STORMWATER MANAGEMENT

CASPER, WYOMING

DESIGN MANUAL

Section 6

Culvert Design

Section 6

Culvert Design

The function of a drainage culvert is to pass the design storm flow under a roadway or railroad without causing excessive backwater and without creating excessive downstream velocities.

6.10 Design Criteria

The design flow shall be determined by the Rational Method or the Colorado Urban Hydrograph Procedure as set forth in Section 2.

Design Frequency

Culverts shall be designed to pass the 10-year runoff with a two-foot freeboard and no flow over the roadway. The drainage system shall accommodate a 100-year flood, including provision for limited overflows at bridges and culverts without loss of life or major property damage. At the option of the appropriate agency the designer may design culverts as storm sewers with a 10-year design frequency. Under this option the proposed culvert shall be placed on line and grade to permit connection to the future storm sewer. The major storm impacts shall be investigated for both the culvert configuration and the proposed storm sewer configuration. The major storm analysis of the storm sewer shall include upstream and downstream reaches sufficient to show that provisions for the major storm can be made when the storm sewer is constructed.

In areas where an official floodway exists, increases in the 100-year water-surface elevation shall not be greater than 1 foot above the natural 100-year water-surface elevation.

Culvert Discharge Velocities

The velocity of discharge from culverts should be limited as shown in Table 6-1. Consideration must be given to the effect of high velocities, eddies or other turbulence on the natural channel, downstream property and roadway embankment.

TABLE 6-1

Culvert Discharge Velocity Limitations

Downstream Condition	Maximum Allowable Discharge Velocity (fps)
Earth	6 fps
Seeded or Sodded Earth	8 fps
Paved or Riprap Apron	15 fps

It is recommended that a minimum velocity of 2.5 feet per second at the design flow be maintained in all drainage structures to prevent siltation. Where doubt exists concerning silt or scour, protection commensurate with the value of the structure and surrounding property shall be installed to insure that damage to or failure of the structure will not occcur.

6.20 Culvert Types

Culverts shall be selected based on hydraulic principles, economy of size and shape, and with a resulting headwater depth which will not cause damage to adjacent property for the 100-year storm. It is essential to the proper design of a culvert that the conditions under which the culvert will operate are known. Five types of operating conditons are discussed below.

Type I Flowing Part Full with Outlet Control and Tailwater Depth Below Critical Depth.

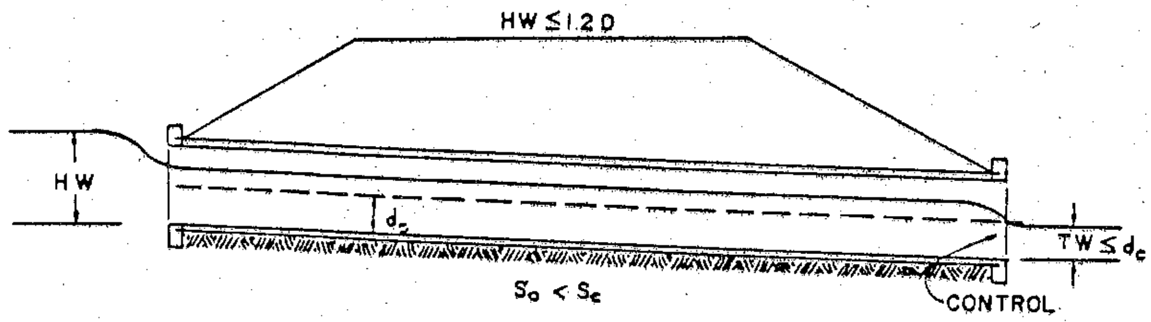


Figure 6-1

CONDITIONS

The entrance is unsubmerged (HW < 1.2D), the slope at design discharge is sub-critical ($S_o < S_c$), and the tailwater is below critical depth ($TW \leq d_c$).

The above condition is a common occurrence where the natural channels are on flat grades and have wide, flat floodplains. The control is critical depth at the outlet.

In culvert design, it is generally considered that the headwater pool maintains a constant level during the design storm. If this level does not submerge the culvert inlet, the culvert flows part full.

If critical flow occurs at the outlet the culvert is said to have "Outlet Control". A culvert flowing part full with outlet control will require a depth of flow in the barrel of the culvert greater than critical depth while passing through critical depth at the outlet.

The capacity of a culvert flowing part full with outlet control and tailwater depth below critical depth is governed by the following equation when the approach velocity is considered zero.

$$HW = d_c + \frac{V_c^2}{2g} + h_e + h_f - S_oL \qquad (6\text{-}1)$$

where:

HW = headwater depth above the invert of the upstream end of the culvert in feet. Headwater must be equal to or less than 1.2D or entrance is submerged and Type IV operation will result.

d_c = critical depth of flow, in feet (Figure 6-25 or 6-26);

D = diameter of pipe or height of box, in feet;

V_c = critical velocity, in feet per second, occurring at critical depth;

h_e = entrance head loss, in feet, where:

$$h_e = k_e \frac{V_c^2}{2g}; \qquad (6\text{-}2)$$

k_e = entrance loss coefficient (Table 6-2, page 6-54);

h_f = friction head loss, in feet = S_fL;

S_f = friction slope or slope that will produce uniform flow. For Type I operation the friction slope is based upon 1.1 d_c;

S_o = slope of culvert, in feet per foot; and

L = length of culvert, in feet.

Type II Flowing Part Full with Outlet Control and Tailwater Depth Above Critical Depth.

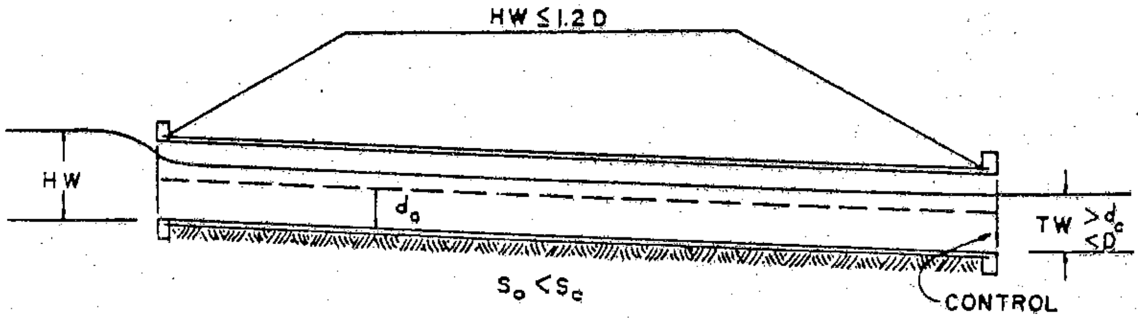


Figure 6-2

CONDITIONS

The entrance is unsubmerged ($HW < 1.2D$), the slope at design discharge is subcritical ($S_o < S_c$), and the tailwater is above critical depth ($TW > d_c$).

The above condition is a common occurrence where the channel is deep, narrow, and well defined.

If the headwater pool elevation does not submerge the culvert inlet, the slope at design discharge is subcritical, and the tailwater depth is above critical depth, the control occurs at the outlet. The capacity of the culvert is governed by the following equation when the approach velocity is considered zero.

$$HW = TW + \frac{V_{TW}^2}{2g} + h_e + h_f - S_oL \qquad (6\text{-}3)$$

where:

HW = headwater depth above the invert of the upstream end of the culvert, in feet. Headwater depth must be equal to or less than 1.2D or entrance is submerged and Type IV operation will result;

TW = tailwater depth above the invert of the downstream end of the culvert, in feet;

V_{TW} = culvert discharge velocity, in feet per second, at tailwater depth;

h_e = entrance head loss, in feet, where;

$$h_e = k_e \frac{V_{TW}^2}{2g}; \qquad (6\text{-}4)$$

k_e = entrance loss coefficient (Table 6-2, page 6-54);

h_f = friction head loss, in feet = S_fL;

S_f = friction slope or slope that will produce uniform flow, in feet per foot. For Type II operation the friction slope is based upon TW depth;

S_o = slope of culvert, in feet per foot; and

L = length of culvert, in feet.

Type III Flowing Part Full with Inlet Control.

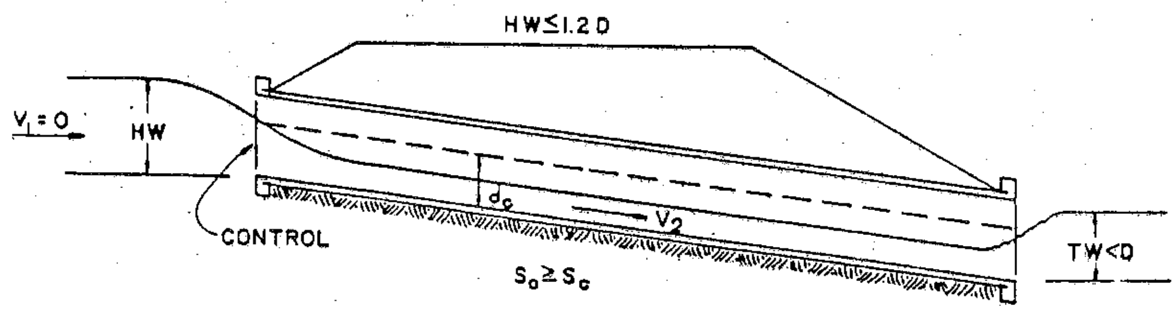


Figure 6-3

CONDITIONS

The entrance is unsubmerged (HW < 1.2D) and the slope at design discharge is equal to or greater than critical slope (supercritical) ($S_0 \geqslant S_c$).

This condition is a common occurrence for culverts in rolling or mountainous country where the flow does not submerge the entrance. The control is critical depth at the entrance.

If critical flow occurs near the inlet, the culvert is operating under "Inlet Control". The maximum discharge through a culvert flowing part full occurs when flow is at critical depth for a given energy head. To assure that flow passes through critical depth near the inlet, the culvert must be laid on a slope equal to or greater than critical slope for the design discharge. Placing culverts which are to flow part full on slopes greater than critical slope will increase the outlet velocities but will not increase the discharge. The discharge is limited by the section near the inlet at which critical flow occurs.

The capacity of a culvert flowing part full with control at the inlet is governed by the following equation when the approach velocity is considered zero.

$$HW = d_c + \frac{V_2^2}{2g} + k_e \frac{V_2^2}{2g} \qquad (6\text{-}5)$$

where:

HW = headwater depth above the invert of the upstream end of the culvert, in feet. Headwater depth must be equal to or less than 1.2D or entrance is submerged and Type IV operation will result.

d_c = critical depth of flow, in feet (Figure 6-25 or 6-26).

V_2 = critical velocity at entrance of culvert, in feet per second.

The velocity of flow varies from critical velocity at the entrance to uniform velocity at the outlet provided the culvert is sufficiently long. Therefore, the outlet velocity is the discharge divided by the area of flow in the culvert.

k_e = entrance loss coefficient (Table 6-2).

Type IVA   Flowing Full with Submerged Outlet.

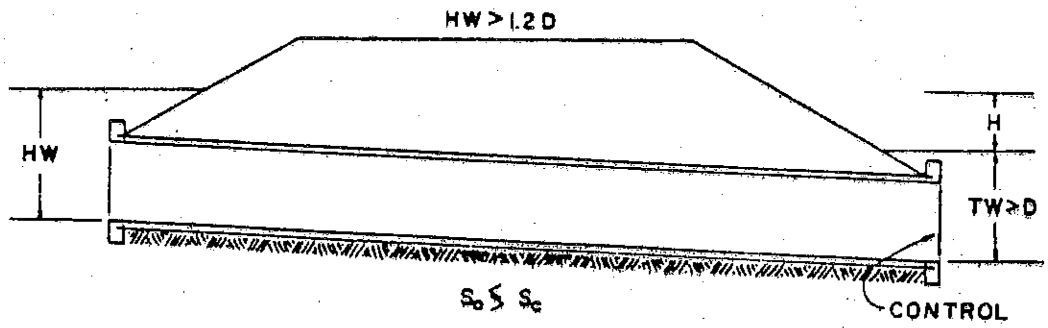


Figure 6-4

CONDITIONS

(Submerged Outlet)

The entrance is submerged ($HW \geqslant 1.2D$). The tailwater completely submerges the outlet.

Most culverts flow with free outlet, but depending on topography, a tailwater pool of a depth sufficient to submerge the outlet may form at some installations. For an outlet to be submerged, the depth at the outlet must be equal to or greater than the diameter of pipe or height of box. The capacity of a culvert flowing full with a submerged outlet is governed by the following equation when the approach velocity is considered zero. Outlet velocity is based on full-pipe flow at the outlet.

$$HW = H + TW - S_oL \qquad (6\text{-}6)$$

where:

HW = headwater depth, in feet, above the invert of the upstream end of the culvert. Headwater depth must be greater than 1.2D for entrance to be submerged.

H = head for culvert flowing full, in feet.

TW = tailwater depth, in feet.

S_O = slope of culvert, in feet per foot.

L = length of culvert, in feet.

Type IVB Flowing Full with Partially Submerged Outlet.

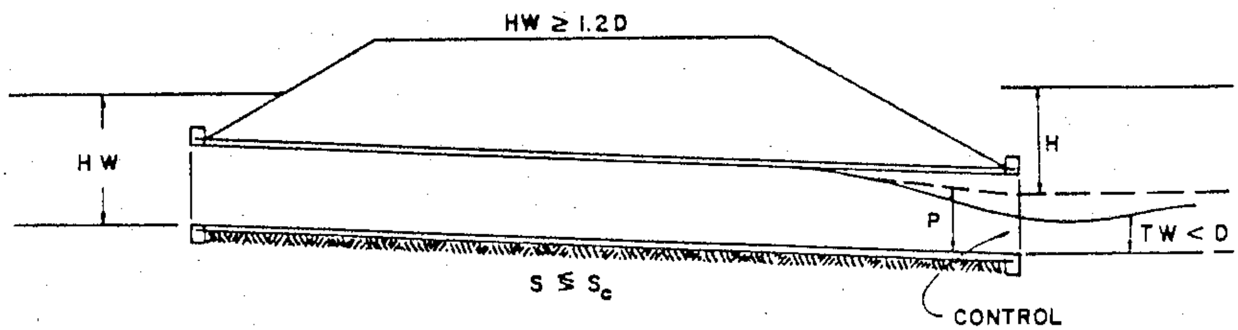


Figure 6-5

CONDITIONS

(Partially Submerged Outlet)

The entrance is submerged (HW ⩾ 1.2D). The tailwater depth is less than D (TW < D).

The capacity of a culvert flowing full with a partially submerged outlet is governed by the following equation when the approach velocity is considered zero. Outlet velocity is based on critical depth if TW depth is less than critical depth. If TW depth is greater than critical depth, outlet velocity is based on TW depth.

$$HW = H + P - S_o L \qquad (6\text{-}7)$$

where:

HW = headwater Depth, in feet, above the invert of the upstream end of the culvert. Headwater depth must be greater than 1.2D for entrance to be submerged.

H = head for culverts flowing full, in feet.

P = pressure line height = $\frac{d_c + D}{2}$, in feet.

d_c = critical depth, in feet.

D = diameter or height of structure, in feet.

S_o = slope of culvert, in feet per foot.

L = length of culvert, in feet.

6.30 End Treatments

The normal functions of properly designed headwalls and endwalls are to anchor the culvert to prevent movement, to control erosion and scour resulting from excessive velocities and turbulence, and to prevent adjacent soil from sloughing into the waterway opening. All headwalls shall be constructed of reinforced concrete and may be either straight parallel headwalls, flared headwalls, or warped headwalls with or without aprons as may be required by site conditions.

Conditions at Entrance

It is important to recognize that the operating characteristics of a culvert may be completely changed by the shape or condition at the inlet or entrance. Design of culverts must involve consideration of energy losses that may occur at the entrance. The entrance head losses may be determined by the following equation.

$$h_e = k_e \frac{V^2}{2g} \qquad (6\text{-}8)$$

where:

h_e = entrance head loss, in feet.

V = velocity of flow in culvert, in feet per second.

k_e = entrance loss coefficient (Table 6-2, page 6-54).

In general the following guidelines should be used in the selection of the type of headwall or endwalls:

Parallel Headwall and Endwall

1. Approach velocities are low (below 6 fps).
2. Backwater pools may be permitted.
3. Approach channel is undefined.
4. Ample right-of-way or easement is available.
5. Downstream channel protection is not required.

Flared Headwall and Endwall

1. Channel is well defined.
2. Approach velocities are between 6 and 10 fps.
3. Medium amounts of debris exist.

Warped Headwall and Endwall

1. Channel is well defined and concrete lined.
2. Approach velocities are between 8 and 20 fps.
3. Medium amounts of debris exist.

These headwalls are effective with drop-down aprons to accelerate flow through culvert, and they are effective endwalls for transitioning flow from closed conduit flow to open channel flow. This type of headwall should be used only where the drainage structure is large and right-of-way or easement is limited.

Improved Inlets

Several types of improved inlets have been developed. The use of these inlets may provide substantial savings by a reduction in the barrel size of the proposed structure. The use of these inlets is optional and should be based on an economic analysis by the designer. For box culverts, reinforced concrete structures, and structures using headwalls the use of beveled inlets or tapered inlets is strongly recommended. For more information, the designer is referred to Hydraulic Design of Improvements for Culverts, Hydraulic Engineering Circular No. 13, August 1973, by the Federal Highway Administration.

6.40 Culvert Design with Standard Inlets

The information and publications necessary to design culverts according to the procedure given in this section can be found in Hydraulic Charts for the Selection of Highway Culverts, Hydraulic Engineering Circular No. 5, March

1965, and Capacity Charts for the Selection of Highway Culverts, Hydraulic Engineering Circular No. 10, March, 1965. Both of which are publications of the FHWA. Some of the charts and nomographs, from these publications, covering the more common requirements are located in the Design Charts at the end of this Section. For special cases and larger sizes, the FHWA publications should be used.

Culvert Sizing

Each chart, Figures 6-13 to 6-20, contains a series of curves which show the discharge capacity per barrel in cfs for each of several sizes of similar type culverts for various headwater depths in feet above the culvert invert at the inlet. The invert of the culvert is defined as the low point of its cross-section.

Each culvert size is described by two lines, one solid and one dashed. The numbers associated with each line are the ratio of the length, L, in feet, to the slope, 100 S_o, in percent. The dashed lines represent the maximum $L/(100S_o)$ ratio for which the curves may be used without modification. The solid line represents the division between outlet and inlet control. For values of $L/(100S_o)$ less than that shown on the solid line, the culvert is operating under inlet control and the headwater depth is determined from the $L/(100S_o)$ value given on the solid line. The solid-line inlet-control curves are plotted from model test data. The dashed-line outlet-control curves were computed for culverts of various lengths with relatively flat slopes. Free outfall at the outlet was assumed, therefore, tailwater depth is assumed to not influence the culvert performance.

For culverts flowing under outlet control, the head loss at the entrance was computed, and the hydraulic roughness of the various materials used in culvert construction was taken into account in computing resistance loss for full or part-full flow. The Manning's "n" values used for each culvert type ranged from 0.012 to 0.032.

Except for large pipe sizes, headwater depths on the charts extend to three times the culvert height. Pipe arches and oval pipe show headwater up to 2.5 times their height since they are generally used in areas of low fill. The dotted line, stepped across the charts, shows headwater depths of about twice the barrel height and indicates the upper limit of restricted use of the charts. Above this line the headwater elevation should be checked with the nomographs found in Figures 6-21 through 6-33.

The headwater depth given by the charts is actually the difference in elevation between the culvert invert at the entrance and the total head, that is depth plus velocity head, for flow in the approach channel. In most cases, the water surface upstream from the inlet is close to this level and the chart determination may be used as headwater depth for practical design purposes. Where the approach velocity is in excess of 3.0 feet per second, the velocity head must be subtracted from the curve determination of headwater to obtain the actual headwater depth.

Proper use of the capacity charts (Figures 6-13 thru 6-20) will minimize problems of scour or sedimentation. The procedure for sizing the culvert is summarized below. Data can be tabulated in the Design Computation Form, Figure 6-6.

1. List design data: Q(cfs), L(ft), allowable HW (ft), S_0(ft/ft), type of culvert barrel and entrance.

2. Compute $L/(100S_0)$.

3. Enter the appropriate capacity chart with the design discharge, Q.

4. Find the $L/(100S_0)$ value for the smallest pipe that will pass the design discharge. If this value is above the dotted line in Figures 6-13 to 6-20, use the nomographs to check headwater conditions.

5. If the computed $L/(100S_0)$ is less than the value of $L/(100S_0)$ given for the solid line then the value of HW is the value obtained from the solid line curve. If the computed L/(100S) is larger than the value for the dashed outlet-control curve, then special measures must be taken, and the reader is referred to the FHWA publications.

   Check the HW value obtained with the allowable HW. If the indicated HW is greater than the allowable HW, then try the next larger pipe size.

The use of the nomographs on Figures 6-21 to 6-33 is limited to cases where tailwater depth is higher than the critical depth in the culvert. The advantage of the capacity charts over the nomographs is that the capacity charts are direct where the nomographs are trial and error. The capacity charts can be used only when the flow passes through critical at the outlet. When the critical depth at the outlet is less than the tailwater depth, the nomographs must be used; however, both give the same results where either of the two methods are applicable. The procedure for design requires the use of both nomographs and is as follows:

1. List design data: Q(cfs), L(ft), invert elevations at an inlet and outlet (ft), allowable HW(ft), mean and maximum flood velocities in the natural stream (ft/sec), type of culvert, and entrance type for first selection.

2. Determine a trial size by assuming a maximum average velocity based on channel considerations to compute the area, A = Q/V.

3. Find HW for trial size culvert for inlet control and outlet control. For inlet control use Figures 6-27 to 6-33, connect a straight line through D and Q to Scale 1 of HW/D scales and project horizontally to the proper Scale (2 or 3). Compute HW and, if too large or too small, try another size before computing HW for outlet control.

Next, compute the HW for outlet control with Figures 6-21 to 6-24. Enter the graph with the length, the entrance coefficient for the entrance type, and the trial size. Connect the length scale and the culvert-size scale with a straight line, pivot on the turning line, and draw a straight line from the design discharge on the discharge scale through the turning point to the head scale (head loss, H). Compute HW from the equation:

$$HW = H + h_o - LS_o$$

For TW greater than or equal to the top of the culvert, $h_o = TW$, and for TW less than the top of the culvert:

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

whichever is the greater. If TW is less than d_c, the nomographs cannot be used. See Figures 6-25 and 6-26 for critical-depth charts.

4. Compare the computed headwaters and use the higher HW to determine if the culvert is under inlet or outlet control. If outlet control governs and the HW is unacceptable, select a larger trial size and find another HW with the outlet-control nomoraphs. Since the smaller size of culvert had been selected for allowable HW by the inlet-control nomographs, the inlet control for the larger pipe need not be checked.

Design Procedure

Due to problems arising from topography and other considerations, the actual design of a culvert installation is more difficult than the simple process of sizing culverts. The procedure is a guide to design since the problems encountered are too varied and too numerous to be generalized. However, the actual process presented should be followed to insure that some special problem is not overlooked.

Design Computation Form

The use of a design computation form is a convenient method to use to obtain consistent designs with a minimum of culvert cost. An example of such a form is Figure 6-6.

Invert Elevations

After determining the allowable headwater elevation, the tailwater elevation, and the approximate length, invert elevations must be assumed. When considering ponded and non-ponded inlets, either for the design discharge or for some lesser storm which will not cause ponding, scour is not likely to occur in an artificial channel, such as a roadside ditch or a major drainage channel, when the culvert has the same slope as the channel. To reduce the chance of failure due to scour, invert elevations corresponding to the natural grade should be used as a first trial. For

PROJECT:

DESIGNER:__________

DATE:__________

HYDROLOGIC AND CHANNEL INFORMATION

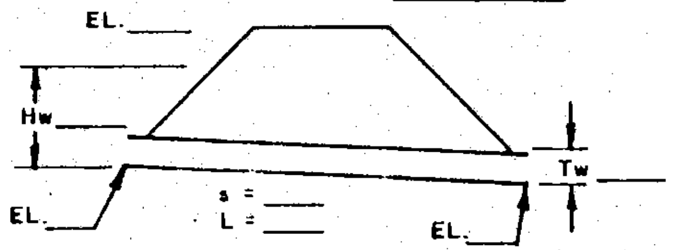
Q_1 = ______ TAILWATER ELEVATION = ____

Q_2 = ______ TAILWATER ELEVATION = ____

(Q_1 = DESIGN DISCHARGE, SAY Q_{25}
Q_2 = CHECK DISCHARGE, SAY Q_{50} OR Q_{100})

SKETCH

STATION: __________



MEAN STREAM VELOCITY = ______

MAX. STREAM VELOCITY = ______

$\frac{L}{100s}$ = ______

CULVERT DESCRIPTION (ENTRANCE TYPE)	Q	SIZE	HEADWATER COMPUTATION													CONTROLLING Hw	OUTLET VELOCITY	COST	COMMENTS
			INLET CONT.		OUTLET CONTROL $HW = H + h_o - L_s$								CHART					
			$\frac{Hw}{D}$	Hw	K_e	H	d_c	$\frac{d_c+D}{2}$	Tw	h_o	Ls	Hw	No.	Hw				

SUMMARY & RECOMENDATIONS:

Figure 6-6 Design Computation Form For Culverts

natural channels, the flow velocity in the channel upstream from the culvert should be investigated to determine if scour will occur.

Culvert Diameter

After the invert elevations have been assumed and using the design computation forms, the capacity charts, and the nomographs, the diameter of pipe that will meet the headwater requirements should be determined. The smallest diameter that appears in the nomographs and capacity charts is 12 inches. Since smaller diameter pipes are often closed by silt, it is recommended that pipe smaller than 18 inches not be used for any drainage where this manual applies. Since the pipe roughness influences the culvert diameter, both concrete and corrugated metal pipe should be considered in design, if both will satisfy the headwater requirements.

Limited Headwater

If there is insufficient headwater elevation to obtain the required discharge, it is necessary to oversize the culvert barrel, lower the inlet invert, use an irregular cross section, or use any combination of the preceding.

If the inlet invert is lowered, special consideration must be given to scour. The use of gabions or concrete drop structures, riprap, and headwalls with aprons and toe walls should be investigated and compared to obtain the proper design.

Culvert Outlet

The outlet velocity must be checked to determine if excessive scour will occur downstream. If scour is indicated, then riprap, an expanding end-section, or a more sophisticated energy dissipating structure should be used as discussed in Section 8 of this manual.

Minimum Slope

To prevent sediment from plugging the culvert, the culvert slope must be equal to or greater than the slope required to maintain a minimum velocity of 2.5 feet per second. The slope should be checked for each design, and if the proper minimum velocity is not obtained, the pipe diameter may be decreased, the slope steepened, a smoother pipe used, or a combination of these may be used.

6.50 Culvert Design with Improved Inlets

The objective of this design procedure is the hydraulic design of culverts, using improved inlets. This design procedure hinges on the selection of a culvert barrel based on its outlet-control performance curve, which is unique when based on elevation. The culvert inlet is then manipulated using edge improvements and adjustment of its elevation in order to achieve inlet-control performance compatible with the outlet-control performance. The resultant culvert design will best satisfy the criteria set by the designer and make optimum use of the barrel selected for the site.

Design Procedure

The flow chart shown in Figure 6-7 outlines the steps of the design procedure, and each step is discussed in detail below. Design calculation forms and design charts and tables that are included are located at the end of this section.

Step 1. Determine and Analyze Site Characteristics.

Step 2. Perform Hydrologic Analysis.

Step 3. Perform Outlet-Control Calculations and Select Culvert.

Outlet-control calculations are performed before inlet-control calculations in order to select the smallest feasible barrel which can be used without the required headwater elevation in outlet control (HW_O) exceeding the allowable headwater elevation (AHW EL.). For use in this procedure, the equation for headwater is in terms of elevation.

The full-flow outlet-control performance curve for a given culvert (size, inlet edge, shape, or material) defines its maximum performance. Therefore, inlet improvements beyond the beveled edge or changes in inlet invert elevation will not reduce the required outlet control headwater elevation. This makes the outlet-control performance curve an ideal limit for improved inlet design.

When the barrel size is increased, the outlet-control curve is shifted to the right, indicating a higher capacity for a given head (see Figure 6-8). Also, it may be generally stated that increased barrel size will flatten the slope of the outlet-control curve.

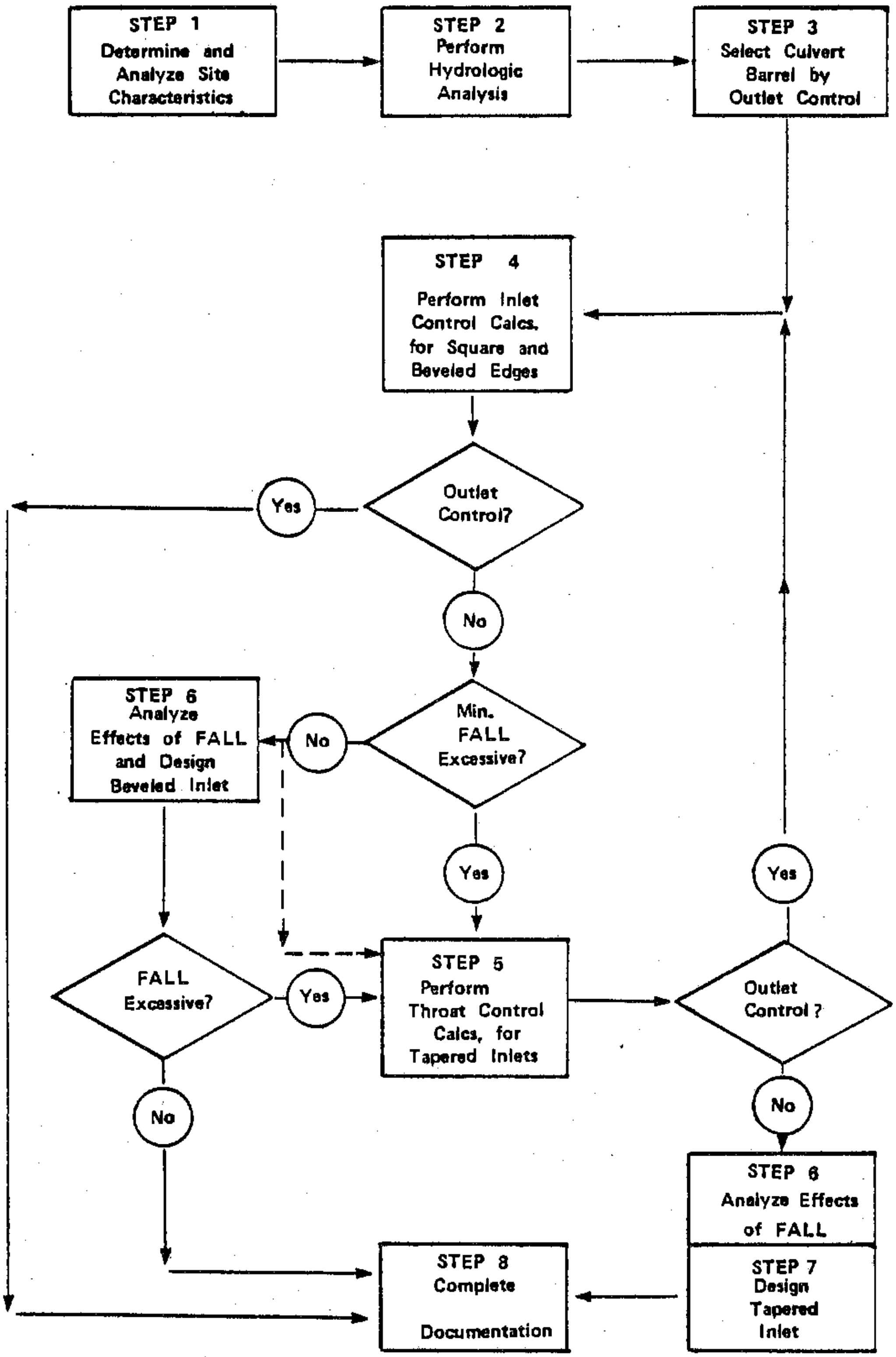


Figure 6-7 Culvert Design Flow Chart

The outlet-control curve passing closest to and below the AHW EL for the design Q. on the performance-curve graph defines the smallest possible barrel which will meet the hydraulic design criteria. However, that curve may be very steep (rapidly increasing headwater requirements for discharges higher than design), or the use of such a small barrel may not be practical. The proper culver selection procedure is:

a. Calculate HW_O at design discharge for trial culvert sizes, entrance conditions, shapes, and materials.

b. Calculate headwater elevations at two additional discharge values in the vicinity of design Q in order to define outlet- control performance.

c. Plot outlet-control performance curves for trial culvert sizes.

d. Select barrel size, shape, and material.

This selection should not be based solely on calculations which indicate that the required headwater at the design discharge is near the AHW EL., but should also be based on outlet velocity as affected by material selection, the designer's evaluation of site characteristics, and the possible consequences of a flood occurrence in excess of the estimated design flood. A sharply rising outlet-control performance curve may be sufficient reason to select a culvert of different size, shape, or material.

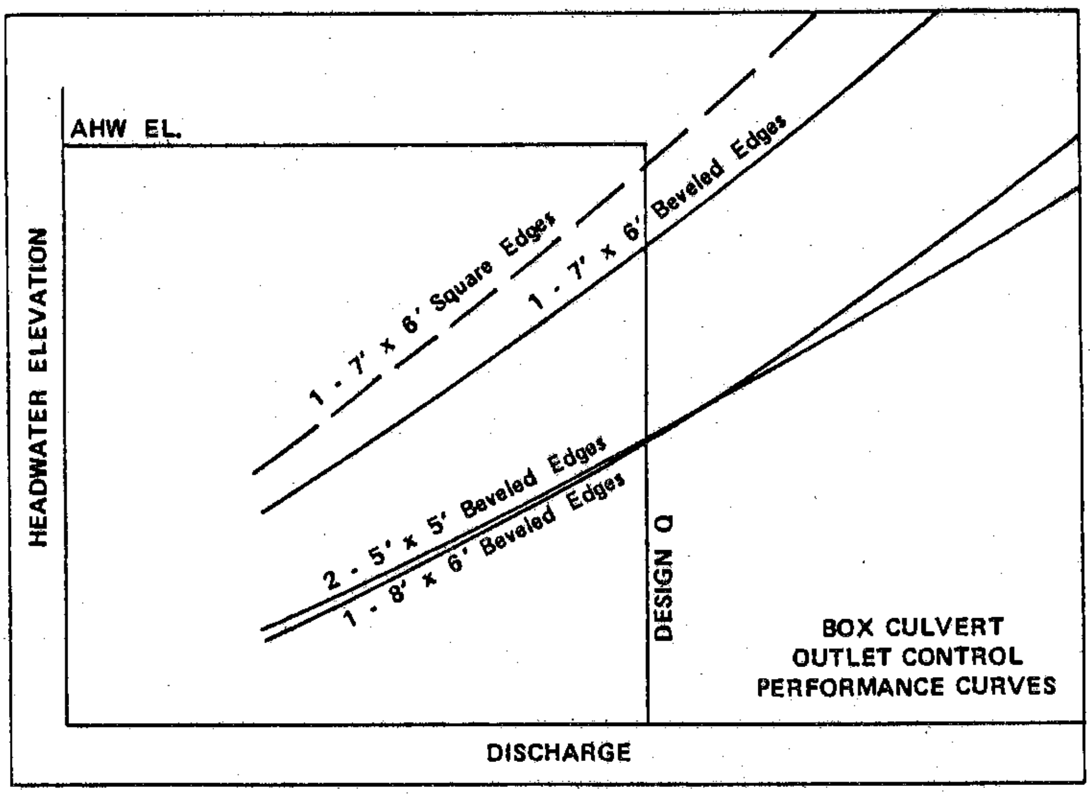


Figure 6-8

In order to calculate the barrel size required in outlet control, the applicable outlet-control nomograph may be used as follows:

a. Intersect the "Turning Line" with a line drawn between Discharge and Head, H. To estimate H, use the following equation:

$$H = \text{AHW EL.} - \text{EL. Outlet Invert} - h_o$$

where h_o may be selected as a culvert height. Accuracy is not critical at this point.

b. Using the point on the "Turning Line", k_e, and the barrel length, draw a line defining the barrel size.

This size gives the designer a good first estimate of the barrel size and more precise sizing will follow rapidly.

Step 4. Perform Inlet-Control Calculations for Conventional and Beveled-Edge Culvert Inlets

The calculation procedure is similar to that used in HEC No. 5, except that headwater is defined as an elevation rather than a depth, a FALL may be incorporated upstream of the culvert face, and performance curves are an essential part of the procedure. The depression or FALL should have dimensions as described for side-tapered inlets.

a. Calculate the required headwater depth, H_f, at the culvert face at design discharge for the culvert selected.

b. Determine required face invert elevation to pass design discharge by subtracting H_f from the AHW EL.

c. If this invert elevation is above the stream bed elevation at the face, the invert would generally be placed on the stream bed and the culvert will then have a capacity greater than design Q with headwater at the AHW EL.

d. If this invert elevation is below the stream bed elevation at the face, the invert must be depressed, and the amount of depression is termed the FALL.

e. Add H_f to the invert elevation to determine HW_f. If HW_f is lower than HW_o, the barrel operates in outlet control at design Q. Proceed to Step 8, Figure 6-7.

f. If the FALL is excessive in the designer's judgment from the standpoint of aesthetics, economy and other engineering reasons, a need for inlet geometry refinements is indicated. If square edges were used in Steps 3 and 4 above, repeat with beveled edges. If beveled edges were used, proceed to Step 5.

g. If the FALL is within acceptable limits, determine the inlet-control performance by calculating required headwater elevation using the flow rates from Step 3 and the FALL determined above. $HW_f = H_f$ + EL. face invert.

h. Plot the inlet-control performance curve with the outlet-control performance curve plotted in Step 3.

i. Proceed to Step 6.

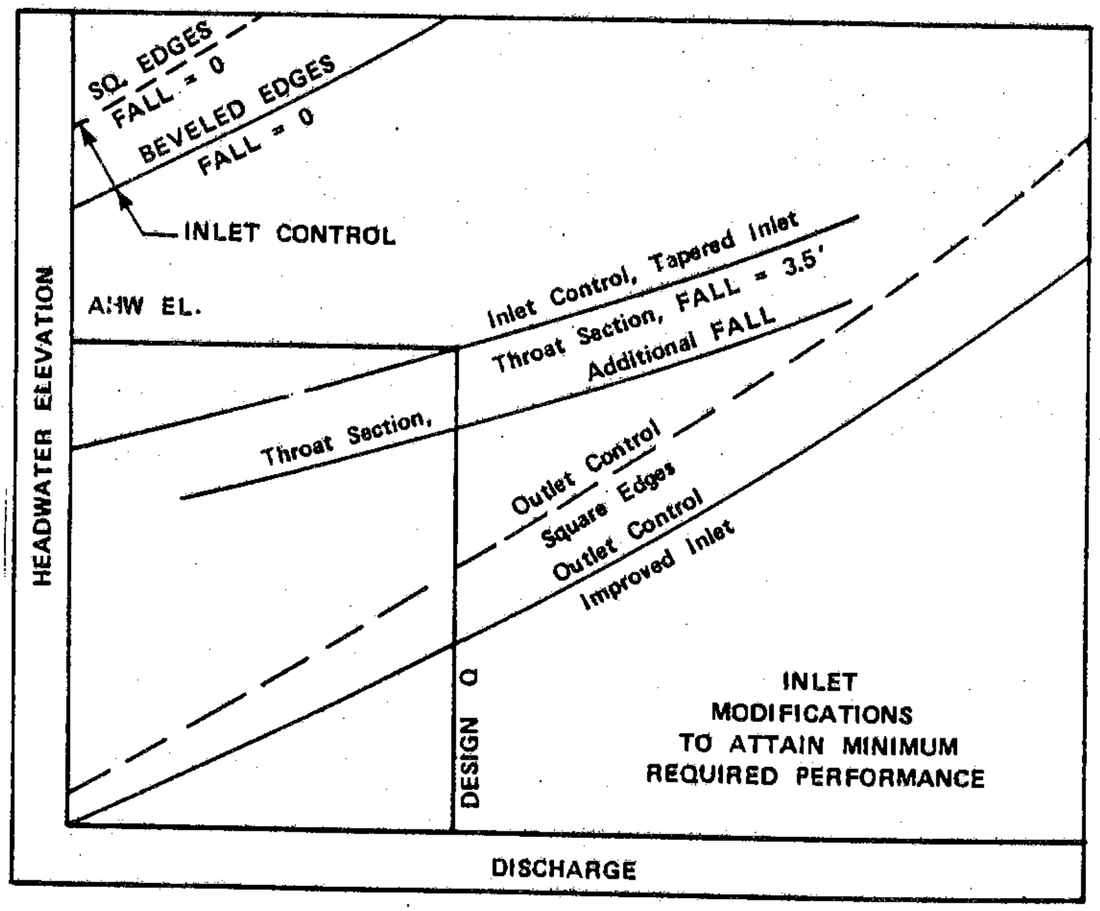


Figure 6-9

Step. 5 Perform Throat-Control Calculations for Side- and Slope-Tapered Inlets

The same concept is involved here as with conventional or beveled-edge culvert design.

a. Calculate required headwater depth on the throat (H_t) at design Q for the culvert selected in Step 3.

b. Determine required throat elevation to pass design discharge by subtracting H_t from the AHW EL.

c. If this throat invert elevation is above the stream bed elevation, the invert would probably be placed on the stream bed and the culvert throat will have a capacity greater than the design Q with headwater at the AHW EL.

d. If this throat invert elevation is below the stream bed elevation, the invert must be depresesed, and the elevation difference between the stream bed at the face and the throat invert is termed the FALL. If the FALL is determined to be excessive, a larger barrel must be selected. Return to Step 5(a).

e. Add H_t to the invert elevation to determine HW_t. If HW_t is lower than HW_o, the culvert operates in outlet control at design Q. In this case, adequate performance can probably be achieved by the use of beveled edges with a FALL. Return to Step 4.

f. Define and plot the throat-control performance curve.

Step 6. Analyze the effect of FALLS on Inlet-Control Section Performance

It is apparent from Figure 6-9 that either additional FALL or inlet improvements would increase the culvert capacity in inlet control by moving the inlet-control performance curve to the right toward the outlet-control performance curve. If the outlet-control performance curve of the selected culvert passes below the point defined by the AHW EL. and the design Q, there is an opportunity to optimize the culvert design by selecting the inlet so as to either increase its capacity to the maximum at the AHW EL. or to pass the design discharge at the lowest possible headwater elevation.

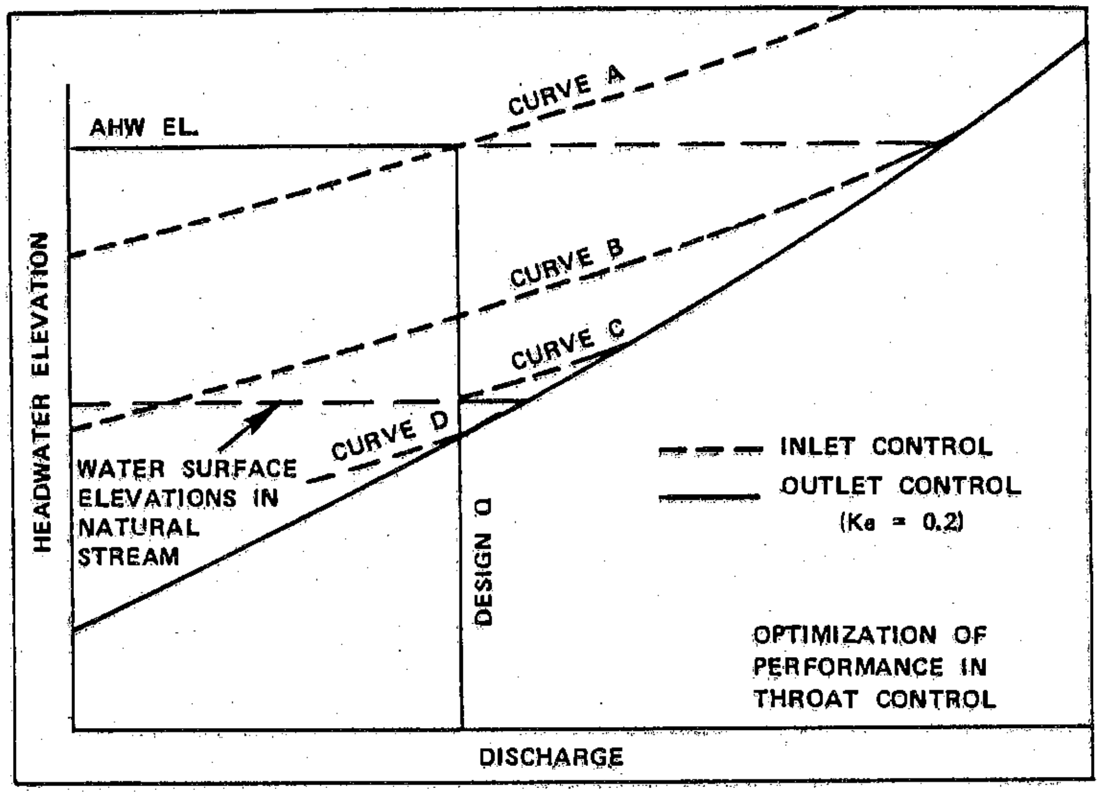


Figure 6-10

Some possibilities are illustrated in Figure 6-10. The minimum inlet-control performance which will meet the selected design criteria is illustrated by Curve A. This design has merit in that minimum expense for inlet improvements and/or FALL is incurred and the inlet will pass a flood in excess of design Q before performance is governed by outlet control. This performance is adequate in many locations, including those locations where headwaters in excess of the AHW EL, would be tolerable on the rare occasion of floods in excess of design Q.

Curve B illustrates the performance of a design which takes full advantage of the potential capacity of the selected culvert and the site to pass the maximum possible flow at the AHW EL. A safety factor in capacity is thereby incorporated in the design. This can be accomplished by the use of a FALL, by geometry improvements at the inlet or by a combination of the two. Additional inlet improvement and/or FALL will not increase the capacity at or above the AHW EL.

There may be reason to pass the design flow at the lowest possible headwater elevation even though the reasons are insufficient to cause the AHW EL. to be set at a lower elevation. The maximum possible reduction in headwater at design Q is illustrated by Curve C. Additional inlet improvement and/or FALL will not reduce the required headwater elevation at design Q.

The water-surface elevation in the natural stream may be a limiting factor in design, i.e., it is not productive to design for headwater at a lower elevation than natural streamflow elevations. The reduction in headwater elevation illustrated by Curve C is limited by natural water surface elevations in the stream. If the water surface elevations in the natural stream had fallen below Curve D, this curve would illustrate the maximum reduction in headwater elevation at design Q. Tailwater depths calculated by assuming normal depth in the stream channel may be used to estimate natural water-surface elevations in the stream at the culvert inlet. These may have been computed as a part of Step 3.

Curve A has been established in either Step 4 for conventional culverts or Step 5 for improved inlets. To define any other inlet-control performance curve such as B, C, or D for the same control section:

a. Select a point on the outlet-control performance curve.

b. Measure the vertical distance from this point to Curve A. This is the difference in FALL between Curve A and the curve to be established, e.g., the FALL on the control section for Curve A plus the distance between Curves A and B is the FALL on the control section for Curve B.

For conventional culverts only:

c. Estimate and compare the costs incurred in FALLS (structural excavation and additional culvert length) to achieve various levels of inlet performance.

d. Select design with increment in cost warranted by increased capacity and improved performance.

e. If FALL required to achieve desired performance is excessive, proceed to Step 5.

f. If FALL is acceptable and performance achieves the design objective, proceed to Step 8.

Step 7. Design Side- and/or Slope-Tapered Inlet (Figures 36 through 39).

Either a side- or slope-tapered inlet may be used if a FALL is required on the throat by use of a depression (FALL) upstream of the face of a side-tapered inlet or a FALL in the inlet of a slope-tapered inlet.

The face of the side- or the slope-tapered inlet should be designed to be compatible with the throat performance defined in Step 6. The basic principles of selecting the face design are illustrated in Figure 6-11.

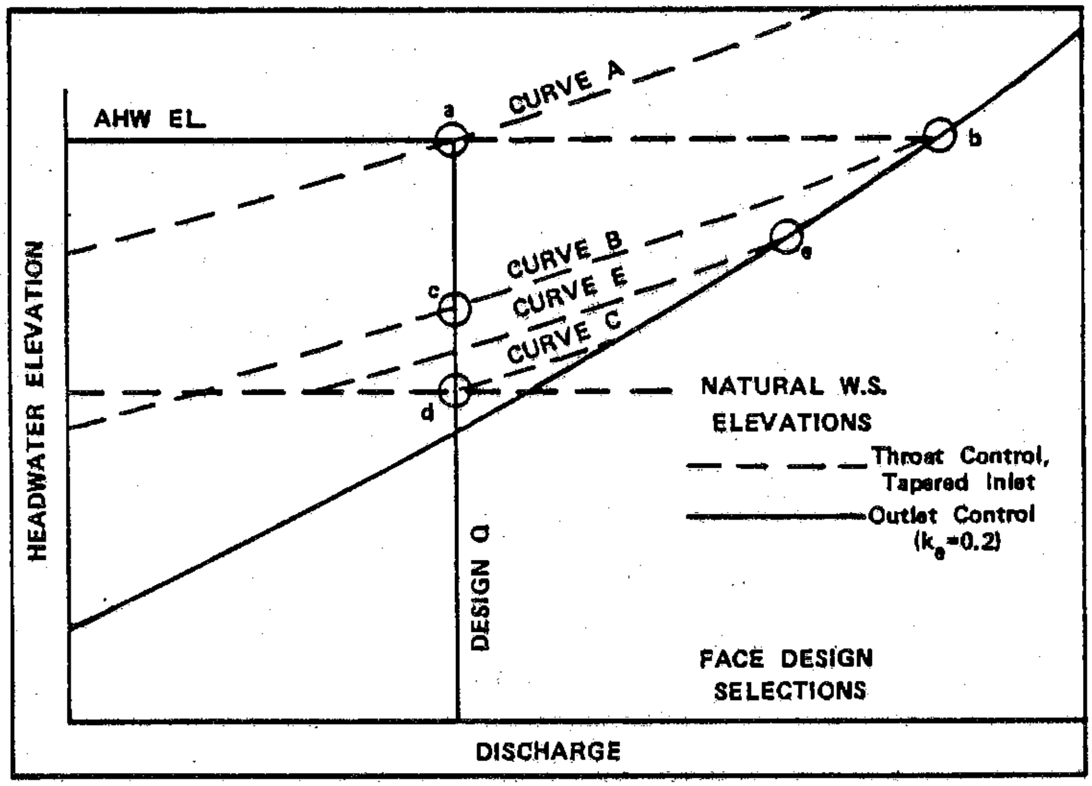


Figure 6-11

The minimum face design is one whose performance curve does not exceed the AHW EL. at design Q. However, a "balanced" design requires that full advantage be taken of the increased capacity and/or lower headwater requirement gained through use of various FALLS. This suggests a face-control curve which intersects the throat-control curve: (1) at the AHW EL., (2) at design Q, (3) at its intersection with the outlet-control curve, or (4) other. These options are illustrated in Figure 6-11 by points a through e representing the intersections of face-control curves with the throat-control curves. The options are explained as follows: (1) Intersection of face- and throat-control curves at the AHW EL. (Point a or b): For the minimum acceptable throat-control performance (Curve A), this is the minimum face size that can be used without the required headwater elevation (HW_f) exceeding the AHW EL. at design Q (Point a). For throat-control performance greater than minimum but equal to or less than Curve B, this is the minimum face design which makes full use of the FALL placed on the throat to increase culvert capacity at the AHW EL. (Point b). (2) Intersection of face- and throat-control curves at design Q (Points a, c, or d): This face design results in throat control at discharges equal to or greater than design Q. It makes full use of the FALL to increase capacity and reduce headwater requirements at flows equal to or greater than the design Q. (3) Intersection of the face-control curve with throat-control curve at its intersection with the outlet-control curve (Points b or e): This option is the minimum face design which can be used to make full use of the increased capacity available from the FALL placed on the throat. It cannot be used where HW_f would exceed AHW EL. at design Q; e.g., with the minimum acceptable throat-control curve. (4) Variations in the above options available to the designer. The culvert face can be designed so that culvert performance will change from face control to throat control at any discharge at which inlet control governs. Options (1) through (3), however, fulfill design objectives of minimum face size to achieve the maximum increase in capacity possible for a given FALL, or the maximum possible decrease in the required headwater for a given FALL for any discharge equal to or greater than design Q.

