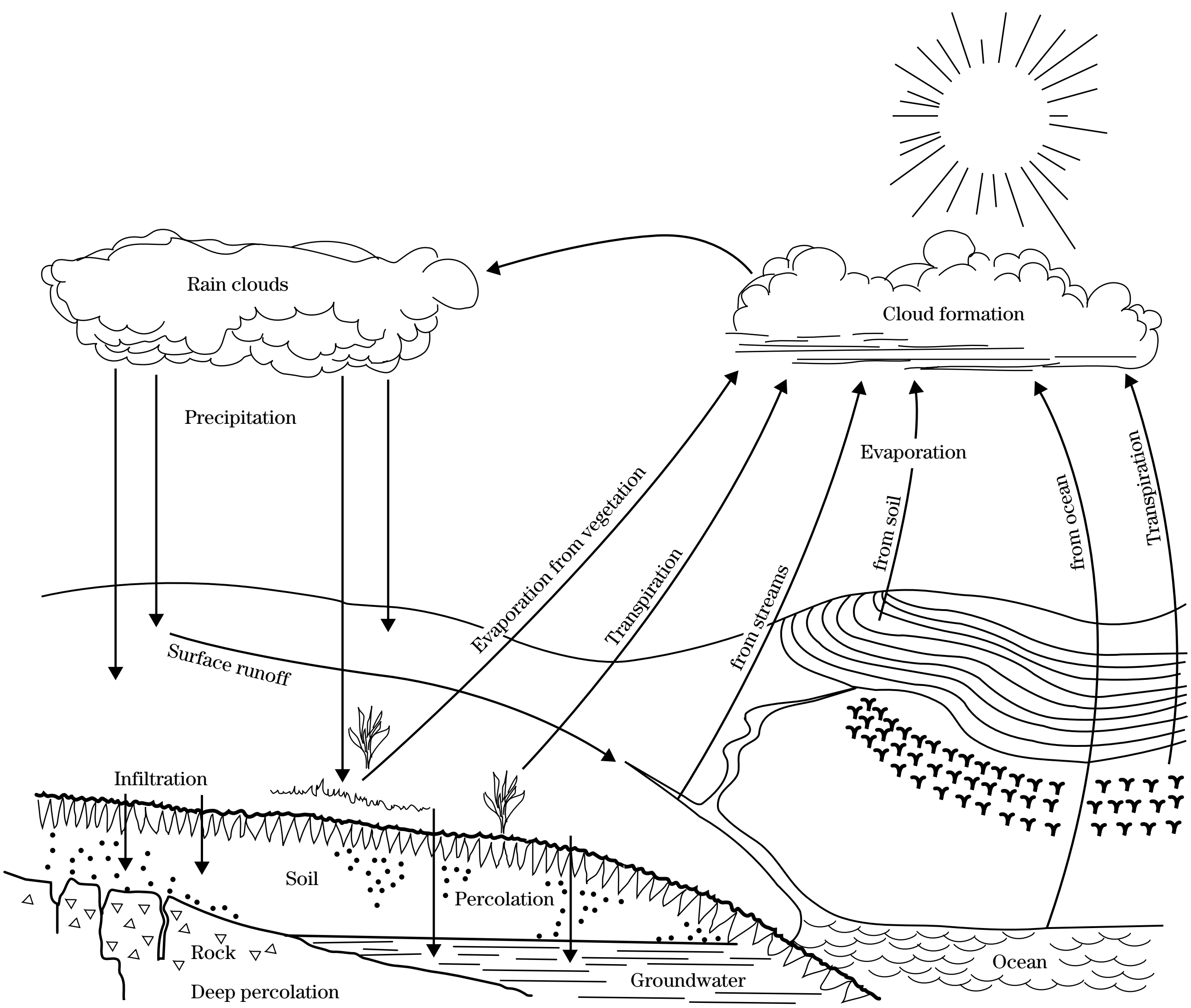
United States Department of Agriculture
Natural Resources Conservation Service

Part 630 Hydrology
National Engineering Handbook

Chapter 14 Stage Discharge Relations



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Chapter 14 Stage Discharge Relations

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630.1400 Introduction

In planning and evaluating aspects of watershed protection, it is necessary for U.S. Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS) hydrologists and engineers to develop stage discharge curves at selected locations on natural streams.

Many hydraulics textbooks, handbooks (NRCS National Engineering Handbook (NEH) Section 5, Hydraulics, 1956), and others contain methods for developing stage discharge curves assuming nonuniform steady flow. Some of these methods are elaborate and time consuming. The type of available field data and the use to be made of these stage discharge curves should dictate the method used in developing the curves.

This chapter presents methods of developing these curves at selected points on a natural stream. This chapter demonstrates hand calculation methods. Many of these are now automated by computer programs such as HEC–RAS (2010b).

The examples in this chapter contain many tables generated by calculator or spreadsheet. Some values may be different from values calculated by other methods. The hand methods, including charts, graphs, and tables, represent valid engineering methods and principles. Numerical accuracy is a function of the number of significant digits and the algorithms used in data processing, so some slight differences in numbers may be found if the examples are checked by other means.

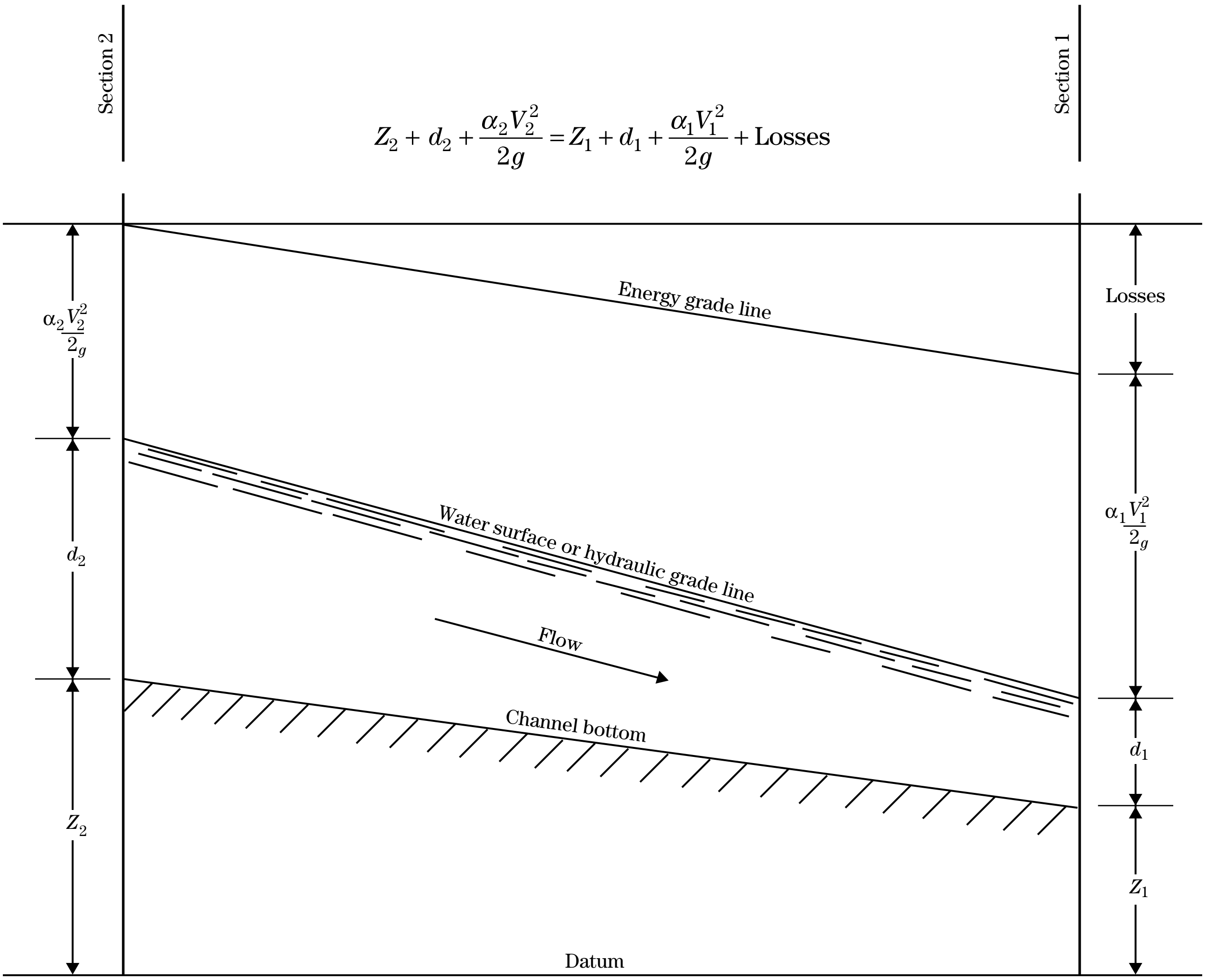
The rate of change of discharge for a given portion of the stage discharge curve can differ between the rising and falling sides of a hydrograph. Some streams occupy relatively small channels during low flows, but overflow onto wide floodplains during high discharges. During the rising stage, the flow away from the stream causes a steeper slope than that for a constant discharge and produces a highly variable discharge with distance along the channel. After passage of the flood crest, the water reenters the stream and again causes a highly variable discharge, together with a stream slope less than that for a constant discharge. The effect on the stage discharge relation is to produce what is called a loop rating for each flood (Chow 1959). Generally, in the work performed by the NRCS, the maximum stage the water reaches is of primary interest. Therefore, the stage discharge curve used for routing purposes is a plot for the maximum elevation obtained during the passage of flood hydrographs of varying magnitudes. This results in the plot being a single line. A detailed treatment of unsteady flow is included in standard textbooks, such as Chow (1959), and is beyond the scope of this chapter.

One of the basic equations used throughout this chapter, which constitutes the basis of many hydraulic calculations, is the energy balance equation, sometimes referred to as the "Bernoulli equation." At two locations along a stream, the energy of the water at the upstream location equals the energy of the water at the downstream location plus energy losses that occur between the two locations. Figure 14–1 shows a diagram of how this is applied along a stream.

The hydraulic grade line is a hydraulic profile of the piezometric level of water at all points along the line. The term is usually applied to water moving in a conduit, open channel, stream, etc., but it may also be applied to free or confined groundwater. In an open channel it is the free water surface. The energy grade line is a line drawn above the hydraulic grade line a distance equivalent to the velocity head of the flowing water at each section along a stream, channel, or conduit (ASCE 1962).

The energy at the upstream cross section 2 consists of the channel bottom elevation (Z_2) plus the depth of water (d_2) plus the velocity head, which is calculated based on the velocity head correction factor (α_2), the average velocity of flowing water (V_2), and the acceleration of gravity (g). The energy is generally represented in units of elevation such as feet or meters. The velocity head correction coefficient (α) has a value of 1.0 if the velocity of water is the same for the entire cross section. If the velocity varies within the cross section, such as in a cross section with channel and floodplain segments, the value of α is greater than 1.0. This energy is to be balanced against the energy at the downstream cross section, which consists of the channel bottom elevation (Z_1) depth of water (d_1), and velocity head of flowing water plus the energy losses occurring between the two cross sections. The energy losses are due to two major factors. One is friction loss. Friction loss between two cross sections is generally estimated using Manning's equation (described in detail in NEH630.1401). Energy losses can also occur through expansion or contraction of the flow. For ex-

Figure 14–1 Energy balance equation



ample, if the velocity decreases from the upstream to downstream direction, it is referred to as "expansion loss." Much more detail concerning the energy balance equation is available in standard hydraulic engineering texts such as Chow (1959).

630.1401 Development of stage discharge curves

(a) Direct measurement

The most direct method of developing stage discharge curves for natural streams is to obtain velocities at selected points through a cross section. The most popular method is to use a current meter; other methods include the use of dynamometers, floats, Pitot tubes, and chemical and electrical methods. From these velocities and associated cross-sectional areas, the discharge is computed for various stages on the rising and falling side of a flood flow and a stage discharge curve developed.

The current meter method is described in detail in the U.S. Geological Survey (USGS) Water Supply Paper 2175, Measurement and Computation of Streamflow, Volume 1, Measurement of Stage and Discharge (USGS 1982).

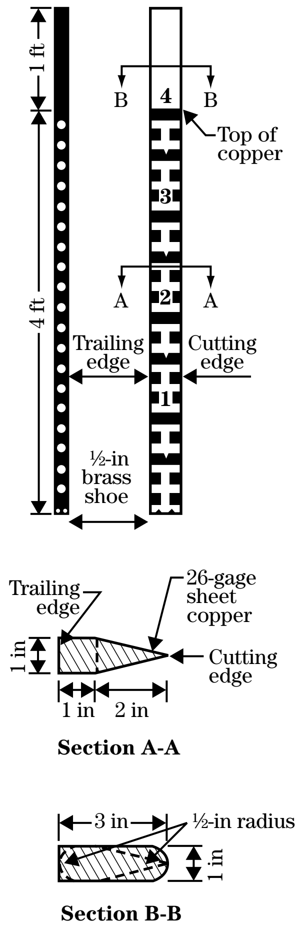
The velocity head rod (fig. 14–2) may be used to measure flows in small streams or baseflow in larger streams (Carufel 1980). In making a measurement with a velocity head rod, a tape is stretched across the flowing stream, and both depth and velocity head readings are taken at selected points that represent the cross section of the channel. Table 14–1 is an example of a discharge determined by the velocity head rod. The data is tabulated as shown in columns 1, 2, and 3, and the computation made as shown.

The total area of flow in the section is shown in column 9, and the total discharge in column 10. The average velocity is 45.19/15.00, or 3.01 feet per second.

(b) Indirect measurements

Indirectly, discharge is measured by methods such as slope-area, contracted-opening, flow over a dam, flow through a culvert, and critical depth. These methods, described in Techniques of Water Resources Investigations of the U.S. Geological Survey, Book 3, chapters 3–7 (USGS 1967, 68), use information on the water-surface profile for a specific flood peak and the hydraulic characteristics of the channel to determine the peak discharge.

Figure 14–2 Velocity head rod for measuring streamflow



Velocity head rod developed at San Dimas experimental forest

The rod is first placed in the water with its foot on the bottom and the sharp edge facing directly upstream. The stream depth at this point is indicated by the water elevation at the sharp edge, neglecting the slight ripple or bow wave. If the rod is now revolved 180 degrees so that the flat edge is turned upstream, a hydraulic jump will be formed by the obstruction to the flow of the stream. After the depth or first reading has been subtracted from the second reading, the net height of the jump equals the actual velocity head at that point. Velocity can then be computed by the standard formula:

$$V = \sqrt{2g\Delta h} = 8.02\sqrt{\Delta h}$$

where:
V = velocity in ft/s
g = acceleration of gravity, 32.16 ft/s^2
Δh = velocity head in ft

The average discharge for the stream is obtained by taking a number of measurements of depth and velocity throughout its cross section and applying the equation:

$$Q = AV$$

where:
Q = discharge in ft^3/s
A = cross-sectional area in ft^2
V = velocity in ft/s

Velocity for different values of Δh:

$$V = 8.02\sqrt{\Delta h}$$

Velocity head, h (ft)	**Velocity (ft/s)**
0.05	1.79
0.10	2.54
0.15	3.11
0.20	3.59
0.25	4.01
0.30	4.39
0.35	4.74
0.40	5.07
0.45	5.38
0.50	5.67

Table 14–1 Computation of discharge using velocity head rod (VHR) measurements

	Depth of flow using VHR			Velocity [1/]					
Distance along section ft (1)	**Cutting edge ft (2)**	**Flat edge ft (3)**	**Δh col (3)–col (2) ft (4)**	**At point ft/s (5)**	**Average for section ft/s (6)**	**Mean depth from col (2) ft (7)**	**Width from col (1) ft (8)**	**Area col (7) × col (8) ft^2 (9)**	**Discharge col (9) × col (6) ft^3/s (10)**
3.5	0	0	0	0					
					0.9	0.68	1.00	0.68	0.61
4.5	1.35	1.4	0.05	1.8					
					2.15	2.05	0.40	0.82	1.76
4.9	2.75	2.85	0.1	2.5					
					3.35	2.90	1.00	2.90	9.72
5.9	3.05	3.32	0.27	4.2					
					4.05	3.03	1.50	4.54	18.39
7.4	3.01	3.25	0.24	3.9					
					3.40	2.60	0.50	1.30	4.42
7.9	2.18	2.31	0.13	2.9					
					2.35	1.48	2.80	4.14	9.73
10.7	0.78	0.83	0.05	1.8					
					0.9	0.39	1.60	0.62	0.56
12.3	0	0							
						Totals		15.00	45.19

Note:
1/ Column 5 is read from table in figure 14–2 using Δh in column 4.

It should be remembered that no indirect method of discharge determination can be of an accuracy equal to a meter measurement.

Fairly accurate discharges may be computed from measurements made of flows over different types of weirs by using the appropriate formula and coefficients selected from the Handbook of Hydraulics (Brater and King 1976), sections 4 and 5. Overfall dams or broad-crested weirs provide an excellent location to determine discharges. Details on procedures for broad-crested weirs may be found in Brater and King (1976).

(c) Slope-area estimates

Field measurements taken after a flood are used to determine one or more points on the stage discharge curve at a selected location. The peak discharge of the flood is estimated using high water marks to determine the slope.

Three or four cross sections are usually surveyed so that two or more independent estimates of discharge, based on pairs of cross sections, can be made and averaged. Additional field work required for slope-area estimates consists of selecting the stream reach, estimating *n* values, and surveying the channel profile and high water profile at selected cross sections. The work is guided by the following:

- The selected reach is as uniform in channel alignment, slope, size, and shape of cross section, and factors affecting the roughness coefficient *n* as is practicable to obtain. The selected reach should not contain sudden breaks in channel bottom grade such as shallow drops or rock ledges.
- Elevations of selected high water marks are determined on both sides of the channel for each cross section.
- The three or more cross sections are located to represent as closely as possible the hydraulic characteristics of the reach. Distances between sections must be long enough to keep the errors in estimating stage or elevation small.

The flow in a channel reach is computed by one of the open-channel formulas. The most commonly used formula in the slope-area method is Manning's equation:

$$Q = \frac{1.486AR^{\frac{2}{3}}S^{\frac{1}{2}}}{n} \qquad \text{(eq. 14–1)}$$

where:
Q = discharge in ft³/s
n = Manning's coefficient of roughness
A = cross-sectional area in ft²
R = hydraulic radius in ft
S = slope of the energy gradient in ft/ft

Rearranging equation 14–1 gives:

$$Kd = \frac{Q}{S^{\frac{1}{2}}} = \frac{1.486AR^{\frac{2}{3}}}{n} \qquad \text{(eq. 14–2)}$$

where:
Kd = cross-sectional conveyance

The right side of equation 14–2 contains only the physical characteristics of the cross section and is referred to as the "conveyance," Kd. The slope is determined from the elevations of the high water marks and the distances between the high water marks along the direction of flow.

Manning's formula has been used to develop stage discharge curves for natural streams assuming the water surface to be parallel to the slope of the channel bottom. This can lead to large errors since this condition can only exist in long reaches having the same bed slope without a change in cross section shape or retardance.

In computing the hydraulic parameters of a cross section on a natural stream when floodplain flow exists, it is desirable to divide the cross section into segments. The number of segments will depend on the irregularity of the cross section and the variation in *n* values assigned to the different portions. Appendix A gives a method of determining *n* values for use in computing stage discharge curves. For a description of determining the water surface in curved channels, see NEH654.06, Stream Hydraulics (NRCS 2007).

Example 14–1 Peak discharge computation from slope-area measurements

Using data for the Concho River near San Angelo, Texas, for the September 17, 1936, flood, compute the peak discharge that occurred. Figure 14–3 shows sections A and B with the high water mark profile along the stream reach between the two sections.

Step 1 Draw a water surface through the average of the high water marks. From figure 14–3, the elevation of the water surface at the lower cross section B is 55.98 feet designated in the example as E_2. The elevation of the water surface at cross section A is 56.50 feet designated as E_1. The difference in elevation between E_2 and E_1 is 0.52 feet.

Step 2 Compute the length of reach between the two sections. From figure 14–3, the length of reach is 680 feet (1,100 – 420). The slope S of the reach $E_2 - E_1$ is (0.52 ft)/(680 ft) or 0.000765. This value is used in step 10 to calculate the discharge Q.

Step 3 Divide each cross section into segments as needed due to different *n* values as shown in figure 14–3.

Step 4 Compute the cross-sectional area (A) and wetted perimeter (WP) for each segment of each cross section. Tabulate in columns 2 and 3 of table 14–2(a) for cross section A and table 14–2(b) for cross section B.

Step 5 Compute hydraulic radius R (R = A/WP) for each segment, and tabulate in column 4, table 14–2(a) and 14–2(b). For segment 1 of section B = 1,598/236 = 6.77 feet.

Step 6 Compute $R^{2/3}$ for each segment, and tabulate in column 5. For segment 1 of cross section B, $R^{2/3} = (6.77)^{2/3} = 3.58$ feet.

Step 7 Tabulate the Manning's *n* value. Tabulate the *n* value assigned to each segment in column 6 of table 14–2(a) and 14–2(b).

Step 8 Compute Kd for each segment. Kd is computed by multiplying 1.486 times columns (2) times (5) and dividing by column (6). This is commonly called the conveyance. For segment 1 of cross section A, Kd = 1.486(2,354)(4.44)/.08 = 1.94×10^5.

Step 9 Compute the total area and the total Kd sum in columns 2 and 7 of table 14–2(a) and 14–2(b).

Step 10 Compute the velocity for each segment of cross sections A and B. In column 8 of tables 14–2(a) and (b), compute the velocity for each segment by multiplying column 7 by the square root of the slope (0.000765) and dividing by column 2.

Step 11 Compute the velocity cubed multiplied by the area for each segment. Raise the value in column 8 to the third power, and multiply by column 2, and tabulate the results in column 9, table 14–2 (a) and (b). Sum the values for all segments for each cross section.

Step 12 Compute the velocity head correction coefficient alpha (α) for cross sections A and B. Compute the average velocity (column 10), average velocity cubed times total area (column 11), and alpha (column 12). The average velocity (column 10) equals the total conveyance (the sum of the values in column 7) multiplied by the square root of the slope (0.000765) divided by the sum of the values in column 2. The average velocity cubed times the area (column 11) equals the average velocity (column 10) raised to the third power multiplied by the sum of the values in column 2. Alpha (column 12) equals the sum of values in column 9 divided by the value in column 11.

Step 13 Use the energy balance between sections A and B to estimate the discharge. The calculations are in table 14–2 (c). Select a discharge to see if energy balances. For the first try, use 240,000 cubic feet per second. Enter the value in column 1, table 14–2 (c).

Step 14 Compute the velocity at cross section A in column 2. Divide the discharge in column 1 by 34,729 square feet (total cross section area in table 14–2 (a) column 2), and enter the result 6.91 in column 2, table 14–2(c).

Step 15 Compute velocity head for cross section A in column 3. Square the value in column 2, multiply by alpha (α) in table 14–2 (a) column 12, and divide by 64.4. Enter the result in column 3, table 14–2 (c).

Step 16 Compute velocity at cross section B. Divide the discharge in column 1 by 32,771 square feet (total cross section area in table 14–2 (b) column 2). Enter the result in column 4, table 14–2 (c).

Step 17 Compute velocity head at cross section B in column 5. Square the value in column 4, multiply by alpha (α) in table 14–2 (b) column 12, and divide by 64.4. Enter the result in column 5, table 14–2 (c).

Figure 14–3 High water mark profile and cross sections, Concho River near San Angelo, Texas

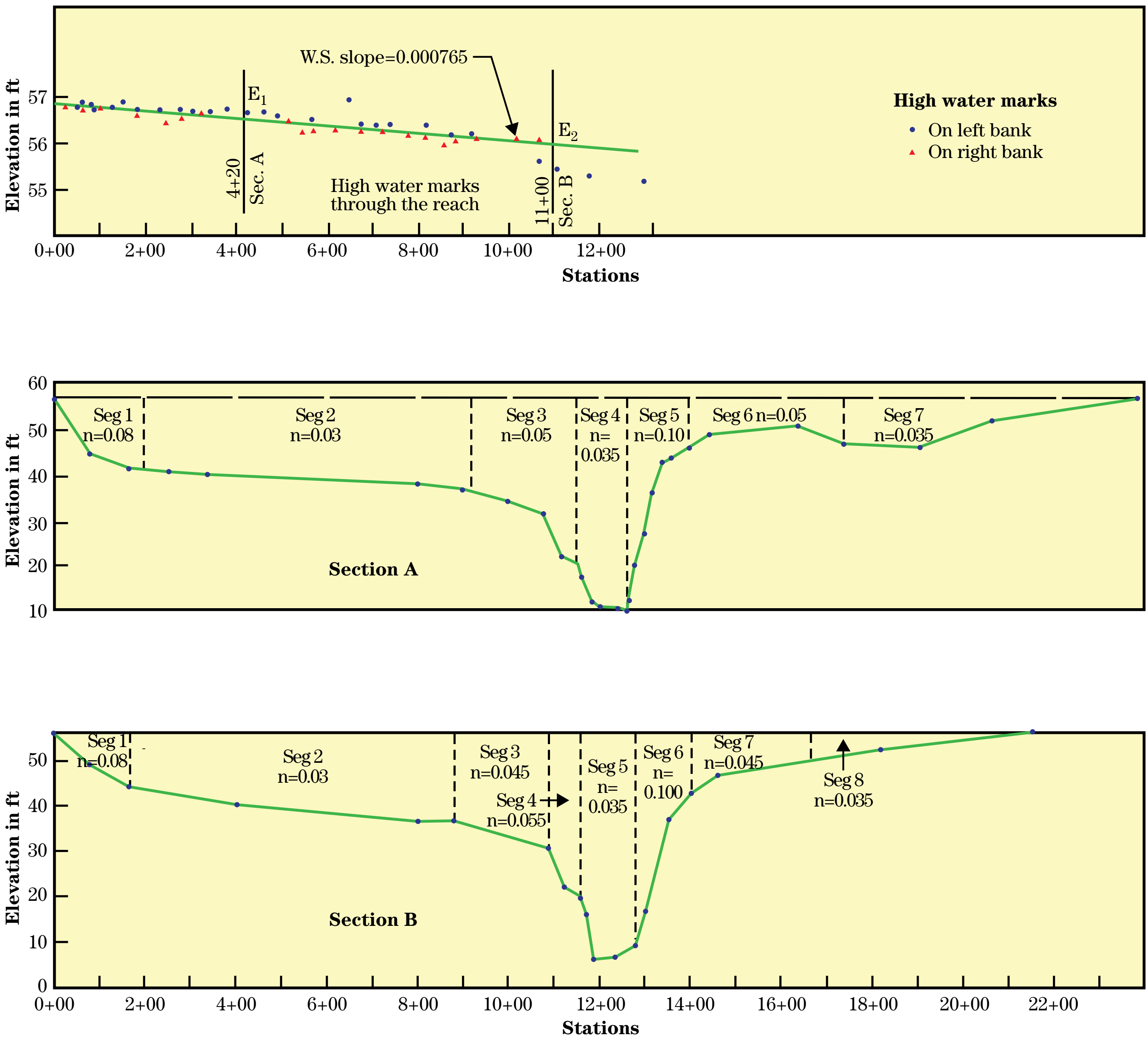


Table 14–2 Data for computing discharge from slope-area measurements, example 14–1

(a) Cross section A at station 4+20

section A

Segment (1)	Area A ft² (2)	Wetted perimeter WP ft (3)	Hydraulic radius R ft (4)	$R^{2/3}$ ft (5)	*n* (6)	Kd (7)	Velocity V ft/s (8)	V^3 A (9)	V_{ave} ft/s (10)	$V_{ave}^3 A_{tot}$ (11)	Alpha α (12)
1	2,354	252	9.34	4.44	0.080	1.94×10^5	2.28	2.79×10^4			
2	12,691	735	17.27	6.68	0.030	4.20×10^6	9.15	9.73×10^6			
3	5,862	231	25.38	8.64	0.050	1.50×10^6	7.10	2.10×10^6			
4	5,385	167	32.25	10.13	0.035	2.32×10^6	11.9	9.07×10^6			
5	2,523	135	18.69	7.04	0.100	2.64×10^5	2.89	6.12×10^4			
6	2,498	350	7.14	3.71	0.050	2.75×10^5	3.05	7.07×10^4			
7	3,416	645	5.30	3.04	0.035	4.41×10^5	3.57	1.55×10^5			
sum	34,729					9.19×10^6		2.12×10^7	7.32	1.36×10^7	1.56

(b) Cross section B at station 11+00

section B

Segment (1)	Area A ft² (2)	Wetted perimeter WP ft (3)	Hydraulic radius R ft (4)	$R^{2/3}$ ft (5)	*n* (6)	Kd (7)	Velocity V ft/s (8)	V^3 A (9)	V_{ave} ft/s (10)	$V_{ave}^3 A_{tot}$ (11)	Alpha α (12)
1	1,598	236	6.77	3.58	0.080	1.06×10^5	1.84	9.95×10^3			
2	11,750	725	16.21	6.40	0.030	3.73×10^6	8.77	7.93×10^6			
3	4,750	227	20.93	7.59	0.045	1.19×10^6	6.94	1.59×10^6			
4	2,486	78	31.87	10.05	0.055	6.75×10^5	7.51	1.05×10^6			
5	4,944	153	32.31	10.15	0.035	2.13×10^6	11.91	8.35×10^6			
6	3,455	134	25.78	8.73	0.100	4.48×10^5	3.59	1.60×10^5			
7	2,270	273	8.32	4.10	0.045	3.08×10^5	3.75	1.20×10^5			
8	1,518	513	2.96	2.06	0.035	1.33×10^5	2.42	2.15×10^4			
sum	32,771					8.72×10^6		1.92×10^7	7.36	1.31×10^7	1.47

(c) Using energy balance between cross sections A and B to estimate discharge

Discharge Q ft³/s (1)	Velocity V V_A ft/s (2)	$(\alpha V_A^2)/2g$ ft (3)	Velocity V V_B ft/s (4)	$(\alpha V_B^2)/2g$ ft (5)	S_f (6)	Friction loss ft (7)	Contraction loss ft (8)	H_L ft (9)	Upstream energy ft (10)	Downstream energy ft (11)	Difference ft (12)
240,000	6.91	1.16	7.32	1.22	0.000718	0.49	0.01	0.50	57.66	57.70	–0.04
235,000	6.77	1.11	7.17	1.17	0.000689	0.47	0.01	0.48	57.61	57.63	–0.02
230,000	6.62	1.06	7.02	1.12	0.000660	0.45	0.01	0.46	57.56	57.56	0.00

Step 18 Compute friction slope (column 6) for the discharge in column 1. Divide the discharge value (column 1) by the average conveyance at cross sections A and B (8,955,000), and then square this number. Place the result in column 6, table 14–2 (c).

