	3.8
2	0.0300	0.74	0.0018	2.5	0.2	3.3	4.2

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. Step-beveled inlets were not included in FHWA criteria.

3) Special improved inlet configurations can reduce headwater depths.

4) Coefficient k and k_d are not dimensionless.



■ **Figure 4.36** Headwater depth for long span corrugated steel structures under inlet control.

Design Calculations

Inlet Control

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

For unsubmerged inlets:

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

For submerged inlets:

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

where:
- HW = headwater depth from the invert to the water surface, ft
- H_c = critical head, ft
- H_e = increment of head above the critical head, ft
- S_o = slope of the structure, ft/ft
- D = rise of the structure, ft
- V_1 = approach velocity, ft/s
- g = gravitational constant, 32.2 ft/s^2
- k_d, k_p = coefficients based on inlet type (Table 4.15)
- Q = design discharge, ft^3/s
- A = full cross-sectional end area of the structure, ft^2

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 4.15. 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 4.36, 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, ft
- V_c = critical velocity, ft/s

The critical depth can be interpolated from Tables 4.16 through 4.18. 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 4.16

Hydraulic section parameters for long span structural plate horizontal ellipses

Span x Rise (B x D) ft - in.	Full Flow Data				Discharge - (Q), ft²/sec					
					Critical Depth Factor*					
	Area ft^2	WP ft	R ft	$AR^{2/3}$	0.40	0.50	0.60	0.70	0.80	0.90
19-4 x 12-9	191	50.7	3.77	462.7	769	1204	1714	2316	3083	4183
20-1 x 13-0	202	52.3	3.86	497.1	823	1282	1832	2478	3298	4502
20-2 x11-11	183	50.7	3.61	430.6	708	1110	1584	2153	2871	3935
20-10 x 12-2	194	52.3	3.71	464.9	756	1154	1694	2298	3088	4194
21-0 x 15-2	248	57.1	4.35	660.9	1073	1684	2390	3225	4336	5901
21-11 x 13-1	221	55.5	3.98	555.0	897	1403	2005	2725	3640	4966
22-6 x 15-8	274	60.3	4.55	752.4	1228	1921	2732	3687	4886	5979
23-0 x 14-1	249	58.7	4.25	653.3	1051	1645	2347	3185	4256	5801
23-3 x 15-11	288	61.9	4.65	802.3	1298	2033	2889	3903	5179	7046
24-4 x 16-11	320	65.1	4.92	925.7	1486	2327	3307	4464	5914	8055
24-6 x 14-8	274	61.9	4.43	739.1	1177	1843	2634	3577	4785	6518
25-2 x 14-11	287	63.5	4.53	785.7	1242	1947	2782	3780	5060	6881
25-5 x 16-9	330	66.7	4.95	958.5	1523	2383	3391	4588	6101	8295
26-1 x 18-2	369	69.9	5.28	1118.9	1775	2778	3949	5331	7046	9611
26-3 x 15-10	320	66.9	4.80	910.6	1430	2240	3196	4340	5804	7902
27-0 x 16-2	334	68.3	4.89	962.2	1503	2356	3366	4572	6113	8303
27-2 x 19-1	405	73.1	5.54	1268.0	1999	3131	4448	6004	7953	10817
27-11 x 19.5	421	74.7	5.64	1334.0	2095	3278	4660	6290	8329	11325
28-1 x 17-1	369	71.5	5.16	1101.9	1714	2683	3830	5192	6943	9438
28-10 x 17-5	384	73.1	5.26	1161.4	1795	2812	4016	5452	7288	9919
29-5 x 19-11	455	77.9	5.84	1475.5	2289	3587	5098	6887	9143	12434
30-1 x 20-2	472	79.5	5.94	1548.1	2391	3744	5326	7198	9563	13008
30.3 x 17-11	415	76.3	5.44	1283.6	1968	3084	4406	5985	8014	10900
31-2 x 21-2	513	82.7	6.20	1731.3	2659	4166	5925	8003	10622	14429
31-4 x 18-11	454	79.5	5.71	1450.4	2212	3467	4950	6720	8983	12205
32-1 x 19-2	471	81.1	5.81	1522.2	2309	3617	5173	7020	9389	12774
32-3 x 22-2	555	85.9	6.46	1925.1	2947	4615	6561	8870	11752	15964
33-0 x 22-5	574	87.5	6.56	2011.5	3064	4798	6825	9220	12236	16639
33-2 x 20-1	512	84.3	6.08	1705.6	2577	4038	5762	7819	10451	14210
34-1 x 23-4	619	90.7	6.82	2226.1	3376	5286	7495	10150	13464	18280
34-7 x 20-8	548	87.5	6.26	1861.4	2792	4372	6245	8481	11320	15424
34-11 x 21-4	574	89.1	6.44	1986.9	2975	4661	6652	9070	12053	16397
35-1 x 24-4	665	93.9	7.08	2452.0	3705	5801	8246	11131	14754	20048
36-0 x 22-4	619	92.3	6.71	2202.1	3286	5146	7341	9950	13283	18054
37-2 x 22-2	631	93.9	6.72	2247.0	3328	5215	7450	10100	13537	18381

* Multiply factor by structure rise.

Table 4.17

Hydraulic section parameters for long span structural plate low profile arches

Span x Rise (B x D) ft - in.	Full Flow Data				Discharge - (Q), ft^3/sec					
					Critical Depth Factor*					
	Area ft^2	WP ft	R ft	AR $^{2/3}$	0.4D	0.5D	0.6D	0.7D	0.8D	0.9D
20-1 x 7-6	120	47.9	2.51	223	579	819	1119	1443	1839	2448
19-5 x 6-9	105	45.6	2.30	183	480	681	933	1206	1532	2049
21-6 x 7-9	133	51.0	2.62	253	656	929	1266	1632	2081	2723
22-3 x 7-11	140	52.5	2.67	269	698	986	1344	1720	2207	2926
23-0 x 8-0	147	54.1	2.72	286	738	1042	1420	1829	2324	3083
23-9 x 8-2	154	55.6	2.77	304	784	1112	1508	1944	2466	3267
24-6 x 8-3	161	57.2	2.82	321	830	1180	1593	2054	2612	3422
25-2 x 8-5	168	58.7	2.86	339	876	1249	1682	2166	2756	3640
25-11 x 8-7	176	60.2	2.91	359	923	1320	1774	2290	2901	3848
27-3 x 10-0	217	64.8	3.34	485	1228	1733	2340	3017	3836	5046
28-1 x 9-6	212	65.6	3.23	463	1179	1666	2252	2901	3685	4858
28-9 x 10-3	234	67.9	3.44	533	1343	1896	2558	3297	4193	5538
28-10 x 9-8	220	67.1	3.28	486	1236	1747	2361	3040	3863	5105
30-3 x 9-11	237	70.2	3.38	534	1353	1914	2586	3326	4220	5543
30-11 x 10-8	261	72.5	3.59	612	1536	2168	2922	3761	4760	6292
31-7 x 12-1	309	76.1	4.06	786	1901	2684	3607	4676	5961	7836
31-0 x 10-1	246	71.7	3.43	560	1416	2004	2702	3476	4411	5806
32-4 x 12-3	319	77.6	4.11	819	1979	2795	3760	4869	6201	8142
31-9 x 10-2	255	73.3	3.47	585	1480	2099	2824	3631	4611	6086
33-1 x 12-5	330	79.1	4.17	855	2085	2907	3916	5065	6443	8500
33-2 x 11-1	289	77.1	3.74	696	1741	2459	3313	4264	5413	7099
34-5 x 13-3	367	83.0	4.42	988	2372	3346	4467	5815	7400	9730
34-7 x 11-4	308	80.2	3.84	755	1886	2674	3589	4614	5853	7722
37-11 x 15-7	477	92.9	5.13	1419	3356	4726	6290	8158	10390	13646
35-4 x 11-5	318	81.7	787	1962	2784	3730	4782	6089	8015	
38-8 x 15-9	490	94.4	1469	3466	4881	6496	8426	10727	14122	

* Multiply factor by structure rise.

Housing Development in Thornton, Colorado. Super Cor Box Culvert 35’-9” span x 7’-9” rise.