Figure 6-12 illustrates the optional tapered-inlet designs possible. Note that the inlet dimensions for the side-tapered inlet are the same for all options. This is because performance of the side-tapered inlet nearly parallels the performance of the throat, and an increase in headwater on the throat by virtue of an increased FALL results in an almost equal increase in headwater on the face. Each foot of FALL on the throat of a culvert with a side-tapered inlet requires additional barrel length equal to the fill slope; e.g., if the fill slope is 3:1, use of 4 ft of FALL rather than 3 ft results in a culvert barrel 3 ft longer as well as increased culvert capacity and/or reduced headwater requirements.

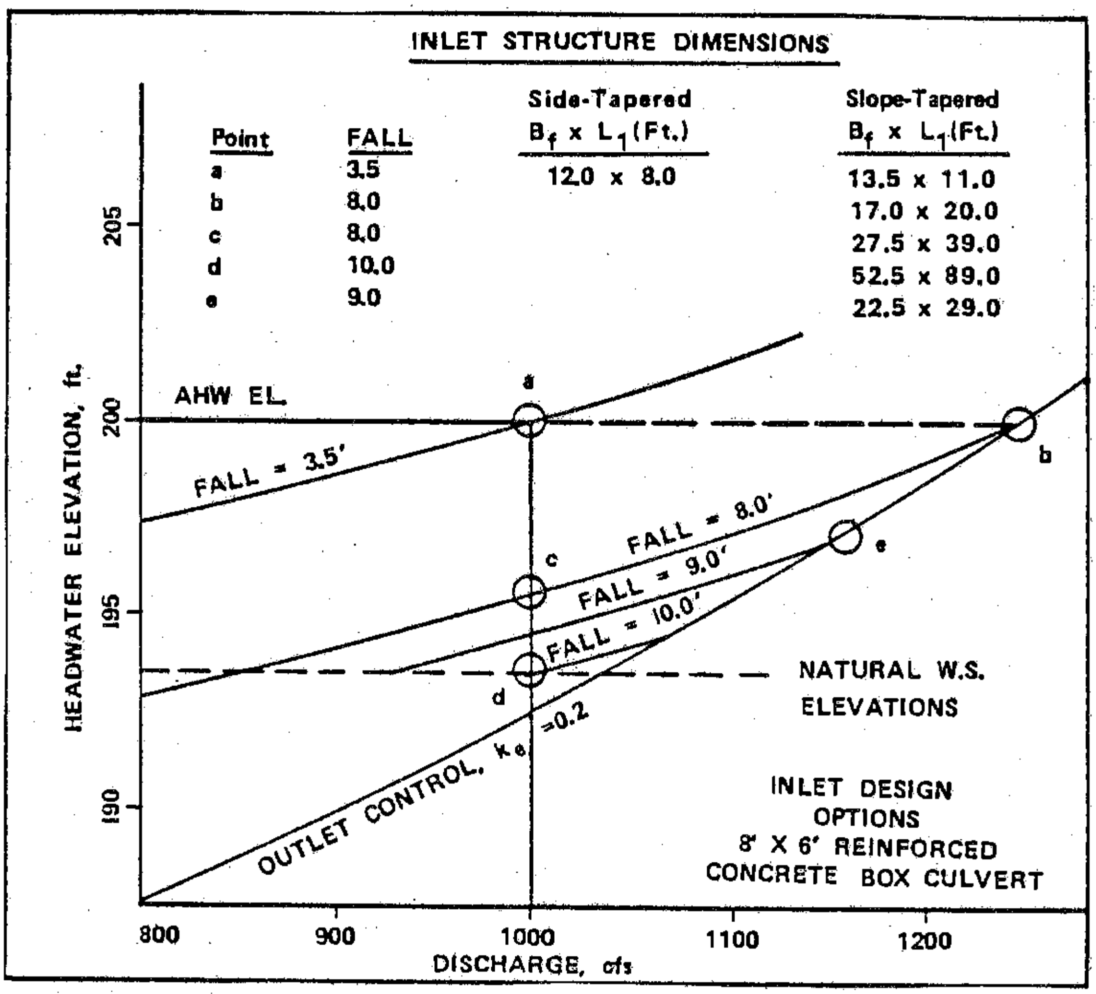


Figure 6-12

Face dimensions and inlet length increase for the slope-tapered inlet as the capacity of the culvert is increased by additional FALL on the throat. No additional head is created for the face by placing additional FALL on the throat. On the other hand, use of a greater FALL at the throat of a culvert with a slope-tapered inlet does not increase culvert length.

The steps followed in the tapered-inlet designs are:

a. Compute Hf for side- and slope-tapered inlets for various FALLS at design Q and other discharges. Side-Tapered Inlet: $H_f = H_t - 1.0$ (approximate). Slope-Tapered Inlet: H_f = HW EL. - Streambed EL. at face.

b. Determine dimensions of side- and slope-tapered inlets for trial options.

c. For slope-tapered inlets with mitered face, check for crest control.

d. Compare construction costs for various options, including the cost of FALL on the throat.

e. Select design with incremental cost warranted by increased capacity and improved performance.

From the above, it is apparent that in order to optimize culvert design, performance curves are an integral part of the design procedure. At many culvert sites, designers have valid reasons for providing a safety factor in designs. These reasons include uncertainty in the design discharge estimate, potentially disastrous results in property damage or damage to the highway from headwater elevations which exceed the allowable, the potential for development upstream of the culvert, and the chance that the design flood will be exceeded during the life of the installation.

Dimensional Limitations

Side-Tapered Inlets (See List of Symbols.)

1. $6:1 \geqslant$ Taper $\geqslant 4:1$

   Tapers greater than 6:1 may be used but performance will be underestimated.

2. Wingwall flare angle from 15 degrees to 26 degrees with top edge beveled or from 26 degrees to 90 degrees with or without bevels.

3. If FALL is used upstream of face, extend barrel slope upstream from face a distance of D/2 before sloping upward more steeply.

4. For pipe culverts, these additional requirements apply:

   a. $D \leqslant E \leqslant 1.1D$

   b. Length of square to round transition $\geqslant 0.5D$

   c. FALL $\geqslant D/4$

   d. $P \geqslant 3T$

   e. $W_p = B_f + T$ or $4T$, whichever is larger.

Slope-Tapered Inlet (See List of Symbols.)

1. $6{:}1 \geqslant$ Taper $\geqslant 4{:}1$

   Tapers > 6:1 may be used, but performance will be underestimated.

2. $3{:}1 \geqslant S_f \geqslant 2{:}1$

   If $S_f > 3{:}1$, use side-tapered design

3. Minimum $L_3 = 0.5B$

4. $1.5D \geqslant$ FALL $\geqslant D/4$

   For FALL < D/4, use side-tapered design

   For FALL > 1.5D, estimate friction losses between face and throat.

5. Wingwall flare angle from 15 degrees to 26 degrees with top edge beveled or from 26 degrees to 90 degrees with or without bevels.

6. For pipe culvert, these additional requirements apply:

   a. Square to circular transition length > 0.5D.

   b. Square-throat dimension equal to barrel diameter. It is not necessary to check square-throat performance.

Example 1

Box Culvert

Given: Design Discharge, Q = 1,000 cfs, for a 10-year recurrence interval

Slope of stream bed, S_o = 0.05 ft/ft

Allowable Headwater Elevation = 200.0 ft msl

Elevation Outlet Invert = 172.5 ft msl

Culvert Length, L_a = 350 ft

Downstream channel approximates an 8-ft wide trapezoidal channel with 2:1 side slopes and a Manning's "n" of 0.03.

Requirements: This box culvert will be located in a rural area where the Allowable Headwater Elevation is not too critical; that is, the damages are low due to exceeding that elevation at infrequent times. Thus, the culvert should have the smallest possible barrel to pass design Q without exceeding AHW EL. Use a reinforced concrete box with n = 0.012.

PROJECT: *Example No. 1* OUTLET CONTROL DESIGNER: *JMN*

DESIGN CALCULATIONS

STATION: ______ DATE: *12-10-73*

INITIAL DATA:

Q_{10} = *1000* cfs

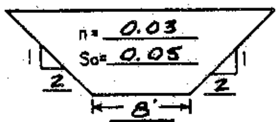
AHW El. = *200* ft.

S_o = *0.05*

L_a = *350* ft.

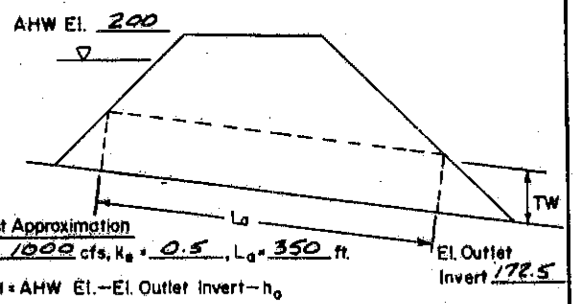
El. Outlet Invert *172.5* ft.

Stream Data:



Barrel Shape and Material *Rect. Conc. Box* Barrel n = *0.012*

SKETCH



First Approximation

Q = *1000* cfs, k_e = *0.5*, L_a = *350* ft.

H = AHW El. − El. Outlet Invert − h_o

= *200* − *172.5* − *5* = *22.5*

∴ A = *40* ft² or D = ____ ft.; Try *7'x6'*

Q	$\frac{Q}{N}$	* H	$\frac{Q}{NB}$	(1) d_c	$\frac{d_c+D}{2}$	Qn	(2) TW	(3) h_o	(4) HW_o	(5) V_o	COMMENTS
Trial No. *1*, N = *1*, B = *7*, D = *6*, k_e = *0.5*											*Square edges*
1000	*1000*	*21*	*143*	*76*	*6.0*		*3.5*	*6.0*	*199.5*	*23.8*	*Ok - Close to AHW El.*
800	*800*	*13.2*	*114*	*76*	*6.0*		*3.25*	*6.0*	*191.7*		
1200	*1200*	*30*	*172*	*76*	*6.0*		*3.8*	*6.0*	*208.5*		
Trial No. *2*, N = *1*, B = *7*, D = *6*, k_e = *0.2*											*Beveled edges*
1000	*1000*	*19*	*143*	*76*	*6.0*		*3.5*	*6.0*	*197.5*	*23.8*	*Ok - Lowered HW_o 2'*
800	*800*	*12*		*Same as*				*6.0*	*190.5*		*Try 1-6'x6'*
1200	*1200*	*27*		*sq. edge*				*6.0*	*205.5*		
Trial No. *3*, N = *1*, B = *6*, D = *6*, k_e = *0.2*											*Beveled edges*
1000	*1000*	*26*	*167*	*76*	*6.0*		*3.5*	*6.0*	*204.5*	*27.8*	*No good - Does not meet*
											design Criteria
											exceeds AHW El.

Notes and Equations:

(1) d_c cannot exceed D

(2) TW based on d_n in natural channel, or other downstream control.

(3) $h_o = \frac{d_c+D}{2}$ or TW, whichever is larger.

(4) HW_o = H + h_o + El. Outlet Invert.

(5) Outlet Velocity (V_o) = Q/Area defined by d_c or TW, not greater than D. Do not compute until control section is known.

SELECTED DESIGN

N = *1*

B = *7* ft.

D = *6* ft.

k_e = *0.2*

At Design Q:

HW_o = *197.5* ft.

V_o = *23.8* f/s

$$* \quad H = \left[1 + k_e + \frac{29n^2 \cdot L}{R^{1.33}}\right] \frac{V^2}{2g}$$

PROJECT: *Example No. 1* — CULVERT INLET CONTROL SECTION DESIGN CALCULATIONS — DESIGNER: *JMN*

STATION: ________ — DATE: *12-10-73*

INITIAL DATA:

Q_{10} = *1000* cfs

AHW El. = *200* ft.

S_o = *0.05*

L_a = *350* ft.

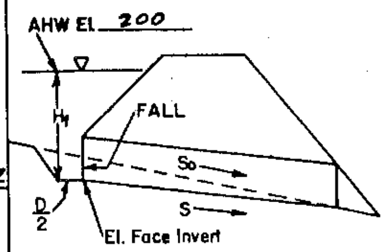
El. Stream Bed at Face *190* ft.

Barrel Shape and Material *RCB* Barrel n = *0.012*

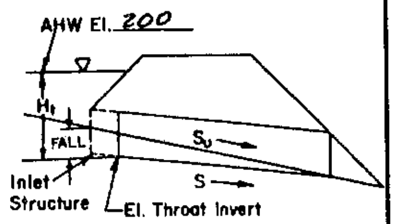
N = *1*, B = *7*

D = *6*, $NBD^{3/2}$ = *102.9 (TABLE 3)*

(Pipe) $ND^{5/2}$ = ________



CONVENTIONAL or BEVELED INLET: FACE CONTROL SECTION (Upper Headings)



TAPERED INLET THROAT CONTROL SECTION (Lower Headings)

DEFINITIONS OF INLET CONTROL SECTION

Q	$\frac{Q}{NB}$ / $\frac{Q}{NBD^{3/2}}$	$\frac{H_f}{D}$ / $\frac{H_t}{D}$	H_f / H_t	(1) El. Face Invert / El. Throat Invert	El. Stream Bed At Face	(2) FALL	(3) HW_f / HW_t	(4) S	(5) V_o	COMMENTS (Note: Use Upper Headings for Coventional or Beveled Face; Lower Headings for Tapered Inlet Throat.)
Trial No. *1* Inlet and Edge Description *Beveled-edged Inlet*										
1000	*143*	*3.9*	*23.4*	*176.6*	*192**	*15.4*	*200*			*FALL too large, try tapered*
** Adjusted upstream*										*inlet – Do not use beveled-*
due to FALL										*edged inlet*
Trial No. *2* Inlet and Edge Description *Tapered inlet throat*										
1000	*9.72*	*2.65*	*15.9*	*184.1*	*190*	*5.9*	*200*	*0.033*	*34.2*	*Ok - Calc. Perf. Curves*
800	*7.79*	*2.05*	*12.3*				*196.4*			*From plot, Opportunity to gain (FALL = 1.3 + 5.9 = 7.2')*
1200	*11.68*	*3.4*	*20.4*				*204.5*			*Max. Capacity at AHW = 200*
Trial No. *3* Inlet and Edge Description *Tapered inlet throat, FALL = 7.2'*										
1000			*15.9*	*182.8*	*190*	*7.2*	*198.7*	*0.029*	*33.3*	*Ok - Capacity at AHW = 200*
800			*12.3*				*195.1*			*1062 c.f.s.*
1200			*20.4*				*203.2*			

Notes and Equations:

(1) El. Face (or throat) invert = AHW El. − H_f (or H_t)

(2) FALL = El. Stream Bed at Face − El. face (or throat) invert

(3) HW_f (or HW_t) = H_f (or H_t) + El. face (or throat) invert, where El. face (or throat) invert should not exceed El. stream bed.

(4) $S \approx S_o - FALL/L_a$

(5) Outlet Velocity = Q/Area defined by d_n at S

SELECTED DESIGN

Inlet Description:

FALL = *7.2* ft.

Invert El. = *182.8* ft.

Bevels:

Angle = ________

b = ________ in., d = ________ in.

PROJECT: *Example No. 1* DESIGNER: JMN

SIDE-TAPERED INLET DESIGN CALCULATIONS

STATION: ______ DATE: 12-10-73

INITIAL DATA

Q_{10} = *1000* cfs S_o = *0.05*

AHW El. = *200* ft. L_a = *350* ft.

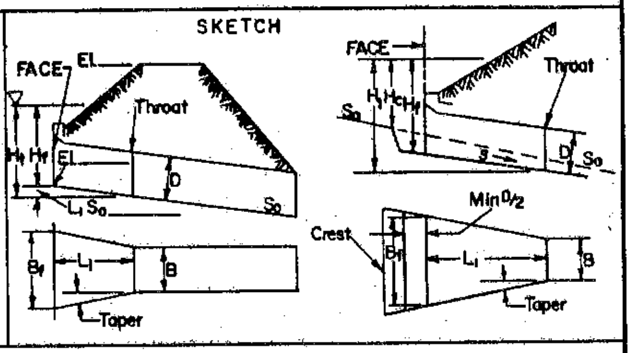
TAPER = *4* : 1

Barrel Shape and Material *RCB; n = 0.012*

Face Edge Description *45° Bevels*

N = *1*, B = *7* ft, D = *6* ft

SKETCH



Upper Headings for Box Culverts, Lower Headings for Pipes

Q	El. Throat Invert	(1) $\frac{H_f}{D}$ / ~~$\frac{H_f}{E}$~~	$\frac{Q}{B_f D^{3/2}}$ / ~~$\frac{Q}{A_f E^{1/2}}$~~	$D^{3/2}$ / ~~$E^{1/2}$~~	(2) Min. B_f / ~~Min. A_f~~	B_f	(3) L_1	(4) S	L_1S	(5) El. Face Invert	COMMENTS
Trial No. *1*, Q = *1000*, HW_f = *200* *(Min. required) FALL = 5.9'*											
1000	*184.1*	*2.48*	*6.6*	*14.7*	*10.3*	*10.5*	*7.0*	*0.033*	*0.2*	*184.3*	$B_f D^{3/2}$ [or $A_f E^{1/2}$] = *154*
											H_f *HW_f*
900		*2.14*	*5.84*								*12.8* *197.1*
1000		*2.48*	*6.50*								*14.5* *198.8*
1100		*2.77*	*7.14*								*16.6* *200.9*
Trial No. *2*, Q = *1000*, HW_f = *198.7* *(FALL = 7.2')*											
1000	*182.8*	*2.48*	*6.6*	*14.7*	*10.3*	*10.5*	*7.0*	*0.029*	*0.2*	*183.0*	$B_f D^{3/2}$ [or $A_f E^{1/2}$] = ______
Trial No. *3*, Q = *1062*, HW_f = *200* *(FALL = 7.2')*											
1062	*182.8*	*2.70*	*7.05*	*14.7*	*10.3*	*10.5*	*7.0*	*0.029*	*0.2*	*183.0*	$B_f D^{3/2}$ [or $A_f E^{1/2}$] = ______

Notes and Equations:

(1) H_f/D [or H_f/E] = (HW_f − El. Throat Invert − 1)/D [or E]
$D \le E \le 1.1D$

(2) Min. $B_f = Q / \left[(D^{3/2}) \frac{Q}{B_f D^{3/2}}\right]$

Min. $A_f = Q / \left[(E^{1/2}) \frac{Q}{A_f E^{1/2}}\right]$

(3) $L_1 = \left[\frac{B_f - NB}{2}\right]$ TAPER

(4) From throat design

(5) El. Face Invert − El. Throat Invert > 1 ft., recompute.
Face and Throat may be lowered to better fit site, but do not raise.

SELECTED DESIGN

B_f = *10.5* ft.

L_1 = *7.0* ft.

Bevels; Angle *45* °

d = *3* in., b = *5.3* in.

Crest Check:

HW_c = *198.7* ft.

H_c = *7.7* ft.

Q/W = *59* (Chart 17)

Min. W = *17* ft.

PROJECT: *Example No. 1* SLOPE-TAPERED INLET DESIGN CALCULATIONS DESIGNER: *JMN*

STATION: ______ DATE: *12-10-73*

INITIAL DATA:

Q_{10} = *1000* cfs S_o = *0.05*

AHW EL. *200* ft. L_a = *350* ft.

El. Stream bed at crest *191* ft.

El. stream bed at face *190* ft.

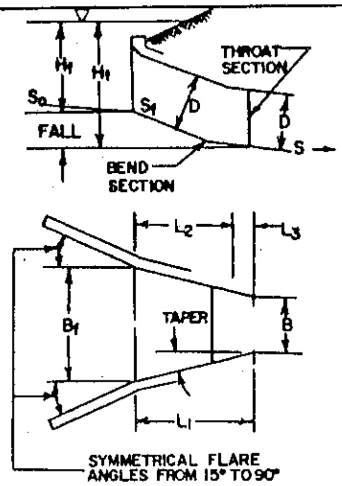
TAPER = *4* :1 (4:1 to 6:1)

S_f = *2* :1 (2:1 to 3:1)

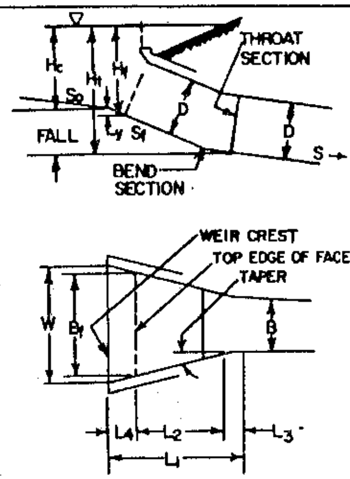
Barrel Shape and Material *RCB; n=0.012*

Inlet Edge Description *45° Bevels*

N= *1*, B= *7* ft, D= *6* ft.



VERTICAL



MITERED

	Q	HW_f	El. Throat Invert	(1.) El. Face Invert	(2.) H_f	$\frac{H_f}{D}$	$\frac{Q}{B_f D^{3/2}}$	$D^{3/2}$	(3.) Min. B_f	B_f	S	Comments
Trial 1	*1000*	*200*	*184.1*	*190*	*10*	*1.67*	*5.1*	*14.7*	*13.3*	*14*	*0.035*	$B_f D^{3/2}$ = ______
												Vertical face point no. 1
Trial 2	*1000*	*198.7*	*182.8*	*190*	*8.7*	*1.43*	*4.45*	*14.7*	*15.3*	*16*	*0.029*	$B_f D^{3/2}$ = ______
												Vertical face point no. 2

Note: Use only throat designs with FALL > 0.25D

(1.) El. face invert: Vertical = Approx. stream bed elevation
Mitered = El. Crest − y, where y = 0.4D (Approx.), but higher than throat invert elevation.

(2.) H_f = HW_f − El. face invert

(3.) Min. $B_f = Q/((D^{3/2})\ Q/B_f D^{3/2})$

(4.) Min. L_3	(5.) L_4	(6.) L_2	(7.) Check L_2	(8.) Adj L_3	(9.) Adj TAPER	(10.) L_1	(11.) W	$\frac{Q}{W}$	H_c	(12.) Max. Crest El.	GEOMETRY
3.5	—	*11.8*	*10.5*	—	*4.4:1*	*15.3*					B_f = ___ ft. L_3 = ___ ft. L_1 = ___ ft. L_4 = ___ ft. L_2 = ___ ft. d = ___ in. δ = ___ in.
3.5	—	*14.4*	*14.4*	*3.6*	—	*18.0*					TAPER = ___ :1

(4.) Min. $L_3 = 0.5NB$

(5.) $L_4 = S_f y + D/S_f$

(6.) L_2 = (El. Face (Crest) Invert − El. Throat Invert) $S_f - L_4$

(7.) Check $L_2 = \left[\frac{B_f - NB}{2}\right]$ TAPER $- L_3$

(8.) If (7) > (6), Adj. $L_3 = \left[\frac{B_f - NB}{2}\right]$ TAPER $- L_2$

(9.) If (6) > (7) Adj. TAPER $= (L_2 + L_3) / \left[\frac{B_f - NB}{2}\right]$

(10.) $L_1 = L_2 + L_3 + L_4$

(11.) Mitered: $W = NB + 2\left[\frac{L_1}{TAPER}\right]$

(12.) Max. Crest El. = $HW_f - H_c$

PROJECT: *Example No. 1* SLOPE-TAPERED INLET DESIGN CALCULATIONS DESIGNER: *JMN*

STATION: ________ DATE: *12-10-73*

INITIAL DATA:

Q_{10} = *1000* cfs S_o = *0.05*

AHW EL. *200* ft. L_a = *350* ft.

El. Stream bed at crest *191* ft.

El. stream bed at face *190* ft.

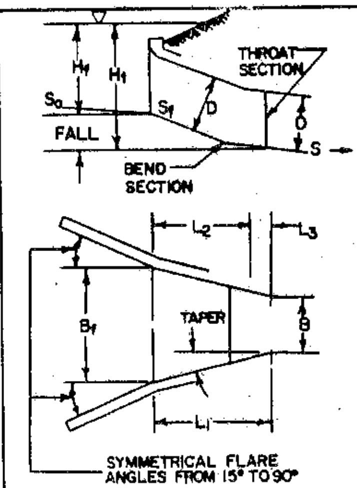
TAPER = *4* :1 (4:1 to 6:1)

S_f = *2* :1 (2:1 to 3:1)

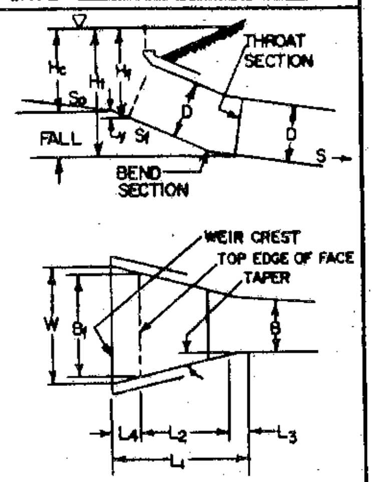
Barrel Shape and Material *RCB ; n = 0.012*

Inlet Edge Description *45° Bevels*

N = *1*, B = *7* ft, D = *6* ft.



VERTICAL



MITERED

	Q	HW_f	El. Throat Invert	(1.) El. Face Invert	(2.) H_f	$\frac{H_f}{D}$	$\frac{Q}{B_f D^{3/2}}$	$D^{3/2}$	(3.) Min. B_f	B_f	S	Comments
Trial 1	*1062*	*200*	*182.8*	*190*	*10*	*1.67*	*5.1*	*14.7*	*14.2*	*15.0*	*0.029*	$B_f D^{3/2}$ = ______
												Vertical face point no. 3
Trial 2	*1000*	*200*	*184.1*	*188.6*	*11.4*	*1.90*	*5.65*	*14.7*	*12.0*	*12.0*	*0.033*	$B_f D^{3/2}$ = ______
												Mitered face point no. 1

Note: Use only throat designs with FALL > 0.25D

(1.) El. face invert: Vertical = Approx. stream bed elevation
Mitered = El. Crest − y, where y = 0.4D (Approx.), but higher than throat invert elevation.

(2.) $H_f = HW_f$ − El. face invert

(3.) Min. $B_f = Q/((D^{3/2})\ Q/B_f D^{3/2})$

(4.) Min. L_3	(5.) L_4	(6.) L_2	(7.) Check L_2	(8.) Adj L_3	(9.) Adj TAPER	(10.) L_1	(11.) W	$\frac{Q}{W}$	H_c	(12.) Max. Crest El.
3.5	*—*	*14.4*	*12.5*	*—*	*4.5:1*	*17.9*				
3.5	*7.8*	*6.0*	*6.5*	*4.0*	*—*	*17.8*	*15.9*	*63.0*	*8.0*	*192.0*

GEOMETRY

B_f = ___ ft. L_3 = ___ ft.
L_1 = ___ ft. L_4 = ___ ft.
L_2 = ___ ft. d = ___ in.
b = ___ in.
TAPER = ___ :1

(4.) Min. $L_3 = 0.5NB$

(5.) $L_4 \approx S_f y + D/S_f$

(6.) L_2 = (El. Face (Crest) Invert − El. Throat Invert) $S_f - L_4$

(7.) Check $L_2 = \left[\frac{B_f - NB}{2}\right]$ TAPER $- L_3$

(8.) If (7)>(6), Adj. $L_3 = \left[\frac{B_f - NB}{2}\right]$ TAPER $- L_2$

(9.) If (6)>(7) Adj. TAPER $= (L_2 + L_3) / \left[\frac{B_f - NB}{2}\right]$

(10.) $L_1 = L_2 + L_3 + L_4$

(11.) Mitered: $W = NB + 2\left[\frac{L_1}{TAPER}\right]$

(12.) Max. Crest El. = $HW_f - H_c$

PROJECT: *Example No. 1* SLOPE-TAPERED INLET DESIGN CALCULATIONS DESIGNER: *JMN*

STATION: DATE: *12-10-73*

INITIAL DATA:

Q_{10} = *1000* cfs S_o = *0.05*

AHW EL. *200* ft. L_a = *350* ft.

El. Stream bed at crest *191* ft.

El. stream bed at face *190* ft.

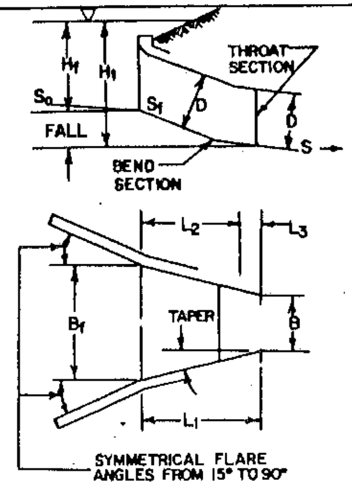
TAPER = *4* :1 (4:1 to 6:1)

S_f = *2* :1 (2:1 to 3:1)

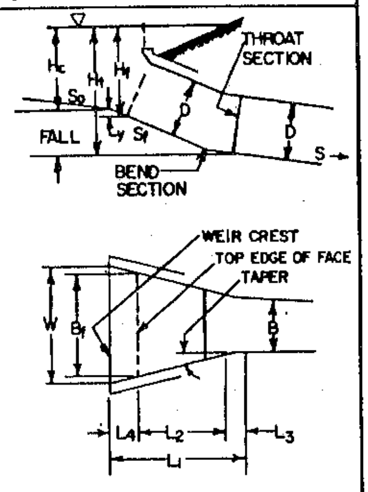
Barrel Shape and Material *RCB ; n = 0.012*

Inlet Edge Description *45° Bevels*

N = *1*, B = *7* ft, D = *6* ft.



VERTICAL



MITERED

	Q	HW_f	El. Throat Invert	(1.) El. Face Invert	(2.) H_f	$\frac{H_f}{D}$	$\frac{Q}{B_f D^{3/2}}$	$D^{3/2}$	(3.) Min. B_f	B_f	S	Comments
Trial 1	*1000*	*198.7*	*182.8*	*188.6*	*10.1*	*1.68*	*5.15*	*14.7*	*13.5*	*14.0*	*0.029*	$B_f D^{3/2}$ =
												Mitered face point no. 2
Trial 2	*1062*	*200*	*182.8*	*188.6*	*11.4*	*1.90*	*5.65*	*14.7*	*12.8*	*13.0*	*0.029*	$B_f D^{3/2}$ =
												Mitered face point no. 3

Note: Use only throat designs with FALL > 0.25D

(1.) El. face Invert: Vertical = Approx. stream bed elevation
Mitered = El. Crest − y, where y = 0.4D (Approx.), but higher than throat invert elevation.

(2.) $H_f = HW_f -$ El. face invert

(3.) Min. $B_f = Q/((D^{3/2})\ Q/B_f D^{3/2})$

(4.) Min. L_3	(5.) L_4	(6) L_2	(7.) Check L_2	(8.) Adj L_3	(9.) Adj TAPER	(10) L_1	(11.) W	$\frac{Q}{W}$	H_c	(12.) Max. Crest El.
3.5	*7.8*	*8.6*	*10.5*	*5.4*	*—*	*21.8*	*17.9*	*55.9*	*7.4*	*191.3*
3.5	*7.8*	*8.6*	*8.5*	*—*	*4.03:1*	*19.9*	*16.8*	*63.3*	*7.9*	*192.1*

GEOMETRY

B_f = ___ ft. L_3 = ___ ft.

L_1 = ___ ft L_4 = ___ ft.

L_2 = ___ ft. d = ___ in.

b = ___ in.

TAPER = ___ :1

(4.) Min. $L_3 = 0.5NB$

(5.) $L_4 = S_f y + D/S_f$

(6.) $L_2 =$ (El. Face (Crest) Invert − El. Throat Invert) $S_f - L_4$

(7.) Check $L_2 = \left[\frac{B_f - NB}{2}\right]$ TAPER $- L_3$

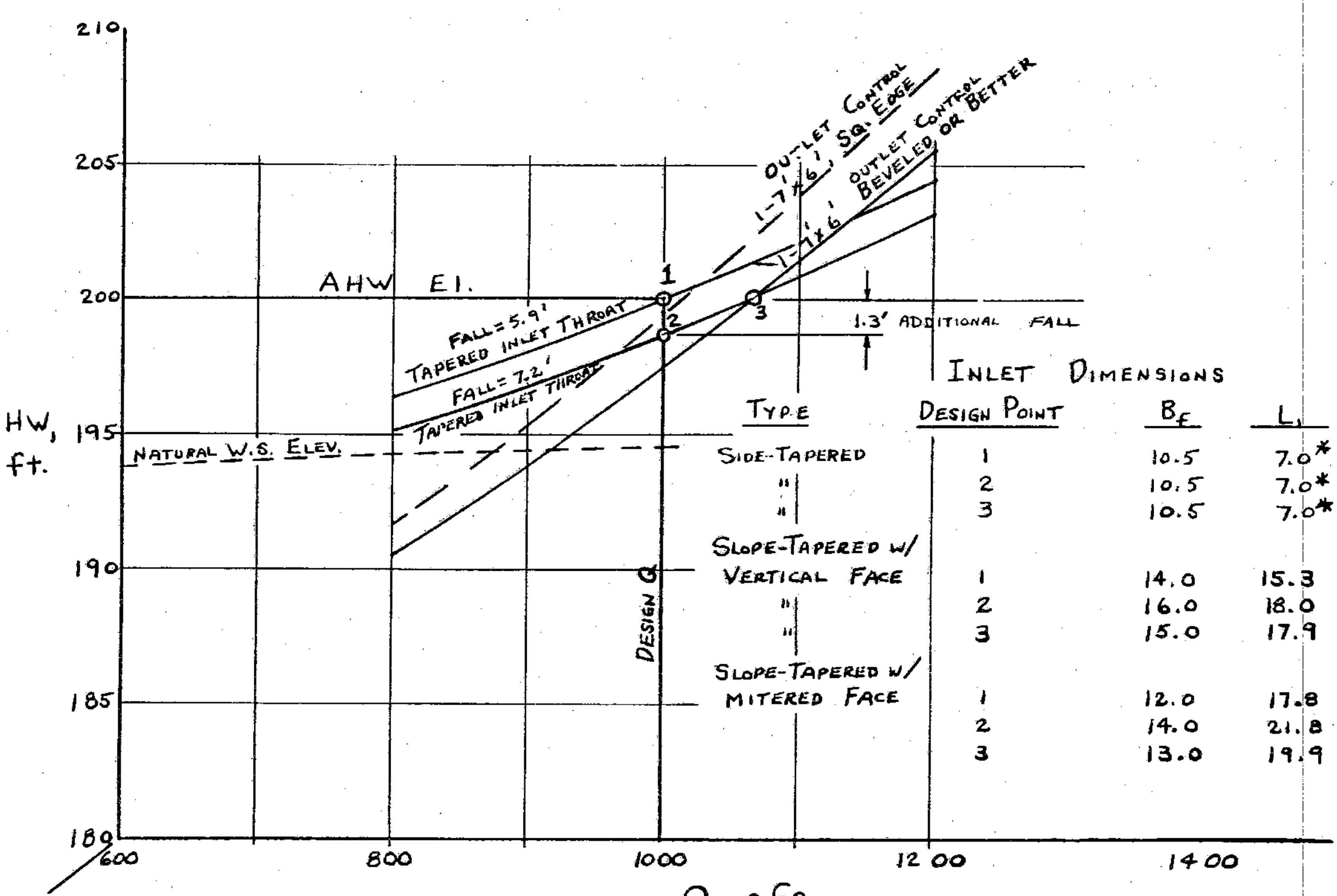
(8.) If (7) > (6), Adj. $L_3 = \left[\frac{B_f - NB}{2}\right]$ TAPER $- L_2$

(9.) If (6) > (7) Adj. TAPER $= (L_2 + L_3) / \left[\frac{B_f - NB}{2}\right]$

(10.) $L_1 = L_2 + L_3 + L_4$

(11.) Mitered: $W = NB + 2\left[\frac{L_1}{TAPER}\right]$

(12.) Max. Crest El. $= HW_f - H_c$



Inlet Dimensions

Type	Design Point	B_f	L_1
Side-Tapered	1	10.5	7.0*
"	2	10.5	7.0*
"	3	10.5	7.0*
Slope-Tapered w/ Vertical Face	1	14.0	15.3
"	2	16.0	18.0
"	3	15.0	17.9
Slope-Tapered w/ Mitered Face	1	12.0	17.8
	2	14.0	21.8
	3	13.0	19.9

Design Performance Curves - 1-7'x6' RCB

Example 1

Conclusion

Since the requirements called for the smallest possible reinforced concrete box culvert, the barrel should be a single 7 ft x 6 ft.

Selection of the inlet would be based on cost. The additional 1.3 ft of FALL gains 62 cfs at AHW EL. = 200.0, but this is not significant at this site. It appears that a side- or slope-tapered design meeting the Q and HW requirements of point 1 would be adequate and the least expensive.

Examination of the outlet control curve shows that a discharge of 1,200 cfs (20% above design) results in an AHW EL. 5.5 ft above design. At this site, no serious flooding of upstream property or the roadway will be caused by such a headwater, and no larger barrel is required.

The dimensions of several alternate inlet structure designs are presented, based on points 1, 2, and 3 on the culvert performance curves. Note that the side-tapered inlets remain about the same size for all FALL values, while the slope-tapered inlets increase in size as FALL increases. However, the side-tapered inlets require an increasingly larger upstream sump as FALL increases. Which design will be more favorable will be a matter of economics and site considerations.

Example 2

Pipe Culvert

Given: Design Discharge, Q_{10} = 150 cfs

Allowable Headwater Elevation = 100.0 ft msl

Elevation Outlet Invert = 75.0 ft msl

Culvert Length, L_a = 350 ft

Downstream channel approximates a 5-ft-wide trapezoidal channel with 2:1 side slopes, a Manning's "n" of 0.03, and S_o = 0.05.

Requirements: This pipe culvert is located in a suburban area where the AHW EL. may be exceeded by 2 to 3 ft without extreme damage. However, headwater elevations greater than 103.0 ft should be avoided for flows significantly higher than the design Q of 150 cfs.

PROJECT: *Example No. 2*

OUTLET CONTROL
DESIGN CALCULATIONS

DESIGNER: *JMN*

STATION: ______

DATE: *12-10-73*

INITIAL DATA:

Q *0* = *150* cfs

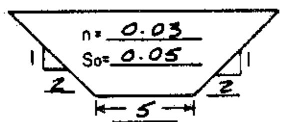
AHW El. = *100* ft.

S_o = *0.05*

L_a = *350* ft.

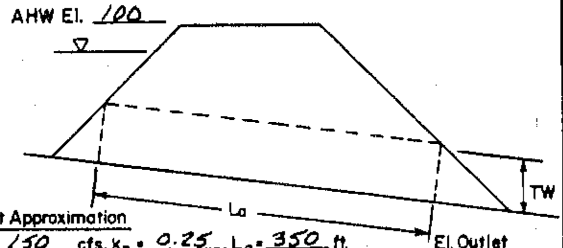
El. Outlet Invert *75* ft.

Stream Data:



Barrel Shape and Material *Circular CMP* Barrel n = *0.024*

SKETCH



First Approximation

Q = *150* cfs, k_e = *0.25*, L_a = *350* ft.

H = AHW El. − El. Outlet Invert − h_o

= *100′ − 75′ − 5′ = 20′*

∴ A = ______ ft.² or D = *46″* ft.; Try *42″*

Q	Q/N	* H	Q/NB	(1) d_c	$\frac{d_c+D}{2}$	Qn	(2) TW	(3) h_o	(4) HW_o	(5) V_o	COMMENTS
Trial No. *1*, N = *1*, B = *—*, D = *3.5′*, k_e = *0.25*											
150	*150*	*31*	*150*	*>3.5*	*3.5*	*—*	*1.6*	*3.5*	*109.5*		*75+31+3.5 = 109.5*
											HW_o > AHW El. Try 48″
Trial No. *2*, N = *1*, B = *—*, D = *4′*, k_e = *0.25*											
150	*150*	*15.6*	*150*	*3.6*	*3.8*	*—*	*1.6*	*3.8*	*94.4*		*Ok – Check square edge*
100	*100*	*7.0*	*100*	*3.1*	*3.5*	*—*	*1.4*	*3.5*	*85.5*		
200	*200*	*27.8*	*200*	*>4*	*4.0*	*—*	*1.9*	*4.0*	*106.8*		
Trial No. *3*, N = *1*, B = *—*, D = *4′*, k_e = *0.5*											
150	*150*	*16.2*					*1.6*	*3.8*	*95.0*		*From inlet Control section*
100	*100*	*7.2*	*Same as trial #2*				*1.4*	*3.5*	*85.7*		*Calculations, FALL req'd*
200	*200*	*28.8*					*1.9*	*4.0*	*107.8*		*∴ Use improved inlet*

Notes and Equations:

(1) d_c cannot exceed D

(2) TW based on d_n in natural channel, or other downstream control.

(3) $h_o = \frac{d_c+D}{2}$ or TW, whichever is larger.

(4) $HW_o = H + h_o +$ El. Outlet Invert.

(5) Outlet Velocity (V_o) = Q/Area defined by d_c or TW, not greater than D. Do not compute until control section is known.

SELECTED DESIGN

N = *1*

B = *—* ft.

D = *4* ft.

k_e = *0.25 or 0.5*

At Design Q:

HW_o = *94.4* ft.

V_o = ______ f/s

$$* \quad H = \left[1 + k_e + \frac{29n^2 \cdot L}{R^{1.33}}\right] \frac{V^2}{2g}$$

PROJECT: *Example No. 2* | CULVERT INLET CONTROL SECTION DESIGN CALCULATIONS | DESIGNER: *JMN*

STATION: ______ | DATE: *12-10-73*

INITIAL DATA:

Q *0* = *150* cfs

AHW El. = *100* ft.

S_o = *0.05*

L_a = *350* ft.

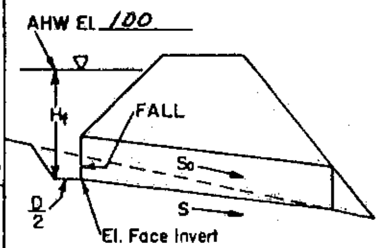
El. Stream Bed at Face *92.5* ft.

Barrel Shape and Material *Circ. CMP* Barrel n = *0.024*

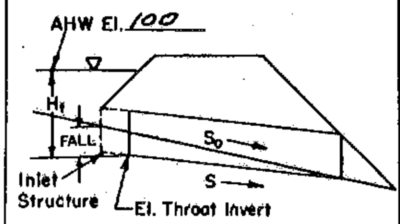
N = *1*, B = *—*

D = *4'*, $NBD^{3/2}$ = ______

(Pipe) $ND^{5/2}$ = *32* *(Table 5)*



CONVENTIONAL or BEVELED INLET: FACE CONTROL SECTION (Upper Headings)



TAPERED INLET THROAT CONTROL SECTION (Lower Headings)

DEFINITIONS OF INLET CONTROL SECTION

Note: Use Upper Headings for Coventional or Beveled Face; Lower Headings for Tapered Inlet Throat.

Q	$\frac{Q}{NB}$ / $\frac{Q}{NBD^{3/2}}$	$\frac{H_f}{D}$ / $\frac{H_t}{D}$	H_f / H_t	(1) El. Face Invert / El. Throat Invert	El. Stream Bed At Face	(2) FALL	(3) HW_f / HW_t	(4) S	(5) V_o	COMMENTS
Trial No. *1* Inlet and Edge Description *Square edges*										
150	*150*	*2.07*	*8.3*	*91.7*	*92.5*	*0.8*	*100.0*	*0.048*		*FALL required, use bevels*
Trial No. *2* Inlet and Edge Description *Beveled edges*										
150	*150*	*1.92*	*7.7*	*92.3*	*92.5*	*~0*	*100*	*0.05*	*HDS 3* *16*	*Check tapered inlet*
100	*100*	*1.25*	*5.0*				*97.3*			*throat*
200	*200*	*2.90*	*11.6*				*103.9*			
Trial No. *3* Inlet and Edge Description *Tapered inlet throat, rough*										
150	*4.7*	*1.65*	*6.6*	*92.5*	*92.5*	*0*	*99.1*	*0.05*	*16*	*Increases Q at AHW*
100	*3.1*	*1.21*	*4.8*				*97.3*			*El. from 100 to 170 c.f.s.*
200	*6.2*	*2.22*	*8.9*				*101.4*			

Notes and Equations:

(1) El. Face (or throat) invert = AHW El. − H_f (or H_t)

(2) FALL = El. Stream Bed at Face − El. face (or throat) invert

(3) HW_f (or HW_t) = H_f (or H_t) + El. face (or throat) invert, where El. face (or throat) invert should not exceed El. stream bed.

(4) $S \approx S_o - FALL/L_a$

(5) Outlet Velocity = Q/Area defined by d_n at S

SELECTED DESIGN

Inlet Description: *Beveled edges*

FALL = *0* ft.

Invert El. = *92.5* ft.

Bevels:

Angle = *45°*

b = ______ in., d = *2* in.

PROJECT: *Example No. 4*

SIDE-TAPERED INLET
DESIGN CALCULATIONS

DESIGNER: *JMN*

STATION: ______

DATE: *12-10-73*

INITIAL DATA

$Q_{\ 0}$ = *150* cfs S_o = *0.05*

AHW El. = *100* ft. L_a = *350* ft.

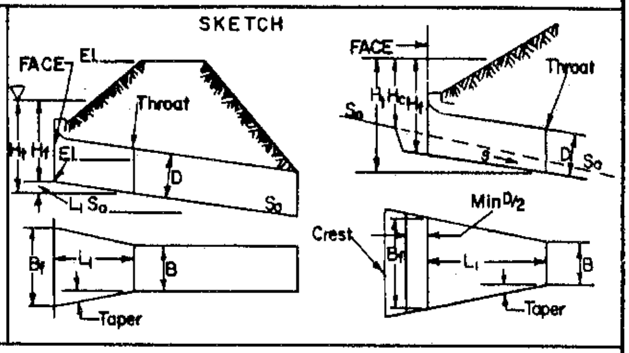
TAPER = *4* : 1

Barrel Shape and Material *Circular CMP*

Face Edge Description *45° Bevels*

N = *1*, B = *–* ft, D = *4'* ft

SKETCH



		(1)			(2)		(3)	(4)		(5)	Upper Headings for Box Culverts, Lower Headings for Pipes
		~~H_f/D~~	~~$\frac{Q}{B_f D^{3/2}}$~~	~~$D^{3/2}$~~	~~Min. B_f~~						
Q	El. Throat Invert	$\frac{H_f}{E}$	$\frac{Q}{A_f E^{1/2}}$	$E^{1/2}$	Min. A_f	B_f	L_1	S	L_1 S	El. Face Invert	COMMENTS
Trial No. *1*, Q = *150*, HW_f = *99.1* *(Use lower column headings)*											
150	*92.5*	*1.4*	*4.0*	*2.0*	*18.8*	*6.0*	*4.0*	*0.05*	*0.2*	*92.7*	~~$B_f D^{3/2}$~~ [or $A_f E^{1/2}$] = *18.85*
											Std. design: B_f = 1.5D
											= 6' ∴ std. design O.k.
Trial No. ___, Q = ___, HW_f = ___											
											$B_f D^{3/2}$ [or $A_f E^{1/2}$] = ___
Trial No. ___, Q = ___, HW_f = ___											
											$B_f D^{3/2}$ [or $A_f E^{1/2}$] = ___

Notes and Equations: *(99.1 − 92.5 − 1)/4 = 1.4*

(1) H_f/D [or H_f/E] = (HW_f − El. Throat Invert − 1)/D [or E]
   $D \le E \le 1.1D$

(2) Min. $B_f = Q / (D^{3/2}) \left[\frac{Q}{B_f D^{3/2}}\right]$

   Min. $A_f = Q / (E^{1/2}) \left[\frac{Q}{A_f E^{1/2}}\right]$

(3) $L_1 = \left[\frac{B_f - NB}{2}\right]$ TAPER   *$\left[\frac{6.0 - 4.0}{2}\right] 4 = 4.0'$*

(4) From throat design

(5) El. Face Invert − El. Throat Invert > 1 ft., recompute.
   Face and Throat may be lowered to better fit site, but do not raise.

SELECTED DESIGN

B_f = *6.0* ft.

L_1 = *4.0* ft.

Bevels: Angle *45* °

d = *–* in., b = *3* in.

Crest Check:

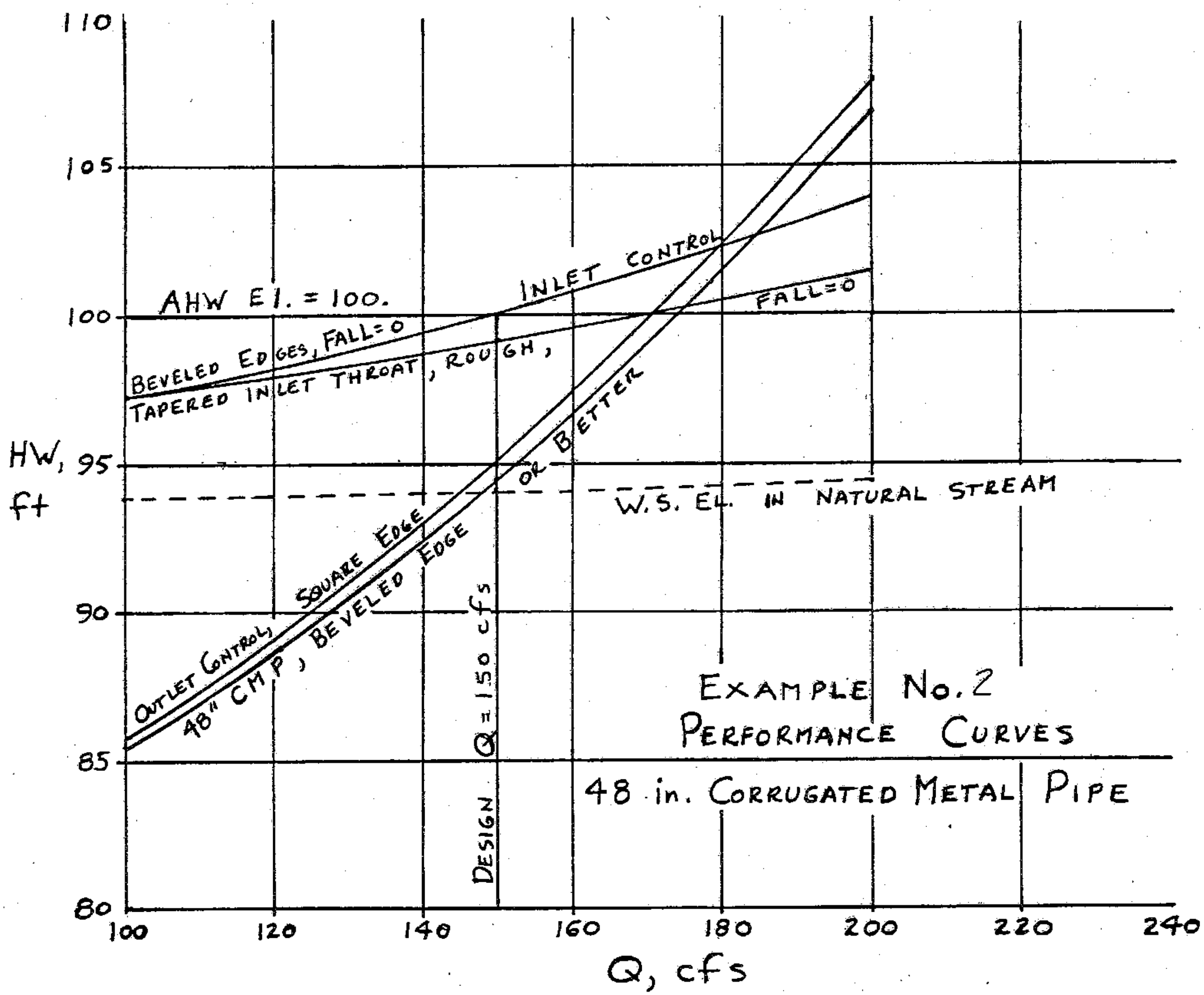
HW_c = *99.1* ft.   *99.1*

H_c = *6.1* ft.   *−93.0*

6.1

Q/W = *44.0* (Chart 17)

Min. W = *3.4* ft.