Step 19 Compute friction loss in column 7. Multiply the value in column 6 by the reach length of 680 feet. Place result in column 7, table 14–2 (c).

Step 20 Compute contraction loss in column 8. Using a contraction coefficient of 0.1, multiply 0.1 times the difference in velocity heads (column 5 minus column 3). In this example, values are rounded to the nearest 0.01 foot. Place the result in column 8, table 14–2 (c).

Step 21 Compute total head loss between cross sections A and B. Add the friction loss in column 7 and the contraction loss in column 8. Place the result in column 9, table 14–2 (c).

Step 22 Compute upstream energy. Add the water surface elevation at the upstream cross section A (56.50 feet) and the velocity head from column 3. Place result in column 10, table 14–2 (c).

Step 23 Compute downstream energy plus head loss. Add the water surface elevation at the downstream cross section B (55.98 feet), the velocity head from column 5, and head loss from column 9. Place result in column 11, table 14–2 (c).

Step 24 Compute the difference in energy between upstream and downstream cross sections. Subtract the value in column 11 from the value in column 10. If the result is negative, upstream energy is too high to be balanced. Try a lower discharge, and repeat steps 1 through 12. In this example, the second estimated discharge is 235,000 cubic feet per second. If the energy difference in column 12 is less than 0.01 feet, then stop as the energy balances within the tolerance. The third estimate of discharge (230,000 ft^3/s) produces a match between the upstream and downstream energy and is the final discharge estimate.

This compares favorably with the 230,000 cubic feet per second computed in Water Supply Paper 816 for the Concho River. The solution just given for example 14–1, and similar cases can be automated by freely available tools, such as the Cross-Section Hydraulic Analyzer (Moore 2010) spreadsheet available at *http://go.usa.gov/0Eo*.

(d) Synthetic methods

There are various methods that depend entirely on data that may be gathered at any time. These methods establish a water surface slope based entirely on the physical elements present such as channel size and shape, floodplain size and shape, and the roughness coefficient. The method generally used by the NRCS is the standard step method in the U.S. Army Corps of Engineers (USACE) HEC–RAS water surface computer program (2010b).

This method bases the average rate of friction loss in the reach between two cross sections on the elements of those two cross sections. Manning's equation is applied to these elements, and the difference in elevation of the water surface plus the difference in velocity head between the two cross sections is assumed to be equal to the total energy loss in the reach. This method, ignoring the changes in velocity head, is illustrated later in example 14–6.

630.1402 Collection of field data

(a) Selecting Manning's *n* values

The selection of appropriate Manning's *n* values is important in the computation of water surface profiles and flow in open channels. The selected values should be representative of flow conditions at the cross section location. The cross section can be subdivided into several segments to properly represent the resistance to flow within a cross section. Normally, a natural cross section is subdivided into three segments: channel, right and left overbanks. The overbank segments can be further subdivided depending on the conditions. Manning's *n* values can vary with the depth of water within a segment. Chow (1959) includes a table of Manning's *n* values for five land uses with different water depths, and many of the other references cited contain similar information.

Standard textbooks and hydraulic references such as Brater and King (1976) and various NRCS and USGS books provide examples of Manning's *n* values in constructed and natural channels. The reference section lists some of the available Manning's *n* values references, such as U.S. Geological Survey (1967), Aldridge and Garrett (1973), U.S. Geological Survey (1989a) U.S. Army Corps of Engineers (USACE 2010b), Chow (1959), and Yochum (2010). Appendix A gives a procedure for estimating Manning's *n* values in natural streams, floodways, and similar streams (Cowan 1956).

(b) Selecting cross section locations

Valley sections can serve many needs (geologic, engineering, economic, hydraulic), and all of them should be considered when selecting the location. For hydraulic purposes, valley sections are surveyed at points along the valley length and need to be representative of parameters such as flow area, wetted perimeter, and roughness.

With the increased availability and quality of digital topography and imagery, such as light detection and ranging (LiDAR) and digital elevation models (DEM), cross section locations, reach lengths, and even Manning roughness estimates may be determined remotely at a project site. While locations of structures, such as bridges, culverts, or weirs, may be seen and located, often detailed information still needs to be collected for these in the field. Also, if there is water in a stream, elevations below the water surface need to be collected in the field if they are necessary for the analyses. Certain software packages, such as the HEC–RAS model (2010b) and computer aided drafting, save time in developing cross section and other hydraulic information. These software packages are generally designed to merge digital data and data collected in the field. Based on the project purpose, scope, and budget, use of digital topographic data may greatly enhance the quality of analyses.

The HEC–RAS model considers energy losses due to friction and expansion or contraction and uses the rate of friction loss at the upstream and downstream sections to estimate the rate of friction loss throughout the reach. Therefore, valley sections should be located as follows: divide the valley length into reaches with nearly constant parameters that affect hydraulics and locate the valley section near the upstream end of the reach. Two cross sections should be placed immediately above and below a road-type restriction and placed as noted in the HEC–RAS user's manual (2010c). Always start cross sections some distance either below (for subcritical flow) or above (for supercritical flow) the point where reliable estimates are needed. The distance above or below depends on slope and the landscape variability, but should be a minimum of five cross sections normally spaced. This will allow the HEC–RAS model to mitigate any errors in assumed or estimated starting elevations. This will improve the accuracy of the predicted water surface profiles for a given flow within the area of interest. For more detail on this procedure, see the technical references available on the USACE HEC–RAS Web site (*http://www.hec.usace.army.mil/software/hec-ras/*).

Survey sections should be perpendicular to the direction of flow and not necessarily straight across the valley. The Manning's *n* values used for the reach should be representative of the channel and floodplain downstream of the cross section.

(c) Selecting representative reach lengths

The flow distance between one section and the next has an important bearing on the friction losses between sections. For flows that are entirely within the channel the channel, distance should be used. On a meandering stream, the overbank portion of the flow may have a flow distance less than the channel distance. This distance approaches but does not equal the floodplain distance due to the effect of the channel on the flow. Determination of this average reach length will be important for use in reach routing equations and models.

From a practical standpoint, the water surface is considered level across a cross section. Thus, the elevation difference between two cross sections is considered equal for both the channel flow portion and the overbank portion.

It has been common practice to compute the conveyance for the total section and then compute the discharge by using a given slope with this conveyance, where the slope used is an average slope between the slope of the channel portion and the overbank portion. The average slope is computed by the formula:

$$S_a = \frac{H}{L_a} \qquad \text{(eq. 14–3)}$$

where:
S_a = average slope of energy gradient in reach
H = elevation difference of the energy level between sections, ft
L_a = average reach length, ft

The reach length L_a can be computed as follows:

$$Q_c = Kd_c \times S_c^{\frac{1}{2}} \qquad \text{(eq. 14–4)}$$

$$Q_f = Kd_f \times S_f^{\frac{1}{2}} \qquad \text{(eq 14–5)}$$

$$Q_t = Kd_t \times S_a^{\frac{1}{2}} \qquad \text{(eq. 14–6)}$$

where:
Q_c = discharge in channel portion, ft³/s
Kd_c = conveyance in channel portion
S_c = energy gradient in channel portion
Q_f = discharge in floodplain portion, ft³/s
Kd_f = conveyance in floodplain portion
S_f = energy gradient in floodplain portion
Q_t = total discharge, ft³/s
Kd_t = total conveyance
S_a = average slope of energy gradient

The subscript f refers to the combined right and left overbanks. The total discharge in a reach is equal to the flow in channel plus the flow in the overbank.

Then:

$$Q_t = Q_c + Q_f \qquad \text{(eq. 14–7)}$$

Substituting from equations 14–4, 14–5, and 14–6

$$Kd_t \times S_a^{\frac{1}{2}} = Kd_c \times S_c^{\frac{1}{2}} + Kd_f \times S_f^{\frac{1}{2}}$$

$$\text{Let } S_c = \frac{H}{L_c} \text{ and } S_f = \frac{H}{L_f} \qquad \text{(eq. 14–8)}$$

where:
H = elevation of reach head minus elevation of reach foot, ft
L_c = length of reach for channel, ft
L_f = length of reach for floodplain, ft

Then, substituting into equation 14–8 using the proper subscripts:

$$Kd_t \times \left(\frac{H}{L_a}\right)^{\frac{1}{2}} = Kd_c \times \left(\frac{H}{L_c}\right)^{\frac{1}{2}} + Kd_f \times \left(\frac{H}{L_f}\right)^{\frac{1}{2}} \qquad \text{(eq. 14–9)}$$

Divide both sides by $H^{½}$.

$$\frac{Kd_t}{L_a^{\frac{1}{2}}} = Kd_c \times \left(\frac{1}{L_c}\right)^{\frac{1}{2}} + Kd_f \times \left(\frac{1}{L_f}\right)^{\frac{1}{2}} = \frac{Kd_c}{L_c^{\frac{1}{2}}} + \frac{Kd_f}{L_f^{\frac{1}{2}}}$$

$$L_a = \left(\frac{Kd_t}{\frac{Kd_c}{L_c^{\frac{1}{2}}} + \frac{Kd_f}{L_f^{\frac{1}{2}}}}\right)^2 \qquad \text{(eq. 14–10)}$$

If the average reach length is plotted versus elevation for a section, it is possible to read the reach length directly to use with the Kd for any desired elevation. The data will plot in a form as shown in figure 14–4.

This procedure is somewhat difficult to use because each time a new elevation is selected for use, a new reach length must also be used. The procedure can be modified slightly by using a constant reach length in all computations.

Multiply both sides of equation 14–5 by:

$$\left(\frac{S_c}{S_c}\right)^{\frac{1}{2}}$$

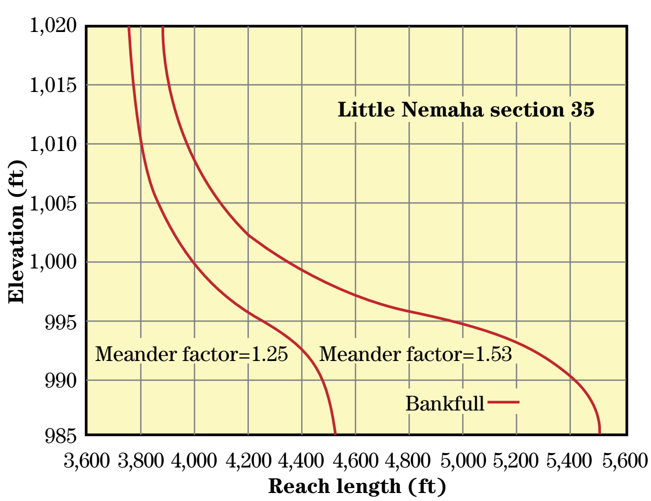
This gives:

$$Q_f\left(\frac{S_c}{S_c}\right)^{\frac{1}{2}} = \left(Kd_f\right)\left(S_c\right)^{\frac{1}{2}}\left(\frac{S_f}{S_c}\right)^{\frac{1}{2}} \qquad \text{(eq. 14–11)}$$

The $\left(\frac{S_c}{S_c}\right)^{\frac{1}{2}}$ on the left hand side drops out with a value of 1, giving:

$$Q_f = \left(Kd_f\right)\left(S_c\right)^{\frac{1}{2}}\left(\frac{S_f}{S_c}\right)^{\frac{1}{2}} \qquad \text{(eq. 14–12)}$$

Figure 14–4 Reach length versus elevation, Little Nemaha, section 35



S_f and S_c can be represented as:

$$S_f = \left(\frac{H}{L_f}\right) \quad \text{or} \quad \left(S_f\right)^{\frac{1}{2}} = \left(\frac{H}{L_f}\right)^{\frac{1}{2}} \qquad \text{(eq. 14–13)}$$

$$S_c = \left(\frac{H}{L_c}\right) \quad \text{or} \quad \left(S_c\right)^{\frac{1}{2}} = \left(\frac{H}{L_c}\right)^{\frac{1}{2}} \qquad \text{(eq. 14–14)}$$

Divide equation 14–13 by equation 14–14.

$$\left(\frac{S_f}{S_c}\right)^{\frac{1}{2}} = \frac{\left(\frac{H}{L_f}\right)^{\frac{1}{2}}}{\left(\frac{H}{L_c}\right)^{\frac{1}{2}}} = \left(\frac{L_c}{L_f}\right)^{\frac{1}{2}} \qquad \text{(eq. 14–15)}$$

Equation 14–12 becomes by substitution:

$$Q_f = \left(Kd_f\right)\left(S_c\right)^{\frac{1}{2}}\left(\frac{L_c}{L_f}\right)^{\frac{1}{2}} \qquad \text{(eq. 14–16)}$$

The term "L_c/L_f" is commonly referred to as the "meander factor."

Then, substituting equations 14–16 and 14–4 into equation 14–7:

$$Q_t = \left(Kd_c\right)\left(S_c\right)^{\frac{1}{2}} + \left(Kd_f\right)\left(S_c^{\frac{1}{2}}\right)\left(\frac{L_c}{L_f}\right)^{\frac{1}{2}}$$

Rearranging:

$$Q_t = \left(Kd_c + \left(Kd_f\right)\left(\frac{L_c}{L_f}\right)^{\frac{1}{2}}\right)\left(S_c\right)^{\frac{1}{2}} \qquad \text{(eq. 14–17)}$$

Equation 14–17 can be used to compute the total stage discharge at a section by using the meander factor, rather than a variable reach length. Example 14–5 illustrates the use of modifying the floodplain conveyance by the square root of the meander factor in developing a stage discharge curve. Equation 14–17 applies to a single cross section. It does not apply to water surface profiles, which introduce more than one section along a stream reach.

630.1403 Discharge versus drainage area

It is desirable for the water surface profile to represent a flow that has the same occurrence interval throughout the watershed. The cubic feet per second per square mile ($ft^3/s/mi^2$) values for most floods vary within a channel system having a smaller value for larger drainage areas. Thus, when running a profile for the 50 cubic feet per second per square mile rate at the outlet, the actual cubic feet per second per square mile rate will increase as the profile progresses up the watershed. The HEC–RAS model requires the user to input the discharge range at each cross section where the stage discharge relation is computed.

The rate of discharge at any point in the watershed can be based on the formula (Creager, Justin, and Hines 1945):

$$q_p = 46\,c\,a^{\left(\left(\frac{0.894}{a_1^{0.048}}\right)-1\right)} \qquad \text{(eq. 14–18)}$$

where:

q_p = discharge in $ft^3/s/mi^2$ for any specific location in the watershed

a = drainage area in mi^2

c = coefficient depending on the characteristics of the watershed

Equation 14–18 was derived based on a plot of the largest peak discharge at 720 locations in the contiguous United States (48 States), 5 locations in Hawaii, and 25 locations in foreign countries. The data includes years through 1940. Watersheds in the data set have drainage areas between 0.17 square miles and more than 2 million square miles (Amazon River, Brazil). The curve was developed to estimate extreme flood discharges at ungaged locations where only the drainage area is known. For this chapter, the equation is used only in a relative way to proportion the peak discharge along a stream and its tributaries. This procedure will provide an initial estimate of discharge along a stream and its tributaries. After estimating the peak discharge ($ft^3/s/mi^2$) at locations in the watershed for various return periods using a hydrologic model, these discharges (which provide more accurate values) may be entered into the hydraulic model to complete water surface profiles.

Assuming that c remains constant for any point in the watershed, then the discharge at any point in the watershed may be related to the discharge of any other point in the watershed by the formula.

$$\frac{q_{p1}}{q_{p2}} = K = \frac{46\,c\,a_1^{\left(\left(\frac{0.894}{a_1^{0.048}}\right)-1\right)}}{46\,c\,a_2^{\left(\left(\frac{0.894}{a_2^{0.048}}\right)-1\right)}}$$

Canceling 46 c, the resulting equation is:

$$\frac{q_{p1}}{q_{p2}} = K = \frac{a_1^{\left(\left(\frac{0.894}{a_1^{0.048}}\right)-1\right)}}{a_2^{\left(\left(\frac{0.894}{a_2^{0.048}}\right)-1\right)}} \qquad \text{(eq. 14–19)}$$

where:

q_{p1} and a_1 = discharge rate in $ft^3/s/mi^2$ and drainage area in mi^2 of one point in the watershed

q_{p2} and a_2 = discharge rate in $ft^3/s/mi^2$ and drainage area in mi^2 at another point

K = ratio of q_{p1} and q_{p2}

The coefficient c does not need to be estimated because for most watersheds the coefficient cancels out in equation 14–19. Equation 14–19, therefore, applies to all watershed regardless of what c equals for an individual watershed.

In practice, q_{p2} and a_2 usually represent the outlet of the watershed and remain constant, and a_1 is varied to obtain q_{p1} at other points of interest within the watershed.

Equation 14–19 is plotted in exhibit 14–1 for the case where a_2 is 400 square miles. This curve may be used directly to obtain the cubic feet per second per square mile discharge of the outlet if the outlet is at 400 square miles as shown in example 14–2. Example 14–3 shows how to use exhibit 14–1 if the drainage area at the outlet is not 400 square miles.

Other methods can also be used to determine the flow for various watershed locations. Some of the other acceptable methods would include hydrology programs such as WinTR–20 Computer Program for Project Formulation Hydrology, ver. 1.11 (2009) or HEC–HMS, Hydrologic Modeling System, ver. 3.5 (2010a) and regression techniques such as those published by USGS.

The USGS regional regression curves can also be used to compute the discharge at the upstream cross sections when using the HEC–RAS water surface profile computer program. Be sure to use USGS regional regression curves and equations within their valid range for drainage area, location, percent impervious areas, and other watershed characteristics, as apply.

Example 14–2 Peak discharge computation using exhibit 14-1 for drainage area of 400 square miles

Find the cubic feet per second per square mile discharge value to be used for a reach with a drainage area of 50 square miles when the cubic feet per second per square mile discharge at the outlet is 80 cubic feet per second per square mile. The drainage area at the outlet is 400 square miles.

Step 1 Determine K for a drainage area of 50 square miles. From Exhibit 14–1 with a drainage area of 50 square miles read K = 2.61.

Step 2 Determine the cubic feet per second per square mile rate for 50 square miles. Multiply the cubic feet per second per square mile rate at the outlet by K computed in step 1:

$$(80)\ (2.61) = 209\ \text{ft}^3/\text{s/mi}^2 \text{ at } 50\ \text{mi}^2$$

This example could also be solved using equation 14–19.

Example 14–3 Peak discharge computation using exhibit 14-1 for drainage area of 50 square miles

Find the cubic feet per second per square mile discharge value to be used for a reach with a drainage area of 20 square miles if the drainage area at the outlet is 50 square miles. The cubic feet per second per square mile rate at the outlet is 60 cubic feet per second per square mile.

Step 1 Determine K for a drainage area of 20 square miles. From exhibit 14–1 with a drainage area of 20 square miles read K=3.66.

Step 2 Determine K for a drainage area of 50 square miles. From exhibit 14–1 for a drainage area of 50 square miles read K=2.61.

Step 3 Compute a new K value for a drainage area of 20 square miles. Divide step 1 by step 2.

$$\frac{3.66}{2.61} = 1.40$$

Step 4 Determine cubic feet per second per square mile rate for the 20 square mile drainage area. Multiply K obtained in step 3 by the cubic feet per second per square mile at the outlet.

$$(1.40)(60) = 84\ \text{ft}^3/\text{s/mi}^2$$

630.1404 Computing profiles

When using water surface profiles to develop stage discharge curves for flows at more than critical depth, it is necessary to have a stage discharge curve for a starting point at the lower end of a reach. This starting point may be a stage discharge curve developed by current meter measurements or one computed from a control section where the flow passes through critical discharge, or it may be one computed from the elements of the cross section and an estimate of the slope. The latter case is the most commonly used by the NRCS since the more accurate stage discharge curves are not generally available on small watersheds. Locate four to five normally spaced cross sections downstream (or upstream for supercritical flow) of the first point of interest where reliable data is needed. This allows for any errors in the starting elevation to be mitigated by distance and elevation. The actual distance needed (and number of cross sections) to obtain reliable model results increases with lesser slopes or increased errors in the assumed or computed starting elevation.

Example 14–4 Develop stage discharge curve for cross section

Develop the starting stage discharge curve for cross section M–1 (fig. 14–5) shown as the first cross section at the outlet of the watershed, assuming an energy gradient S_a of 0.001 feet per foot.

Step 1 Plot the surveyed cross section. From field survey notes, plot the cross section, figure 14–6(a), noting the points where there is an apparent change in the *n* value.

Step 2 Divide the cross section into segments. An abrupt change in shape or a change in *n* is the main factor to be considered in determining extent and number of segments required for a particular cross section. Compute the *n* value for each segment using appendix A, or the *n* may be based on other data or publications.

Step 3 Plot the channel segment on an enlarged scale. Figure 14–6(b) is for use in computing the area and measuring the wetted perimeter at selected elevations in the channel. The length of the segment at selected elevations is used as the wetted perimeter for the floodplain segments. The division line between each segment is not considered as wetted perimeter.

Step 4 Tabulate elevations to be used in making computations. Start at the lowest point of the channel, and proceed to an elevation equal to or above any flood of record, and tabulate in column 1 of table 14–3 elevations that will be required to define the hydraulic elements of each segment.

Step 5 Compute the wetted perimeter at each elevation listed in step 4. Starting at the lowest elevation in column 1, measure or calculate the wetted perimeter of each segment at each elevation, and tabulate in columns 3, 6, 9, and 12 of table 14–3. Note that the maximum wetted perimeter for the channel segment is 62 at elevation 94.

Step 6 Compute the cross-sectional area for each elevation listed in step 4. Starting at the lowest elevation, compute the accumulated cross-sectional area for each segment at each elevation in column 1, and tabulate in columns 2, 5, 8, and 11 of table 14–3.

Step 7 Compute Kd for each elevation. Compute:

$$\text{Kd} = \frac{1.486(\text{A})\left(\text{R}^{\frac{2}{3}}\right)}{n}$$

and tabulate in columns 4, 7, 10, and 13, table 14–3.

Step 8 Sum columns 4, 7, 10, and 13, and tabulate in column 14. A plot of column 14 and elevation is shown on figure 14–7. The elevation scale is selected based on the elevation of the channel bottom.

Step 9 Compute the discharge for each elevation. Using the average slope at cross section M–l, S = .00l feet per foot, develop stage discharge data for cross section M–l, Q = $(S_a^{1/2})$(Kd). The stage discharge curve for cross section M–l is shown on figure 14–8.

The next example shows the effect of a meandering channel in a floodplain on the elevation discharge relation. Equation 14–17 will be used to determine the discharge.

Figure 14–5 Schematic of watershed outlet with cross sections and floodplain for examples 14–4, 14–5, 14–6, 14–8, 14–9, and 14–10

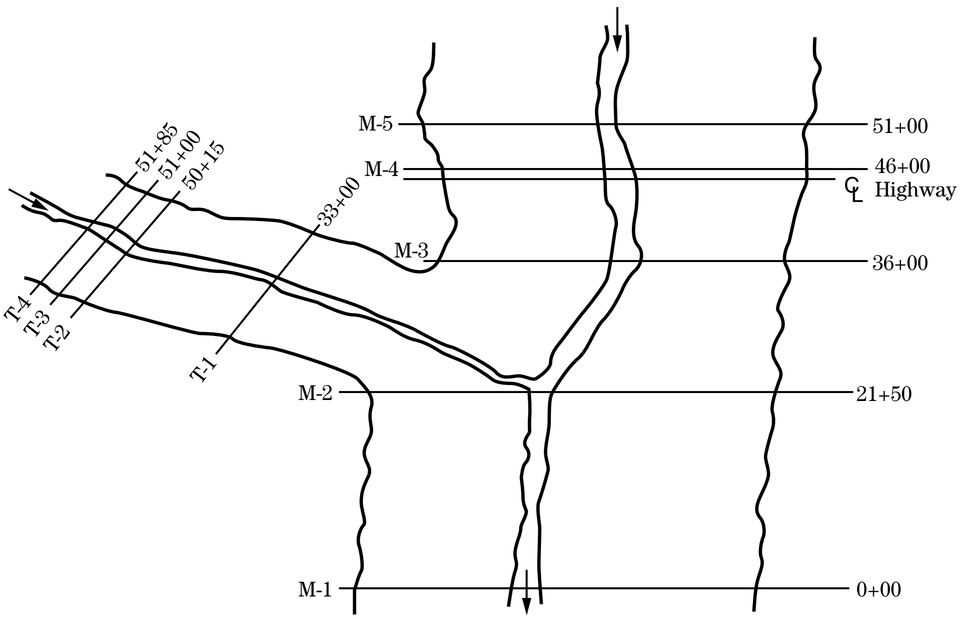


Figure 14–6 Cross section M–1, examples 14–4 and 14–5

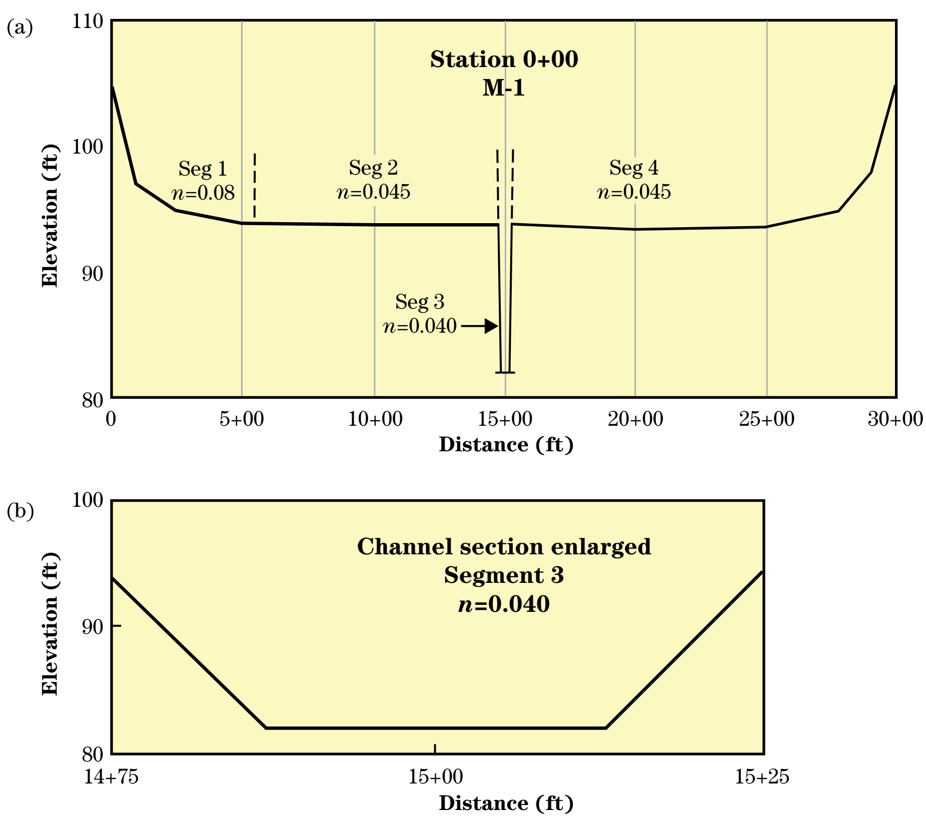
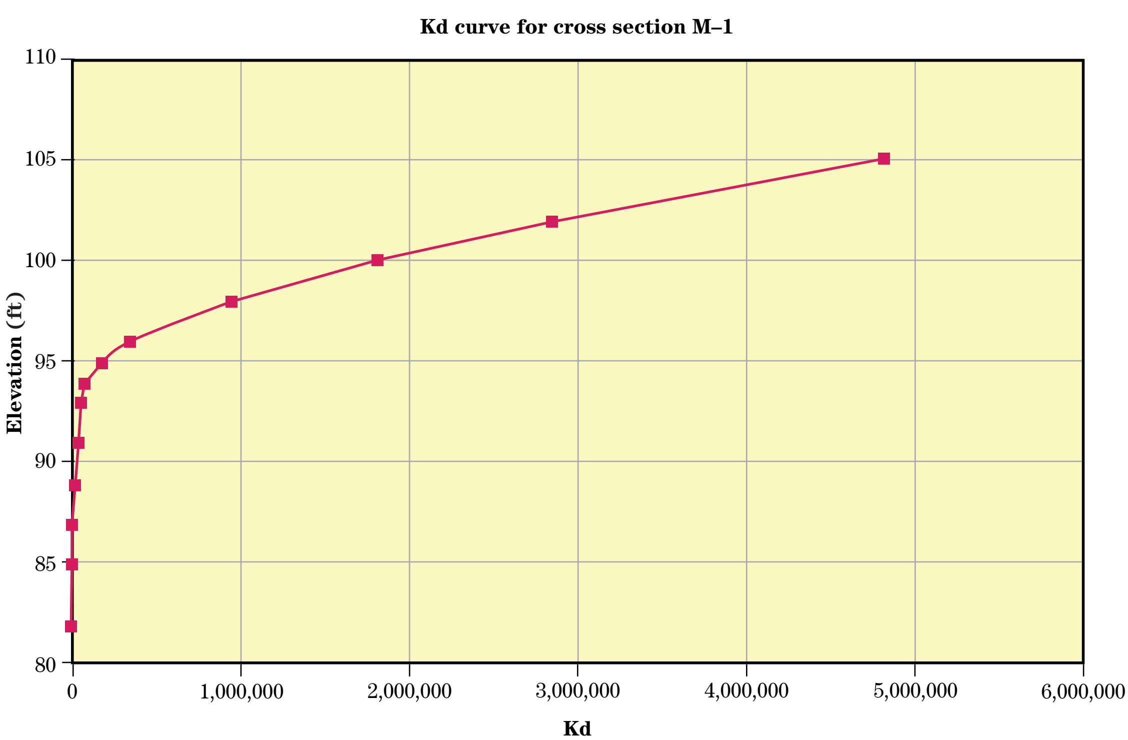


Table 14–3 Hydraulic parameters for starting cross section M–1, example 14–4

	n = 0.08 Segment 1			*n* = 0.045 Segment 2			*n* = 0.04 Segment 3			*n* = 0.045 Segment 4			Totals	
Elev. ft (1)	A ft^2 (2)	WP ft (3)	Kd (4)	A ft^2 (5)	WP ft (6)	Kd (7)	A ft^2 (8)	WP ft (9)	Kd (10)	A ft^2 (11)	WP ft (12)	Kd (13)	Kd (14)	Q ft^3/s (15)
82							0	0					0	0
85							87	35	5.93×10^{3}				5.93×10^{3}	188
87							155	41	1.40×10^{4}				1.40×10^{4}	442
89							231	46	2.52×10^{4}				2.52×10^{4}	796
91							315	52	3.89×10^{4}				3.89×10^{4}	1,230
93			0	0	0	0	407	58	5.54×10^{4}	0	0	0	5.54×10^{4}	1,753
94	0	0	0	93	925	6.64×10^{2}	456	62	6.41×10^{4}	378	1,050	6.32×10^{3}	7.10×10^{4}	2,247
95	150	300	1.76×10^{3}	1,018	925	3.58×10^{4}	506	62	7.63×10^{4}	1,543	1,275	5.79×10^{4}	1.72×10^{5}	5,428
96	487	375	1.08×10^{4}	1,943	925	1.05×10^{5}	556	62	8.92×10^{4}	2,833	1,300	1.57×10^{5}	3.62×10^{5}	11,500
98	1,322	460	4.96×10^{4}	3,793	925	3.21×10^{5}	656	62	1.17×10^{5}	5,523	1,380	4.60×10^{5}	9.48×10^{5}	30,000
100	2,272	490	1.17×10^{5}	5,643	925	6.22×10^{5}	756	62	1.49×10	8,325	1,400	9.02×10^{5}	1.79×10^{6}	56,600
102	3,272	510	2.10×10^{5}	7,493	925	9.98×10^{5}	856	62	1.83×10^{5}	11,175	1,440	1.45×10^{6}	2.84×10^{6}	89,700
105	4,862	550	3.86×10^{5}	10,268	925	1.69×10^{6}	1006	62	2.40×10^{5}	15,555	1,460	2.49×10^{6}	4.80×10^{6}	152,000

Figure 14–7 Conveyance values section M–1, example 14–4



Example 14–5 Develop stage discharge curve for meandering reach

Develop the stage discharge curve for cross section M–l (fig. 14–5) if M–l represents a reach having a channel length of 2,700 feet and a floodplain length of 2,000 feet. The energy gradient of the channel portion is 0.001 feet per foot.