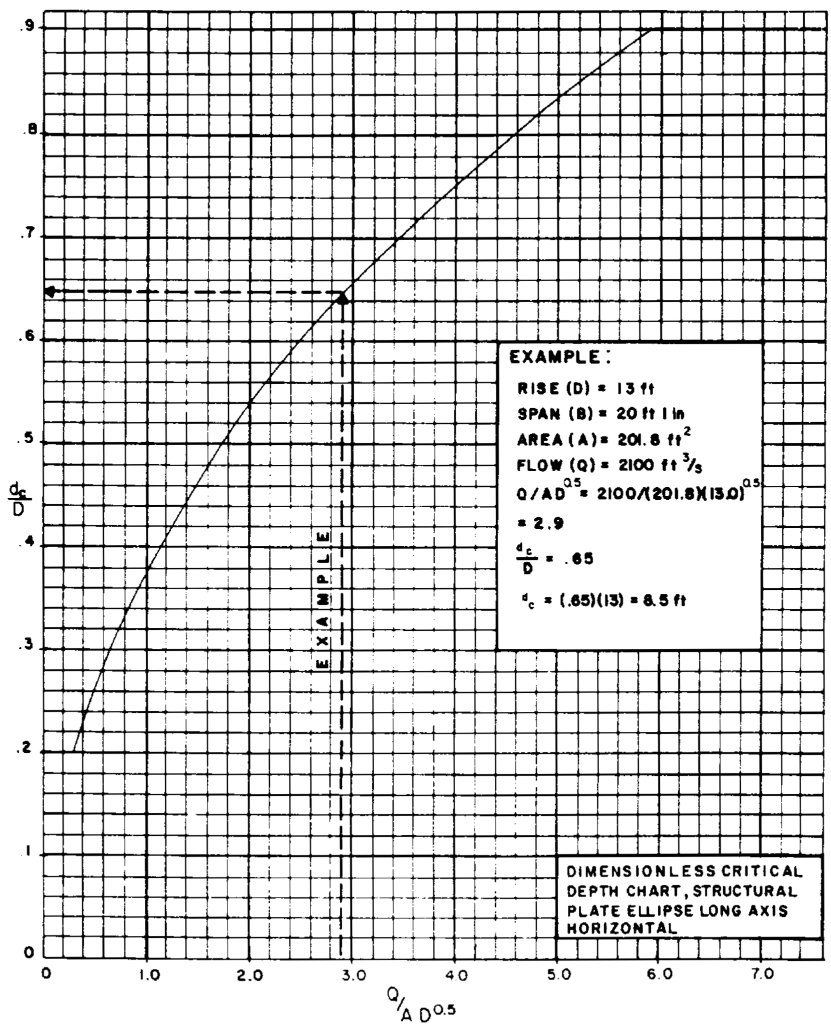
Table 4.18

Hydraulic section parameters for long span structural plate high profile arches

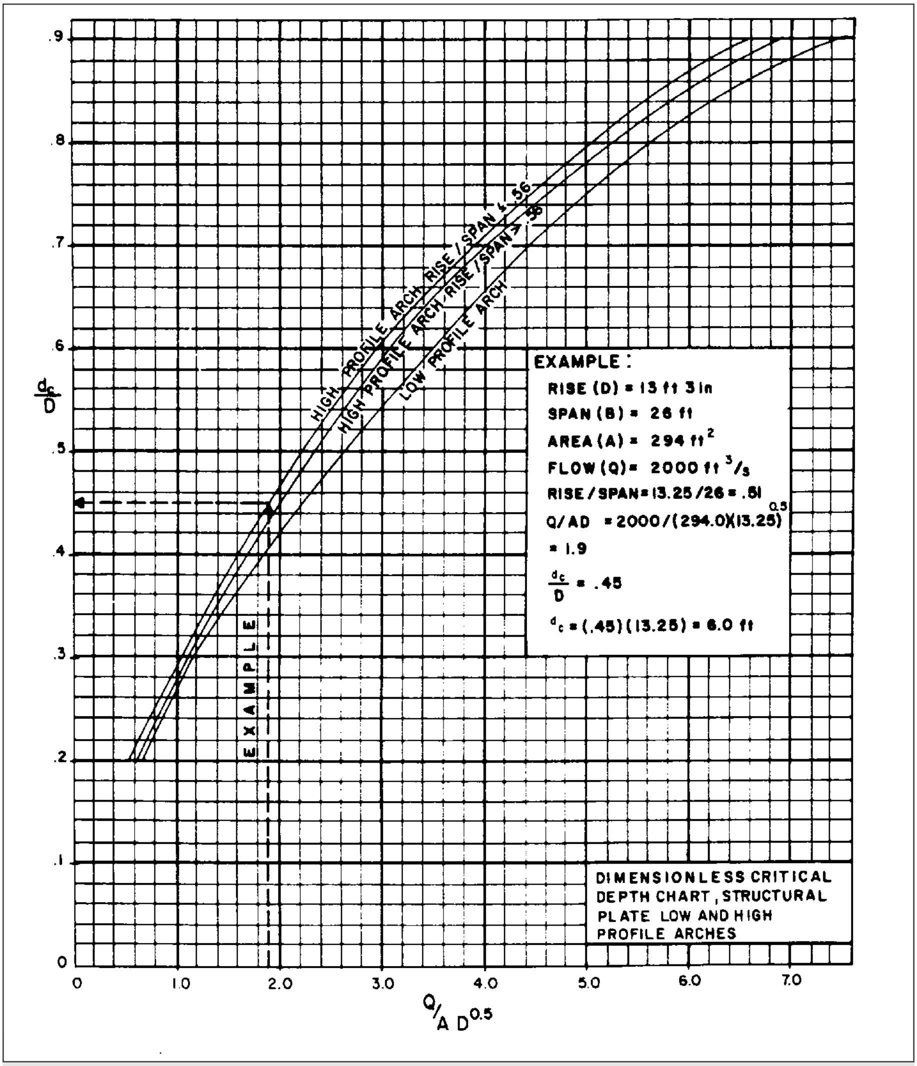
Span x Rise (B x D) ft - in.	Full Flow Data				Discharge - (Q), ft^3/sec					
					Critical Depth Factor*					
	Area ft^2	WP ft	R ft	AR $^{2/3}$	0.4D	0.5D	0.6D	0.7D	0.8D	0.9D
20-1 x 9-1	152	50.8	2.99	315.5	785	1107	1480	1923	2466	3282
20-8 x 12-1	214	56.5	3.78	518.6	1191	1687	2264	2936	3790	5044
21-6 x 11-8	215	57.5	3.73	516.1	1179	1669	2234	2911	3765	4989
22-10 x 14-6	284	63.9	4.45	768.9	1690	2402	3227	4193	5412	7209
22-3 x 11-10	224	59.1	3.80	546.5	1246	1762	2361	3077	3974	5297
22-11 x 14-0	275	63.3	4.34	731.3	1601	2279	3050	3969	5140	6853
23-0 x 11-11	234	60.7	3.86	576.7	1315	1858	2491	3246	4191	5589
24-4 x 14-10	309	67.2	4.60	854.4	1874	2658	3569	4636	5989	7980
23-9 x 12-1	244	62.3	3.93	608.8	1385	1956	2623	3418	4417	5869
24.6 x 13-9	288	66.0	4.37	770.5	1680	2376	3187	4154	5391	7200
25-9 x 15-1	334	70.5	4.74	942.5	2063	2924	3923	5096	6586	8769
25-2 x 13-1	283	66.6	4.25	742.0	1650	2331	3125	4079	5280	7030
26-6 x 15-3	347	72.1	4.81	988.1	2161	3062	4106	5312	6896	9184
25-11 x 13-3	294	68.2	4.31	778.4	1730	2445	3276	4280	5534	7348
27-3 x 15-5	360	73.7	4.88	1034.6	2260	3201	4292	5577	7803	9584
27-5 x 13-6	317	71.3	4.44	855.0	1896	2679	3591	4692	6064	8068
29-5 x 16-5	412	79.2	5.20	1235.4	2697	3820	5118	6639	8570	11390
28-2 x 14-5	348	74.0	4.70	976.0	2123	2998	4019	5255	6802	9050
30-1 x 18-0	466	82.8	5.63	1474.0	3111	4402	5920	7694	9952	13266
30-3 x 15-5	399	79.5	5.02	1169.0	2539	3589	4811	6278	8114	10775
31-7 x 18-4	496	86.1	5.77	1596.6	3366	4768	6405	8315	10760	14291
31-0 x 15-7	412	81.1	5.08	1216.8	2642	3734	5004	6534	8437	11173
31-8 x 17-9	483	85.4	5.65	1531.5	3222	4556	6114	7960	10323	13760
32-4 x 19-11	553	90.0	6.18	1863.2	3808	5404	7259	9450	12256	16350
31-9 x 17-2	469	84.8	5.53	1466.4	3080	4353	5836	7615	9890	13190
33-1 x 20-1	570	91.2	6.25	1934.9	3940	5610	7534	9807	12721	16963
32-6 x 17-4	484	86.4	5.60	1524.8	3200	4522	6061	7917	10270	13675
33-10 x 20-3	587	92.9	6.33	2009.9	4106	5820	7814	10172	13197	17607
34-0 x 17-8	513	89.6	5.73	1643.2	3445	4867	6524	8532	11054	14703
34-7 x 19-10	590	93.9	6.28	2007.6	4095	5797	7775	10136	13176	17575
34-8 x 17-9	528	91.2	5.79	1703.0	3572	5043	6762	8844	11458	15210
35-4 x 20-0	607	95.5	6.35	2080.4	4255	6022	8076	10534	13697	18270

* Multiply factor by structure rise.

Some of the long span culverts and special culvert shapes had no critical depth charts. These special shapes are available in numerous sizes, making it impractical to produce individual critical depth curves for each culvert size and shape. Hence, dimensionless critical depth curves, Figures 4.37 and 4.38, have been developed for the shapes which have adequate geometric relationships. It should be noted that these special shapes are not truly geometrically similar, and any generalized set of geometric relationships will involve some degree of error. The amount of error is unknown.



■ **Figure 4.37** Critical depth for horizontal ellipse long span structural plate corrugated steel bridges.

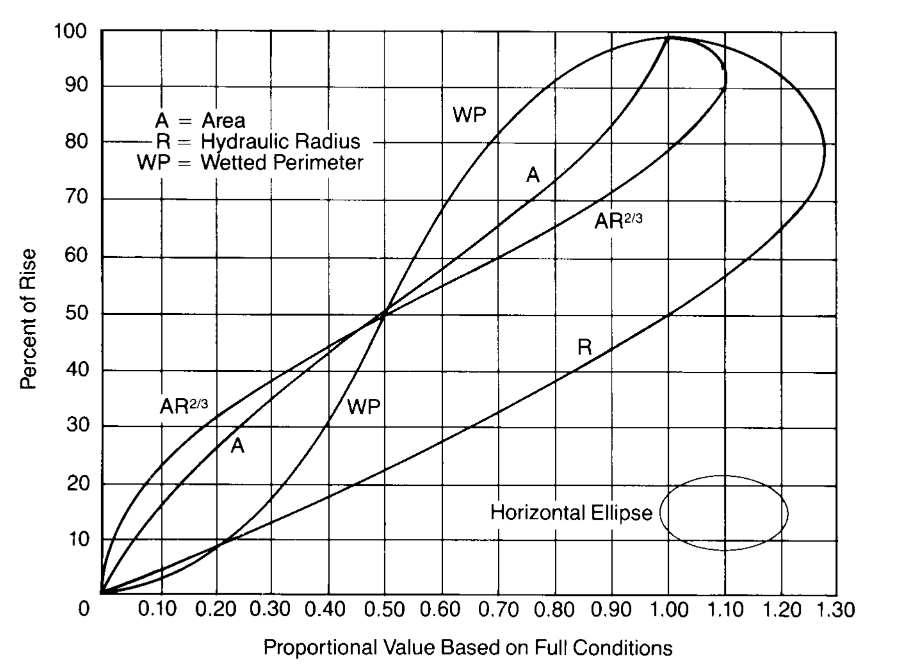


■ **Figure 4.38** Critical depth for low and high profile arch long span structural plate corrugated steel bridges.

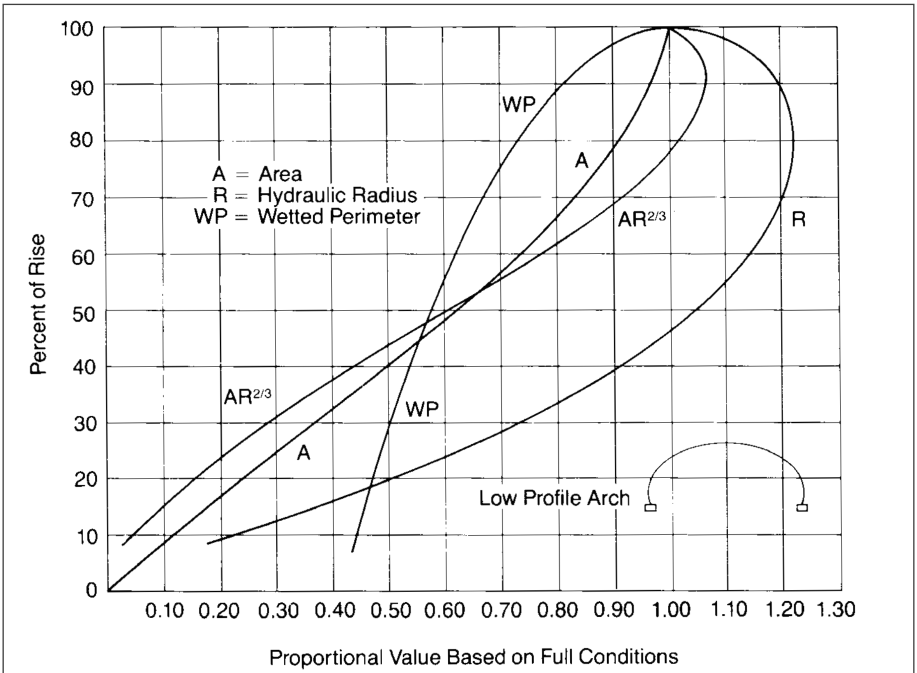
The critical velocity is calculated by dividing the design discharge by the partial flow area corresponding to the critical depth. The partial flow area can be determined from Figures 4.39 through 4.41 using the critical depth as a percentage of the structure rise. The partial flow area is the product of the proportional value from the figure and the full cross sectional area of the structure. The critical velocity is then:

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

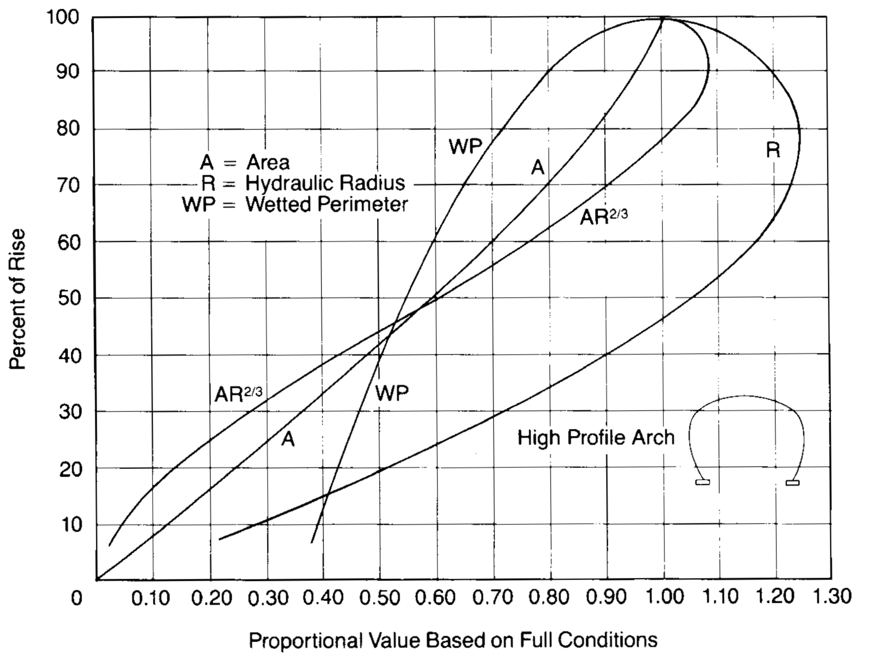
where: A_c = partial flow area based on the critical depth, ft^2



■ **Figure 4.39** Hydraulic section parameters for long span horizontal ellipses.



■ **Figure 4.40** Hydraulic section parameters for long span low profile arches.



■ **Figure 4.41** Hydraulic section parameters for long span high profile arches.

The accuracy of the critical depth may be checked using the basic equation 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, ft

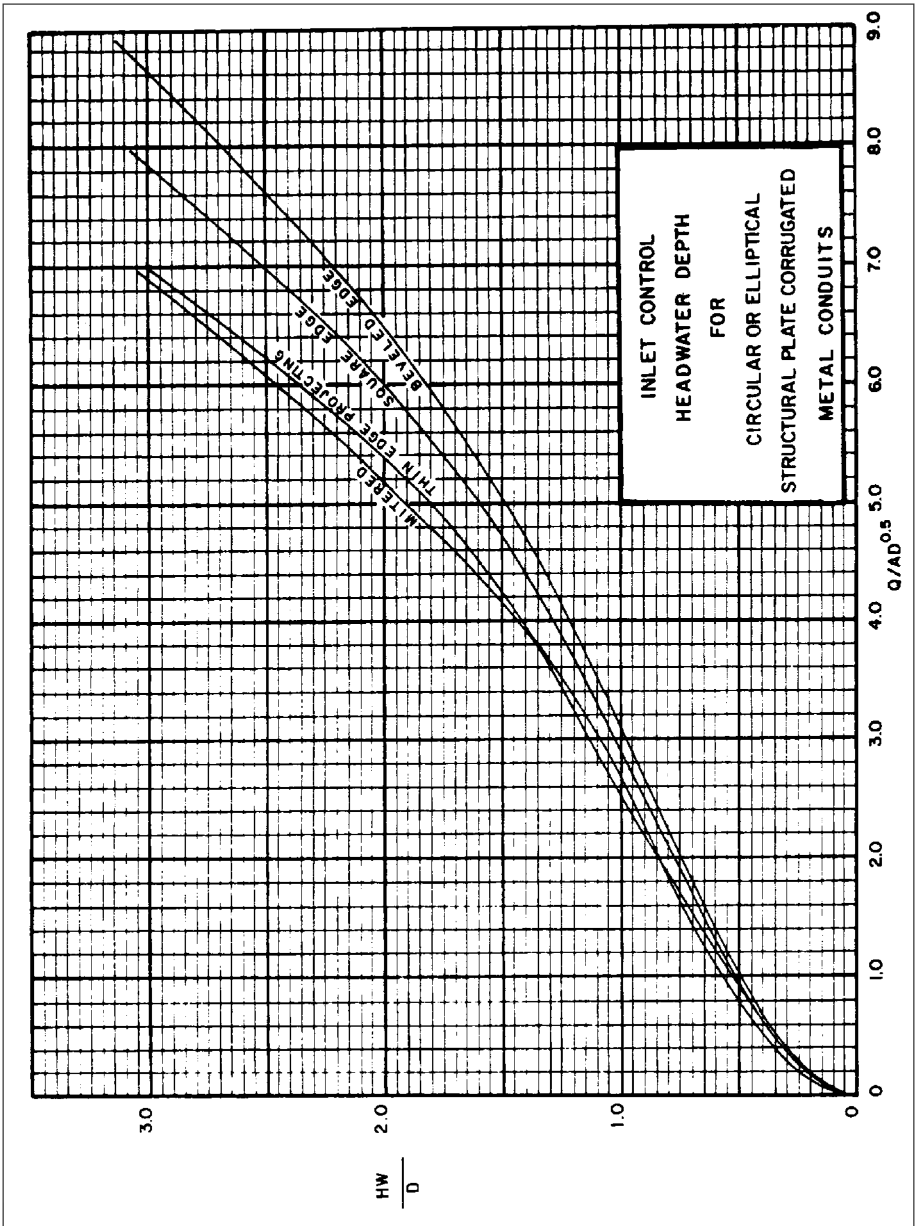
For this calculation, detailed structure cross section geometry is required in order to calculate the water surface width when the water depth is the critical depth.

The increment of head above the critical head is:

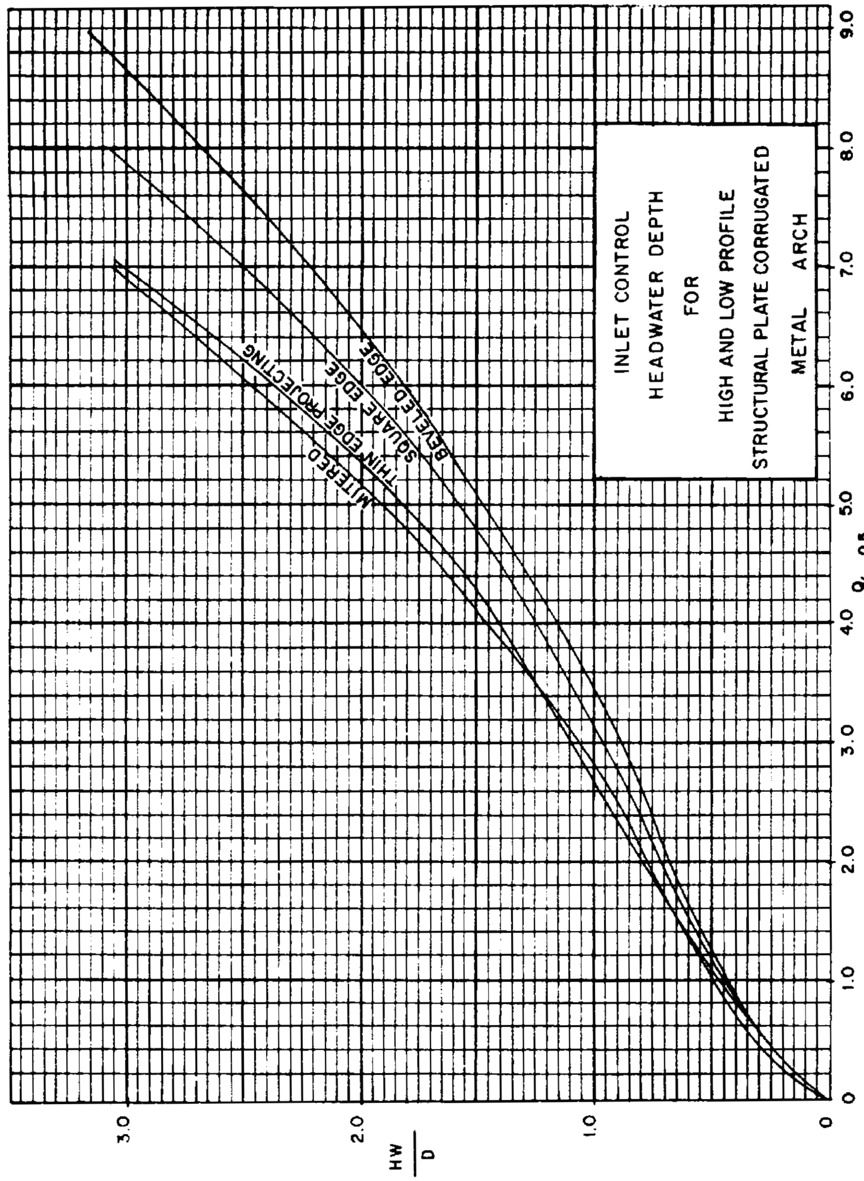
$$H_e = k\,D\left\{\frac{Q}{AD^{1/2}}\right\}^j$$

where: k, j = coefficients based on inlet type (Table 4.15)

By plotting the results of the unsubmerged and submerged calculations and connecting the resultant curves with transition lines, the dimensionless design curves shown in Figures 4.42 and 4.43 were developed. All circular and elliptical shapes can be represented by a single curve for each inlet edge configuration. A similar set of curves was developed for high and low profile arches. It is recommended that the curves shown in Figure 4.42 be used for curved shapes including circles, ellipses and pear shapes, and that the high and low profile arch curves in Figure 4.43 be used for all true arch shapes (those with a flat bottom).



■ **Figure 4.42** Headwater depth for circular and elliptical long span structural plate corrugated steel bridges.



■ **Figure 4.43** Headwater depth for high and low profile arch long span structural plate corrugated steel bridges.

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 4.15)

Under this condition, the headwater depth must be determined by a backwater analysis if accurate results are required. Datum points d1 and d2 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.

As discussed previously, long span hydraulic properties are provided in Tables 4.16 through 4.18 and Figures 4.39 through 4.41, and entrance loss coefficients are in Table 4.15. 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 design manual. 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 following:

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

where: HW = headwater depth, ft
k_e = entrance loss coefficient (Table 4.15)
g = gravitational constant = 32.2 ft/s^2
n = Manning's friction factor (Table 4.7)
L = length of long span, ft
R = hydraulic radius, ft = A / WP

A = full cross sectional area of the long span, ft^2
WP = perimeter of the long span, ft
V = velocity, ft/s
h_o = outlet datum, ft
S_o = slope of structure, ft/ft
V_1 = approach velocity, ft/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, ft
n_i = Manning's n value for material i
p = total wetted perimeter, ft

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:

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

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 4.36 and the design parameters to obtain a value for HW + Φ 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 $D + (1 + k_e)\dfrac{V_c^2}{2g}$

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.

HYDRAULICS OF STEEL BOX CULVERTS

Where large waterway openings are required with no or minimal ponding, a box culvert is often used. With a HW ratio of less than one (1.0), the steel structural plate box culvert may be designed as an open channel. This is the most efficient hydraulic design for this condition.

By examining the geometry, it can be seen that the nearly vertical legs and flat bottom will provide a linear relationship with lower depths of flow (to 0.6D, where D is the box culvert rise). As the water surface elevation increases and begins to contact the corner or haunch sections, the wetted perimeter increases at a rate faster than the rate of increase in the waterway area. At water depths of 0.8D to 1.0D, there is a rapid increase in wetted perimeter and very little increase in area. Therefore, it can be seen that maximum flow will occur at a point somewhat less than full (0.8 to 0.9D).

Manning's equation is the accepted design method for open channel flow. Table 4.19 and Figures 4.44 through 4.57 provide hydraulic design information for steel box culverts with a 6 x 2 inch corrugation. The procedure is similar to that summarized previously.