EXAMPLE No. 2
PERFORMANCE CURVES
48 in. CORRUGATED METAL PIPE
HW, ft
Q, cfs
AHW El. = 100.
INLET CONTROL
BEVELED EDGES, FALL=0
TAPERED INLET THROAT, ROUGH,
FALL=0
OR BETTER
W.S. EL. IN NATURAL STREAM
OUTLET CONTROL, SQUARE EDGE
48" CMP, BEVELED EDGE
DESIGN Q = 150 cfs
110
105
100
95
90
85
80
100
120
140
160
180
200
220
240

Example 2

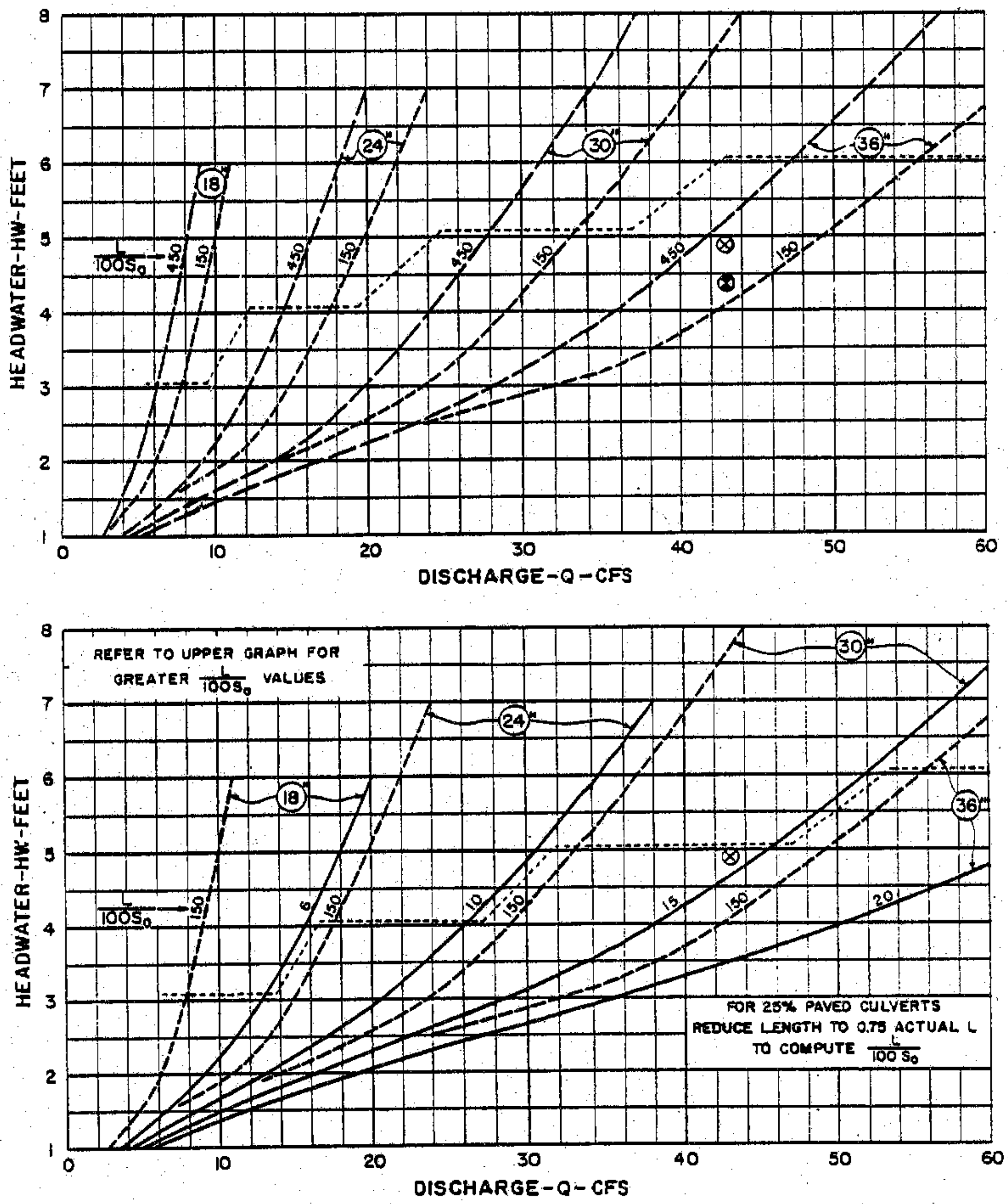
Conclusion

From the performance curves, beveled edges meet the AHW EL. of 100 ft and Q = 150 cfs, while the use of a side-tapered inlet would increase Q to 170 cfs at AHW El. = 100 ft. In both cases, the FALL = 0. It appears that the beveled-edge inlet would be sufficient and the least costly in this case, since the culvert performance curve does not exceed 103.0 ft until Q is 186 cfs.

For additional examples consult Appendix A of Reference 4.

Design Figures

Capacity charts and nomographs covering the range of applications commonly encountered in urban drainage are contained in this section. These charts are from Federal Highway Administration publications. For situations beyond the range covered by these charts reference should be made to the original publications.



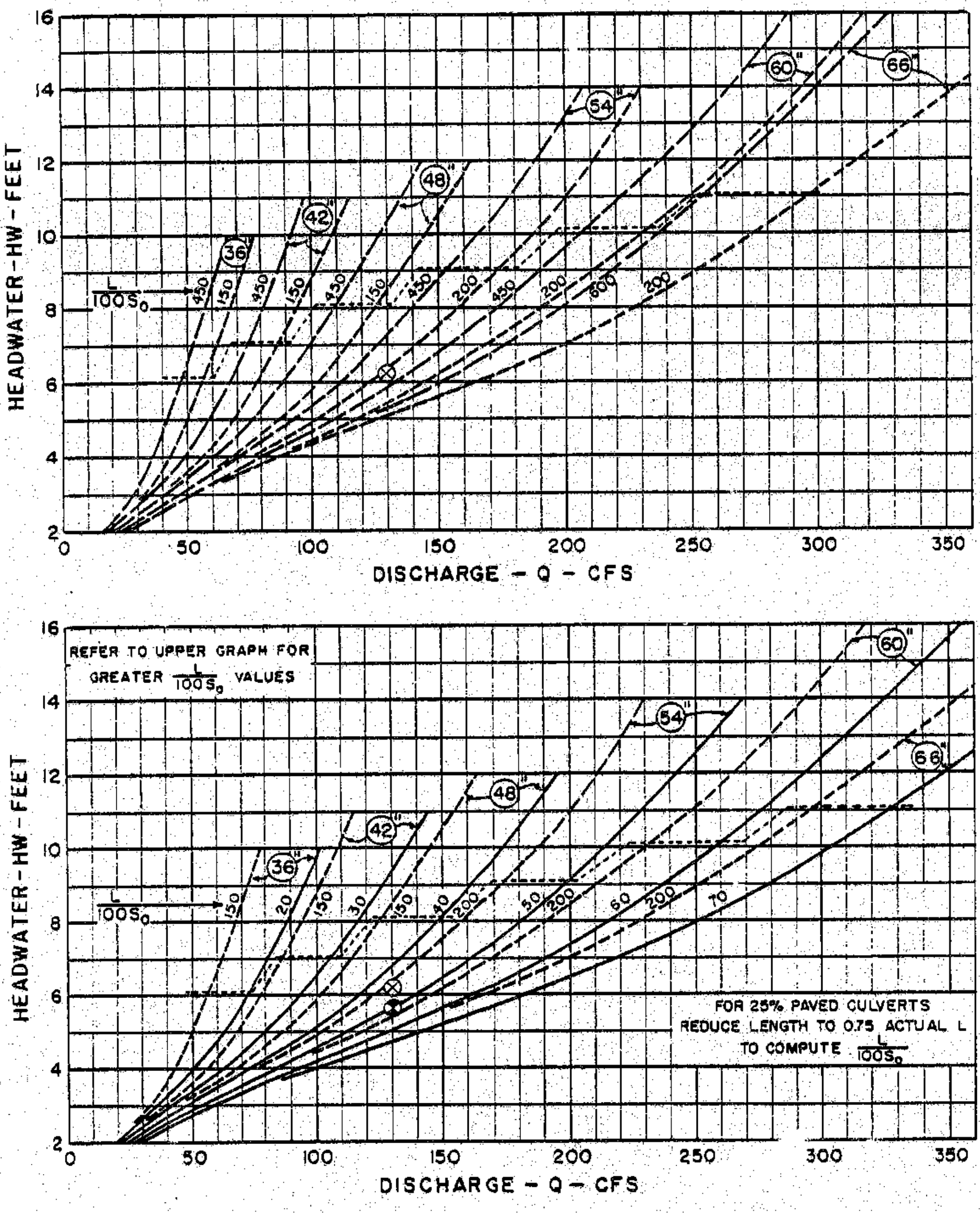
EXAMPLE

⊗ GIVEN:
43 CFS; AHW = 4.9 FT.
L = 72 FT.; S_0 = 0.003

⊕ SELECT 36" UNPAVED
HW = 4.4 FT.

STANDARD
CIRCULAR CORR. METAL PIPE
HEADWALL ENTRANCE
18" TO 36" ○

Figure 6-13 Culvert Capacity



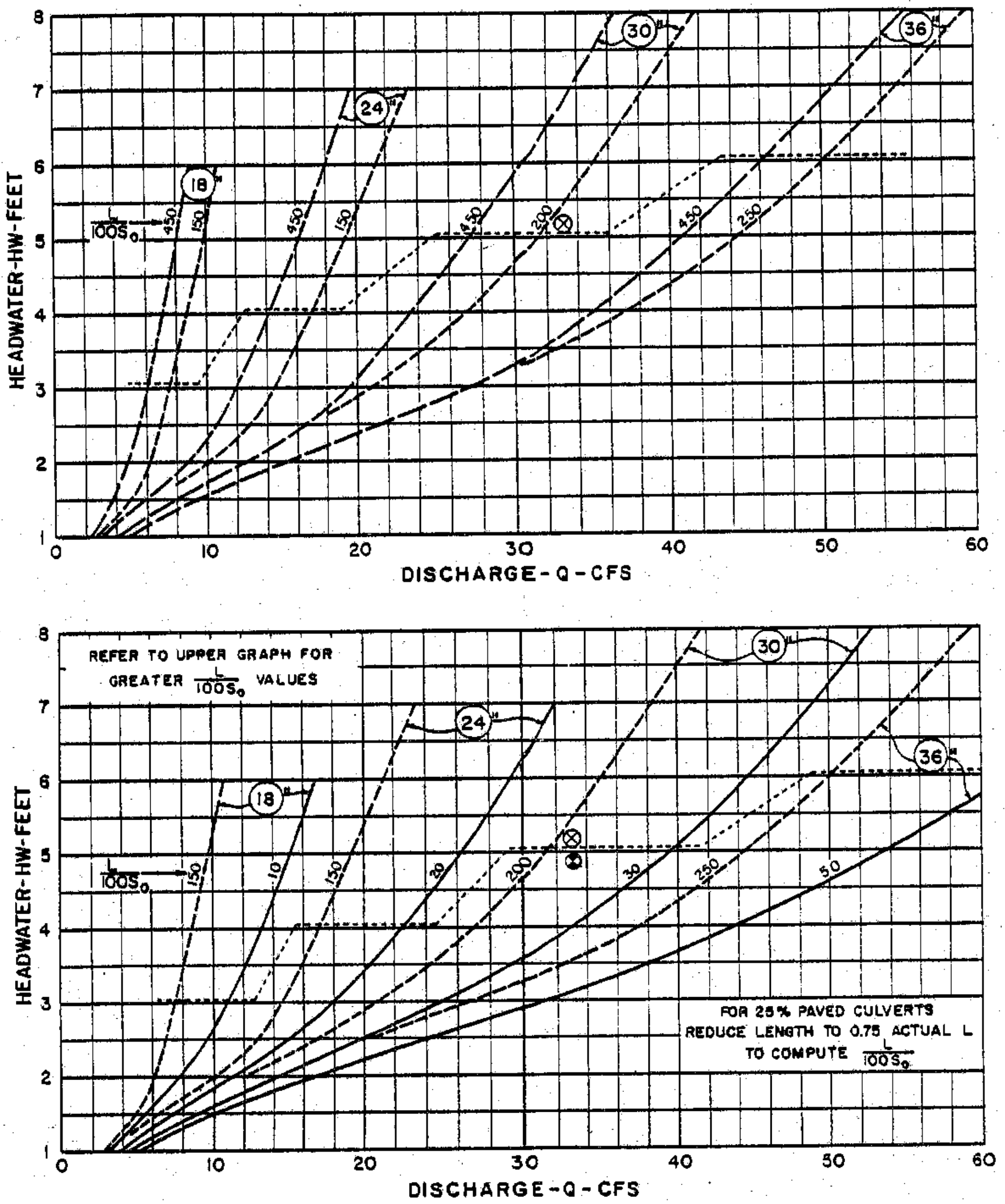
EXAMPLE

⊗ GIVEN:
130 CFS; AHW = 6.2 FT.
L = 120 FT.; S_0 = 0.025

② SELECT 54" UNPAVED
HW = 5.6 FT.

STANDARD
CIRCULAR CORR. METAL PIPE
HEADWALL ENTRANCE
36" TO 66" ○

Figure 6-14 Culvert Capacity



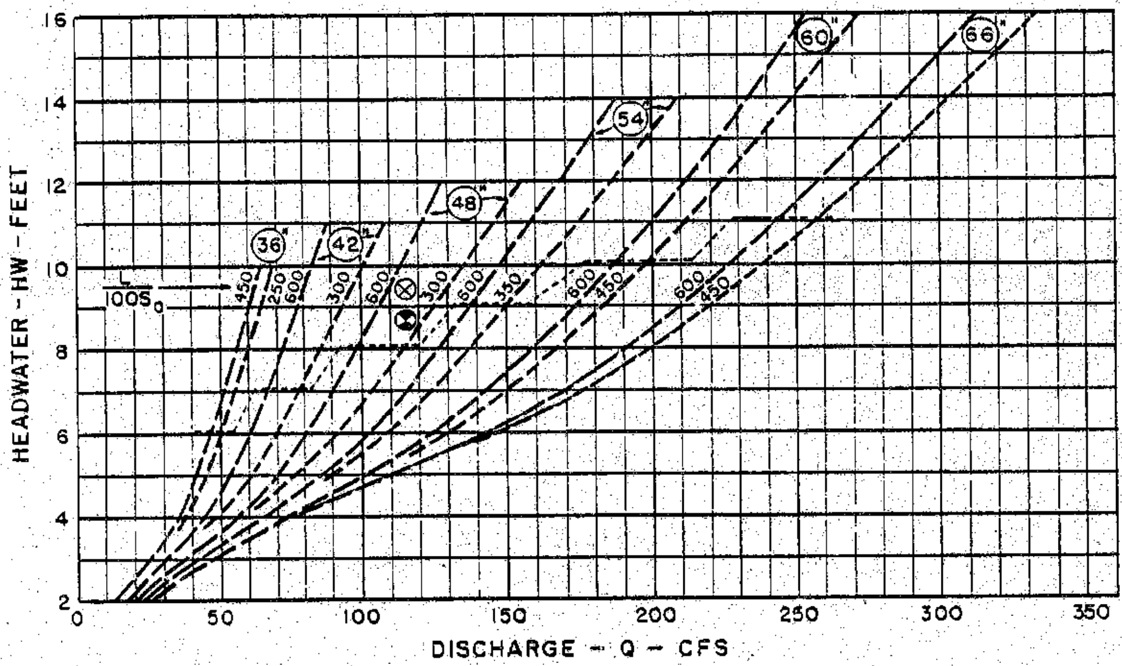
EXAMPLE

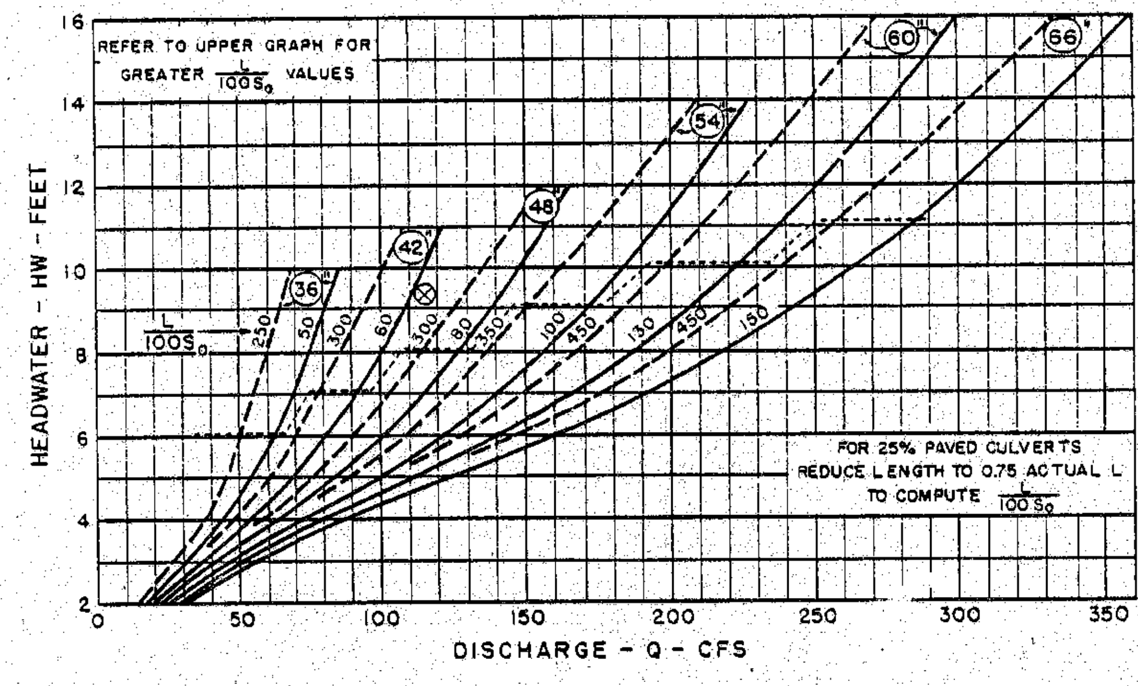
⊗ GIVEN:
33 CFS; AHW = 5.2 FT.
L = 70 FT.; S_o = 0.005

⊗ SELECT 30" UNPAVED
HW = 4.9 FT.

STANDARD
CIRCULAR CORR. METAL PIPE
PROJECTING ENTRANCE
18" TO 36" ○

Figure 6-15 Culvert Capacity





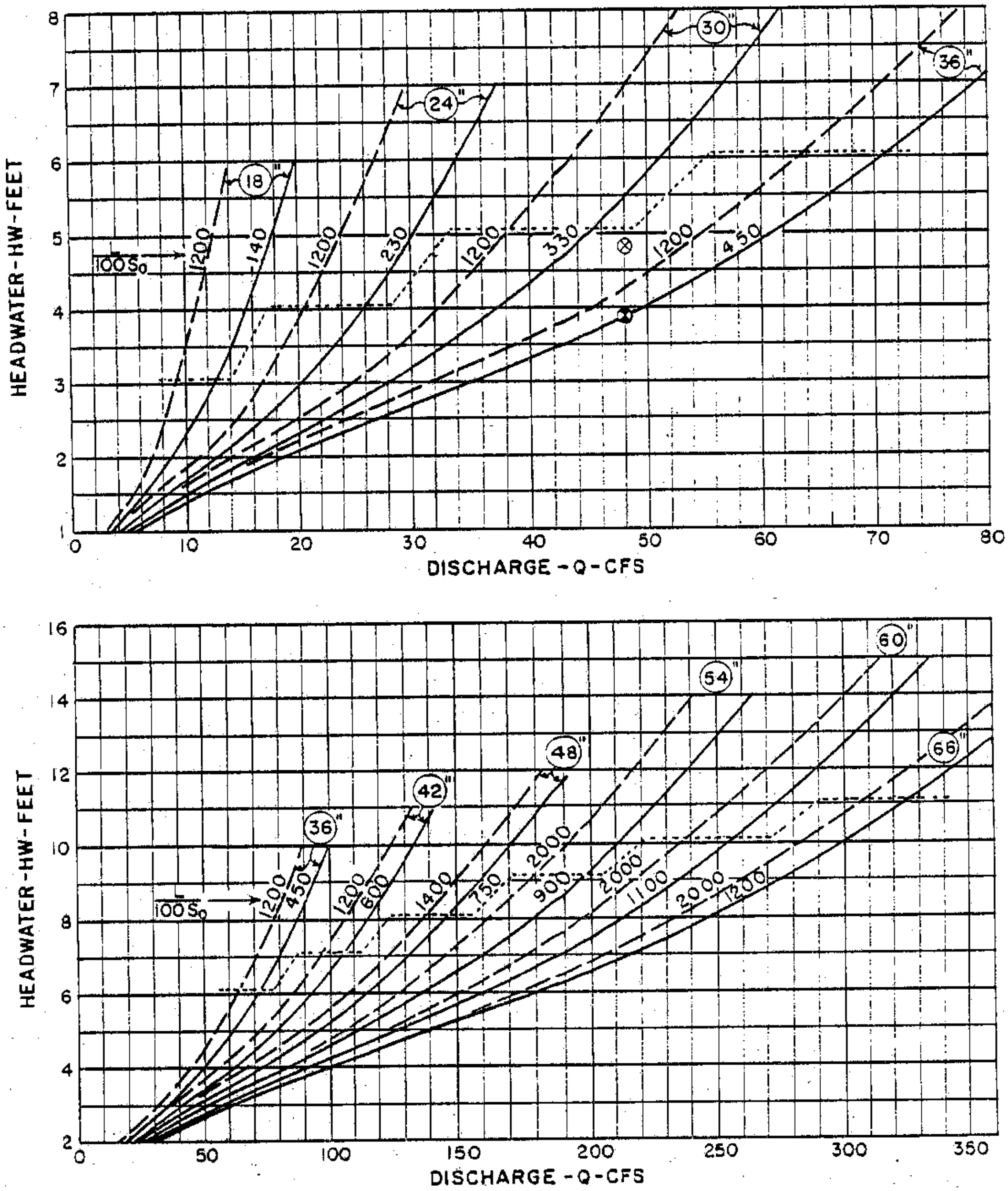
EXAMPLE

⊗ GIVEN:
115 CFS; AHW = 9.4 FT.
L = 135 FT.; S_0 = 0.0034

SELECT 48" UNPAVED
HW = 8.6 FT.

STANDARD
CIRCULAR CORR. METAL PIPE
PROJECTING ENTRANCE
36" TO 66" ○

Figure 6-16 Culvert Capacity



EXAMPLE

⊗ GIVEN:
48 CFS; AHW = 4.8 FT.
L = 60 FT.; S_0 = 0.003

SELECT 36"
HW = 3.9 FT.

CIRCULAR CONCRETE PIPE
SQUARE-EDGED ENTRANCE
18" TO 66"

Figure 6-17 Culvert Capacity

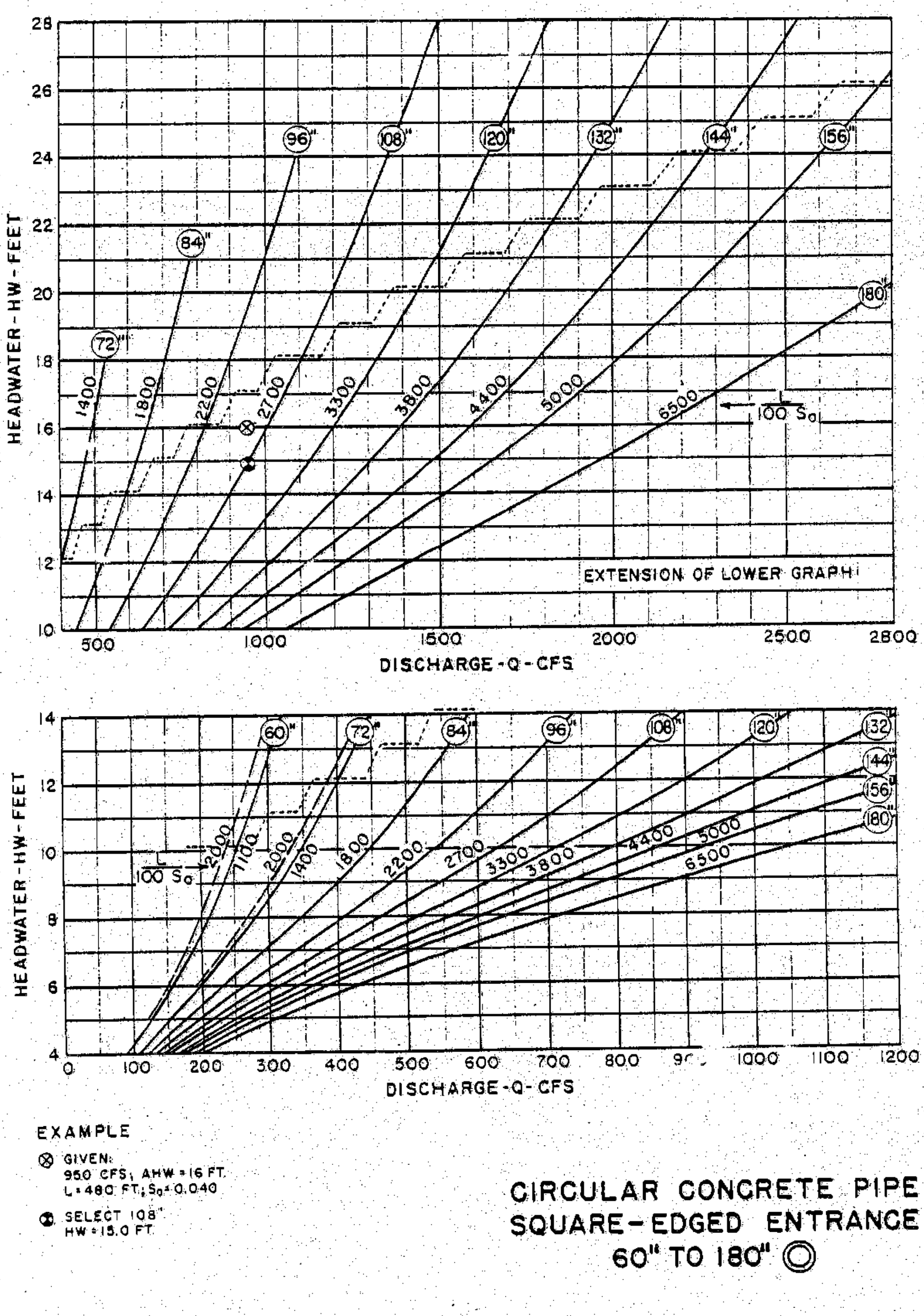


Figure 6-18 Culvert Capacity

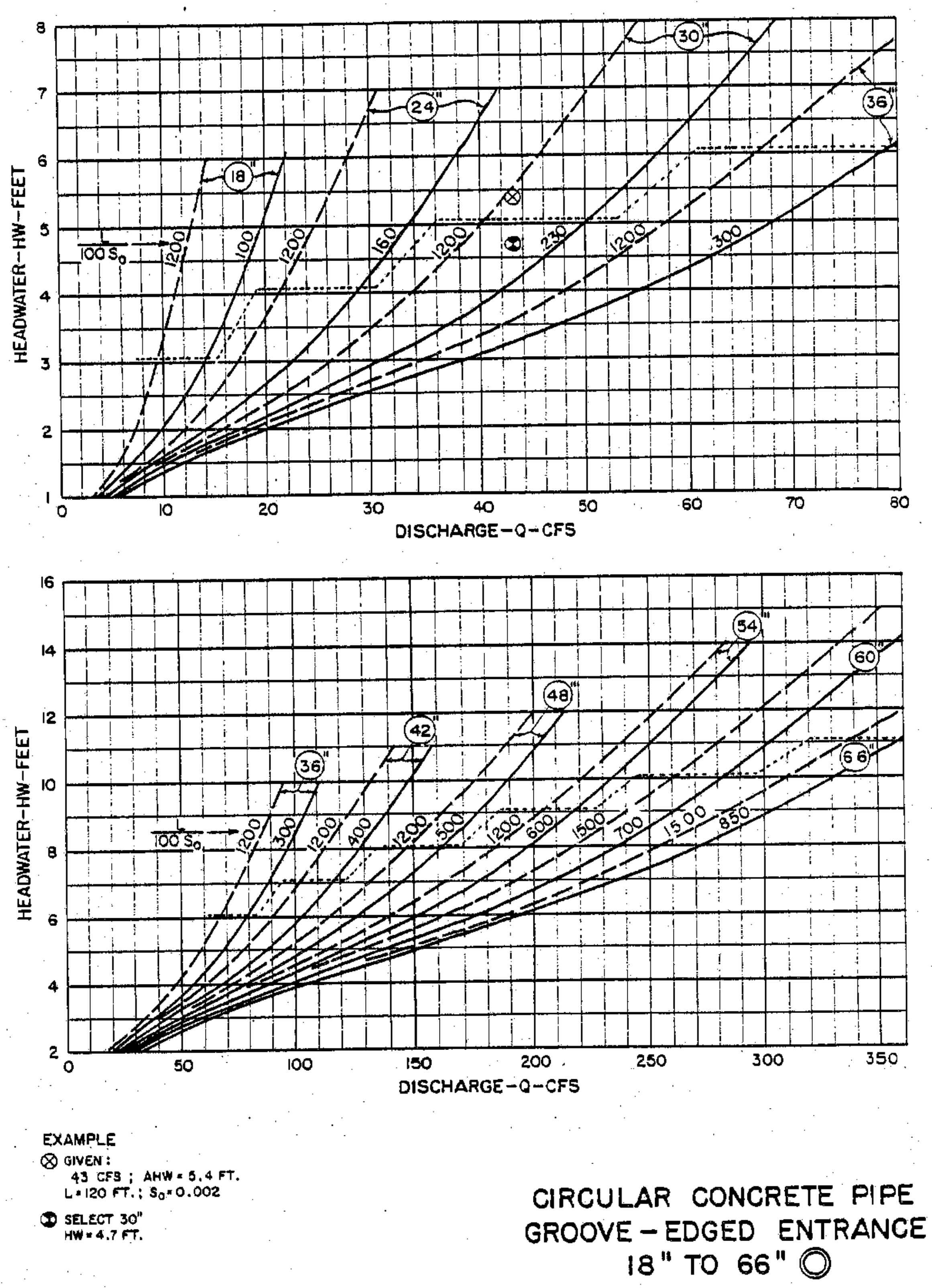
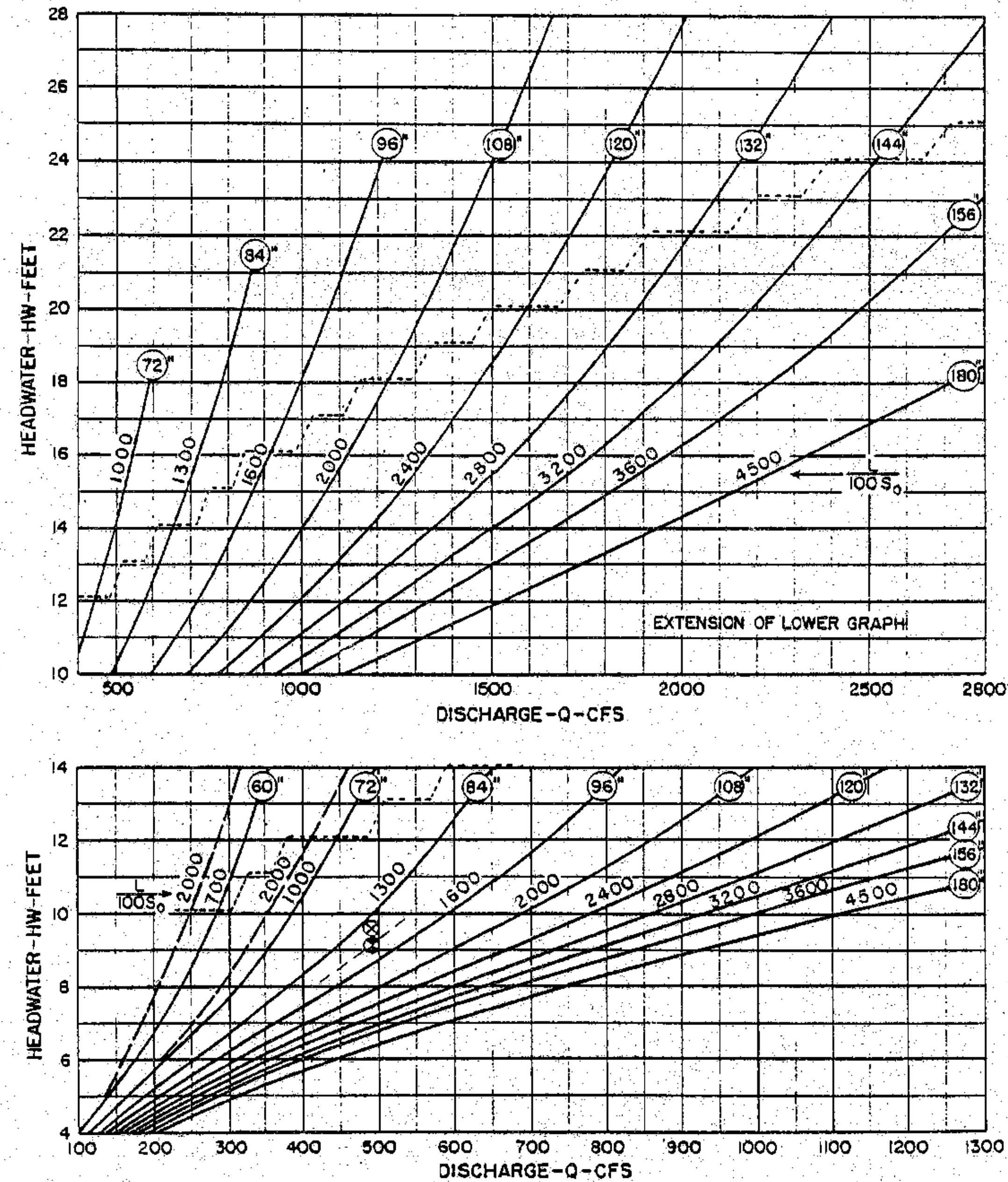


Figure 6-19 Culvert Capacity



EXAMPLE

⊗ GIVEN:
490 CFS ; AHW = 9.6 FT.
L = 60 FT. ; S_0 = 0.000

SELECT 90" ($\frac{L}{D}$ = 8)
HW = 9.2 FT.

CIRCULAR CONCRETE PIPE
GROOVE – EDGED ENTRANCE
60" TO 180" ◎

Figure 6-20 Culvert Capacity

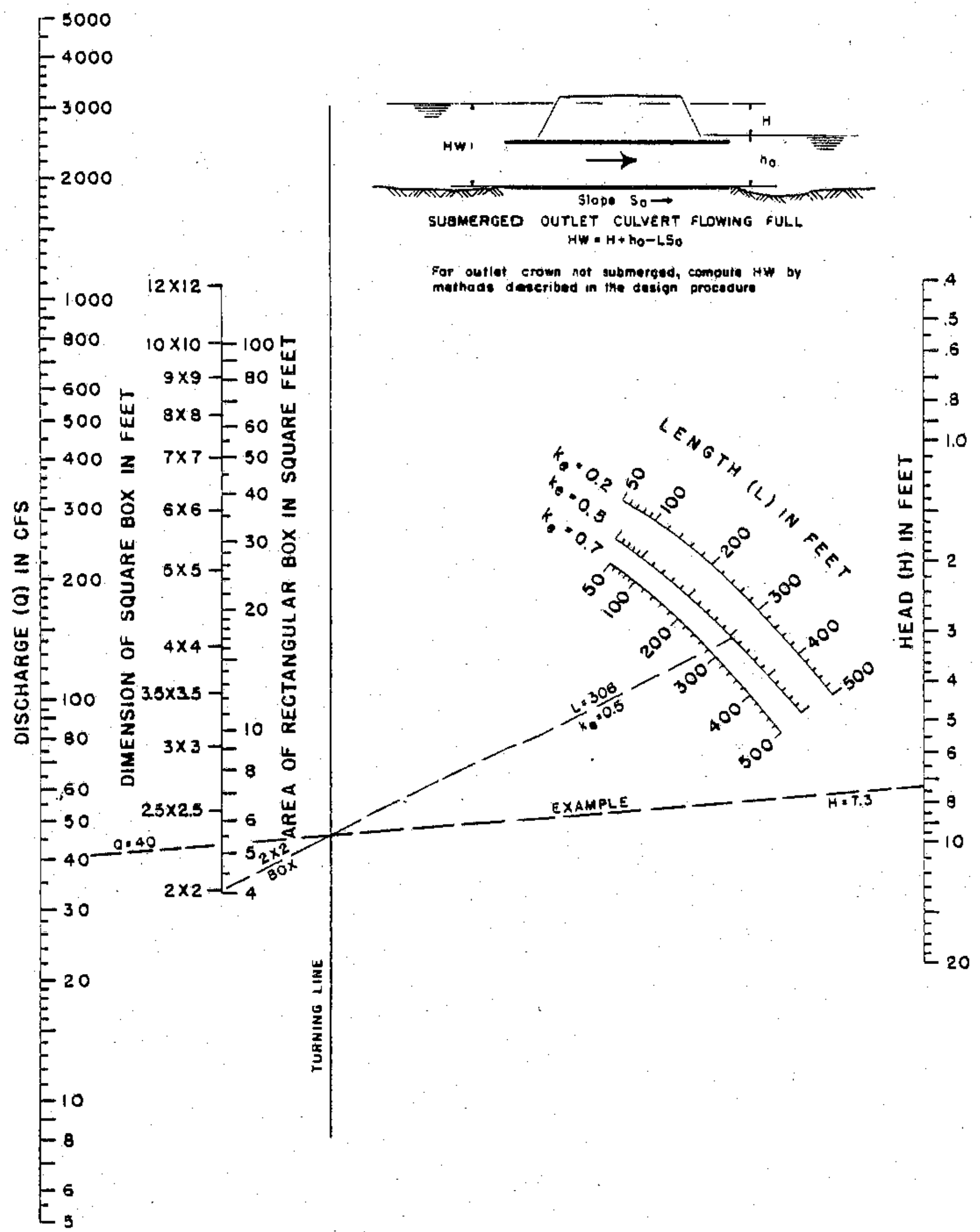
TABLE 6-2

Entrance Loss Coefficients

Outlet Control, Full or Partly Full

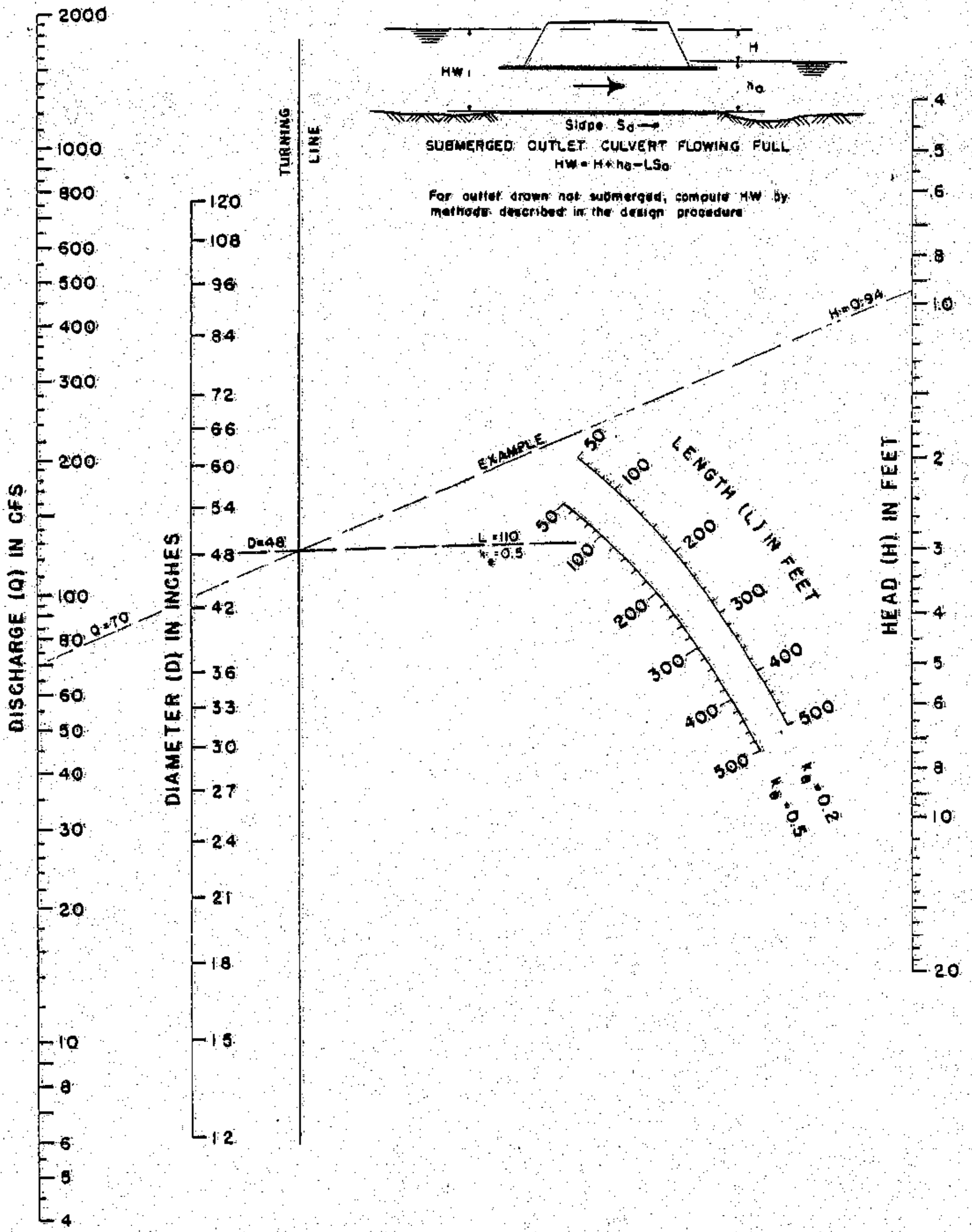
Type of Structure and Design of Entrance	Coefficient k_e
Pipe, Concrete	
Projecting from fill, socket end (groove end)	0.2
Projecting from fill, square-cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove end)	0.2
Square-edged	0.5
Rounded (radius = 1/12D)	0.2
Mitered to conform to fill slope	0.7
*End section conforming to fill slope	0.5
Beveled edges, 33.7-degree to 45-degree 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-degree to 45-degree 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 degrees to 75 degrees to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of 1/12 barrel dimension, or beveled top edge	0.2
Wingwall at 10 degrees to 25 degrees 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

*Note: End sections conforming to fill slope are the sections commonly available from manufacturers. From limited hydraulic tests they are equivalent in operation to a headwall in both inlet and outlet control. Some end sections incorporating a closed taper in their design have a superior hydraulic performance.