Step 1 Compute the total floodplain conveyance Kd_f. Figure 14–6 shows that segments 1, 2, and 4 of section M–l are floodplain segments. Table 14–3 was used to develop the hydraulic parameters for section M–l for each segment. From table 14–3, add the Kd values for each elevation from columns 4, 7, and 13, and tabulate as Kd_f in column 2 of table 14–4.

Step 2 Determine the meander factor L_c/L_f. For the channel length of 2,700 feet and the floodplain length of 2,000 feet, the meander factor is:

$$\frac{2700}{2000} = 1.35$$

Step 3 Determine $(L_c/L_f)^{1/2}$.

$$(1.35)^{\frac{1}{2}} = 1.16$$

Step 4 Compute $(Kd_f)\,(L_c/L_f)^{1/2}$. For each elevation in column 1 of table 14–4, multiply column 2 by $(L_c/L_f)^{1/2}$ and, tabulate in column 3.

$$\left(4.56 \times 10^{6}\right)(1.16) = 5.29 \times 10^{6}$$

Step 5 Compute the channel conveyance Kd. From table 14–3, the channel is segment 3, and the conveyance has been calculated in column 10. Tabulate Kd_c in column 4 of table 14–4.

Step 6 Compute $Kd_c + (Kd_f)\,(L_c/L_f)^{1/2}$. From table 14–4, add columns 3 and 4, and tabulate in column 5.

Step 7 Compute the discharge for each elevation. Use $S_c = 0.001$ and equation 14–17. Multiply column 5 by $S_c^{1/2}$, and tabulate in column 6.

Figure 14–8 Stage discharge curve, section M–1, example 14–4

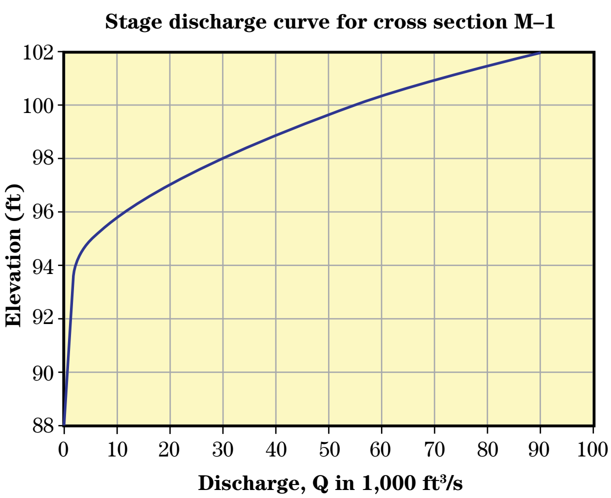


Table 14–4 Stage discharge for section M–1 with meander correction, example 14–5

Elevation ft	Floodplain Kd_f	$Kd_f(L_c/L_f)^{1/2}$	Channel Kd_c	Col 3 + Col 4	Q ft³/s
(1)	(2)	(3)	(4)	(5)	(6)
91	0	0	3.9×10^4	3.9×10^4	1.2×10^3
93	0	0	5.5×10^4	5.5×10^4	1.7×10^3
94	7.0×10^3	8.1×10^3	6.4×10^4	7.2×10^4	2.3×10^3
95	9.5×10^4	1.1×10^5	7.6×10^4	1.9×10^5	5.9×10^3
96	2.7×10^5	3.2×10^5	8.9×10^4	4.1×10^5	1.3×10^4
98	8.3×10^5	9.6×10^5	1.2×10^5	1.1×10^6	3.4×10^4
100	1.6×10^6	1.9×10^6	1.5×10^5	2.1×10^6	6.5×10^4
102	2.7×10^6	3.1×10^6	1.8×10^5	3.3×10^6	1.0×10^5
105	4.6×10^6	5.3×10^6	2.4×10^5	5.5×10^6	1.7×10^5

$$Q_t = \left[\left(Kd_c + \left(Kd_f\right)\left(\frac{L_c}{L_f}\right)^{\frac{1}{2}}\right)\right]\left(S_c\right)^{\frac{1}{2}}$$
$$= \left(5.54 \times 10^6\right)(0.0316)$$
$$= 1.75 \times 10^5$$
$$= 175{,}000 \text{ ft}^3/\text{s}$$

Example 14–6 shows the use of the modified step method in computing water surface profiles. It is a trial and error procedure based on estimating the elevation at the upstream section, determining the conveyance, Kd, for the estimated elevation, and computing $S_a^{1/2}$ by using Manning's equation in the form:

$$Q = Kd\left(S_a^{\frac{1}{2}}\right)$$

where:

$$Kd = \frac{1.486(A)\left(R^{\frac{2}{3}}\right)}{n}$$

S_a is the head loss per foot (neglecting velocity head) from the downstream to the upstream section. This head loss added to the downstream water surface elevation should equal the estimated upstream elevation.

Example 14–6 Compute water surface profiles and develop stage discharge curves for cross sections

Using the rating curve developed in example 14–4 for cross section M–l and parameters plotted on figures 14–9 and 14–10 for cross sections M–2 and T–l, compute the water surface profiles required to develop stage discharge curves for cross sections M–2 and T–l. The changes in velocity head will be ignored for these computations. The drainage area at section M–l is 400 square miles, at M–2 is 398 square miles, and at T–l is 48 square miles. The reach length between M–l and M–2 is 2,150 feet and between M–2 and T–l is 1,150 feet. Assume the meander factor for this example is 1.0.

Step 1 Determine the range of cubic feet per second per square mile needed to define the stage discharge curve. One or more of the cubic feet per second per square mile values selected should be contained within the channel. Tabulate in column 1, table 14–5(a).

Step 2 Compute the discharge in cubic feet per second for each cubic foot per second per square mile at the two cross sections M–l and M–2. At section M–l, the drainage area is 400 square miles. Using exhibit 14–l, the K factor is 1.0, and the cubic feet per second for 2 cubic feet per second per square mile is 2 × 400 × 1.0 = 800 cubic feet per second. At section M–2, the drainage is 398 square miles, and from exhibit 14–l, the K factor is 1.002. For 2 cubic feet per second per square mile, the discharge at M–2 is 2 × 398 × 1.002 = 798 cubic feet per second. Tabulate the discharges at M–l and M–2 on table 14–5(a), columns 2 and 3.

Step 3 Tabulate the reach length between the two cross sections in column 8. The reach length between section M–l and M–2 is 2,150 feet.

Step 4 Determine the water surface elevation at M–l. For the discharge listed in column 2, read the elevation from figure 14–8, and tabulate in column 4 of table 14–5(a).

Step 5 Assume a water surface elevation at section M–2. For the smallest discharge of 798 cubic feet per second, assume an elevation of 90.0 at M–2, and tabulate in column 5 of table 14–5(a).

Step 6 Determine Kd for assumed elevation. Read $Q_{M-2}/S_a^{1/2}$ or Kd_{M-2} of 3.70×10^4 at elevation 90.0 from figure 14–9 and tabulate in column 6 of table 14–5(a).

Step 7 Determine S_a.

$$S_a = \left(\frac{Q_{M-2}}{Kd_{M-2}}\right)^2$$

Divide column 3 by column 6, and square the results $(798/37{,}000)^2 = 0.00046$, and tabulate in column 7 of table 14–5(a).

Step 8 Determine $S_a \times L$. Multiply column 8 by column 7: 0.00046 × 2,150 = 0.99, and tabulate in column 9 of table 14–5(a).

Step 9 Compute elevation at M–2. Add column 9 (S_f) to column 4 (elevation at M–l), and tabulate in column 10 of table 14–5(a).

Figure 14–9 Conveyance values, section M–2, example 14–6

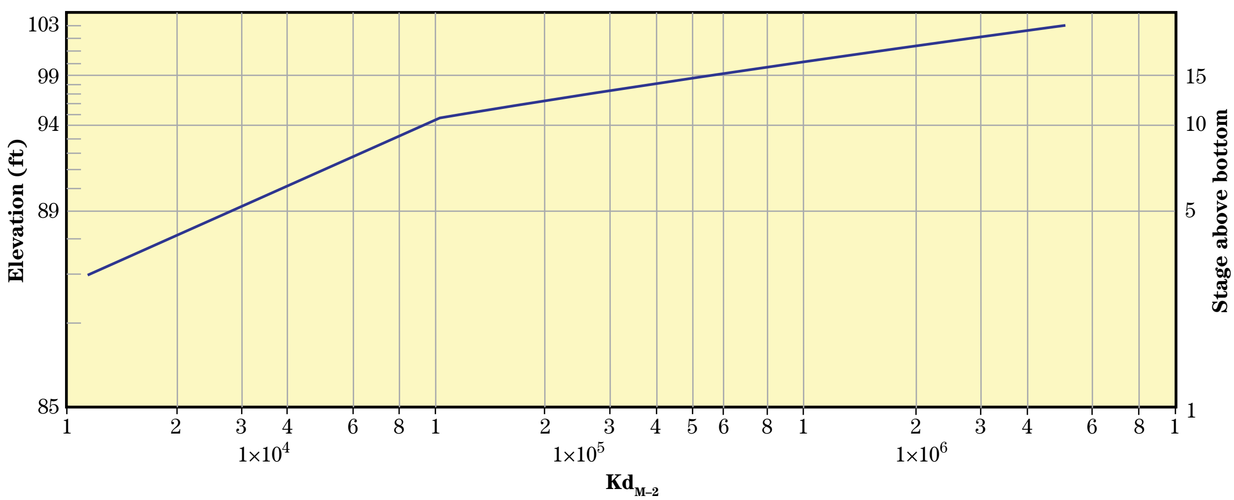


Figure 14–10 Conveyance values, section T–1, example 14–6

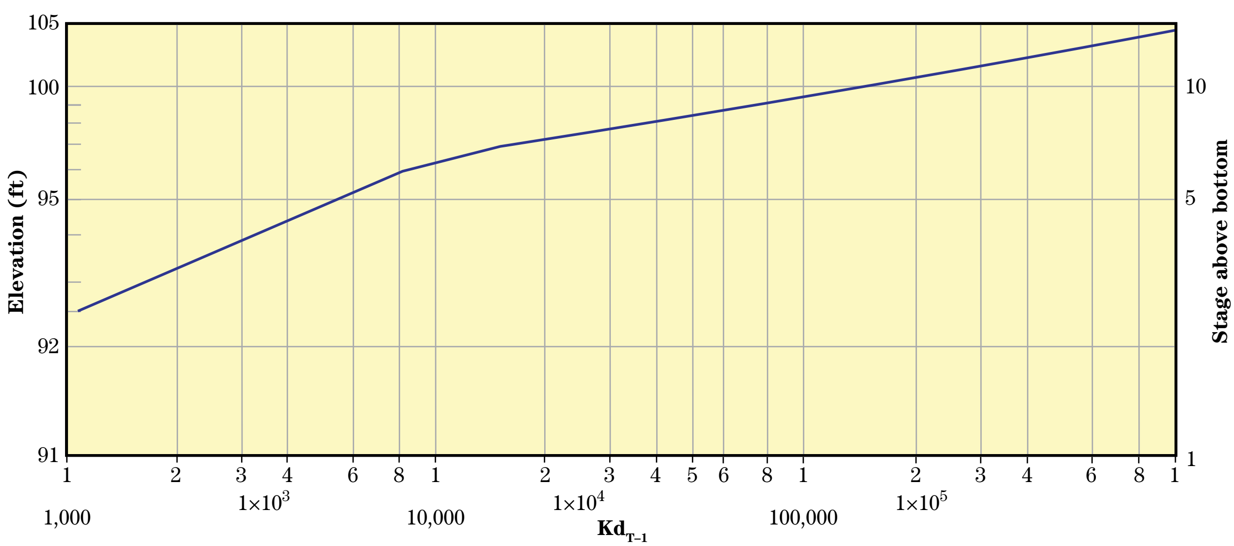


Table 14–5 Water surface profiles from cross sections, example 14–6

(a) M–1 to M–2

Discharge per mi² ft³/s/mi² (1)	Q_{M-1}, ft³/s (2)	Q_{M-2}, ft³/s (3)	Elevation at M–1 ft (4)	Assumed elevation at M–2 ft (5)	Kd_{M-2} (6)	$(Q_{M-2}/Kd_{M-2})^2=S_a$ (7)	L ft (8)	$S_f=L\times S_a$ ft (9)	Col 4 + col 9 estimated elevation at M–2 ft (10)	Computed elevation at M–2 ft (11)
2	800	798	89.02	90.00	37,000	0.00046	2,150	0.99	90.01	90
10	4,000	3,990	94.55	95.30	140,000	0.00081	2,150	1.74	96.29	
		3,990	94.55	95.70	170,000	0.00055	2,150	1.18	95.73	95.7
20	8,000	7,980	95.42	97.40	330,000	0.00058	2,150	1.25	96.67	
		7,980	95.42	97.10	290,000	0.00076	2,150	1.63	97.05	97.1
50	20,000	19,940	96.92	99.50	740,000	0.00073	2,150	1.57	98.49	
		19,940	96.92	99.10	650,000	0.00094	2,150	2.02	98.94	
		19,940	96.92	99.10	640,000	0.00097	2,150	2.09	99.01	99
100	40,000	39,900	98.75	101.10	1,400,000	0.00081	2,150	1.74	100.49	
		39,900	98.75	100.90	1,300,000	0.00094	2,150	2.02	100.77	
		39,900	98.75	100.80	1,280,000	0.00097	2,150	2.09	100.84	100.8
200	80,000	79,800	101.41	103.20	3,200,000	0.00062	2,150	1.33	102.74	
		79,800	101.41	103.00	3,000,000	0.00071	2,150	1.53	102.94	102.9

Table 14–5 Water surface profiles from cross sections, example 14–6—Continued

(b) M–2 to T–1

Discharge per mi² ft³/s/mi² (1)	Q_{M-2}, ft³/s (2)	Q_{T-1}, ft³/s (3)	Elevation at M–2 ft (4)	Assumed elevation at T–1 ft (5)	Kd_{T-1} (6)	$(Q_{T-1}/Kd_{T-1})^2=S_a$ (7)	L ft (8)	$S_f=S_a \times L$ ft (9)	Col 4 + col 9 estimated elevation at T–1 ft (10)	Computed elevation at T–1 ft (11)
2	798	260	90.0	93.74	1,700	0.02339	1,150	26.90	116.9	
		260	90.0	94.71	3,400	0.00585	1,150	6.72	96.7	
		260	90.0	95.07	4,400	0.00349	1,150	4.02	94.0	
		260	90.0	94.93	4,000	0.00422	1,150	4.86	94.9	94.9
10	3,990	1,290	95.7	97.06	16,500	0.00611	1,150	7.03	102.7	
		1,290	95.7	98.36	35,000	0.00136	1,150	1.56	97.3	
		1,290	95.7	98.04	29,000	0.00198	1,150	2.28	98.0	98.0
20	7,980	2,580	97.1	99.20	57,000	0.00205	1,150	2.36	99.5	
		2,580	97.1	99.29	60,000	0.00185	1,150	2.13	99.2	99.2
50	19,950	6,450	99.0	101.62	232,000	0.00077	1,150	0.89	99.9	
		6,450	99.0	101.23	185,000	0.00122	1,150	1.40	100.4	
		6,450	99.0	100.98	160,000	0.00163	1,150	1.87	100.9	100.9
100	39,900	12,900	100.8	102.61	410,000	0.00099	1,150	1.14	101.9	
		12,900	100.8	102.47	380,000	0.00115	1,150	1.33	102.1	
		12,900	100.8	102.38	360,000	0.00128	1,150	1.48	102.3	102.3
200	79,800	25,800	102.9	103.88	860,000	0.00090	1,150	1.04	103.9	103.9

Step 10 Compare computed elevation with assumed elevation. Compare column 10 with column 5, and adjust column 5 up if column 10 is greater and down if it is less. For 2 cubic feet per second per square mile discharge, the computed elevation is 90.01, and the estimated elevation is 90.0. Since column 10 is very close in the value to column 5, a revision to the estimated elevation at M–2 in column 5 is not needed.

If the difference between elevations on columns 10 and 5 are more than 0.1 foot, repeat steps 5 through 10 until a reasonable balance between columns 10 and 5 is obtained. A tolerance of 0.1 foot was used in this example.

Step 11 Repeat steps 5 through 10 for each cubic feet per second per square mile value selected.

Step 12 Plot stage discharge curve, columns 3 and 11 as shown on figure 14–11.

Table 14–5(b) shows computations similar to step 1 through step 11 computing water surface profiles between cross section M–2 on the main stem and T–l, the first cross section on a tributary. Discharge values at T–1 are based on exhibit 14–1. Use of this exhibit is demonstrated in example 14–3 and in step 3 of example 14–6. Kd values are shown on figure 14–10. Figure 14–12 was plotted from table 14–5(b).

630.1405 Road crossings

(a) Bridges

In developing the hydraulics of natural streams, bridges of all types and sizes are encountered. These bridges may or may not have a significant effect on the stage discharge relation in the reach above the bridge. Many of the older bridges were designed without regard to their effect on flooding in the reach upstream from the road crossing.

The U.S. Department of Transportation (DOT) Federal Highway Administration (FHWA), formerly known as the Bureau of Public Roads, initiated a research project with Colorado State University in 1954 that culminated in the investigation of several features of the bridge problem. Included in these investigations was a study of bridge backwater. Laboratory studies using hydraulic models as the principal research tools were completed and considerable progress made in the collection of field data by the USGS to substantiate the model results and extend the range of application. The procedure developed is explained in the publication entitled "Hydraulics of Bridge Waterways" (DOT FHWA 1978). This is one method that is recommended by the NRCS for use in computing effects of bridges in natural channels and floodplains.

The FHWA method has been formulated by applying the principle of conservation of energy or momentum between the point of maximum backwater upstream

Figure 14–11 Stage discharge, section M–2, example 14–6

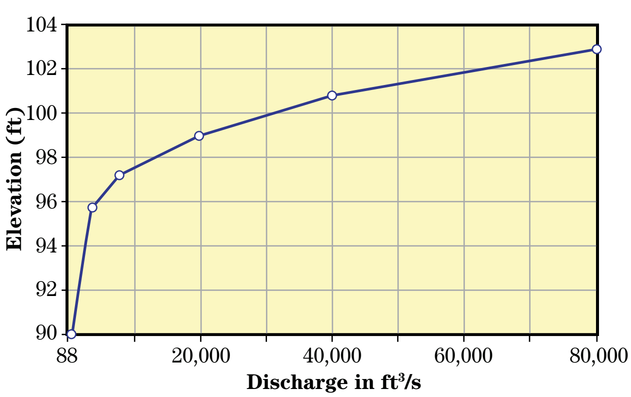
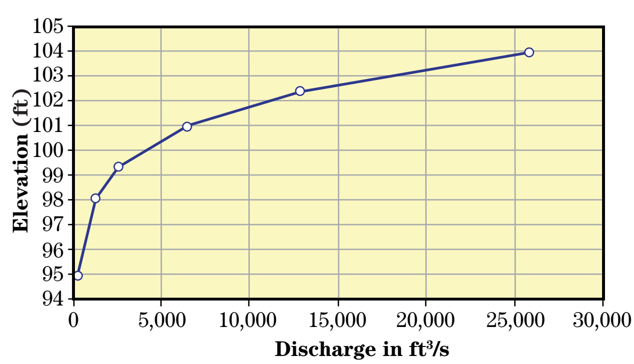


Figure 14–12 Stage discharge, section T–1, example 14–6



from the bridge and a point downstream from the bridge at which normal stage has been reestablished. The general expression for the computation of backwater upstream from a bridge constricting the flow is:

$$h_1^* = K^* \frac{\alpha_2 V_{n2}^2}{2g} + \left(\frac{\alpha_4 V_4^2}{2g} - \frac{\alpha_1 V_1^2}{2g} \right) \qquad \text{(eq. 14–20)}$$

where:
h_1^* = total backwater, ft
K^* = total backwater coefficient
$\alpha_1, \alpha_2, \alpha_4$ = velocity head energy coefficients at the upstream, constriction, and downstream section
V_{n2} = average velocity in constriction or Q/A, ft/s
V_1 = average velocity at section 1 upstream, ft/s
V_4 = average velocity at section 4 downstream, ft/s

For a more detailed explanation of each term and the development of the equation, refer to DOT FHWA (1978).

Equation l4–20 is reasonably valid if the channel in the vicinity of the bridge is essentially straight, the cross-sectional area of the stream is fairly uniform, the gradient of the bottom is approximately constant between sections 1 and 4, the flow is free to expand and contract, there is no appreciable scour of the bed in the constriction, and the flow is in the subcritical range.

This procedure relates the total backwater effect to the velocity head caused by the constriction times the total backwater coefficient. The total backwater coefficient is comprised of the effect of constriction as measured by the bridge opening coefficient, M; type of bridge abutments; size, shape, and orientation of piers; and eccentricity and skew of the bridge.

For a detailed description of the backwater coefficient and the effect of constriction, abutments, piers, eccentricity, and skew of bridges, refer to DOT FHWA (1978).

A preliminary analysis may be made to determine the maximum backwater effect, of a bridge. If the analysis shows a significant bridge effect then a more detailed procedure should be used. If the analysis shows only a minor effect, then the bridge may be eliminated from the backwater computation.

The examples shown in this chapter are based on the approximate equation to compute bridge head losses taken from the DOT FHWA (1978) report:

$$h_1^* = K^* \frac{V^2}{2g} \qquad \text{(eq. 14–21)}$$

where:
h_1^* = total backwater, ft
K^* = total backwater coefficient
V = average velocity in constriction Q/A, ft/s
A = gross water area in constriction measured below normal stage, ft^2

The following data are the minimum needed for estimating the maximum backwater effect of a bridge using equation 14–21.

- total area of bridge opening
- length (span) of bridge opening; span being defined as the distance or space between supports of a bridge
- cross section upstream from the bridge at a distance approximately equal to the length of the bridge opening, also called the approach section (DOT FHWA 1978)
- area of approach section at elevation of the bottom of bridge stringers or at the low point in the road embankment (DOT FHWA 1978)
- width of floodplain in approach section
- estimate of the velocity of unrestricted flow at the elevation of the bottom of the bridge stringers or at the low point in the road embankment

A preliminary analysis to determine an estimate of the maximum backwater effect of a bridge is shown in example 14–7. Exhibits 14–2 and 14–3 were developed only for use in making preliminary estimates and should not be used in a more detailed analysis.

Example 14–7 Bridge backwater effect

Estimate the backwater effect of a bridge with 45-degree wingwalls given the following data:

area of bridge = 4,100 ft^2

length (span) of bridge = 400 ft

area of approach = 11,850 ft^2

width of floodplain = 2,650 ft

estimated velocity in the natural stream = 2.5 ft/s

Step 1 Compute the ratio of the area of the bridge to the area of approach section. From the given data: 4,100/11,850 = 0.346

Step 2 Compute the ratio of length of bridge to the width of the floodplain. From the given data: 400/2,650 = 0.151

Step 3 Determine the change in velocity head. Using the results of step 1 (0.346) and the estimated velocity in the natural stream (2.5 ft/s), read the velocity head h from exhibit 14–2. This is the velocity head, $V^2/2g$, in equation 14–21 and (from exhibit 14–2), it equals 0.8 feet.

Step 4 Estimate the constriction ratio, M. Using the results from step 1 (0.346) and step 2 (0.151), read M = 0.67 from exhibit 14–3.

Step 5 Estimate the total backwater coefficient. Using M = 0.67 from step 4, read from exhibit 14–4 curve 1, Kb = 0.6. Kb is the base curve backwater coefficient, and for estimating purposes, is considered to be the total backwater coefficient, K^*, in equation 14–21.

Step 6 Compute the estimated total change in water surface, h_1^*. From equation 14–21 the total change in water surface is:

$$
\begin{aligned}
h_1^* &= K^* \frac{V^2}{2g} \\
&= (0.6)(0.8) \\
&= 0.48 \text{ ft}
\end{aligned}
$$

If the estimate shows a change in water surface that would have an appreciable effect on the evaluation or level of protection of a plan or the design and construction of proposed structural measures, a more detailed survey and calculation should be made for the bridge and flood in question.

Example 14–8 shows a more detailed solution to the backwater loss using equation 14–21. To use the FHWA method, it is necessary to develop stage discharge curves for an exit and an approach section assuming no constriction between the two cross sections. The exit section should be located downstream from the bridge a distance approximately twice the length of the bridge. The approach section should be located upstream from the upper edge of the bridge a distance approximately equal to the length of the bridge.

Based on the BPR manual, HEC–RAS (2010d) has different recommendations.

If the elevation difference between the water surface at the exit section and the approach section prior to computing head loss is relatively small, the bridge tailwater may be taken as the elevation of the exit section and the bridge head loss simply added to the water elevation of the approach section. However, if this difference is not small, the bridge tailwater should be computed by interpolation of the water elevation at the approach section and exit section and the friction loss from the bridge to the approach section recomputed after the bridge headwater is obtained.

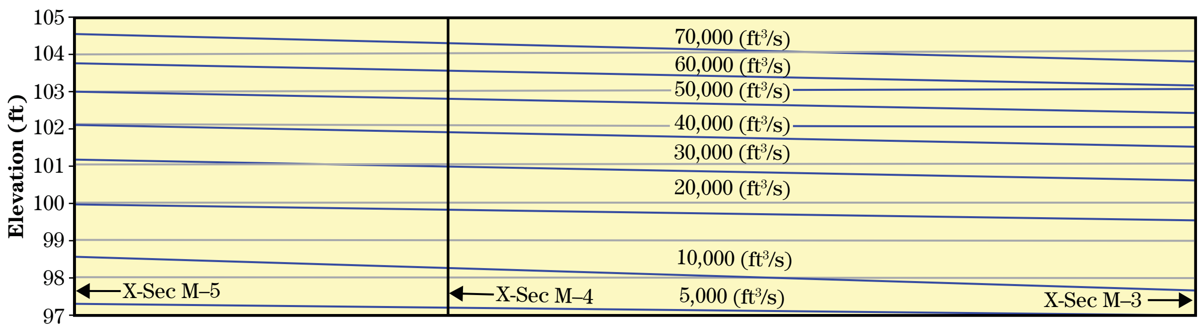
In example 14–8, it is assumed that all preliminary calculations have been made. The profiles are shown on figure 14–13(a), and the stage discharge curve for cross section M–5 is shown on figure 14–14, natural condition without constriction.

Example 14–8 Stage discharge curves for bridge alternatives

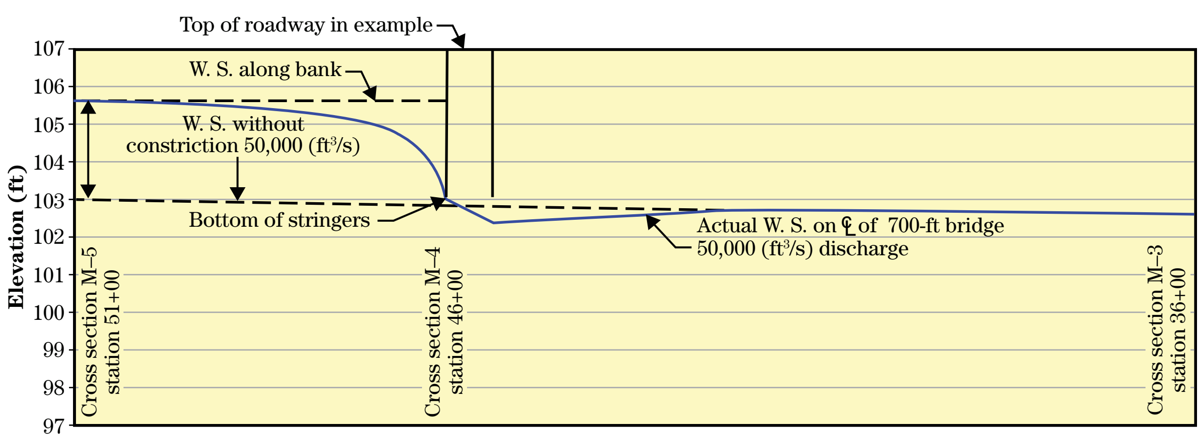
Develop stage discharge curves for each of four bridge span alternatives located at cross section M–4 (fig. 14–5), 300, 400, 500, and 700 feet long (fig. 14–13(c)) with 45-degree wingwalls. The elevation of the bottom of the bridge stringer is 103 feet for each trial bridge length. The span over the channel is 100 feet with the remaining portion of the bridge supported by 24-inch H piers placed 25 feet on center (o. c.). Assume the fill is sufficiently high to prevent overtopping for the maximum discharge (70,000 ft^3/s) studied. It is assumed that water surface profiles have been run for present conditions through section M–5 and that this

Figure 14–13 Water surface profiles, example 14–8

(a) Water surface profiles without constriction, example 14–8



(b) Water surface profile with constriction, example 14–8



(c) Cross section of road at section M–4, example 14–8

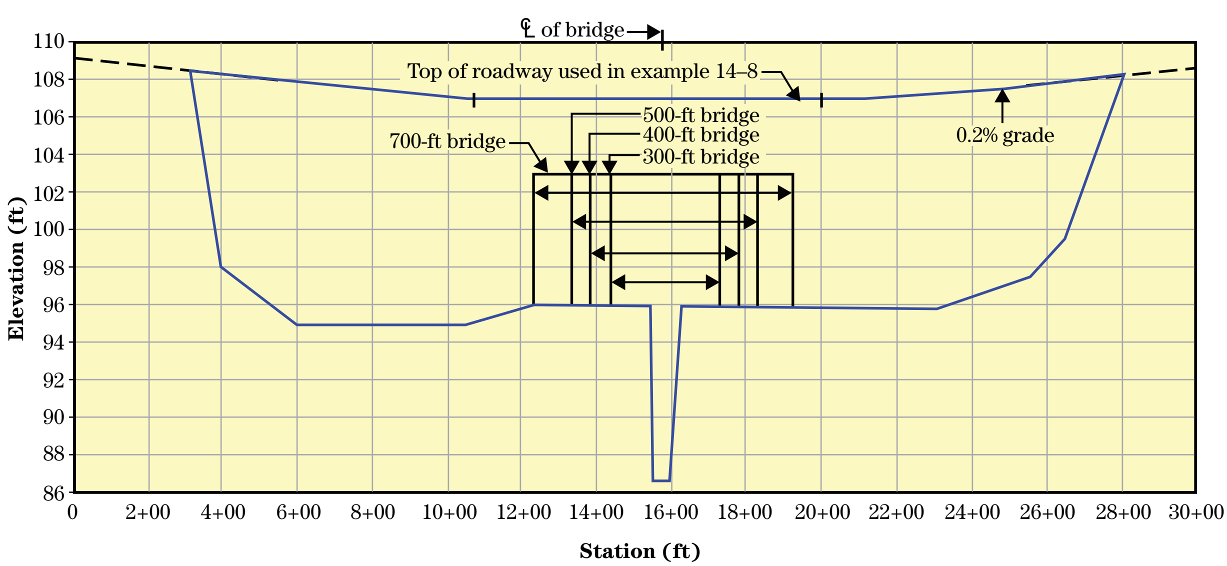
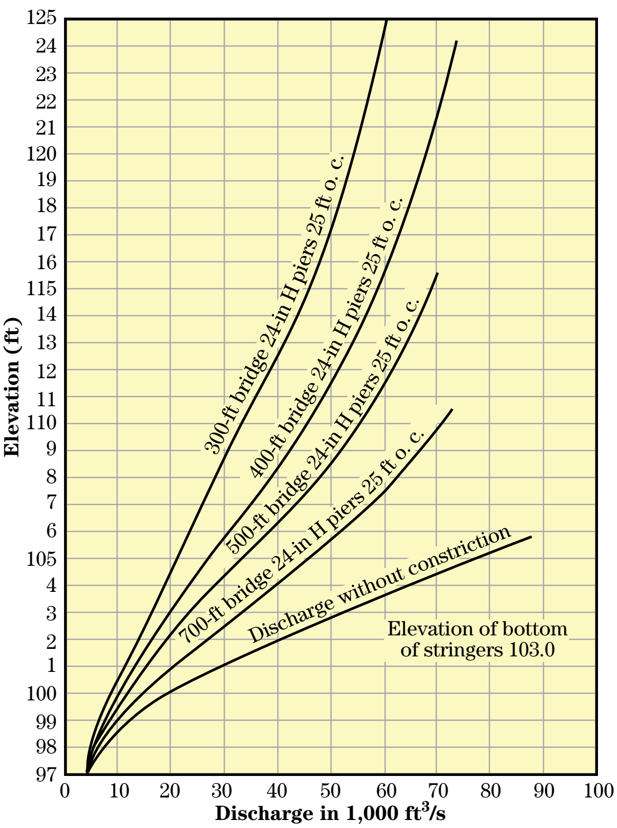


Figure 14–14 Stage discharge without embankment overflow, section M–5, example 14–8



information is available for use in analyzing the effect of bridge losses.

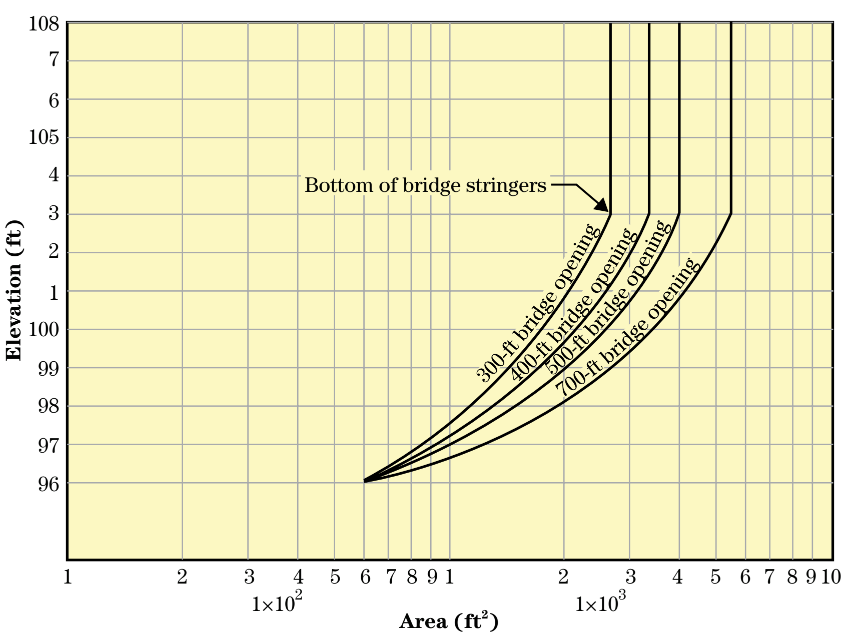
Step 1 Select a range of discharges that will define the rating curve. For this problem, select a range of discharges from 5,000 to 70,000 cubic feet per second for each bridge length, and tabulate in column 1 of table 14–6.

Step 2 Determine the present condition elevation for each discharge at the bridge section M–4. For this example water surface profiles have been computed from section M–3 to M–5 without the bridge in place. The results are plotted in figure 14–13(a). From figure 14–13(a), read the normal elevation without constriction for each discharge at cross section M–4, and tabulate in column 2 of table 14–6.

Step 3 Compute the elevation versus gross bridge opening area. The gross area of the bridge is the total area of the bridge opening at a given elevation without regard to the area of piers. The channel area is 600 square feet, and for the 300-foot-long bridge, the gross bridge area is plotted in figure 14–15.

Plot the elevation versus gross bridge opening area as shown in figure 14–15.

Figure 14–15 Bridge opening areas, example 14–8



Step 4 Determine the gross area of the bridge opening at each water surface elevation. Using figure 14–15, read the gross area at each elevation tabulated in column 2, and tabulate in column 3 of table 14–6.

Step 5 Compute the average velocity through the bridge opening.

Divide column 1 by column 3, and tabulate in column 4 of table 14–6. For the 300-foot-long bridge:

$$\begin{aligned} V &= \frac{Q}{A} \\ &= \frac{5{,}000}{885} \\ &= 5.65 \text{ ft/s} \end{aligned}$$

Step 6 Compute the velocity head $V^2/2g$. Using the velocities from column 4, compute the velocity head for each discharge, and tabulate in column 11 of table 14–6. For a discharge of 5,000 cubic feet per second and a bridge length of 300 feet, the velocity head is:

$$\frac{(5.65)^2}{(2)(32.16)} = 0.496$$

Step 7 Determine the elevation for each discharge at section M–5 under natural conditions. Using figure 14–13(a) or figure 14–14 (natural condition without constriction curve), read the elevation for each discharge at cross section M–5, and tabulate in column 5 of table 14–6.

Step 8 Compute M versus elevation for each bridge size. M is computed as outlined in Hydraulics of Bridge Waterways, (DOT FHWA 1978). It is computed as the ratio of that portion of the discharge at the upstream section computed for a width equal to the length of the bridge to the total discharge of the channel system. If Q_b is the discharge at the upstream section computed for a floodplain or channel width equal to the length of the bridge, and Q_a and Q_c are the remaining discharges on either side of $Q_{b,}$ then:

$$\begin{aligned} M &= \frac{Q_b}{Q_a + Q_b + Q_c} \\ &= \frac{Q_b}{Q} \end{aligned}$$

The bridge opening ratio, M, is most easily explained in terms of discharges, but it is usually determined from conveyance relations. Since conveyance (Kd) is proportional to discharge, assuming all subsections to have the same slope, M can be expressed also as:

$$\begin{aligned} M &= \frac{Kd_b}{Kd_a + Kd_b + Kd_c} \\ &= \frac{Kd_b}{Kd_t} \end{aligned}$$

The approach section information is not shown for this example.

Plot M versus elevation for each bridge size as shown in figure 14–16.

Step 9 Read M for each elevation. Using figure 14–16 prepared in step 8, read M for each elevation in column 2, and tabulate in column 6 of table 14–6.

Step 10 Determine the base backwater coefficient, K_b. Using M from step 9, read K_b from exhibit 14–4 for bridges having 45-degree wingwalls, and tabulate in column 7 of table 14–6.

Step 11 Compute the area of pier/area of bridge versus elevation.

$$\frac{\text{area of piers}}{\text{gross area of bridge opening}} = \frac{A_p}{A_{n2}} = J$$

For the 300-foot-long bridge, the piers are located in an area 200 feet wide. (300 ft – 100 ft clear span = 200 ft). The piers are on 25-foot centers and are 2 feet wide. Within the 200-foot width, the piers will occupy:

$$\frac{(200)(2)}{(25)} = 16 \text{ ft}$$

At an elevation of 103, the piers will occupy an area 25 feet wide by 7 feet deep (103 – 96 = 7 ft). From figure 14–15, the gross area of the bridge opening is 2,700 square feet.

Then:
$$\begin{aligned} \frac{A_p}{A_{n2}} &= \frac{(16)(7)}{2{,}700} \\ &= 0.041 \end{aligned}$$

Table 14–6 Backwater computations through bridges, example 14–8

Bridge span, ft (see note 2)	Q in 1000 ft³/s (1)	Normal el. @ x-sec M–4 ft (2)	Bridge opening area A_{n2} ft² (3)	Velocity through bridge opening, ft/s (4)	Normal el. @ x-sec M–5 ft (5)	M 1/ (6)	K_b 1/ (7)	J 1/ (8)	ΔK_p 1/ (9)	K* 1/ (10)	$\frac{V_{n2}^2}{2g}$ 1/ ft (11)	h_1^* ft (12)	Elev. with 24-in piers; 25 ft on centers ft (13)
300	5	97.20	885	5.65	97.35	0.470	1.24	0.020	0.06	1.30	0.496	0.64	97.99
	10	98.25	1280	7.81	98.55	0.350	1.74	0.028	0.08	1.82	0.947	1.72	100.27
	20	99.82	1720	11.63	100.00	0.276	2.08	0.035	0.08	2.16	2.10	4.54	104.54
	30	100.95	2060	14.56	101.15	0.243	2.24	0.038	0.08	2.32	3.29	7.63	108.78
	40	101.90	2340	17.09	102.10	0.222	2.34	0.040	0.08	2.42	4.54	10.99	113.09
	50	102.80	2600	19.23	103.00	0.208	2.41	0.041	0.08	2.49	5.74	14.29	117.29
	60	103.55	2660	22.56	103.75	0.183	2.54	0.042	0.08	2.62	7.90	20.70	124.45
	70	104.25	2660	26.32	104.50	0.160	2.66	0.042	0.07	2.73	10.76	29.37	133.87
400	5	97.20	1030	4.85	97.35	0.510	1.09	0.027	0.10	1.19	0.365	0.43	97.78
	10	98.25	1470	6.80	98.55	0.385	1.59	0.036	0.12	1.71	0.718	1.23	99.78
	20	99.82	2070	9.66	100.00	0.315	1.90	0.043	0.12	2.02	1.45	2.93	102.93
	30	100.95	2540	11.81	101.15	0.282	2.05	0.046	0.12	2.17	2.17	4.71	105.86
	40	101.90	2950	13.56	102.10	0.265	2.13	0.048	0.12	2.25	2.86	6.44	108.54
	50	102.80	3300	15.15	103.00	0.250	2.21	0.049	0.12	2.33	3.56	8.29	111.29
	60	103.55	3380	17.75	103.75	0.220	2.35	0.049	0.11	2.46	4.89	12.03	115.78
	70	104.25	3380	20.71	104.50	0.192	2.49	0.049	0.10	2.59	6.66	17.25	121.75
500	5	97.20	1160	4.31	97.35	0.525	1.03	0.032	0.13	1.16	0.288	0.33	97.68
	10	98.25	1670	5.99	98.55	0.420	1.44	0.042	0.15	1.59	0.557	0.89	99.44
	20	99.82	2550	7.84	100.00	0.350	1.74	0.049	0.16	1.90	0.954	1.81	101.81
	30	100.95	3050	9.84	101.15	0.325	1.85	0.052	0.16	2.01	1.50	3.02	104.17
	40	101.90	3520	11.36	102.10	0.310	1.92	0.054	0.16	2.08	2.00	4.16	106.26
	50	102.80	3950	12.66	103.00	0.298	1.98	0.055	0.16	2.14	2.49	5.33	108.33
	60	103.55	4050	14.81	103.75	0.262	2.15	0.055	0.14	2.29	3.41	7.81	111.56
	70	104.25	4050	17.28	104.50	0.230	2.30	0.055	0.13	2.43	4.64	11.28	115.78
700	5	97.20	1420	3.52	97.35	0.580	0.84	0.040	0.19	1.03	0.192	0.20	97.55
	10	98.25	2170	4.61	98.55	0.480	1.20	0.050	0.21	1.41	0.330	0.47	99.02
	20	99.82	3300	6.06	100.00	0.415	1.46	0.056	0.21	1.67	0.570	0.95	100.95
	30	100.95	4080	7.35	101.15	0.394	1.55	0.058	0.21	1.76	0.839	1.48	102.63
	40	101.90	4750	8.42	102.10	0.377	1.62	0.059	0.21	1.83	1.10	2.01	104.11
	50	102.80	5380	9.29	103.00	0.367	1.67	0.060	0.20	1.87	1.34	2.51	105.51
	60	103.55	5520	10.87	103.75	0.325	1.85	0.061	0.19	2.04	1.83	3.73	107.48
	70	104.25	5520	12.68	104.50	0.285	2.04	0.061	0.17	2.21	2.50	5.53	110.03

Notes:
1/ Letters and symbols are the same as used in Hydraulics of Bridge Waterways, U.S. Dept. of Transportation, Bureau of Public Roads (1978)
2/ 45-degree wingwall abutments are assumed for all four bridge span trials

Compute and plot $A_p/A_{n2} = J$ versus elevation for each bridge length as shown in figure 14–17.

Step 12 Determine J for each elevation. Read J from figure 14–17 for each elevation in column 2, and tabulate in column 8 of table 14–6.

Step 13 Determine the incremental backwater coefficient ΔK_p. Using J from step 12, read ΔK from the appropriate curve (for this example curve 1) from exhibit 14–5(a). Using M from step 9, read σ from the appropriate curve (curve l) from exhibit 14–5(b). Multiply ΔK by σ, and tabulate as ΔK_p in column 9 of table 14–6.

For 5,000 cubic feet per second and a 300-foot-long bridge:

$$\Delta K = 0.105 \qquad \sigma = 0.59$$

$$\begin{aligned} \Delta K_p &= \Delta K \sigma \\ &= (0.105)(0.59) \\ &= 0.06 \end{aligned}$$

Step 14 Determine the total backwater coefficient, K^*. Add columns 7 and 9, and tabulate as K^* in column 10. This is the total backwater coefficient for the bridge that will be considered for this example. If there are other losses that appear to be significant, the user should follow the procedure shown in the FHWA report (1978) for computing their effects.

Step 15 Determine the total change in water surface h_1^*. Multiply column 10 by column 11, and tabulate in column 12. From equation 14–21:

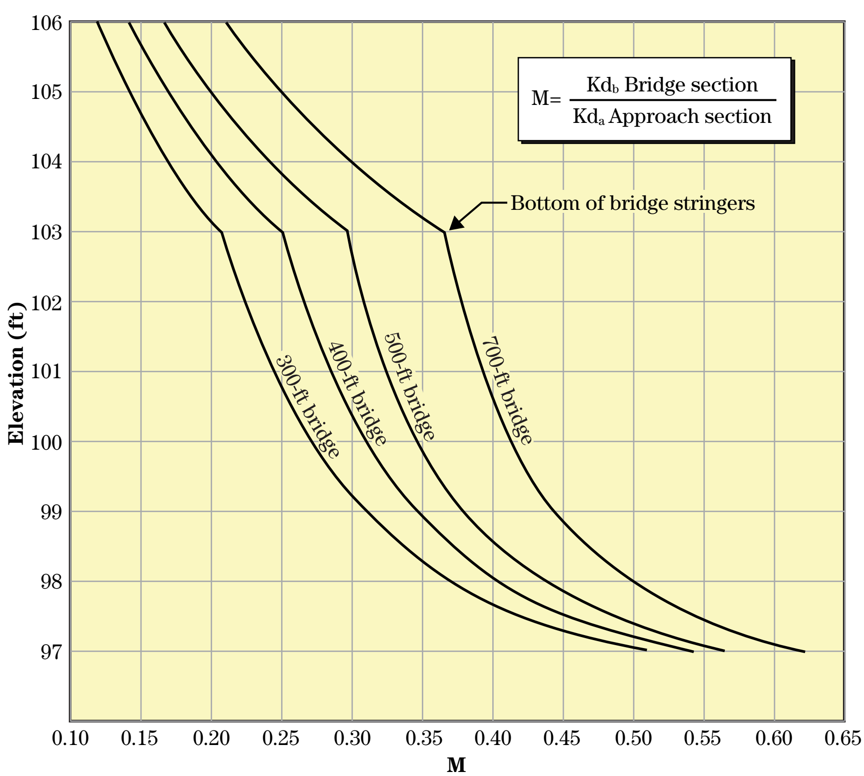
$$h_1^* = K^* \frac{V^2}{2g}$$

For 5,000 cubic feet per second and a 300-foot-long bridge with piers:

$$\begin{aligned} h_1^* &= (1.30)(0.496) \\ &= 0.64 \text{ ft} \end{aligned}$$

If the example did not include piers or if the effect of eliminating the piers are desired, the h_1^* could be determined by multiplying column 7 by column 11.

Figure 14–16 M values for bridge, example 14–8



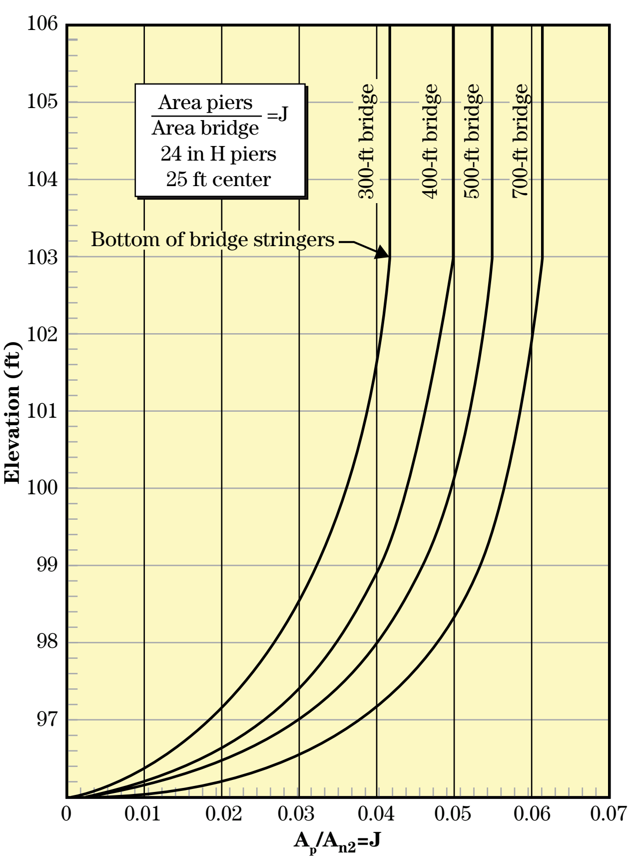
For 5,000 cubic feet per second and a 300-foot-long bridge without piers:

$$h_1^* = (1.24)(0.496)$$
$$= 0.62 \text{ ft}$$

Step 16 Determine the elevation with bridge losses. Add column 5 and column 12, and tabulate in column 13. Column 13 is plotted on figure 14–14, which shows the stage discharge curve for cross section M–5, assuming the fill to be high enough to force all of the 70,000 cubic feet per second discharge through the bridge opening.

The water surface at the bridge with a 700-foot opening at 50,000 cubic feet per second is plotted on figure 14–13(b).

Figure 14–17 J values for bridge, example 14–8



(b) Full bridge flow

The analysis of flood flows past existing bridges involves flows that submerge all or a part of the bridge girders. When this condition occurs, the computation of the head loss through the bridge must allow for the losses imposed by the girders. This may be accomplished in several ways.

One method is to continue using the FHWA report (1978), but hold the bridge flow area and Kd constant for all elevations above the bridge girder. Example 14–8 uses this procedure (fig. 14–15).

Another approach commonly taken is to compute the flow through the bridge opening by the orifice flow equation.

$$Q = CA\sqrt{2g\Delta h} \qquad \text{(eq. 14–22)}$$

where:
Q = discharge in ft³/s
Δh = the difference in water surface elevation between headwater and tailwater in ft
A = flow area of bridge opening in ft²
g = acceleration of gravity, ft/s²
C = coefficient of discharge

In estimating C, if conditions are such that flow approaches the bridge opening with relatively low turbulence, the appropriate value of C is about 0.90. In the majority of cases, C probably is in the 0.70 to 0.90 range. For very poor conditions (much turbulence), it may be as low as 0.40 to 0.50. In judging a given case, consider the following:

- whether the abutments are square-cornered or shaped to reduce turbulence
- number and shape of piers
- degree of skew
- number and spacing of piles and the pile bents that cap them since closely spaced piles and bents increase turbulence
- existence of trees, drift, or other types of obstruction at the bridge or in the approach reach

Using a C value of 0.8 has given approximately the same results as the method for example 14–7; however, the C value varied with discharge.

(c) Overtopping of bridge embankment

When the fill of a bridge is overtopped, the total discharge at the bridge section is equal to the discharge through the bridge opening plus the discharge over the embankment. A reliable estimate of the effect of the bridge constriction on stages upstream under these conditions is difficult to obtain.

A generally accepted procedure to use in analyzing flows over embankments is to consider the embankment as acting as a broad crested weir. The broad crested weir equation is:

$$Q = CLh_e^{\frac{3}{2}} \qquad \text{(eq. 14–23)}$$

where:
L = length of weir, ft
h_e = energy head, which is comprised of the velocity head at the upstream section plus the depth of flow over the weir, ft
C = coefficient
Q = discharge, ft^3/s

The following approximate ranges of C values for flows over embankments are recommended for use in equation 14–23. For road and highway fills, C = 2.5 to 2.8; for single-track railroad fills, C = 2.2 to 2.5; for double-track railroad fills, C = 1.9 to 2.2. The weir coefficient for double-track railroad fills is less than the weir coefficient for single-track railroad fills because the double-track railroad fills create more turbulence and energy loss.

Equation 14–23 was developed for use in rectangular weir sections. Since road profiles encountered in the field seldom represent rectangular sections, it becomes difficult to determine the weir length to use. Many approaches have been formulated to approximate this length.

A method suggested for use in this chapter substitutes the flow area A for the weir length and flow depth over the weir in equation 14–23.

$$Q = C' Ah^{\frac{1}{2}} \qquad \text{(eq. 14–24)}$$

where:
A = flow area over the embankment at a given depth, h, in ft^2
h = flow depth measured from the low point on the embankment in ft
C´ = coefficient, which accounts for the velocity of approach
Q = discharge in ft^3/s

The coefficient C´ can be computed by equating equations 14–23 and 14–24 and solving for C´.

$$C' = C\frac{1}{\left(\frac{\text{depth}}{\text{depth + velocity head}}\right)^{\frac{3}{2}}} \qquad \text{(eq. 14–25)}$$

In equation 14–25, the depth is measured from the low point on the embankment of the bridge section, and the velocity head is computed at the upstream section for the same elevation as water is flowing over the embankment. The approach velocity may be approximated by V = Q/A, where Q is the total discharge, and A is the total flow area at the upstream section for the given elevation. In cases where the approach velocity is sufficiently small, C´ will equal C, and no correction for velocity head will be needed to use equation 14–24.

The free discharge over the road computed using equation 14–24 must be modified when the tailwater elevation downstream is great enough to submerge the embankment of the bridge section. The modification to the free discharge, Q_f, is made by computing a submergence ratio, H_2/H_1, where H_2 and H_1 are the depths of water downstream and upstream, respectively, above the low point on the embankment. A submergence factor, R, is read from figure 3–4, USDA SCS (1986), and the submerged discharge is computed as $Q_s = RQ_f$. Then the total discharge at the bridge section is equal to the discharge through the bridge opening plus the submerged discharge over the embankment.

Example 14–9 shows the use of equations 14–24 and 14–25 in computing flows over embankments using a trial and error procedure to determine C´.

Example 14–9 Stage discharge curve for overflow section of highway bridge

Develop a stage discharge curve for the overflow section of the highway analyzed in example 14–8 (fig. l4–13c) for the bridge opening of 300 feet. The top of embankment is at elevation 107. Assume a C value of 2.7.

Step 1 Select a range of elevations that will define the rating curve over the road. Tabulate in column 1 of table 14–7. The low point on the road is at elevation 107.

Step 2 Compute the depth of flow, h, over the road. For each elevation listed in column 1, compute h and list in column 2 of table 14–7.

Step 3 Compute $h^{1/2}$, and tabulate in column 3 of table 14–7.

Step 4 Compute the flow area, A, over the road. For each elevation listed in column 1, compute the area over the road and tabulate in column 4 of table 14–7.

Steps 5 through 11 are used to calculate the modified coefficient, C´ to account for the approach velocity head. If it is determined that no modification to the coefficient C is required, these steps may be omitted.

Step 5 Compute the flow area at the upstream section. For each elevation listed in column 1, compute the total area at the upstream section, and tabulate in column 5 of table 14–7. The flow area can be obtained from the Kd computations at the upstream section or computed directly from the surveyed cross section.

Step 6 Determine the discharge through the bridge. For the elevation in column 1, read the discharge through the bridge opening previously computed using bridge loss equations, and tabulate in column 6 of table 14–7.

Step 7 Estimate the discharge over the road using equation 14–24. Tabulate the discharge in column 7 of table 14–7.

Step 8 List the total estimated discharge going past the bridge section. Sum columns 6 and 7, and tabulate in column 8 of table 14–7.

Step 9 Compute the average velocity at the upstream section. The velocity can be estimated by using the total upstream area from column 5 and the estimated discharge from column 8 for the elevations listed in column 1 in the equation V = Q/A. For example, for elevation 107.5:

$$\begin{aligned} V &= \frac{28,250 \text{ ft}^3/\text{s}}{26,700 \text{ ft}^2} \\ &= 1.06 \text{ ft/s} \end{aligned}$$

Tabulate the velocity in column 9 of table 14–7.

Step 10 Compute the velocity head. Using the velocity from column 9, compute $\frac{V^2}{2g}$, and tabulate in column 10 of table 14–7.

Step 11 Compute C´. Using equation 14–25 and data from table 14–7, compute C´. For example, at elevation 107.5:

$$\begin{aligned} C' &= 2.7\frac{1}{\left(\frac{0.5}{0.5+0.017}\right)^{\frac{3}{2}}} \\ &= \frac{2.7}{(0.967)^{\frac{3}{2}}} \\ &= 2.84 \end{aligned}$$

List C´ in column 11 in table 14–7.

Step 12 Compute discharge over the road. Using equation 14–24 and data from table 14–7, compute the discharge over the road. For example, at elevation 107.5:

$$\begin{aligned} Q &= C'Ah^{\frac{1}{2}} \\ &= 2.84(625)(0.707) \\ &= 1,254 \text{ ft}^3/\text{s} \end{aligned}$$

Round to the nearest hundreds value, 1,300 cubic feet per second, and list in column 12. Compare this discharge value to the estimated discharge listed in column 7. If the computed discharge is less than or greater than the estimated discharge, modify the estimated discharge in column 7, and recompute C´ following steps 8 through 12.

Sum columns 6 and 12, and tabulate in column 13 of table 14–7.

Step 13 List the total discharge going past the bridge section.

Step 14 Plot the stage discharge curve. Using the computations shown in columns 1 and 13 of table 14–7, plot the elevation versus discharge. The portion of the discharge flowing over the road (column 12) and the total discharge curve is shown in figure 14–18 for the 300-foot-long bridge. This is the total stage discharge curve for the approach section (M–5).

(d) Multiple bridge openings

Multiple openings in roads occur quite often and must be considered differently from single openings. The M ratio in the BPR procedure (DOT FHWA 1978) is defined as:

$$M = \frac{\text{Kd bridge}}{\text{Kd approach}}$$

When multiple openings are present, the proper ratio must be assigned to each opening and then the capacity computed accordingly. If the flow is divided on the approach, the problem is then one of divided flow with single openings in each channel. In many cases, the flow is not divided for overbank flows. In these cases, the headwater elevation must be considered to be the same elevation for each opening, and the solution becomes trial and error until the head losses are equal for each opening and the sum of the flows equals the desired total.

The approaches are divided as shown in figure 14–19. When the headwater is below the physical dividing point as illustrated by level A, then the M ratio is computed as in a single opening.

When the headwater is above the physical dividing point, crossflow can occur. When this occurs, the approach used to compute the M ratio and J is as follows:

Step 1 Compute the Kd value for each bridge opening.

Step 2 Compute the Kd value for the total approach section.

Table 14–7 Stage discharge over roadway at cross section M–4 without submergence, example 14–9

Elevation ft (1)	h ft (2)	$h^{1/2}$ ft (3)	A over road, ft² (4)	A upstream ft² (5)	Q through bridge, ft³/s (6)	Q est. over road ft³/s (7)	Q est. total ft³/s (8)	V ft/s (9)	$V^2/2g$ ft (10)	C´ (11)	Q over road, ft³/s (12)	Q total ft³/s (13)
107.0	0	0	0	25,500	26,000	0	26,000	1.02	0.016	0	0	26,000
107.5	0.5	0.707	625	26,700	27,000	1,300	28,300	1.06	0.017	2.84	1,300	28,300
108.0	1.0	1.000	1500	28,000	28,000	4,300	32,300	1.15	0.021	2.79	4,200	
						4,200	32,200	1.15	0.021	2.79	4,200	32,200
108.5	1.5	1.225	2525	29,200	29,300	8,500	37,800	1.29	0.026	2.76	8,500	37,800
109.0	2.0	1.414	4000	30,400	30,300	12,000	42,300	1.39	0.030	2.76	15,600	
						15,600	45,900	1.51	0.035	2.76	15,600	45,900
110.0	3.0	1.732	7500	32,800	32,800	35,000	67,800	2.07	0.067	2.79	36,200	
						36,200	69,000	2.10	0.068	2.79	36,200	69,000

Step 3 Proportion the approach Kd value for each opening by the relationship:

$$\text{Kd appr}_x = \frac{\text{Kd}_{\text{bridge}_x}}{\text{Kd}_{\text{bridge}_1} + \text{Kd}_{\text{bridge}_2} + \ldots \text{Kd}_{\text{bridge}_n}} \times \text{total approach Kd}$$

Step 4 Compute M as before using the Kd value computed in step 3 for the approach.

Step 5 Compute the approach area contributing to this opening by the relationship:

$$\text{Area appr}_x = \frac{\text{Kd}_{\text{bridge}_x}}{\text{Kd}_{\text{bridge}_1} + \text{Kd}_{\text{bridge}_2} + \ldots \text{Kd}_{\text{bridge}_n}} \times \text{ total approach area}$$

Step 6 Compute J as before using the area computed in step 5 for the approach area.

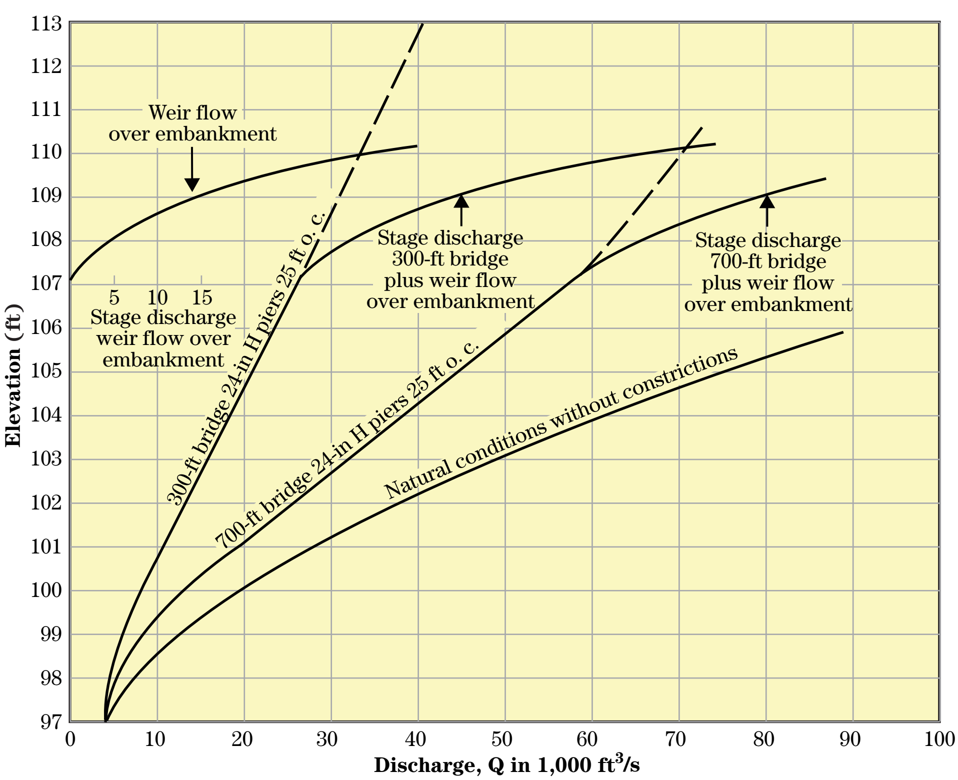
(e) Culverts

Culverts of all types and sizes are encountered when computing stage discharge curves in natural streams. These culverts may or may not have a significant effect on the development of a watershed work plan. However, in many cases, the acceptable plan can be installed without enlarging or replacing the existing culvert.

The FHWA has developed procedures based on research data for use in designing culverts. This procedure is documented in Hydraulic Design of Highway Culverts (1985).

Culverts of various types, installed under different conditions, were studied to develop procedures to determine the backwater effect for the two flow conditions: culverts flowing with inlet control and culverts flowing with outlet control.

Figure 14–18 Stage discharge with embankment overflow, section M–5, example 14–9



(1) Inlet control

Inlet control means that the capacity of the culvert is controlled at the culvert entrance by the depth of headwater (HW_I) and the entrance geometry of the culvert including the barrel shape and cross-sectional area and the type of inlet edge, shape of headwall, and other losses. With inlet control, the entrance acts as an orifice and the barrel of the culvert is not subjected to pressure flow. Figures 14–20(a) and 14–20(b) show sketches of two types of inlet controlled flow.

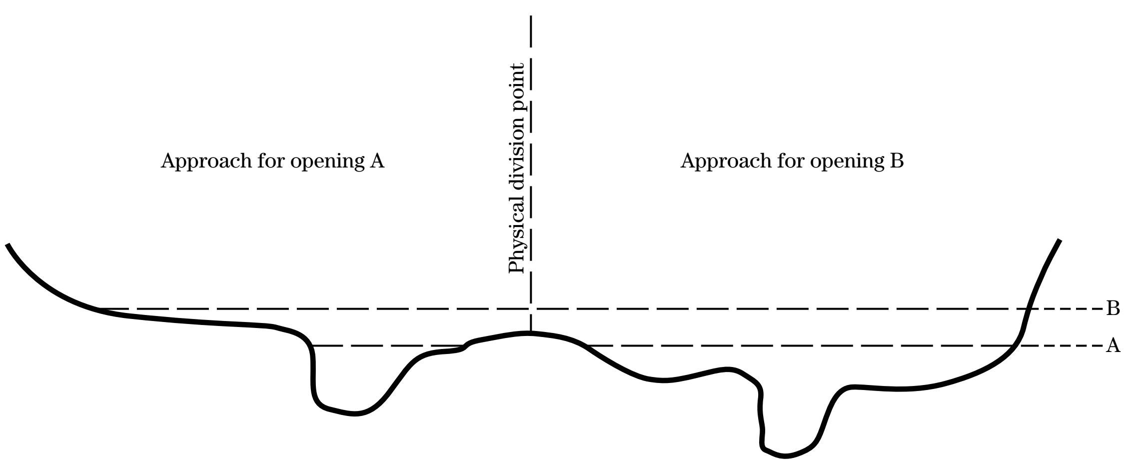
The nomographs shown in exhibits 14–6 through l4–10 were developed from research data by FHWA (1985). They have been checked against actual measurements made by USGS with favorable results.

Types of inlets

The following descriptions are taken from Hydraulic Design of Highway Culverts (1985). Some of the types of inlets are illustrated in figure 14–21.

- Tapered—this inlet is a type of improved entrance that can be made of concrete or metal. The larger diameter of the inlet gradually reduces to the diameter of the culvert.
- Bevel A and Bevel B—these bevels, a type of improved entrance, can be formed of concrete or metal. The square edge of a culvert inlet has been formed to an angled entrance to improve flow conditions into the culvert.
- Angled wingwall—similar to headwall, but at an angle with the culvert.
- Projecting —the culvert barrel extends from the embankment. The transverse section at the inlet is perpendicular to the longitudinal axis of the culvert.
- Headwall—a concrete or metal structure placed around the entrance of the culvert. Headwalls considered are those giving a flush or square edge with the outside edge of the culvert barrel. No distinction is made for wingwalls with skewed alignment.
- Mitered—the end of the culvert barrel is on a miter or slope to conform with the fill slope. All degrees of miter are treated alike since research data on this type of inlet are limited. Headwater is measured from the centerline elevation of the pipe inlet.
- End section—the common prefabricated end made of either concrete or metal and placed on the inlet or outlet ends of a culvert. The closed portion of the section, if present, is not tapered (not illustrated).

Figure 14–19 Approach section for a bridge opening



When water elevation is at A approaches act as directed by the physical division point.
When water elevation is at B approaches act according to the ratio of Kds of openings.

- Grooved edge—the bell or socket end of a standard concrete pipe is an example of this entrance (not illustrated).

(2) Outlet control

Culverts flowing with outlet control can flow with the culvert barrel full or part full for part of the barrel length or for all of it. Figure 14–20(c), (d), (e), and (f) show the various types of outlet control flow. The equation and graphs for solving head loss give accurate results for the first three conditions. For the fourth condition shown in figure 14–20(f), the accuracy decreases as the head decreases. The head H_t (fig. 14–20(c) and (d)) or the energy required to pass a given discharge through the culvert flowing in outlet control with the barrel flowing full throughout its length consists of three major parts: velocity head H_v, entrance loss H_e, and friction loss H_f, all expressed in feet.

From figure 14–22a:

$$H_t = H_v + H_e + H_f \qquad \text{(eq. 14–26)}$$

where:
H_t = total head in ft
H_v = $\frac{V^2}{2g}$ when V is the average velocity in the culvert barrel, ft
H_e = entrance loss, which depends on the geometry of the inlet. The loss is expressed as a coefficient K_e (exhibit 14–21) times the barrel velocity head, ft
H_f = friction loss in barrel, in ft

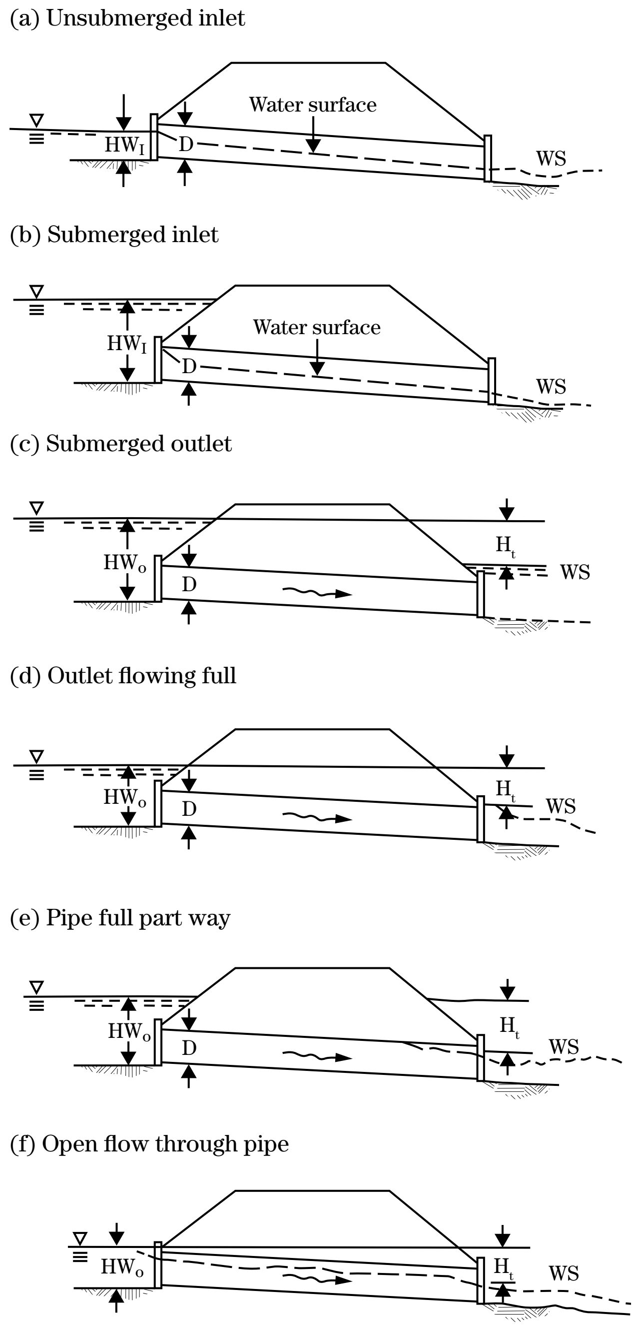
$$H_e = K_e \frac{V^2}{2g} \qquad \text{(eq. 14–27)}$$

where:
K_e = entrance energy loss coefficient

$$H_f = \frac{29n^2L}{R^{1.33}} \times \frac{V^2}{2g} \qquad \text{(eq. 14–28)}$$

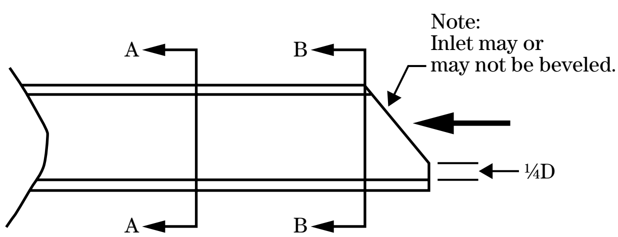
where:
n = Manning's friction factor
L = length of culvert barrel, ft
V = velocity in culvert barrel, ft/s
g = acceleration of gravity, ft/s²
R = hydraulic radius, ft

Figure 14–20 Culvert flow conditions

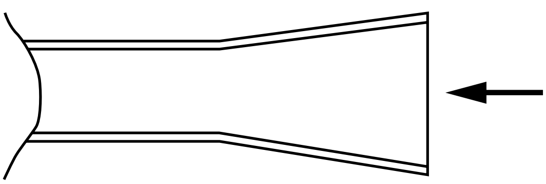


HW_I is associated with inlet control.
HW_o is associated with outlet control.

Figure 14–21 Types of culvert inlets



Plan view

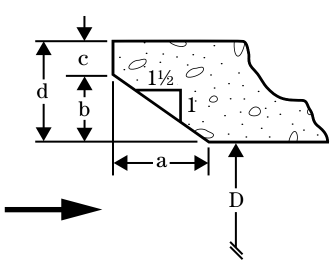


Plan view

Section A–A

Section B–B

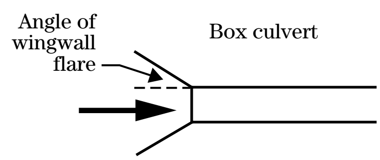
Tapered inlet



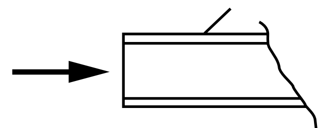
Bevel can be made of metal or concrete
D=inside diameter of pipe

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

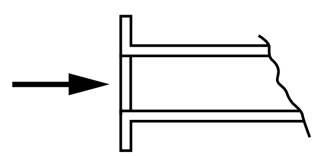
Bevels A and B



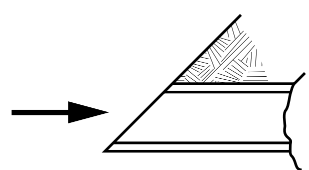
Angled wing wall



Projecting



Headwall



Mitered

Substituting in equation 14–26:

$$H_t = \left(1 + K_e + \frac{29n^2L}{R^{1.33}}\right)\left(\frac{V^2}{2g}\right) \qquad \text{(eq. 14–29)}$$

Figure 14–22(a) shows the terms of equation 14–26, hydraulic grade line, energy grade line, and headwater depth HW_o.

The expression for H_t is derived by equating the total energy upstream from the culvert to the energy at the invert of the culvert outlet.

$$\begin{aligned} H_t &= d_1 + \frac{V_1^{\,2}}{2g} + LS_o - d_2 \\ &= H_v + H_e + H_f \end{aligned} \qquad \text{(eq. 14–30)}$$

where:
V_1 = velocity in the approach section, ft/s
S_o = slope of the channel bed, ft/ft
d_1 = depth of water at inlet, ft
d_2 = depth of water at outlet, ft

From figure 14–22(a):

$$HW_o = H_t + d_2 - LS_o \qquad \text{(eq. 14–31)}$$

If the velocity head in the approach section $\frac{V_1^{\,2}}{2g}$ is low, it can be ignored, and HW_o is considered to be the difference between the water surface and the invert of the culvert inlet.

The depth, d_2, for culverts flowing full is equal to the culvert height in figure 14–20(d), or the tailwater depth (TW), whichever is greater, figure 14–22(b).

The hydraulic grade line for culverts flowing with the barrel part full for part of the barrel length passes through a point where the water breaks with the top of the culvert, and if extended as a straight line, will pass through the plane of the outlet end of the culvert at a point above the critical depth, d_c. This point is approximately halfway between d_c and the crown of the culvert, or equal to:

$$\frac{d_c + D}{2}$$

The depth d_2 or h_o (fig. 14–22(c)) for this type of flow is equal to:

$$\frac{d_c + D}{2}$$

or TW, whichever is greater.

With the definition of d_2, which will be designated as h_o, an equation common to all outlet control conditions can be written:

$$HW_o = H_t + h_o - LS_o \qquad \text{(eq. 14–32)}$$

This equation was used to develop the nomographs shown in exhibits 14–11 through 14–15, which can be used to develop stage discharge curves for the approach section to culverts flowing with outlet control.

Exhibit 14–16 shows d_c for discharge per foot of width for rectangular sections. Exhibits 14–17 to 14–20 show d_c for discharges for various nonrectangular culvert sections.

Example 14–10 Culvert analysis

Develop a stage discharge curve for cross section T–4 (fig. 14–5) showing the backwater effect of eight 16-foot by 8-foot concrete box culverts for each of three conditions: inlet control; outlet control, present channel; and outlet control, improved channel. Figure 14–23(a) shows a cross section along the centerline of the roadway at cross section T–3. Figure 14–23(b) shows a section through the roadway with water surface profiles prior to and after the construction of the culverts and roadway embankment.

The culvert headwalls are parallel to the embankment with no wingwalls, and the entrance is square on three edges.

Figure 14–22 Elements of culvert flow for outlet control

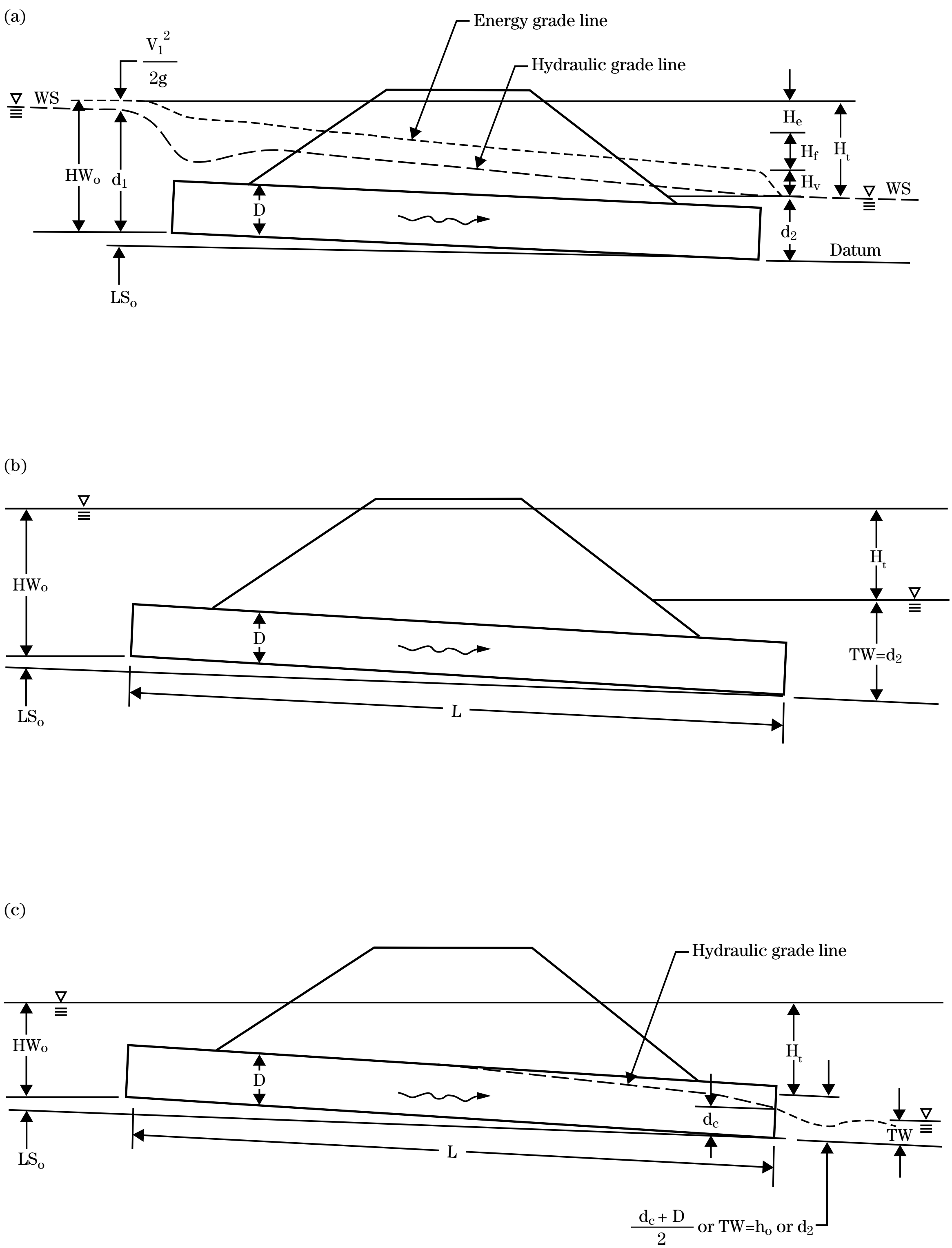
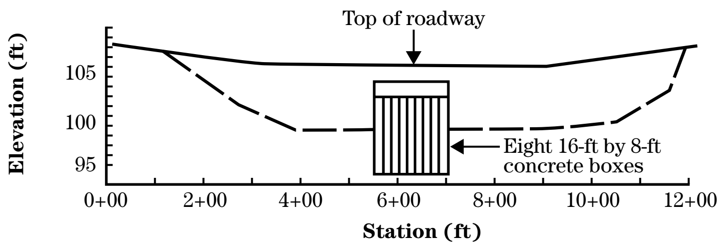
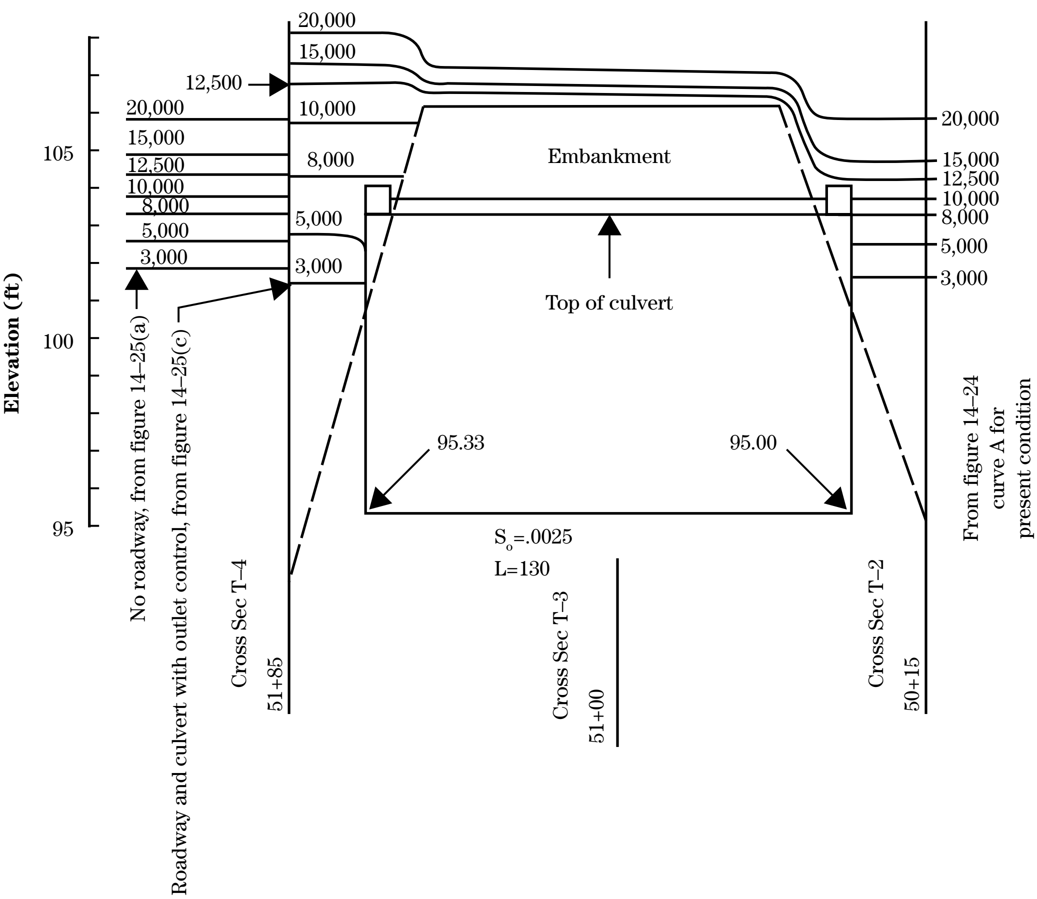


Figure 14–23 Plots of data for cross section T–3, example 14–10

(a) Cross section T–3



(b) Profile through culvert



The following are given in this example: a stage discharge curve for cross section T–2, present condition and with proposed channel improvement (fig. 14–24, curves A and B). Also given is a stage discharge curve for cross section T–4 disregarding the effect of the culverts and roadway fill (fig. 14–25(a)).

Condition 1—Inlet control

Step 1 Select a range of discharges sufficient to define the new stage discharge curve, and tabulate in column 1 of table 14–8.

Step 2 Determine the discharge for each culvert. Divide the discharges in column 1 by the number of culverts (8), and tabulate in column 2 of table 14–8.

Step 3 Determine the discharge per foot of width (Q/B). Divide the discharges in column 2 by the width of each culvert (16 ft), and tabulate in column 3 of table 14–8.

Step 4 Compute HW/D. Using exhibit 14–6, read HW/D for each discharge per foot of width in column 3, and tabulate in column 4 of table 14–8. Referring to exhibit 14–6, project a line from the depth of culvert (8 ft) through the discharge per foot of width (line Q/B) to the first HW/D line, then horizontal to line (3), which is the HW/D for the type of culvert in this example.

Step 5 Compute HW. Multiply column 4 by the depth of the culvert (8 ft), and tabulate in column 5 of table 14–8.

Step 6 Add the invert elevation at the entrance to the culvert (elev. 95.33 ft) to column 5, and tabulate in column 6 of table 14–8.