Table 4.19

Hydraulic section parameters for structural plate box culverts

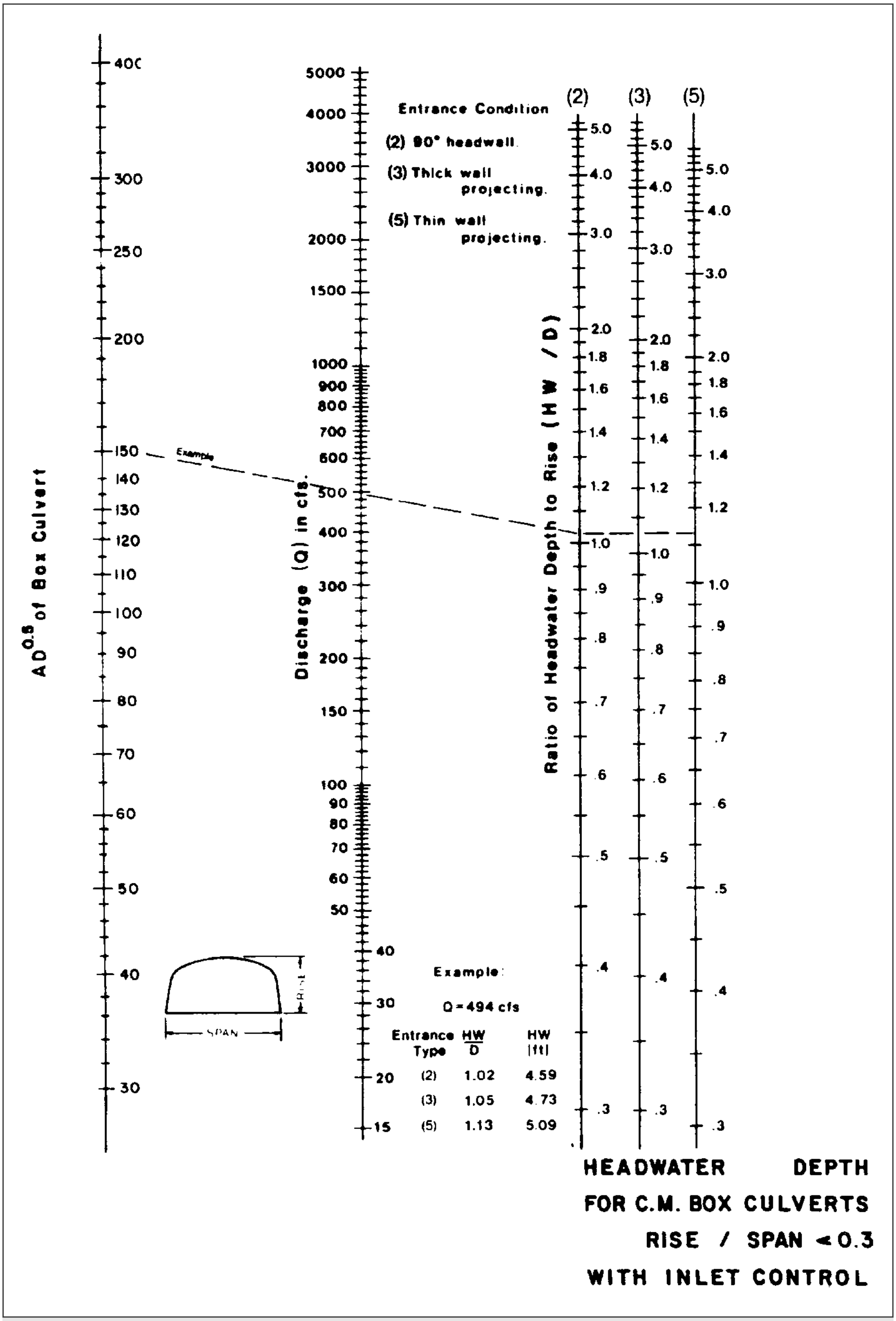
No.	Span ft-in.	Rise ft-in.	Area (sq ft)	WP ft.	$AR^{2/3}$	$AD^{1/2}$	No.	Span ft-in.	Rise ft-in.	Area (sq ft)	WP ft.	$AR^{2/3}$	$AD^{1/2}$
1	9-2	2-6	18.4	20.0	17.4	29.1	28	11-2	4-3	39.4	27.7	49.8	81.2
2	9-8	2-7	20.2	22.2	19.0	32.5	29	19-5	4-3	66.0	42.6	88.4	136.1
3	10-6	2-8	22.6	23.8	21.8	36.9	30	11-9	4-4	42.4	28.3	55.6	88.3
4	11-1	2-9	24.8	24.3	25.1	41.1	31	16-3	4-4	59.5	37.0	81.7	123.9
5	11-10	2-10	27.8	26.7	28.5	46.8	32	12-6	4-5	46.9	30.7	62.3	98.6
6	12-9	2-11	30.6	28.3	32.2	52.3	33	13-3	4-6	49.4	31.5	66.7	104.8
7	13-2	3-1	33.5	29.6	36.3	58.8	34	16-10	4-6	64.1	38.8	89.5	136.0
8	14-1	3-2	36.6	31.3	40.6	65.1	35	20-0	4-6	70.8	43.2	98.5	150.2
9	14-6	3-3	39.0	31.7	44.8	70.3	36	17-9	4-7	67.2	40.0	95.0	143.9
10	9-0	3-4	24.2	21.4	26.3	44.2	37	20-8	4-7	74.7	45.0	104.7	159.9
11	10-1	3-4	27.7	23.5	30.9	50.6	38	13-9	4-8	54.8	33.0	76.8	118.4
12	10-10	3-5	30.8	25.7	34.8	56.9	39	14-7	4-9	59.1	35.0	83.9	128.8
13	15-4	3-5	43.3	34.1	50.8	80.0	40	18-4	4-9	73.1	42.0	105.8	159.3
14	11-6	3-6	33.2	26.6	38.5	62.1	41	10-0	4-11	39.1	25.7	51.7	86.7
15	16-0	3-6	46.2	35.5	55.0	86.4	42	11-0	4-11	44.2	28.2	59.7	98.0
16	12-2	3-8	37.2	28.6	44.3	71.2	43	15-0	4-11	63.2	35.7	92.5	140.1
17	16-8	3-8	50.7	37.3	62.2	97.1	44	19-2	4-11	78.2	43.2	116.1	173.4
18	12-10	3-9	39.7	29.4	48.5	76.9	45	21-6	4-11	83.8	47.3	122.6	185.8
19	13-6	3-10	44.0	31.0	55.6	86.1	46	11-8	5-0	48.2	29.0	67.6	107.8
20	17-6	3-10	54.0	38.0	68.2	105.7	47	15-10	5-0	68.1	37.8	100.8	152.3
21	14-4	4-0	47.8	33.2	61.0	95.6	48	12-5	5-1	52.5	31.3	74.2	118.4
22	18-2	4-0	58.8	40.0	76.0	117.6	49	19-8	5-1	82.3	44.5	124.0	185.6
23	9-6	4-1	31.1	23.6	37.4	62.8	50	12-10	5-2	56.6	32.0	82.8	128.7
24	14-10	4-1	51.3	34.2	67.2	103.7	51	16-4	5-2	72.2	38.8	109.2	164.1
25	10-7	4-2	35.9	26.2	44.3	73.3	52	17-2	5-3	77.6	40.6	119.5	177.8
26	15-7	4-2	55.6	36.1	74.1	113.5	53	20-8	5-3	88.4	46.5	135.6	202.5
27	18-9	4-2	62.2	40.7	82.6	127.0	54	13-8	5-4	60.8	34.0	89.6	140.4

(continued)

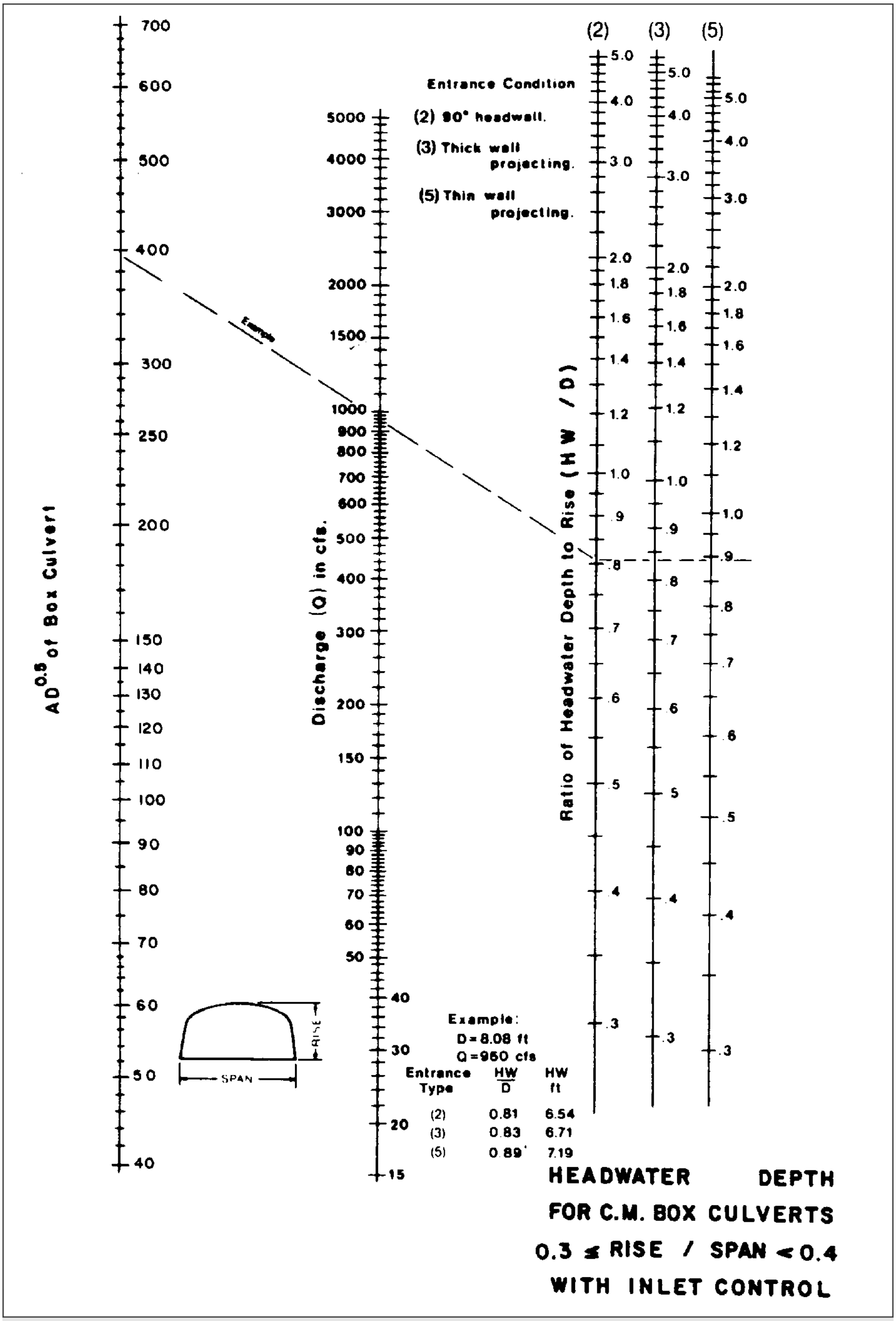
Table 4.19 (continued)

Hydraulic section parameters for structural plate box culverts

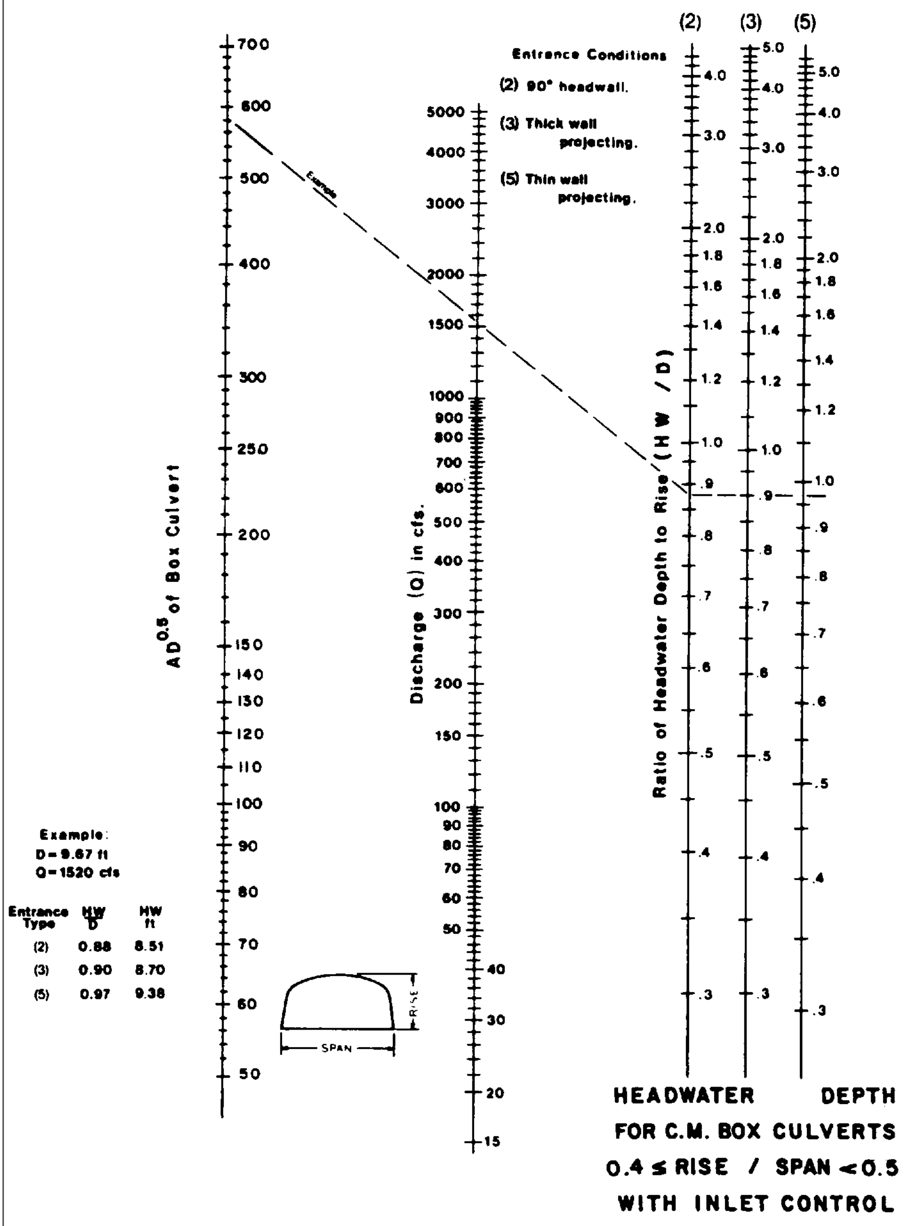
No.	Span ft-in.	Rise ft-in.	Area (sq ft)	WP ft.	AR $^{2/3}$	AD $^{1/2}$	No.	Span ft-in.	Rise ft-in.	Area (sq ft)	WP ft.	AR $^{2/3}$	AD $^{1/2}$
55	22-8	5-4	95.0	49.7	146.2	219.4	109	15-8	7-3	98.7	41.1	176.9	265.8
56	14-0	5-5	65.6	64.7	100.3	152.7	110	20-7	7-3	125.0	49.4	232.1	336.6
57	18-0	5-5	82.2	42.2	128.3	191.3	111	22-7	7-3	135.9	52.9	255.0	365.9
58	21-2	5-5	94.1	47.5	148.4	219.0	112	12-10	7-4	76.0	35.0	127.4	205.8
59	11-0	5-7	47.9	29.0	66.9	113.2	113	19-1	7-4	121.5	47.6	226.8	329.0
60	14-10	5-7	70.7	36.8	109.2	167.1	114	24-5	7-4	146.8	56.5	277.6	397.5
61	18-4	5-7	87.9	43.5	140.5	207.7	115	13-4	7-5	80.4	36.1	137.2	219.0
62	11-5	5-8	52.6	29.7	77.0	125.2	116	16-6	7-5	106.5	43.3	194.1	290.0
63	15-5	5-8	75.1	37.6	119.2	178.8	117	16-10	7-6	111.5	44.1	207.0	305.4
64	19-3	5-8	93.3	45.1	151.5	222.1	118	19-10	7-6	129.8	49.5	246.7	355.5
65	22-2	5-8	101.0	49.3	163.0	240.4	119	14-0	7-7	88.4	38.4	154.0	243.4
66	12-0	5-9	57.2	31.1	85.9	137.2	120	14-4	7-8	91.0	38.6	161.3	252.0
67	16-0	5-10	80.7	39.8	129.4	194.9	121	17-6	7-8	119.0	45.6	225.6	329.5
68	19-10	5-10	97.2	46.3	159.4	234.8	121	20-0	7-8	127.1	48.7	240.9	351.9
69	23-9	5-10	108.2	52.8	174.6	261.3	123	21-2	7-8	134.8	50.9	258.1	373.2
70	12-10	5-11	62.3	33.1	94.9	151.5	124	23-6	7-8	150.6	55.6	292.7	417.0
71	16-8	5-11	84.7	40.7	138.0	206.0	125	15-0	7-9	99.4	40.6	180.6	276.7
72	13-2	6-0	66.6	33.8	104.7	163.1	126	25-4	7-9	161.3	59.1	315.1	449.0
73	14-0	6-1	72.1	35.9	114.8	177.8	127	18-3	7-10	125.5	47.2	240.9	351.2
74	17-2	6-1	91.3	42.5	152.1	225.2	128	15-8	7-11	106.8	42.5	197.3	300.5
75	19-2	6-1	97.1	44.7	162.8	239.5	129	18-10	7-11	132.1	48.7	256.8	371.7
76	23-4	6-1	113.7	51.9	191.7	280.4	130	20-10	7-11	143.3	51.7	282.6	403.2
77	20.11	6-2	105.5	48.2	177.9	262.0	131	16-0	8-1	111.8	43.2	210.9	317.9
78	24-10	6-2	119.0	54.3	200.7	295.5	132	19-3	8-1	135.9	49.2	267.4	386.4
79	14-4	6-3	76.6	36.7	125.2	191.5	133	20-11	8-1	141.2	51.3	277.2	401.4
80	18-0	6-3	96.9	43.8	164.4	242.3	134	22-10	8-1	153.7	54.7	306.1	437.0
81	10-10	6-4	53.8	29.6	80.2	135.4	135	16-8	8-2	119.6	44.8	230.2	341.8
82	15-1	6-4	82.5	38.7	136.6	207.6	136	24-6	8-2	166.2	58.1	334.9	475.0
83	18-4	6-4	101.9	44.9	176.0	256.4	137	16-10	8-3	124.4	45.6	243.0	357.3
84	11-9	6-5	61.2	31.7	94.9	155.0	138	20-2	8-3	146.2	51.5	293.2	419.9
85	15-6	6-6	87.1	39.2	148.2	222.1	139	17-1	8-3	125.8	45.7	246.9	363.2
86	19-2	6-6	106.6	46.6	185.2	271.8	140	20-4	8-5	142.6	50.7	284.0	413.7
87	20-2	6-6	109.2	47.5	190.3	278.4	141	21-10	8-5	155.7	53.9	315.9	451.7
88	22-5	6-6	118.4	51.0	207.5	301.9	142	17-8	8-6	133.0	47.5	264.2	387.8
89	24-4	6-6	127.2	55.1	222.3	324.3	143	23-8	8-6	169.3	57.2	349.0	493.6
90	12-5	6-7	66.4	33.0	105.8	170.4	144	18-4	8-7	139.9	48.7	282.5	409.9
91	16-3	6-7	93.5	41.6	160.5	239.9	145	21-2	8-7	160.7	54.1	332.0	470.8
92	13-1	6-8	72.3	35.1	117.1	186.7	146	25-4	8-7	182.4	60.8	379.4	534.4
93	19-8	6-8	113.3	47.6	202.0	292.5	147	18-11	8-9	147.0	50.0	301.7	434.8
94	13-6	6-9	76.9	35.8	128.1	199.8	148	21-3	8-10	157.6	53.4	324.2	468.4
95	16-10	6-9	98.3	42.2	172.8	255.4	149	23-0	8-10	171.6	56.4	360.1	510.0
96	17-5	6-10	105.1	44.3	186.8	274.7	150	19-4	8-11	152.1	51.2	314.3	454.2
97	21-3	6-10	121.7	50.3	219.4	318.1	151	24-6	8-11	185.7	59.9	394.8	554.5
98	14-2	6-11	83.1	31.8	140.6	218.5	152	20-0	9-1	161.6	53.0	339.9	487.0
99	19-7	6-11	111.9	46.8	200.0	294.3	153	21-10	9-3	173.3	55.9	368.4	527.1
100	23-5	6-11	132.1	53.7	240.7	347.4	154	23-10	9-3	188.1	59.2	406.8	572.1
101	14-8	7-0	88.4	38.8	153.1	233.9	155	21-3	9-5	177.4	55.6	384.4	544.4
102	18-1	7-0	111.1	45.2	202.5	293.9	156	25-4	9-5	202.9	62.2	446.1	622.6
103	21-2	7-0	127.4	50.8	235.3	337.1	157	23-2	9-8	189.8	58.4	416.4	590.1
104	25-4	7-0	142.2	57.6	259.8	376.2	158	24-8	9-9	205.3	61.5	458.7	641.0
105	15-5	7-1	94.5	40.6	166.1	251.5	159	24-0	10-1	207.0	60.9	468.0	657.3
106	11-4	7-2	63.0	31.8	99.3	168.7	160	25-5	10-2	222.7	94.0	511.4	710.1
107	18-9	7-2	117.3	46.5	217.2	314.0	161	24-9	10-6	225.0	63.2	524.6	729.1
108	12-3	7-3	71.2	34.4	115.6	191.7							



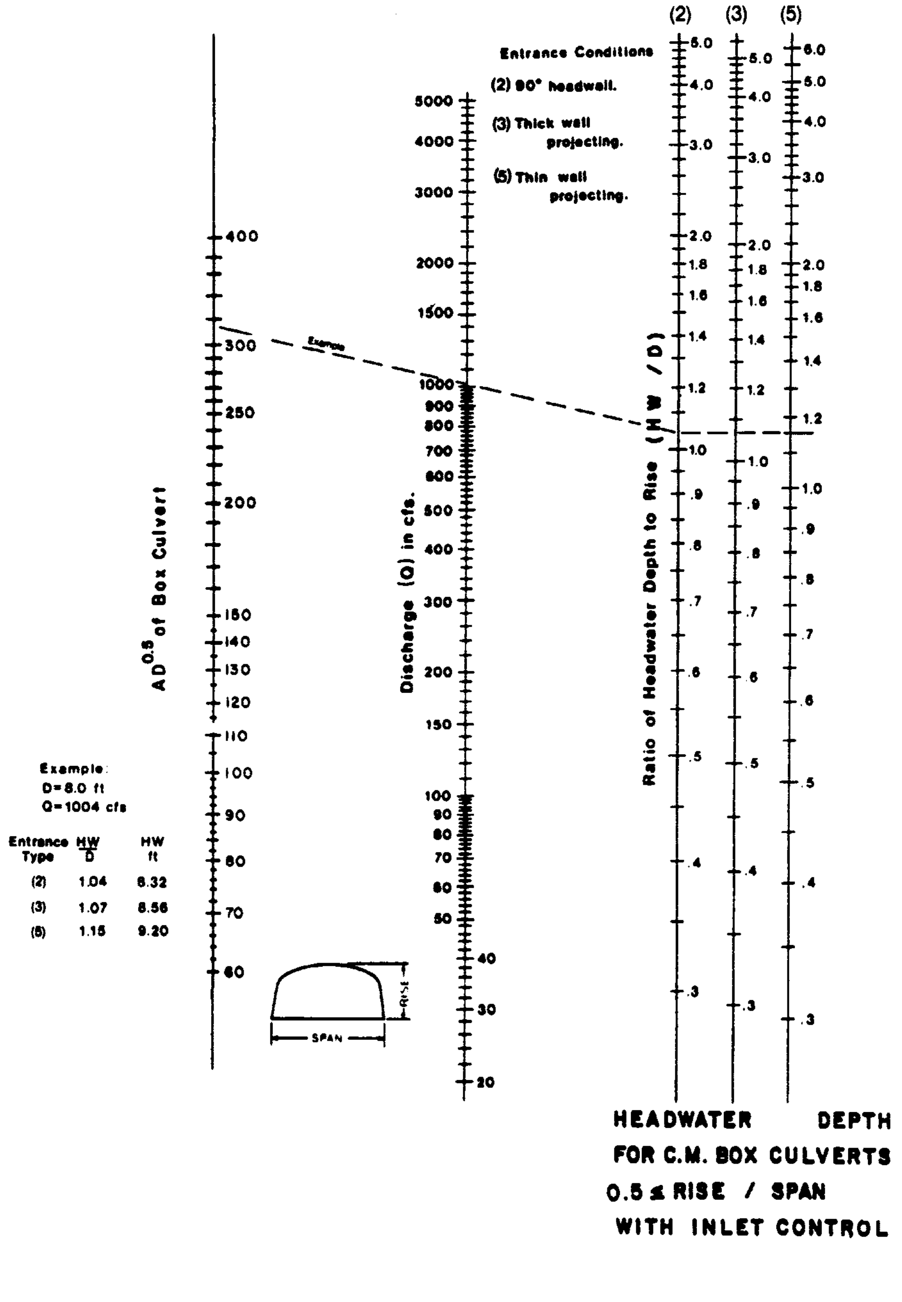
■ **Figure 4.44** Headwater depths for structural plate box culverts, with rise/span < 0.3, under inlet control.



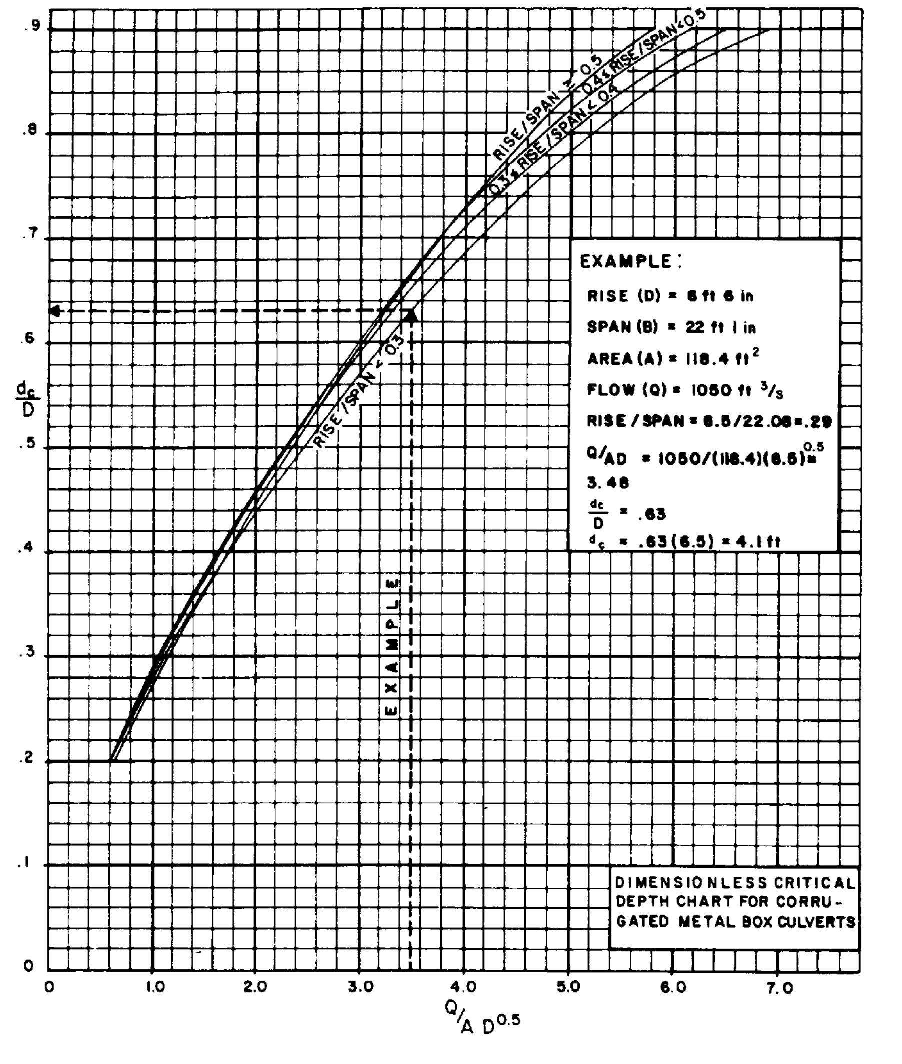
■ **Figure 4.45** Headwater depths for structural plate box culverts, with 0.3 <= rise/span < 0.4, under inlet control.



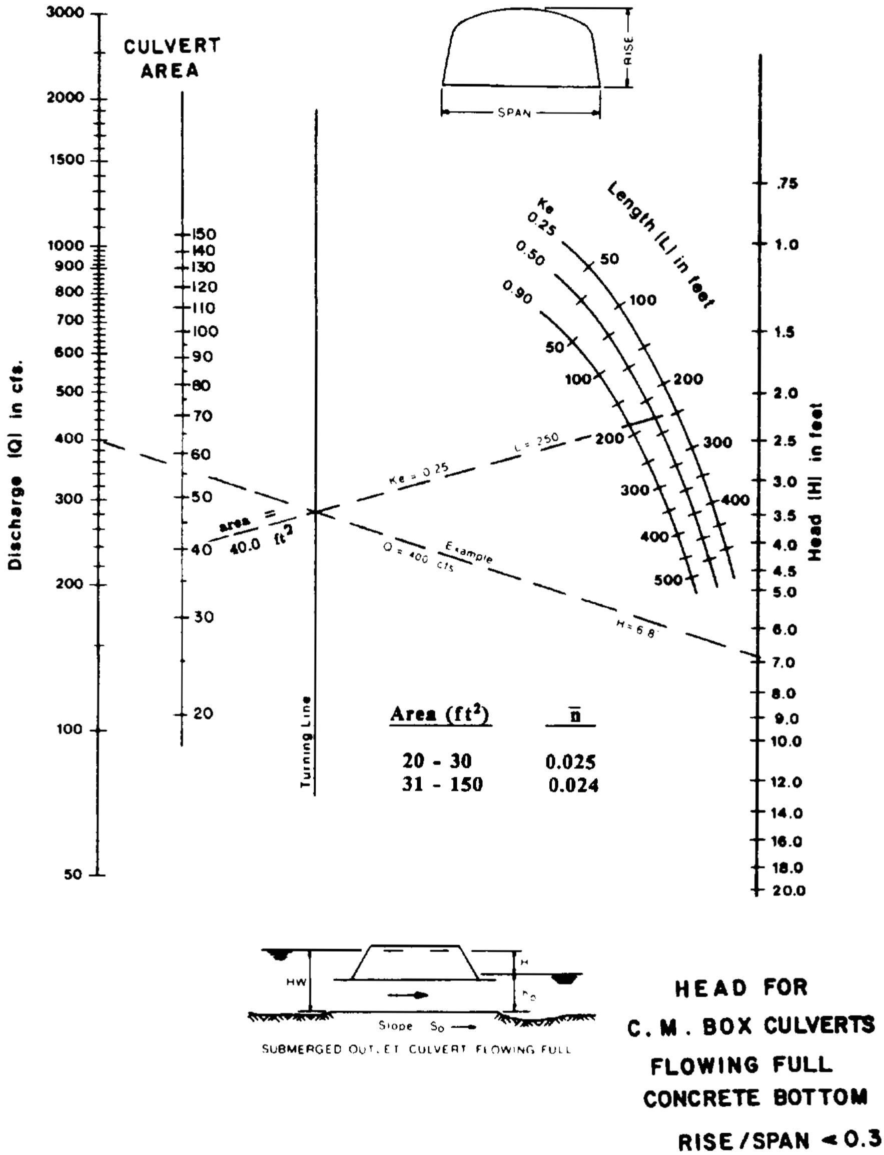
■ **Figure 4.46** Headwater depths for structural plate box culverts, with 0.4 <= rise/span < 0.5, under inlet control.



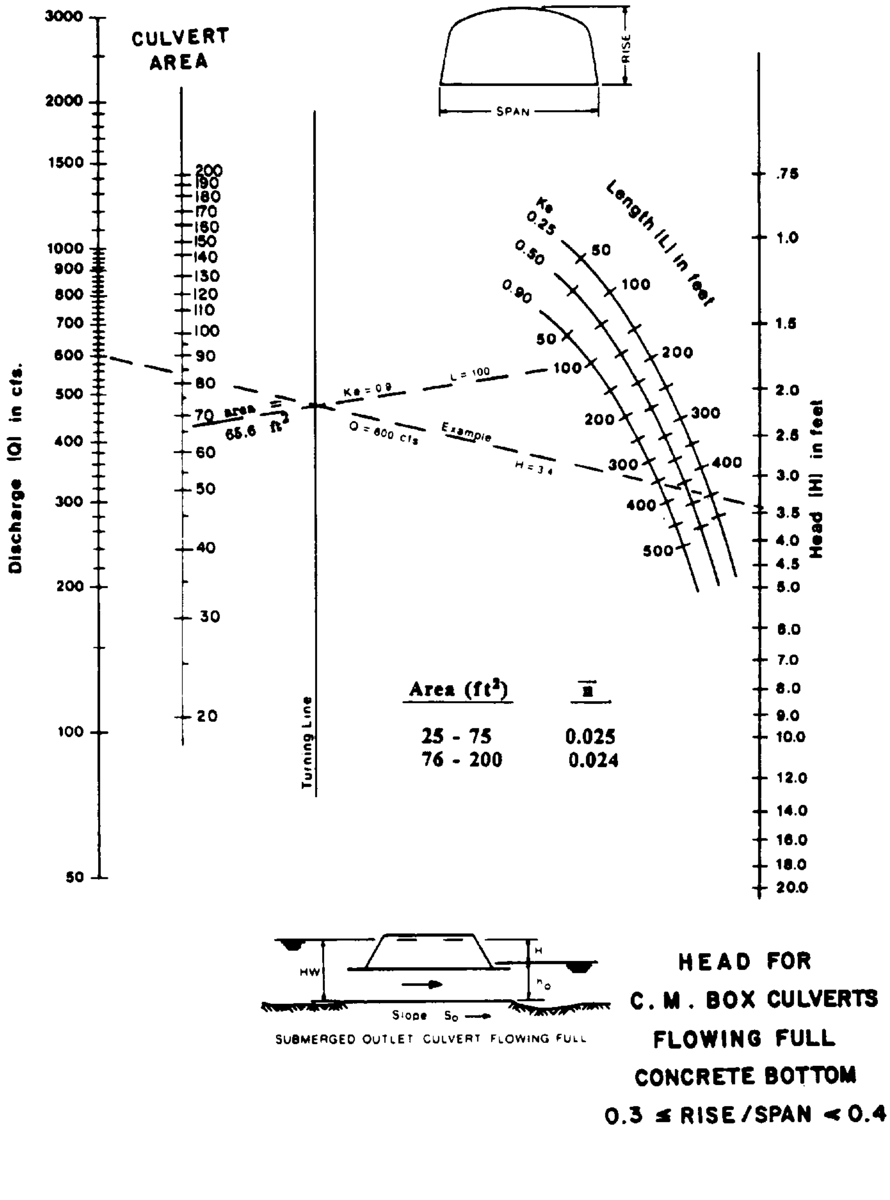
■ **Figure 4.47** Headwater depths for structural plate box culverts, with 0.5 <= rise/span, under inlet control.



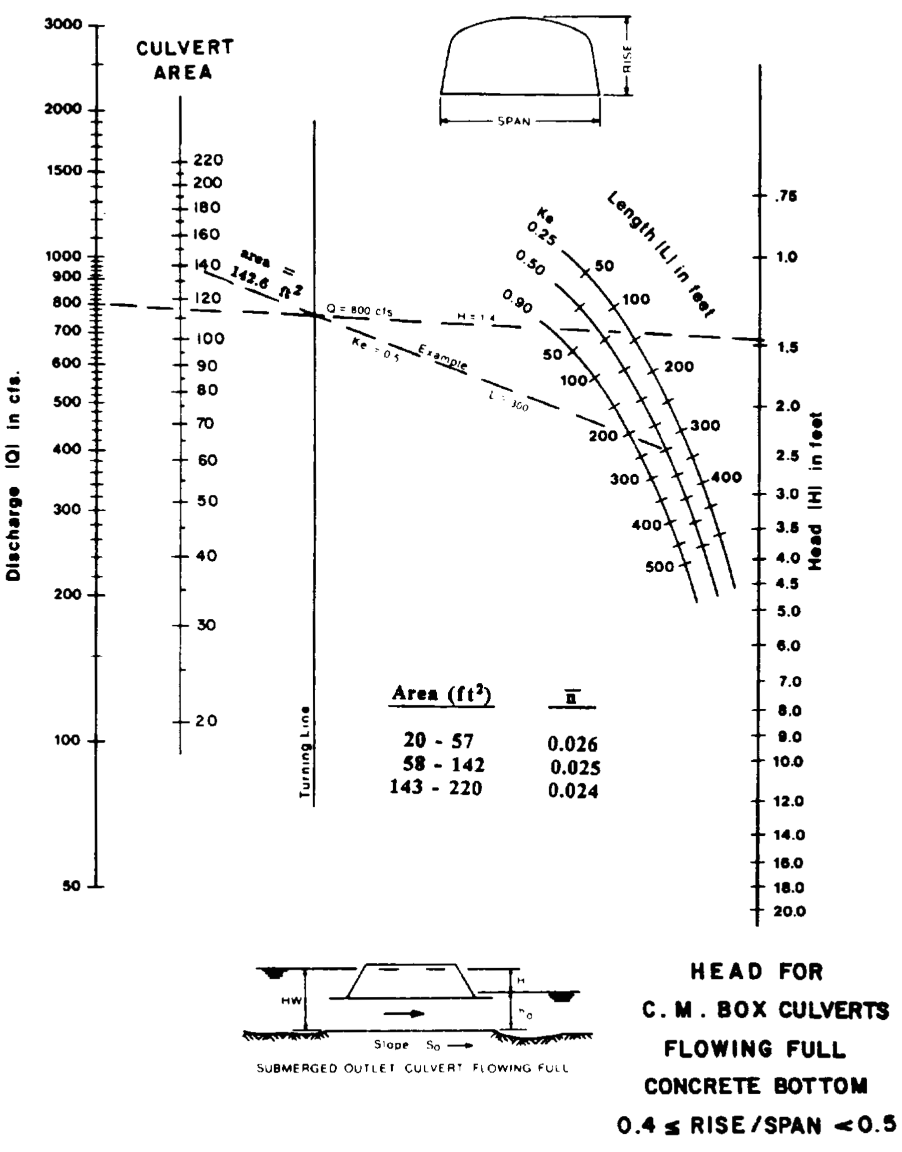
■ **Figure 4.48** Critical depth for corrugated steel structural plate box culverts.



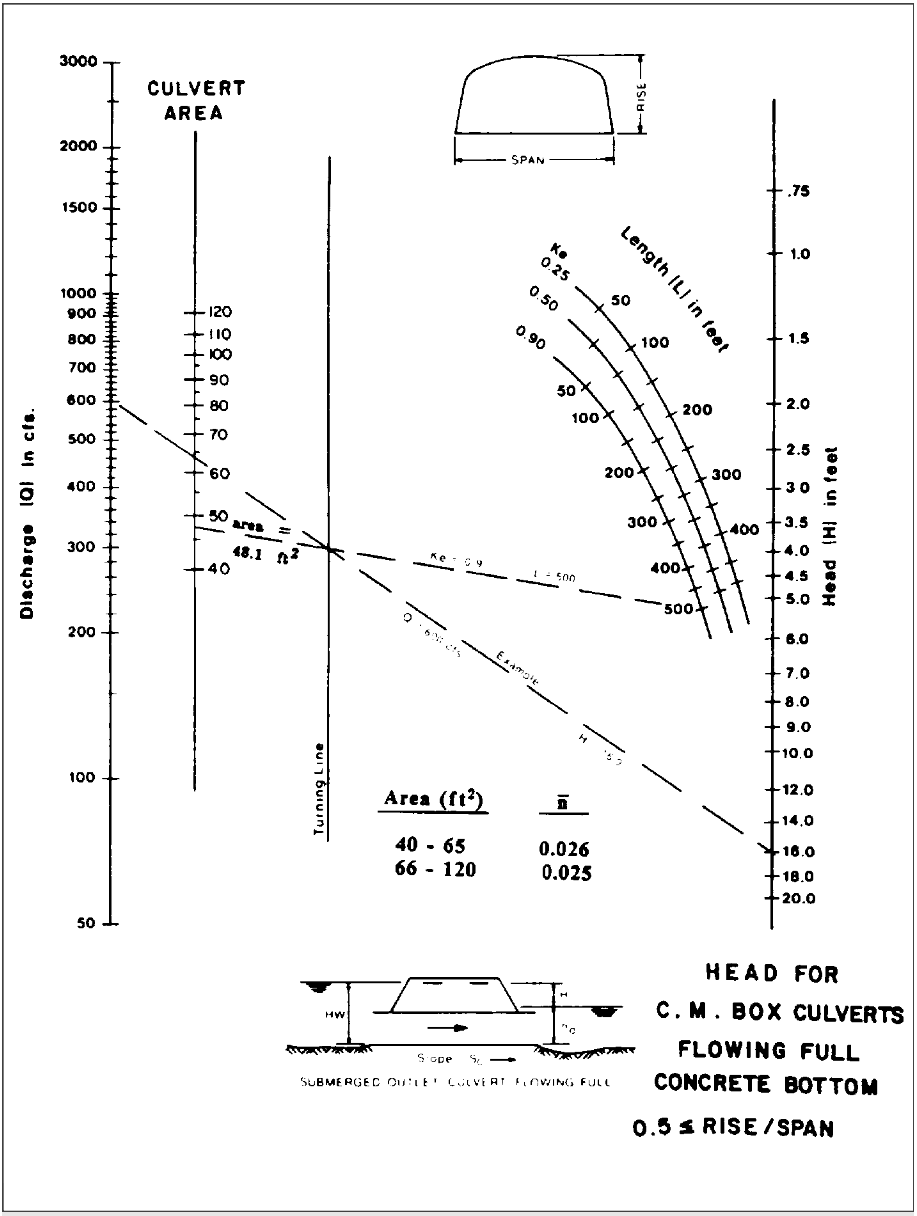
■ **Figure 4.49** Head for 6x2 structural plate box culverts, with concrete invert and rise/span < 0.3, flowing full under outlet control.



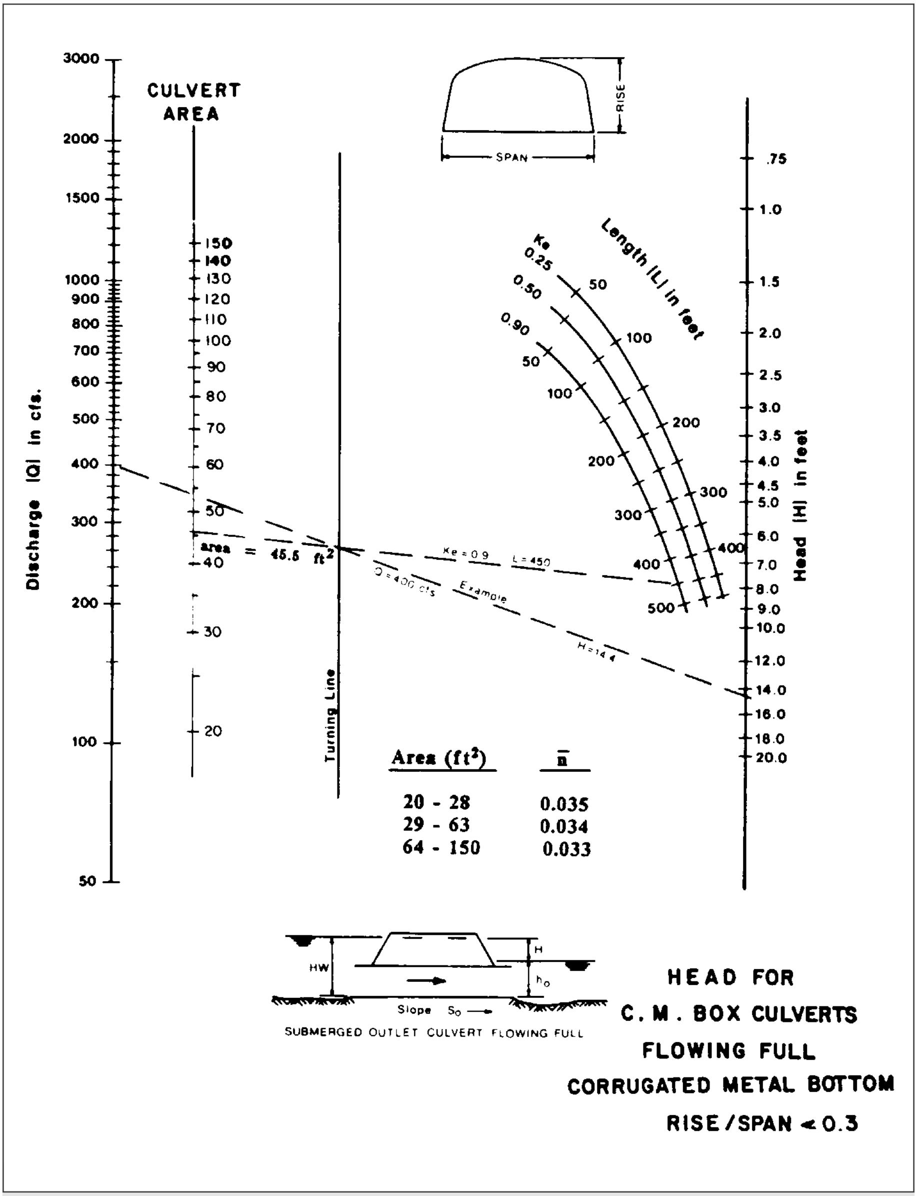
■ **Figure 4.50** Head for 6x2 structural plate box culverts, with concrete invert and 0.3 <= rise/span < 0.4, flowing full under outlet control.



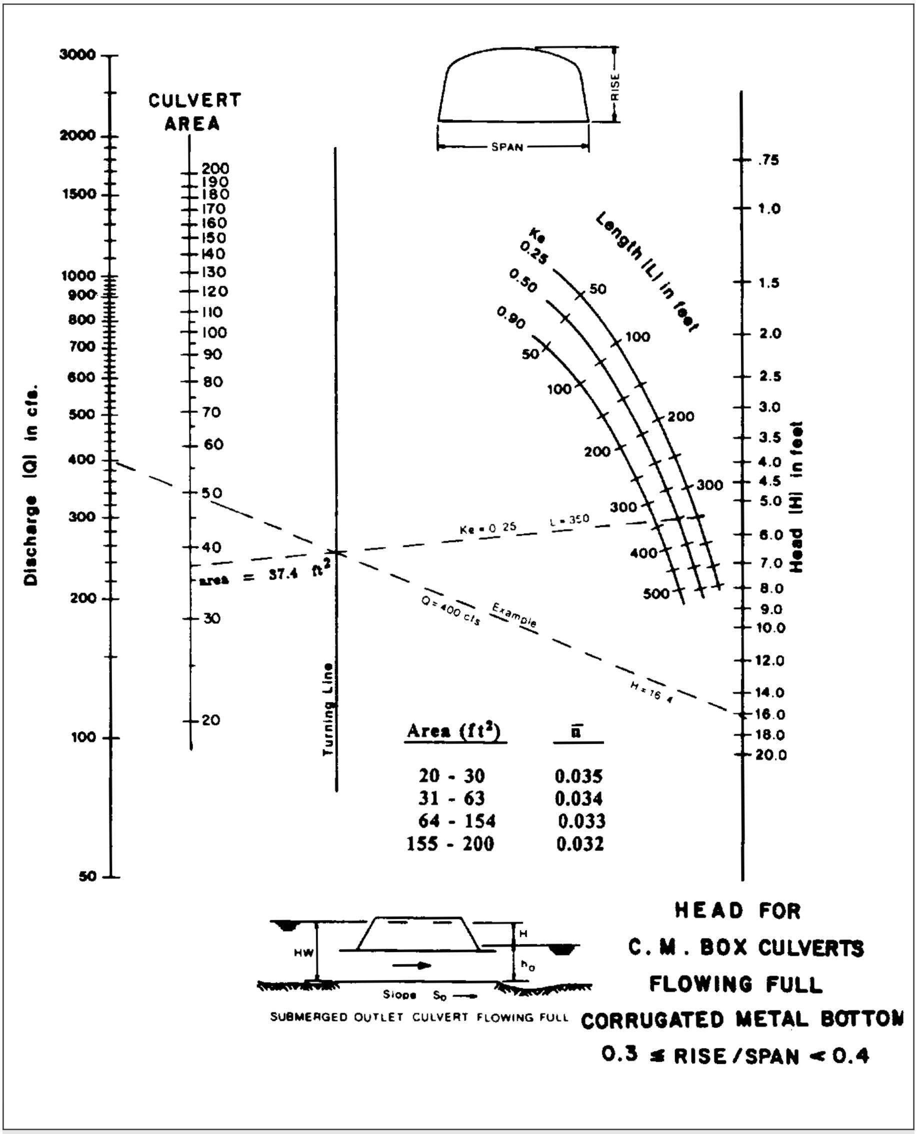
■ **Figure 4.51** Head for 6x2 structural plate box culverts, with concrete invert and 0.4 <= rise/span < 0.5, flowing full under outlet control.



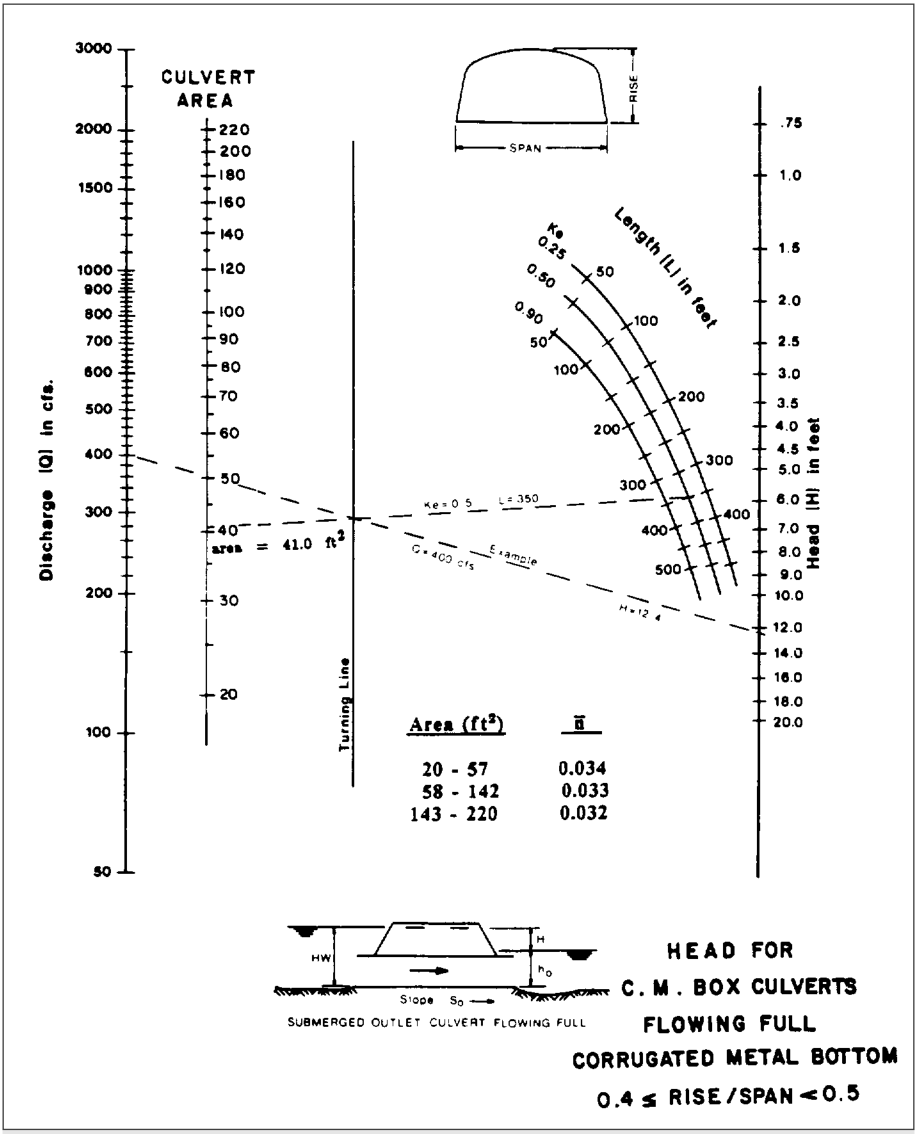
■ **Figure 4.52** Head for 6x2 structural plate box culverts, with concrete invert and 0.5 <= rise/span, flowing full under outlet control.



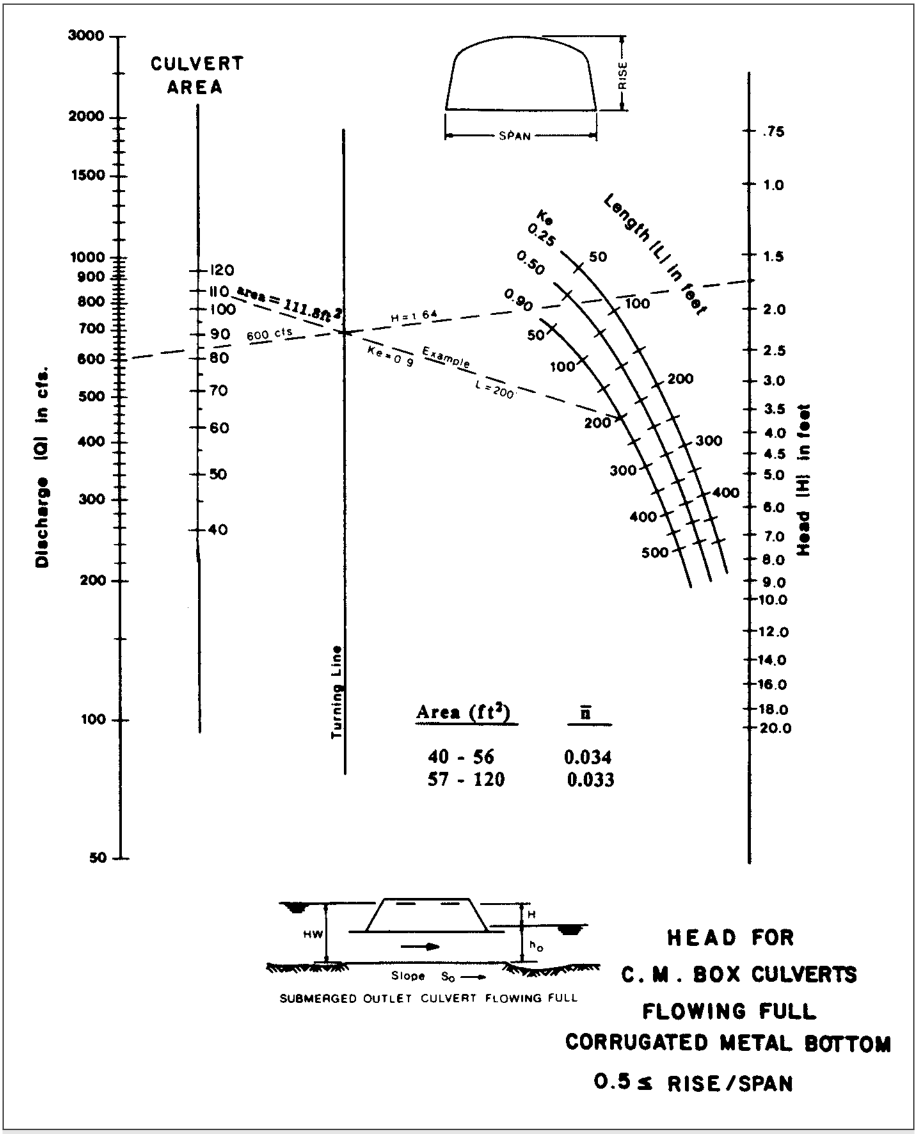
■ **Figure 4.53** Head for 6x2 structural plate box culverts, with corrugated invert and rise/span < 0.3, flowing full under outlet control.



■ **Figure 4.54** Head for 6x2 structural plate box culverts, with corrugated invert and 0.3 <= rise/span < 0.4, flowing full under outlet control.



■ **Figure 4.55** Head for 6x2 structural plate box culverts, with corrugated invert and 0.4 <= rise/span < 0.5, flowing full under outlet control.



■ **Figure 4.56** Head for 6x2 structural plate box culverts, with corrugated invert and 0.5 <= rise/span, flowing full under outlet control.

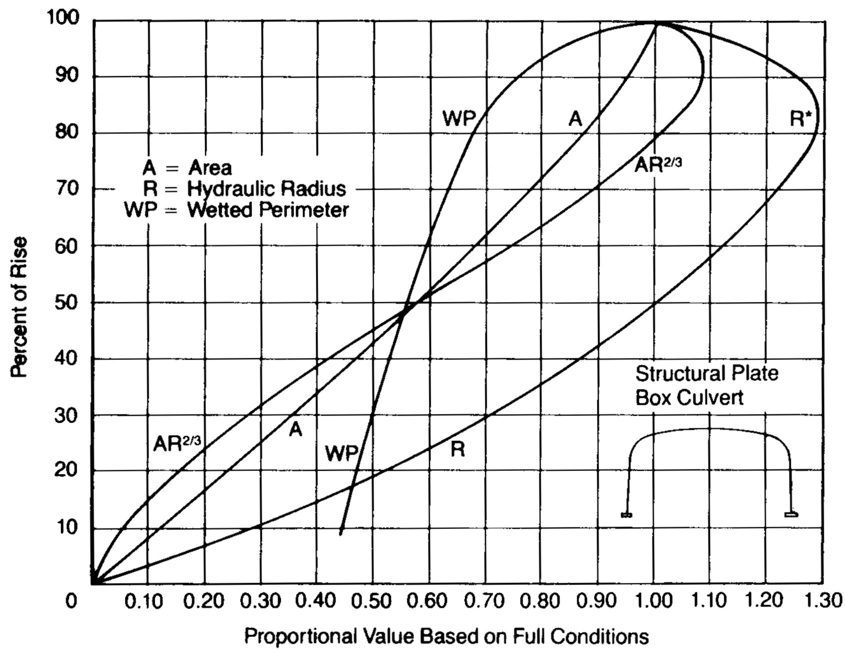


Figure 4.57 Hydraulic section parameters for structural plate box culverts.

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: 1) Vortexes and eddy currents that cause scour, which in turn undermine the inlet and erode the culvert supporting embankment slope; 2) 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 3) 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

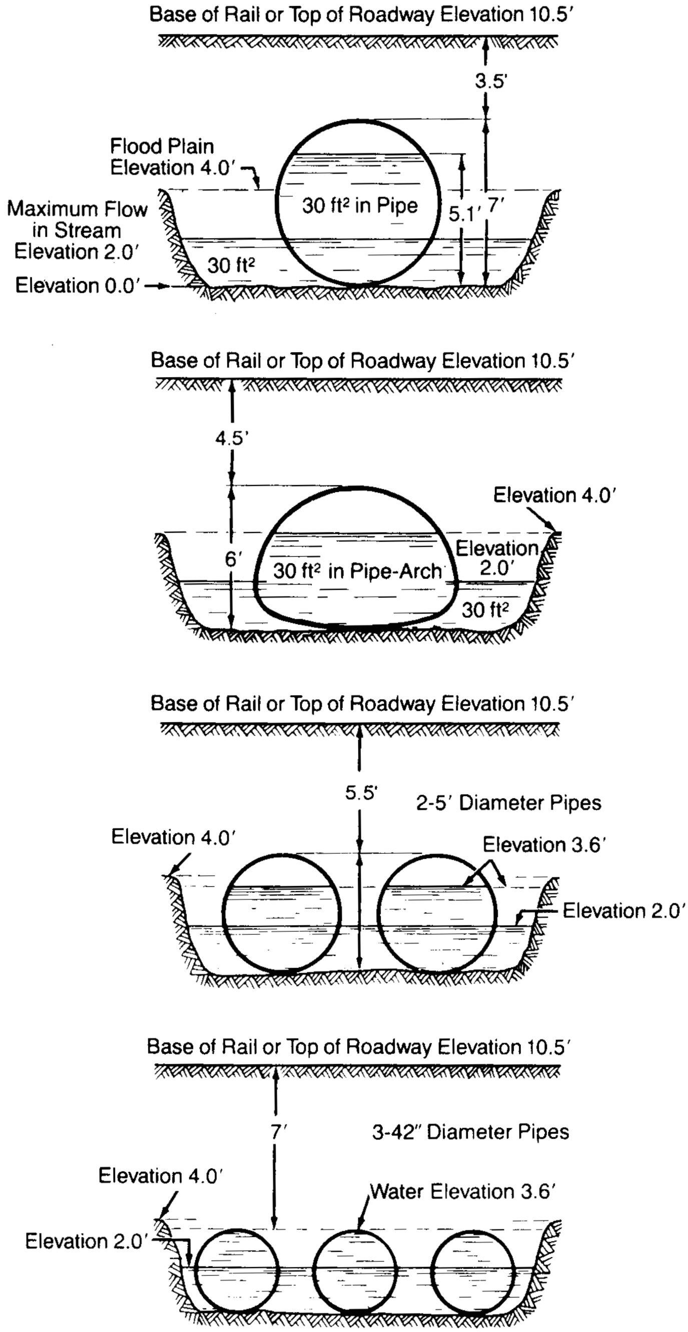
These are perforations in the culvert barrel which are used to relieve pore pressure in the embankment. Generally, weep holes are not required in culvert design. For an installation involving prolonged ponding, there may be merit in considering a separate subdrainage system to relieve pore pressure and 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 4.58.



■ **Figure 4.58** Comparison of single and multiple opening installations, and round and pipe arch installations.

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Pipe inspection.