HEAD FOR
CONCRETE BOX CULVERTS
FLOWING FULL
n = 0.012

Figure 6-21 Outlet-Control Nomograph



HEAD FOR
CONCRETE PIPE CULVERT
FLOWING FULL
n = 0.012

Figure 6-22 Outlet-Control Nomograph

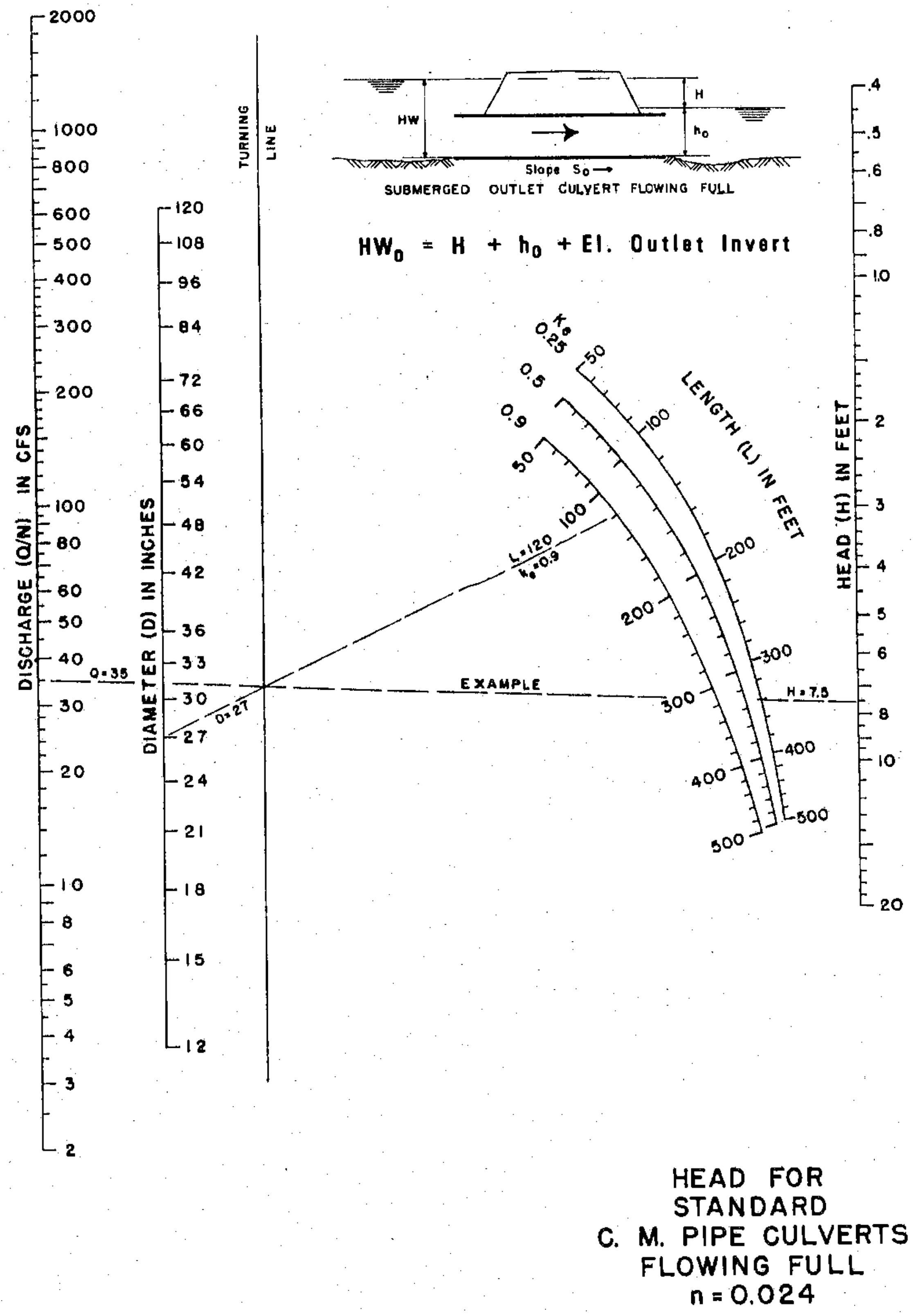


Figure 6-23 Outlet-Control Nomograph

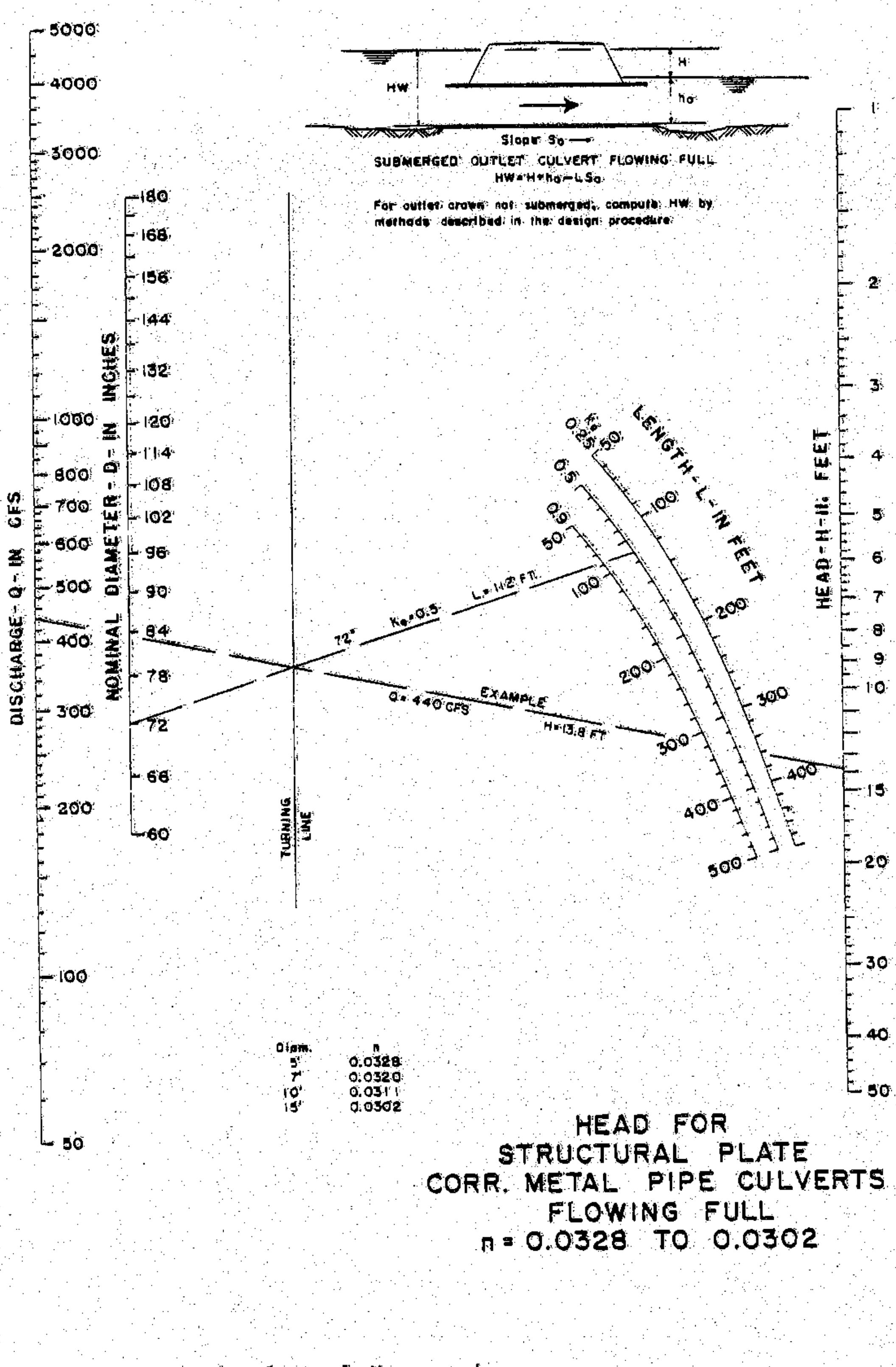
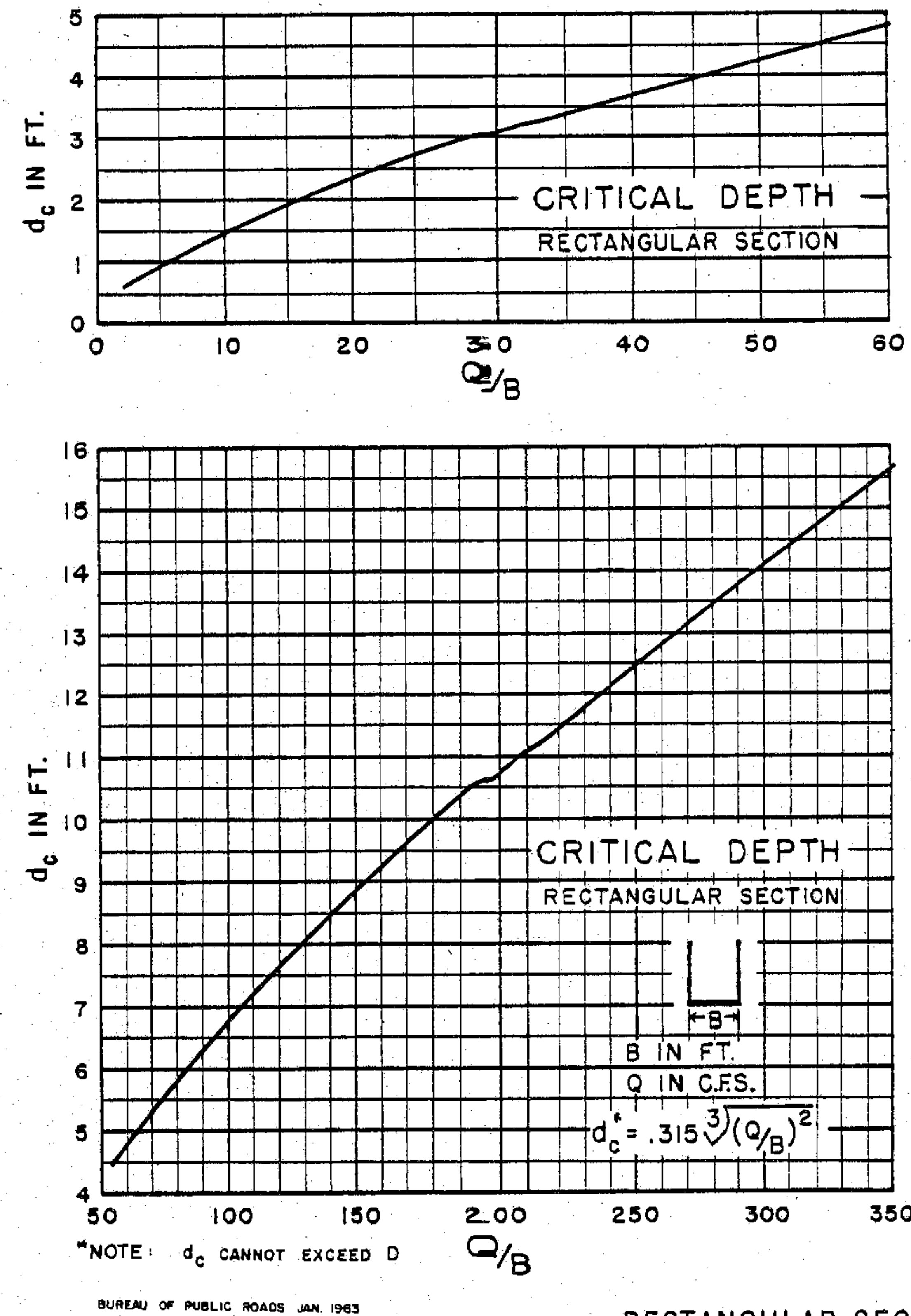


Figure 6-24 Outlet-Control Nomograph



RECTANGULAR SECTION

Figure 6-25 Critical-Depth Chart

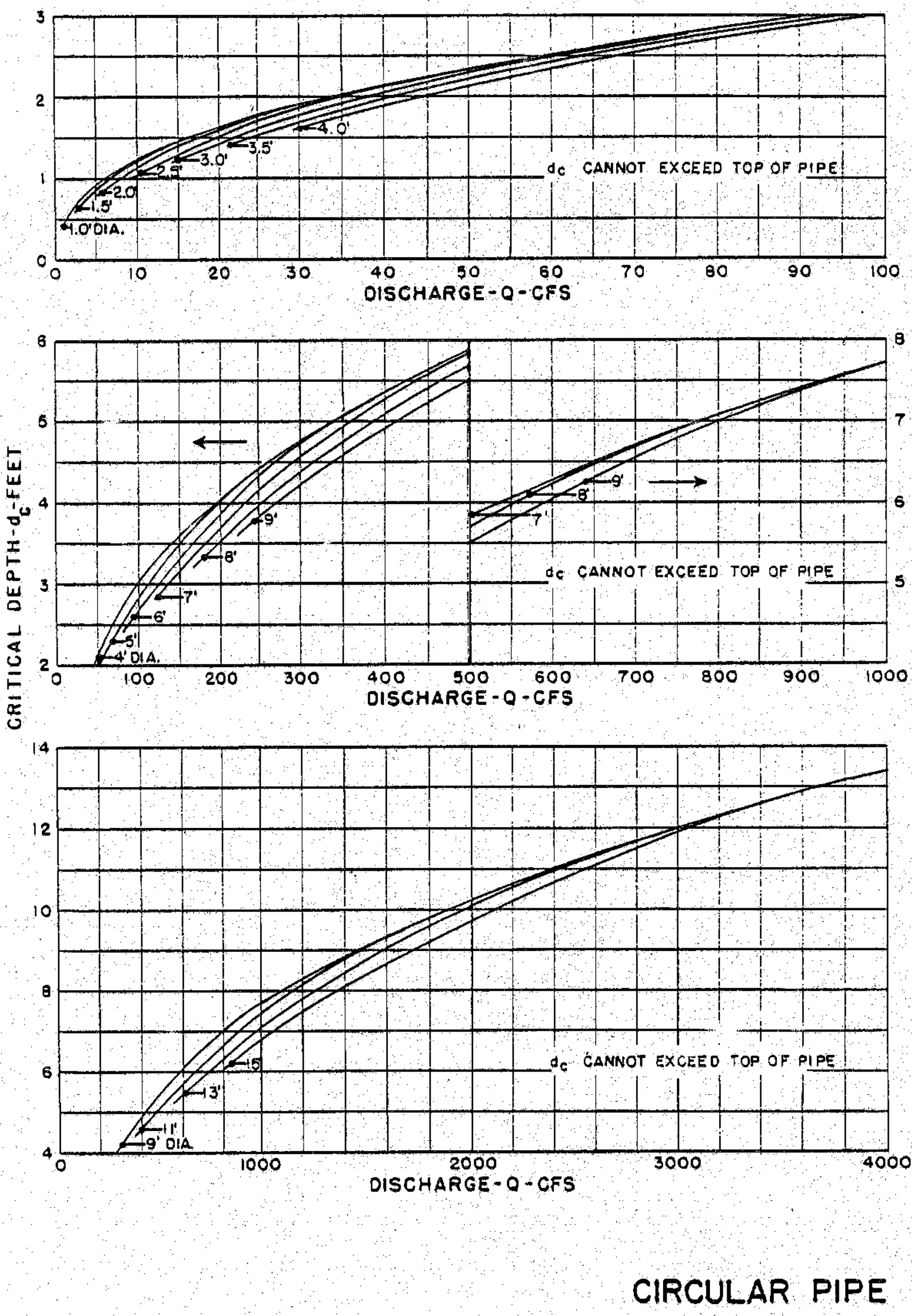


Figure 6-26 Critical-Depth Chart

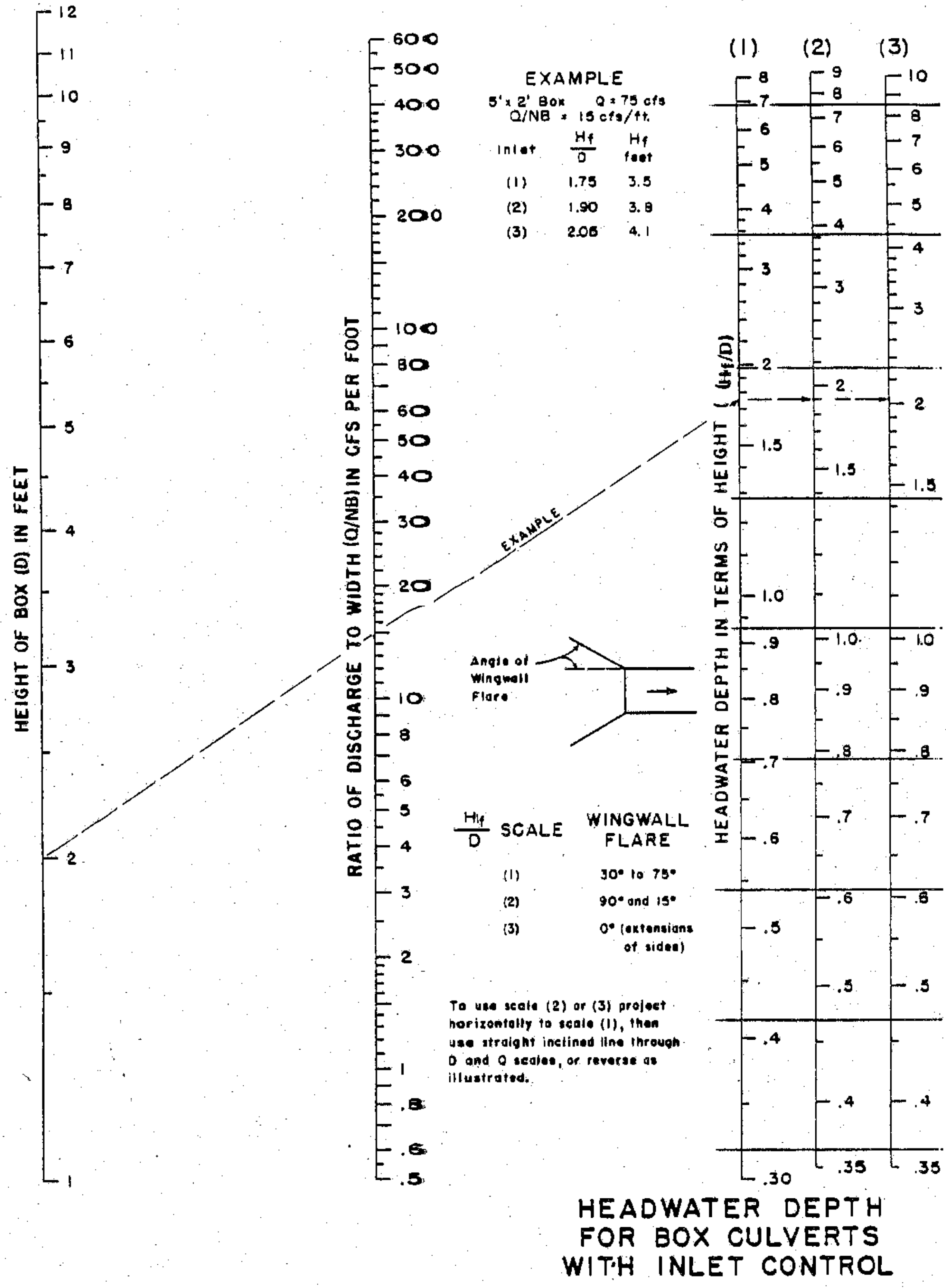
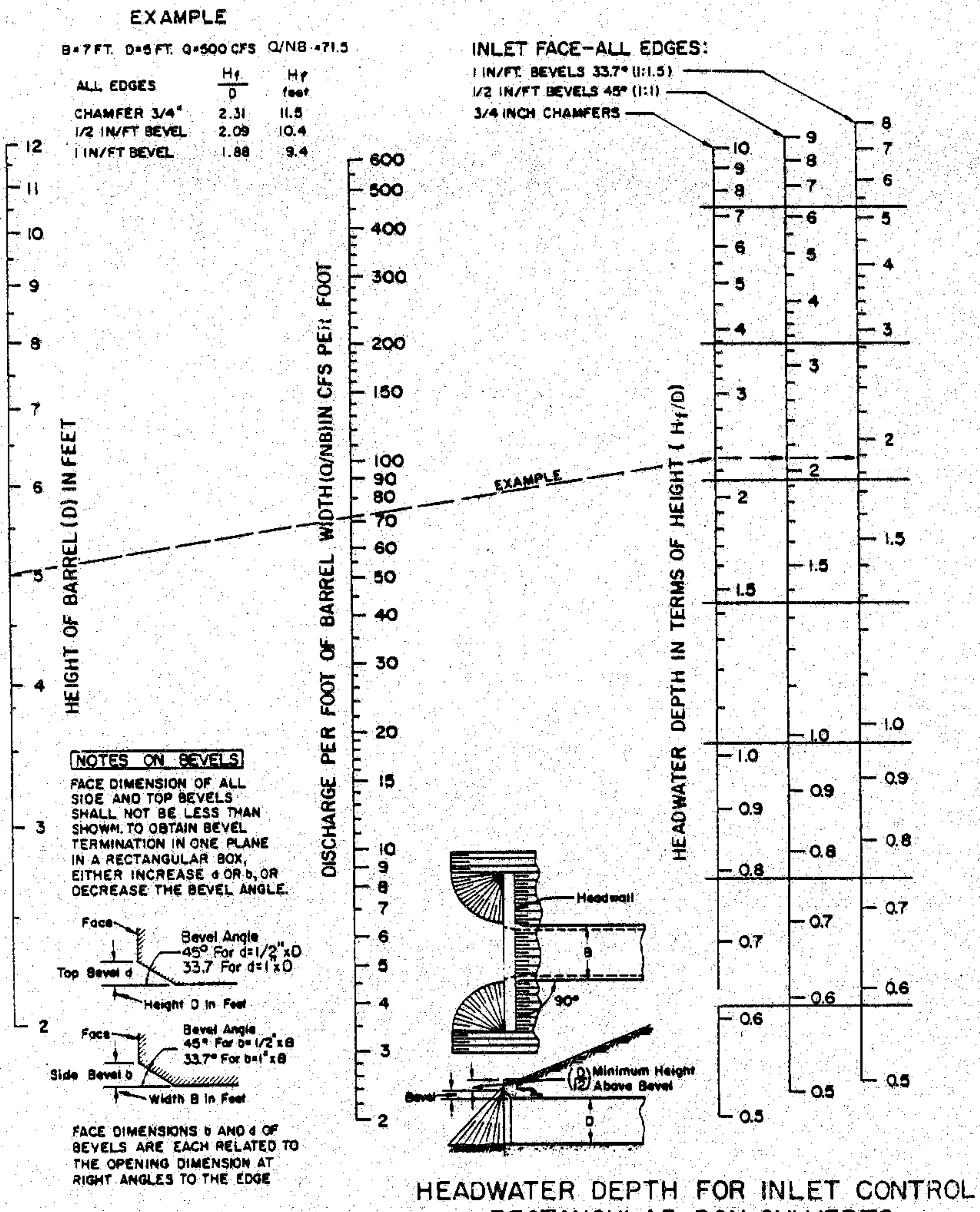


Figure 6-27 Inlet-Control Nomograph



HEADWATER DEPTH FOR INLET CONTROL
RECTANGULAR BOX CULVERTS
90° HEADWALL
CHAMFERED OR BEVELED INLET EDGES

Figure 6-28 Inlet-Control Nomograph

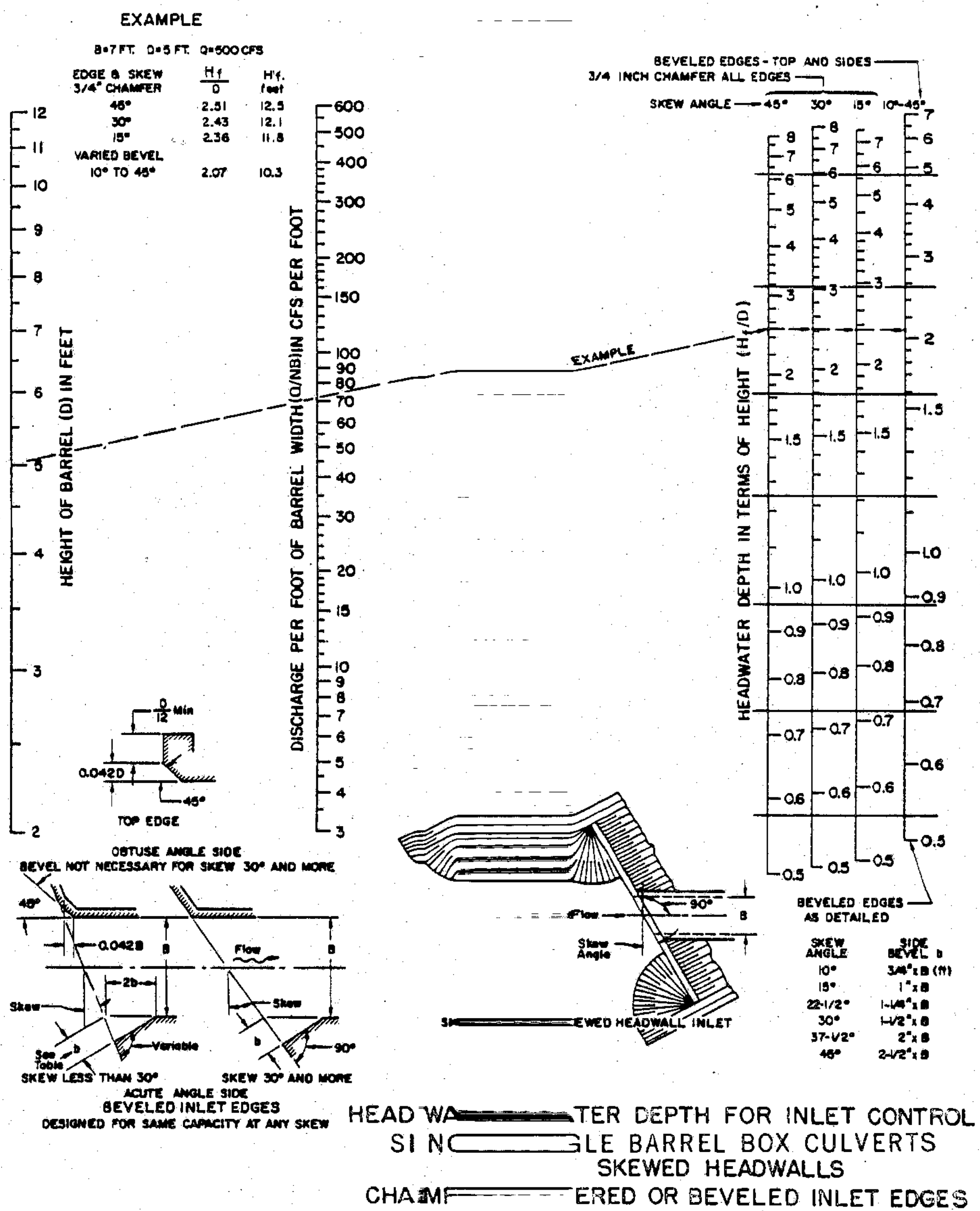
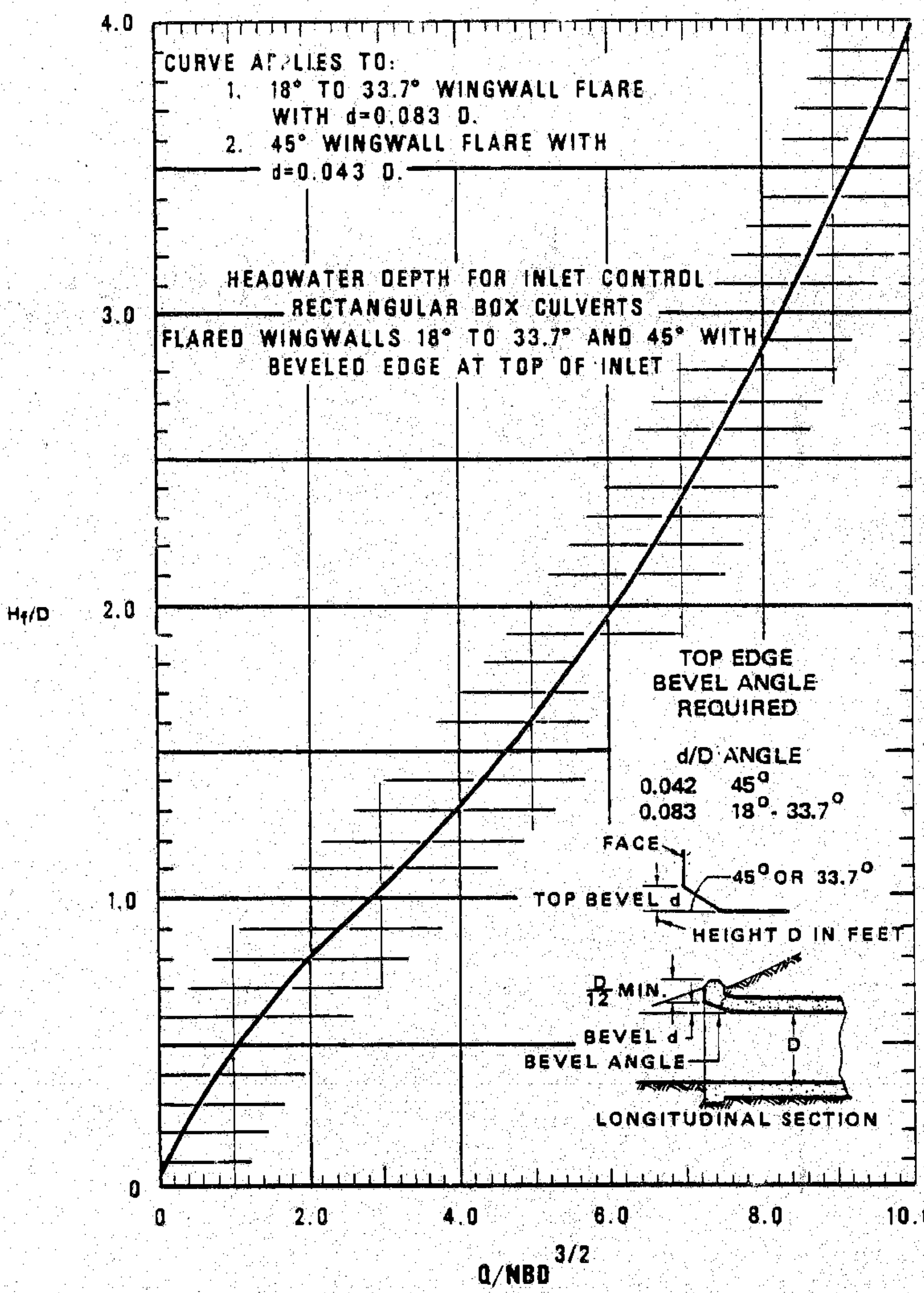
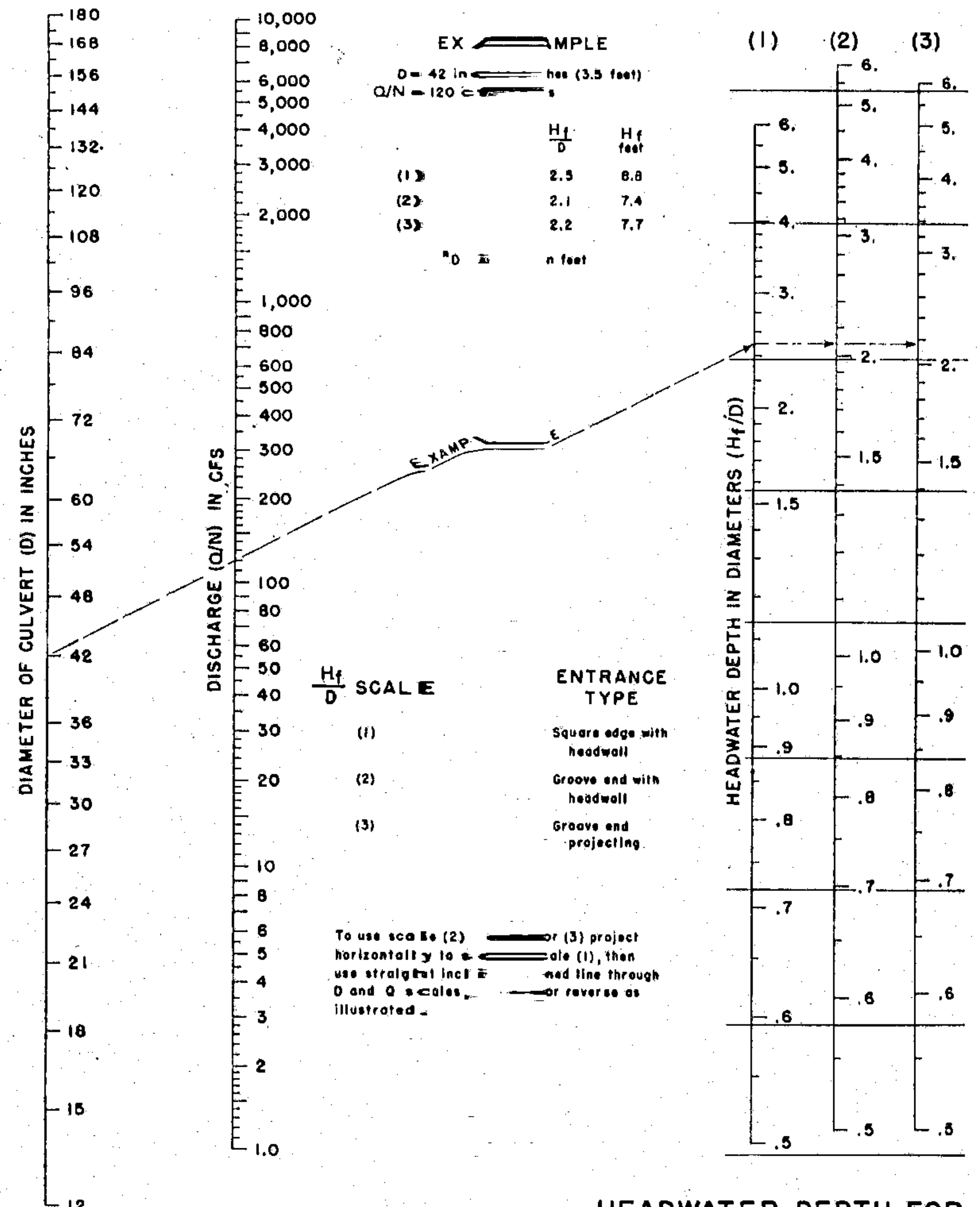


Figure 6-29 Inlet-Control Nomograph



HEADWATER DEPTH FOR INLET CONTROL
RECTANGULAR BOX CULVERTS
FLARED WINGWALLS 18° TO 33.7° AND 45°
WITH BEVELED EDGE AT TOP OF INLET

Figure 6-30 Inlet-Control Nomograph



HEADWATER DEPTH FOR CONCRETE PIPE CULVERTS WITH INLET CONTROL

Figure 6-31 Inlet-Control Nomograph

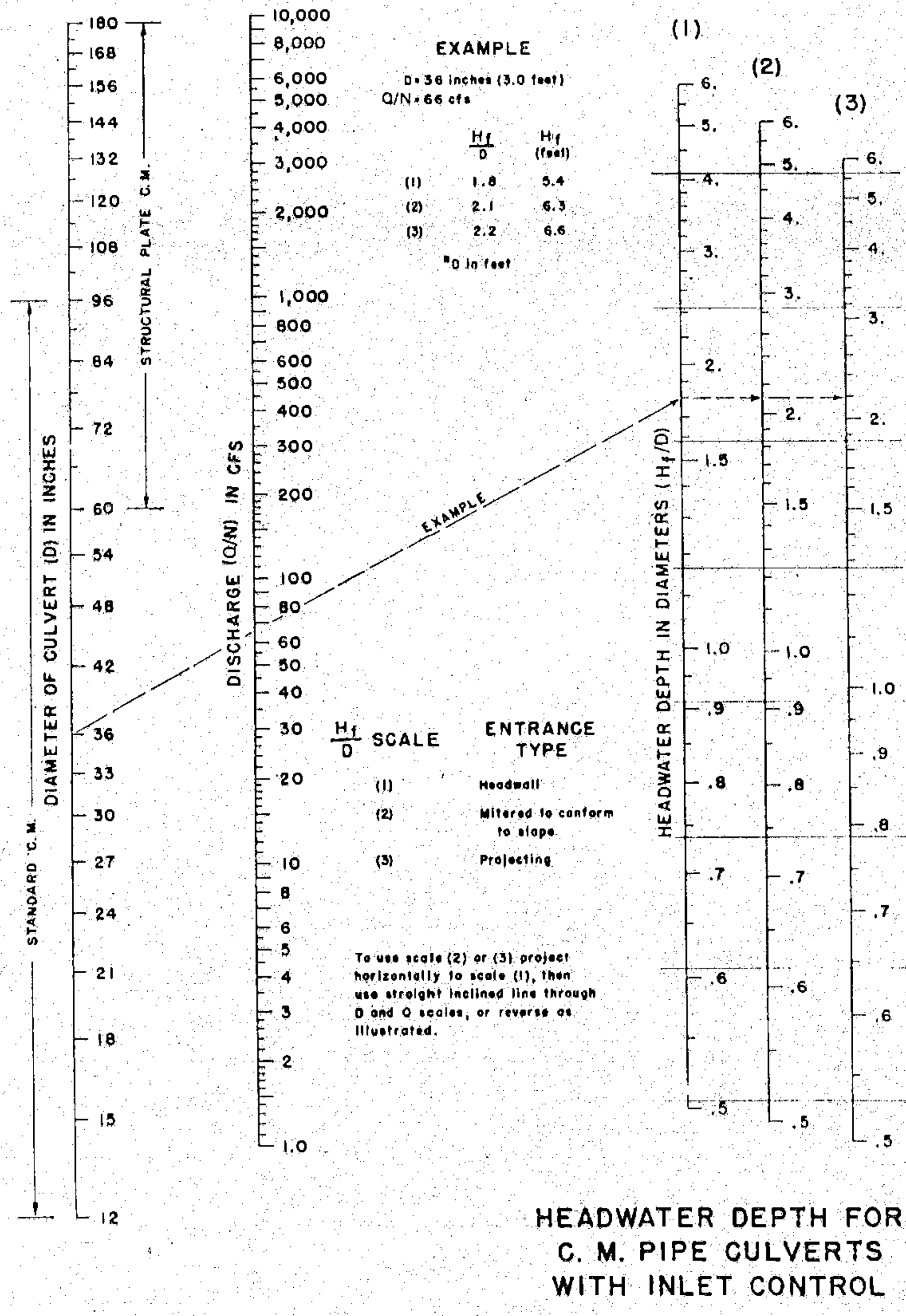


Figure 6-32 Inlet-Control Nomograph

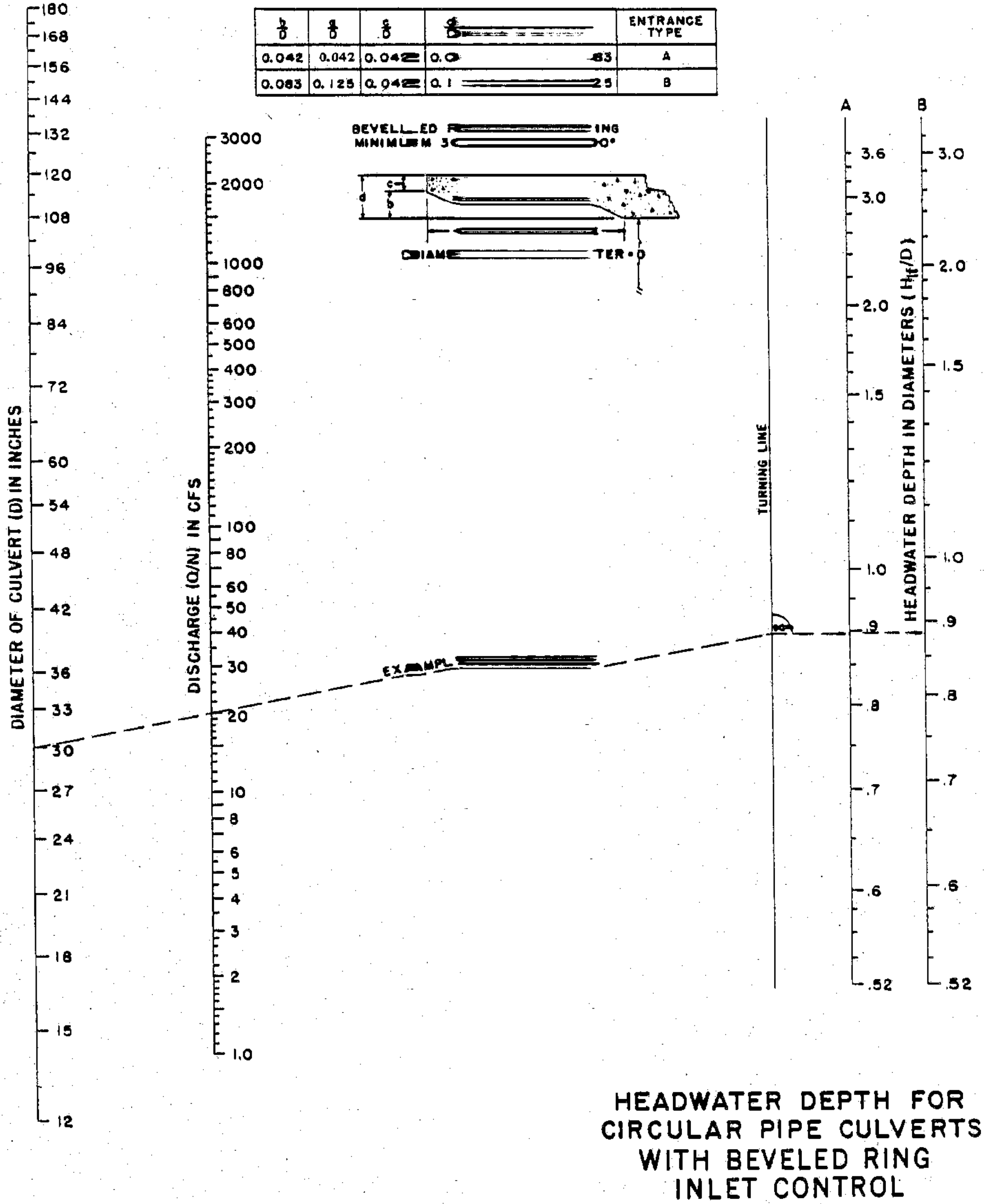


Figure 6-33 Inlet-Control Nomograph

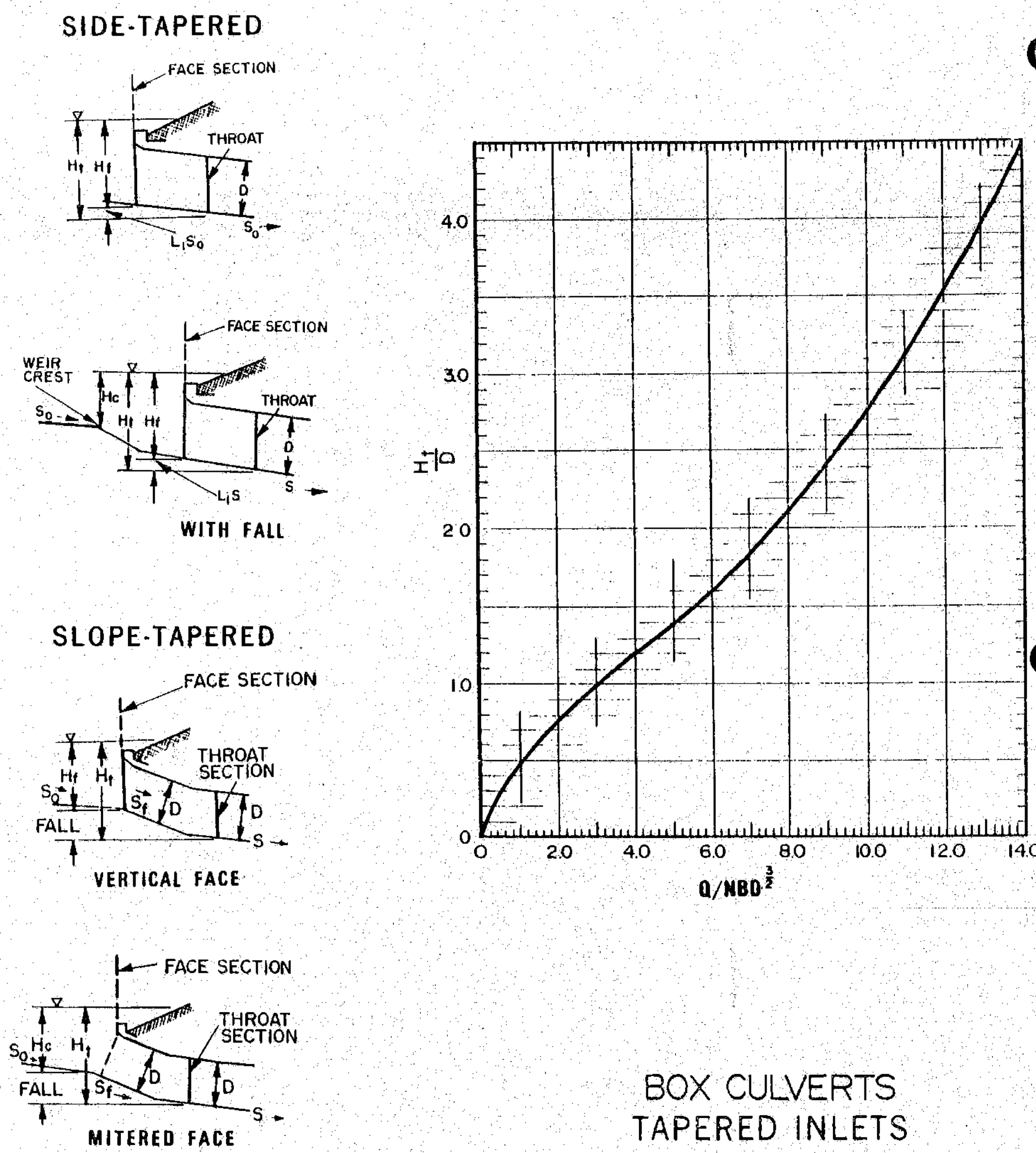


Figure 6-34 Throat-Control Curve

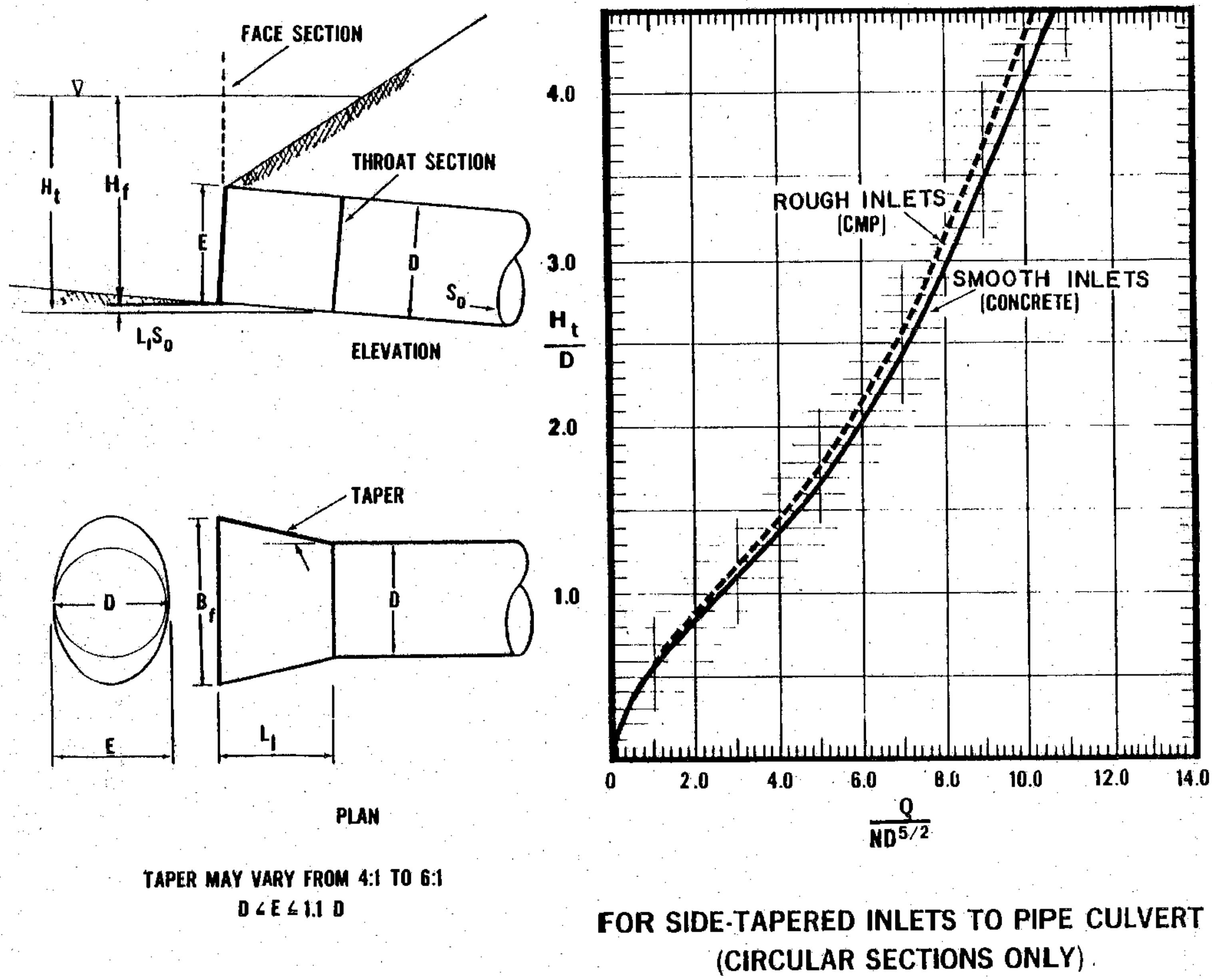
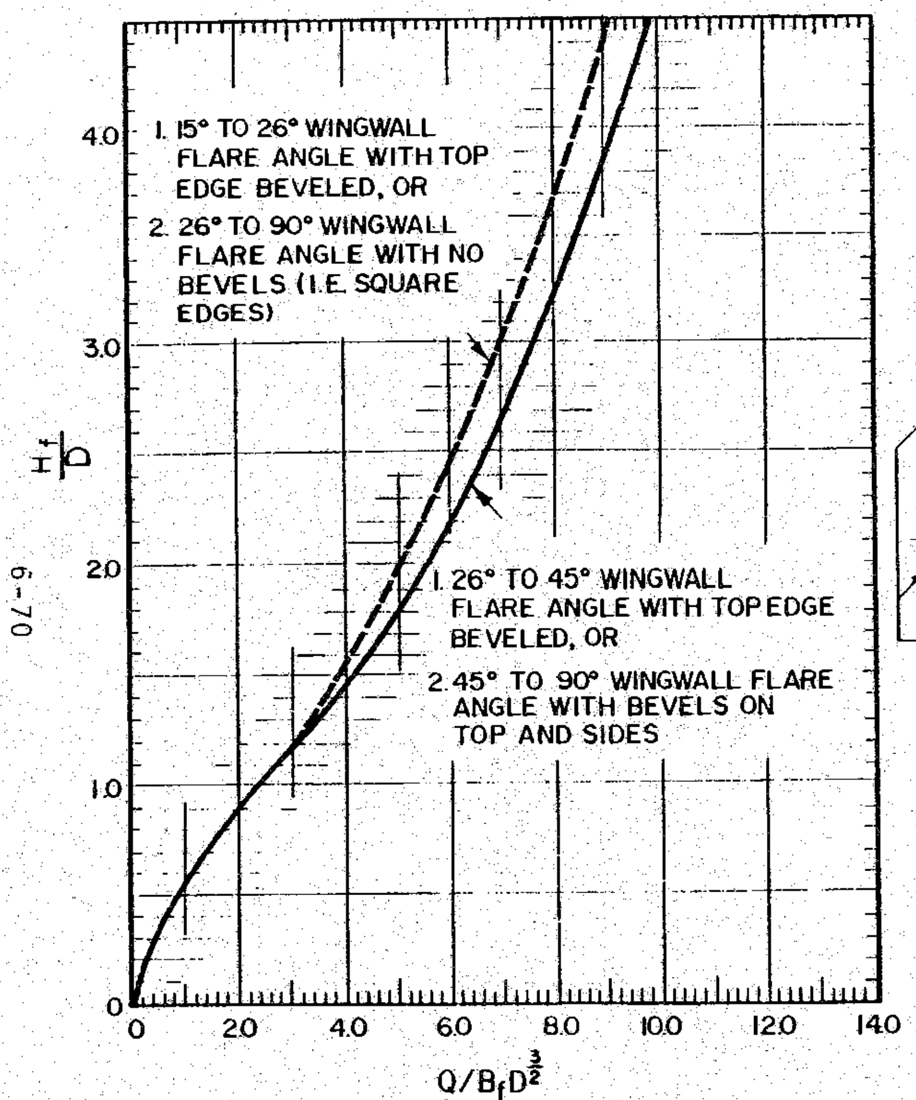
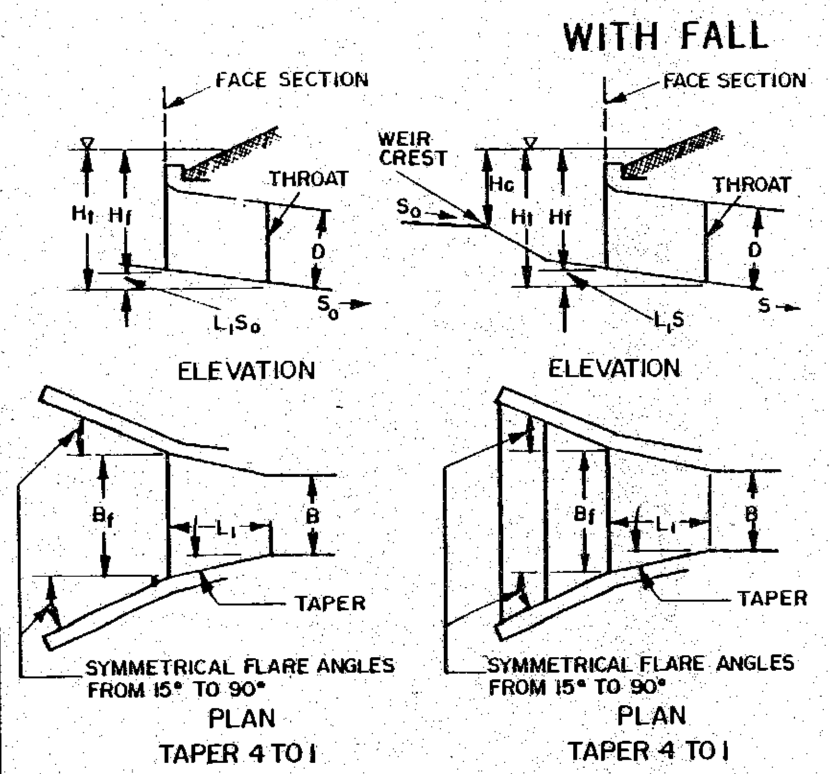


Figure 6-35 Throat-Control Curve





BOX CULVERTS
SIDE-TAPERED INLETS

Figure 6-36 Face-Control Curve

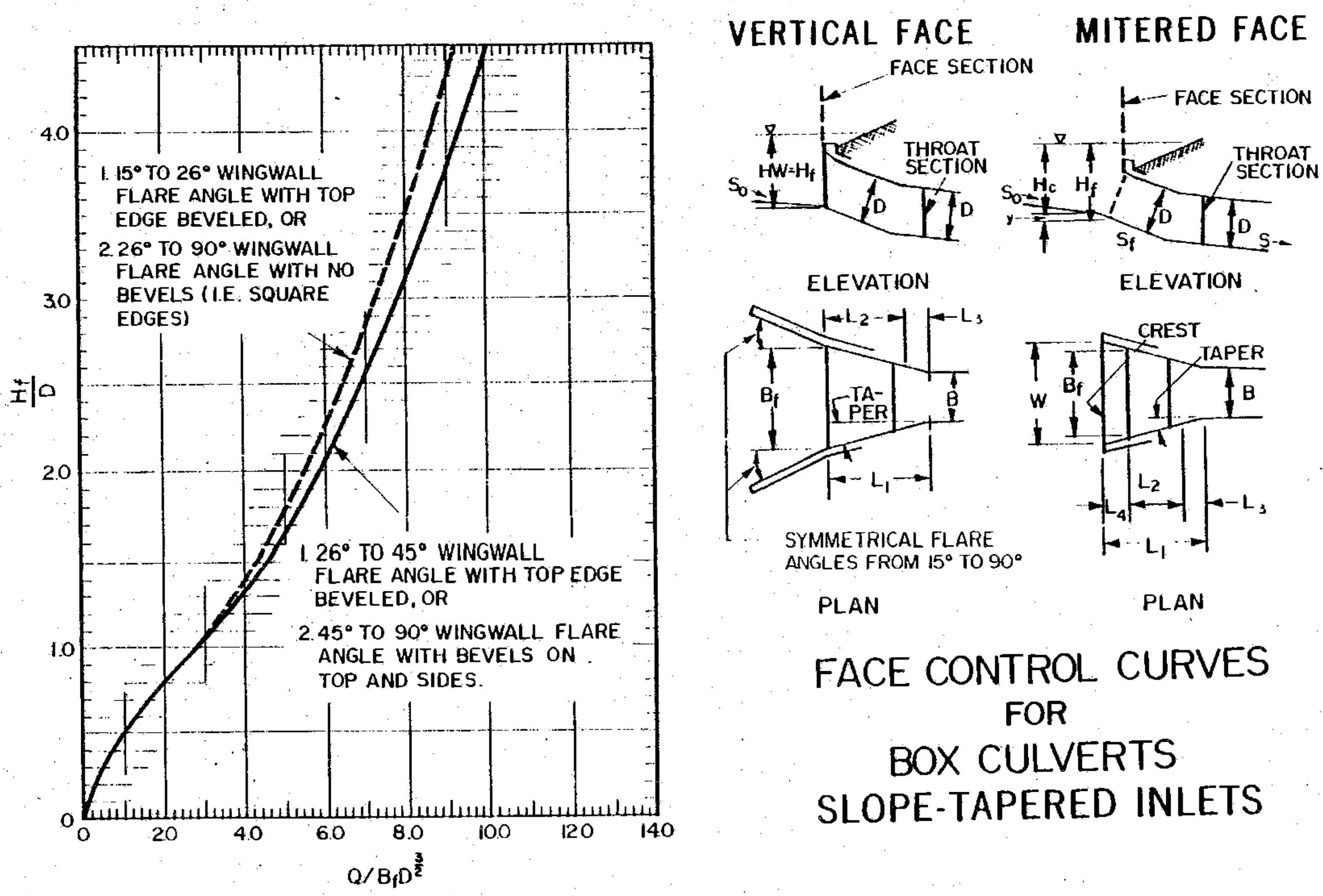
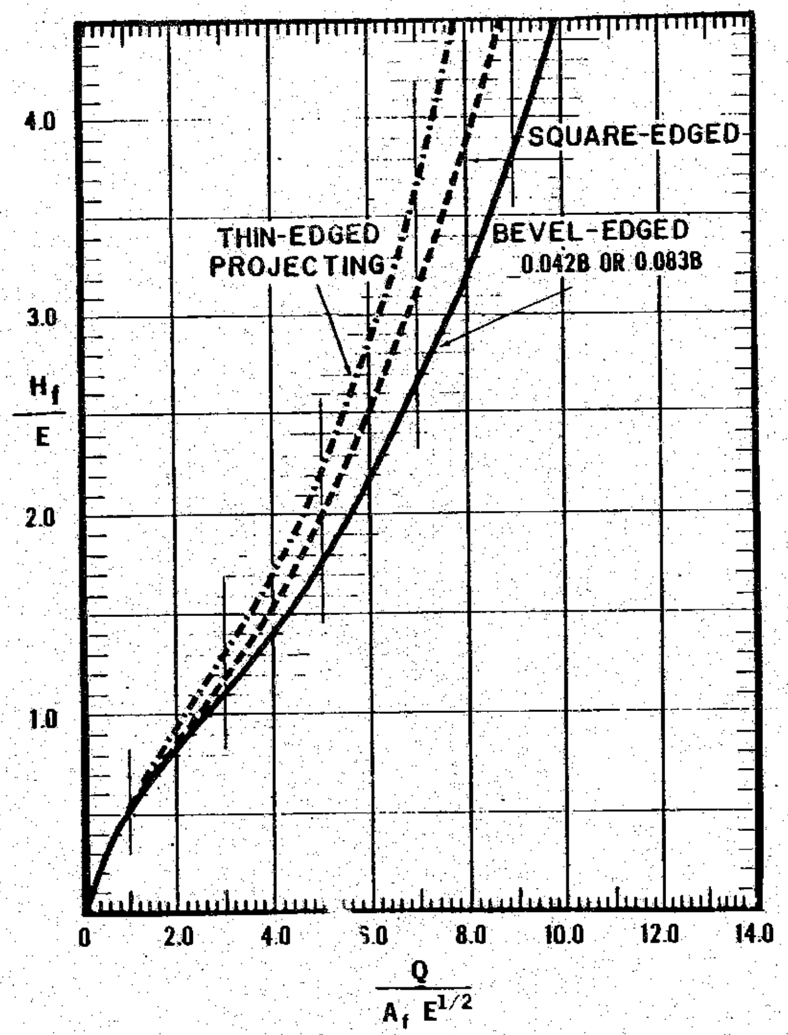
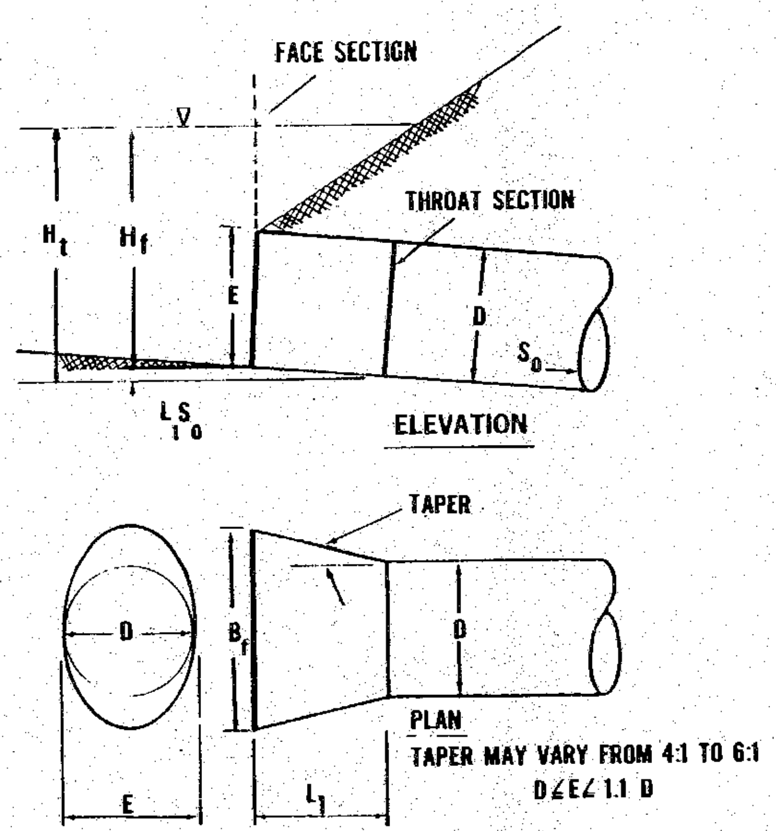


Figure 6-37 Face-Control Curve





FACE CONTROL CURVES
FOR SIDE- TAPERED INLETS TO PIPE CULVERTS
(NON-RECTANGULAR SECTIONS ONLY)

NOTE: FOR MULTIPLE BARRELS, DESIGN SIDE-TAPERED INLETS AS INDIVIDUAL STRUCTURES

Figure 6-38 Face-Control Curve

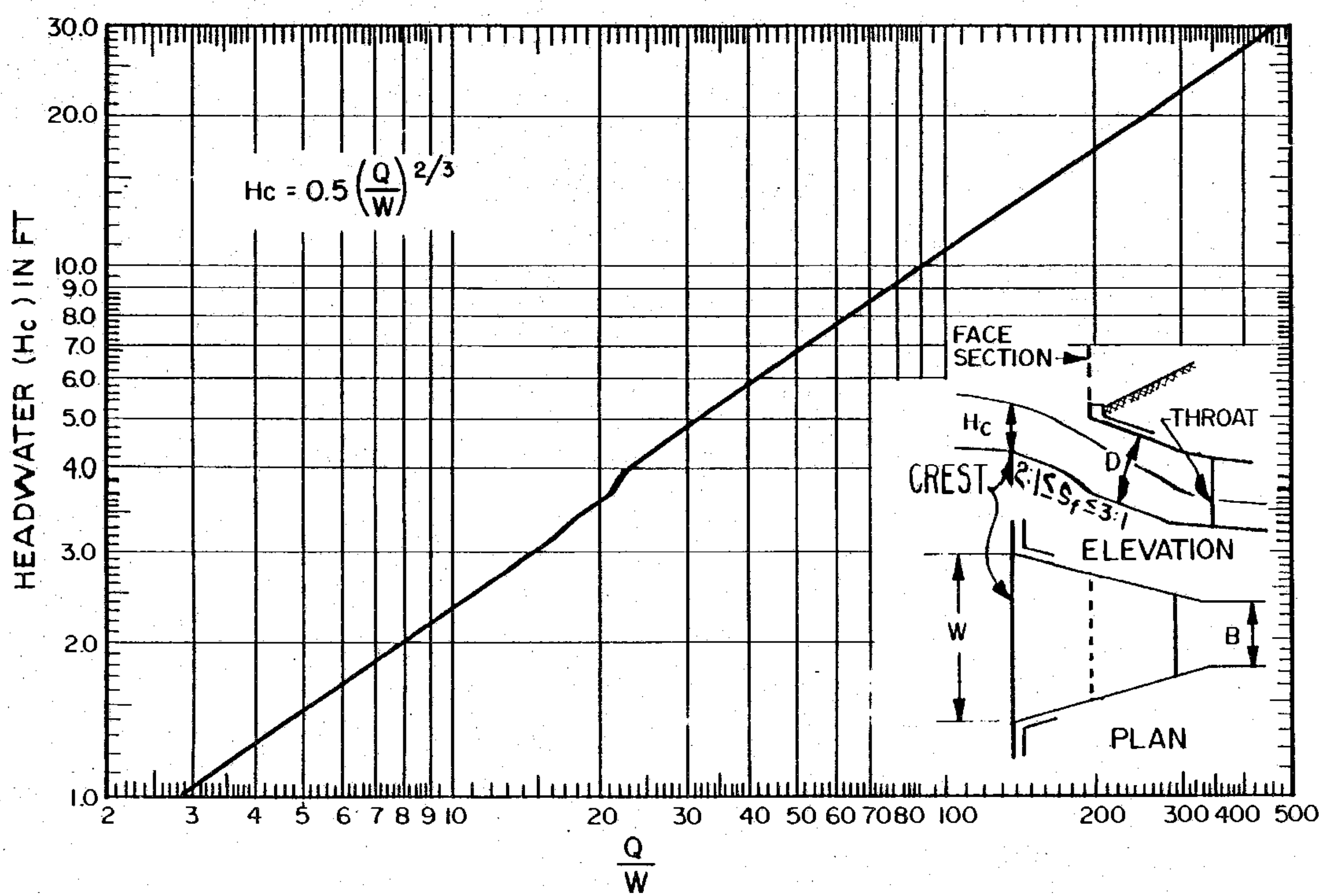


Figure 6-39 Headwater Required for Crest Control

PROJECT: ____________

STATION: ____________

OUTLET CONTROL
DESIGN CALCULATIONS

DESIGNER: ____________

DATE: ____________

INITIAL DATA:

Q ______ = ______ cfs

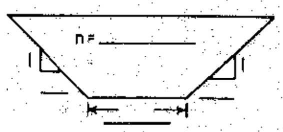
AHW El. = ______ ft.

S_o = ______

L_a = ______ ft.

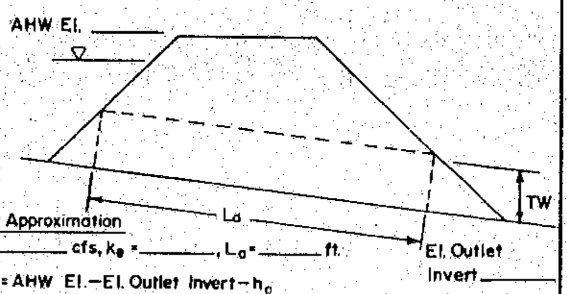
El. Outlet Invert ______ ft.

Stream Data:



Barrel Shape and Material ______ Barrel n= ______

SKETCH



First Approximation

Q= ______ cfs, k_e = ______, L_a = ______ ft.

H = AHW El. − El. Outlet Invert − h_o

= ______ − ______ − ______ ______

∴ A = ______ $ft.^2$ or D= ______ ft.; Try ______

Q	$\frac{Q}{N}$	* H	$\frac{Q}{NB}$	(1) d_c	$\frac{d_c+D}{2}$	Qn	(2) TW	(3) h_o	(4) HW_o	(5) V_o	COMMENTS
Trial No. ______, N= ______, B= ______, D= ______, k_e = ______											
Trial No. ______, N= ______, B= ______, D= ______, k_e = ______											
Trial No. ______, N= ______, B= ______, D= ______, k_e = ______											

Notes and Equations:

(1) d_c cannot exceed D

(2) TW based on d_n in natural channel, or other downstream control.

(3) $h_o = \frac{d_c+D}{2}$ or TW, whichever is larger.

(4) HW_o = H + h_o + El. Outlet Invert.

(5) Outlet Velocity (V_o=Q/Area defined by d_c or TW, not greater than D. Do not compute until control section is known.

SELECTED DESIGN

N= ______ At Design Q:

B= ______ ft.

D= ______ ft. HW_o = ______ ft.

k_e = ______ V_o = ______ f/s

$$* \quad H = \left[1 + k_e + \frac{29 n^2 \cdot L}{R^{1.33}}\right] \frac{V^2}{2g}$$

Figure 6-40 Design Computation Form for Improved Culverts

PROJECT:________________ **CULVERT INLET CONTROL SECTION DESIGN CALCULATIONS** DESIGNER:____________

STATION:________________ DATE:____________

INITIAL DATA:

Q ______ = ______ cfs

AHW El. = ________ ft.

S_o = ______

L_a = ______ ft.

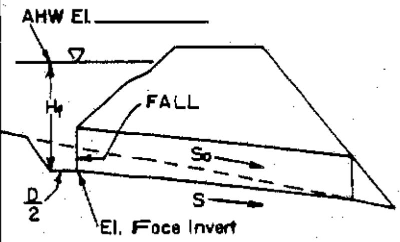
El. Stream Bed at Face ______ ft.

Barrel Shape and Material ____________ Barrel n= ______

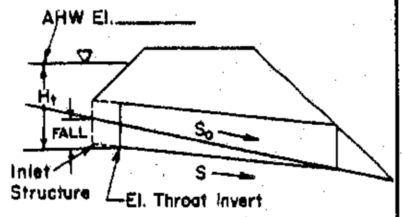
N = ______ , B = ______

D = ______ , $NBD^{3/2}$ = ______

(Pipe) $ND^{5/2}$ = ____________



CONVENTIONAL or BEVELED INLET: FACE CONTROL SECTION (Upper Headings)



TAPERED INLET THROAT CONTROL SECTION (Lower Headings)

DEFINITIONS OF INLET CONTROL SECTION

	$\frac{Q}{NB}$	$\frac{H_f}{D}$	H_f	(1) El. Face Invert	El. Stream Bed At Face	(2)	(3) HW_f	(4)	(5)	Note: Use Upper Headings for Coventional or Beveled Face; Lower Headings for Tapered Inlet Throat.
Q	$\frac{Q}{NBD^{3/2}}$	$\frac{H_t}{D}$	H_t	El. Throat Invert		FALL	HW_t	S	V_o	COMMENTS
Trial No. ______ Inlet and Edge Description ____________										
Trial No. ______ Inlet and Edge Description ____________										
Trial No. ______ Inlet and Edge Description ____________										

Notes and Equations:

(1) El. Face (or throat) invert = AHW El. − H_f (or H_t)

(2) FALL = El. Stream Bed at Face − El. face (or throat) invert

(3) HW_f (or HW_t) = H_f (or H_t) + El. face (or throat) invert, where El. face (or throat) invert should not exceed El. stream bed.

(4) $S \approx S_o - FALL/L_a$

(5) Outlet Velocity = Q/Area defined by d_n at S

SELECTED DESIGN

Inlet Description:

FALL = ______ ft.

Invert El. = ______ ft.

Bevels:

Angle = ______

b = ______ in., d = ______ in.

Figure 6-41 Design Computation Form for Improved Culverts

PROJECT: ____________ DESIGNER: ____________

SIDE-TAPERED INLET
DESIGN CALCULATIONS

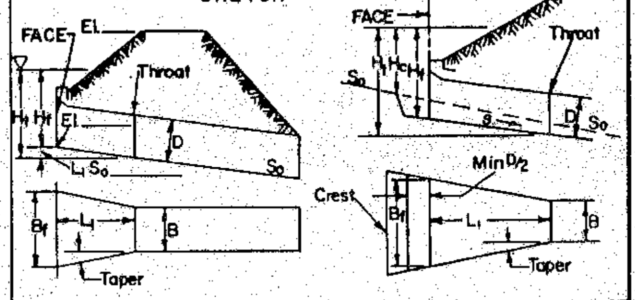
STATION: ____________ DATE: ____________

INITIAL DATA

Q ______ = ______ cfs S_o = ______
AHW El. = ______ ft. L_a = ______ ft.
TAPER = ______ : 1
Barrel Shape and Material ____________
Face Edge Description ____________

N = ______, B = ______ ft, D = ______ ft

SKETCH



Q	El. Throat Invert	(1) H_f/D H_f/E	$Q/(B_f D^{3/2})$ $Q/(A_f E^{1/2})$	$D^{3/2}$ $E^{1/2}$	(2) Min. B_f Min. A_f	B_f	(3) L_1	(4) S	$L_1 S$	(5) El. Face Invert	Upper Headings for Box Culverts, Lower Headings for Pipes COMMENTS
Trial No. ___, Q = ______, HW_f = ______											
											$B_f D^{3/2}$ [or $A_f E^{1/2}$] = ______
Trial No. ___, Q = ______, HW_f = ______											
											$B_f D^{3/2}$ [or $A_f E^{1/2}$] = ______
Trial No. ___, Q = ______, HW_f = ______											
											$B_f D^{3/2}$ [or $A_f E^{1/2}$] = ______

Notes and Equations:

(1) H_f/D [or H_f/E] = (HW_f − El. Throat Invert − 1) / D [or E]
$D \le E \le 1.1D$

(2) $\text{Min. } B_f = \dfrac{Q}{(D^{3/2})(Q/B_f D^{3/2})}$

$\text{Min. } A_f = \dfrac{Q}{(E^{1/2})(Q/A_f E^{1/2})}$

(3) $L_1 = \left[\dfrac{B_f - NB}{2}\right]$ TAPER

(4) From throat design

(5) El. Face Invert − El. Throat Invert > 1 ft., recompute.
Face and Throat may be lowered to better fit site, but do not raise.

SELECTED DESIGN

B_f = ______ ft.
L_1 = ______ ft.
Bevels: Angle ______ °
d = ______ in., b = ______ in.
Crest Check:
HW_c = ______ ft.
H_c = ______ ft.
Q/W = ______ (Chart 17)
Min. W = ______ ft.

Figure 6-42 Design Computation Form for Improved Culverts

PROJECT: ______ STATION: ______

SLOPE-TAPERED INLET DESIGN CALCULATIONS

DESIGNER: ______ DATE: ______

INITIAL DATA:

Q____ = ____cfs So = ______

AHW EL. ____ ft. L_a = ______ ft.

El. Stream bed at crest ______ ft.

El. stream bed at face ______ ft.

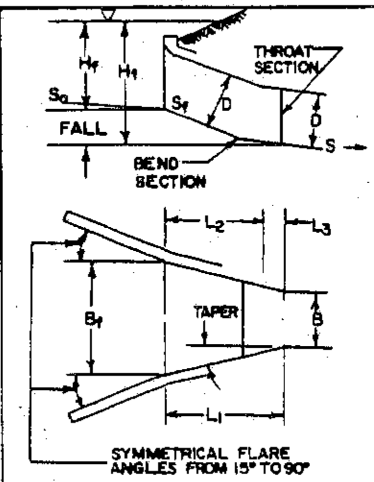
TAPER = ____:1 (4:1 to 6:1)

S_f = ____ :1 (2:1 to 3:1)

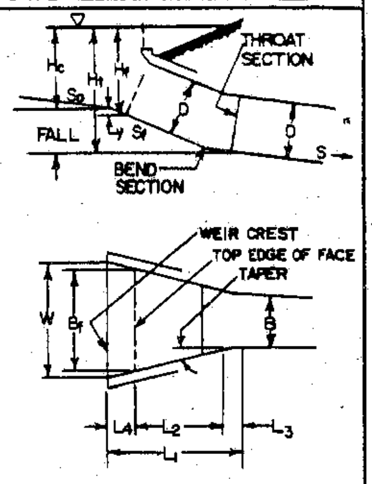
Barrel Shape and Material ______

Inlet Edge Description ______

N=______, B=______ft, D=______ft.



VERTICAL



MITERED

	Q	HW_f	El. Throat Invert	(1.) El. Face Invert	(2.) H_f	$\frac{H_f}{D}$	$\frac{Q}{B_f D^{3/2}}$	$D^{3/2}$	(3.) Min. B_f	B_f	S	Comments
Trial 1												$B_f D^{3/2}$ = ______
Trial 2												$B_f D^{3/2}$ = ______

Note: Use only throat designs with FALL > 0.25D

(1.) El. face invert: Vertical = Approx. stream bed elevation at face
Mitered = El. Crest − y, where y = 0.4D (Approx.), but higher than throat invert elevation.

(2.) Hf = HW_f − El. face invert

(3.) Min. $B_f = \frac{Q}{(D^{3/2})(Q / B_f D^{3/2})}$

(4.) Min. L_3	(5.) L_4	(6.) L_2	(7.) Check L_2	(8.) Adj L_3	(9.) Adj TAPER	(10.) L_1	(11.) W	$\frac{Q}{W}$	H_c	(12.) Max. Crest El.	GEOMETRY
											B_f = ____ft. L_3 = ____ft.
											L_1 = ____ft. L_4 = ____ft.
											L_2 = ____ft. d = ____in.
											b = ____in.
											TAPER = ____:1

(4.) Min. L_3 = 0.5NB

(5.) $L_4 = S_{fy} + D/S_f$ (Mitered only)

(6.) L_2 = (El. Face (Crest) Invert − El. Throat Invert) $S_f - L_4$

(7.) Check $L = \left[\frac{B_f - NB}{2}\right]$ TAPER $- L_3$

(8.) If (7) > (6), Adj. $L_3 = \left[\frac{B_f - NB}{2}\right]$ TAPER $- L_2$

(9.) If (6) > (7) Adj. TAPER $= (L_2 + L_3) / \left[\frac{B_f - NB}{2}\right]$

(10.) $L_1 = L_2 + L_3 + L_4$

(11.) Mitered: $W = NB + 2\left[\frac{L_1}{TAPER}\right]$

(12.) Max. Crest El. = $HW_f - H_c$

Figure 6-43 Design Computation Form for Improved Culverts

6.70 List of Symbols

Symbol	Units	Description
A_b	sq ft	Area of bend section of slope-tapered inlets
A_f	sq ft	Area of inlet face section
A_t	sq ft	Area of inlet throat section
AHW EL.	ft	Allowable headwater elevation at culvert entrance
B	ft	Width of culvert barrel or diameter of pipe culvert
b	in.	Dimension of side be bevel
B_b	ft	Width of bend section of slope-tapered inlets
B_f	ft	Width of face section of improved inlets
C_d		Discharge coefficient based on bend section control
C_f		Discharge coefficient based on face section control
C_t		Discharge coefficient based on throat section control
D	ft	Height of box culvert or diameter of pipe culvert
d	in.	Dimension of top bevel
d_c	ft	Critical depth of flow
E	ft	Height of side-tapered pipe-culvert face section, excluding bevel dimension
FALL	ft	Approximate depression of control section below the stream bed
g	ft/sec^2	Acceleration of gravity = 32.2
H	ft	Head or energy required to pass a given quantity of water through a culvert flowing in outlet control
H_b	ft	Depth of pool, or head, above the bend section invert
H_c	ft	Depth of pool, or head, above the crest
H_f	ft	Depth of pool, or head, above the face section invert
H_t	ft	Depth of pool, or head, above the throat section invert
HG Line	ft	Hydraulic grade line

Symbol	Units	Descriptions
HW	ft	Headwater elevation; subscript indicates control section (HW, as used in HEC #5, is a depth and is equivalent to H_f in this Manual)
HW_c	ft	Headwater elevation required for flow to pass crest in crest control
HW_f	ft	Headwater elevation required for flow to pass face section in face control
HW_o	ft	Headwater elevation required for culvert to pass flow in outlet control
HW_t	ft	Headwater elevation required for flow to pass throat section in throat control
h_o	ft	Elevation of equivalent hydraulic grade line referenced to the outlet invert
k_e		Entrance energy loss coefficient
k_b		A dimensionless effective pressure term for bend section control
k_t		A dimensionless effective pressure term for inlet throat control
L_a	ft	Approximate total length of culvert, including inlet face section control
L_1, L_2, L_3, L_4	ft	Dimensions relating to the improved inlet as shown in sketches of the different types of inlets
N		Number of barrels
n		Manning's roughness coefficient
P	ft	Length of depression
Q	cfs	Volume rate of flow
R	ft	Hydraulic radius = $\frac{\text{Area}}{\text{Wetted Perimeter}}$
S	ft/ft	Slope of culvert barrel
S_e	ft/ft	Slope of embankment
S_f	ft/ft	Slope of FALL for slope-tapered inlets (a ratio of horizontal to vertical)

Symbol	Units	Description
S_o	ft/ft	Slope of natural channel
T	ft	Depth of the depression
Taper	ft/ft	Sidewall flare angle (also expressed as the cotangent of the flare angle)
TW	ft	Tailwater depth at outlet of culvert referenced to outlet invert elevation
V	ft/sec	Mean velocity of flow
W	ft	Width of weir crest for slope-tapered inlet with mitered face
W	ft	Top width of depression
y	ft	Difference in elevation between crest and face section of a slope-tapered inlet with mitered face

6.80 Bibliography

1. City of Austin, Texas, Engineering Department et al, Drainage Criteria Manual, Austin, Texas, 1977.

2. Federal Highway Administration, Hydraulic Charts for the Selection of Highway Culverts, U.S. Government Printing Office, Washington, D.C., 1965.

3. Federal Highway Administration, Capacity Charts for the Hydraulic Design of Highway Culverts, U.S. Government Printing Office, Washington, D.C., 1965.

4. Federal Highway Administration, Hydraulic Design of Improved Inlets for Culverts, U.S. Government Printing Office, Washington, D.C., 1973.

Section 7

Open-Channel Flow

Section 7

Open-Channel Flow

Open channels for use in the major drainage system, have significant advantage in regard to cost, capacity, multiple use for recreational and aesthetic purposes, and potential for detention storage. Disadvantages include right-of-way needs and maintenance costs. Careful planning and design are needed to minimize the disadvantages, and to increase the benefits.

The ideal channel is a natural one carved by nature over a long period of time. The benefits of such a channel are that:

Velocities are usually low, resulting in a longer time of concentration and lower downstream peak flows.

Channel storage tends to decrease peak flows.

Maintenance needs are usually low because the channel is somewhat stabilized.

The channel provides a desirable green belt and recreational area adding significant social benefits.

Generally speaking, the natural channel, or the man-made channel which most nearly conforms to the character of a natural channel, is the most efficient and the most desirable.

In many areas facing urbanization, the runoff has been so minimal that natural channels do not exist. However, small trickle paths nearly always exist which provide an excellent basis for location and construction of channels. Good land planning should reflect even these minimal trickle channels to reduce development costs and minimize drainage problems. In some cases the prudent utilization of natural water routes in the development of a major drainage system will reduce the requirements for an underground storm sewer system.

Channel stability is a well recognized problem in urban hydrology because of the significant increase in low flows and peak storm runoff rates. A natural channel must be studied to determine the measures needed to avoid future bottom scour and bank cutting. Erosion control measures can be taken at reasonable cost which will preserve the natural appearance without sacrificing hydraulic efficiency.

7.10 Channel Discharge

Manning's Equation

Careful attention must be given to the design of drainage channels to assure adequate capacity and minimum maintenance to overcome the results of erosion and silting. The hydraulic characteristics of channels shall be determined by Manning's equation.

$$Q = \frac{1.49}{n} A \, R^{2/3} \, S^{1/2} \quad (7\text{-}1)$$

where:

Q = discharge in cfs

n = coefficient of roughness

A = cross-sectional area of channel in sq. ft.

R = hydraulic radius of channel, $\frac{A}{P}$, in feet;

P = wetted perimeter, in feet; and

S = slope of the frictional gradient, in feet per foot.

Uniform Flow

For a given channel condition of roughness, discharge, and slope, there is only one possible depth for maintaining a uniform flow. This depth is the normal depth. When roughness, depth, and slope are known at a channel section, there can only be one discharge for maintaining a uniform flow through the section. This discharge is the normal discharge.

If the channel is uniform and resistance and gravity forces are in exact balance, the water surface will be parallel to the bottom of the channel, this is the condition of uniform flow.

Uniform flow is more often a theoretical abstraction than an actuality. True uniform flow is difficult to find in the field or to obtain in the laboratory. Channels are sometimes designed on the assumption that they will carry uniform flow at the normal depths, but because of conditions difficult if not impossible to evaluate and hence not taken into account, the flow will actually have depths considerably different from uniform depth. The engineer must be aware of the fact that uniform flow computation provides only an approximation of what will occur.

Normal Depth

The normal depth is computed so frequently that it is convenient to use nomographs for various types of cross sections to eliminate the need for trial and error solutions, which are time consuming. Nomographs for uniform flow are given in Figures 7-1 and 7-2.

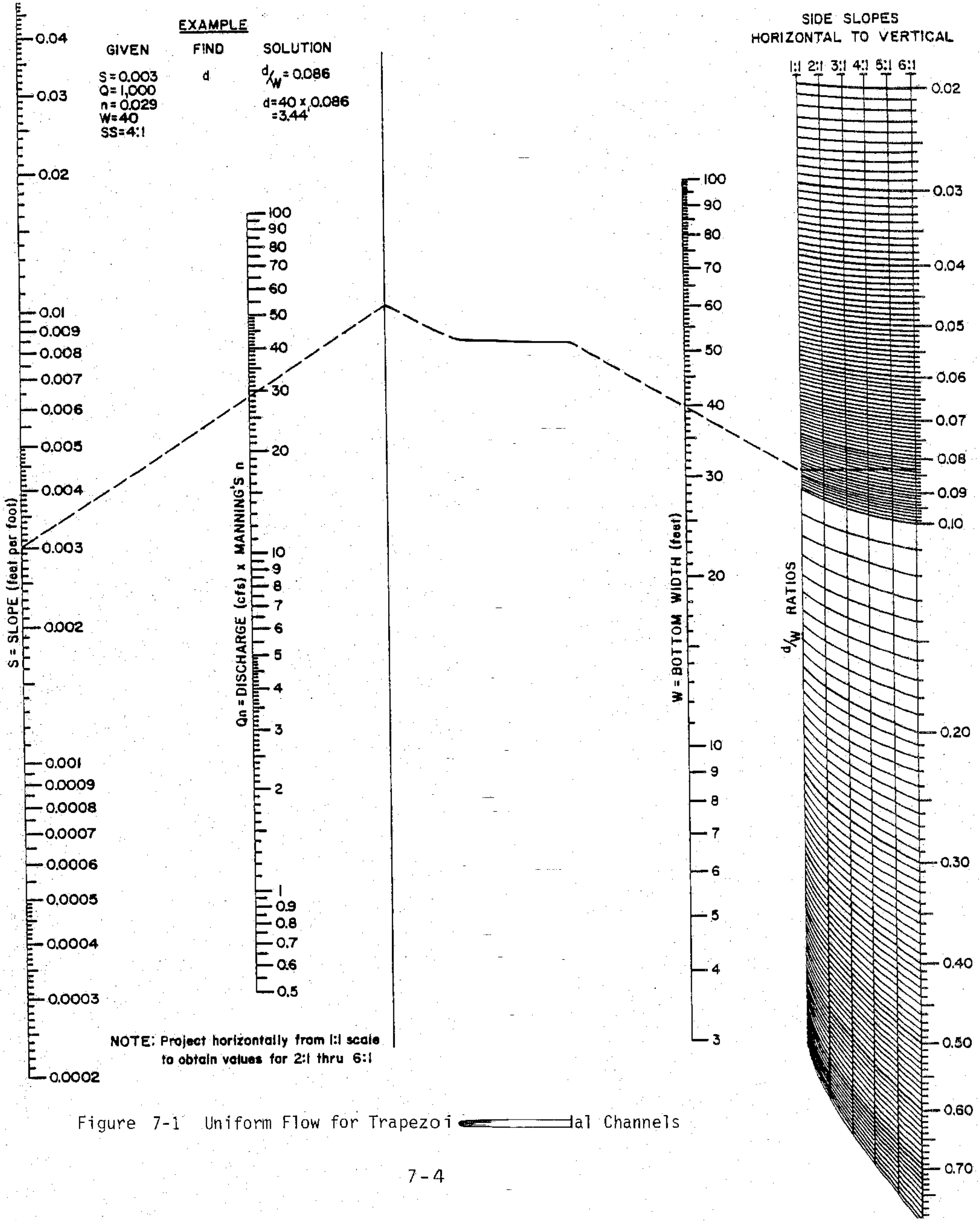


Figure 7-1 Uniform Flow for Trapezoidal Channels

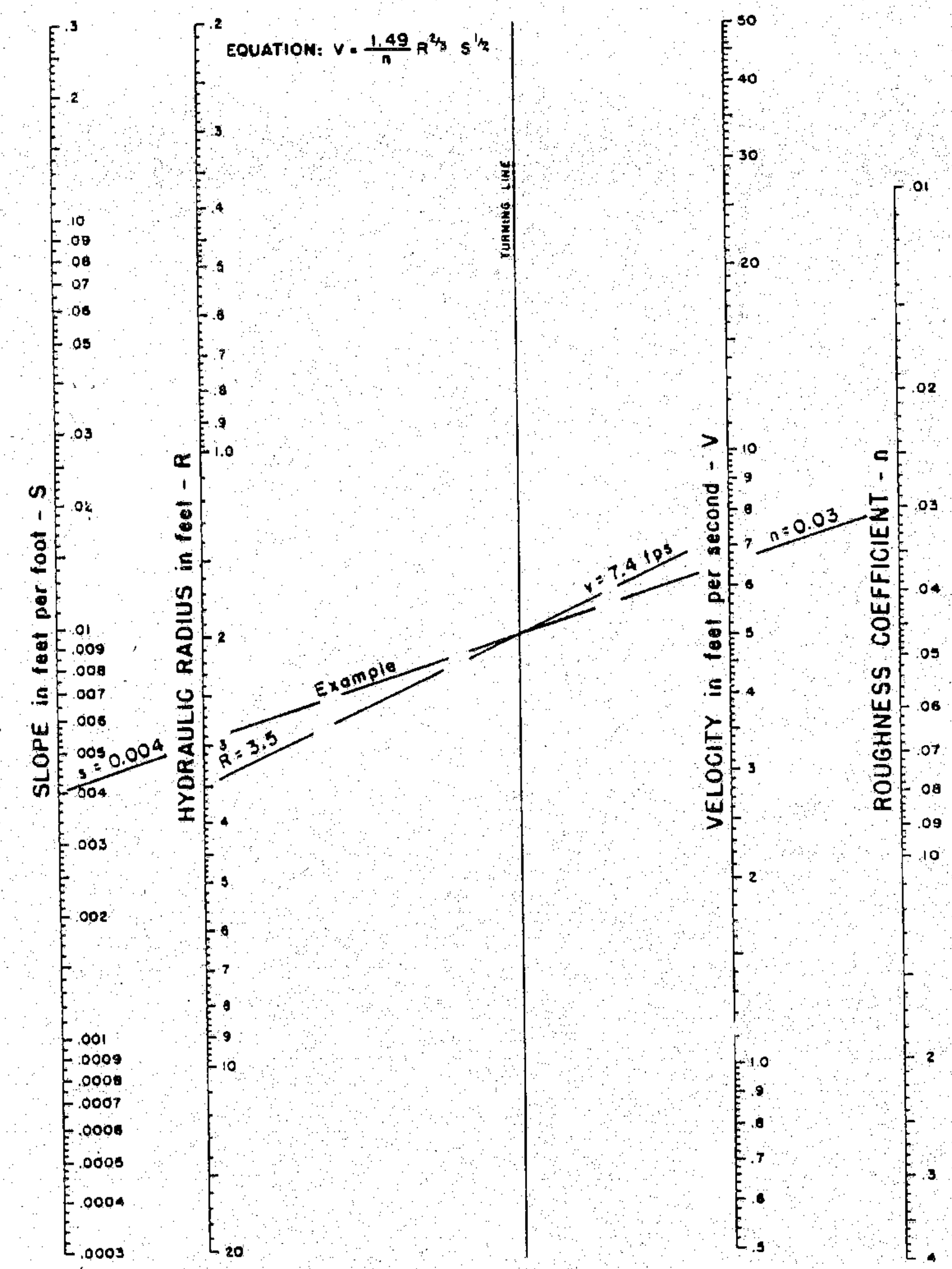


Figure 7-2 Manning's Formula Nomograph

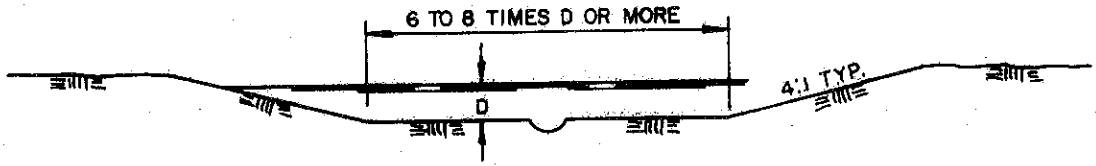
7.20 Design Considerations

Man-made channels should have trapezoidal sections of adequate cross-sectional areas to take care of uncertainties in runoff estimates, changes in channel coefficients, channel obstructions and silt accumulations. Figure 7-3 shows several typical cross sections used for urban drainage channels with grass cover.

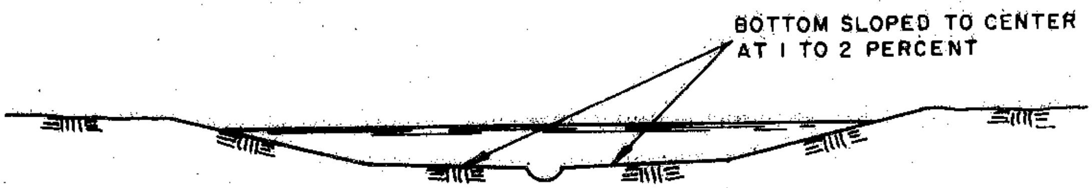
Accurate determination of the "n" value is critical in the analysis of the hydraulic characteristics of a channel. The "n" value for each channel reach should be based on experience and judgment with regard to the individual channel characteristics. Table 7-1 gives a method of determining the composite roughness coefficient based on actual channel conditions.

Where practicable, unlined channels should have sufficient gradient, depending upon the type of soil, to provide velocities that will be self-cleaning but will not be so great as to create erosion. Lined channels, drop structures, check dams, or concrete spillways may be required to control erosion that results from the high velocities of large volumes of water. Unless approved otherwise by the appropriate agency, channel velocities in man-made, unlined channels should not exceed 6 fps.

Maximum permissible design velocities are shown in Table 7-3 for seed mixtures recommended for use in the Casper area.

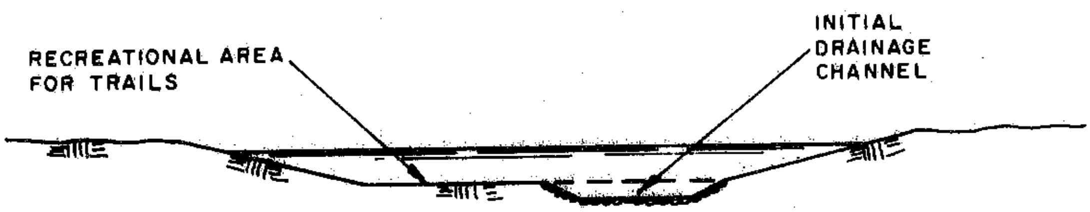


FLAT BOTTOM WITH TRICKLE CHANNEL



OVAL OR SLOPED BOTTOM WITH TRICKLE CHANNEL

INITIAL DRAINAGE CHANNEL WITH OVERFLOW
AREA FOR MAJOR DRAINAGE RUNOFF



RIPRAPPED INITIAL DRAINAGE CHANNEL WITH
OVERFLOW FOR MAJOR DRAINAGE RUNOFF

Figure 7-3 Typical Channel Sections

TABLE 7-1

Composite Roughness Coefficients for Channels

$$n = (n_0 + n_1 + n_2 + n_3 + n_4)\ m \qquad (7\text{-}2)$$

	Channel Conditions	Value
Material Involved n_0	Earth	0.020
	Fine Gravel	0.024
	Coarse Gravel	0.028
Degree of Irregularity n_1	Smooth	0.000
	Minor	0.005
	Moderate	0.010
	Severe	0.020
Variation of Channel Cross Section n_2	Gradual	0.000
	Alternating Occasionally	0.005
	Alternating Frequently	0.010 - 0.015
Relative Effect Of Obstructions n_3	Negligible	0.000
	Minor	0.010 - 0.015
	Appreciable	0.020 - 0.030
	Severe	0.040 - 0.060
Vegetation n_4	Low	0.005 - 0.010
	Medium	0.010 - 0.025
	High	0.025 - 0.050
	Very High	0.050 - 0.100
Degree of Meandering m	Minor	1.000 - 1.200
	Appreciable	1.200 - 1.500
	Severe	1.500

TABLE 7-2

Roughness Coefficients for Channels

Type of channel and description	Minimum	Normal	Maximum
EXCAVATED OR DREDGED			
a. Earth, straight and uniform			
1. Clean, recently completed	0.016	0.018	0.020
2. Clean, after weathering	0.018	0.022	0.025
3. Gravel, uniform section, clean	0.022	0.025	0.030
4. With short grass, few weeds	0.022	0.027	0.033
b. Earth, winding and sluggish			
1. No vegetation	0.023	0.025	0.030
2. Grass, some weeds	0.025	0.030	0.033
3. Dense weeds or aquatic plants in deep channels	0.030	0.035	0.040
4. Earth bottom and rubble sides	0.028	0.030	0.035
5. Stony bottom and weedy banks	0.025	0.035	0.040
6. Cobble bottom and clean sides	0.030	0.040	0.050
c. Dragline-excavated or dredged			
1. No vegetation	0.025	0.028	0.033
2. Light brush on banks	0.035	0.050	0.060
d. Rock cuts			
1. Smooth and uniform	0.025	0.035	0.040
2. Jagged and irregular	0.035	0.040	0.050
e. Channels not maintained, weeds and brush uncut			
1. Dense weeds, high as flow depth	0.050	0.080	0.120
2. Clean bottom, brush on sides	0.040	0.050	0.080
3. Same, highest stage of flow	0.045	0.070	0.110
4. Dense brush, high stage	0.080	0.100	0.140

TABLE 7-2 (continued)

Roughness Coefficients for Channels

Type of channel and description	Minimum	Normal	Maximum
NATURAL STREAMS			
Minor streams (top width at flood stage 100 ft)			
a. Streams on plain			
1. Clean, straight, full stage, no rifts or deep pools	0.025	0.030	0.033
2. Same as above, but more stones and weeds	0.030	0.035	0.040
3. Clean, winding, some pools and shoals	0.033	0.040	0.045
4. Same as above, but some weeds and stones	0.035	0.045	0.050
5. Same as above, lower stages, more ineffective slopes and sections	0.040	0.048	0.055
6. Same as 4, but more stones	0.045	0.050	0.060
7. Sluggish reaches, weedy, deep pools	0.050	0.070	0.080
8. Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.075	0.100	0.150
LINED OR BUILT-UP CHANNELS			
a. Corrugated Metal	0.021	0.025	0.030
b. Concrete			
1. Trowel finish	0.011	0.013	0.015
2. Float finish	0.013	0.015	0.016
3. Finished, with gravel on bottom	0.015	0.017	0.020
4. Unfinished	0.014	0.017	0.020
5. Gunite, good section	0.016	0.019	0.023
6. Gunite, wavy section	0.018	0.022	0.025
7. On good excavated rock	0.017	0.020	
8. On irregular excavated rock	0.022	0.027	

TABLE 7-2 (continued)

Roughness Coefficients for Channels

Type of channel and description	Minimum	Normal	Maximum
c. Concrete bottom float finished with sides of			
1. Dressed stone in mortar	0.015	0.017	0.020
2. Random stone in mortar	0.017	0.020	0.024
3. Cement rubble masonry, plastered	0.016	0.020	0.024
4. Cement rubble masonry	0.020	0.025	0.030
5. Dry rubble or riprap	0.020	0.030	0.035
d. Gravel bottom with sides of			
1. Formed concrete	0.017	0.020	0.025
2. Random stone in mortar	0.020	0.023	0.026
3. Dry rubble or riprap	0.023	0.033	0.036
e. Asphalt			
1. Smooth		0.013	
2. Rough		0.016	
f. Grassed	0.030	0.040	0.050

TABLE 7-3

Maximum Permissible Design Velocities[1]

	Slope (percent)		
Cover	0-5	5-10	> 10
Kentucky bluegrass	6	5	4
Wheatgrass	6	5	4
Smooth brome	6	5	4
Tall fescue	6	5	4
Reed canarygrass	4.5	3.5	NR
Annuals[2] Rye, Oats, Ryegrass	2.5	NR	NR

[1] Velocity in fps.
[2] Annuals: use only as temporary protection until permanent vegetation is established.

NR - Not recommended.

7.30 Channel Cross Sections

The channel shape may be almost any type suitable to the location and to the environmental conditions. Often the shape can be chosen to suit open space and recreational needs to create additional benefits.

Side Slope

Except in horizontal curves the flatter the side slope, the better. Normally slopes shall be no steeper than 3:1, which is also the practical limit for mowing equipment. Rock or concrete-lined channels or those which for other reasons do not require slope maintenance may have slopes as steep as 1.5:1.

Depth

Deep channels are difficult to maintain and can be hazardous. Constructed channels should therefore be as shallow as practical.

Bottom Width

Channels with narrow bottoms are difficult to maintain and are conducive to high velocities during high flows. It is desirable to design open channels such that the bottom width is at least twice the depth with 6 to 8 times the depth desirable.

Trickle Channels

The low flows, and sometimes base flows, from urban areas must be given specific attention. If erosion of the bottom of the channel appears to be a problem, low flows shall be carried in a ripraped trickle channel which has a capacity of 5 percent of the design peak flow. Care must be taken to insure that low flows do not create an erosion problem. Water-tolerant vegetation such as Reed canarygrass may be used in lieu of riprap for low-velocity channels.

Freeboard

For channels with flow at high velocities, surface roughness, wave action, air bulking, and splash and spray are quite erosive along the top of the flow. Freeboard height should be chosen to provide a suitable safety margin. The height of freeboard shall be a minimum of one foot, or provide an additional capacity of approximately one-third of the design flow. For deep flows with high velocities one may use the formula:

$$\text{Freeboard (in feet)} = 1.0 + 0.025\ vd^{0.33} \qquad (7\text{-}3)$$

where:

v = velocity of flow, in fps; and
d = depth of flow, in ft.

For the freeboard of a channel on a sharp curve, extra height must be added to the outside bank or wall in the amount:

$$H' = \frac{v^2 (T + B)}{2gR} \qquad (7\text{-}4)$$

where:

H'	=	additional height on outside edge of channel, in ft;
v	=	velocity of flow in channel, in fps;
T	=	width of flow at water surface, in ft;
B	=	bottom width of channel, in ft;
R	=	centerline radius of turn, in ft; and
g	=	acceleration of gravity, 32.2 ft/sec ^{2}.

If R is equal or greater than 3B additional freeboard is not required.

7.40 Channel Drops

The use of channel drops permits adjustment of channel gradients which are too steep for the design conditions. In urban drainage work it is often desirable to use several low head drops in lieu of a few higher drops. Special attention must be given to protecting the channel from erosion in the area of channel drops.

Section 8, Structures, should be consulted when considering drops in either open-channel or closed-conduit flow.

7.50 Supercritical Flow

The specific energy for open-channel flow may be expressed by the following equation:

$$H = d + \frac{v^2}{2g} \qquad (7\text{-}5)$$

where:

H	=	total energy head, in feet;
d	=	depth of flow, in feet;
v	=	velocity of flow, in fps; and
g	=	acceleration of gravity, in feet/sec ^{2}.

Then depth of flow is plotted against specific energy for a given channel and discharge, the resulting curve shows that, at a given specific energy, there are two possible depths. At minimum energy, only one depth of flow exists, known as the critical depth. At critical depth, the following relationship applies:

$$d = \frac{v^2}{g} \tag{7-6}$$

The Froude Number, F, is defined as:

$$F = \frac{V}{\sqrt{gd}} \tag{7-7}$$

It can be shown that F = 1 for critical flow. If the Froude Number is greater than 1, the flow is subcritical, but when the Froude Number is less than 1, the flow is supercritical.

Supercritical flow in an open channel in an urbanized area creates certain hazards which the designer must take into consideration. From a practical standpoint it is generally not possible to have any curvature in such a channel. Careful attention must be taken to insure against excessive oscillatory waves which may extend down the entire length of the channel from only minor obstructions upstream. Imperfections at joints of lined channels may rapidly cause a deterioration of the joints, in which case a complete failure of the channel can rapidly occur. In addition, high-velocity flow entering cracks or joints creates an uplift force by the conversion of velocity head to pressure head which can damage the channel lining. It is evident that when designing a lined channel with supercritical flow the designer must use utmost care and consider all relevant factors. Section 8, Structures, should be consulted when designing channels for supercritical flow.

7.60 Maintenance of Grassed Waterways

Because of the irregular and generally sparse precipitation in the Casper area (10-12 inches annually), irrigation of seeded areas is necessary depending on the time of planting. Seedings between May 1 and August 15 will require sprinkler irrigation. Seedings after August 15 may not become established before the first frost. Irrigation should continue until an adequate grass cover has been established to protect against erosion (a stand at least 3 inches high).

After preparing the site by removing debris and providing at least 4 inches of topsoil, at least 20 pounds per acre of nitrogen fertilizer should be applied and incorporated to a depth of 3 inches. Seeding rates shall be proportioned by percentage of species in the mixture according to the following table:

Table 7-4

Seeding Rates for Grassed Waterways

Seed	Rate (pounds per acre) Dryland	Irrigated
Spring grains[1]	--	40-60
Streambank wheatgrass[2]	10	20
Thickspike wheatgrass[2]	12	24
Western wheatgrass[2]	12	24
Intermediate wheatgrass	10	20
Kentucky bluegrass	6	12
Pubescent Wheatgrass	14	24
Reed canarygrass	6	12
Smooth brome	14	28
Tall fescue	10	20
Tall wheatgrass	16	32

[1] Annuals - use on slopes less than 5 percent or as temporary protection until permanent cover is established.

[2] Native grasses

At least 50 percent of any permanent seeding mixture shall consist of native grasses. The seeding depth is determined by the species requiring the shallowest depth and should be drilled in rows perpendicular to the flow direction. In addition, mulching is recommended on slopes exceeding 5 percent.

Timely maintenance is important to keep a waterway in good condition. Fertilizing and mowing or spraying for weed control should be done frequently enough to keep vegetation in vigorous condition. Grassed waterways are not permanent measures; they should be inspected and repaired after major storm events if necessary.

7.70 Bibliography

1. City of Austin, Texas, Engineering Department et al, Drainage Criteria Manual, Austin, TX, 1977.

2. Chow, Ven Te. Open-Channel Hydraulics, McGraw-Hill Book Company, Inc., New York, NY, 1959.

3. Resource Consultants, Inc., Larimer County Stormwater Management Manual, Fort Collins, CO, 1979.

4. Soil Conservation Service, Grassed Waterway or Outlet, Wyoming Engineering Standard 412, Section IV, Technical Guide, Casper, WY, 1979.

5. Wright-McLaughlin Engineers, Urban Storm Drainage Criteria Manual, Denver Regional Council of Governments, CO, 1969.

Section 8

Structures

Section 8

Structures

Hydraulic structures include energy dissipators, channel drops and transitions, baffle chutes, riprap, and many other specific drainage works. Their shape, size, and other features vary widely depending upon the function to be served.

8.10 Energy Dissipators

Energy dissipators are often necessary at the end of outfall sewers, culverts or channels. Stilling basins, a type of energy dissipator, are useful at locations were the designer wants to convert supercritical flow to subcritical flow downstream from a high velocity channel or conduit.

Impact Stilling Basin

Generally, this type of basin lends itself to use with pipes. It is an effective stilling device even with deficient tailwater where the discharge is relatively small. This basin can be used with either an open chute or a closed conduit structure. The design shown in Figure 8-1 has been used for discharges up to about 400 cfs, for larger discharges multiple basins could be placed side-by-side.