Step 7 Plot the stage discharge curve assuming inlet control. Plot column 1 and column 6 of table 14–8 as the stage discharge curves for cross section T–4 (fig. 14–25(b), curve A). This assumes inlet con-

Figure 14–24 Stage discharge exit section T–2, example 14–10

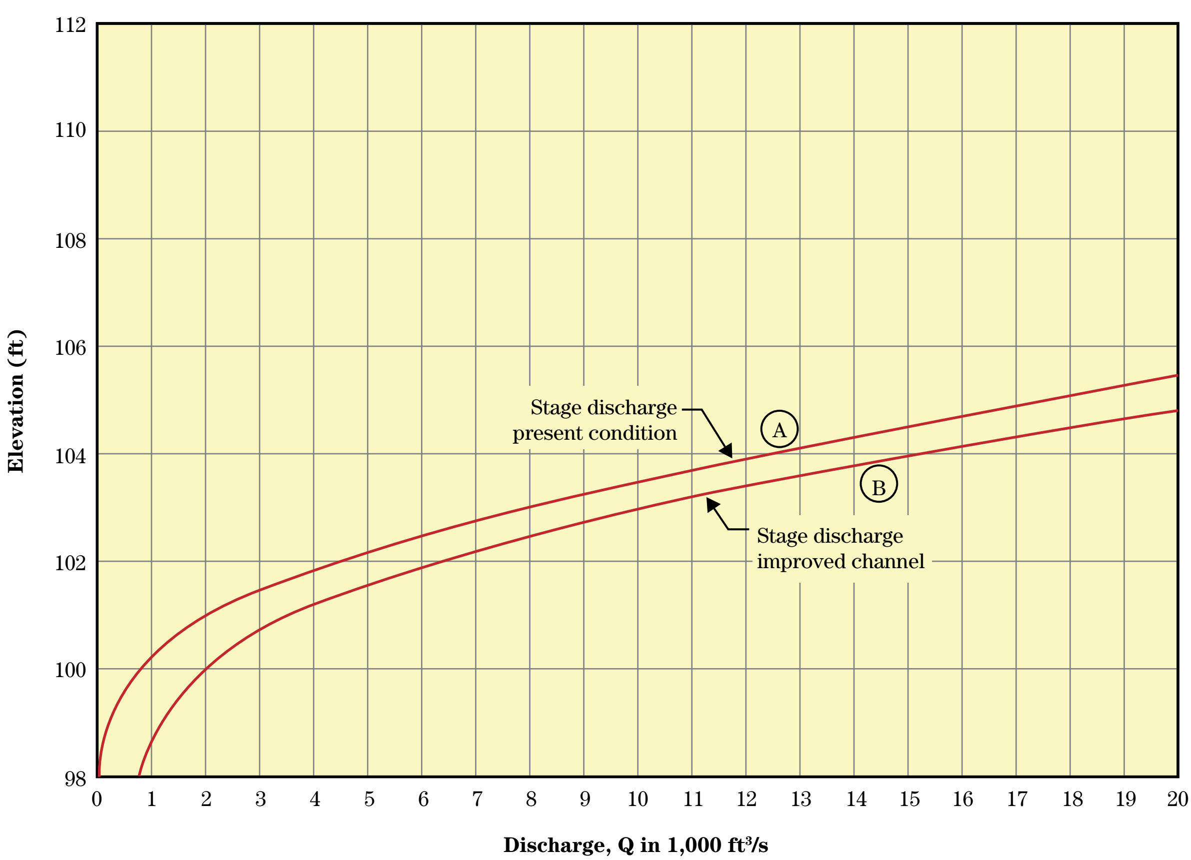


Figure 14–25 Rating curves, cross section T–4, example 14–10

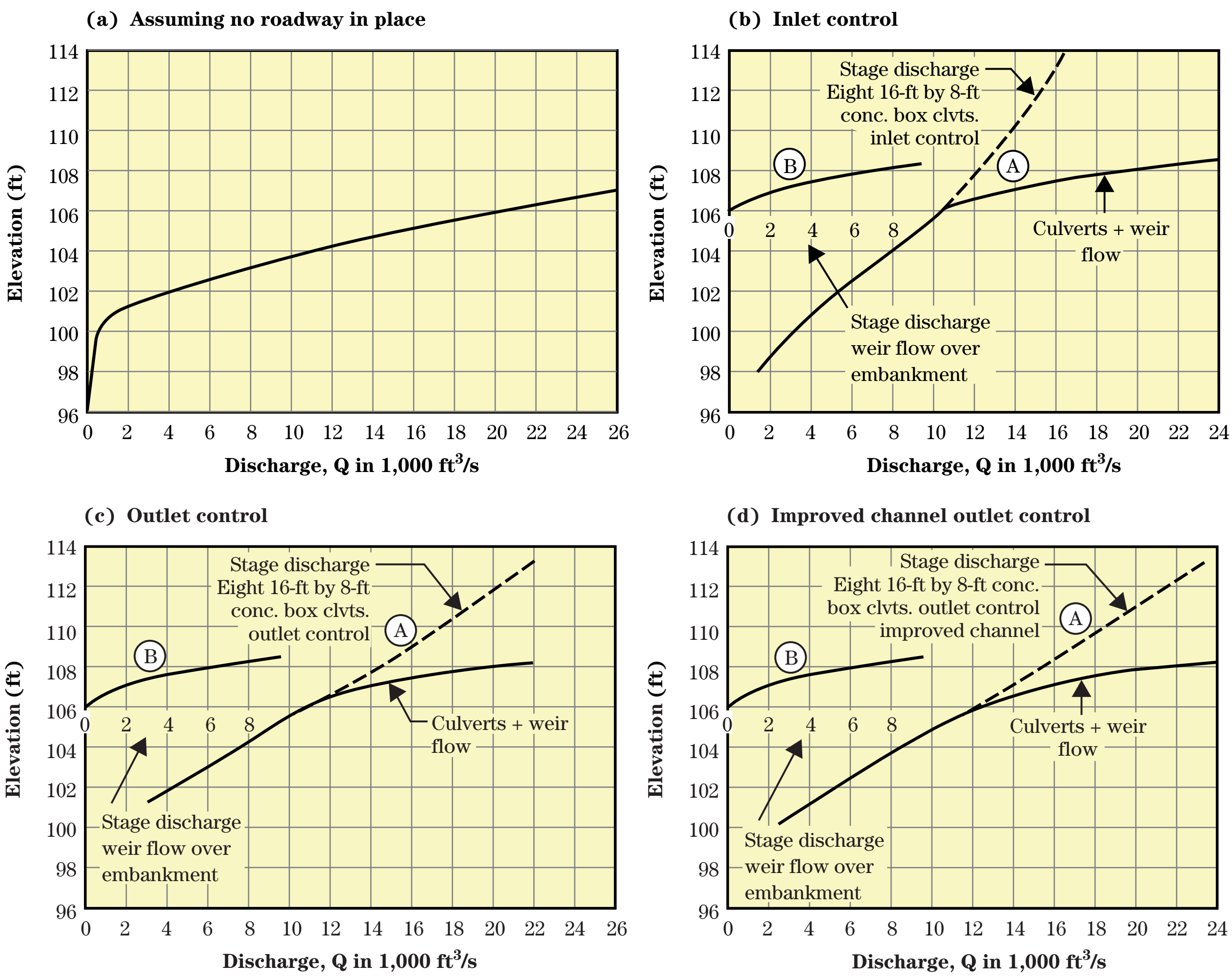


Table 14–8 Headwater computations for eight 16- by 8-foot concrete box culverts, headwalls parallel to embankment (no wingwalls), square-edged on three sides, example 14–10

Total discharge	Discharge for each culvert	Discharge per foot of width	Inlet control			Outlet control present channel								Outlet control improved channel	
Q ft³/s	Q ft³/s	Q/B ft³/s/ft	HW/D	HW ft	HW_I [1] ft	K_e	H_t ft	d_c ft	$(d_c+D)/2$ ft	h_o [2] elev. ft	TW elev. ft	LS_o ft	HW_o [3] elev. ft	TW elev. ft	HW_o elev. ft
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
3,000	375	23.4	0.55	4.40	[4]	0.5	0.22	2.6	5.30	100.30	101.4	0.33	[4]	100.7	[4]
5,000	625	39.1	0.77	6.16	[4]	0.5	0.60 [5]	3.6	5.80	100.80	102.3	0.33	102.57	101.6	101.87
8,000	1,000	62.5	1.08	8.64	103.97 [6]	0.5	1.40	4.9	6.45	101.45	103.0	0.33	104.07 [6]	102.6	103.67 [6]
10,000	1,250	78.1	1.31	10.48	105.81	0.5	2.00	5.7	6.85	101.85	103.5	0.33	105.17	103.0	104.67
12,500	1,563	97.7	1.61	12.88	108.21	0.5	3.00	6.7	7.35	102.35	104.0	0.33	106.67	103.6	106.27
15,000	1,875	117.2	2.01 [7]	16.08	111.41	0.5	4.10	7.5	7.75	102.75	104.5	0.33	108.27	104.0	107.77
20,000	2,500	156.3	— [8]	— [8]	— [8]	0.5	6.50	9.1 [8]	8.00 [9]	103.00	105.5	0.33	111.67	104.8	110.97

Notes:
1/ HW_I = HW+95.33 (invert elevation at entrance end of culvert = 95.33)
2/ $h_o = (d_c+D)/2 + 95.00$ (invert elevation at outlet end of culvert = 95.00)
3/ $HW_o = H_t+TW-LS_o$ or $H_t+h_o-LS_o$, whichever is greater
4/ Tailwater elevation is higher than the computed elevation and open channel flow exists
5/ See example on exhibit 14–11
6/ With channel improvement the control switches from outlet to inlet control between 5,000 and 8,000 ft³/s.
7/ See example on exhibit 14–6
8/ If $d_c \geq D$, the outlet always controls
9/ $(d_c+D)/2$ cannot exceed D

trol with the road sufficiently high to prevent over topping.

Condition 2—Outlet control, present channel

Step 1 Compute the entrance loss coefficient, K_e. Read K_e = 0.5 from exhibit 14–21 for the type of headwall and entrance to box culvert, and tabulate in column 7 of table 14–8.

Step 2 Compute the head loss, H, for the concrete box culvert flowing full. Using exhibit 14–11, draw a line from L =130 feet on the K_e = 0.5 scale to the cross-sectional area scale, 16 feet × 8 feet = 128 square feet, and establish a point on the turning line. Draw a line from the discharge (Q) line for each of the discharges shown in column 2 through the turning point to the head (H_t) line. Tabulate H_t in column 8 of table 14–8.

Step 3 Compute the critical depth, d_c, for each discharge per foot of width. Using exhibit 14–16, read d_c for each discharge per foot of width shown in column 3, and tabulate in column 9 of table 14–8.

Step 4 Compute $(d_c + D)/2$. Tabulate in column 10 of table 14–8. D is the inside diameter of pipe.

Note: $(d_c + D)/2$ cannot exceed D.

Step 5 Compute h_o. Add the invert elevation of the outlet end of the culvert (elevation. 95.00) to $(d_c + D)/2$, and tabulate as h_o in column 11 of table 14–8.

Step 6 Compute the TW elevation for each discharge in column 1. Using figure 14–24, curve A, read the elevation for each discharge in column 1, and tabulate as TW elevation in column 12 of table 14–8.

Step 7 Compute the difference in elevation of the inlet and outlet inverts of the culverts. Multiply L × S_o = 130 × 0.0025 = 0.33, and tabulate in column 13 of table 14–8.

Step 8 Compute the water surface elevation, HW_o, assuming outlet control. Add values in column 8 to the larger of column 11 or column 12 minus column 13, and tabulate as HW_o in column 14 of table 14–8.

Step 9 Plot the stage discharge curve assuming outlet control. Plot column 1 and column 14 on figure 14–25(c) as curve A assuming outlet control with the roadway sufficiently high to prevent over topping.

Condition 3—Outlet control, improved channel

Step 1 Compute the tailwater elevation at the culvert for the improved channel condition. Using figure 14–24, curve B, read the elevation for each discharge in column 1, and tabulate as TW elevation in column 15 of table 14–8.

Step 2 Compute the elevation assuming outlet control, improved channel. Add column 8 plus column 15 minus column 13, and tabulate in column 16 of table 14–8.

Step 3 Plot the stage discharge curve assuming outlet control with improved channel. Plot column 1 and column 16 on figure l4–25(d) as curve A, the stage discharge curve for cross section T–4 assuming outlet control with improved channel and the roadway sufficiently high to prevent over topping.

Condition 4—flow over roadway

Assume the approach velocity head for this example is negligible, and the coefficient C will equal C´ used in equation 14–24. If the velocity head is significant, and a correction to the coefficient C is desired by using equation 14–25, follow steps 5 through 9 of example 14–9.

Step 1 Select a range of elevations that will define the rating curve over the road. Tabulate in column 1 of table 14–9. The low point on the road is at elevation 106.

Step 2 Compute the depth of flow, H, over the road. For each elevation in column 1, compute H and list in column 2 of table 14–9.

Step 3 Compute $H^{\frac{1}{2}}$. Tabulate in column 3 of table 14–9.

Step 4 Compute the flow area, A, over the road. For each elevation listed in column 1, compute the area over the road, and tabulate in column 4 of table 14–9.

Step 5 Determine coefficient, C. Assume C = 2.7 for this example, and assume C = C´. Tabulate C´ in column 5 of table 14–9.

Step 6 Compute the discharge over the roadway using equation 14–24.

Step 7 Plot the stage discharge curve. Using the computations shown in table 14–9, plot columns 1 and 6 shown on figure 14–25(b), (c), and (d) as curve B.

Step 8 Graphically combine curves A and B in figure 14–25(b), (c), and (d) to form the stage discharge curve for the culverts and weir flow over the roadway.

Each of the three flow conditions were computed independent of each other. The flow condition that actually controls is that which requires the greater upstream elevation for the discharge being considered. By comparing elevations for the same discharge for the three conditions tabulated on table 14–8 and plotted on figure 14–25(b) and (c), the type of control at any given discharge can be determined. It may be advantageous to plot all the curves on one graph to better define points of intersection.

Under the old channel conditions, it can be determined that open channel flow conditions exist for discharges less than about 4,000 cubic feet per second, outlet control governs between about 4,000 and 8,300 cubic feet per second, and inlet control governs for discharges greater than 8,300 cubic feet per second.

Under new channel conditions, open channel flow exists for discharges less than 3,800 cubic feet per second, outlet control governs for discharges between 3,800 and 6,600 cubic feet per second, and inlet control governs for discharges greater than 6,600 cubic feet per second. Also, in both cases, discharges greater than 10,200 cubic feet per second flow will occur over the road embankment.

Table 14–9 Stage discharge over roadway at cross section T–3, figure 14–5, example 14–10

Elevation ft	H ft	$H^{1/2}$ ft	A ft^2	C′	Q ft^3/s
(1)	(2)	(3)	(4)	(5)	(6)
106	0	0	0	2.7	0
106.5	0.5	0.707	340	2.7	650
107	1	1	750	2.7	2,020
107.5	1.5	1.225	1,230	2.7	4,070
108	2	1.414	1,790	2.7	6,830

If the actual profile for discharges occurring under open channel flow conditions is desired, water surface profiles should be run through the culverts.

It can also be seen from figure 14–25(a) and (b) that by constructing the highway with eight 16- by 8-foot concrete box culverts, elevations upstream will increase over present conditions for discharges greater than 5,000 cubic feet per second. For improved outlet conditions, upstream elevations will not be increased above present conditions until a discharge of 6,500 cubic feet per second occurs.

630.1406 References

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Exhibit 14–1 K values for converting cubic feet per second per square mile to cubic feet per second

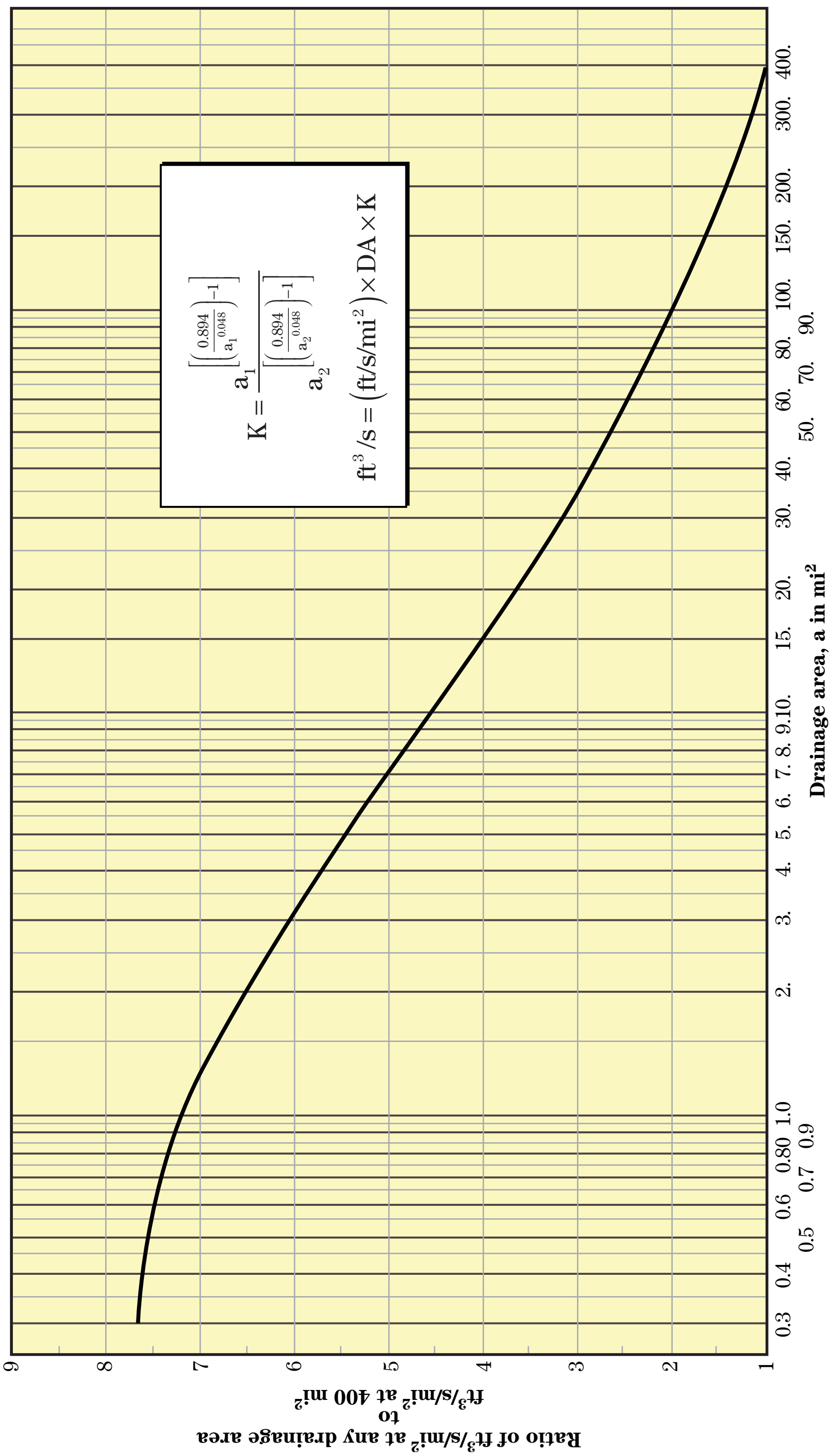


Exhibit 14–2 Estimate of velocity head in bridges

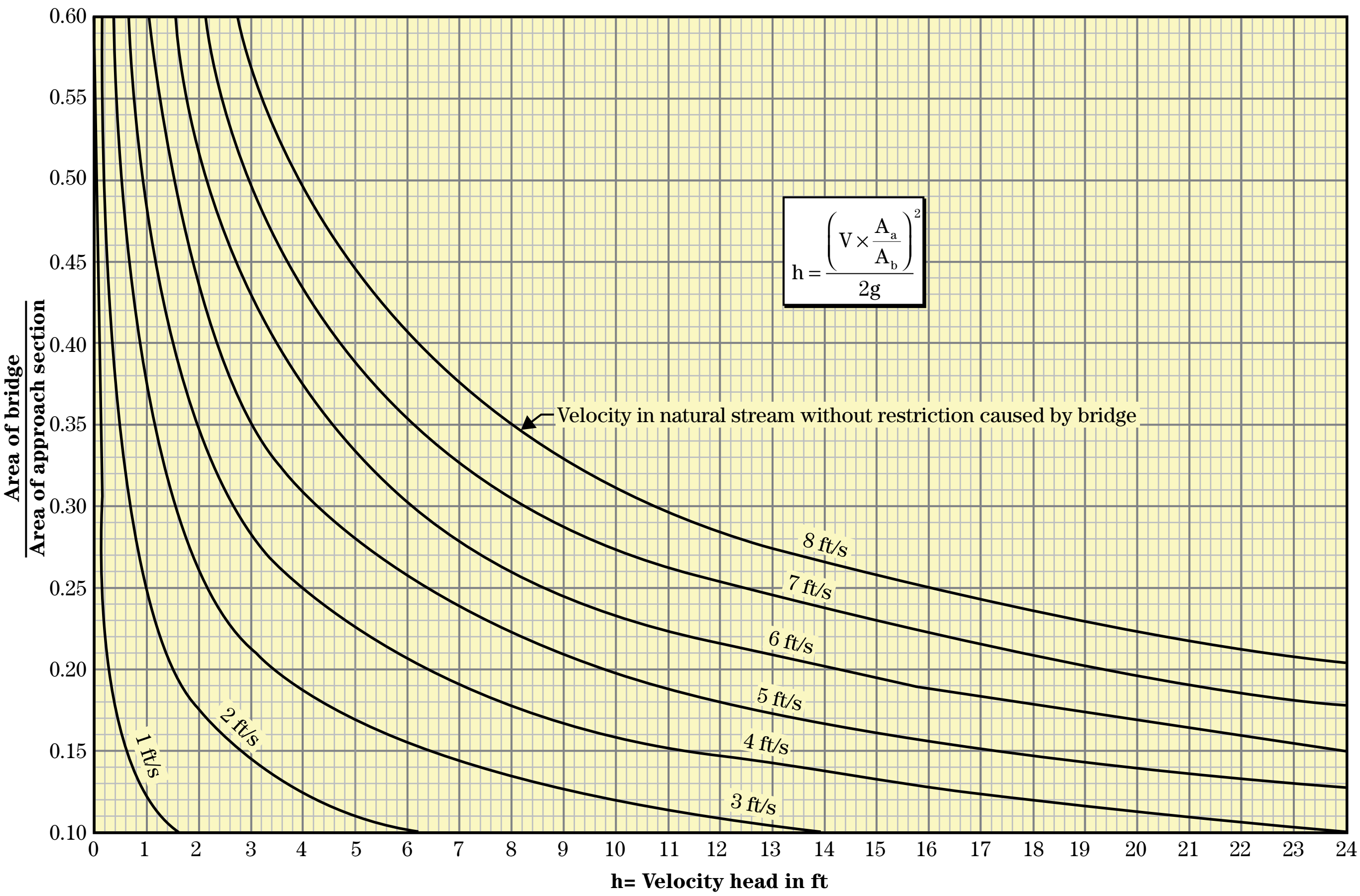


Exhibit 14–3 Estimate of M for use in BPR equation

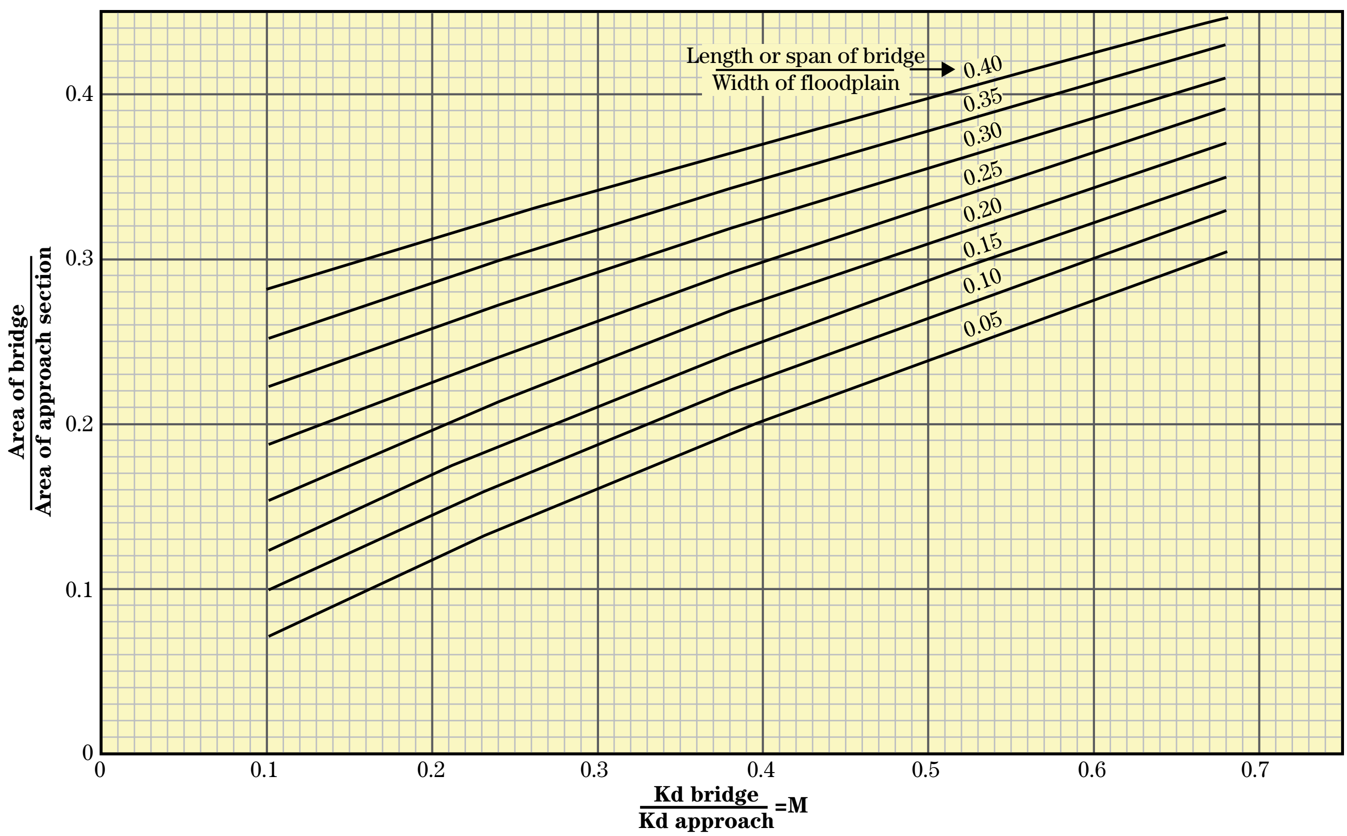
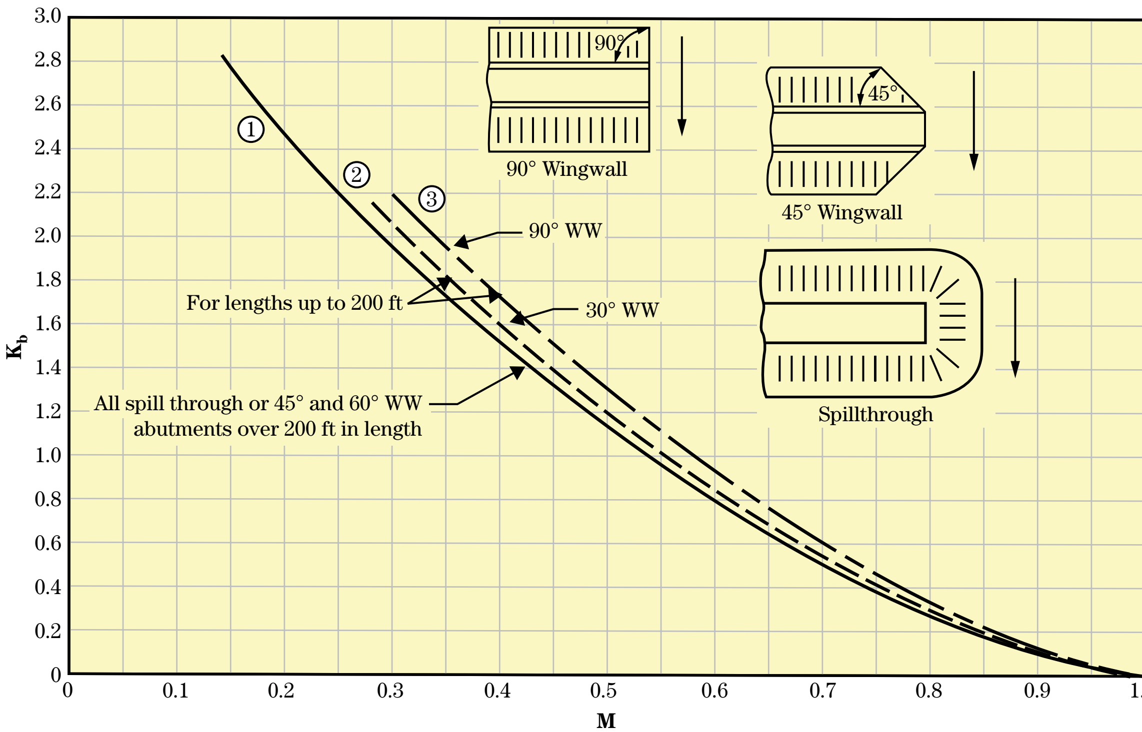
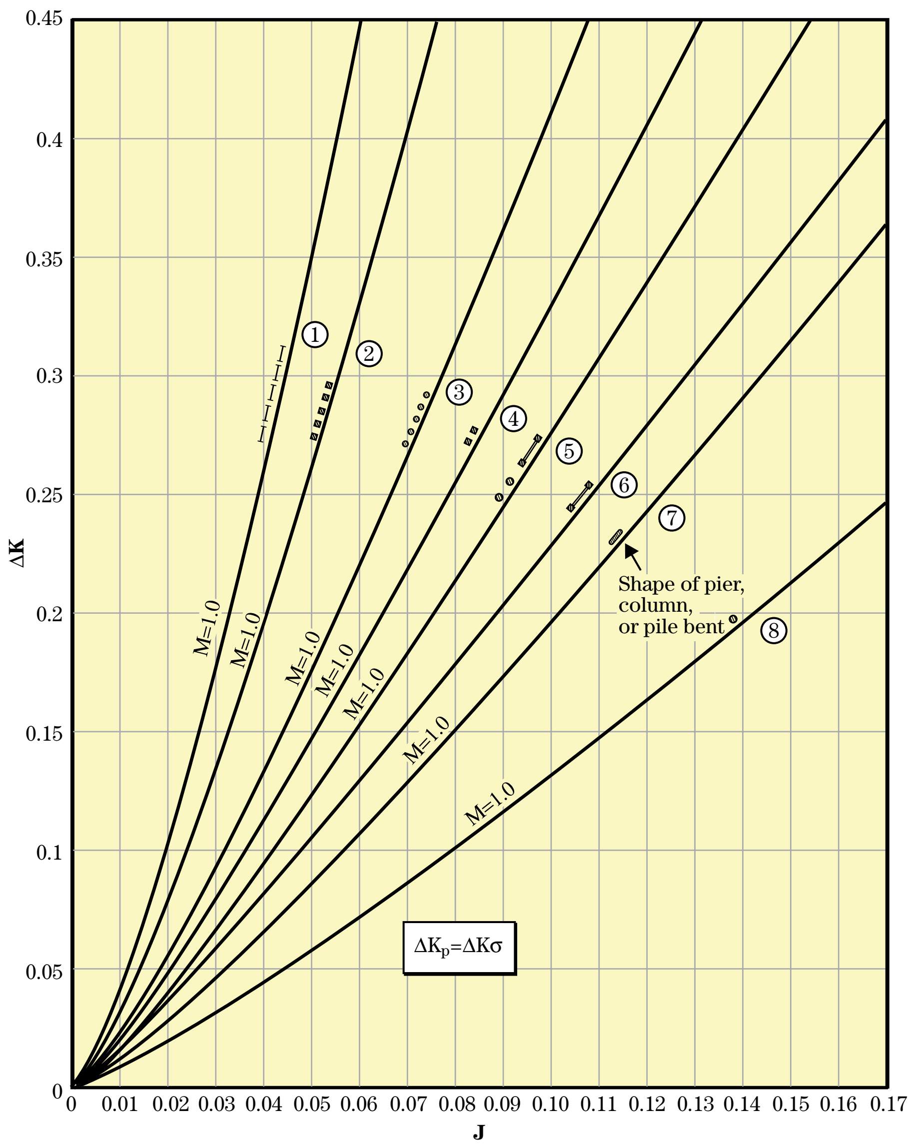


Exhibit 14–4 BPR base curve for bridges, K_b



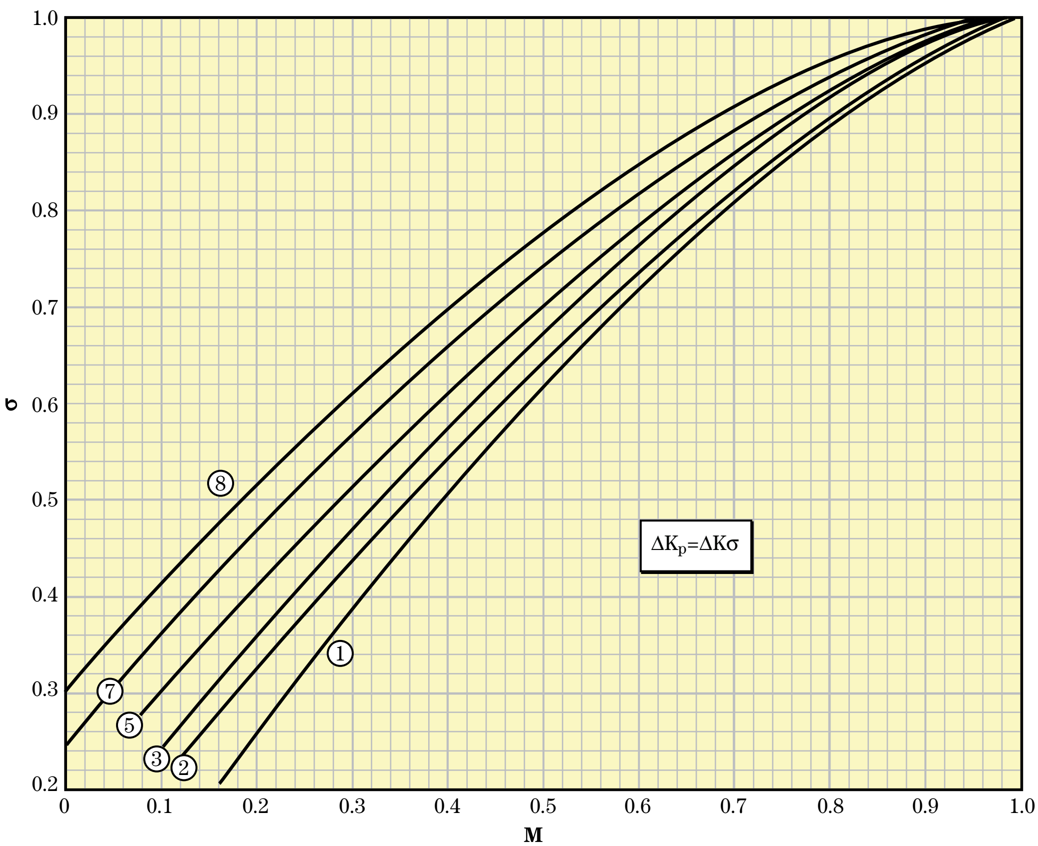
Note: Exhibits 14-4 and 14-5 are from U.S. Department of Department of Transportation documents (DOT FHWA) 1965, 1978, and 1985.

Exhibit 14–5(a) Incremental backwater coefficients ΔK and J for the more common types of columns, piers, and pile bents



Notes: Exhibits 14-4 and 14-5 are from U.S. Department of Department of Transportation documents (DOT FHWA) 1965, 1978, and 1985.
J is defined in step 11 of example of 14–8 as the area of the piers divided by the gross area of the bridge opening.

Exhibit 14–5(b) Incremental backwater coefficients σ and M for the more common types of columns, piers, and pile bents



Note: Exhibits 14-4 and 14-5 are from U.S. Department of Department of Transportation documents (DOT FHWA) 1965, 1978, and 1985.

Exhibit 14–6 Headwater depth for box culverts with inlet control

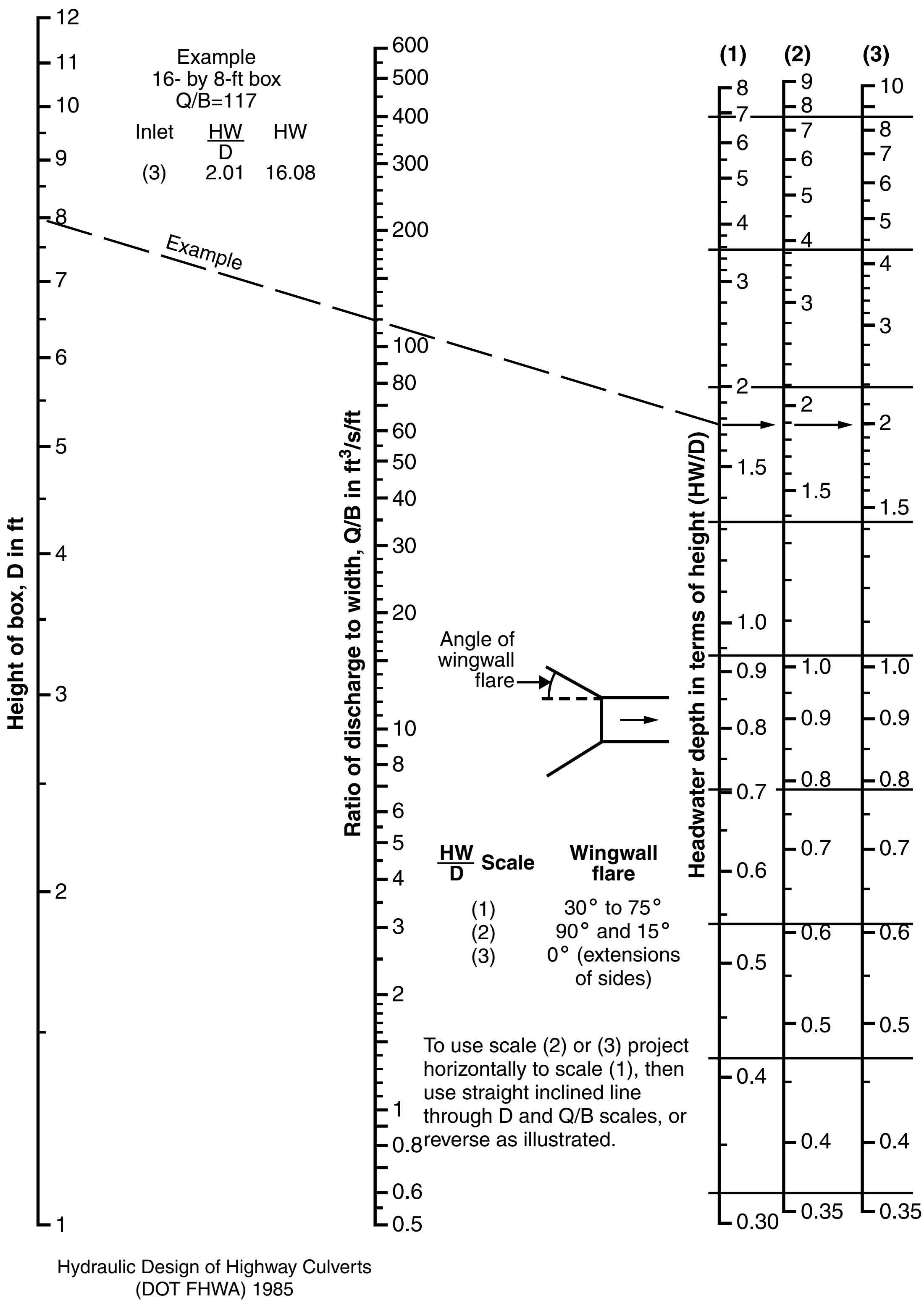


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Exhibit 14–7 Headwater depth for concrete pipe culverts with inlet control

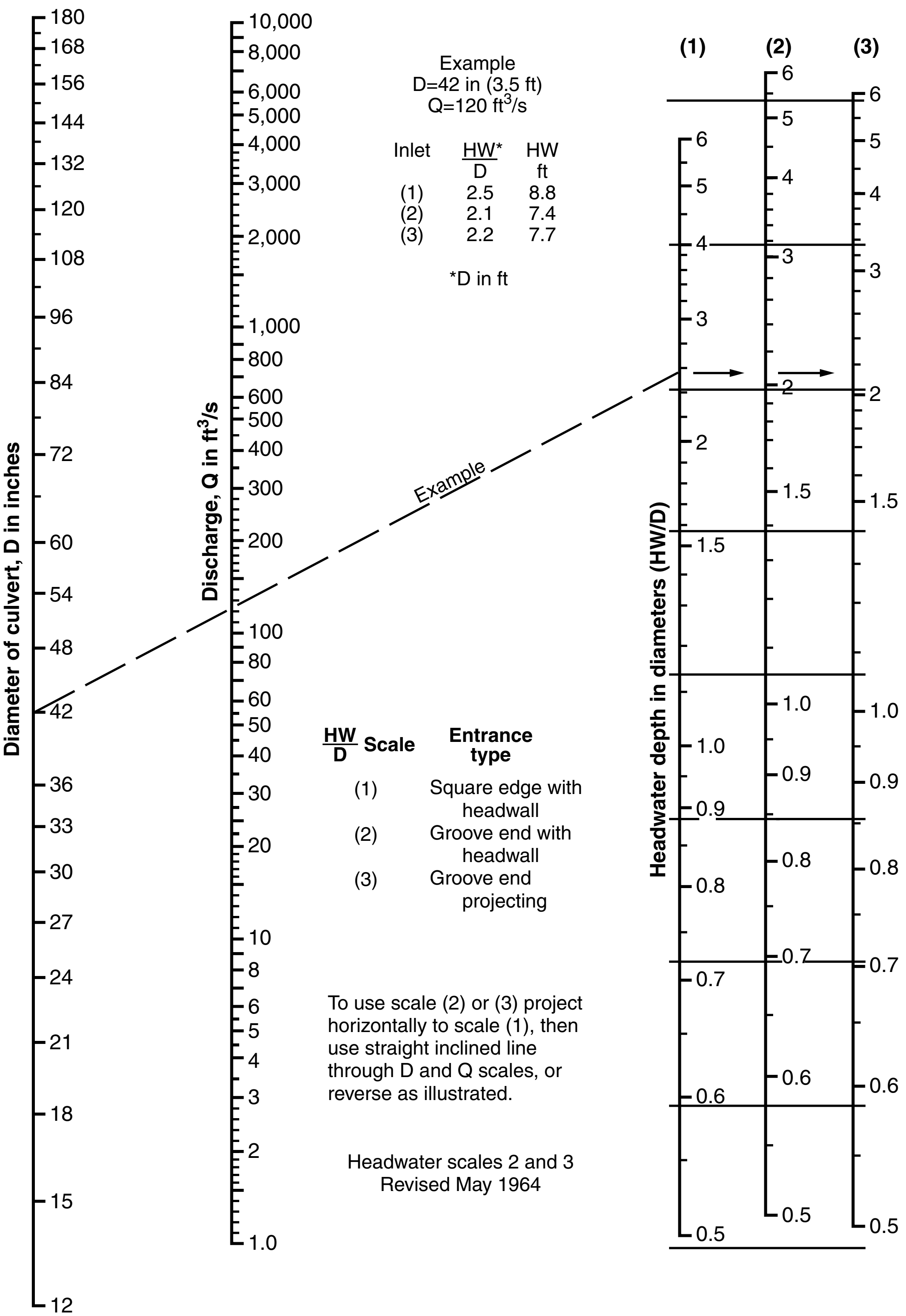

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Exhibit 14–8 Headwater depth for oval concrete pipe culverts long axis horizontal with inlet control

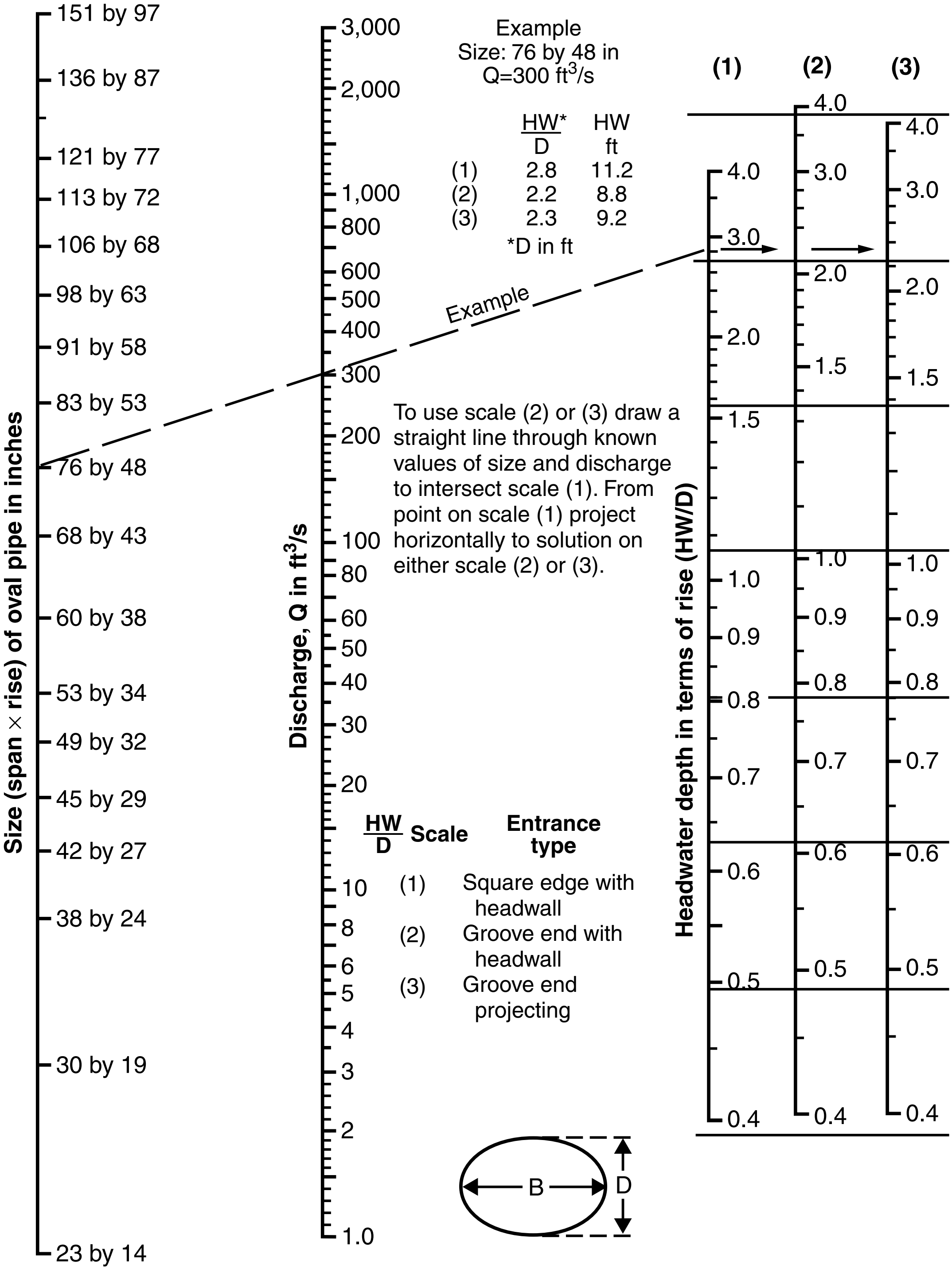

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Exhibit 14–9 Headwater depth for corrugated metal (C.M.) pipe culverts with inlet control

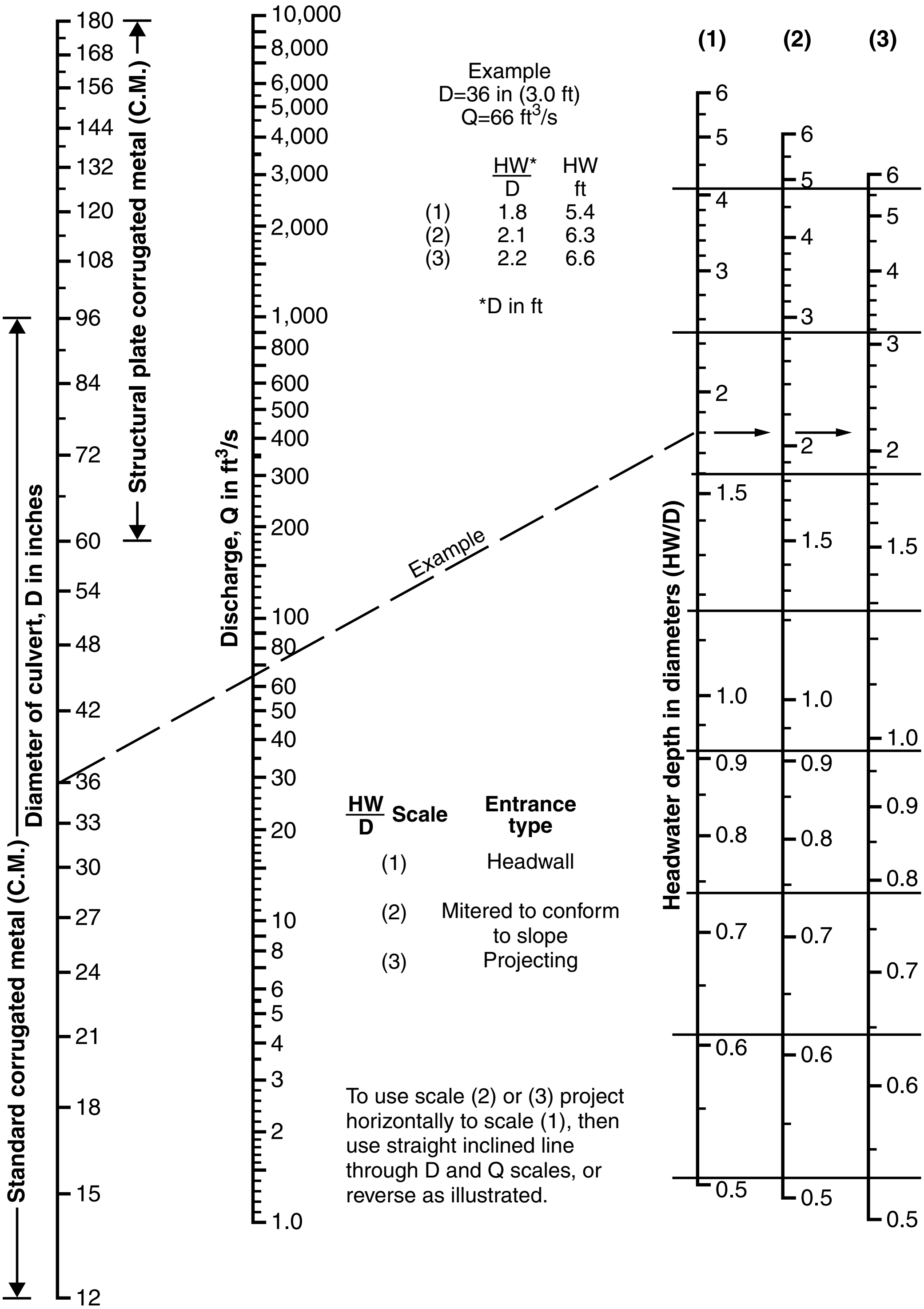

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Exhibit 14–10 Headwater depth for corrugated metal (C.M.) pipe-arch culverts with inlet control

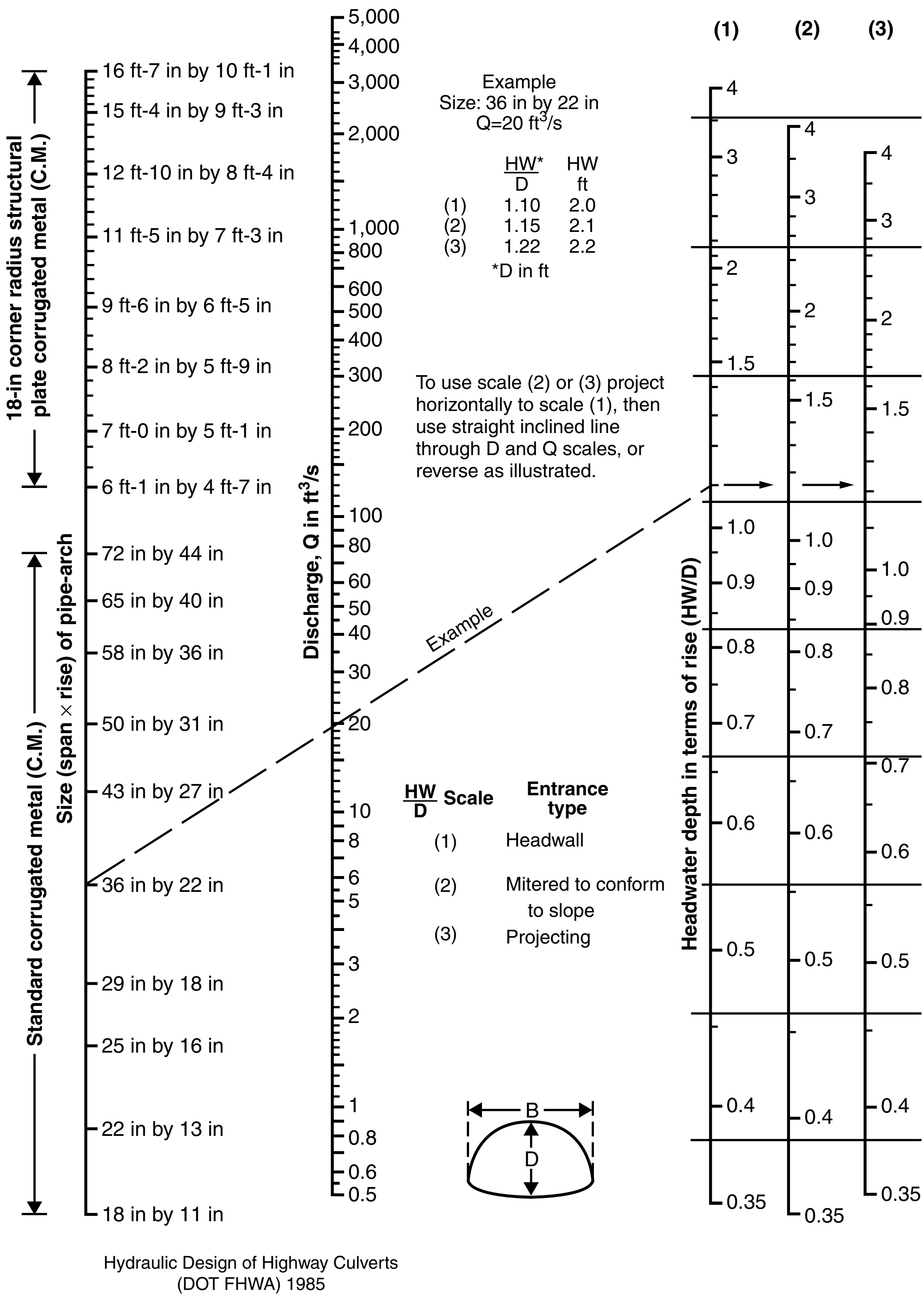

Exhibit 14–11 Head for concrete box culverts flowing full—n = 0.012

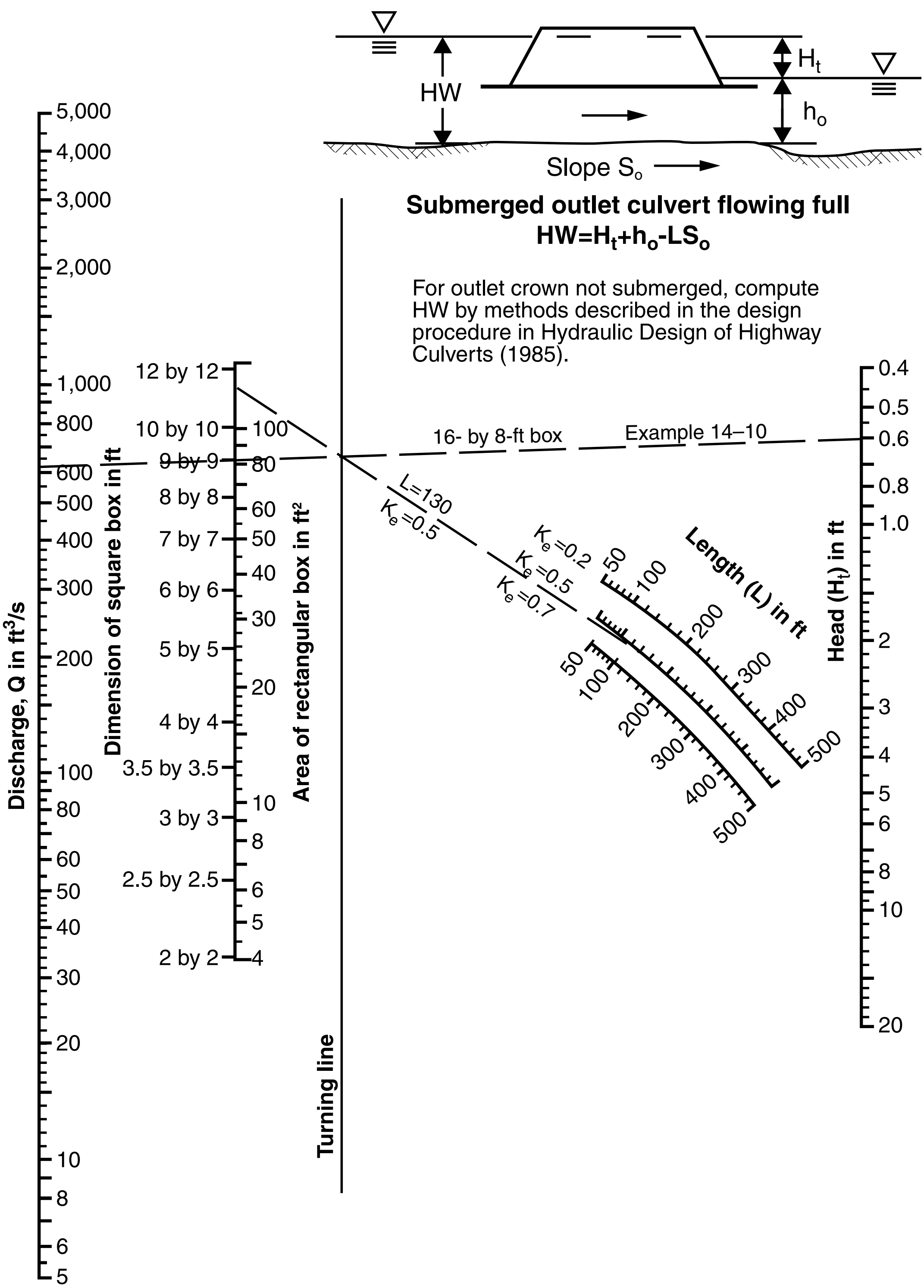

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Exhibit 14–12 Head for concrete pipe culverts flowing full—n = 0.012

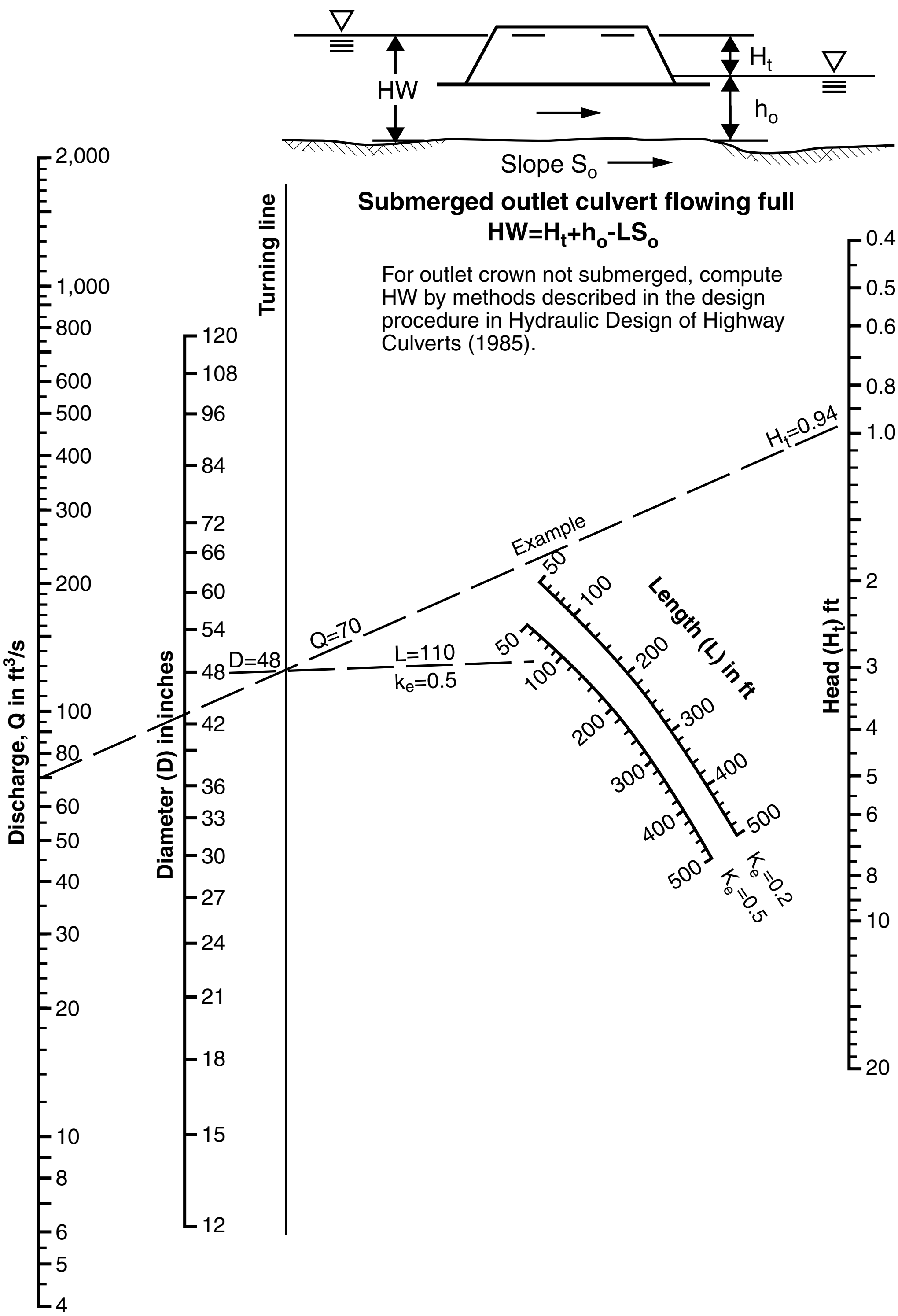

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Exhibit 14–13 Head for oval concrete pipe culverts long axis horizontal or vertical flowing full—n = 0.012

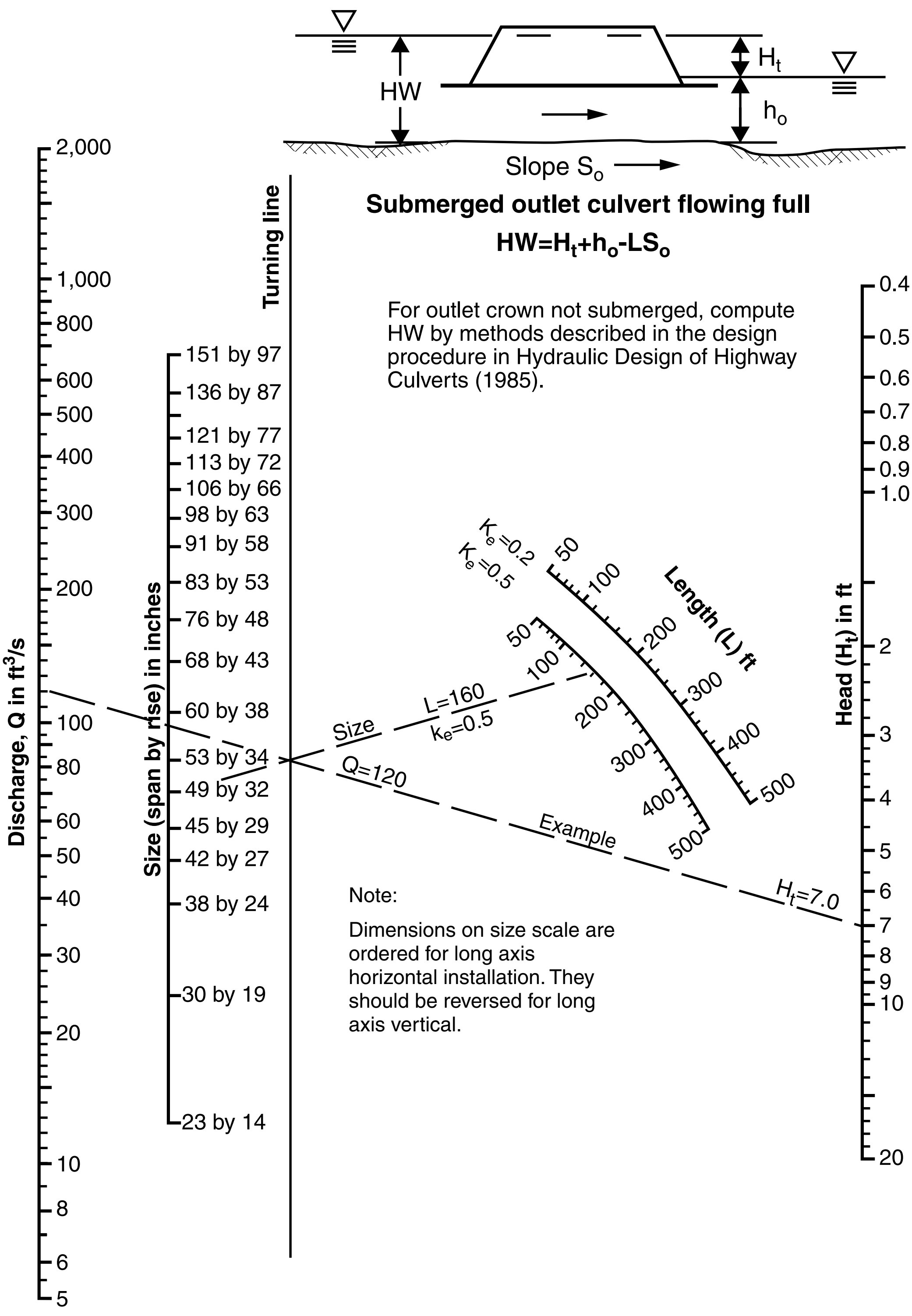

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Exhibit 14–14 Head for standard corrugated metal (C.M.) pipe culverts flowing full—n = 0.024

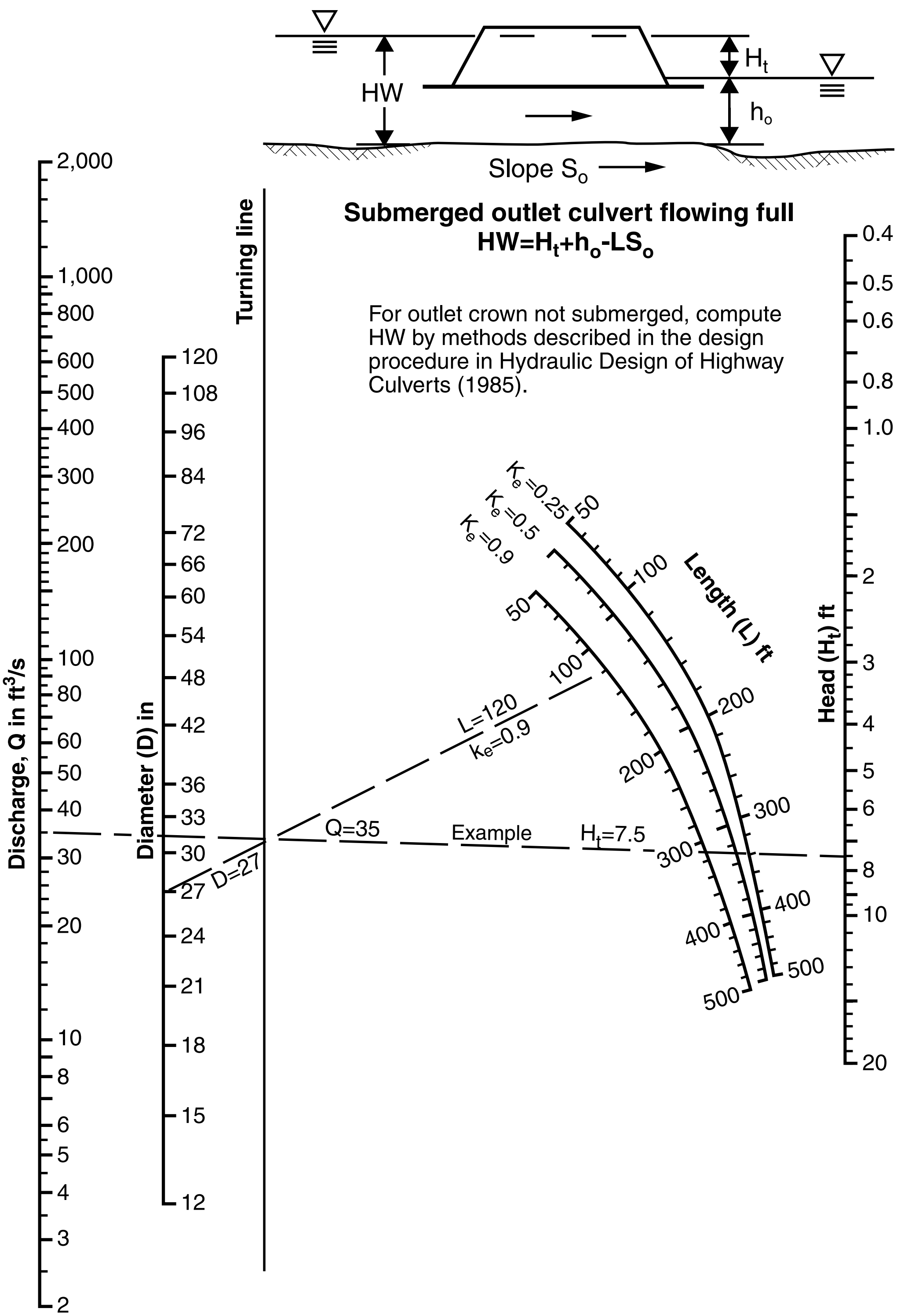

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Exhibit 14–15 Head for standard corrugated metal (C.M.) pipe-arch culverts flowing full—n = 0.024

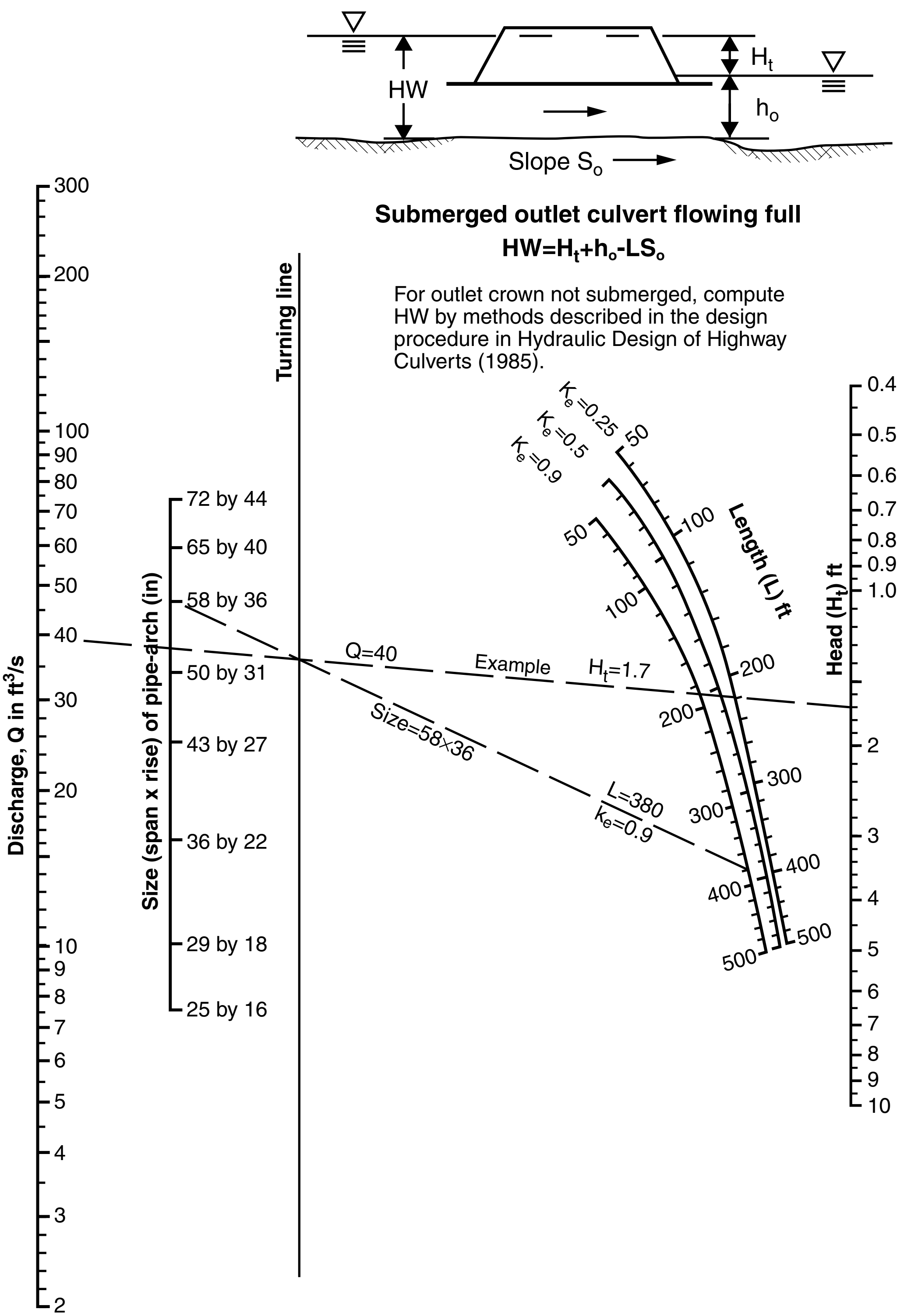

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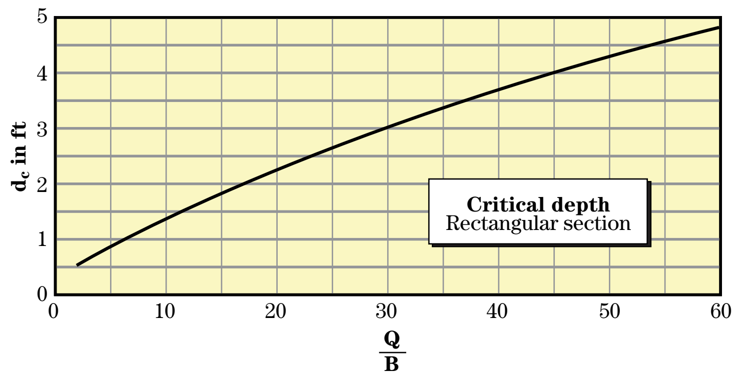
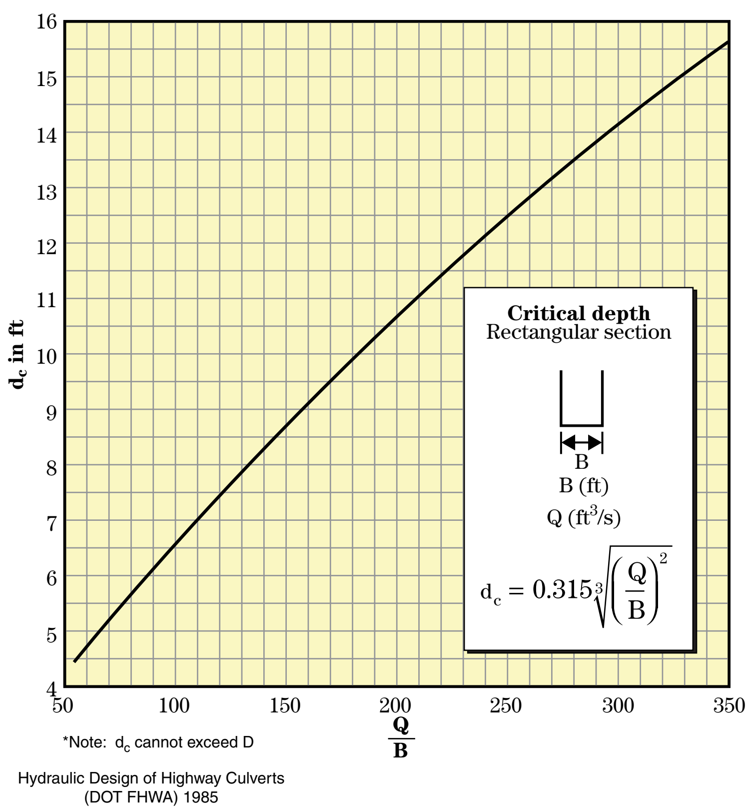
Exhibit 14–16 Critical depth—rectangular section

(a)



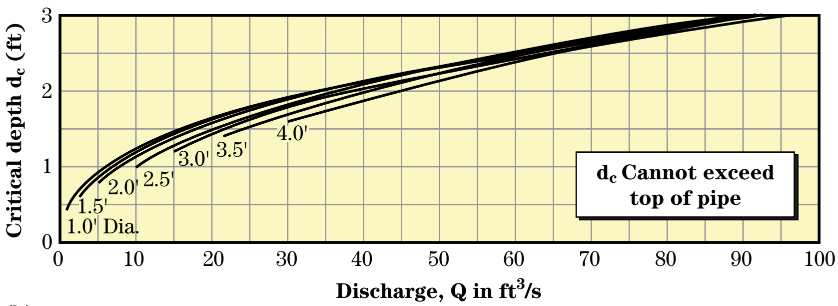
(b)



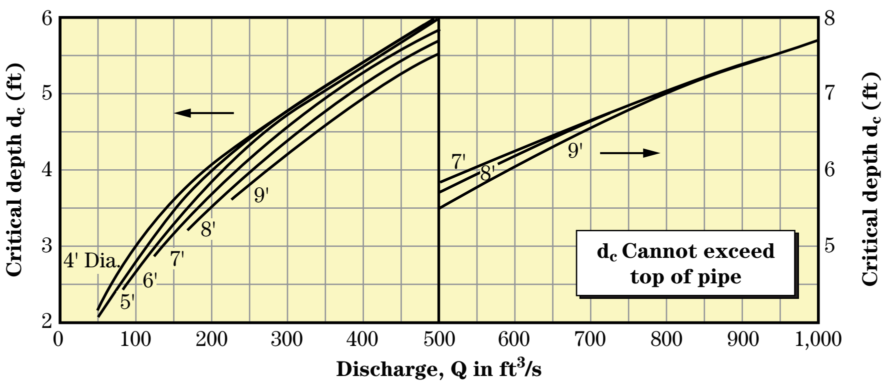
Note: The diagram and equation in (b) applies to (a) also.

Exhibit 14–17 Critical depth—circular pipe

(a)



(b)



(c)

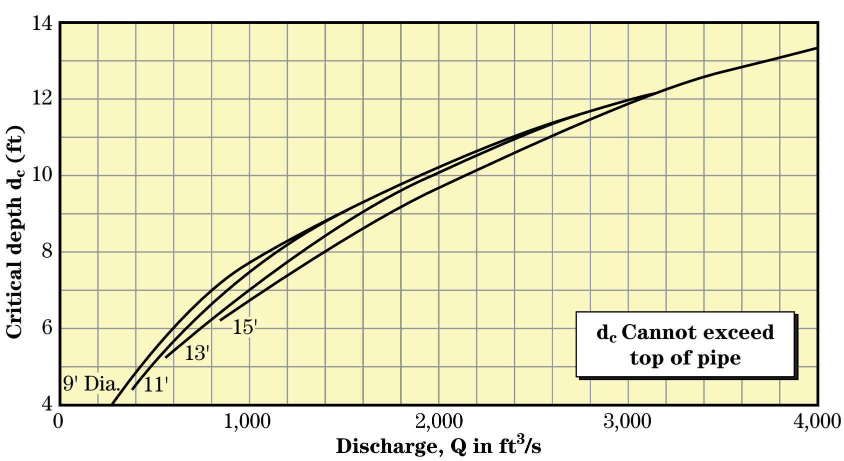


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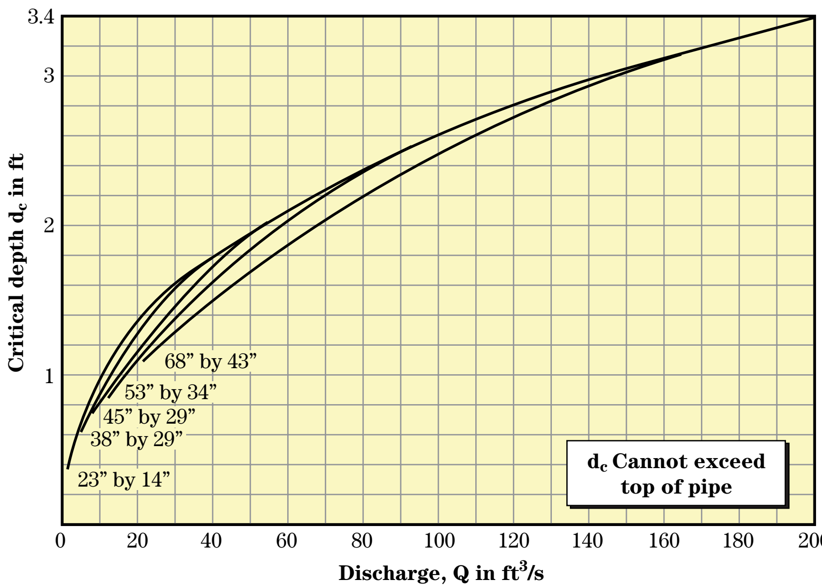
Exhibit 14–18 Critical depth—oval concrete pipe—long axis horizontal

(a)



(b)

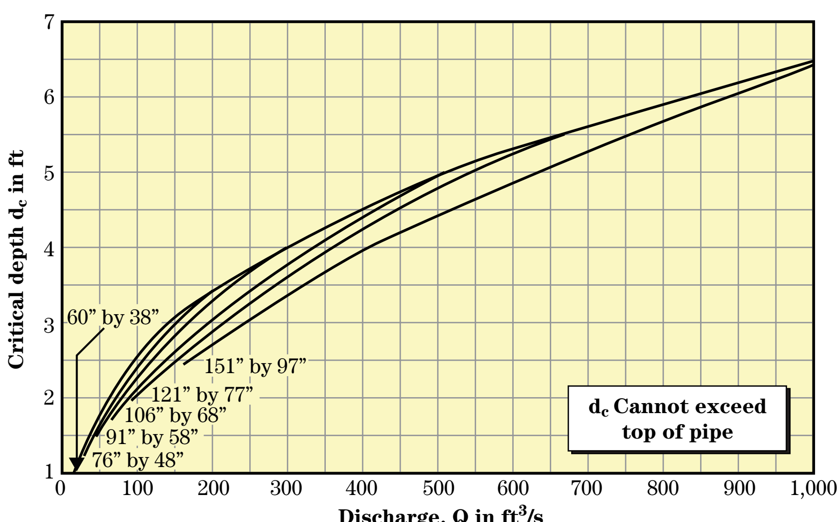


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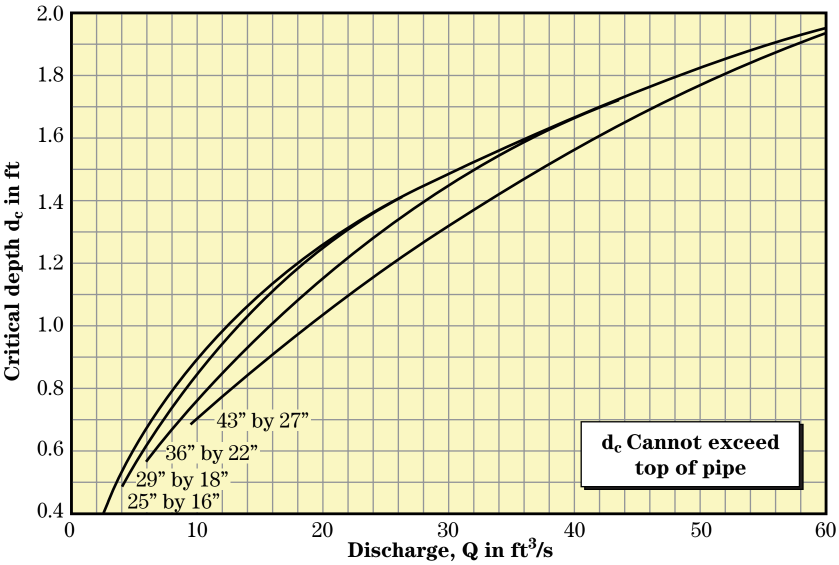
Exhibit 14–19 Critical depth—standard corrugated metal (C.M.) pipe-arch

(a)



(b)

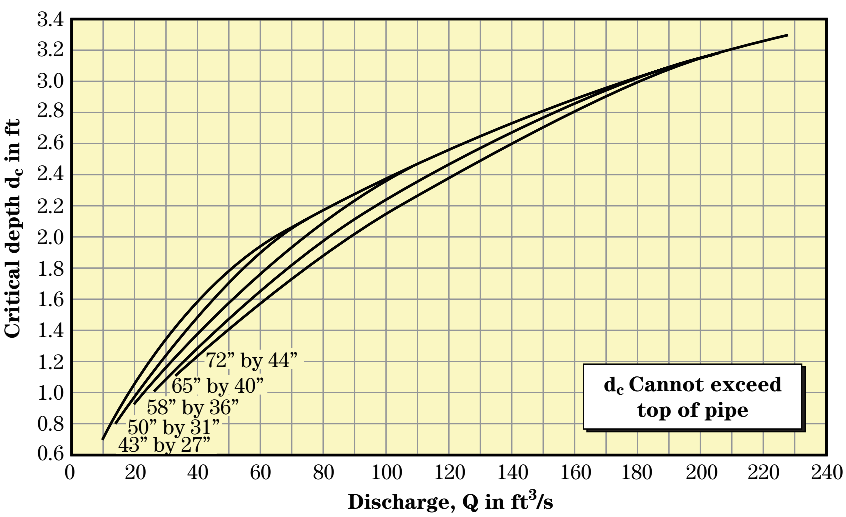


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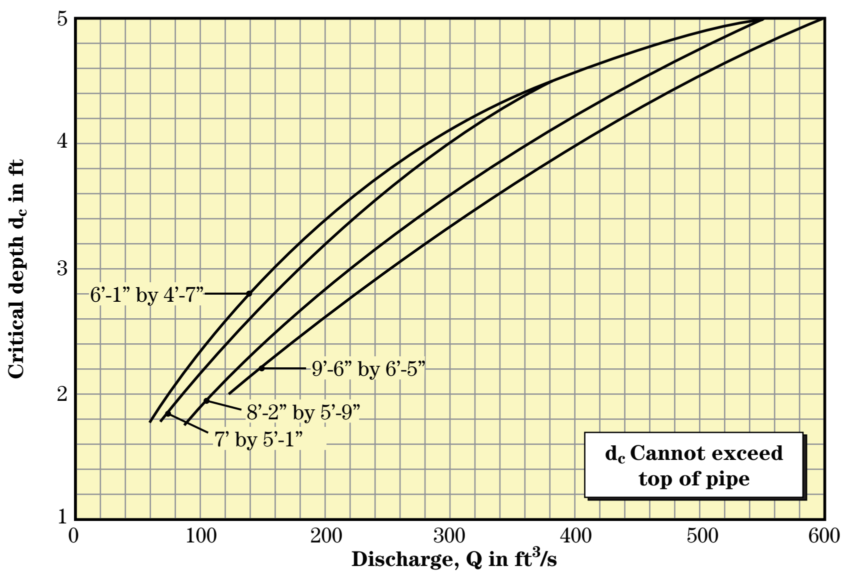
Exhibit 14–20 Critical depth—structural plate—corrugated metal (C.M.) pipe-arch, 18-inch corner radius

(a)



(b)

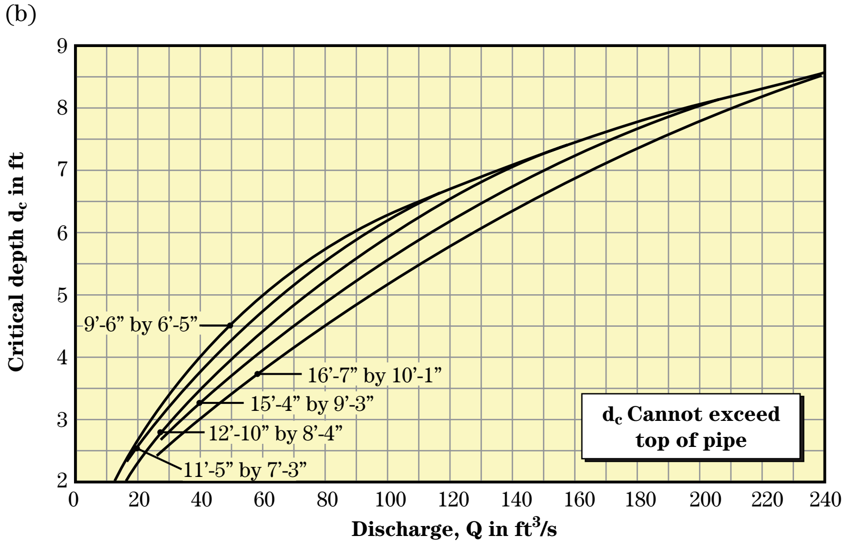


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Exhibit 14–21 Entrance loss coefficients

Coefficient K_e to apply to velocity head $V^2/2g$ for determination of head loss at entrance to a structure, such as a culvert or conduit, operating full or partly full with control at the outlet.

Entrance head loss $$H_e = K_e \frac{V^2}{2g}$$

Type of structure and design of entrance	Coefficient K_e
Pipe, concrete	
Projecting from fill, socket end (groove end)	0.2
Projecting from fill, square cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove end)	0.2
Square-edge	0.5
Rounded (radius = 1/12D)	0.2
Mitered to conform to fill slope	0.7
*End-section conforming to fill slope	0.5
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	0.7
*End-section conforming to fill slope	0.5
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	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	0.2
Wingwalls 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

* "End-section conforming to fill slope," made of either metal or concrete, are the sections commonly available from manufacturers. From limited hydraulic tests, they are equivalent in operation to a headwall in both inlet and outlet control. Some end sections incorporating a closed taper in their design have a superior hydraulic performance. These latter sections can be designed using the information given for the beveled inlet, p. 5–13, DOT FHWA (1985).

Appendix A

Estimating the Roughness Coefficient *n* for Use in Hydraulic Computations Associated with Natural Streams, Floodways, and Similar Streams

This appendix (Cowan 1956) describes a method for estimating the roughness coefficient *n* for use in hydraulic computations associated with natural streams, floodways, and similar streams. The procedure proposed applies to the estimation of *n* in Manning's formula. This formula is now widely used, it is simpler to apply than other widely recognized formulas, and has been shown to be reliable.

Manning's formula is empirical. The roughness coefficient *n* is used to quantitatively express the degree of retardation of flow. The value of *n* indicates not only the roughness of the sides and bottom of the channel, but also all other types of irregularities of the channel and profile. In short, *n* is used to indicate the net effect of all factors causing retardation of flow in a reach of channel under consideration.

There seems to have developed a tendency to regard the selection of *n* for natural channels as either an arbitrary or an intuitive process. This probably results from the rather cursory treatment of the roughness coefficient in most of the more widely used hydraulic textbooks and handbooks. The fact is that the estimation of *n* requires the exercise of critical judgment in the evaluation of the primary factors affecting *n*. With any method of estimating *n*, care should be applied to ensure reasonable answers. For example, if this seven-step process yields a resulting *n* value greater than one for a natural stream, the result is very likely too high. Comparing the systematic method results with pictorial *n* values in references listed in NEH630.1402(a) will ensure the reasonableness of systematically computed *n* values. These factors are irregularity of the surfaces of the channel sides and bottom, variations in shape and size of cross sections, obstructions, vegetation, and meandering of the channel.

The need for realistic estimates of *n* justifies the adoption of a systematic procedure for making the estimates.

Procedure for estimating *n*—The general procedure for estimating *n* involves first the selection of a basic value of *n* for a straight, uniform, smooth channel in the natural materials involved; then secondly, through critical consideration of the factors listed, the selection of a modifying value associated with each factor. The modifying values are added to the basic value to obtain *n* for the channel under consideration.

In the selection of the modifying values associated with the five primary factors, it is important that each factor be examined and considered independently. In considering each factor, it should be kept in mind that *n* represents a quantitative expression of retardation of flow. Turbulence of flow can, in a sense, be visualized as a measure or indicator of retardance. Therefore, in each case, more critical judgment may be exercised if it is recognized that as conditions associated with any factor change so as to induce greater turbulence, there should be an increase in the modifying value. A description and tabulated guide to the selection of modifying values for each factor is given under the following procedural steps.

Step 1 Select the basic *n* value. This step requires the selection of a basic *n* value for a straight, uniform, smooth channel in the natural materials involved. The selection involves consideration of what may be regarded as a hypothetical channel. The conditions of straight alignment, uniform cross section, and smooth side and bottom surfaces without vegetation should be kept in mind. