9

CLOSED CONDUITS

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Storm drains and culverts are generally minor parts of an overall drainage system. Storm drains are required when other minor parts of the overall system, primarily curb, gutter and roadside ditches, no longer have capacity for additional flow. Culverts, on the other hand, are generally part of a drainage system that is more natural and exists because of an interruption in the system such as roadway or other types of manufactured embankment.

In either case, particular care must be taken during design to avoid adverse effects on the traveling public, receiving waters or facilities and upstream and downstream properties. The following section presents information on the appropriate design of storm drains, culverts, inlets and outlets, and appurtenant facilities.

9.1 STORM DRAINS

A storm drain system refers to a system of inlets, conduits, manholes and other appurtenances which are designed to collect and convey storm runoff from the design storm to a point of discharge.

Storm drain system design is based on the design storm. The design storm is the storm associated with the governing return period from Table 6.2 (page 6.3). In the upper reaches of a system the 10-year design storm will govern. However, farther downstream in the system, the storm drain design for the 10-year storm may not meet the stated criteria. In this case, the storm drain will need to be upsized to meet the criteria for the 100-year design storm. However, this does not necessarily imply storm drain designed for 100-year discharges.

Both return periods must be checked. The storm condition governing design at any point is the design storm.

9.1.1 General Hydraulic Criteria

Closed conduits shall be designed to convey the design storm flood peak at full flow, whenever possible, and may be allowed to flow under pressure except when the following conditions exist:

- In areas of high debris potential, there is a possibility of stoppage occurring in drains. In situations where debris may be expected, City staff should be consulted for a determination of the appropriate bulking factor.

- In certain situations open channel sections upstream of the proposed closed conduit may be adversely affected by backwater effects.

If the proposed conduit is to be designed for pressure conditions, the hydraulic grade line shall not be higher than 1.0 foot below the normal gutter flow line at any point in a specific reach of any storm drain. However, in those reaches where no surface flow will be intercepted, a hydraulic grade line which encroaches on, or is slightly higher than, the ground or street surface will be acceptable.

9.1.2 Hydraulic Design

To ensure the objectives of Section 9.1.1 are achieved, the hydraulic grade line shall be calculated by accounting for pipe friction losses and other minor losses that may occur along a particular reach.

Most procedures for calculating hydraulic grade line profiles are based on the Bernoulli Equation. This equation can be expressed as follows:

$$\frac{V_1^2}{2g} + D_1 + S_o L = \frac{V_2^2}{2g} + D_2 + S_f L + h_{minor} \qquad \textbf{(9.1)}$$

in which D is the vertical distance from the invert to the HGL, S_o is the invert slope, L is the projected, horizontal length of the conduit, S_f is the friction slope, V is the average velocity and h_{minor} are the minor head losses. Minor losses have been included in the Bernoulli Equation because of their importance in calculating hydraulic grade line profiles and are assumed to be uniformly distributed along any pipe length.

The total hydraulic loss in a system is equivalent to the sum of the losses due to friction and the minor losses in that system. Minor losses include those caused by expansion or contraction of flow, changes in pressure and momentum at junctions, pipe bends and manholes.

9.2 HEAD LOSS IN CLOSED CONDUITS

Generally, between the inlet and outlet of a storm drain the flow encounters a variety of configurations such as changes in pipe size, laterals, bends, junctions, expansions and contractions. These variations in shape or flow pattern impose losses in addition to those resulting from friction.

Head losses can generally be stated as a function of velocity head multiplied by a particular constant. the following discussion presents the most common head losses encountered in a storm drain system.

9.2.1 Friction Loss

Friction loss for closed conduits carrying storm water can be calculated from Manning's Equation (see Equation 7.1) or a derivative thereof. When rearranged into a more useful form,

$$S_f = \left(\frac{Qn}{1.486AR^{2/3}}\right)^2 = \left(\frac{Q}{K}\right)^2 \qquad \textbf{(9.2)}$$

in which,

$$K = \frac{1.486AR^{2/3}}{n} \qquad \textbf{(9.3)}$$

The loss due to friction throughout a reach of length L is, therefore, calculated by:

$$h_f = S_f L = \left(\frac{Q}{K}\right)^2 L \qquad \textbf{(9.4)}$$

The value of K is dependent upon only two factors: the geometric shape of the flow cross section as expressed by the quantity $AR^{2/3}$, and the roughness coefficient, n.

9.2.2 Transition Loss

Transition loss shall be calculated from the Equations 9.5 and 9.6 shown below. These equations are applicable when no change in flow rate occurs and where the horizontal angle of divergence or convergence between the two sections does not exceed 5°45'.

For velocities which increase in the direction of flow ($V_2 > V_1$),

$$h_t = 0.1\left(\frac{V_2^2}{2g} - \frac{V_1^2}{2g}\right) \qquad \textbf{(9.5)}$$

For velocities which decrease in the direction of flow ($V_2 < V_1$),

$$h_t = 0.2\left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g}\right) \qquad \textbf{(9.6)}$$

9.2.3 Junction Loss

In general, junction losses shall be calculated by equating pressure plus momentum through the confluences under consideration. This can be done by using any pressure plus momentum method or the City of Los Angeles' Thompson equation. Both methods are applicable in all cases for pressure flow and will give the same results.

For the special case of pressure flow with upstream and downstream areas being equivalent and friction neglected,

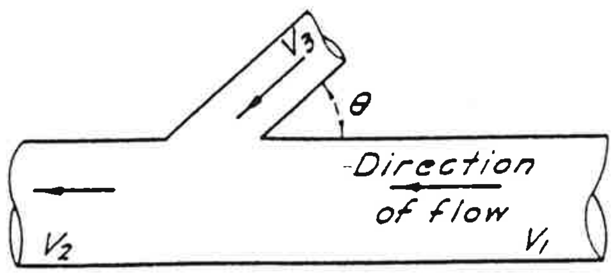


Figure 9.1

$$h_j = \frac{V_2^2}{2g} - \frac{V_1^2}{2g} - \frac{2A_3}{A_2} \times \frac{V_3^2}{2g} \times COS\theta \qquad (9.7)$$

It is important to note that the junction loss calculated using Equation 9.7 should be applied to the energy grade line as opposed to the hydraulic grade line.

The *Thompson Equation:* The Thompson Equation for junction loss is described by the following:

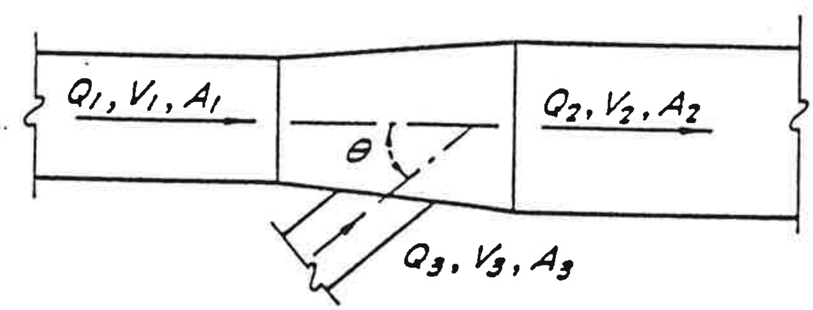


Figure 9.2

$$\Delta y \times A_{AVG.} = \frac{Q_2 V_2 - Q_1 V_1 - Q_3 V_3 COS\theta}{g} \qquad (9.8)$$

where delta y is the difference in hydraulic gradient of the two sections in feet and $A_{avg.}$ is the average area in square feet.

The above equation is applicable only to prismoidal and circular conduits or channels. The friction force may be considered negligible or can be calculated and taken into account. The Thompson Equation should not be used for open channel junction analysis when the open channel changes side slope going through the junction.

For details of the above method, refer to *Hydraulic Analysis of Junctions, 1968, Storm Drain Design Division, Bureau of Engineering, City of Los Angeles.*

9.2.4 Manhole Loss

Manhole losses shall be calculated using Equation 9.9 and shall be applied only to manholes where no lateral connections are made.

$$h_{M.H.} = 0.5(\frac{V^2}{2g}) \qquad \textbf{(9.9)}$$

When a change in cross section and/or change flow rate occurs, the head loss across the manhole shall be calculated in accordance with Sections 9.2.2 and 9.2.3.

9.2.5 Bend Loss

Bend losses are determined by the application of the following equation:

$$h_b = K_b(\frac{V^2}{2g}) \qquad \textbf{(9.10)}$$

The bend loss coefficient, K_b, is a function of the central angle of the bend. For any central angle not exceeding 90 degrees, K_b can be determined by:

$$K_b = 0.2(\frac{V^2}{2_g})^{0.5} \qquad \textbf{(9.11)}$$

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

9.2.6 Angle Point Loss

Angle point losses have been determined to be a function of the deflection angle, Θ, and can be calculated using the following equation:

$$h_{\Delta} = .0033\theta(\frac{V^2}{2g}) \qquad \textbf{(9.12)}$$

In most common applications, the deflection angle should not exceed 6 degrees. Should a deflection angle of greater magnitude be necessary, approval of the City Engineer is necessary.

9.2.7 Special Cases

Branching of Flow in Pipe - Head Loss: The following equation may be used to determine the loss of head in cases where it may be necessary to split or branch the flow into another drain.

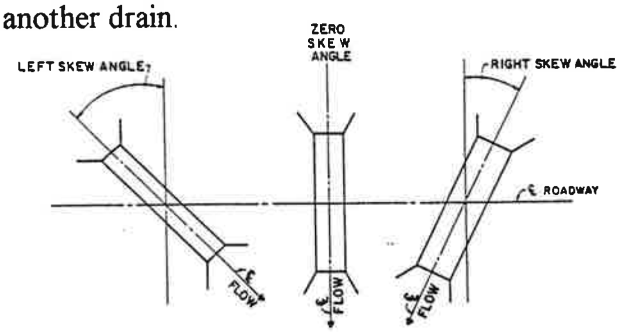


Figure 9.3

$$h_{br} = C\frac{V_1^2}{2g} \quad \textbf{(9.13)}$$

Values for the coefficient C may be obtained from the Table 9.1 below and apply only to straight reaches of pipe of constant diameter. Typical values for the coefficient C are listed for various ratios of Q_3/Q_1 and various angles of divergence.

Divergence Angle - ⊖	$\frac{Q_3}{Q_1} = 0.3$	$\frac{Q_3}{Q_1} = 0.5$	$\frac{Q_3}{Q_1} = 0.7$
90°	0.76	0.74	0.80
60°	0.59	0.54	0.52
45°	0.35	0.32	0.30

Table 9.1
Head Loss Coefficient, C For Use With Equation 9.13

For angles of divergence and ratios of Q_3/Q_1 other than those shown, values of C may be interpolated.

9.3 DESIGN REQUIREMENTS FOR MAINTENANCE AND ACCESS

9.3.1 Manholes

Spacing:

- *Conduit 45 inches in diameter of smaller*: Manholes shall be spaced at intervals of approximately 400 feet.

- *Conduit Diameter 45 inches or larger*: Manholes shall be spaced at intervals of approximately 500 feet.

The spacing requirements shown above apply regardless of design velocities. Deviations from the criteria shall be subject to approval by the City Engineer.

Location: Manholes should not be located in street intersections, especially when one or more streets are heavily traveled.

In situations where the proposed conduit is to be aligned within an easement and in street right-of-way, manholes should be located in the street right-of-way, wherever possible.

Manholes should be located as close to changes in grade as feasible when the following condition exists:

- When the upstream conduit is steeper than the downstream conduit and the change in grade is greater than 10 percent, sediment tends to deposit at the point where the change in grade occurs.

Manhole Design: When the design flow in a pipe flowing full has a velocity of 20 feet per second or greater, or is supercritical in a partially full pipe, the total horizontal angle of divergence or convergence between the walls of the manhole and its center line shall not exceed 5°45".

Deep Manholes: A manhole shaft safety ledge shall be provided in all instances when the manhole shaft is 20 feet or greater in depth. Installation shall be in accordance with Los Angeles County Flood Control District Standard Drawing No. 2-D430.

Pressure Manholes: A pressure manhole shaft and a pressure frame and cover shall be installed in a pipe or box storm drain whenever the design water surface (HGL) is above the ground surface.

9.3.2 Minimum Slope

The minimum slope for main line conduit shall be 0.10 percent, unless otherwise approved by the City Engineer.

9.3.3 Minimum Pipe Size

The minimum diameter of main line conduit shall be 24 inches, unless otherwise approved by the City Engineer.

In cases where the conduit may carry a significant amount of debris, the minimum diameter of the main line conduit shall be 30 inches.

9.3.4 Inlet Structures

An inlet structure shall be provided for storm drains accepting storm flows from any natural conveyance. The structure should generally consist of a headwall, wingwalls to protect any adjacent banks from erosion, and a paved inlet apron. The apron slope should be limited to a maximum of 2:1.

Wall heights should conform to the height of the water upstream of the inlet, and be adequate to protect both the fill over the drain and the embankment. Headwall and wingwall fencing and a protection barrier to prevent public entry shall be provided.

If debris is prevalent, barriers consisting of vertical 3-inch or 4-inch diameter steel pipe at 24 to 36 inches on center should be embedded in concrete immediately upstream of the inlet apron.

9.3.5 Outlet Structures

When a storm drain outlets into a natural channel or other historic conveyance, an outlet structure shall be provided which prevents erosion and property damage. Velocity of flow at the outlet should agree as closely as possible with existing channel velocity. The following guidelines shall apply:

- When the discharge velocity is low, or subcritical, the outlet structure shall consist of a headwall, wingwalls and an apron. The apron may consist of a concrete slab, riprap or grouted rock.

- When the discharge velocity is high, or supercritical, the designer should consider bank protection in the vicinity of the outlet and an energy dissipator structure. Please refer to Section 8 for discussion of energy dissipators.