The general arrangement of the basin and the dimensional requirements for various discharges are shown on Figure 8-1 and Table 8-1. This type of basin is subject to large dynamic forces and turbulences which must be considered in the structural design. The structure must be made sufficiently stable to resist sliding against the impact load on the baffle wall. The entire structure must resist the severe vibrations inherent in this type of device, and the individual structural members must be sufficiently strong to withstand the large dynamic loads.

Riprap should be provided along the bottom and sides adjacent to the structure to avoid the tendency for scour of the outlet channel downstream from the end sill when a shallow tailwater exists. Downstream wingwalls placed at 45 degrees may also be effective in reducing scouring tendencies and flow concentrations downstream.

Plunge Pools

The plunge pool is a free-failing overflow which drops vertically into a pool.

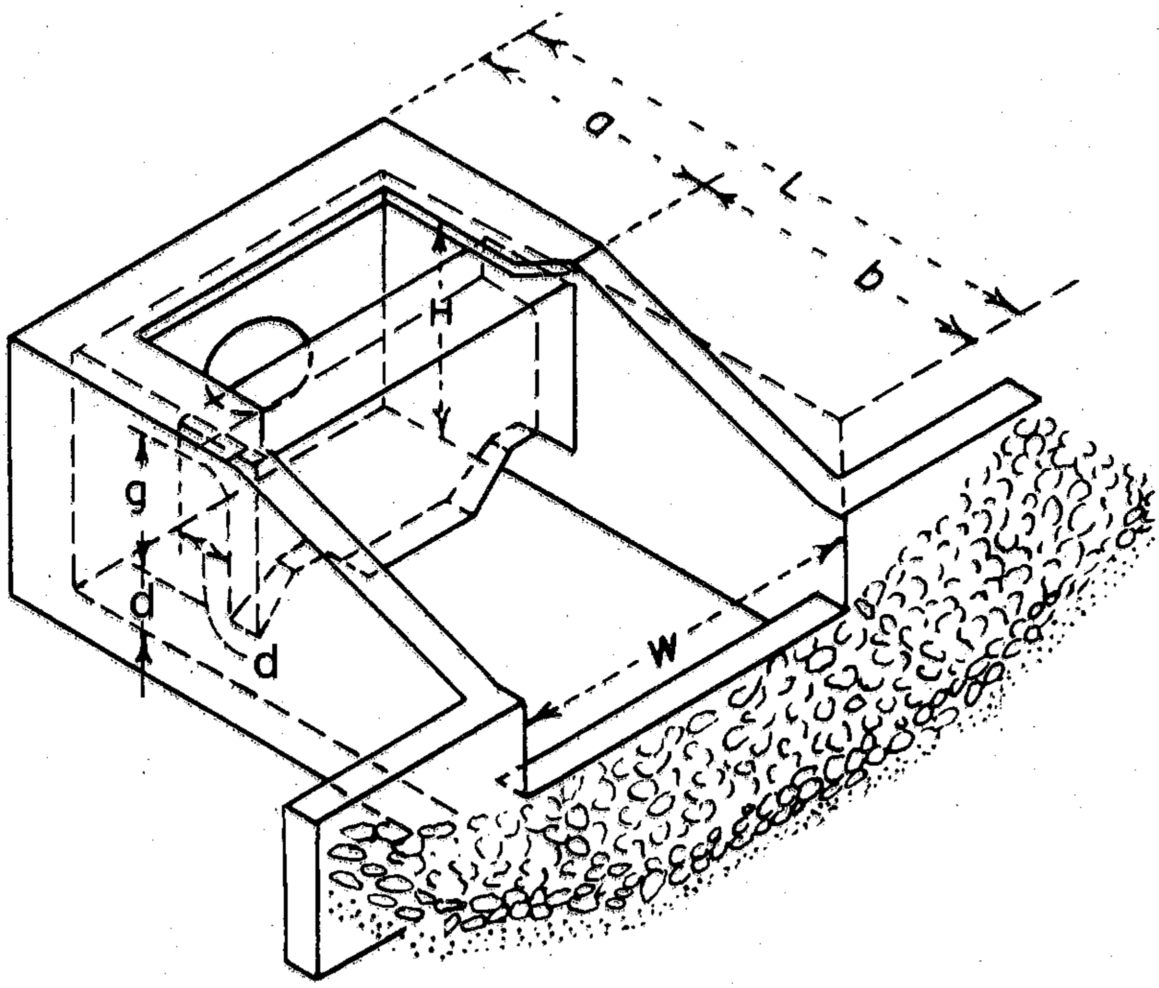
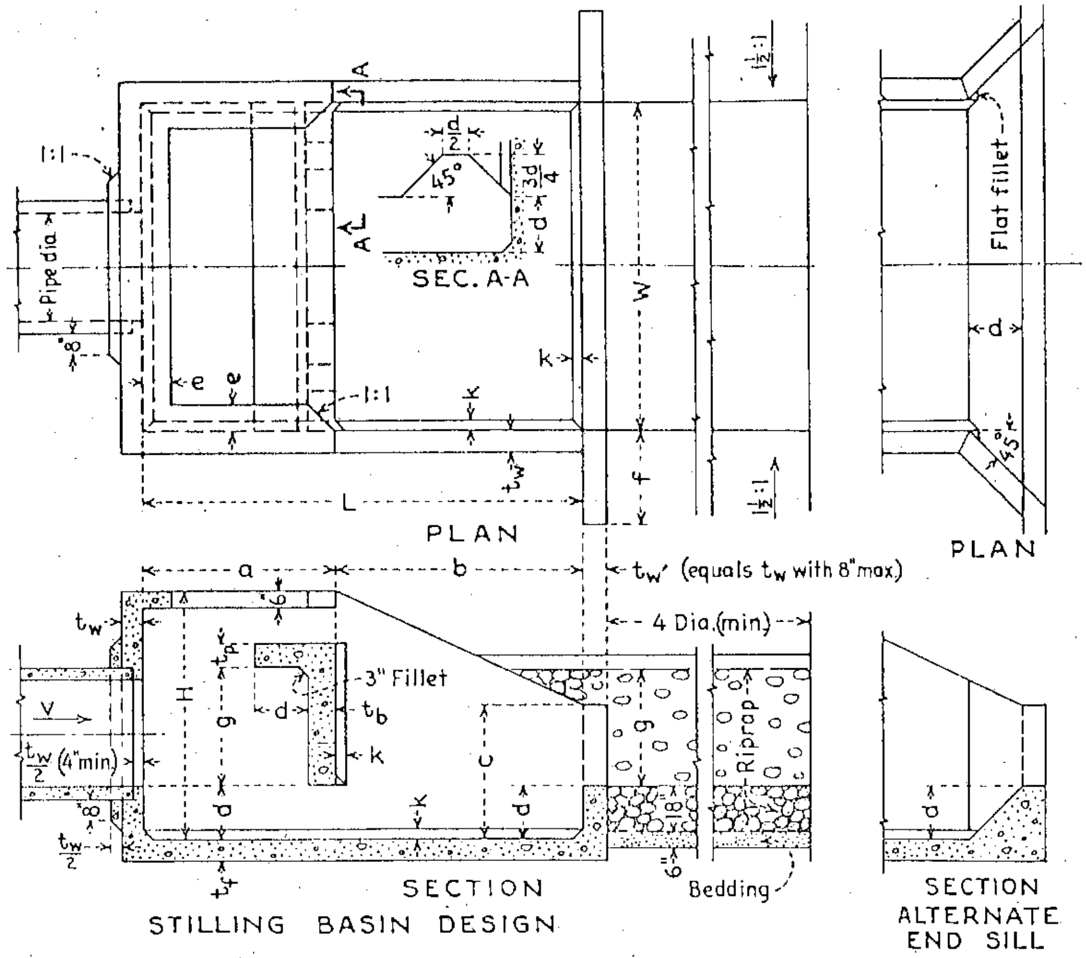


Figure 8-1a Dimensional Criteria for Impact-Type Stilling Basins



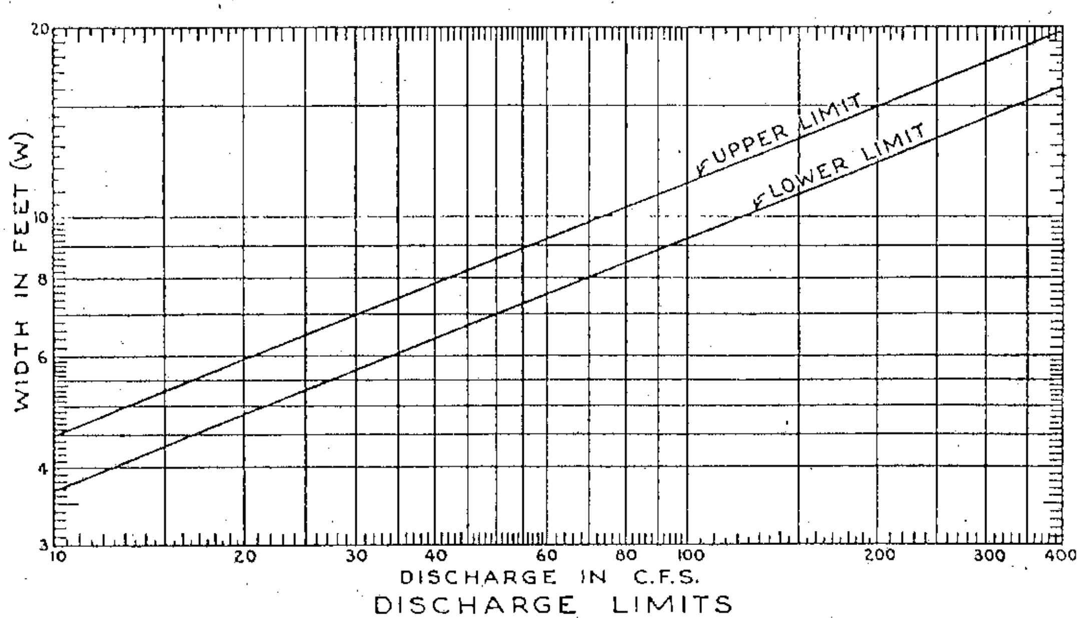


Figure 8-1b Dimensional Criteria for Impact-Type Stilling Basins, Continued

Table 8-1 Dimensions for Impact-Type Stilling Basins

Suggested pipe size[1]		Max discharge Q	Feet and Inches										Inches					
Dia in.	Area (sq ft)		W	H	L	a	b	c	d	e	f	g	t_w	t_f	t_s	t_p	K	Suggested riprap size
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)[3]
18	1.77	[2] 21	5-6	4-3	7-4	3-3	4-1	2-4	0-11	0-6	1-6	2-1	6	6½	6	6	3	4.0
24	3.14	38	6-9	5-3	9-0	3-11	5-1	2-10	1-2	0-6	2-0	2-6	6	6½	6	6	3	7.0
30	4.91	59	8-0	6-3	10-8	4-7	6-1	3-4	1-4	0-8	2-6	3-0	6	6½	7	7	3	8.5
36	7.07	85	9-3	7-3	12-4	5-3	7-1	3-10	1-7	0-8	3-0	3-6	7	7½	8	8	3	9.0
42	9.62	115	10-6	8-0	14-0	6-0	8-0	4-5	1-9	0-10	3-0	3-11	8	8½	9	8	4	9.5
48	12.57	151	11-9	9-0	15-8	6-9	8-11	4-11	2-0	0-10	3-0	4-5	9	9½	10	8	4	10.5
54	15.90	191	13-0	9-9	17-4	7-4	10-0	5-5	2-2	1-0	3-0	4-11	10	10½	10	8	4	12.0
60	19.63	236	14-3	10-9	19-0	8-0	11-0	5-11	2-5	1-0	3-0	5-4	11	11½	11	8	6	13.0
72	28.27	339	16-6	12-3	22-0	9-3	12-9	6-11	2-9	1-3	3-0	6-2	12	12½	12	8	6	14.0

1Suggested pipe will run full when velocity is 12 feet per second or half full when velocity is 24 feet per second. Size may be modified for other velocities by Q=AV, but relation between Q and basin dimensions shown must be maintained.

2For discharges less than 21 second-feet, obtain basin width from curve of Fig. 8-1b. Other dimensions proportional to W; $H=\frac{3W}{4}$, $L=\frac{4W}{3}$, $d=\frac{W}{6}$, etc.

3Determination of riprap size explained in Sec. 10.

See Reference 1 for additional information.

The pool must be heavily protected with large riprap or reinforced concrete. The approximate pool depth is given by the following equation:

$$d_S = 1.32\, H_T^{\,0.225}\, q^{\,0.54} \qquad (8\text{-}1)$$

where:

d_S = the maximum depth of scour below tailwater level, in feet;

H_T = the head from the reservoir to tailwater levels, in feet; and

q = the unit discharge, in cfs per foot of stilling basin width.

A plunge pool may only be used with a continuous low flow present in the channel because of the health and safety hazards which could be created by a stagnant pool.

Drops

The function of drop structures is to convey water from a higher to a lower elevation and to dissipate excess energy resulting from its drop. A canal along this same terrain would ordinarily be steep enough to cause severe erosion in earth canals or disruptive flow in lined canals. The water must therefore be conveyed with a drop structure designed to safely dissipate the excess energy. Different kinds of drops that may be used are vertical, baffled apron, rectangular inclined, and pipe drops.

Vertical drops are often the most economical for small drops of less than three feet. They consist of a simple weir above a vertical retaining wall structure and a splash-pool-type energy dissipator that are consolidated in a single inexpensive structure. These structures can be constructed from sheet pile and riprap, gabion retaining walls and channel mats, or reinforced concrete.

Baffled apron drops may be used for nearly any decrease in water-surface elevation where the horizontal distance to accomplish the drops is relatively short. They are particularly adaptable to the situation where the downstream water-surface elevation may vary because of such things as degradation or an uncontrolled water surface. A further discussion on baffled aprons may be found in References 1 and 2.

Rectangular inclined (RI) drops and pipe drops are used where the decrease in elevation is in the range of 3 to 15 feet over a relatively short distance. Economics dictate if it is more practicable to use a pipe drop or an RI drop. Usually a pipe drop will be selected for the smaller flow and an RI drop will be selected for the larger flows. If the drop crosses another waterway or a roadway it will probably be more economical to use a pipe drop.

Chutes are usually used where the drop in elevation is greater than 15 feet and where the water is conveyed over long distances and along grades that may be flatter than those for drops but yet steep enough to maintain supercritical velocities. The decision as to whether to use a chute or a series of drops should be based upon a hydraulic and economic study of the two alternatives. Drops should not be so closely spaced as to possibly preclude uniform flow between outlet and inlet of consecutive structures, particularly where checks or control notches are not used at the inlets. The danger is that sufficient tailwater depths may not exist to produce hydrualic jumps in the pools, and thus shooting flow may develop through the series of drops and possibly damage the canal. Also, with drops too closely spaced on a steep slope, problems of excavation and backfill may make such construction undesirable or prohibitive. About 200 feet of canal should be the minimum distance between the inlet and outlet of consecutive drop structures. The economic study should compare costs of a series of drops and a chute considering advantages and disadvantages pertinent to the specific conditions. Since the maintenance costs for a series of drops is usually considerably more than for a chute accomplishing the function, it is sometimes economically justifiable to spend considerably more for a chute than for a series of drops. A more complete discussion on chutes will be found in References 1, 2, and 4.

8.20 Flow Transitions

A flow transition is a change of cross section designed to be accomplished in a short distance with a minimum amount of flow disturbance. The types of transitions are shown in Figure 8-2. Of these the abrupt (headwall) and the straight line (wingwall) are the most common.

Special inlet transitions are useful when the conservation of flow energy is essential because of allowable headwater considerations.

Outlet transitions (expansions) must be considered in the design of all culverts, channel protection, and energy dissipators. The standard wingwall-apron combinations and expansions upstream of dissipator basins are most common.

8.30 Riprap

Preventing bank damage caused by surges from a stilling basin and forestalling possible undermining of the structure caused by erosive currents passing over the end sill usually requires placing riprap on the channel bottom and banks downstream.

Experience has shown that the primary reason for riprap failure is undersized individual stones in the maximum size range. Failure has occurred because of an underestimation of the required stone size, and a general tendency for the riprap in place to be smaller than specified, despite quality-control procedures.

The curve in Figure 8-3 gives the individual minimum stone size (diameter and weight of a spherical specimen) for a range of bottom velocities up to 17 feet per second.

A well-graded riprap layer containing about 40 percent of the rock pieces smaller than the required size is as stable, or more stable, than a uniformly-graded layer consisting entirely of stones of the required size.

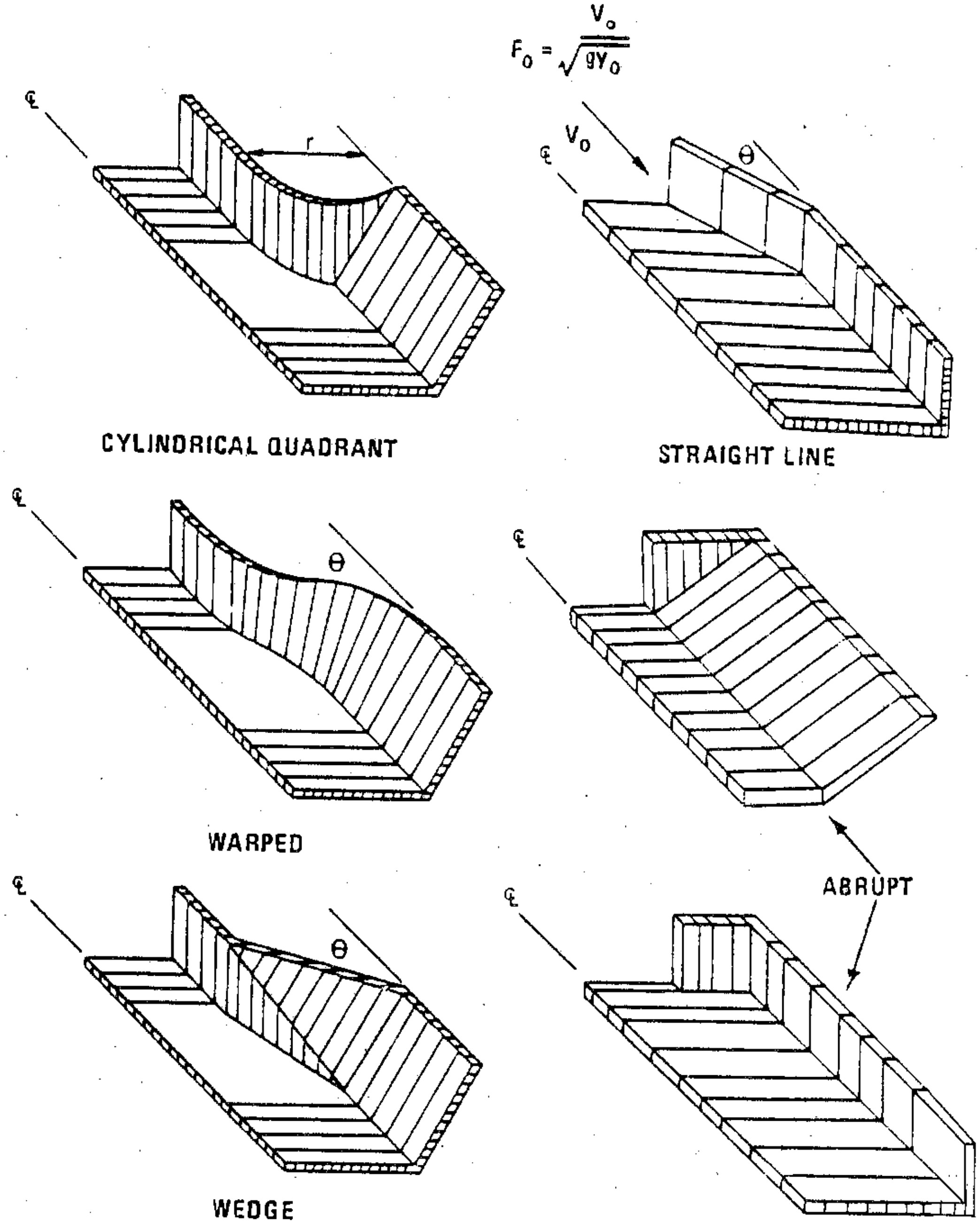


Figure 8-2 Transition Types.

Most of the mixture should consist of stones having nearly symmetrical dimensions and of the size shown in Figure 8-3 or larger; else the stones should be of curve weight as shown in Figure 8-3 or more (weight is computed on the basis of 165 pounds per cubic foot) and should not be flat slabs.

The riprap layer should be a minimum of 1.5 times as thick as the dimension of the large stones (curve size) and should be placed over a gravel or reverse filter layer.

A minimum of 12 inches of granular bedding or suitable filter fabric covered with 4 inches of sand shall be provided as a filter layer beneath the riprap layer. For noncohesive soils the 15-percent size (d_{15}) of the granular bedding or filter material should be less than 4 to 5 times the 85-percent size (d_{85}) of the adjacent soil layer. The 15-percent size of the filter material should also be at least 4 to 5 times the 15-percent size of the protected soil. For cohesive soils such as highly plastic clays, the 15-percent size of the filter material may be as great as 0.4 mm so long as the material is well graded with the ratio of the 60-percent size to the 10-percent size (coefficient of uniformity) not exceeding 20. Section 11 should also be consulted regarding erosion control practices utilizing riprap lining.

8.40 Bibliography

1. Federal Highway Administration, Hydraulic Design of Energy Dissipators for Culverts and Channels, Hydraulic Engineering Circular No. 14, Government Printing Office, Washington, DC, 1975.

2. Cedergren, H.R., Seepage, Drainage, and Flow Nets, 2nd ed., John Wiley & Sons, New York, NY, 1977.

3. Peterka, A.J., Hydraulic Design of Stilling Basins and Energy Dissipators, U.S. Department of the Interior, Bureau of Reclamation, Denver, CO 1978.

4. Wright-McLaughlin Engineering, Urban Storm Drainage Criteria Manual, Denver Regional Council of Governments, CO, 1969.

5. U.S. Department of the Interior, Bureau of Reclamation Design of Small Canal Structures, Government Printing Office, Washington, DC, 1978.

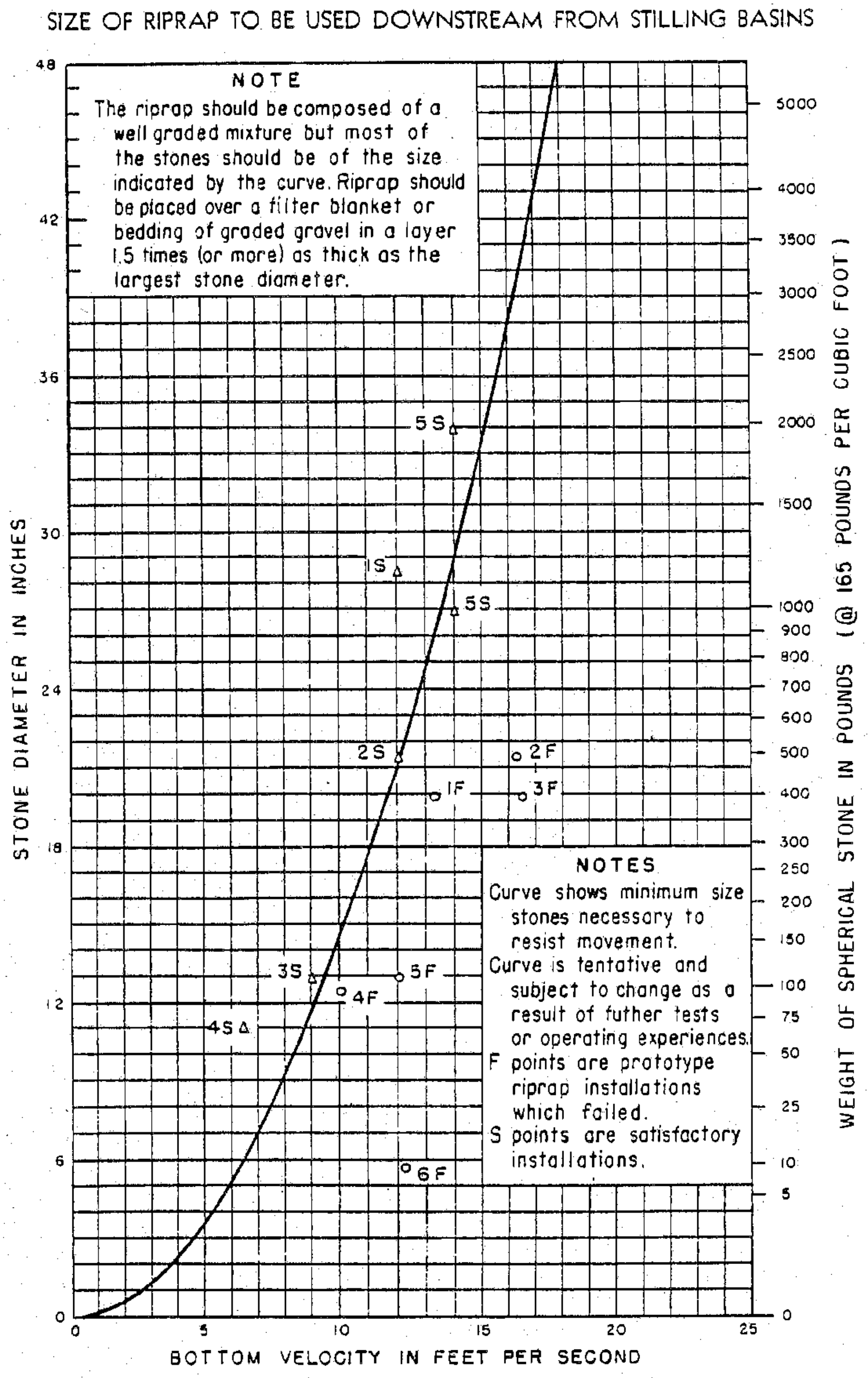


Figure 8-3 Maximum Stone Size for Riprap

Section 9

Storage

Section 9

Storage

On-site detention of runoff is an alternative to other methods of urban stormwater management. Storage, which involves collecting excess runoff before it enters the main drainage system, can often be an effective and economical means of reducing peak flowrates and mitigating problems of flooding, pollution, soil erosion, and siltation.

The purpose of this Section is to introduce the design procedures and methods of application of on-site detention facilities. This includes rooftop storage, parking lot storage, recreational area storage, and small detention basins and ponds constructed within the limits of development areas.

On-site detention of stormwater generally refers to storage of excess runoff on the site of a development prior to its entry into a sewer system and the gradual release of the stored runoff after the peak flow has passed. Generally, detention facilities will not reduce the total volume of runoff, but will redistribute the rate of runoff over a longer time period.

9.10 Upstream Storage

Upstream storage utilization is usually controlled by the planner in the early stages of development. However, architects, engineers, home builders, land developers, and governmental officials all have a responsibility to work towards more upstream storage. It is with upstream storage that the greatest potential exists for reducing the cost of urban drainage.

Rooftop Ponding

In the Casper area building codes require the design of roofs for a snowload of 30 pounds per square foot. In terms of equivalent water this equals 5.8 inches. Thus, most existing buildings are already designed to carry the load which would be imposed by rooftop rainfall ponding.

The drainage outlet from the rooftop storage should be sized to release approximately 0.5 inches per hour. During most storms, no water will be stored because it will runoff as quickly as it falls. Only in the larger storms will water accumulate. For instance, in a heavy thunderstorm lasting 30 minutes having a precipitation rate of 2 inches per hour, the maximum depth of ponding will be about 0.75 inches. About 90 minutes after the storm all water will have drained off the roof.

Parking Lots

There are two general forms of stormwater detention on parking lot surfaces. One form involves the storage of runoff in depressions constructed at drain locations. The stored water is drained into the sewer system slowly, using restrictions, such as orifice plates, in the drain. Proper design of such paved areas would restrict ponding to areas which would cause the least amount of inconvenience to the users of the parking areas. For example, the parking lot of a shopping center would have the ponding areas located in the least-used portions of the lot, allowing customers to walk to their vehicles in areas of no ponding, except when the entire lot is filled with vehicles. Drainage of ponded water would be fairly rapid as compared to rooftop ponding, to prevent customer inconvenience. In most cases, the water would pond to a depth not to exceed 12 inches and the ponding area would most likely be drained within 30 minutes or less after the rainfall. Computation of the amount of storage needed would be similar to the analysis used in designing detention basins on ground surfaces.

Another type of stormwater detention on parking lots consists of using the paved areas of the lot to channel the runoff to grassed areas or gravel-filled seepage pits. The flow then infiltrates into the ground. Soil conditions and the effects of siltation in reducing infiltration must be considered.

Minimum slopes of 1.0 percent are recommended in parking lot detention areas. Maximum slopes should not exceed 4 percent to avoid gasoline spillage from tanks and to minimize vehicle traction problems on icy paving.

The use of parking lots for the storage of rainfall, if done properly, can be accomplished without inconvenience to the parking lot users and without interfering with the layout and functioning of the parking lot. Furthermore, controlled rates of release are generally large enough so that water is ponded only during the larger storms.

Recreational Areas

Recreational areas generally have a substantial area of grass cover which often has a high infiltration rate. Storm runoff from such fields is generally minimal. The best use of such recreational fields can be made by providing for the ponding of runoff from adjacent areas. Grassed recreational fields can be utilized for the temporary detention of the storm runoff without adversely affecting their primary function.

Most urban areas contain parks, both the neighborhood type and the large central type. Parks, like recreational fields, create little runoff of their own; however, parks provide excellent detention storage potential for the storage of runoff from adjacent areas.

Property-Line Swales

Subdivision planning and layout requires adequate surface drainage away from buildings. This is obtained by sloping the finished grade at approximately 1 percent in all directions away from the buildings. The layout often calls for a swale to be located along the back property line, which then drains longitudinally through the block. The final grading plan for the lot layout can readily be done in such a manner as to cause up to six inches of temporary ponding along the property line.

Temporary ponding facilities along the rear lot line may include small controlled discharges or, if the subsoil conditions are favorable, such water may be percolated into the ground.

Road Embankments

The temporary detention of storm runoff behind road embankments is a good practice and is encouraged. The reader is referred to Sections 5 and 6 of this Manual dealing with inlets and culverts for design criteria relative to the use of ponding behind road embankments. It is, of course, necessary to review the damage potential to upstream property.

The design criteria to be used for the temporary detention of water behind road embankments should include consideration of the major storm runoff, i.e., the runoff to be expected once each 100 years.

The use of roadway embankments to help reduce downstream peak flows is encouraged in the Casper area. Planning for such use of embankments must avoid unnecessary damage to the embankment, structure, and adjacent property.

On-Site Ponds

The construction of on-site ponds which have recreational benefits provides significant detention benefits when properly planned and designed. The use of such ponds is particularly encouraged in planned unit developments where large areas of grass and open space are common.

Controlled outlets for the surcharge storage can be used, and it is suggested that such outlets be designed to release at a rate that does not exceed the rate estimated for natural condtions for the 10-year storm.

Porous Pavements

The use of porous pavements as a method of storing or attenuating storm runoffs is in the development stage. However, porous pavement used in parking and other areas can reduce flood peaks by allowing water to infiltrate.

Combinations

In many instances, one on-site detention method cannot conveniently or economically satisfy the required amount of stormwater storage. Limitations in storage capacities, site development conditions, soils limitations, and other related constraints may require that more than one method be utilized. For example, rooftop, parking lot, and surface pond storage might all be required to compensate for increases in runoff due to development of a particular site. Whichever combinations are suitable should be incorporated into the site development plan.

9.20 Design Criteria

Design Storms

The 10-year, 2-hour storm hydrograph (or in the case of the Modified Rational Method, the 10-year storm of the critical duration) shall be routed through the storage area with a maximum release rate which does not exceed the peak discharge for the same storm under natural conditions. Either a constant release rate or the actual discharge for the principal outlet structure obtained from a rating curve may be utilized. The maximum storage volume and water-surface elevation for the initial storm routing shall be indicated.

The 100-year, 2-hour storm hydrograph is then routed through the storage area with a starting water-surface elevation equal to the maximum elevation obtained from the initial storm routing. Either a riser pipe or spillway shall be used to convey the routed outflow from the 100-year, 2-hour storm. The crest of the riser pipe or spillway shall be established at the starting water-surface elevation. Site conditions will determine whether a riser pipe or spillway is selected.

Principal Outlets

Either corrugated metal or reinforced concrete pipes may be utilized for the principal outlet. The minimum pipe diameter shall be 18 inches unless a waiver is obtained from the Wyoming State Engineer. The principal outlet shall be able to completely drain the storage area.

To prevent the formation of vortices at the inlet, a hood or anti-vortex baffle shall be installed. Vortices can significantly reduce the discharge for a given headwater because of energy losses.

Depending on the geometry of the outlet structure (either drop-inlet riser or hood-inlet pipe) discharge for various headwater depths can be controlled by the inlet crest (weir control), the riser or barrel opening (orifice control), or the riser or barrel pipe (pipe control). Each of these flow controls shall be evaluated when determining the rating curve of the principal outlet. In general, the riser pipe diameter is at least 6 inches greater than the barrel pipe diameter. Therefore, the minimum riser pipe diameter shall be 24 inches.

Weir flow may be computed by the following equation:

$$Q = CLH^{1.5} \tag{9-1}$$

where:

Q = discharge, in cfs;
C = weir coefficient. For riser pipes, C = 3.1 may be used;
L = length of the weir, in feet. For circular riser pipes, L is the pipe circumference; and
H = the depth of flow over the pipe crest, in feet.

Orifice flow may be computed by the following equation:

$$Q = CA(2gH)^{0.5} \tag{9-2}$$

where:

C = orifice coefficient. For sharp-edged orifices, $C = 0.6$;
A = cross-sectional area of the pipe, in ft^2;
g = gravitational acceleration, 32.2 ft/sec^2; and
H = head above the centerline of the pipe, in feet.

Pipe flow may be computed by the following equation:

$$Q = A\sqrt{2g(H-S_fL)/(1+k_b+k_e)} \tag{9-3}$$

where:

H = the difference between headwater and tailwater elevations, in feet;
k_b = bend loss coefficient;
k_e = entrance loss coefficient;
S_f = friction slope, in feet per foot; and
L = length of pipe, in feet.

Spillways

Spillway velocities shall be limited by the criteria listed in Section 6 (Open-Channel Flow). At least 5 feet of freeboard shall be provided between the top of the embankment and the crest of the spillway or riser pipe. Also, at least 1.5 feet of freeboard shall be provided between the top of the embankment and the maximum 100-year water-surface elevation for the major storm routing.

The outlet channel slope should be greater than critical slope for one-fourth of the peak outflow to assure that flow will be controlled at the spillway section. Otherwise, both weir control and open-channel control should be evaluated over the range of headwater depths.

Retention of Stormwater

Note

Retention storage is prohibited by the Wyoming State Engineer. All storage areas shall be drained within 72 hours of the end of the storm by gravity flow through the principal outlet.

Site Conditions

The Geologic Hazard Map of the site shall be consulted to determine the suitability for impoundment of surface water. Groundwater level increases downstream of the storage area are to be avoided. A Geologic Hazard Map may be obtained from the Natrona County Planning Department.

Embankments

Proposed embankment slopes shall not be steeper than 3:1 on the upstream face. The sum of the upstream and downstream slopes shall not be less (steeper) than 5:1. The width of the top of the embankment shall not be less than the fill height divided by 5 plus 4 feet, or 8 feet, whichever is greater.

Design Plans

Note → All design plans shall be submitted to the State Engineer's Office for approval. Variances from the above criteria must be obtained in writing from the State Engineer.

9.30 Hydraulic Design Methods

The two methods suggested for predicting the volume of runoff over time and the peak flow are the Modified Rational Method and the Colorado Urban Hydrograph Procedure.

Modified Rational Method Analysis

The term Modified Rational Method Analysis refers to a procedure for manipulating the basic Rational Method to reflect the fact that storms with durations greater than the normal time of concentration for a basin will result in a larger volume of runoff even though the peak discharge is reduced. This greater volume of runoff produced by longer storm durations must be analyzed to determine the correct sizing for detention facilities.

The approach becomes more valid on progressively smaller basins, eventually reaching a size so small that watershed modeling is approached. The procedure should, therefore, be limited to relatively small areas such as rooftops, parking lots, or other upstream areas with tributary basins less than 20 acres. This would minimize major damage which could result from overtopping or failure of the proposed detention facility.

Figure 9-1, Modified Rational Method Hydrographs, presents a family of curves for a theoretical basin. These hydrographs are developed by using the basic Rational Method assumptions of constant rainfall intensity, time of concentration for the longest flow path, and the coefficient of runoff. The typical Rational Method hydrograph with the peak discharge coinciding with the time of concentration for the basin (T_C) is first calculated using the normal formula $Q = CiA$. Following this, a family of hydrographs representing storms of greater duration are developed. The peak runoff rate for each hydrograph is equal to CiA where i is the rainfall intensity for the storm duration in question. The rising limb and falling limb of the hydrograph are, in each case, equal to T_C for the basin. The basic assumption of this method is that the area under the assumed trapezoidal hydrograph equals the volume of runoff from the theoretical rainfall. The area under the hydrograph is also equal to the peak discharge rate for that particular rainfall multiplied by the duration of the rainfall.

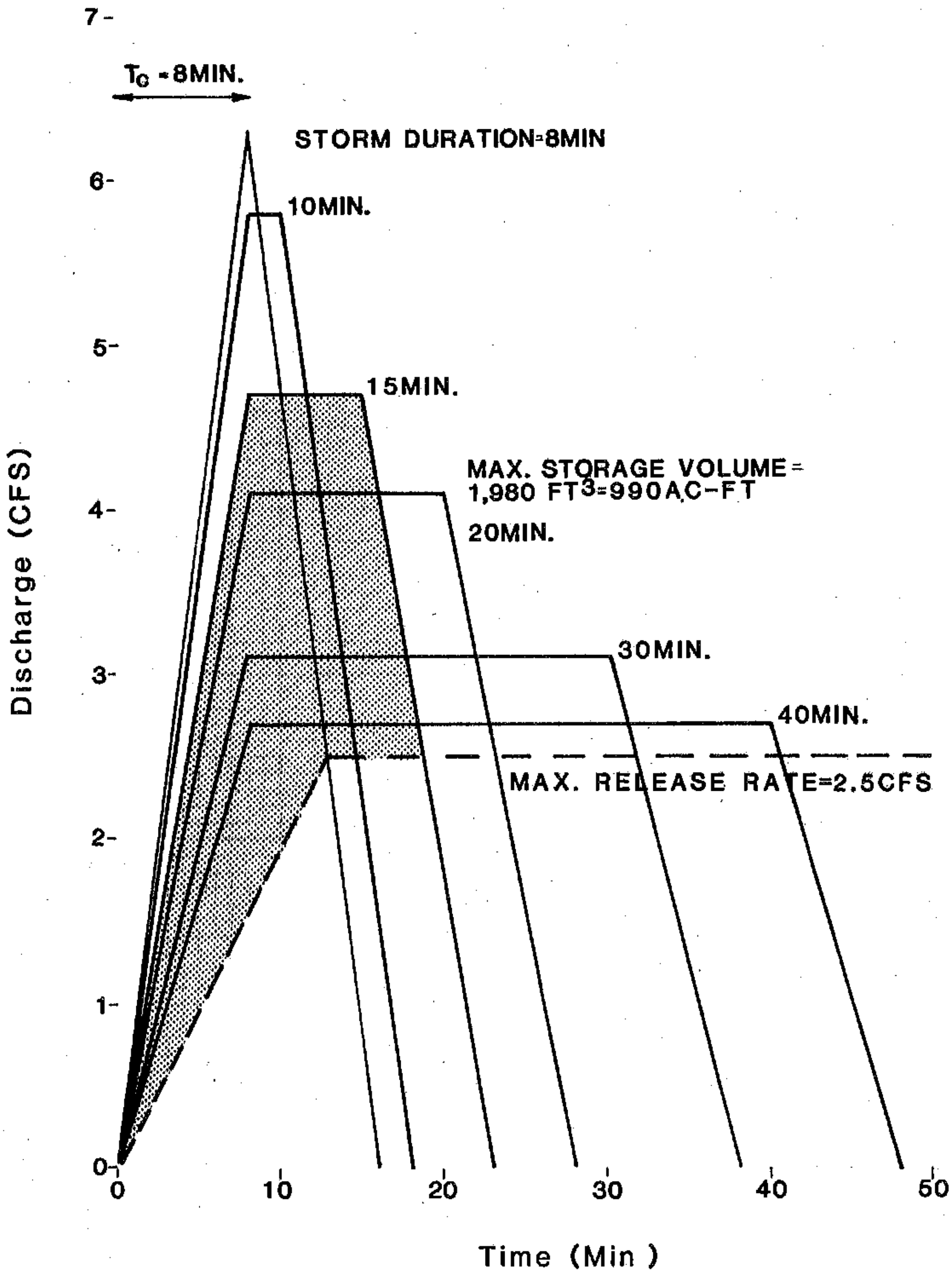


Figure 9-1 Modified Rational Method Hydrographs

The following example presents the calculation method for a typical two-acre basin.

Example 1

Given: Area: A = 2.0 acres
Type of development: commercial parking lot, fully paved, C = 0.9
Time of Concentration: T_C = 8 minutes
Design Frequency = 10 years
Use Rainfall-Intensity Duration Curves in Figure 2-1.

Required: Develop family of curves representing Modified Rational Method hydrographs for the 8-, 10-, 15-, 20-, 30- and 40-minute rainfall durations.

Rainfall Duration (min)	Rainfall Intensity (in./hr)	Peak Runoff Rate (cfs)
8	3.5	6.3
10	3.2	5.8
15	2.6	4.7
20	2.3	4.1
30	1.8	3.2
40	1.5	2.7

Answer: The resulting storm hydrographs are depicted in Figure 9-1.

It is recommended that a coefficient be added to the Rational Method to account for antecedent precipitation conditions for major storms with recurrence intervals greater than 10 years. These coefficients are presented in Table 2-3. Under these conditions, the Rational Formula becomes $Q = C_fCiA$. Although this approach does not totally reconcile the difficulties in representing volume of runoff by the Rational Method, it does attempt to predict more realistic hydrograph volumes characteristic of high-frequency storms.

The next step in determining the necessary storage volume for the detention facility is to set a release rate and determine the volume of storage necessary to accomplish this release rate.

To determine the storage volume required, a reservoir routing procedure should be accomplished for each of the hydrographs, with the critical storm duration and required volume being determined. The importance of the particular project should govern the type of routing utilized. For small areas requiring repetitive calculations, such as in bays of a parking lot, an assumed release curve is normally satisfactory. For larger areas, such as a pond in a small park with 20 acres or more of tributary area, a reservoir routing procedure would be in order.

Figure 9-1, Modified Rational Method Hydrographs, represents a method for small area detention analyses. The assumed release curve approximates a formal reservoir routing in much the same way the Rational Method Hydrograph approximates a true storm hydrograph. The curve allows for the low release rate at the beginning of a storm and an increasing release rate as the storage volume increases.

In normal flood routing, the maximum release rate will always occur at the point where the outflow hydrograph crosses the receding limb of the inflow hydrograph. For this reason, the design release rate is forced to coincide with that point on the falling limb of the hydrograph resulting from the storm of duration equal to the time of concentration for the basin. The release rate is held constant past this point. The storage volume is then found by determining the area between the inflow and release hydrographs. Example 2 continues the calculations initiated in Example 1 to determine the required storage volume. The equation for storage volume, V_s, in ft^3, is as follows:

$$V_s = 60\ D(Q_p - Q_o) \qquad (9-4)$$

where:

D = storm duration, in min;
Q_p = peak discharge of the inflow hydrograph, in cfs; and
Q_o = maximum release rate, in cfs.

Example 2

Given: Drainage basin and other hydrologic information presented in Example 1.

Allowable release rate: $Q_o = 2.5$ cfs

Required: Determine the critical storage volume.

Storm Duration (min)	Storm Runoff Volume (ft^3)	Release Flow Volume (ft^3)	Required Storage Volume (ft^3)
8	3020	1200	1820
10	3480	1500	1920
15	4230	2250	1980
20	4920	3000	1920
30	5760	4500	1260
40	6480	6000	480

The critical storage volume is then 1,980 ft^3 occurring for a 15-minute rainfall duration.

The limitations in the assumptions behind this method are evident. The approach becomes more valid on progressively smaller basins. The procedures should, therefore, be limited to relatively small areas where no major damage would result from over-topping or failure of the proposed detention facility. Care should be used when applying this method to areas in excess of 20 acres.

The Colorado Urban Hydrograph Procedure for Storage Analysis

The Colorado Urban Hydrograph Procedure (CUHP) develops a hydrograph which provides a reliable solution for detention storage effects. Procedures for the CUHP provide the designer greater flexibility for the representation of actual conditions to be modeled.

The development of the runoff hydrograph is presented in Section 2 of this Manual. A flood routing procedure may be used to determine required volume of the detention basin. Several flood routing procedures are available in published texts. The data needed for the routing computations are the inflow hydrograph, the physical dimensions of the storage basin, the maximum outflow allowed, and the hydraulic characteristics of the outlet structure or spillway. This method is referred to as the Modified Puls routing procedure.

After the inflow hydrograph, depth-storage relationship, and depth-outflow relationship have been determined, they are combined in a routing routine. The results of the routing are the ordinates of the outflow hydrograph, the depths of storage, and the volumes of storage at each point in time of the flood duration.

The routing period, or time interval, Δt, is selected small enough so that there is a good definition of the hydrograph and the variation in the hydrograph during the period Δt is approximately linear. This can be accomplished by setting Δt = 5 minutes, or the same time interval as in the CUHP.

Several assumptions are made in this procedure and include the following:

1. The entire inflow hydrograph is known.

2. The storage volume is known at the beginning of the routing.

3. The outflow rate is known at the beginning of the routing.

4. The outlet structures are such that the outflow is uncontrolled and the outflow rate is dependent only on the headwater.

The derivation of the routing equation begins with the conservation of mass which states that the difference between the average inflow and average outflow during some time period t, is equal to the change in storage during that time period. This can be written in equation form

as:

$$\bar{I} - \bar{O} = \Delta S / \Delta t \qquad (9\text{-}5)$$

If inflow during the period is greater than outflow, then ΔS is positive and the pond gets deeper. If inflow is less than outflow during the period, then ΔS is negative and the pond gets shallower. Using the assumptions made previously, this equation can be rewritten as:

$$\frac{I_1 + I_2}{2} - \frac{O_1 + O_2}{2} = \frac{S_2 - S_1}{\Delta t} \tag{9-6}$$

Multiplying both sides by two and separating the right-hand side yields:

$$I_1 + I_2 - O_1 - O_2 = \frac{2S_2}{\Delta t} - \frac{2S_1}{\Delta t} \tag{9-7}$$

Rearranging so that all the known terms are on the left-hand side and all the unknown terms on the right-hand side yields the final routing equation:

$$(I_1 + I_2) + \left(\frac{2S_1}{\Delta t} - O_1\right) = \left(\frac{2S_2}{\Delta t} + O_2\right) \tag{9-8}$$

However, the last equation has two unknowns, S_2 and O_2. We need a second equation which relates storage and outflow. If outflow is a direct function of reservoir depth (as it is with uncontrolled outflow), there is a direct relationship that exists between reservoir elevation, reservoir storage, and outflow. Therefore, for a particular elevation, we have an answer for storage and outflow (S and O). A relationship between O and $(2S/\Delta t)+O$ is determined for several elevations and plotted on logarithmic graph paper. The routing equation is solved by adding all the known terms on the left-hand side. This yields a value for $(2S/\Delta t)+O$. This value is found on the log-log plot of $(2S/\Delta t)+O$ versus O and the value of O_2 is obtained from the graph.

The $(2S/\Delta t)+O$ versus O relationship is derived by combining the depth-storage relationship and the depth-outflow relationship, as previously discussed. This is shown in Table 9-1. Columns 1, 2, and 3 are tabulations of the depth-storage and depth-outflow relationships for a specific detention facility.

In column 4, the units of $2S/\Delta t$ and O must be the same. If O is in cfs then $2S/\Delta t$ must be changed to cfs. For a routing time interval of 5 minutes:

$$\frac{2S \text{ ac-ft}}{5 \text{ min}} \times \frac{1 \text{ cfs-day}}{1.98 \text{ ac-ft}} \times \frac{1440 \text{ min}}{1 \text{ day}} = 291S$$

Thus, $\frac{2S + O}{\Delta t} = 291S + O$ for $\Delta t = 5$ min.

where: S has units of acre-feet, O has units of cfs, and $2S/\Delta t$ has units of cfs.

TABLE 9-1

Development of a (2S/ Δt) + O vs. O Relationship

Depth (ft) (1)	Storage,S (ac-ft) (2)	Outflow,O (cfs) (3)	(2S/ Δ t) + O (cfs) (4)
0	0.0	0	0
2	0.1	40	69
4	0.6	138	313
6	3.0	274	1,147
8	11.0	426	3,627
10	32.0	560	9,872
12	72.0	671	21,623
14	131.0	765	38,886

The sizing of the outlet works for detention ponding is a matter of judgment depending upon the actual conditions for the specific case. The designer may approach the sizing of the outlet works on a trial- and-error basis with the objective being the optimum use of the available storage capacity of the ponding area. In addition all ponding areas must be carefully analyzed in regard to the major storm runoff conditions. In many cases it will be found that the initial storm runoff should be routed through the outlet works with only minimal ponding. The storage would then be utilized to reduce the major runoff. If other provisions are made for the major runoff the designer may be primarily concerned with the initial drainage system. The outlet capacity would then be substantially less than the inflow from the initial runoff. It cannot be overemphasized, however, that the use of downstream channel storage requires competent planning and design to avoid the creation of an unnecessary hazard.

9.40 Bibliography

1. City of Austin, Texas, Engineering Department et al., Drainage Criteria Manual, Austin, TX, 1977.

2. Henningson, Durham & Richardson, P.S., City of Yakima, Washington, Stormwater Drainage Control Manual, Yakima, WA, 1982.

3. Poertner, Herbert G., Practices in Detention of Urban Stormwater Runoff, American Public Works Association Special Report No. 43. Chicago, IL, 1974.

4. State Engineer's Office, State of Wyoming, Regulations and Instructions, Part I--Surface Water, Cheyenne, WY, 1974.

5. Ward, Andy; Haan, Tom; and Tapp, John, The DEPOSITS Sedimentation Pond Design Manual, University of Kentucky Institute for Mining and Minerals Research, Lexington, KY, 1979.

6. Wright-McLaughlin Engineering, Urban Storm Drainage Criteria Manual, Denver Regional Council of Governments, CO, 1969.

Section 10

Flood Proofing

Section 10

Flood Proofing

Flood proofing is defined by Federal Insurance Administration as any combination of structural and nonstructural additions, changes or adjustments to structures which reduce or eliminate flood damage to real estate or improved real property and sanitary facilities, structures, and their contents. The Federal Insurance Administration has published several references to provide detailed criteria and design procedures for flood-proofing structures. These references are listed in the bibliography of this Section and should be consulted for detailed design information.

10.10 Flood-Proofing Requirements

Any proposed structures located within the 100-year floodplain should either have the lowest floor elevated to one foot above the 100-year flood elevation or should be flood-proofed to that elevation. The 100-year floodplain and corresponding elevations of the 100-year flood can be determined by reviewing the Flood Hazard Boundary Maps and Flood Insurance Rate Maps published by the Federal Insurance Administration and available for review in the office of the appropriate agency.

10.20 Types of Flood Proofing

There are two basic systems for flood proofing for residential and commercial structures. One system consists of the use of conventional construction methods along with a total drain, sump, and pump operation to keep water away from the lower portions of the structure. This type of system is feasible in regions with soils having low coefficients of permeability (clay soils). This system is termed the Drain or Sump System and is the more economical method. This system accepts the inevitability of some infiltration from a high head of water into a reasonably economical basement construction via the sump. The sump type basement protection system is adaptable to low and medium volumes of inflow of flood waters within the capability of its drainage and pumping system. This system is only applicable to structures having a grade line above the 100-year flood elevation on all sides.

The other system utilizes a watertight wall and slab treatment for all portions of the structure beneath the 100-year flood elevation. This type of system is required for soils having a high coefficient of permeability (sandy soils). This is termed an Undrained or Barge System and is required where inflow is beyond the capacity of a sump drain system or where the grade line of the proposed structure cannot be located above the 100-year flood elevation. An undrained structure is designed to be watertight. All openings below the 100-year flood elevation are equipped with flood-proofed, watertight closures.

Both these systems will allow flood proofing of structures up to 5 feet above the bottom of the lowest floor slab. This height restriction is due mainly to buoyancy considerations. Special design features will allow flood proofing to higher elevations.

10.30 Procedures

Several general procedures to achieve flood proofing are discussed in this Section. Some of these procedures, such as laying out sites and raising buildings, are intended primarily for new construction which would represent a proper use of floodplain sites. Other procedures such as those to keep the water out or those to minimize losses if the water gets in, would apply to both new and existing structures.

Site Layout

The practice of clustering buildings is prevalent in planned unit developments. This clustering permits buildings to be attractively grouped on parts of a site which are above flood levels and reserves the low-lying sections as landscaped green areas and parking facilities.

Where natural high ground does not exist, sites can be raised by filling, providing the fill does not interfere with the flow of flood waters (i.e., the structure is not located within the "floodway").

Flood-proofing measures can be designed to blend with the overall appearance of a structure. When this is done, a structure's appearance can be preserved and in some cases even enhanced by flood-proofing.

Elevated Structures

The practice of elevating a building on "stilts" to provide an "open" effect at ground level can also reduce the flood hazard. If some means of access is maintained and utilities can continue to function, activities would not be interrupted during floods.

Where land is at a premium, as in central business districts, buildings are often placed on stilts with parking facilities on the ground level.

Waterproofing Structures

The design techniques discussed above either achieve flood-proofing through site planning and development or incorporate flood-proofing in the initial construction of the buildings. The location and environment of structures in urban areas may make these solutions impractical. In these circumstances the building owner, architect, or engineer is faced with a job of designing flood-proofing measures for existing conventional buildings which are exposed to flood water. These flood proofed buildings can incorporate many contemporary design features such as large window areas, pedestrian arcades, open floor space, and curtain wall panels.

In designing new structures, or in altering existing ones, thought should be given to the use of recessed flood shields which are normally hidden from view, but can be easily lowered or slid into place upon the receipt of a flood warning. Such shields also escape the danger of being misplaced.

When flood shields must be mounted on the street side of an opening, the brackets to which they would be bolted can be concealed with easily removed aluminum strips or "skins."

Internal Flood-Proofing Measures

Owners of buildings which are subject to flood but cannot be easily altered to keep the water out can consider the use of water-resistant construction materials to reduce flood damage. Even owners of flood-proofed structures generally able to withstand flooding would also be well advised to consider the use of such materials to reduce losses when flooding exceeds the protection level.

10.40 Engineering Aspects

Flood-proofing to keep the water out of buildings falls, in part, within the province of the structural engineer. When flood waters surround a building they impose loads on the structure and substructure beyond those it normally is designed to withstand. A determination of these loads is a prerequisite of flood-proofing efforts.

The following paragraphs discuss some of the more common structural problems that could be encountered. Because of the complexity of these problems, building owners who are contemplating flood-proofing should engage the service of a professional engineer who has a working knowledge of structures and who has had experience with hydraulic structures or flood-proofing. This is necessary to insure that the flood-proofing does not worsen the problem by creating structural damages such as ruptured walls and floors in addition to the damages resulting from water contact and disruption.

Structural Problems

The forces which would act upon a typical building under conditions varying from normal (nonflood) to partial submergence (flood with and without subsurface or foundation drainage) are graphically presented in Figures 10-1, 10-2, and 10-3. The building cross-section shown is considered representative of that which would be commonly encountered in a flood-proofing program.

Loading from Structure and Contents

The weight of the building itself (masonry, concrete, steel, wood, etc.), known as dead load, together with the weight of its live load (furniture, machinery, merchandise, occupants, etc.) will normally be transmitted through the roof and floor systems to supporting columns and walls and thence to the foundations. These loads generally are transmitted directly to the supporting soil or bedrock under the foundation. Loads of this type will normally be unaffected by flooding and will have the same value for both flood and nonflood conditions.

Restraint from Floor and Roof Systems

Flooding produces large lateral forces on the structure. These forces will be resisted by the building walls, floor, and roof systems. Many commercial and industrial buildings are designed and constructed in a manner to provide adequate connection and anchorage between these systems for support and structural unity, but each building must be individually evaluated and strengthened where necessary.

Most residential and many light commercial and industrial buildings, however, do not have the necessary anchorage and would require modification to provide it. This would involve adequate transverse bridging in addition to anchorage into the walls around the entire perimeter. Steel angles bolted into both the floor system and the walls at their juncture would be one method of anchorage.

Resultant of Nonflood and Flood Loading

The nonflood loading is the force exerted by the soil backfill upon the wall. These pressures depend upon the physical characteristics of the soil particles, the degree of compaction, the moisture content, and the movement of the wall caused by the backfill and foundation deformation, if any. Where the top of the zone of saturation (water table) is at an elevation above the base of the foundation, the pressure on the wall and floor slab is due to the buoyant weight of the soil plus the full hydrostatic pressure of the water. When a workable subdrainage system is provided to lower the elevation of the water table, the pressure on the wall will be reduced. The degree to which the water table can be lowered will depend upon the permeability of the soil and the efficiency of the subdrainage system.

Flood loading without subdrainage is the force of the full hydrostatic pressure of the water above as well as below the ground line plus the buoyant weight of the soil. As schematically illustrated in Figure 10-2, the magnitude of this force can be considerably larger than the force developed under nonflood conditions (shown in Figure 10-1 for comparison).

When subdrainage is provided, the flood loading is reduced. However, in the case of an existing building with an existing unmodified subdrainage system, prudence would dictate that no load reduction be assumed. Subdrains, where already installed, are generally provided only to intercept seepage and control uplift on basement floors due to groundwater. If such a subdrainage system were to be modified to attain a known degree of effectiveness, a load reduction could be determined. Obviously, for new construction the subdrainage system can be designed and constructed to afford a predetermined degree of reduction of flood loads.

The magnitude of the flood-induced forces that will be encountered is indicated by the fact that a one-story brick building (3-5/8 inches of brick over wood frame) can be expected to withstand no more than two feet of water above the ground line providing the wall is in good condition. For brick with concrete-block backup, this height would be somewhat greater.

Subsurface Drainage

Groundwater conditions may adversely affect the stability of a building or structure either through uplift which tends to "float" the building or by erosion which can undermine the support. Investigation and analysis of the factors involved at any specific building and the design of control or corrective measures are endeavors requiring the attention of a professional engineer.

Groundwater problems can be controlled by the installation of subdrainage systems (Figures 10-1 and 10-3) to reduce the lateral forces on the

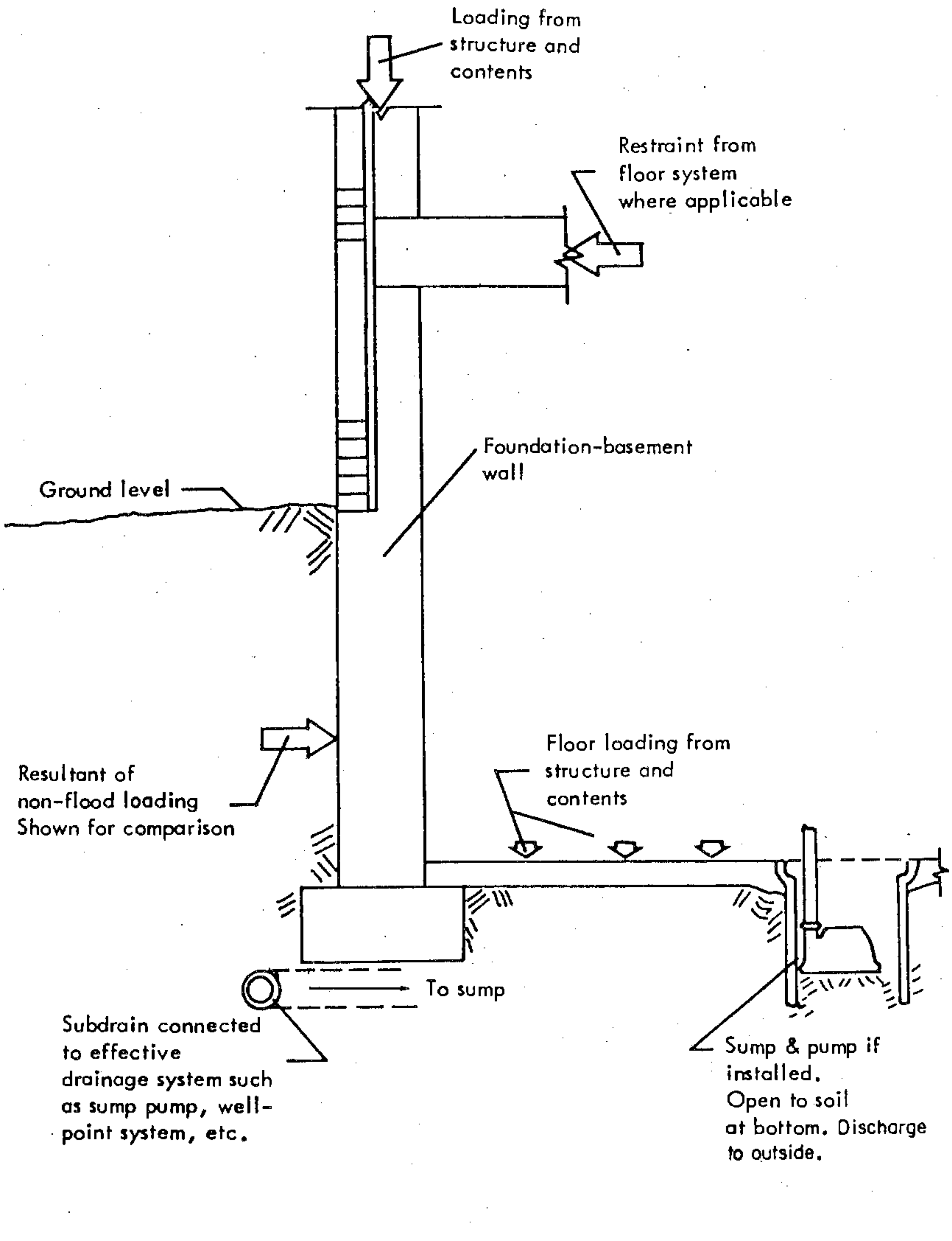


Figure 10-1 Building Loads Under Nonflood Conditions

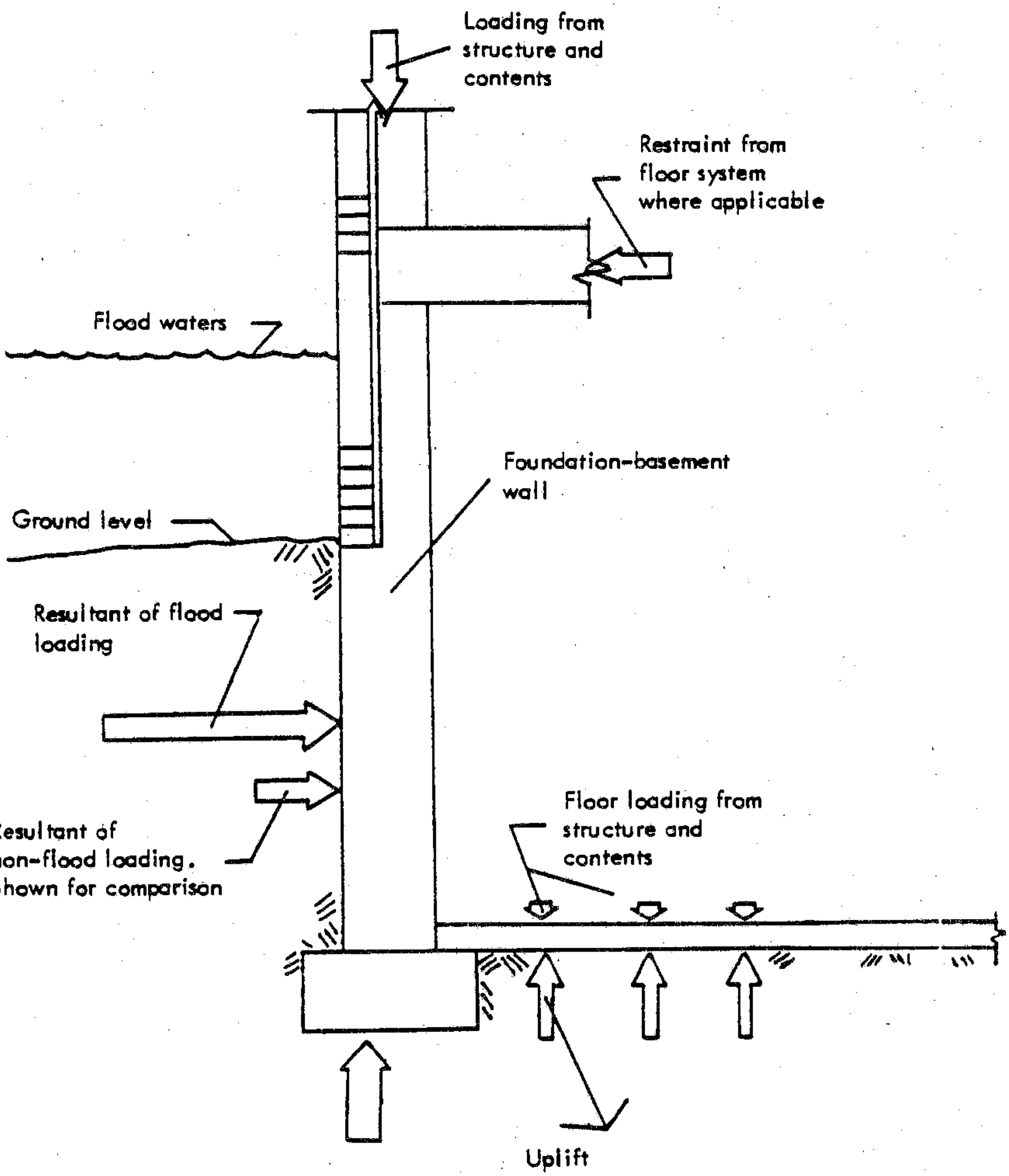


Figure 10-2 Building Loads Without Subsurface Drainage

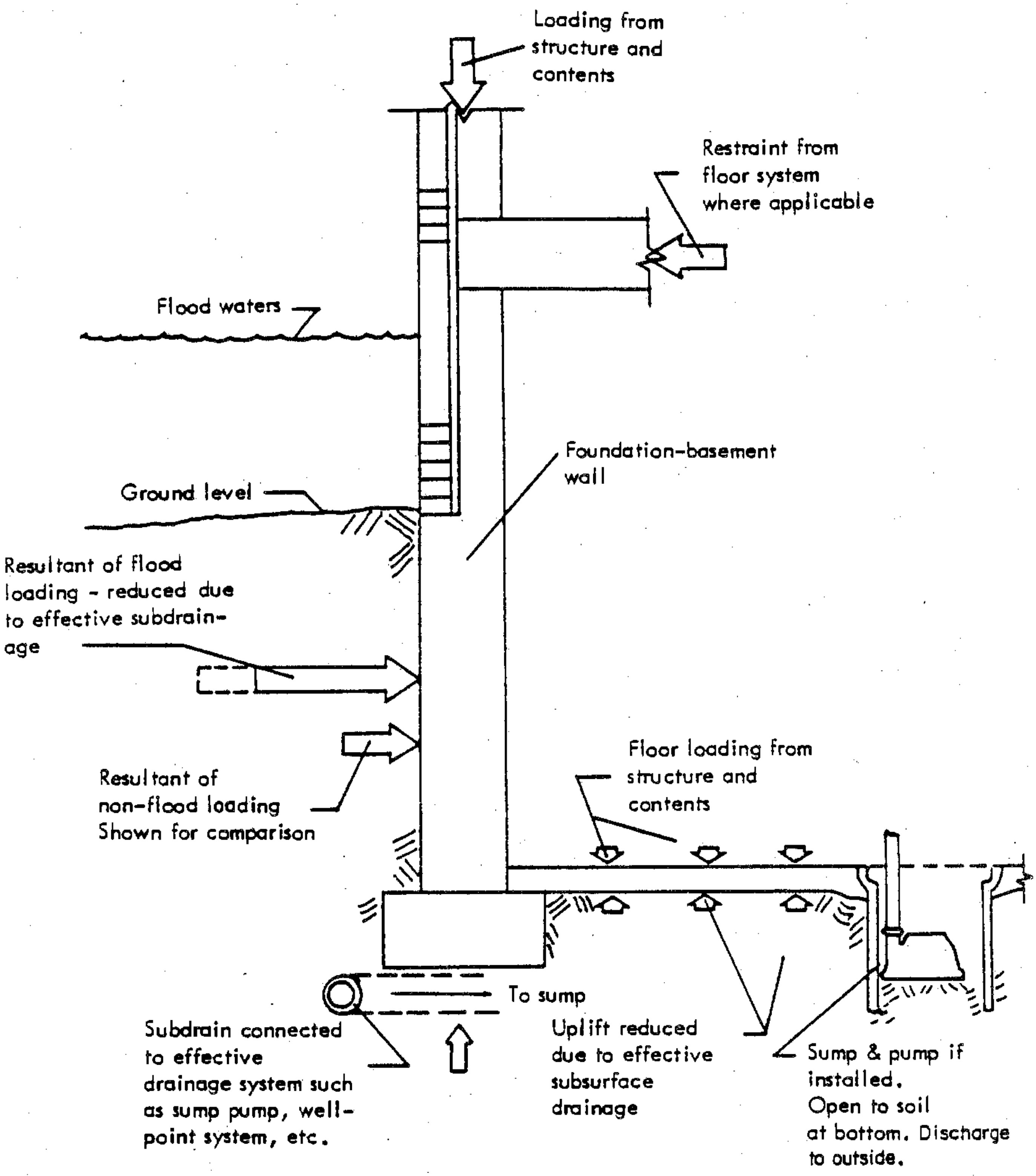


Figure 10-3 Building Loads with Subsurface Drainage

foundation walls and floor slabs. Experience has shown that the composition of soils in a particular area can vary widely with extreme ranges of permeability existing in areas of similar geological origin. Such ranges in permeability call for careful investigation and analysis. The design of a subsurface drainage system must be based on the results of soil investigations of permeability and analyses of structural strength.

A sump and pump system can be employed to help protect the subsurface part of a building. The pump could be designed to accept storm and seepage flows and pump them to a point above the flood waters. The sump should be open to the soil at the bottom and to atmospheric pressure at the top within the basement. This would provide a fail-safe feature, in that power or pump failure would allow water to flood the basement and thereby tend to balance the outside flood-induced pressures upon the basement walls and floor slab. As an alternative, a prearranged program of deliberate flooding with clean water could be employed to minimize the cost of clean up after a flood.

Seepage Control

Foundation walls can be made watertight to minimize water infiltration through cracks and crevices in the walls. In buildings under construction, this can be accomplished through the use of waterproof membranes and seals. Construction joints can be protected by the use of a neoprene or other similar waterstop. Existing masonry or stone foundations are more difficult to waterproof, particularly if the mortar joints have deteriorated with age. Sealing of walls to prevent seepage can be accomplished in many cases by coating them, preferably on the exterior, with hydraulic cement, epoxy paint, or other similar waterproofing materials.

It must be recognized that sealing and waterproofing of walls increases the hydraulic forces acting on the walls unless the drainage through the walls which is afforded by the cracks and crevices prior to sealing is provided by other means. Sometimes the wisest course would be to permit the seepage through the wall and then control it by a floor drain and sump pump. Existing cracks and leaks in walls sometimes can be the most practical form of drainage to relieve pressure. In some cases this drainage can be supplemented by holes drilled through the walls. Structural and hydraulic analyses of alternative designs and associated cost estimates will enable the designer to choose the most suitable means of controlling seepage at a given building.

Sewage Backup

Most existing subdrains, whether connected to sewerage systems or not, are subject to backflow and high pressures during floods. Since these high pressures could burst the usually encountered clay pipe subdrains and endanger basement walls and floors, some device such as a gate valve must be provided for isolating the subdrains around the building from these high pressures.

There are several alternative methods for controlling backflow through sewers. One method would be to install a main valve at a location where the sewer is strong enough to resist the flood-induced pressure and where all possible reverse flows can be stopped. See locations "A" and "B" in

Figure 10-4. This valve should be designed to accommodate grit and other materials which could lodge in it.

If the pipe is of sufficient strength, an alternative would be to install separate valves on all basement fixtures and floor drains (Figure 10-4). These valves could be inflatable rubber plugs or a similar type of mechanically expandable rubber plug. Valves designed for low pressure (20 psi or less) could be installed in drain lines of fixtures which are below design water levels. In either of the above alternatives, it would be necessary to provide adequate sump pumps to handle any leakage.

Figure 10-5 presents another alternative for controlling sewer backup. This alternative provides for conveying all floor drainage, applicance drainage, drain-tile flow, and any seepage that might enter the building to a sump pump. The pump would lift the drainage up to an elevation above the design flood on a permanent basis. By thus eliminating all gravity sewer drains, the problem of flooding backflow can be eliminated and a subsurface area permitted to function during floods.

Structural Engineering

The highly technical and thorough nature of the investigations and analyses required in the design of effective, safe, and reliable flood-proofing measures for both new and existing facilities cannot be overstressed. Construction or modification of subdrainage systems without such investigation and analysis can result in a situation potentially more dangerous to life and property than no flood-proofing program at all.

The large number of factors and the potential magnitude of the forces involved make it impossible to design flood-proofing measures by intuition. Such an approach can lead to loss of property and even life during a flood.

10.50 Flood-proofing Operations

Any individual or organization undertaking a contingent or emergency protection program must have a standard operating procedure to carry out the flood-proofing measures when the need arises. Some buildings can be secured in a short time while others may take considerably longer.

The flood-proofing system should be designed so that it may be put into operation as quickly and as simply as possible. Flood shields, doors and hatches may have to be handled during the most adverse weather conditions (perhaps during the storm which causes the flood), so lightweight metals should be used wherever possible.

Flood-proofing items including bolts, gaskets, caulking, timbers, and flood shields should be stored for easy access. The larger items should be stored close to the point of insertion and in such a manner that they can be easily slid or dropped into position. One lost or improperly mounted flood shield, or the failure to isolate a sewer can defeat the best flood-proofing system.

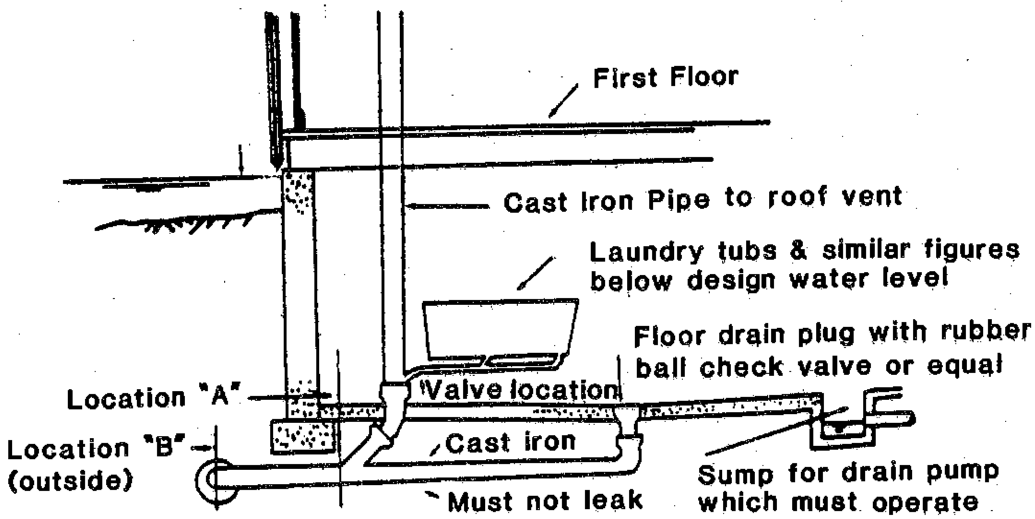


Figure 10-4 Locations for Cutoff Valves on Sewer Lines

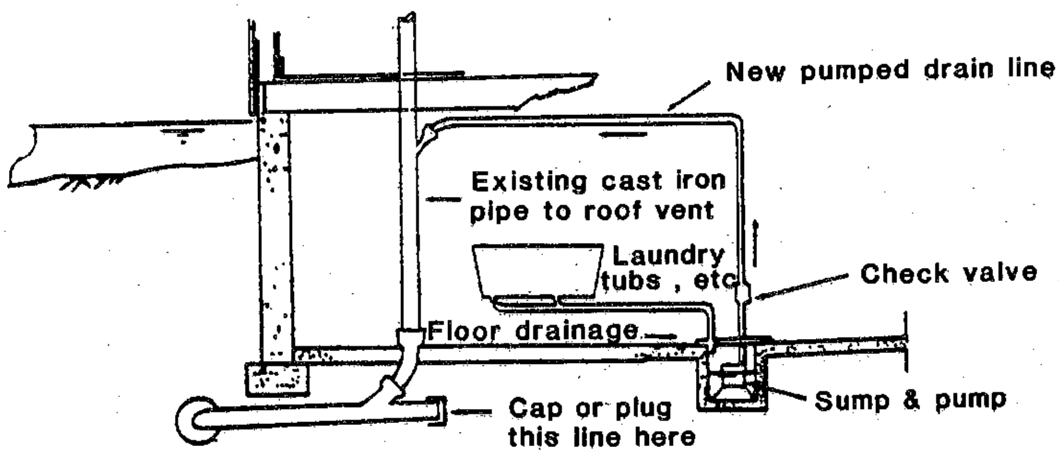


Figure 10-5 Elimination of Gravity- Flow Basement Drains

Basement Rooms

Newly designed buildings may have machinery located on upper floors but most older structures have the electrical machinery and the heating and pumping equipment on lower floors. It is these lower, subsurface floors which first experience flooding problems from seepage and sewer backup. Flood-proofing systems requiring pumps, electrical equipment, and emergency generators as an essential part of the operation must be kept in working condition throughout the crisis.

When buildings have entrances to subsurface levels from ground level, these entrances should be adjusted to prevent the entry of overland flow.

Utilities

When flood-proofing a building, provision should be made to eliminate the threat of flooding by way of gas mains, sewers, conveyor systems, and water pipes or drain tiles which enter the building. Check valves can be installed in utility pipes to protect against this source of flooding.

If sewers are to be isolated, they should be constructed of pressure pipe. Otherwise the pressure in the line could cause a rupture.

Fuses and circuit breakers servicing flooded areas should be clearly marked and easily accessible. Electrical circuits serving lower levels should be designed or modified so that they can be cut off if flooding begins. This will protect against fires and loss of life due to electrical shocks. As another precaution, valuable electrical appliances which cannot be moved should be disconnected at the unit to prevent short circuiting and damage to their power components.

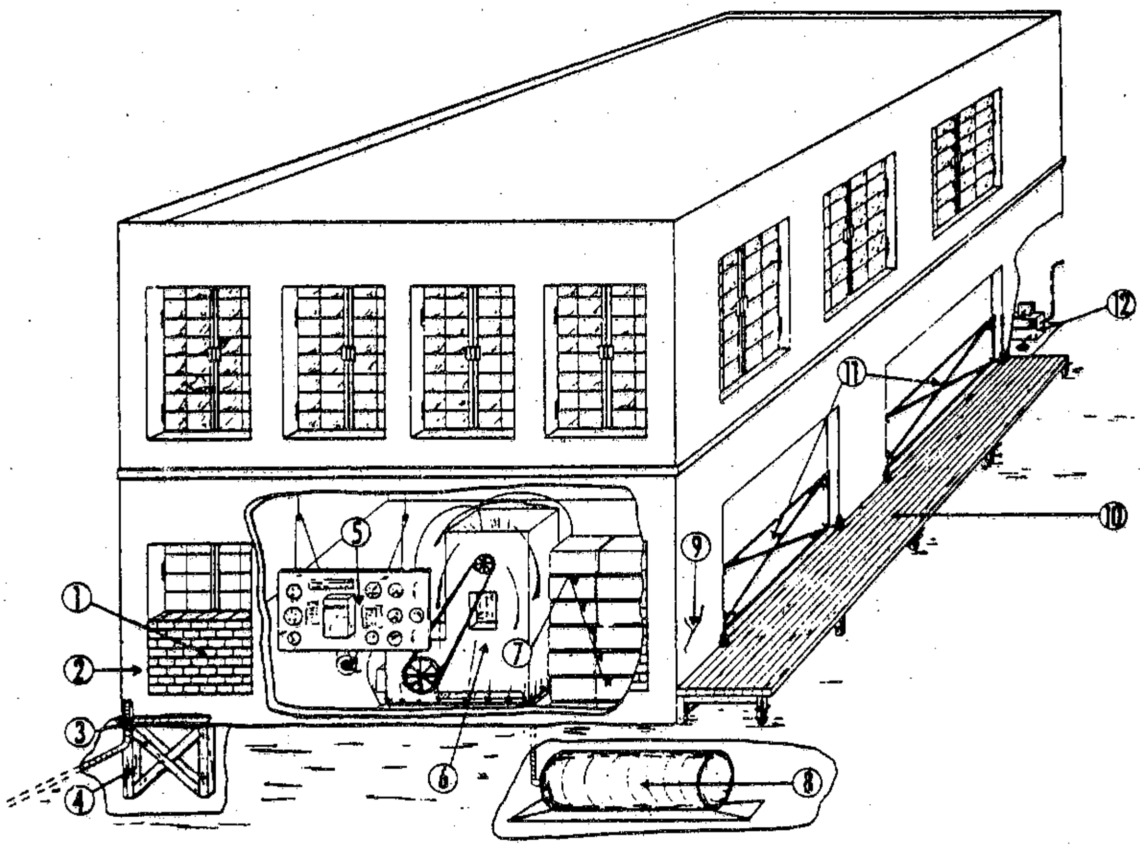
Wall Openings

Windows and vents both above and below the surface should be sealed to prevent the entry of flood waters. They may also need to be reinforced. The walls should be strong enough to support the pressures added by vent and window reinforcements. The wall should also be treated or constructed to prevent large amounts of water from passing through it.

Residential Homes

Residential construction does not lend itself readily to flood-proofing because of the extensive use of materials that do not impede the passage of water. Moreover, houses are seldom designed to withstand any significant horizontal pressures.

In most cases in which an owner has purchased a finished house in an area subject to flooding, his success with flood-proofing will depend on whether flood stages are low on his property and whether the outer walls of the structure are reasonably impervious. Under these conditions flood shields can be designed to restrict the entry of water through openings in the walls, providing the walls are strong enough to resist flood-induced pressures. An effective flood-proofing program must also include measures to cope with sewer backup and groundwater seepage.



EXPLANATION

1. Permanent closure of opening with masonry
2. Thoroseal coating to reduce seepage
3. Valve on sewer line
4. Underpinning
5. Instrument panel raised above expected flood level
6. Machinery protected with polyethylene covering
7. Strips of polyethylene between layers of cartons
8. Underground storage tank properly anchored
9. Cracks sealed with hydraulic cement
10. Rescheduling has emptied the loading dock
11. Steel bulkheads for doorways
12. Sump pump and drain to eject seepage

Figure 10-6 Flood-Proofed Building

Homeowners who have suffered severe basement flooding should consider the relocation of furnaces, hot water heaters, washers, dryers, air conditioners, freezers, refrigerators, power shop equipment, and other appliances as a permanent flood-proofing measure.

The flood-proofing of a structure is analogous in many respects to making a ship watertight and seaworthy. Flood-proofing involves not only adjustments to the foundation and substructure but also modifications of those parts of the superstructure that are below anticipated flood levels (Figure 10-6).

10.60 Bibliography

1. NAHB Research Foundation, Inc., Manual for the Construction of Residential Basements in Non-Coastal Flood Environs, Department of Housing and Urban Development Federal Insurance Administration, Washington, DC, 1977.

2. Office of the Chief of Engineers, U.S. Army, Flood-Proofing Regulations, Washington, DC, 1972.

3. Wright-McLaughlin Engineers, Urban Storm Drainage Criteria Manual, Denver Regional Council of Governments, CO, 1969.

Section 11

Erosion and Sediment Control

Erosion and sediment control are discussed in a manual published under separate cover. Please refer to the current version of the City of Casper Erosion and Sedimentation Control Manual.

Section 12

Design Examples

Section 12

Design Examples

The following examples represent a simplified version of a typical Subdivision Drainage Design. The examples are intended to demonstrate the specific design method presented in Sections 2 through 11. These examples also show the interrelationship between the various methods in design applications. For a detailed discussion of these methods, consult the appropriate sections of this manual and their respective bibliographies.

12.10 General

A developer owns 300 acres which he intends to develop in segments over the next 10 years. The first area to be developed will be a 30 acre tract containing 20 acres of single family residential lots of approximately 11,250 square feet and a 10 acre shopping center.

Due to limited resources, the developer intends to use stormwater management techniques to limit runoff and minimize his overall expenditures for drainage facilities.

Figure 12-1 shows the drainage basin, the proposed development, and drainage facilities. The drainage plan utilizes a detention pond to reduce the peak runoff from the majority of the basin, thus reducing the size of required trunk sewer through the proposed development. The proposed shopping center will utilize parking lot detention and roof storage to reduce peak runoff.

Figure 12-2 shows the proposed residential development layout, with appropriate dimensions and design street grades.

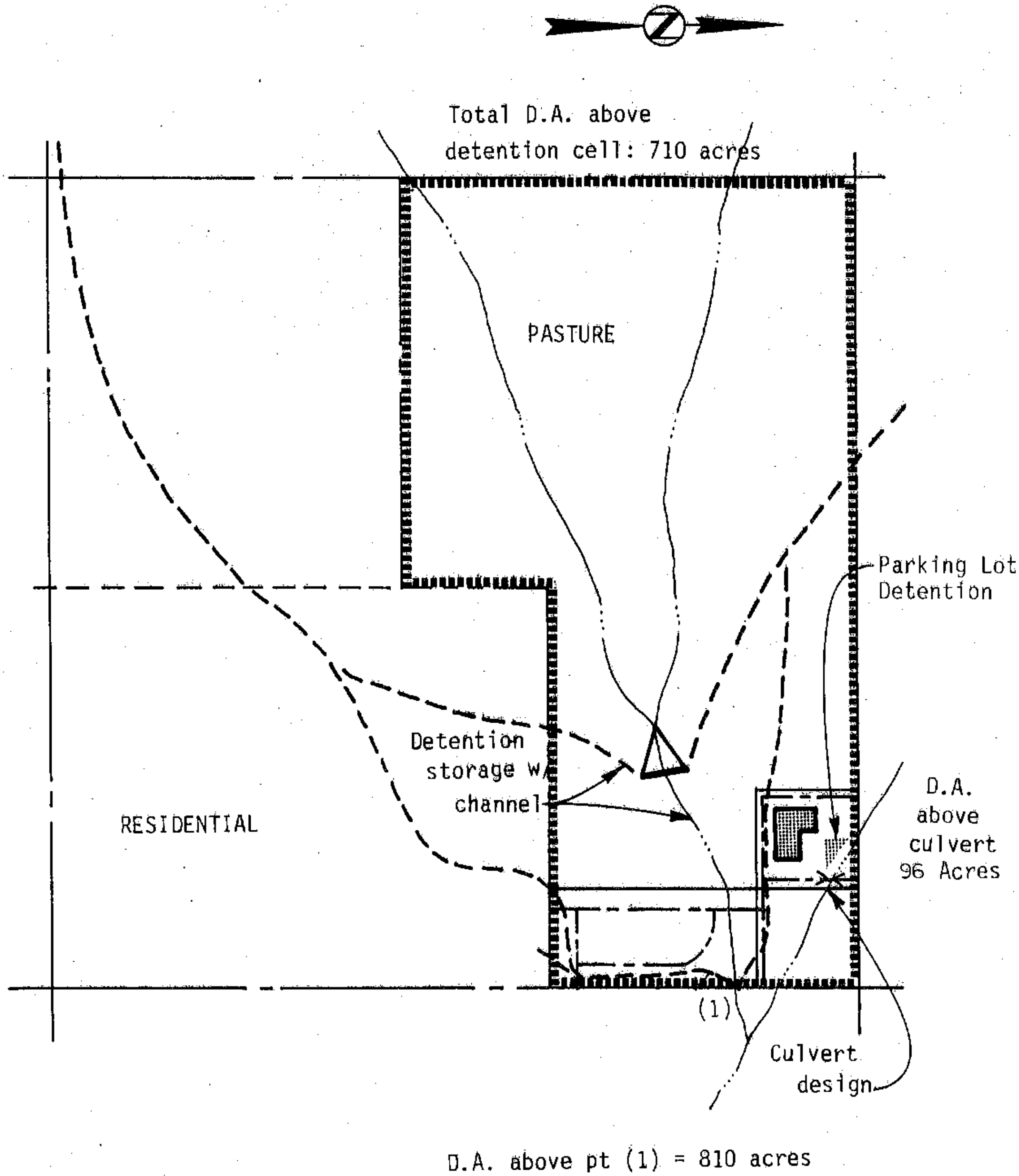


Figure 12-1 Overall Site Plan

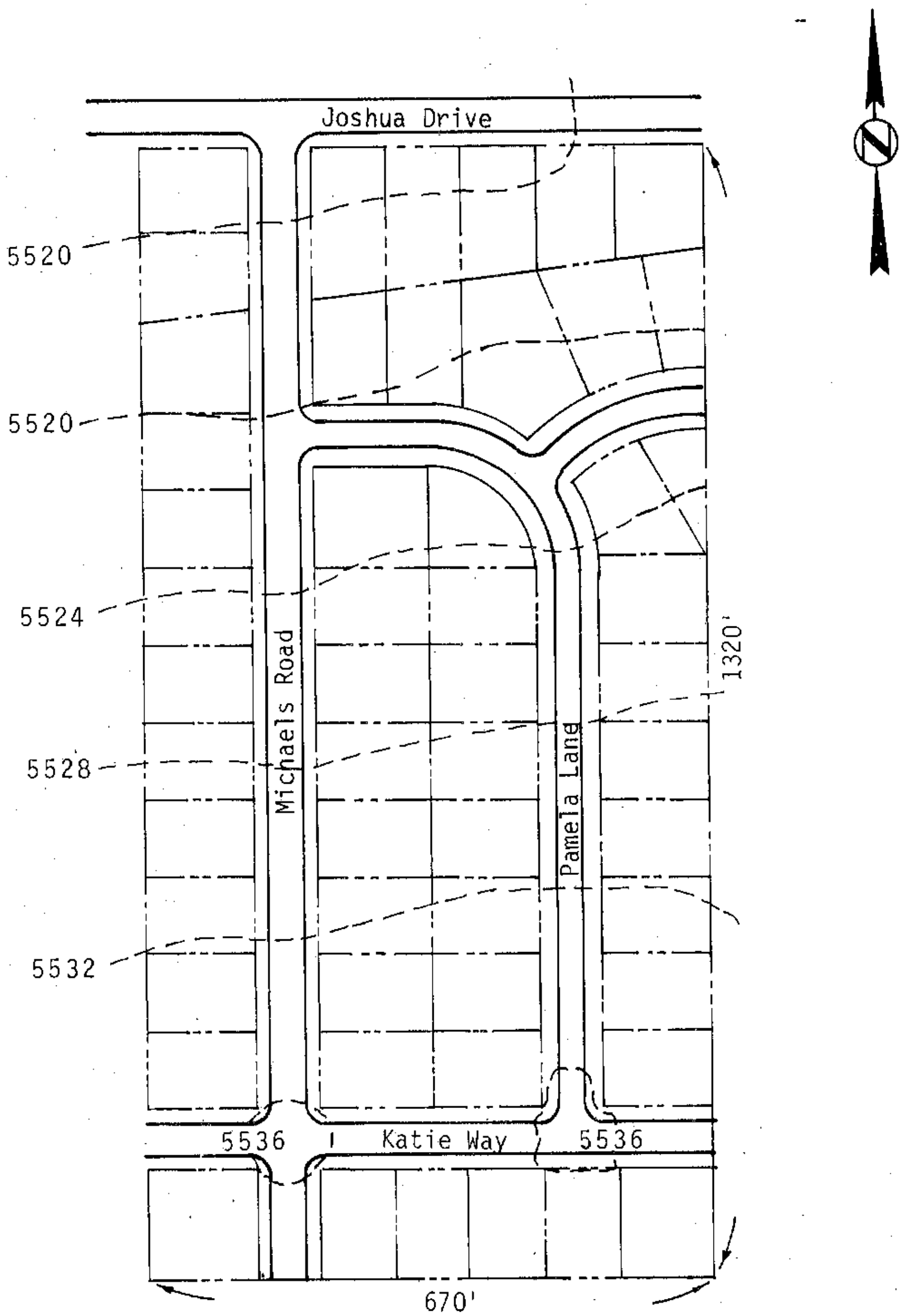


Figure 12-2 Sub-division Layout

	PROJECTS
	SUBJECT
	COMPUTED CHECKED DATE PAGE 12-5 OF

12.20 Street drainage

Michaels Road is a 36' wide collector street on a 2% grade, and drains a uniform width of 530' along its length. Katie Way is also a 36' wide collector, but is graded at 0.8% and -0.8%, creating a sump curve in the middle of the block. It drains a total area of 1.82 acres, with 80% contributing to the east inlet. Pamela Lane is a 25' wide local street, and drains a width of 280' through its length

Using the criteria of Sections 3 and 5, design the surface drainage system for these streets.

Allowable Gutter Flow

Initial Storm

Michaels Road

Given: 6" vertical curb
2% pav't cross slope
36' street width
2% grade

Find: Allowable gutter flow for initial storm of 10 year frequency

1. Determine allowable pav't encroachment
Use Table 3-1: one 12' lane must remain free
36' - 12' = 24'
∴ encroachment = 12' per side

PROJECTS

SUBJECT

COMPUTED CHECKED DATE PAGE 12 OF

2. Calculate theoretical capacity for each gutter.
   Use Figure 3-2.
   $n = 0.016$
   $z = 1/S_x = 1/.02 = 50$
   $\therefore z/n = 3125$
   $y = 12 \times .02 = 0.24$

   $\therefore q = 6.0$ cfs

3. Calculate allowable gutter capacity
   Use Figure 3-3 w/ street slope of 2%
   $q = (0.8)(6.0) = 4.8$ cfs

4. Therefore, allowable gutter capacity for Michaels Road.
   4.8 cfs

5. Using same procedures, allowable gutter capacity for Katie Way:

   2.1 cfs

Pamela Lane
Given: 6" vertical curb
2% pav't cross slope
25' wide street
2% grade

Find: Allowable gutter flow for initial storm

	PROJECTS ______
	SUBJECT ______
	COMPUTED ______ CHECKED ______ DATE ______ PAGE ______ OF ______

1. Determine pav't encroachment
   Use Table 3-1: Flow may not overtop curb
   (25-1) ÷ 2 = 12' ± encroachment

2. Calculate theoretical capacity
   Use Figure 3-2
   $n = 0.016$
   $z = 50$
   $z/n = 3125$
   $y = .50$
   $y' = .50 - (12 \times 0.02) = .26$
   $\therefore q = 40\ cfs - 6\ cfs = 34\ cfs$

3. Calculate allowable gutter capacity
   Use Figure 3-3

   $q = 0.8\,(34) = 27.2\ cfs$

4. Allowable gutter flow for Pamela Lane
   = 27.2 cfs

PROJECTS ______

SUBJECT ______

COMPUTED ______ CHECKED ______ DATE ______ PAGE 12-8 ______ OF ______

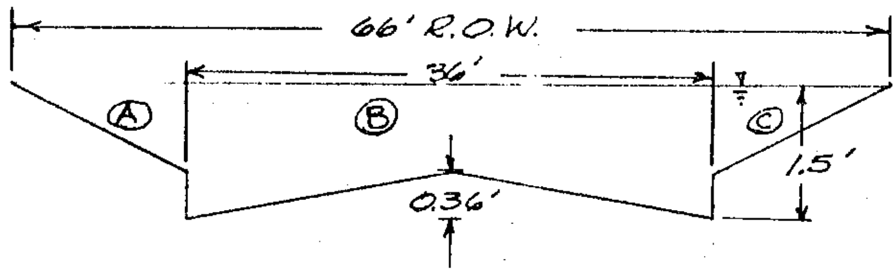
Major Storm

Given: 8 Acre Residential Development
Property lines of 66' R.O.W are one foot higher than curb, minimum, along all streets.

Find: Encroachment due to major storm

1. Allowable depth of flow
Use Table 3-2: 18" above gutter flowline for local and collector.

2. Street capacity: collectors
Use Manning's



3 Assume areas A and C (parking) are non-effective for flow, due to obstructions.

4. Major storm discharge

$$Q = \frac{1.49}{n} A R^{2/3} S^{1/2}$$ eq. (7-1)

$$A = (36' \times 1.5') - (36' \times \tfrac{0.36}{2}) = 47.52 \text{ sf}$$

$$R = \frac{A}{WP} = \frac{47.52}{[2(1.5)+36]} = 1.22$$

PROJECTS

SUBJECT

COMPUTED CHECKED DATE PAGE 11-9 OF

$\therefore Q = \frac{1.49}{0.016} (47.52)(1.22)^{2/3}(0.02)^{1/2}$

$Q = 714$ cfs

5. Area needed to exceed major storm discharge
   100 yr, 5 min. storm
   $i = 6.36$ inches/hr. (Figure 2-1)
   $C = .52$
   $A = \frac{Q}{CI} = \frac{714}{(.52)(6.36)(1.25)}$

   $A = 173$ acres

   but, D.A. above streets < 173 acres;
   therefore street has sufficient capacity

6. Local street

   Use Figure 3-2 for local streets
   grade = 2% Sx = 2% z/n = 50/.016
   Try *y = 0.5'
   Q = 34 cfs

   * y may go to 1.5', if no buildings are endangered
   See Table 3-2

7. Drainage area needed

   $A = \frac{.34}{(.52)(6.36)(1.25)} = 8.55$ acres

   But DA is < 8.55 acres
   therefore street has sufficent capacity

PROJECTS

SUBJECT

COMPUTED ______ CHECKED ______ DATE ______ PAGE 12-10 OF ______

12.30 Inlet design

Using the gutter capacities for Michaels Road, Katie Way, and Pamela Lane developed previously, determine inlet locations for continuous grade and sag conditions.

Use the criteria presented in Sections 2 and 4.

Inlets on a Continuous Grade

Curb-Opening Inlet

Given: Michaels Road gutter flow 4.8 cfs / gutter

$n = 0.016$

$S_o = 0.02 \ (2\%)$

$S = 0.02 \ (2\%)$

$T = 12 \text{ ft}$

$W = 2 \text{ ft}$

Find: Inlet length for curb-opening inlet for various interception rates. From Equations Listed in table 4-1

1. $F_W = 16.4\left[(T-2)S_x\right]^{1/6} S_o^{\ 1/2}$

$= 16.4\left[(10)(.02)\right]^{1/6}(.02)^{1/2}$

$= 1.77$

2. $F_W T = 21.28$

3. $\therefore L_1 = 16.38$

$L_2 = 9.83$

$L_3 = 35.11$

PROJECTS

SUBJECT

COMPUTED ____ CHECKED ____ DATE ____ PAGE ____ OF ____ 12-11

4. $Q_2 = \frac{L_2}{L_1} Q_1 = 2.88$

5. Try $Q_i/Q = 1.00,\ 0.8,\ 0.6$

$$\frac{Q_i}{Q} = \left(\frac{L_i}{L_3}\right)^{.4} \quad \text{for } Q_i > Q_2,\ L_i > L_2$$

rewritten as

$$\frac{L_i}{L_3} = \left(\frac{Q_i}{Q}\right)^{2.5}$$

$\frac{Q_i}{Q}$	$\frac{L_i}{L_3}$	L_i
1.0	1.0	35.11
0.8	0.572	20.1
0.6	0.279	9.79 $< L_2$
0.65	0.342	12.0
0.71	0.427	15.0

A 9.83' Inlet would provide most efficient inlet. However use standard 12' inlet for economy

6. For $L_i = 12'$ $\quad \frac{Q_i}{Q} = .65$

$\therefore Q_i = .65(4.8) \quad \underline{3.12 \text{ cfs}}$

	PROJECTS ___
	SUBJECT ___
	COMPUTED ___ CHECKED ___ DATE ___ PAGE ___ OF ___

Grate Inlet

Check grate inlet for same location.
Use 24" wide by 48'
long, 60% clear opening.
3" bar depth

a. Flow depth @ curb = 0.24'

b. Flow depth @ outside edge of grate:
$0.24 - 0.02(2.0) = 0.20$

c. Gutter q @ outside edge:
$z/n = 3125$

$q_{theory} = 3.1$ cfs

d. Account for reduction factor

$3.1 \times 0.8 = 2.48$ cfs

e. ∴ Discharge captured by grate

$4.8 - 2.48 = \underline{2.32 \text{ cfs}}$

f. Grate efficiency

$\frac{2.48}{4.8} = 0.48$ or 48%

g. Since curb-opening inlet captures 65% of flow, compared to 48% of flow by grate, the curb-opening is more efficient. Assuming costs are equal, use curb-opening.

PROJECTS
SUBJECT
COMPUTED CHECKED DATE PAGE 12-13 OF

Inlet Spacing

Check Inlet spacing for Curb-Opening Inlet Design using Rational Formula.

Assume $t_c = 5$ min. $i = 4.02$ iph

First Inlet

$A = \frac{Q}{Ci}$ Q is max. allowable gutter flow

$A = \frac{4.8 \times 2}{0.52 \times 4.02} = 4.6$ acres or 200,000 sf

Spacing $= \frac{\text{Area}}{\text{Width}} = \frac{200,000}{330} = 606$ ft. Use 600

Second and following inlet spacing

$A = \frac{Q_i}{Ci}$ Q_i is max. amount to pick-up between inlets

$A = \frac{3.12 \times 2}{.52 \times 4.02} = 3.0$ acres or 131,000 sf

Spacing $= \frac{\text{Area}}{\text{Width}} = \frac{131,000}{330} = 397$ ft.

Use 400'

PROJECTS

SUBJECT

COMPUTED CHECKED DATE

Inlet in a Sag Condition

Initial Storm Grate Inlet

Katie Way

Given: Allowable curb depth for initial storm 0.31'

Total drainage area = 1.82 acres, 70% to first inlet

Find: Inlet type and size

1. Total runoff

$Q = CiA$

$= 0.52 \times 4.02 \times 1.82$

$Q = 3.8$ cfs or 2.7 cfs for first inlet

2. Capacity of (24" x 48") grate inlet,

$Q_i = 3.0 \times P \times d^{1.5}$

$= 3.0 \times 8 \times 0.31^{1.5}$

$Q_i = 4.1$ cfs (OK)

PROJECTS
SUBJECT
COMPUTED CHECKED DATE PAGE 12-15 OF

Initial Storm Curb-Opening Inlet.

1. Capacity of curb-opening inlet, $W = 2'$, $a = 2''$

Try standard 4' Inlet

$Q = 3.0\, L_i\, d_i^{1.5}$ $\quad d_i = 0.48'$ $(0.31' + 2'')$

$= 3.0(4)(0.48)^{1.5}$

$= 4.0$ cfs (OK)

Major Storm

1. Provide for the passage of the 100yr. flood (Q100) from Sag Area

$Q_{100} = (C)(i)(A)(cf)$

$Q_{100} = (0.52)(6.36)(1.82)(1.25) = 7.5$ cfs eq (2-1)

2. Subtract storm sewer design capacity

7.5 cfs − 3.8 cfs = 3.7 cfs must go overland

3. Try Property Line swale with 2' bottom and 3 to 1 side slope and 1 foot depth.
for ; $d = .5'$, $s = 0.01'/ft.$ $n = 0.027$
$A = 1.75$ sf $r = 0.34$

$Q = A \frac{1.49}{n} r^{2/3} s^{1/2} = 1.75 \frac{1.49}{0.027} \times .34^{.67} \times 0.01^{.5}$
$= 4.7$ cfs (channel OK) eq (7-1)

PROJECTS

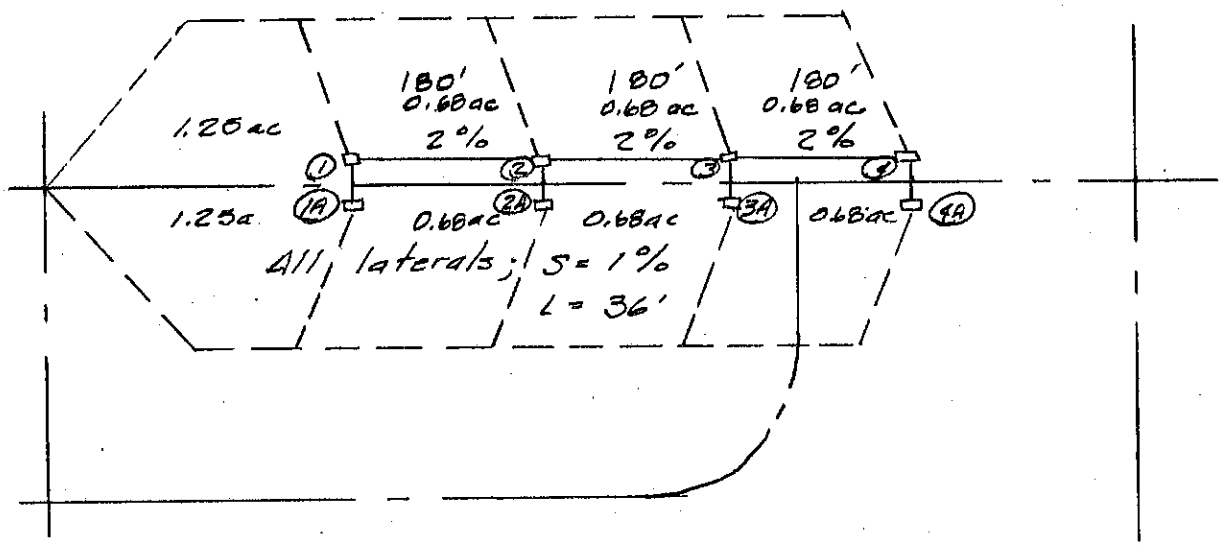
SUBJECT

COMPUTED ______ CHECKED ______ DATE ______ PAGE 12-16 OF ______

12.40 Storm sewer design

Given: Inlet locations as previously determined
Runoff coefficient 0.52; $n = 0.013$
Drainage areas as shown below
12" Minimum pipe size
10-year design frequency

Find: Storm sewer sizes for Michaels Road



$Q = ACi$

A – acres t – min
i – inches/hr. V – ft/sec
Q – cfs · Ca

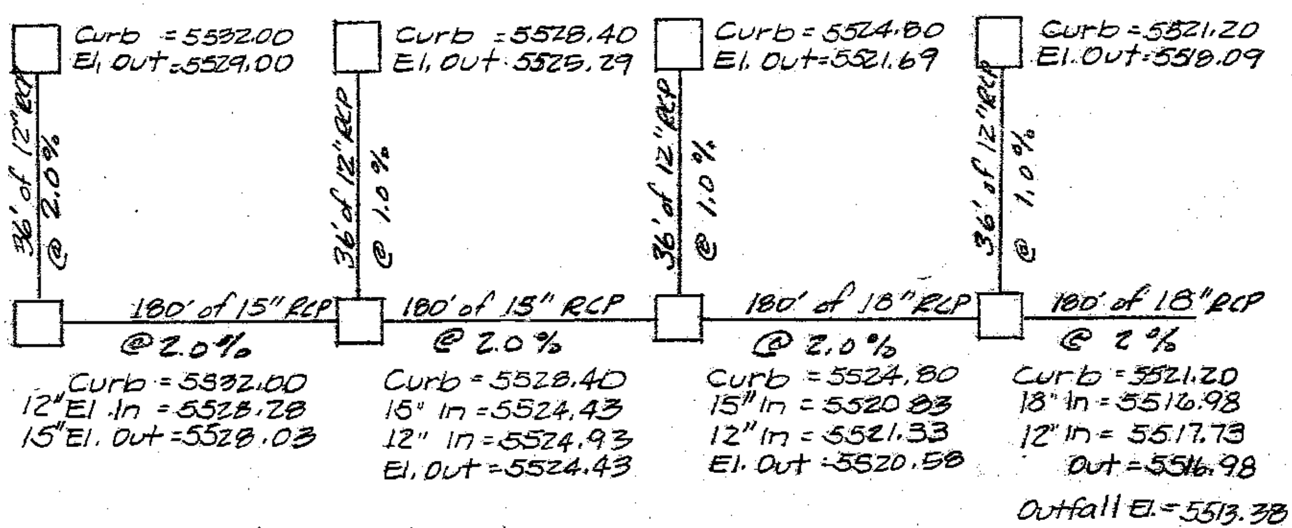
COMPUTATION FORM

DRAINAGE & STORM SEWER DESIGN
RATIONAL METHOD

Project *Michaels Road*
Comp. by ______ date ______
Ck by ______ date ______

LOCATION			A		C	TIME OF CONCENTRATION		i	Q	DESIGN					PROFILE				
STREET	FROM	TO	INCREMENT	TOTAL		TO INLET	PIPE OR CHANNEL			SIZE	SLOPE %	n	CAP. FULL	V	LENGTH FT.	FALL FT.	OTHER LOSSES	UPPER INV. EL.	LOWER INV. EL.
Mains	1A	1	1.25	1.25	.52	5	0	4.0	2.6	12"	2	.013	5.0	6.5	36	.72	—	5529.00	5528.28
"	1	2	1.25	2.5	.52	5	.1	4.0	5.2	15"	2	.013	9.0	7.5	180	3.6	.25	5528.03	5524.43
"	2	3	1.36	3.86	.52	5	.5	3.9	7.9	15"	2	.013	9.0	7.5	180	3.6	—	5524.43	5520.83
"	3	4	1.36	5.22	.52	5	.9	3.9	10.6	18"	2	.013	14.9	8.4	180	3.6	.25	5520.58	5516.98
"	4	5	1.36	6.58	.52	5	1.3	3.8	13.0	18"	2	.013	14.9	8.4	180	3.6	—	5516.98	5513.38
Laterals	2A	2	.68		.52	5	—	4.0	1.4	12"	1	.013	5.0	6.5	36	.36	—	5525.29	5524.93
"	3A	3	.68		.52	5	—	4.0	1.4	12"	1	.013	5.0	6.5	36	.36	—	5521.69	5521.33
"	4A	4	.68		.52	5	—	4.0	1.4	12"	1	.013			36	.36	—	5518.09	5517.73

Final design for sewerage system on Michaels Road



	PROJECTS
	SUBJECT
	COMPUTED CHECKED DATE PAGE 12-18 OF

12.50 Detention Basin Design by the Colorado Urban Hydrograph Procedure

In order to limit the amount of runoff flowing through the planned development, thus minimizing the size of the trunk sewer, the developer has decided to build a small detention pond above the subdivision.

The developer owns approximately 200 acres of the 710 acre drainage area controlled by the detention pond. All of the area is currently devoted to grazing practices. Within the next 10 years, the developer will be constructing additional sub-divisions, but will attempt to maintain the existing drainage courses in their natural conditions.

Minor channel realignment and cleaning, and planting of moderate channel vegetation are anticipated.

However, 510 acres of the drainage area are not owned by the developer. Therefore, to anticipate the development within these areas, current and future zoning maps are consulted. From these maps, it is learned that 510 acres are zoned for future residential areas similar to those being developed. Thus the entire area is considered for future residential development conditions.

Project ______________________ Computed ______

Subject ______________________ Date ______ Sht. 12-19 Of ____

Given: Drainage Area = 710 acres ($1.11\ mi^2$)
Length = 1.94 miles
Length to Centroid = 0.99 miles
Slope = 0.045 ft/ft

Land use within planned development of 200 acres:

a. Residential 55% (12,000 S.F. avg. size)
b. Commercial 5%
c. Streets 15%
d. Open space 25%

Find: Development imperviousness
Natural and ultimate 10-year peak discharges
Ultimate 100-year peak discharge
Unit and storm hydrographs
Required storage

Imperviousness, I

1. Use Table 2-4

Res.	55% × 30%	=	16.5%	
Com.	5% × 85%	=	4.3%	
St.	15% × 100%	=	15.0%	
Open	25% × 5%	=	1.3%	
			37.1%	= I

Ultimate 10-year and 100-year peak discharges, unit hydrograph, and storm hydrographs

1. $L = 1.94$ miles
$L_c \doteq 0.99$ miles
$S = 0.045$ ft/ft
$A = 1.11\ mi^2$
$I = 37\%$

2. From Fig. 2-4, $C_{t_0} = 0.47$ for $I = 37\%$.

Project ______ Computed ______

Subject ______ Date ______ Sht. 12-20 Of ______

3. Adjust C_{t_o} for $S = 0.045$ ft/ft

$C_t = 0.48(0.47)(0.045)^{-0.2}$

$C_t = 0.42$

4. Find t_p. $t_p = 0.42[(1.94)(0.99)]^{0.3}$

$t_p = 0.511$ hrs

5. Find C_p. $C_p = 0.89(0.42)^{0.46}$

$C_p = 0.60$

6. Find q_p.

$q_p = 640(0.60)(1.11)/0.511$

$q_p = 834$ cfs

7. Find T_p. $T_p = 0.511 + 0.042$

$T_p = 0.553$ hrs

8. Find z. $z = 0.992(0.60)(0.553)/0.511$

$z = 0.644$

9. From Fig 2-6, $\omega = 2.77$ for $z = 0.644$

Use $\omega = 3$.

10. Find unit hydrograph.

$q = q_p (T/T_p)^{\omega} e^{(1 - T/T_p)\omega}$

Project ______ Computed ______

Subject ______ Date ______ Sht. 12-21 Of ______

T(min)	T/T_p	q/q_p	q (cfs)
0	0.00	0.00	0
5	0.15	0.04	36
10	0.30	0.22	184
15	0.45	0.47	396
20	0.60	0.72	598
25	0.75	0.89	745
30	0.90	0.98	821
35	1.05	1.00	830
40	1.21	0.94	787
45	1.36	0.85	712
50	1.51	0.75	622
55	1.66	0.63	527
60	1.81	0.52	435
65	1.96	0.42	353
70	2.11	0.34	280
75	2.26	0.26	220
80	2.41	0.20	170
85	2.56	0.16	130
90	2.71	0.12	98
95	2.86	0.09	74
100	3.01	0.07	55
105	3.16	0.05	40
110	3.32	0.03	29
115	3.47	0.03	21
120	3.62	0.02	15
125	3.77	0.01	11
130	3.92	0.01	8
135	4.07	0.01	6
140	4.22	0.00	4
145	4.37	0.00	0

11. Find ultimate 10-year and 100-year rainfall excess and storm hydrographs.

Table 2-9 Determination of Rainfall Excess
Ultimate 10-year Storm

		Pervious Area (63%)				Impervious Area (37%)				
Time (min)	Rainfall Increment (in.)	Infiltration (in.)	Depression Storage (in.)	Rainfall Excess (in.)	63% Rainfall Excess (in)	Depression Storage (in.)	Other Losses (in.)	Rainfall Excess (in.)	37% Rainfall Excess (in.)	Composite Rainfall Excess (in.)
(1) Total	(2) (1.28)	(3) (0.57)	(4) (0.30)	(5) (0.41)	(6) (0.26)	(7) (0.10)	(8) (0.05)	(9) (1.13)	(10) (0.40)	(11) (0.66)
5	.01	.01				.01				
10	.01	.01				.01				
15	.01	.01				.01				
20	.01	.01				.01				
25	.01	.01				.01				
30	.02	.02				.02				
35	.03	.03				.03				
40	.04	.04						.04	.01	.01
45	.06	.04	.02					.06	.02	.02
50	.08	.04	.04					.08	.03	.03
55	.14	.04	.10				.01	.13	.05	.05
60	.34	.04	.14	.16	.10		.02	.32	.12	.22
65	.18	.04		.14	.09		.01	.17	.06	.15
70	.11	.04		.07	.04		.01	.10	.04	.08
75	.07	.04		.03	.02			.07	.03	.05
80	.05	.04		.01	.01			.05	.02	.03
85	.03	.03						.03	.01	.01
90	.02	.02						.02	.01	.01
95	.01	.01						.01		
100	.01	.01						.01		
105	.01	.01						.01		
110	.01	.01						.01		
115	.01	.01						.01		
120	.01	.01						.01		

Table 2-9 Determination of Rainfall Excess
Ultimate 100-year Storm

Time (min)	Rainfall Increment (in.)	Pervious Area (63%) Infiltration (in.)	Pervious Area (63%) Depression Storage (in.)	Pervious Area (63%) Rainfall Excess (in.)	Pervious Area (63%) 63 % Rainfall Excess (in)	Impervious Area (37%) Depression Storage (in.)	Impervious Area (37%) Other Losses (in.)	Impervious Area (37%) Rainfall Excess (in.)	Impervious Area (37%) 37% Rainfall Excess (in.)	Composite Rainfall Excess (in.)
(1) Total	(2) (1.98)	(3) (0.62)	(4) (0.30)	(5) (1.06)	(6) (0.67)	(7) (0.10)	(8) (0.10)	(9) (1.78)	(10) (0.64)	(11) (1.31)
5	.01	.01				.01				
10	.01	.01				.01				
15	.01	.01				.01				
20	.01	.01				.01				
25	.01	.01				.01				
30	.02	.02				.02				
35	.04	.04				.03		.01		
40	.07	.04	[illegible]					.07	.03	.03
45	.10	.04	.06				.01	.09	.03	.03
50	.13	.04	.09				.01	.12	.04	.04
55	.22	.04	.12	.06	.04		.01	.21	.08	.12
60	.53	.04		.49	.31		.03	.50	.19	.50
65	.30	.04		.26	.16		.02	.28	.10	.26
70	.16	.04		.12	.08		.01	.15	.06	.14
75	.11	.04		.07	.04		.01	.10	.04	.08
80	.09	.04		.05	.03			.09	.03	.06
85	.05	.04		.01	.01			.05	.02	.03
90	.04	.04						.04	.01	.01
95	.02	.02						.02	.01	.01
100	.01	.01						.01		
105	.01	.01						.01		
110	.01	.01						.01		
115	.01	.01						.01		
120	.01	.01						.01		

Project ______________________ Computed ______
Subject ______________________ Date ______ Sht. 12-24 Of ____

Using the method outlined in Table 2-10:

	Ultimate Conditions	
T(min)	10-year Q (cfs)	100-year Q (cfs)
0	0	0
5	0	1
10	3	7
15	9	19
20	21	42
25	48	96
30	107	220
35	195	402
40	294	601
45	384	777
50	451	903
55	488	968
60	493	973
65	474	930
70	435	854
75	386	757
80	332	652
85	279	547
90	229	450
95	185	363
100	146	288
105	115	226
110	88	174
115	68	133
120	51	101
125	38	76
130	28	56
135	21	41
140	15	30
145	11	22
150	8	16
155	6	11
160	4	8
165	2	4
170	1	2
175	1	1
180		1

Project ________________ Computed ________

Subject ________________ Date ______ Sht. 12-25 Of ____

Natural 10-year peak discharge, unit hydrograph, and storm hydrograph

1. $L = 1.94$ miles $\quad A = 1.11\ mi^2$

   $L_c = 0.99$ miles

   $S = 0.045$ ft/ft

   $I = 0\%$

2. From Fig. 2-4, $C_{t_0} = 1.12$ for $I = 0\%$

3. Adjust C_{t_0} for $S = 0.045$ ft/ft

   $C_t = 0.48(1.12)(0.045)^{-0.2}$

   $C_t = 1.00$

4. Find C_p. $\quad C_p = 0.89(1.00)^{0.46}$

   $C_p = 0.89$

5. Find t_p. $\quad t_p = 1.00[(1.94)(0.99)]^{0.3}$

   $t_p = 1.216$ hrs

6. Find q_p. $\quad q_p = 640(0.89)(1.11)/1.216$

   $q_p = 520$ cfs

7. Find T_p. $\quad T_p = 1.216 + 0.042$

   $T_p = 1.258$ hrs

8. Find z. $\quad z = 0.992(0.89)(1.258)/1.216$

   $z = 0.913$

9. From Fig 2-6, $\omega = 5.41$ for $z = 0.913$

   Use $\omega = 5$.

Project ______ Computed ______

Subject ______ Date ______ Sht. ___ Of ___ 12-26

10. Find unit hydrograph.

$$q = q_P (T/T_P)^{\omega} e^{(1-T/T_P)\omega}$$

T(min)	T/T_P	q/q_P	q (cfs)
0	0.00	0.00	0
5	0.07	0.00	0
10	0.13	0.00	1
15	0.20	0.02	9
20	0.26	0.05	25
25	0.33	0.11	58
30	0.40	0.21	107
35	0.46	0.31	159
40	0.53	0.44	228
45	0.60	0.57	277
50	0.66	0.69	356
55	0.73	0.80	416
60	0.79	0.88	457
65	0.86	0.95	493
70	0.93	0.99	513
75	0.99	1.00	520
80	1.06	0.99	516
85	1.13	0.96	500
90	1.19	0.92	480
95	1.26	0.87	450
100	1.32	0.81	421
105	1.39	0.74	383
110	1.46	0.67	346
115	1.52	0.60	313
120	1.59	0.53	277
⋮	⋮	⋮	⋮

Since the duration of rainfall excess is only 25 minutes, this should be adequate to define the peak of the storm hydrograph. Find natural 10-year rainfall excess and peak discharge.

Table 2-9 Determination of Rainfall Excess
Natural 10-year Storm

		Pervious Area (100%)				Impervious Area (0%)				
Time (min)	Rainfall Increment (in.)	Infiltration (in.)	Depression Storage (in.)	Rainfall Excess (in.)	100% Rainfall Excess (in.)	Depression Storage (in.)	Other Losses (in.)	Rainfall Excess (in.)	0% Rainfall Excess (in.)	Composite Rainfall Excess (in.)
(1) Total	(2) (1.28)	(3) (0.57)	(4) (0.40)	(5) (0.31)	(6) (0.31)	(7)	(8)	(9)	(10)	(11) (0.31)
5	.01	.01								
10	.01	.01								
15	.01	.01								
20	.01	.01								
25	.01	.01								
30	.02	.02								
35	.03	.03								
40	.04	.04								
45	.06	.04	.02							
50	.08	.04	.04							
55	.14	.04	.10							
60	.34	.04	.24	.06	.06					.06
65	.18	.04		.14	.14					.14
70	.11	.04		.07	.07					.07
75	.07	.04		.03	.03					.03
80	.05	.04		.01	.01					.01
85	.03	.03								
90	.02	.02								
95	.01	.01								
100	.01	.01								
105	.01	.01								
110	.01	.01								
115	.01	.01								
120	.01	.01								

Project ______ Computed ______

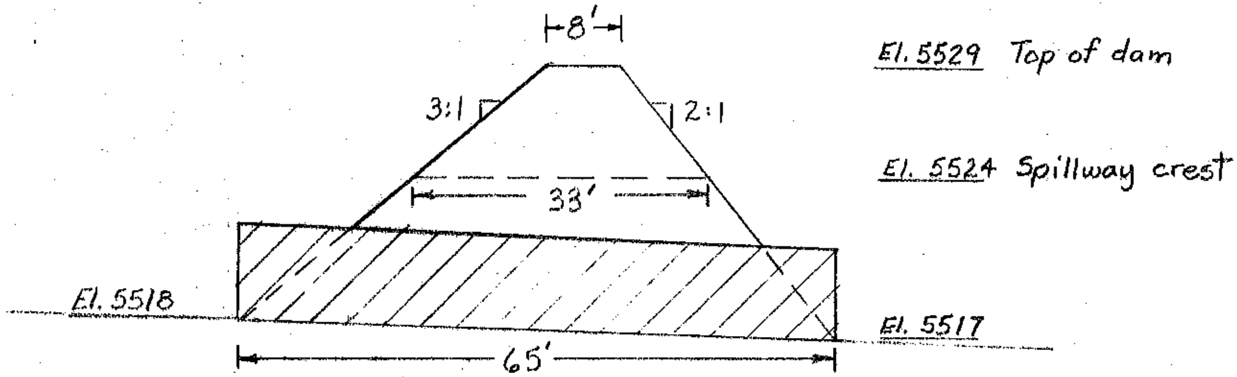
Subject ______ Date ______ Sht. 12-28 Of ______

Using the method outlined in Table 2-10:

T(min)	Natural Conditions 10-year Q (cfs)
⋮	⋮
75	155
80	159
85	159
90	156
⋮	⋮

Therefore the natural 10-year peak discharge is 159 cfs. This will be the target peak outflow from detention storage.

Detention Basin Outlet Works - "Katie Park"



1. Principal outlet: 48" RCP
   $n = 0.013$ $IE = 5518$

2. Emergency spillway: $b = 12'$ $IE = 5524$
   $z = 3$
   $n = 0.04$ from Table 7-2 for poorly maintained grass channel

3. Stage - storage data

El (ft msl)	Area (ac)	Avg. Area (ac)	Depth (ft)	Volume (ac-ft)	Cum. Volume (ac-ft)
5518	1				0
		1.5	2	3	
5520	2				3
		3.5	2	7	
5522	5				10
		7.5	2	15	
5524	10				25
		15	2	30	
5526	20				55
		35	2	70	
5528	50				125

4. Stage - discharge data

A. Weir control at outlet pipe at El. 5520

$Q_w = C P H_w^{1.5}$ where: $C = 3.1$, $P = \pi/2$ ft (effective perimeter); and $H_w = 2'$. eq (9-1)

$Q_w = 14$ cfs

B. Orifice control at outlet pipe for El. $\geq$ 5522

$Q_o = C A \sqrt{2 g H_o}$ where: $C = (1.1 + 0.026 L d^{-1.2})^{-0.5} = 0.84$ for beveled-edged RCP; $A = 4\pi$ sq ft; and $H_o =$ El. $-$ 5520. eq. (9-2)

$Q_o = 85 \sqrt{H_o}$

C. Pipe control at outlet pipe for El. $\geq$ 5522

$Q_p = A \sqrt{2 g H_p / (1 + k_e + k_b + k_f L)}$ eq (9-3)

where: $A = 4\pi$ sq ft; $k_e = 0.2$ for beveled-edged RCP; $k_b = 0$; $k_f = 185 n^2 d^{-1.33} = 0.005$; and $H_p =$ El. $-$ Tailwater El.

$Q_p = 82 \sqrt{H_p}$ (Assuming tailwater does not control.)

D. Critical flow at spillway for El. $\geq$ 5524

$Q_s = 3.087 (b + z H_{ec}) H_{ec}^{1.5}$

where: $H_{ec} = H_s / (1 + 4.315 n^2 L H_{ec}^{-1.33})$; $H_s =$ El. $-$ 5524; $b = 12'$; and $z = 3$.

$V_c = 4.63 \sqrt{H_{ec}}$ and $S_c = 21.4 n^2 / [Q/(b + z H_{ec})]^{0.22}$

where: V_c = critical velocity (fps); and S_c = critical slope in exit channel (ft/ft)

Assume exit channel is at critical slope for the upper range of spillway discharges.

El. (ft msl)	Q_w (cfs)	Q_o (cfs)	Q_s (cfs)	Q_{total} (cfs)
5518	0			0
5520	14			14
5522		120		120
5524		170	0	170
5526		210	130	340
5528		240	550	790

5. Route the ultimate 10-year hydrograph through the basin. Consider only the outlet pipe discharge.

Results: Peak discharge = 163 cfs ≈ 159 cfs (OK)
Peak elevation = 5523.7 < 5524 (OK)

See tabular routing. P. 12-33

6. Route the ultimate 100-year hydrograph through the basin starting at El. 5524. This is done by subtracting 30 acre-feet from each storage value. Assume that the outlet pipe is clogged with debris by considering only the spillway discharge.

Results: Peak discharge = 265 cfs
Peak elevation = 5526.6 < 5527.5 (OK)

See tabular routing. P. 12-33

7. Now that both the spillway crest and top-of-dam elevations have been checked, route the ultimate 100-year hydrograph through the empty basin. Use both the outlet pipe and spillway discharges.

Results: Peak discharge = 301 cfs
Peak elevation = 5525.5

This peak discharge is used for the downstream channel design.

See tabular routing. P. 12-34

8. Determine slope of exit channel. Use 1/4 of the peak discharge from Step 6 (265 cfs).

$$Q = 265/4 = 66 \text{ cfs}$$

$$H_{ec} = 1.23 \text{ ft} = \left[\frac{66}{3.086(12+3(1.23))}\right]^{0.67} \quad \text{(by trial and error)}$$

$$S_c = \frac{21.4\,(0.04)^2}{\left[66/(12+3(1.23)\right]^{0.22}} = 0.025 \text{ ft/ft}$$

9. Check critical velocity at 265 cfs.

$$H_{ec} = 2.65 \text{ ft} = \left[\frac{265}{3.086(12+3(2.65))}\right]^{0.67} \text{(by trial and error)}$$

$$V_c = 4.63\sqrt{2.65} = 7.5 \text{ fps}$$

(High, but is <u>very</u> unlikely too occur.)

10-year routing

FLOOD ROUTING BY
MODIFIED PULS

IMPOUNDMENT: KATIE PARK

DISCHARGE(CFS)	STORAGE(AF)
0.00	0.0
14.00	3.0
120.00	12.0
170.00	25.0
210.00	55.0
240.00	125.0

DISCHARGE(CFS)	S/ΔT+O/2
0.00	0.000
14.00	442.500
120.00	1512.000
170.00	3715.000
210.00	8091.000
240.00	18270.000

T(MIN)	INFLOW(CFS)	OUTFLOW(CFS)
0	0.0	0.0
5	3.0	0.0
10	9.0	.2
15	21.0	.7
20	48.0	1.8
25	107.0	4.2
30	195.0	8.8
35	294.0	21.1
40	384.0	52.6
45	451.0	88.7
50	488.0	121.5
55	493.0	129.9
60	474.0	137.9
65	435.0	145.1
70	386.0	151.1
75	332.0	155.8
80	279.0	159.2
85	229.0	161.3
90	185.0	162.4
95	146.0	162.5
100	115.0	161.7
105	88.0	160.4
110	68.0	158.5
115	51.0	156.2
120	38.0	153.7
125	28.0	151.0
130	21.0	148.1
135	15.0	145.2
140	11.0	142.2
145	8.0	139.1
150	6.0	136.1
155	4.0	133.2
160	2.0	130.2
165	1.0	127.3
170	1.0	124.4
175	0.0	121.6

VOLUME STORED = 22.8 (AF)

100-year spillway routing

FLOOD ROUTING BY
MODIFIED PULS

IMPOUNDMENT: KATIE PARK

DISCHARGE(CFS)	STORAGE(AF)
0.00	0.0
130.00	30.0
550.00	100.0

DISCHARGE(CFS)	S/ΔT+O/2
0.00	0.000
130.00	4422.743
550.00	14800.810

T(MIN)	INFLOW(CFS)	OUTFLOW(CFS)
0	0.0	0.0
5	1.0	0.0
10	7.0	.1
15	19.0	.5
20	42.0	1.4
25	96.0	3.4
30	220.0	7.6
35	402.0	16.9
40	601.0	31.1
45	777.0	50.4
50	903.0	73.7
55	968.0	99.0
60	973.0	124.5
65	930.0	156.1
70	854.0	185.9
75	757.0	211.0
80	652.0	230.9
85	547.0	245.9
90	450.0	256.1
95	363.0	262.2
100	288.0	264.7
105	226.0	264.4
110	174.0	261.8
115	133.0	257.4
120	101.0	251.7
125	76.0	245.1
130	56.0	237.9
135	41.0	230.2
140	30.0	222.3
145	22.0	214.4
150	16.0	206.5
155	11.0	198.7
160	8.0	191.0
165	4.0	183.5
170	2.0	176.2
175	1.0	169.1
180	1.0	162.3
185	0.0	155.5

FLOOD ROUTING BY
MODIFIED PULS

100-yr routing *12-34*

IMPOUNDMENT KATIE PARK

DISCHARGE(CFS)	STORAGE(AF)
0.00	0.0
14.00	3.0
120.00	10.0
170.00	25.0
340.00	55.0
790.00	125.0

DISCHARGE(CFS)	S/ΔT+Q/2
0.00	0.000
14.00	442.600
120.00	1512.000
170.00	3715.000
340.00	8156.000
790.00	18545.000

T(MIN)	INFLOW(CFS)	OUTFLOW(CFS)
0	0.0	0.0
5	1.0	0.0
10	7.0	.1
15	19.0	.5
20	42.0	1.5
25	96.0	3.6
30	220.0	8.5
35	402.0	26.8
40	691.0	73.8
45	777.0	123.4
50	903.0	139.7
55	968.0	157.7
60	973.0	180.4
65	930.0	209.9
70	854.0	236.0
75	757.0	257.8
80	652.0	274.9
95	547.0	287.4
90	450.0	295.4
95	363.0	299.7
100	288.0	300.7
105	226.0	299.0
110	174.0	295.2
115	133.0	289.8
120	101.0	283.2
125	76.0	275.7
130	56.0	267.7
135	41.0	259.3
140	30.0	250.7
145	22.0	242.1
150	16.0	233.6
155	11.0	225.2
160	8.0	216.9
165	4.0	208.8
170	3.0	201.0
175	1.0	193.3
180	1.0	186.0
185	0.0	178.9

PROJECTS ____
SUBJECT ____
COMPUTED ____ CHECKED ____ DATE ____

12.60 Open channel design

Using peak discharge data generated in Section 12.50, design a channel to convey the discharges from the detention cell through the planned development

Given: $Q_{100} = 301$ cfs
channel slope = 0.9 % or .009'/ft.
side slopes = 3 horizontal to 1 vertical
(z = 3)

Find: Dimensions of trapezoidal channel to convey 100-year discharge from cell

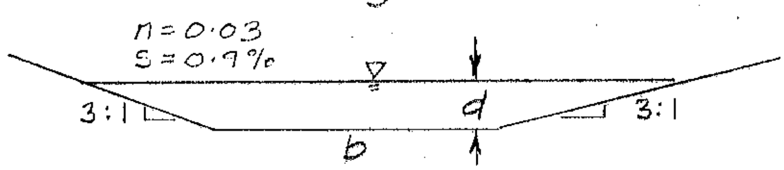
1. S = .009'/ft.
n = .030 from Table 7-2 for well-maintained grass channel

$Qn = 301\,(0.03) = 9.0$

2. From Figure 7-1 Find bottom width for channel such that $W/d \geq 6$
$\therefore\ d/W \leq 0.17$

3. Try W = 25'
$d/W = 0.068$
$d = d/W\,(W)$
$= 0.068(25) = 1.70'$

4. Check with Manning's Formula



	PROJECTS
	SUBJECT
	COMPUTED ______ CHECKED ______ DATE ______ PAGE 12-36 OF ______

$$A = bd + zd^2$$
$$= 25(1.70) + 3(1.70)^2$$
$$= 51.2 \text{ sq ft}$$

$$r = A/P$$
$$= \frac{A}{b + 2\sqrt{d^2 + (zd)^2}}$$

$$= \frac{51.2}{25 + 2\sqrt{(1.70)^2 + [3(1.70)]^2}}$$

$$= 1.43 \text{ ft}$$

$$V = \frac{1.49}{n} r^{2/3} s^{1/2}$$

$$= \frac{1.49}{0.030} (1.43)^{2/3} (0.009)^{1/2}$$

$$= 5.98 \text{ ft/sec.} \quad \text{(OK)}$$

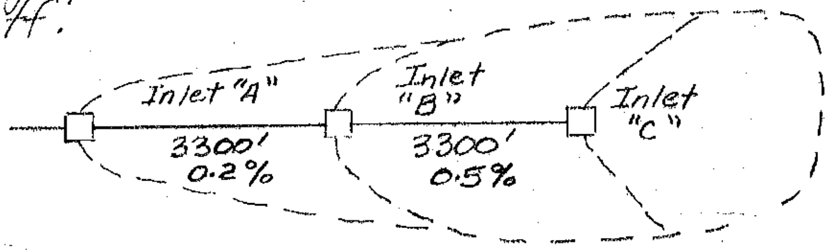
$$Q = VA$$
$$= 5.98 (51.2)$$
$$= \underline{\underline{306 \text{ cfs}}}$$

5. Minimum Freeboard = 1 foot

$\therefore$ use $d = 3'$
$b = 25'$
$Z = 3$
$S = 0.009$ 1/ft.

12.70 Trunk Sewer Design by the Colorado Urban Hydrograph Procedure

A major trunk sewer will be required as a result of this project. Since the drainage area exceeds 200 acres, the Colorado Urban Hydrograph Procedure will be used to determine runoff.



Basin Characteristics

	"A"	"B"	"C"
Area	100 ac	150 ac	150 ac
L	.55 mi	.68 mi	.68 mi
L_c	.28 mi	.35 mi	.35 mi
S	1.5 %	2.0 %	2.0 %
I	60 %	60 %	60 %

1. Develop a 10-year ultimate hydrograph for each sub-basin. (See Section 12.50.)

	"A"	"B"	"C"
C_t	.32	.32	.32
t_p	11.0 min	12.5 min	12.5 min
C_p	.53	.53	.53
q_p	290 cfs	382 cfs	382 cfs
T_p	13.5 min	15.0 min	15.0 min
Z	.65	.63	.63
ω	3	2.5	2.5
Q_{10}	150 cfs	217 cfs	217 cfs

Project ______ Computed ______

Subject ______ Date ______ Sht. 12-38 Of ______

2. Size pipe from "C" to "B."

Sewer grade = 0.005 ft/ft

n = 0.013 (RCP)

Q_{10_C} = 217 cfs

Try 66"ϕ (Capacity = 237 cfs)

From Figures 5-1 and 5-3: $d/D = 0.75$

$V_{66''\phi} = 11.3$ fps (OK)

$$\text{Travel time} = \frac{3300'}{11.3\,(60)} = 4.9 \text{ min}$$

3. Lag hydrograph at "C" by 5 min and add to hydrograph at "B."

$Q_{10_{B+C}}$ = 425 cfs

4. Size pipe from "B" to "A."

Sewer grade = 0.002 ft/ft

n = 0.013

Try 102"ϕ (Capacity = 479 cfs)

From Figures 5-1 and 5-3: $d/D = 0.73$

$V_{102''\phi} = 9.5$ fps (OK)

$$\text{Travel time} = \frac{3300}{9.5\,(60)} = 5.8 \text{ min}$$

Project ______ Computed ______

Subject ______ Date ______ Sht. 12-39 Of ______

5. Lag hydrographs at "C" and "B" by 10 min and 5 min, respectively, and add to hydrograph at "A."

$$Q_{10_{A+B+C}} = 517 \text{ cfs}$$

6. Size pipe below "A."

Sewer grade = 0.002 ft/ft
n = 0.013

Try 108" ϕ (Capacity = 558 cfs)

Assume full-pipe flow because of tailwater.

$V_{108''\phi}$ = 8.6 fps (OK)

12.80 Detention basin design by Rational Formula

Determine the required volume to control runoff of the 10-year storm from an 8 acre parking lot. Assume the lot is divided into 4 cells of 2 acres each, and that the slopes are 2%. The inlet time for each cell is 5 minutes.

Given: Area = 2 acres
C = 0.9
T_c = 5 min
Freq = 10 yr

Find: Family of curves for Modified Rational Formula
Critical volume

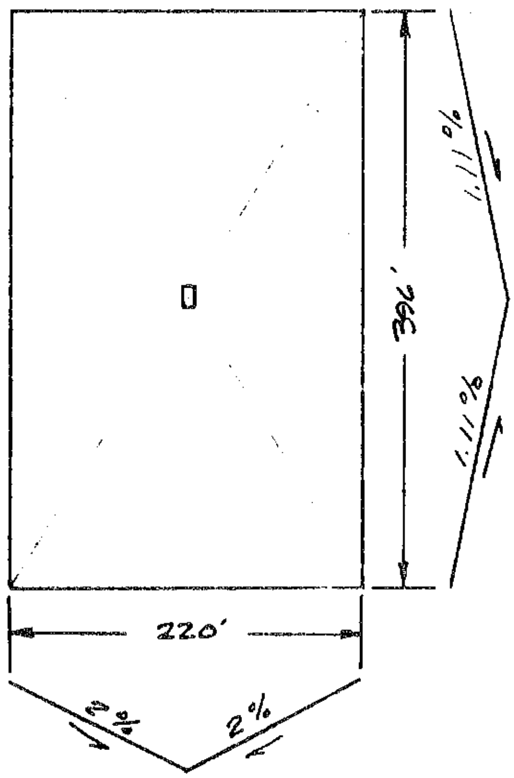
1. Curve family

D	i, "/hr	Q_p, cfs
5	8.8	15.84
10	7.0	12.60
15	5.8	10.44
20	5.0	9.00
30	3.9	7.02
40	3.3	5.94
50	2.8	5.04
60	2.4	4.32

$$\text{Volume} = \frac{Q_p, \text{cf}}{\text{sec}} \Big| \frac{60\,\text{sec}}{\text{min}} \Big| \text{dur, min}$$

	PROJECTS ____
	SUBJECT ____
	COMPUTED ____ CHECKED ____ DATE ____ PAGE 12-41 OF ____

2. Cell dimensions



Cell volume = $\frac{1}{3}$ (220)(396)(2.2)

= 63888 cf

3. Critical volume
Assume release rate = 3.5 cfs

D	St. vol, cf.	Rel. vol, cf	Storage, cf.
5	4752	1050	3702
10	7560	2100	5460
15	9396	3150	6246
20	10800	4200	6600
30	12636	6300	6336
40	14256	8400	5856
50	15120	10500	4620
60	15552	12600	2952

PROJECTS

SUBJECT

COMPUTED CHECKED DATE PAGE 12-42 OF

4. Check ponding depth

$6600 \text{ cf} = \frac{1}{3} A d$

try $d = 0.50$ ft
$A = 4500$
$V = 750$ cft

try $d = 1.04$
$A = 19468.8$ sf
$V = 6749$ cf
~ 12½" - requirement is 12" close

5. Inlet design
$Q_i = 5.37 A d^{0.5}$

Solve for A

$\frac{3.5}{5.37 d^{.5}} = 0.64$ sf

or $Q_i = 3.0 P d^{1.5}$

$P = \frac{3.5}{3 d^{1.5}} = 1.1$ ft

very small grate is needed - 12" ϕ pipe

	PROJECTS ___ ⟨
	SUBJECT ___
	COMPUTED ___ CHECKED ___ DATE ___ PAGE 12-43 OF ___

12.90 Culvert design

A new culvert will be needed under the arterial street which borders the proposed shopping center on the south side. The total drainage area above the culvert is 118 acres, and the time of concentration is 15 minutes. The composite runoff coefficient is 0.8.

Downstream of the culvert the channel has been regraded and aligned with a channel slope of 1%. The outlet elevation will be elevation 5510. The natural stream slope through the area was 2%. The proposed culvert will be 150 feet long, and the natural stream bed elevation at the inlet is elevation 5516. The top of road elevation is elevation 5524.

Design a culvert which will pass the 100-year flood without exceeding the top of road elevation. Determine the culvert size needed assuming inlet control. Also determine the culvert size needed using an improved inlet.
Use the forms and charts provided in Section 6 of the manual.

1. Discharge - use Rational Formula

$Q = 0.8 \times 4.17 iph \times 96 acres \times 1.25 = 400$ cfs

use 400 cfs

PROJECT:

DESIGNER: -.

DATE:

HYDROLOGIC AND CHANNEL INFORMATION

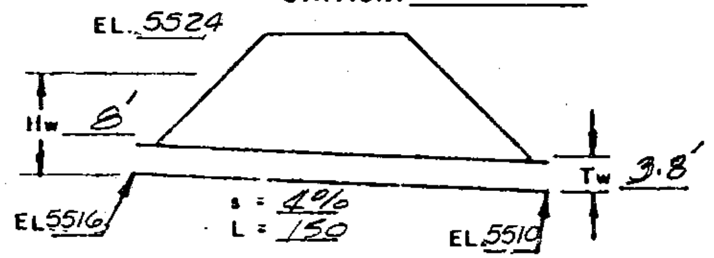
Q_1 = 400 TAILWATER ELEVATION = 5513.8

Q_2 = ____ TAILWATER ELEVATION = ____

(Q_1 = DESIGN DISCHARGE, SAY Q_{25}
Q_2 = CHECK DISCHARGE, SAY Q_{50} OR Q_{100})

SKETCH

STATION: ____



MEAN STREAM VELOCITY = ____ $\frac{L}{100s}$ = 38

MAX. STREAM VELOCITY = ____

CULVERT DESCRIPTION (ENTRANCE TYPE)	Q	SIZE	HEADWATER COMPUTATION: INLET CONT. $\frac{Hw}{D}$	INLET CONT. Hw	OUTLET CONTROL $HW = H + h_o - L_s$: K_e	H	d_c	$\frac{d_c+D}{2}$	Tw	h_o	L_s	Hw	CHART No.	CHART Hw	CONTROLLING Hw	OUTLET VELOCITY	COST	COMMENTS
groove edge proj	400	96"	.93	7.44	0.2	1.35	4.9	6.45		6.45	6	1.8	6.31/6.22	7.4	7.4	13.8		riprap apron

SUMMARY & RECOMENDATIONS:

Inlet control governs; use 96" RCP

PROJECT: ____________

OUTLET CONTROL DESIGN CALCULATIONS

DESIGNER: ____________

STATION: ____________

DATE: ____________

INITIAL DATA:

Q_{100} = 400 cfs

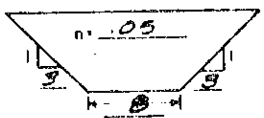
AHW El. = 5524 ft.

S_o = .02 (.01 d/s)

L_a = 150 ft.

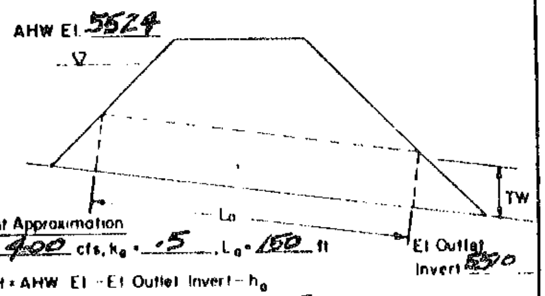
El. Outlet Invert 5510 ft.

Stream Data:



Barrel Shape and Material: RCP Barrel n = 0.012

SKETCH



First Approximation

Q = 400 cfs, k_e = .5, L_o = 150 ft

H = AHW El − El Outlet Invert − h_o

5524 − 5510 − 5 = 9

∴ A = ____ ft² or D = ____ ft; Try ____

Q	Q/N	* H	Q/NB	(1) d_c	$\frac{d_c+D}{2}$	Qn	(2) TW	(3) h_o	(4) HW_o	(5) V_o	COMMENTS
Trial No 1, N = 1, B = 5, D = 5, k_e = 0.5	60" RCP w/ square edge										
400	400	6-22 13	400	>5	5	20	7-1 3.8	5	5528		exceeds allowalle HW
Trial No. 2, N = 1, B = -, D = 66", k_e = 0.2	larger pipe w/ beveled edge										
400	400	7	400	5.3	5.4	20	3.8	5.5	5522.4	16.8	HW_o is OK, but V_o > 15fps
350	350	5.3	350	5.1	5.3			5.1	5520.4		Q_{100} − 50 cfs
450	450	8.8	450	>5.5	5.5			5.5	5524.3		Q_{100} + 50 cfs
Trial No ____, N = ____, B = ____, D = ____, k_e = ____											

Notes and Equations:

(1) d_c cannot exceed D

(2) TW based on d_n in natural channel, or other downstream control.

(3) $h_o = \frac{d_c+D}{2}$ or TW, whichever is larger.

(4) HW_o = H + h_o + El. Outlet Invert.

(5) Outlet Velocity (V_o) = Q/Area defined by d_c or TW, not greater than D. Do not compute until control section is known.

SELECTED DESIGN

N = 1

B = ____ ft.

D = 5.5 ft.

k_e = 0.2

At Design Q:

HW_o = 5522.4 ft

V_o = 16.8 f/s

$$* \quad H = \left[1 + k_e + \frac{29n^2 \cdot L}{R^{1.33}}\right] \frac{V^2}{2g}$$

PROJECT: ______ **CULVERT INLET CONTROL SECTION DESIGN CALCULATIONS** DESIGNER: ______

STATION: ______ DATE: ______

INITIAL DATA:

Q_{100} = 400 cfs

AHW El. = 5524 ft.

S_o = .027

L_a = 150 ft.

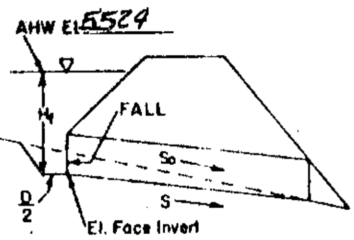
El. Stream Bed at Face 5516 ft.

Barrel Shape and Material RCP Barrel n = 0.012

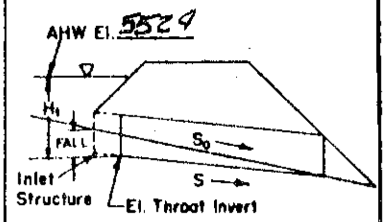
N = 1, B = ______

D = 66", $NBD^{3/2}$ = ______

(Pipe) $ND^{5/2}$ = 70.9



CONVENTIONAL or BEVELED INLET: FACE CONTROL SECTION (Upper Headings)



TAPERED INLET THROAT CONTROL SECTION (Lower Headings)

DEFINITIONS OF INLET CONTROL SECTION

Q	$\frac{Q}{NB}$ / $\frac{Q}{NBD^{3/2}}$	$\frac{H_f}{D}$ / $\frac{H_t}{D}$	H_f / H_t	(1) El. Face Invert / El Throat Invert	El. Stream Bed At Face	(2) FALL	(3) HW_f / HW_t	(4) S	(5) V_o	Note: Use Upper Headings for Coventional or Beveled Face; Lower Headings for Tapered Inlet Throat. / COMMENTS
Trial No 1	Inlet and Edge Description Beveled inlet; Type B									
400	400	6-33 2.1	11.6	5512.4	5516	3.6	5524.		16.8	fall too large; try tapered inlet
–										
Trial No 2	Inlet and Edge Description tapered throat - smooth									
400	5.69	6-35 1.8	9.9	5514.1	5516	1.9	5524		16.	ok; calc. perf. curves
350	4.94	1.65	9.1				5523.2			
450	6.35	2.20	12.1				5526.2			
Trial No ___	Inlet and Edge Description ______									

Notes and Equations

(1) El Face (or throat) invert = AHW El − H_f (or H_t)

(2) FALL = El. Stream Bed at Face − El. face (or throat) invert

(3) HW_f (or HW_t) = H_f (or H_t) + El face (or throat) invert, where El face (or throat) invert should not exceed El. stream bed.

(4) $S \approx S_o - FALL/L_a$

(5) Outlet Velocity = Q/Area defined by d_n at S

SELECTED DESIGN

Inlet Description:

FALL = 1.9 ft

Invert El = 5514.1 ft.

Bevels:

Angle = N/A

b = – in, d = – in.

PROJECT: ______ DESIGNER: ______

SIDE-TAPERED INLET
DESIGN CALCULATIONS

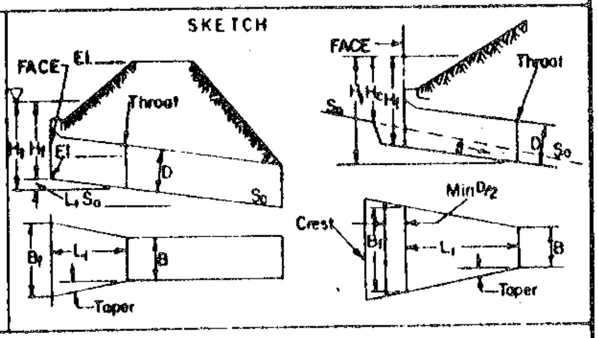
STATION: ______ DATE: ______

INITIAL DATA

Q_{100} = 400 cfs S_o = .02
AHW El = 5524 ft. L_a = 150 ft.
TAPER = 4 : 1
Barrel Shape and Material 66" RCP
Face Edge Description 45° bevels

N = 1, B = – ft, D = 5.5 ft

SKETCH



Q	El. Throat Invert	(1) ~~H_f/D~~ / H_f/E	~~$\frac{Q}{B_f D^{3/2}}$~~ / $\frac{Q}{A_f E^{1/2}}$	~~$D^{3/2}$~~ / $E^{1/2}$	(2) ~~Min B_f~~ / Min A_f	B_f	(3) L_f	(4) S	$L_f S$	(5) El. Face Invert	Upper Headings for Box Culverts, Lower Headings for Pipes COMMENTS
Trial No 1, Q = 400, HW_f = 5524.0											
400	5514.1	1.62	6-38 4.6	2.35	37	10	9	.027	0.24	5514.34	~~$B_f D^{3/2}$~~ [or ~~$A_f E^{1/2}$~~] = 43.20
											H_f HW_f
350		1.20	3.45								6.6 5520.94
400		1.38	3.94								7.6 5521.94
Trial No 1, Q = 400, HW_f = 5524.0											
											~~$B_f D^{3/2}$~~ [or ~~$A_f E^{1/2}$~~] = 43.20
450		1.55	4.43								8.53 5522.87
Trial No ___, Q = ___, HW_f = ___											
											$B_f D^{3/2}$ [or $A_f E^{1/2}$] = ___

Notes and Equations:

(1) H_f/D [or H_f/E] = (HW_f – El. Throat Invert – 1) / D [or E]
D≤E≤1.1D

(2) $\text{Min } B_f = \frac{Q}{(D^{3/2})(Q / B_f D^{3/2})}$

$\text{Min } A_f = \frac{Q}{(E^{1/2})(Q / A_f E^{1/2})}$

(3) $L_f = \left[\frac{B_f - NB}{2}\right]$ TAPER

(4) From throat design

(5) El. Face Invert – El. Throat Invert > 1 ft., recompute
Face and Throat may be lowered to better fit site, but do not raise

Since the HW of the natural stream is ≈5520 at design Q, this design is acceptable.

SELECTED DESIGN

B_f = 10' ft.
L_f = 9' ft.
Bevels: Angle 45 °
d = 5.5 in., b = 2.8 in.
Crest Check:
HW_c = 5524 ft
H_c = 8 ft.
Q/W = 64 (Fig 6-39)
Min. W = 6.25 ft. < B_f (OK)