9.4 DESIGN REQUIREMENTS FOR JUNCTIONS

9.4.1 Inlets to Main Line Drains

Lateral pipe entering a main line storm drain shall be connected at the spring line of main line. Lateral pipe entering a main line box structure shall conform to the following:

- Lateral pipe 24 inches or less in diameter shall enter the main line no more than five feet above the invert.

- Lateral pipe 27 inches in diameter or larger shall enter the main line no more than 18 inches above the invert, with the exception that catch basin connector pipes less than 50 feet in length shall enter no more than 5 feet above the invert.

Exceptions to the above requirements may be permitted where it can be shown that the cost of bringing laterals into a main line box conduit in conformance with the above requirements would be excessive.

9.4.2 Angle of Confluence

In general, the angle of confluence between main line and lateral shall not exceed 45 degrees and, as an additional requirement, shall not exceed 30 degrees under any of the following conditions:

- Where the flow in the proposed lateral exceeds 10 percent of the main line flow;

- The velocity of flow in the proposed lateral is 20 feet per second or greater;

- The size of the proposed lateral is 60 inches or greater; and

- Where hydraulic calculations indicate excessive head losses may occur in the main line due to the confluence.

Connector pipe may be joined to main line pipe at angles greater than 45 degrees up to a maximum of 90 degrees provided none of the above conditions exist. If, in any specific situation, one or more of the above conditions do not apply, the angle of confluence for the connector pipes shall not exceed 30 degrees.

The above requirements may be waived only if calculations are submitted to the City showing that the use of a confluence angle larger than 30 degrees will not unduly increase head losses in the main line.

9.5 CULVERTS

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

9.5.1 Culvert Criteria

Sizing: Crossroad culverts should be sized according to the following criteria:

Other design frequencies shall be used only with the approval of the City Engineer.

- 10-year discharge with no head
- 100-year discharge utilizing available head

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

Maximum Velocity: As a practical limit, outlet velocities should be kept below 15 feet per second. In general, the maximum velocity should be consistent with channel stability requirements at the culvert outlet. As outlet velocities increase, the need for channel stabilization at the culvert outlet increases. If culvert outlet velocities exceed permissible velocities for the outlet channel lining material, suitable outlet protection must be provided. Outlet velocities may exceed permissible downstream channel velocities by up to 10 percent without providing outlet protection if the culvert tailwater depth is greater than the culvert critical depth of flow under design flow conditions.

Skewed Channels: The alignment of a culvert barrel with respect to a line normal to the roadway centerline is referred to as the barrel skew angle. A culvert aligned normal to the roadway centerline has a skew angle equal to zero. Directions right or left must accompany the skew angle.

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

In cases where the culvert barrel cannot be aligned with the channel flowline, such as when runoff is directed parallel to the roadway embankment to a suitable crossing location, the flow enters the culvert barrel at an angle. The approach angle should be limited to a maximum of 90 degrees. When high velocities exist, inlet losses resulting from turning the flow into the culvert should be considered. If backwater computations are not employed and the approach channel velocity is 6 feet per second or greater, the following equation should be used to estimate the loss. The loss should be added to the other inlet losses in the culvert design computation:

$$H_t = \frac{V_a^2}{2g} S14a$$

Bends: When considering a nonlinear alignment, particular attention should be given to erosion, sedimentation and debris control.

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

If these conditions cannot be met, analysis of bend losses is required. To calculate bend losses refer to Equations 9.10 and 9.11 in Section 9.2.5

Trash Racks: When any of the following conditions are met, trash racks will be required on the entrances to all conduits in areas adjacent to schools, recreational and urban areas, and highways which are accessible to the public.

- Side slopes in channels steeper than 4:1 for concrete, grass and earth linings and 3:1 for riprap linings.
- Conduits smaller than 7 feet in diameter, greater than 100 feet in length and without 12 inches of freeboard at the design flow rate.
- Conduits with energy dissipators or exiting multi-use detention facilities.
- Conduits with sufficient curvature that the opposite end cannot be seen.

A plugging factor of 50 percent will be used on all trash racks, and the velocity through the rack shall be less than or equal to 2.0 feet per second (after plugging factors applied).

For trash racks with velocities less than 3.0 feet per second after the plugging factor has been applied, the losses caused by the trash rack can be ignored in computations. For greater velocities, the loss will be computed and added to the calculated water surface.

9.5.2 Inlets

Culvert inlets are used to transition the flow from a ponded condition upstream of the culvert into the culvert barrel. The provision for a more gradual flow transition will decrease the energy loss and thus create a more efficient inlet condition.

The hydraulic performance of culverts operating under inlet control can be improved by changing the inlet geometry. Typical inlet improvements include beveled edges, tapered sides and sloped drop inlets.

Beveled edges: Provide a decrease in flow contraction losses at the inlet. The entrance loss coefficient is reduced from 0.5 to 0.2, which can increase culvert capacity as much as 20 percent.

Tapered sides: Provide an enlarged face area accomplished by tapering wingwalls. This change in geometry can increase flow capacity as much as 25 to 40 percent over square edged inlets.

Sloped drop: Provide additional head at the throat section. This type of inlet can accommodate over 100 percent greater capacity than a conventional culvert with square edges. The degree of increased capacity depends upon the drop between the face and the throat section.

Section 9.5.7 provides inlet control nomographs for various culvert types which provide the headwater depth required to pass the design discharge. Additional design charts for improved inlets are contained in *Hydraulic Design of Highway Culverts* (FHWA, HDS No.5, September 1985).

9.5.3 Outlets

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

The minimum requirement is to provide a riprap pad, designed in accord with the provisions of this manual, with a cutoff wall provided at the downstream end of the wall. Standard outlets for closed conduits are presented in Section 9.3.5.

9.5.4 Design Procedures

The Culvert design method provides a convenient and organized procedure for designing culverts, considering inlet and outlet control. The first step in the design process is to summarize all known data for the culvert at the top of the Culvert Design Form (Figure 9.3, page 9-19). The next step is to select a preliminary culvert material, shape, size, and entrance type. The user then enters the design flow rate and proceeds with the inlet control calculations.

Inlet Control: The inlet control calculations determine the headwater elevation required to pass the design flow through the selected culvert configuration if the culvert opening is operating under inlet control. The inlet control nomographs in Section 9.5.7 are used in the design process. For the following discussion, refer to the schematic inlet control nomograph shown in Figure 9.8, page 9-23.

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

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

3. If another HW/D scale is required, extend a horizontal line from the first HW/D scale to the desired scale.

4. Multiply HW/D by the culvert height, D, to obtain the required (HW) from the invert of the control section to the energy grade line. HW equals the required headwater depth. If trash racks are included, add the loss associated with the trash rack to the headwater.

Outlet Control: The outlet control calculations result in the headwater elevation required to convey the design discharge through the selected culvert if the culvert is operating under outlet control. The critical depth charts and outlet control nomographs of Section 9.5.8 are used to determine the required headwater for a culvert operating under these conditions. For illustration, refer to the schematic critical depth chart and outlet control nomograph shown in Figures 9.5 and 9.11, respectively.

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

2. Enter the appropriate critical depth chart with the flow rate and determine critical depth. If the computed critical depth is greater than D, the height of the culvert, use D for critical depth.

   Note: The critical depth curves are truncated for convenience when they converge. If an accurate critical depth is required, consult the *Handbook of Hydraulics* by King and Brater, or other hydraulic reference.

3. Calculate (dc + D)/2.

4. Determine the depth form the culvert outlet invert to the hydraulic grade line (ho).

$$h_o = TW \text{ or } (d_c + D)/2, \text{ whichever is larger.} \tag{9.13}$$

5. From Table 9.2 (page 9-39) obtain the appropriate entrance loss coefficient, ke, for the culvert inlet configuration.

6. Determine the losses through the culvert barrel, H, using the outlet control nomograph (Figure 9.10) or appropriate equations if outside the range of the nomograph.

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

$$L_1 = L\ [n_1/n]^2 \tag{9.14}$$

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

   b) Connect the culvert size (point 1) with the culvert length on the appropriate ke scale (point 2). This defines a point on the turning line (point 3).

   c) Extend a line from the discharge (point 4) through the point of the turning line (point 3) to the Head Loss (H) scale. Read H, the energy loss through the culvert, including entrance, friction and outlet losses.

7. Calculate the required outlet control headwater elevation.

$$EL_{ho} = EL_o + H + h_o \tag{9.15}$$

   where EL_o is the invert elevation at the outlet.

8. If the outlet control headwater elevation exceeds the design headwater elevation, a new culvert configuration must be selected and the process repeated.

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

The outlet velocity is calculated as follows:

1. If the controlling headwater is based on inlet control, determine the normal depth and velocity in the culvert barrel. The velocity at normal depth is

assumed to be the outlet velocity. Normal depth for circular and rectangular culverts can be found using Figure 9.8, page 9-23.

2. If the controlling headwater is in outlet control, determine the area of flow at the outlet based on the Barrek geometry and the following:

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

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

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

Repeat the design process until an acceptable culvert configuration is determined.

If outlet control governs and the headwater depth (referenced to the inlet invert) is less than 1.2D, it is possible that the barrel flows partly full through its entire length. In this case, caution should be used in applying the approximate method of setting the downstream elevation based on the greater of tailwater or (dc + D)/2. If an accurate headwater is necessary, backwater calculations should be used to check the result from the approximate method. If the headwater depth falls below 0.75D, the approximate method should not be used.

Culvert design should include a culvert rating curve which displays culvert performance over a range of discharges. Development of rating curves is presented in the following section and an example problem is provided in Section 9.5.8.

9.5.5 Culvert Rating Curves

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

Inlet Control: The inlet control rating curves are developed using the inlet control nomographs in Section 9.5.7. The headwaters corresponding to the series of flow rates are determined and then plotted.

Outlet Control: The outlet control rating curves are developed using the outlet control nomographs in Section 9.5.7. Flows bracketing the design flow are selected. For this range of flow rates the total head loss through the barrel is calculated or read from the outlet control nomographs. The losses are added to the elevation of the hydraulic grade line at the culvert outlet to obtain the headwater.

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

Roadway Overtopping: A rating curve showing the culvert flow as well as the flow across the roadway is a useful analysis tool. The rating curve depicts the sum of the flow through the culvert and the flow across the roadway.

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

1. Select the range of flow rates and determine the corresponding headwater elevations for the culvert flow alone. These flow rates should fall above and below the design discharge and cover the entire range of flows anticipated. Both the inlet and outlet control headwaters should be calculated.

2. Combine the inlet and outlet control rating curves to define a single rating curve for the culvert.

3. When the culvert headwater elevations exceed the roadway crest elevation, overtopping will begin. Calculate the equivalent upstream water surface depth above the roadway (crest of weir) for each selected flow rate. Use these water surface depths and Equation 9.16 to calculate flow rates across the roadway.

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

Using the combined culvert rating curve, it is an easy matter to determine the headwater elevation for any flow rate, or to visualize the performance of the culvert installation over a range of flow rates.

9.5.6 Roadway Overtopping

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

$$Q = K_t C_r L_s (HW_r)^{1.5} \qquad \textbf{(9.16)}$$

The charts in Figure 9.5 indicate how to evaluate the correction factors K_t and C_r.

If the elevation of the roadway crest varies, for instance where the crest is defined by a sag vertical curve, the vertical curve can be approximated as a series of horizontal segments. The flow over each is calculated separately, and the total flow across the roadway is the sum of the incremental flows for each segment.

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

9.5.7 Design Aids

Figures 9.3 through 9.22 and Table 9-2 are provided to facilitate completion of the Culvert Design Form and the development of culvert rating curves.

CULVERT DESIGN FORM

Project: ____________________
____

Station: ____________
Sheet ______ of ______

Designer/Date: ________ / ________
Reviewer/Date: ________ / ________

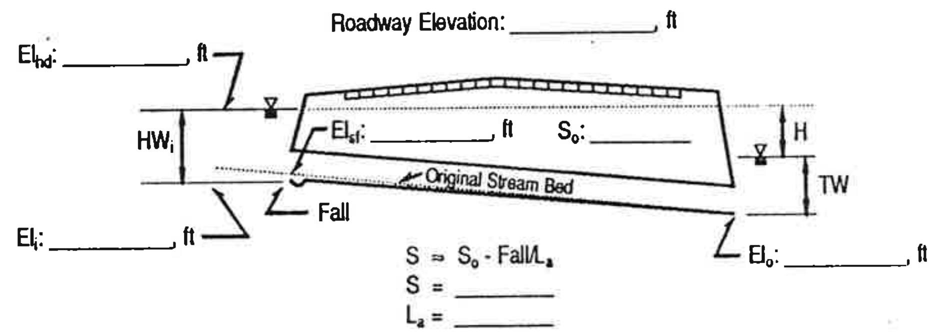
Hydrological Data

See Add'l shts

- ☐ Method: ____________________
- ☐ Drainage Area: ________ ☐ Stream Slope: ______
- ☐ Channel Shape: ____________________
- ☐ Routing: ____________ ☐ Other: ____________

Design Flows/Tailwater

R.I., years	Flow, cfs	TW, ft



Culvert Description: Material - Shape - Size - Entrance	Total Flow Q, cfs	Flow Per Barrel Q/N (1)	Headwater Calculations: Inlet Control: HW_i/D (2)	HW_i	Fall (3)	EL_{hi} (4)	Outlet Control: TW (5)	d_c	$\frac{d_c + D}{2}$	h_o (6)	k_e	H (7)	EL_{ho} (8)	Control Headwater Elevation	Outlet Velocity	Comments

Technical Footnotes:

(1) Use Q/NB for box culverts.
(2) $HW_i/D = HW/D$ or HW_i/D from design charts
(3) $FALL = HW_i - (EL_{hd} - EL_{sf})$; Fall is zero for culverts on grade.
(4) $EL_{hi} = HW_i + EL_i$ (invert of inlet control section)
(5) TW based on downstream control or flow depth in channel.
(6) h_o = TW or $(d_c + D)/2$ (whichever is greater).
(7) $H = \left[1 + k_e + (29n^2L)/R^{1.33}\right] V^2/2g$
(8) $EL_{ho} = EL_o + H + h_o$

Subscript Definitions:

- a. approximate
- f. culvert fact
- hd. Design Headwater
- hj. Headwater in inlet control
- he. Headwater in outlet control
- i. inlet control section
- o. Outlet
- sf. Streambed at culvert face
- TW. Tailwater

Comments / Discussion:

Culvert Barrel Selected:

Size: ____________
Shape: ____________
Material: ________ n: ______
Entrance: ____________

Figure 9.3
Culvert Design Form

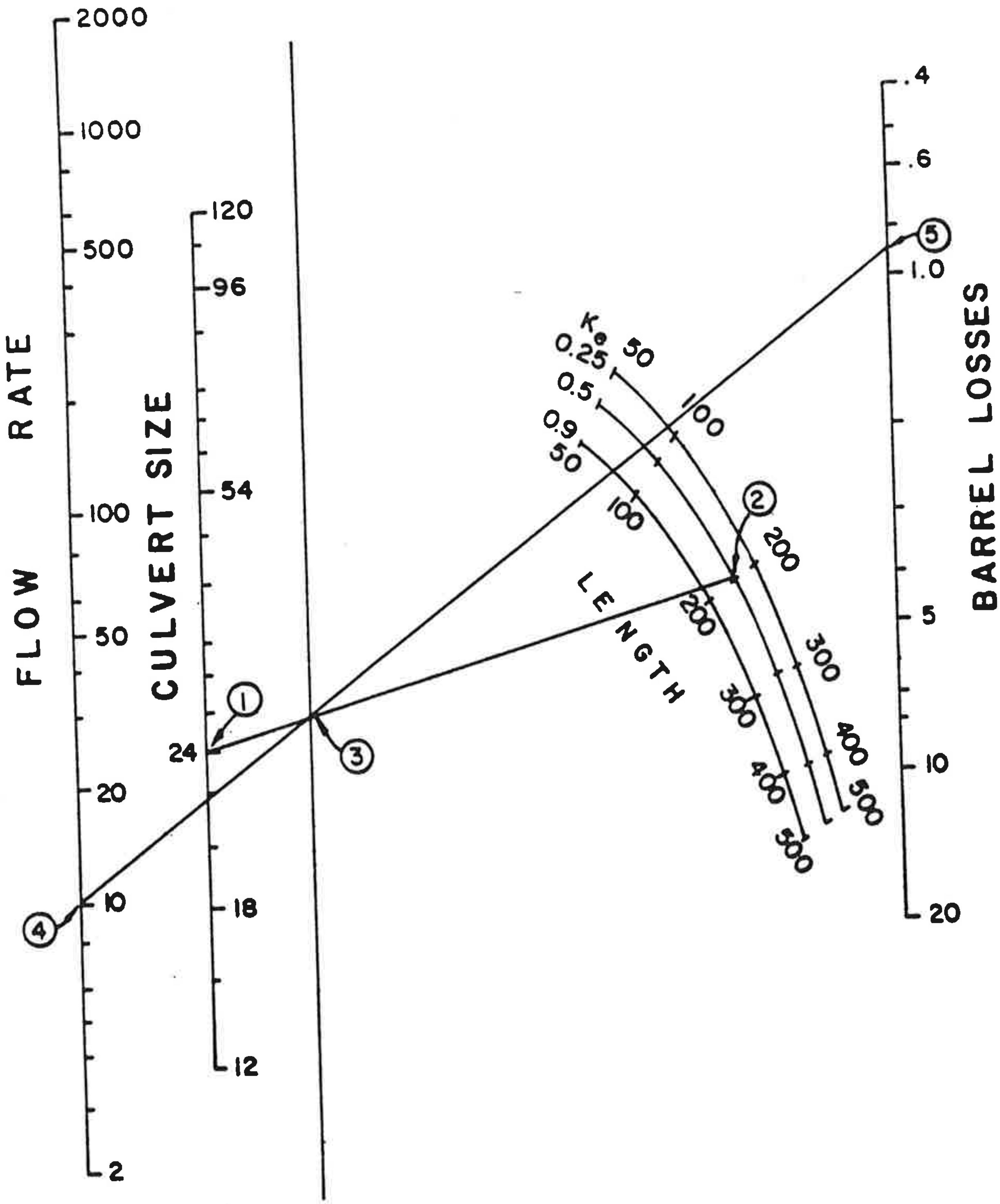


Figure 9.4
Outlet Control Nomograph (Schematic)

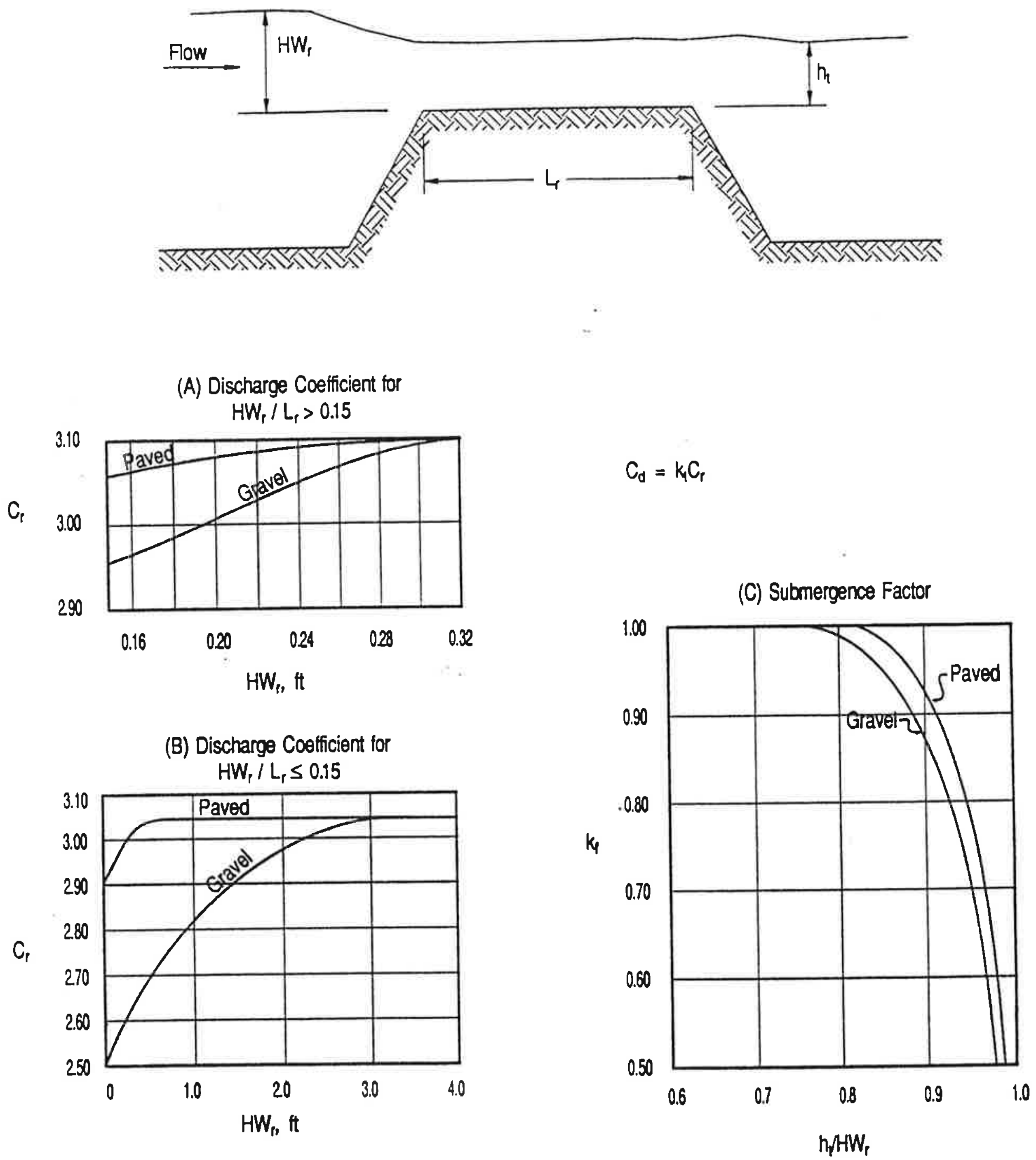


Figure 9.5
Discharge Coefficient
and Submergence Factor for Roadway Overtopping

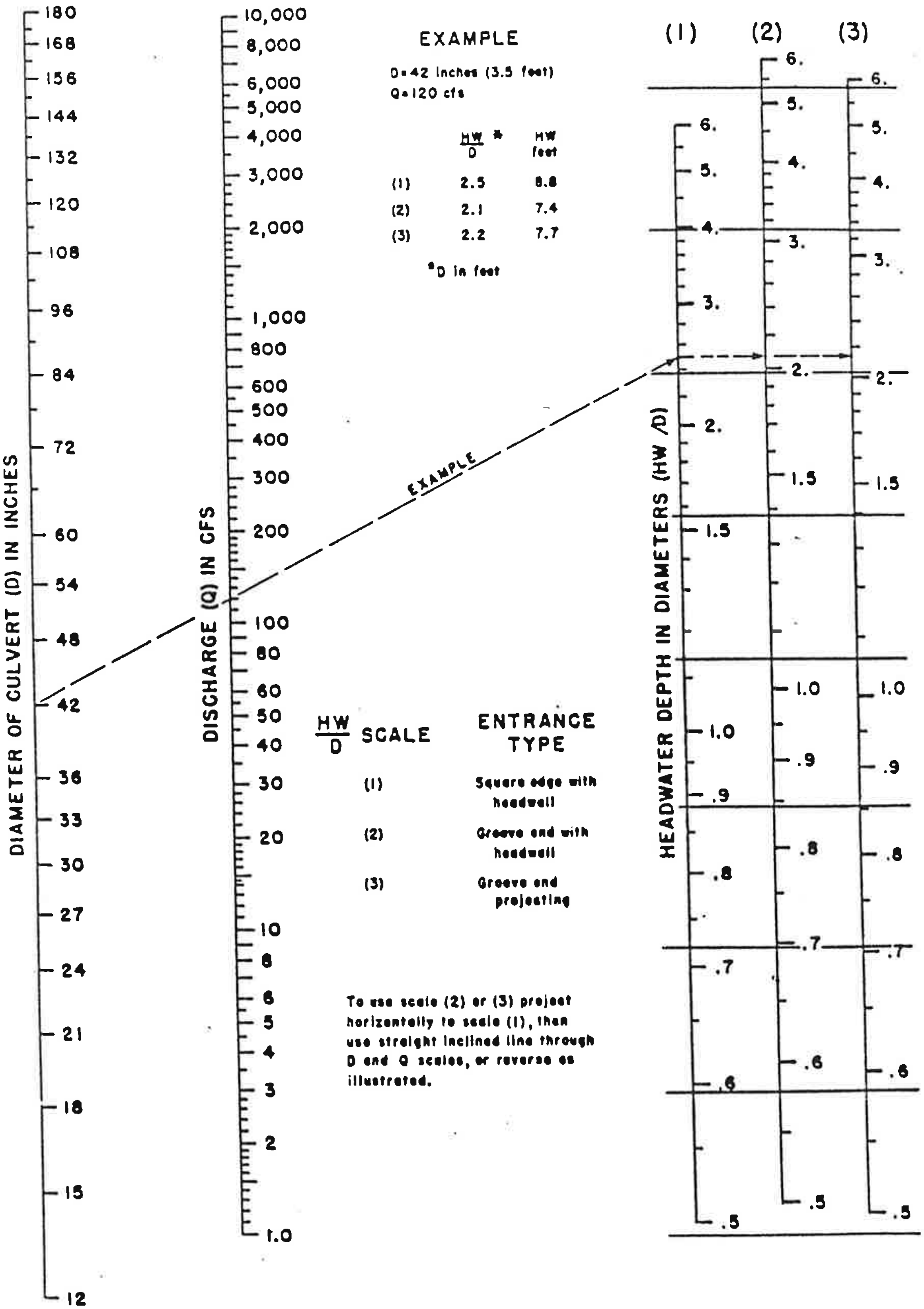


Figure 9.6
Headwater Depth for Concrete Pipe Culverts with Inlet Control
(FHWA, HDS-5, 1985)

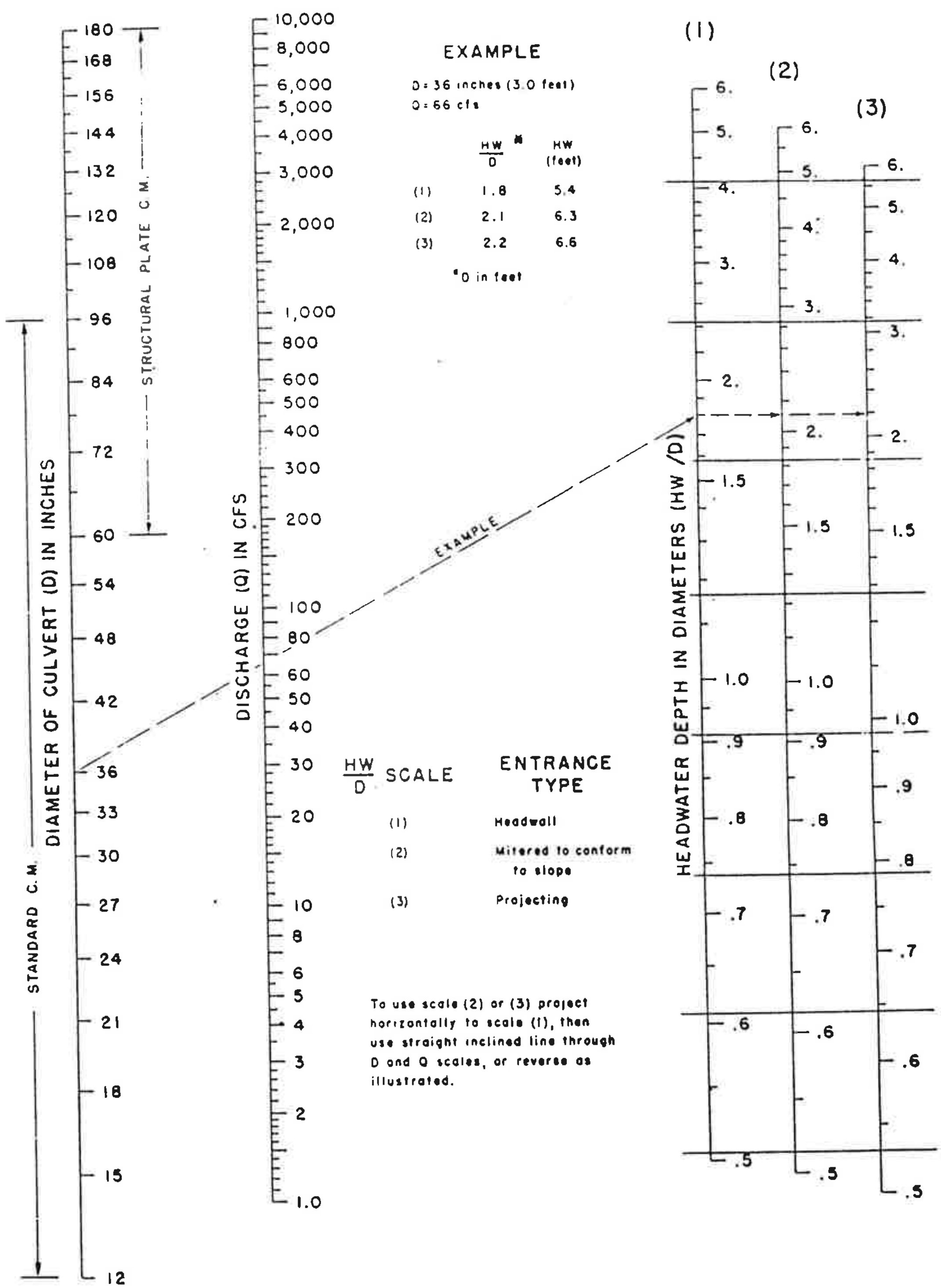


Figure 9.7
Headwater Depth for CMP Culverts
with Inlet Control
(FHWA, HDS-5, 1985)

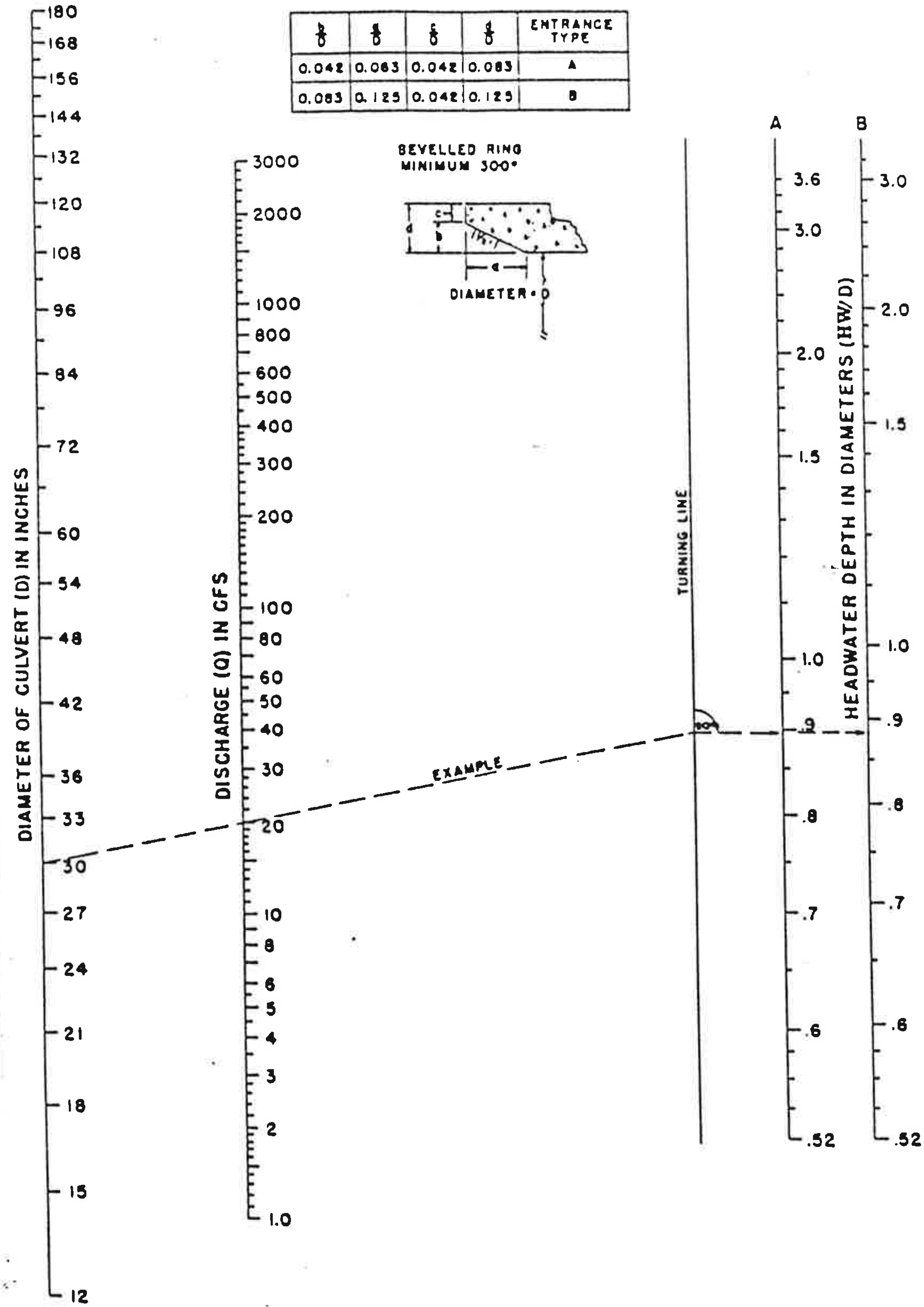


Figure 9.8
Headwater Depth for Circular Pipe Culverts with Beveled Ring Inlet Control
(FHWA, HDS-5, 1985)

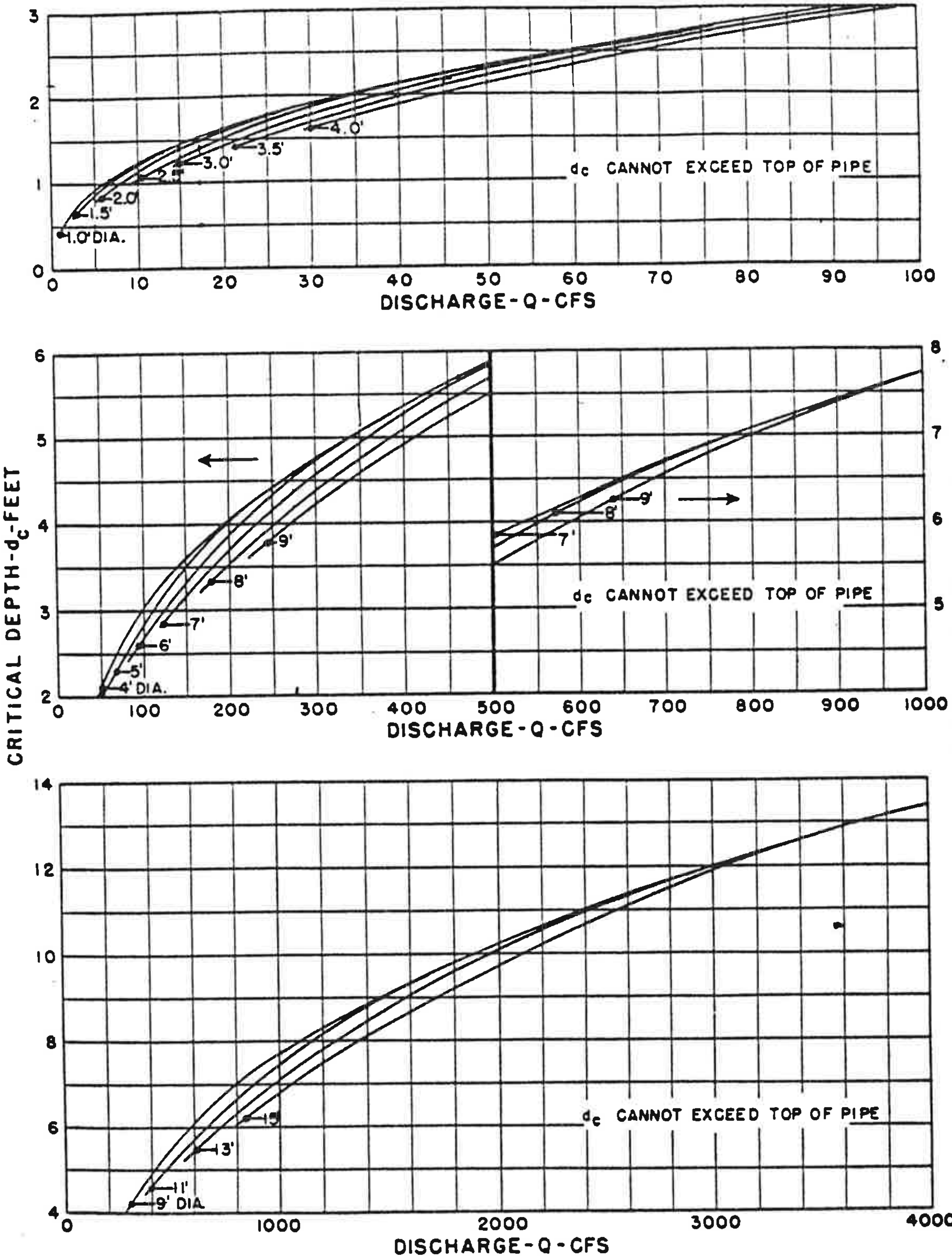


Figure 9.9
Critical Depth for Circular Pipe
(FHWA, HDS-5, 1985)

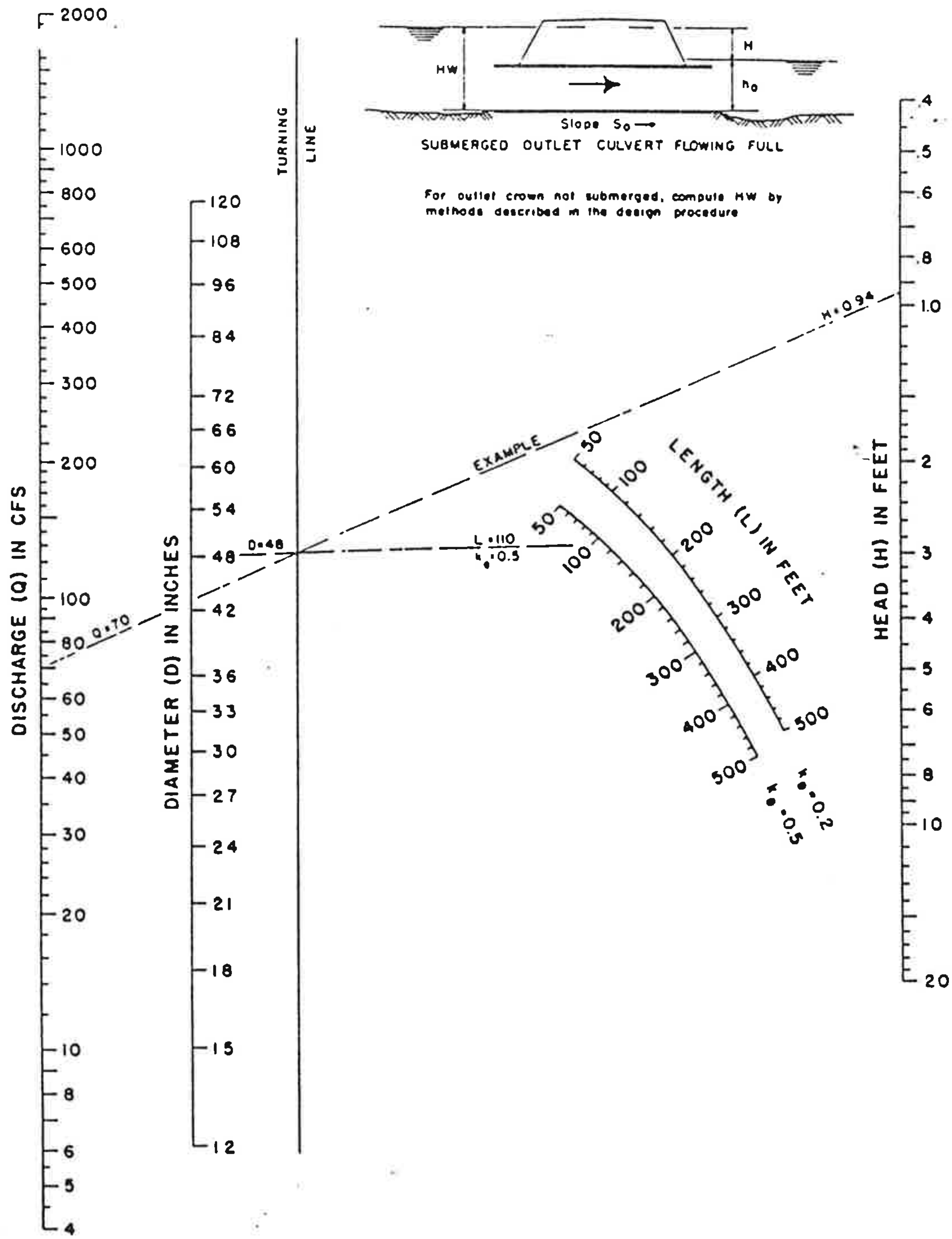


Figure 9.10
Head for Concrete Pipe Culverts Flowing Full
(FHWA, HDS-5, 1985)

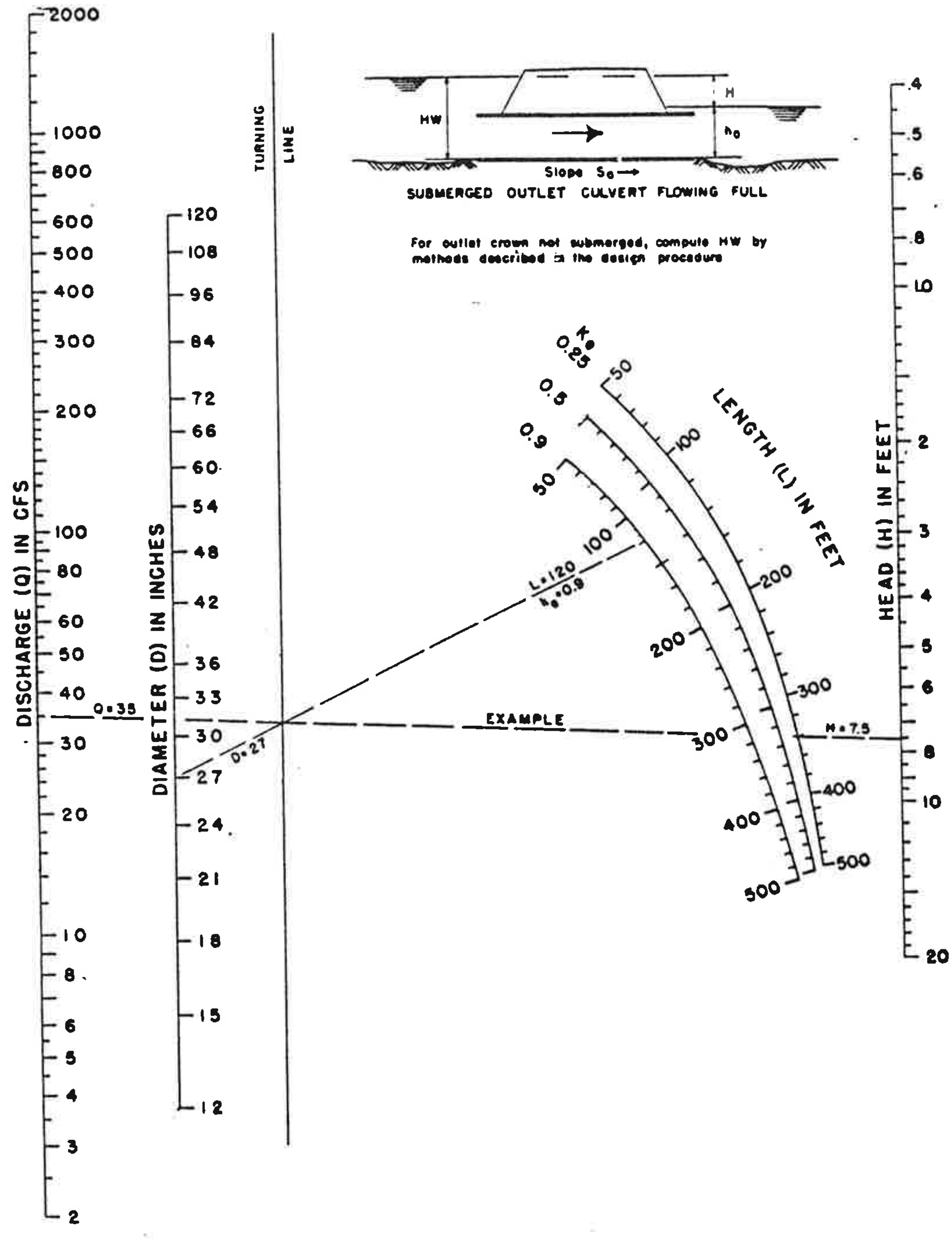


Figure 9.11
Head for Standard C.M. Pipe Culverts Flowing Full
$n = 0.024$
(FHWA, HDS-5, 1985)

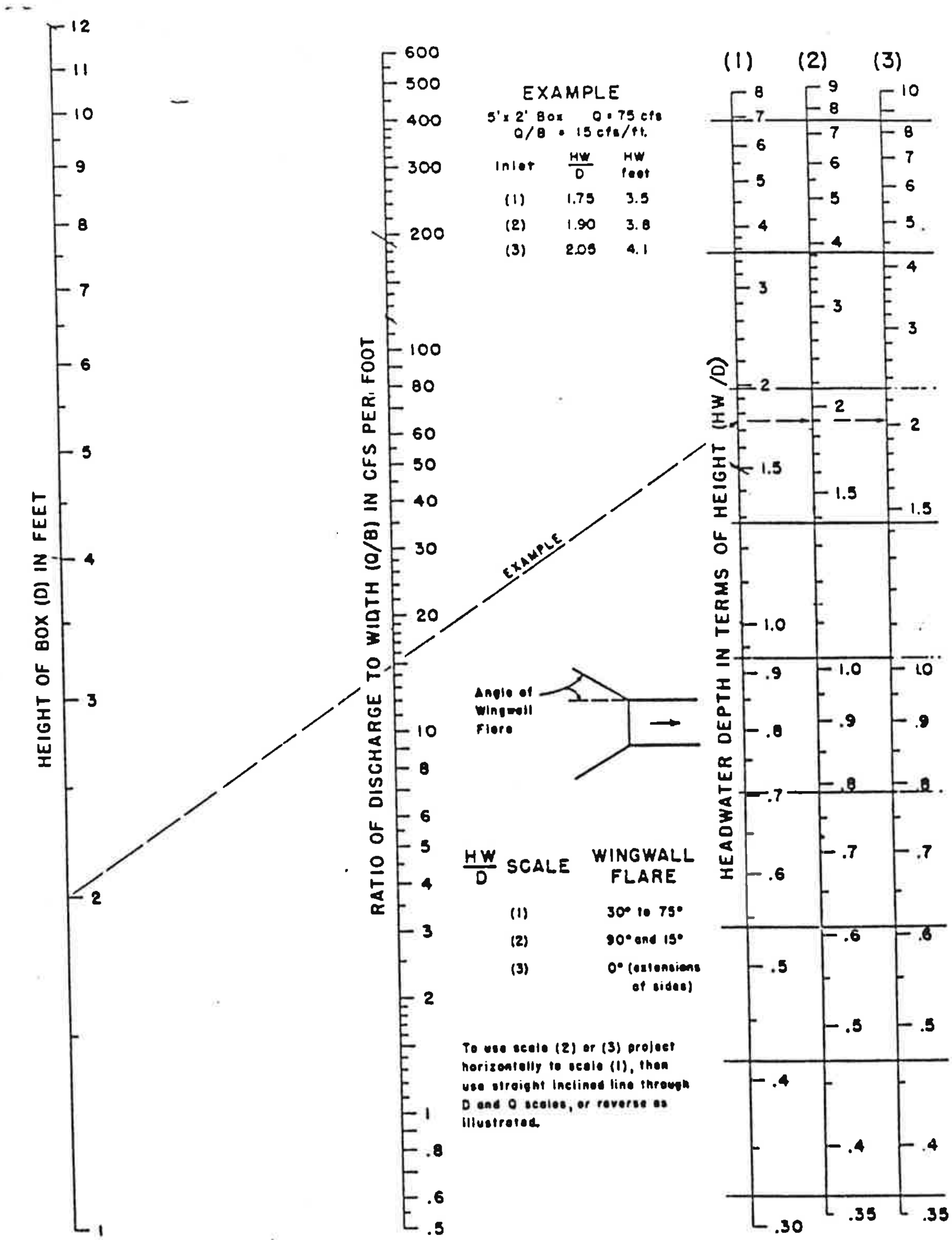


Figure 9.12
Headwater Depth for Box Culverts with Inlet Control
(FHWA, HDS-5, 1985)

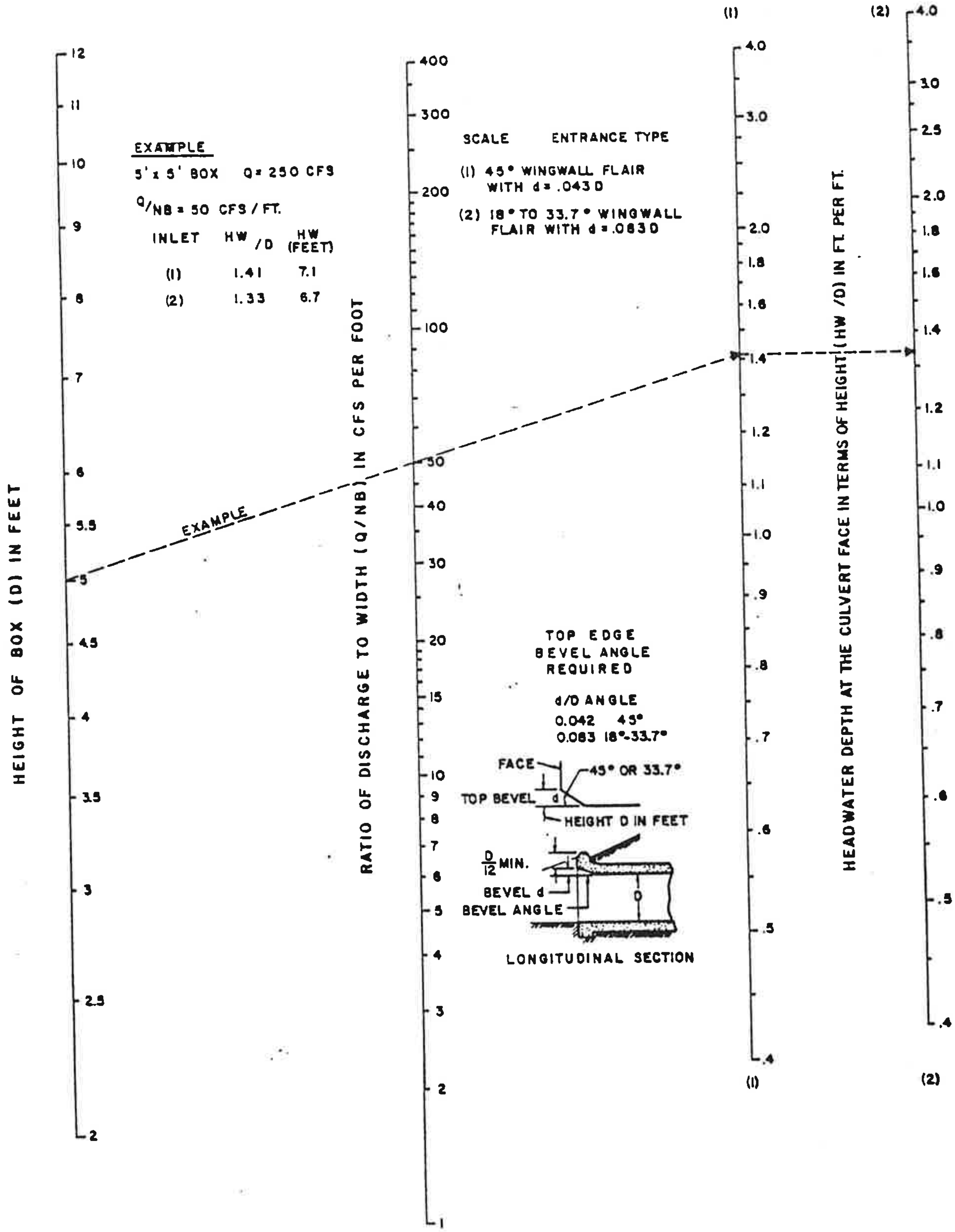


Figure 9.13
Headwater Depth for a Rectangular Box Culvert with Inlet Control, Flared Wingwalls (18 to 33.7 Degrees and 45 Degrees), and Beveled Edge at the Top of the Inlet
(FHWA, HDS-5, 1985)

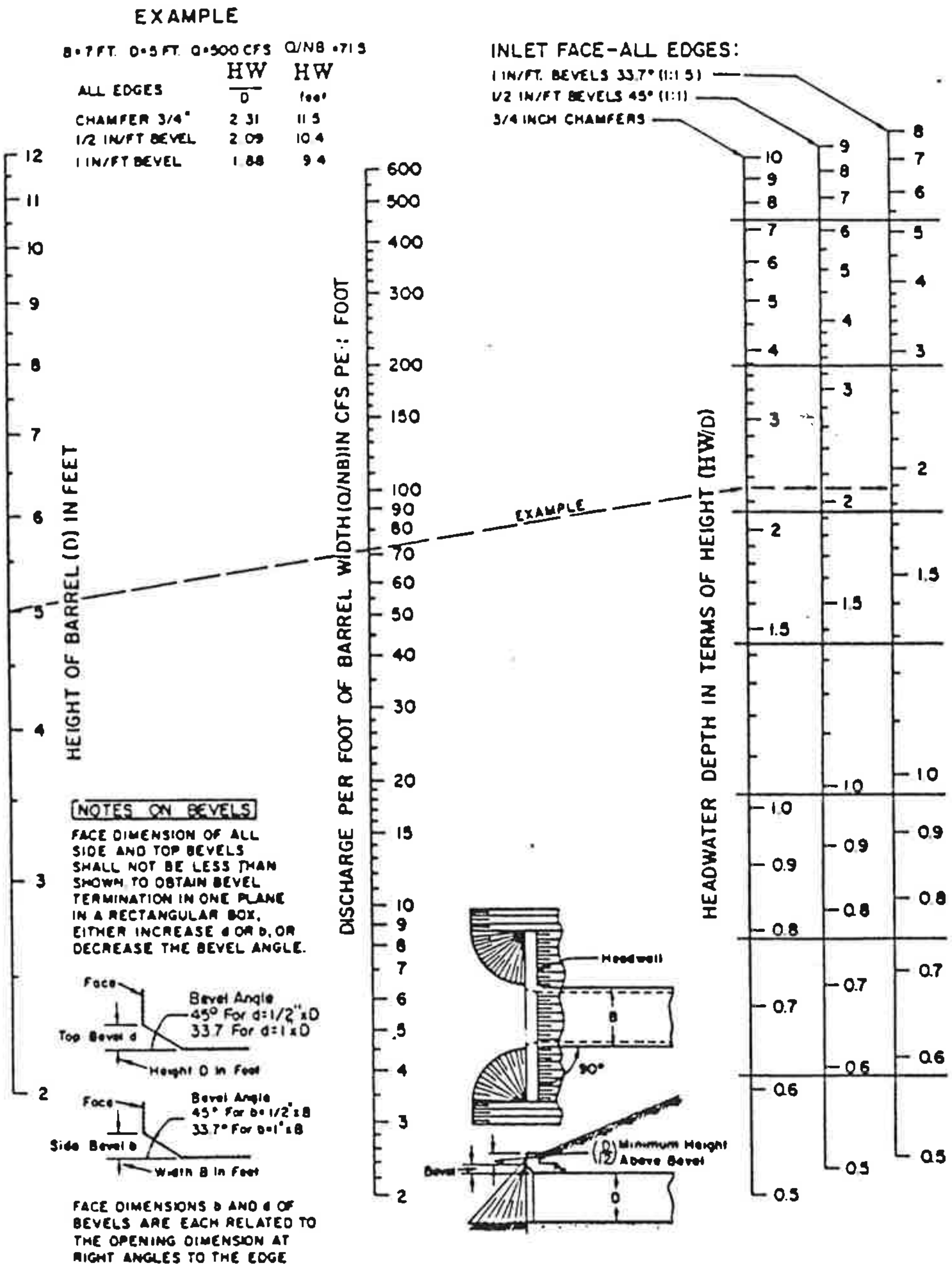
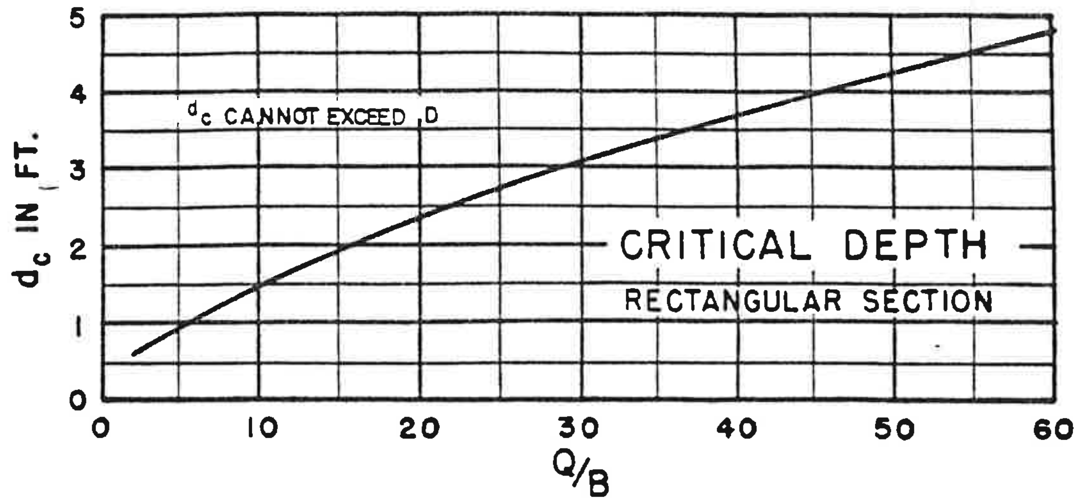


Figure 9.14
Headwater Depth for Inlet Control Rectangular Box Culverts
90 ° Headwall—Chamfered or Beveled Inlet Edges
(FHWA, HDS-5, 1985)



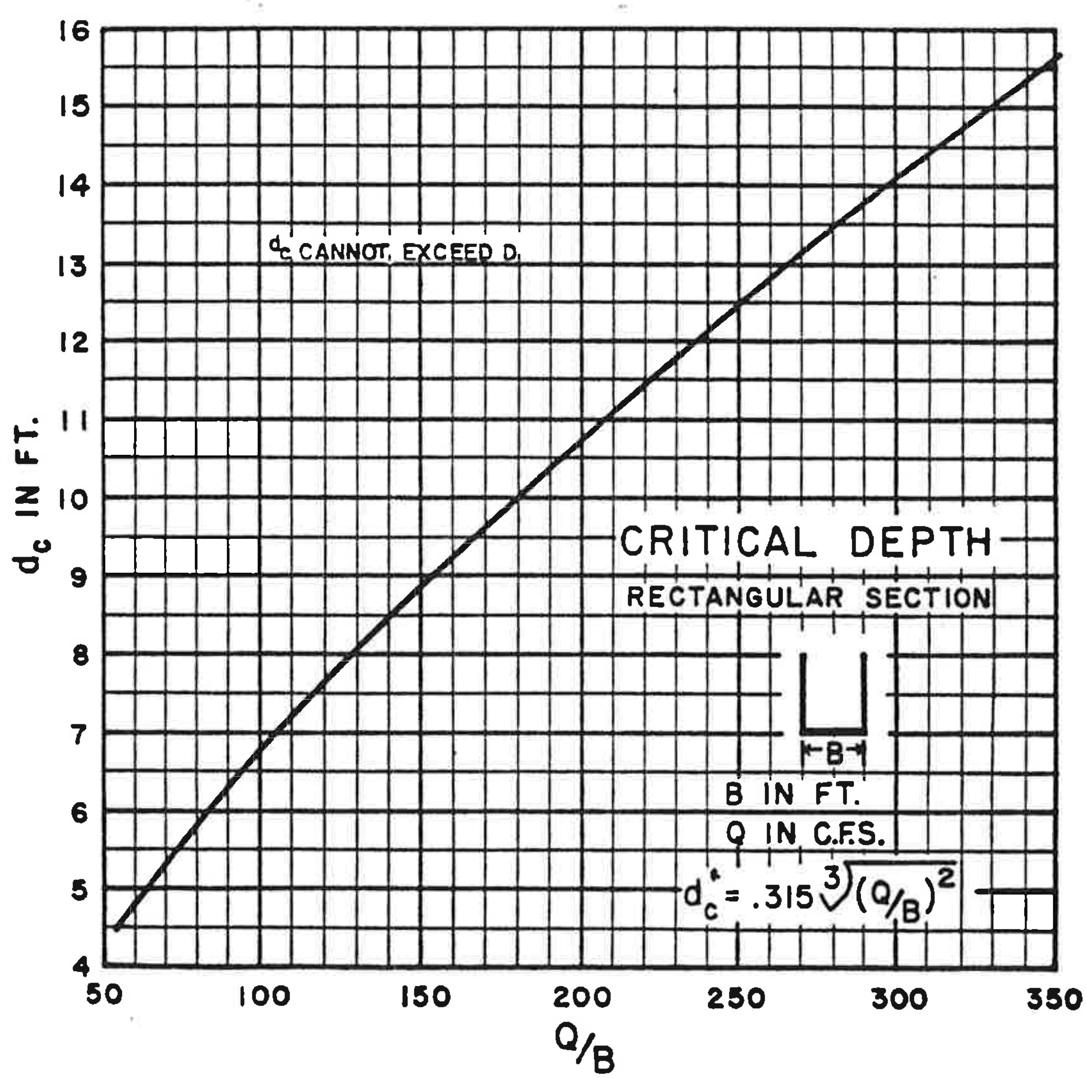


Figure 9.15
Critical Depth Rectangular Section
(FHWA, HDS-5, 1985)

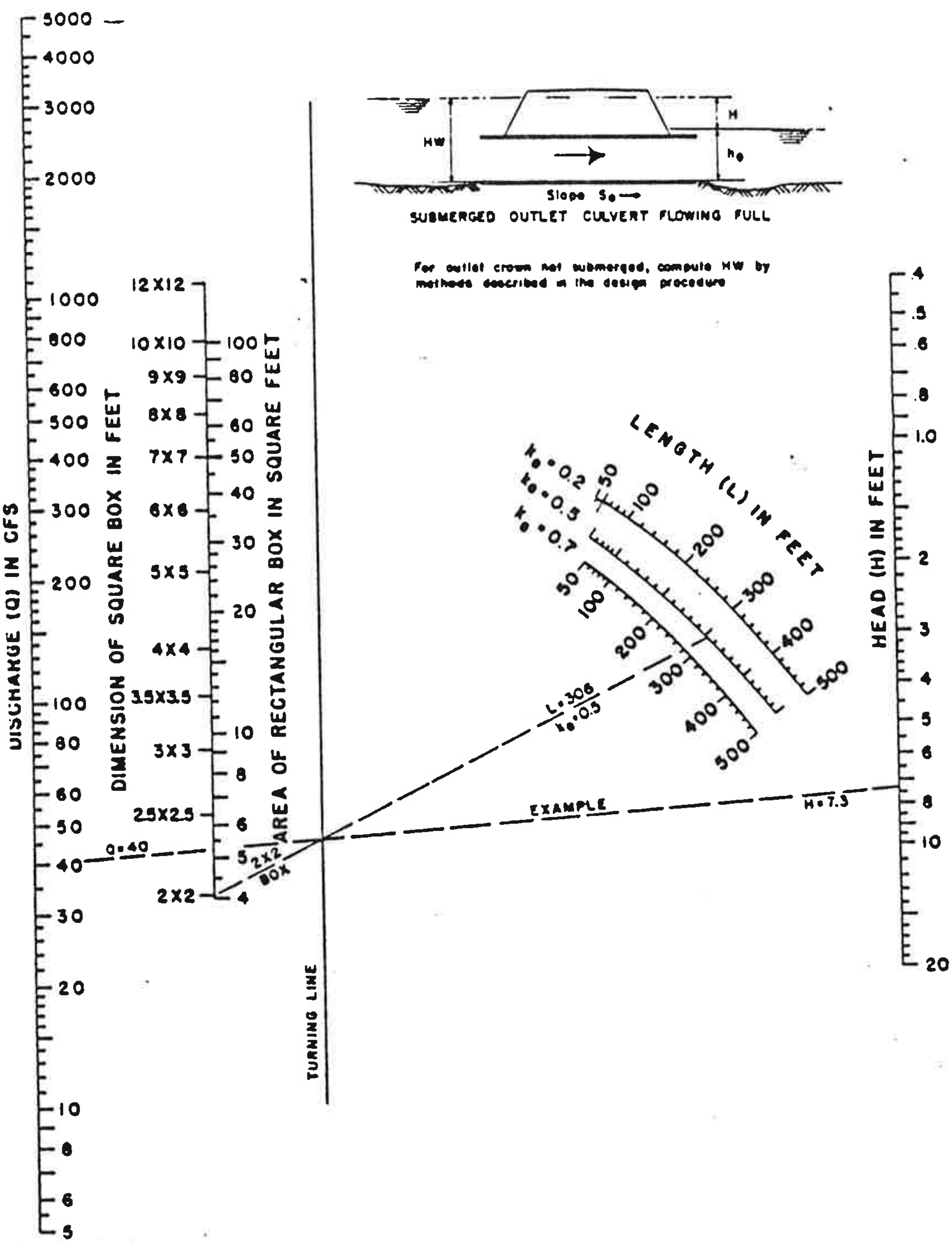


Figure 9.16
Head for Concrete Box Culverts Flowing Full
n=0.012
(FHWA, HDS-5, 1985)

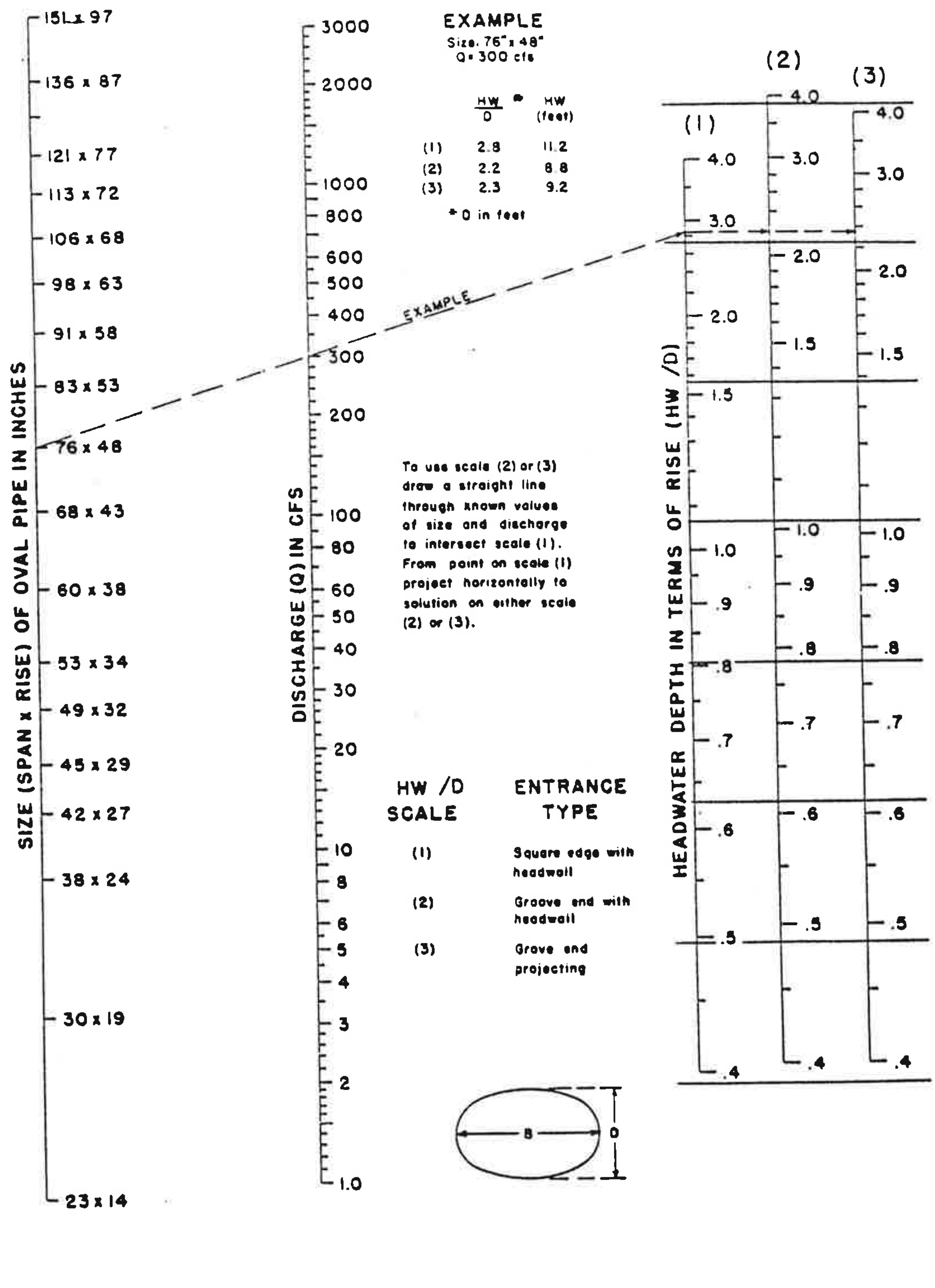


Figure 9.17
Headwater Depth for Oval Concrete Pipe Culverts
Long Axis Horizontal with Inlet Control
(FHWA, HDS-5, 1985)

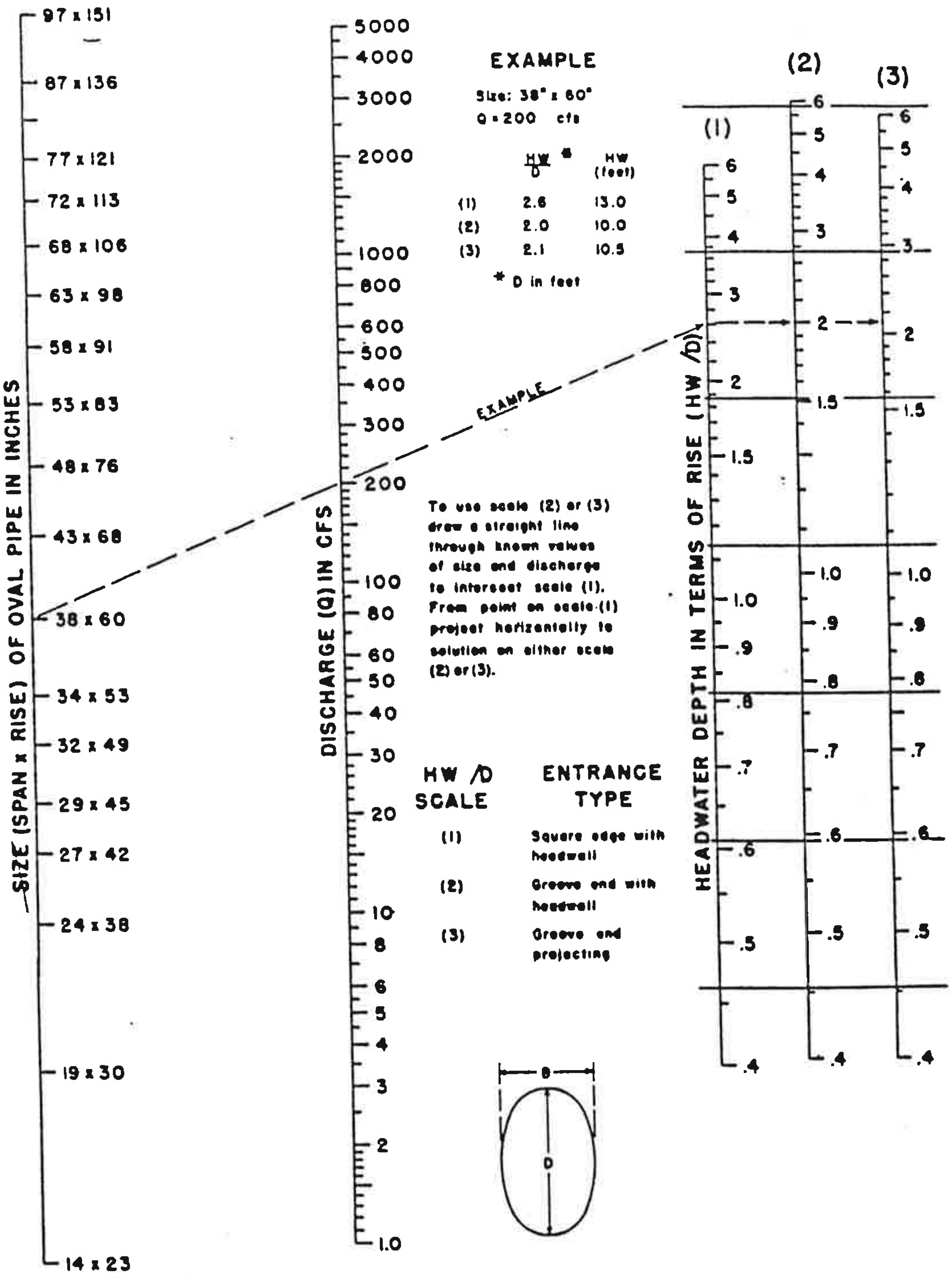


Figure 9.18
Headwater Depth for Oval Concrete Pipe Culverts
Long Axis Vertical with Inlet Control
(FHWA, HDS-5, 1985)

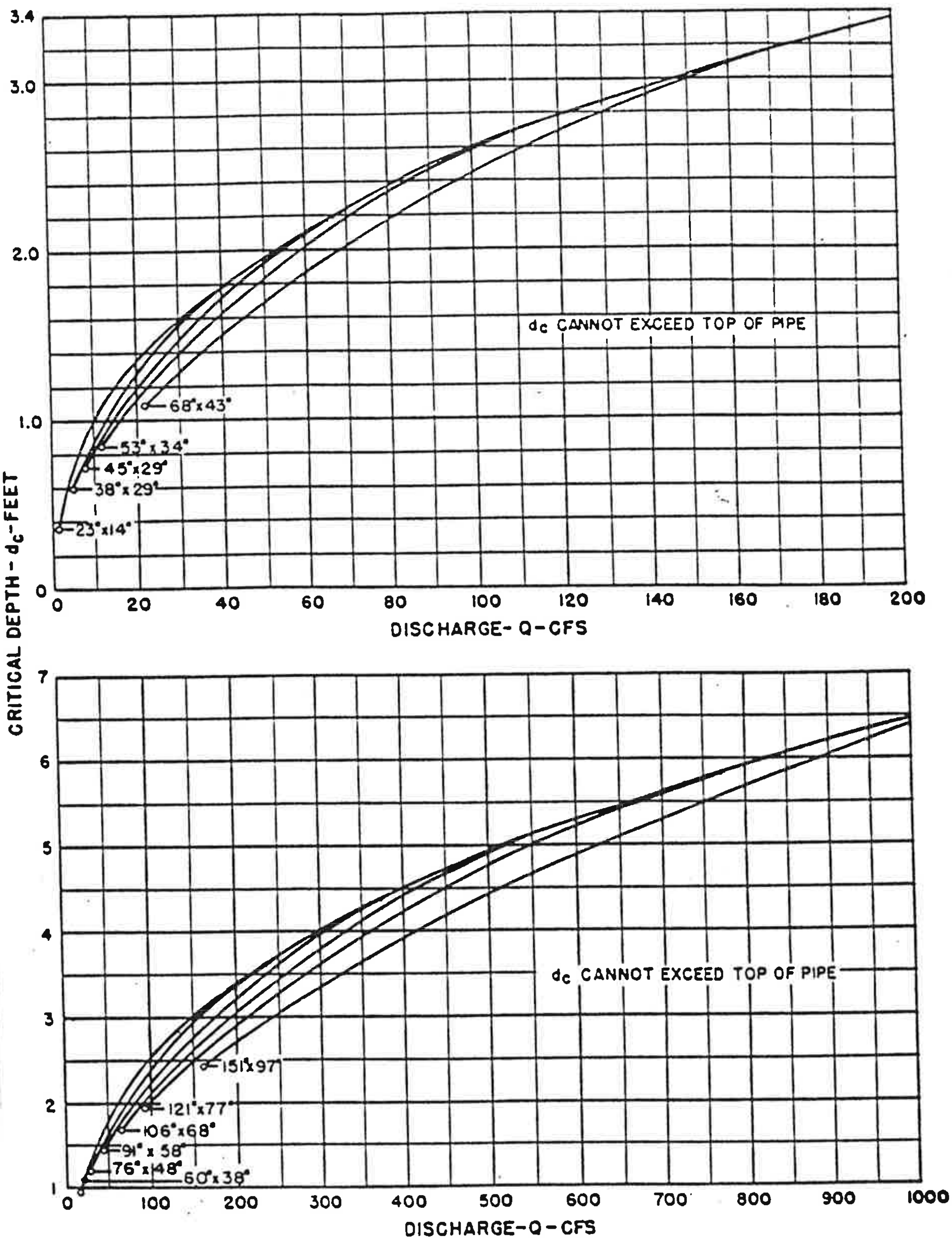


Figure 9.19
Critical Depth for an Oval Concrete Pipe—Long Axis Horizontal
(FHWA, HDS-5, 1985)

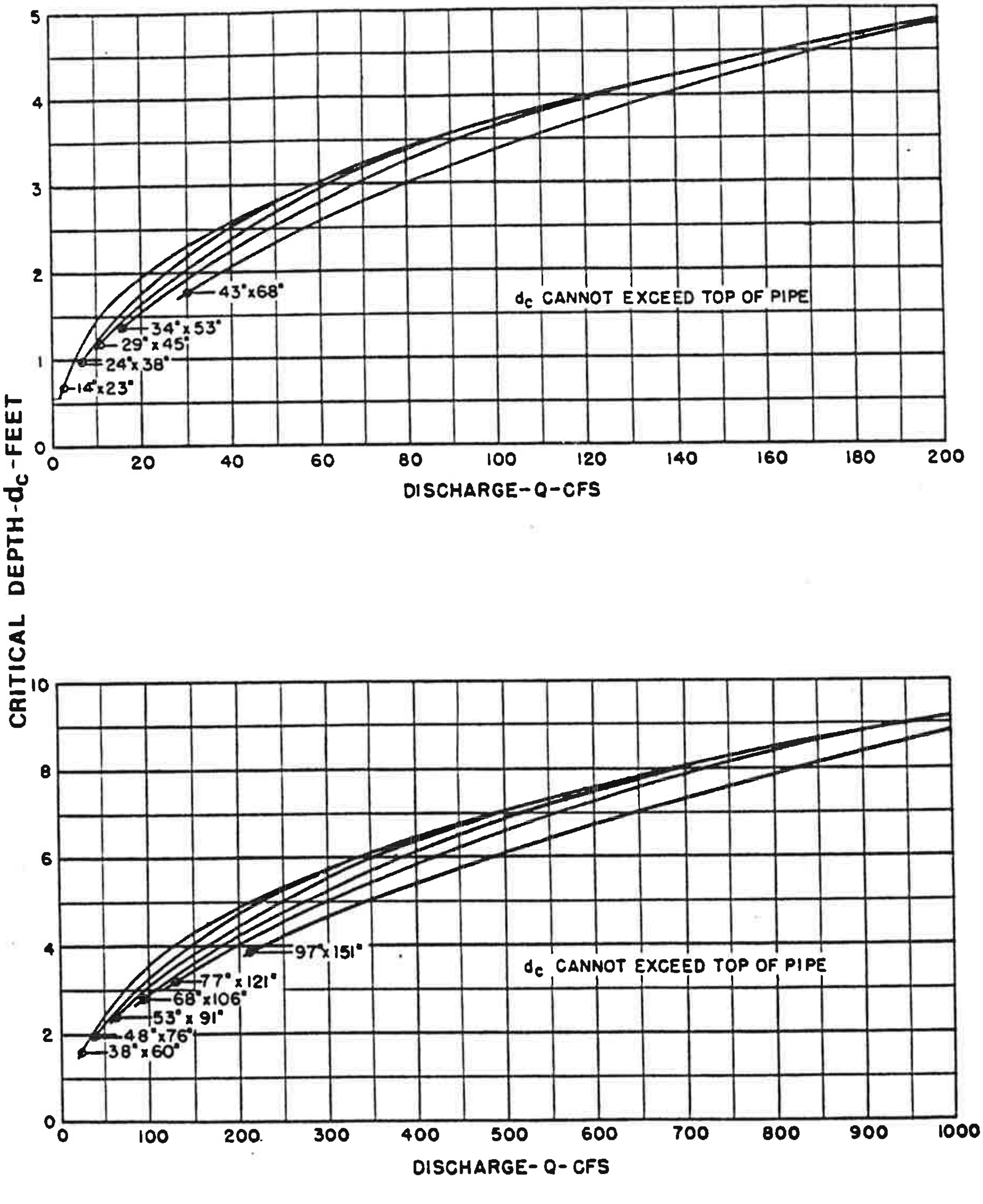


Figure 9.20
Critical Depth for an Oval Concrete Pipe—Long Axis Vertical
(FHWA, HDS-5, 1985)

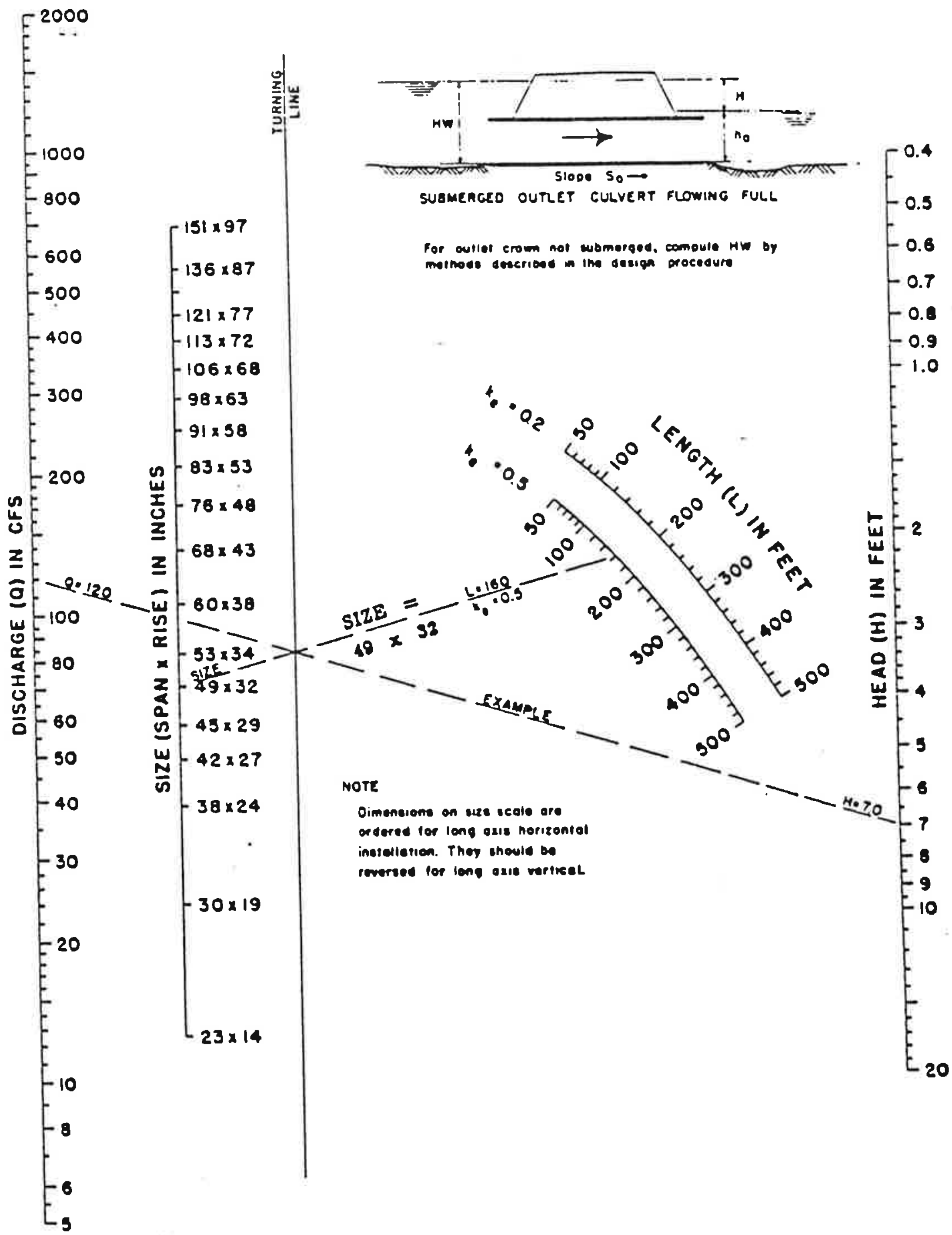


Figure 9.21
Head for Oval Concrete Pipe Culverts
Long Axis Horizontal or Vertical—Flowing Full
n=0.012
(FHWA, HDS-5, 1985)

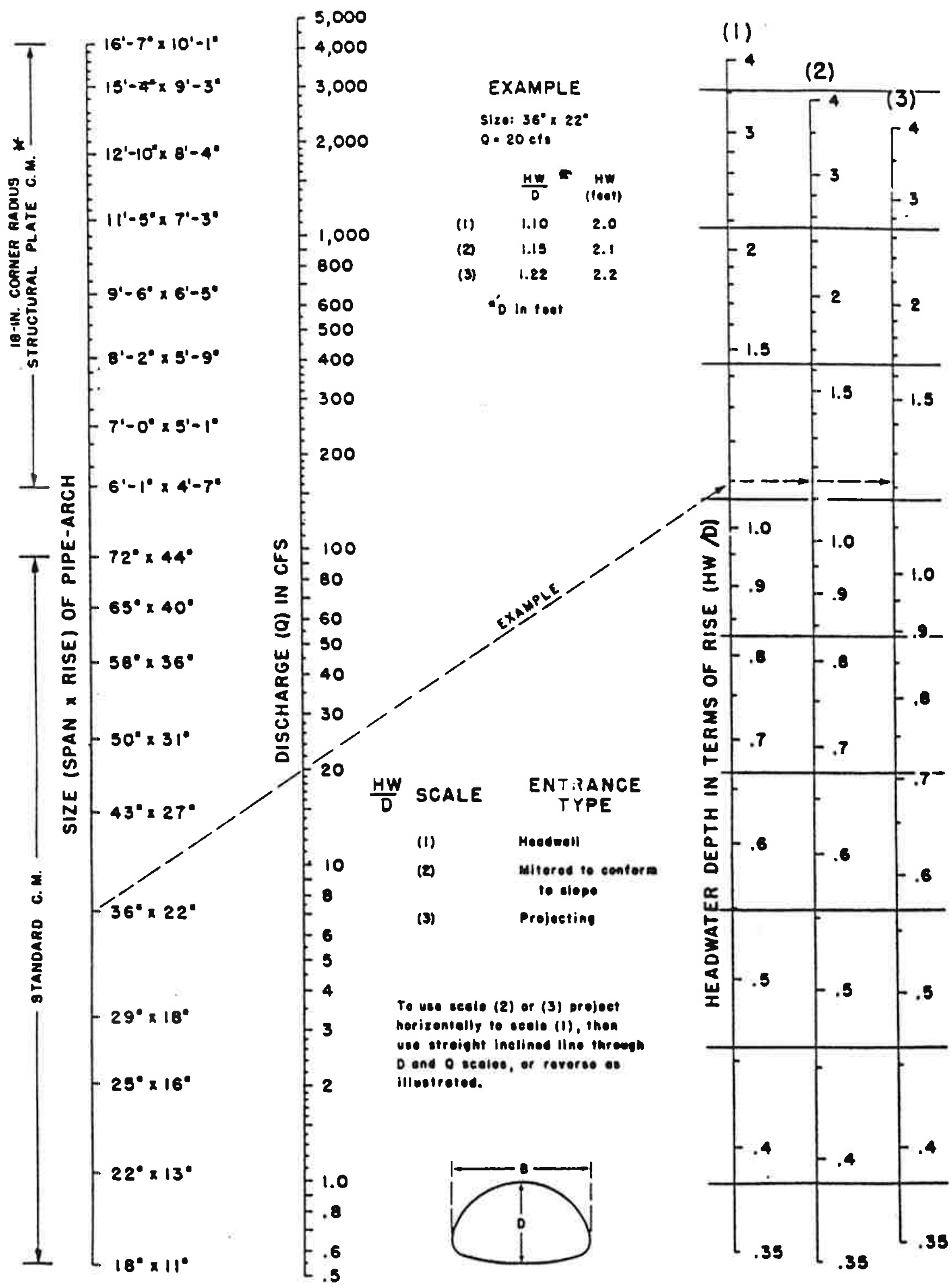


Figure 9.22
Headwater Depth for C.M. Pipe-Arch Culvert with Inlet Control
(FHWA, HDS-5, 1985)

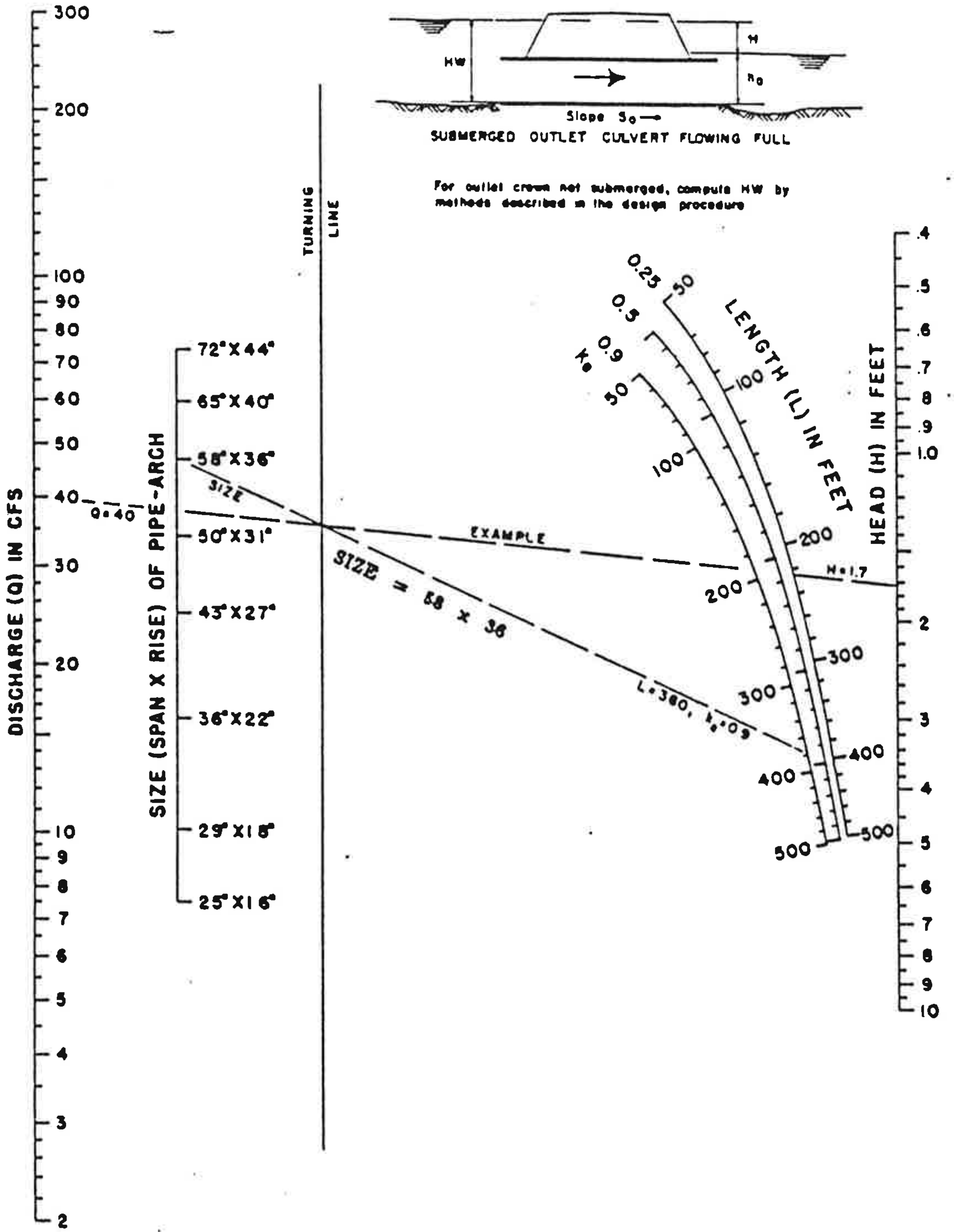


Figure 9.23
Head for Standard C.M. Pipe-Arch Culverts Flowing Full
n=0.024
(FHWA, HDS-5, 1985)

Table 9.2
Entrance Loss Coefficients
Outlet Control, Full or Partly Full Entrance Head Loss

Type of Structure and Design of Entrance	Coefficient k_e
Pipe, Concrete	
Projecting from fill, socket end (groove-end)	0.2
Projecting from fill, square cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove-end)	0.2
Square-edge	0.5
Rounded (radius = 1/12D)	0.2
Mitered to conform to fill slope	0.7
End-Section conforming to fill slope	0.5
Beveled edges, 33.7 or 45° bevels	0.2
Side-or slope-tapered inlet	0.2
Pipe, or Pipe-Arch, Corrugated Metal	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls square-edge	0.5
Mitered to conform to fill slope, paved or unpaved slope	0.7
End-Section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side- or slope-tapered inlet	0.2
Box, Reinforced Concrete	
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of 1/12 barrel dimension, or beveled edges on 3 sides	0.2
Wingwalls at 30° to 75° to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius to 1/12 barrel dimension, or beveled top edge	0.2
Wingwall at 10° to 25° to barrel	
Square-edged at crown	0.5
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7
Side- or slope-tapered inlet	0.2

9.6 LIST OF VARIABLES

A	=	Cross-sectional area
C_r	=	Length of the roadway crest along the roadway, feet
D	=	Depth of flow in feet
h	=	Head loss
HW_r	=	Flow depth above the roadway in feet
K_E	=	Submergence factor
L	=	Length in feet
L_s	=	Length of the roadway crest along the roadway, feet
n	=	Manning's Roughness coefficient
R	=	Hydraulic radius
S_f	=	Friction slope in feet per foot
S_o	=	Longitudinal slope in feet per foot
TW	=	Tailwater
V	=	Velocity in feet per second
Θ	=	Inlet skew angle

9.7 REFERENCES

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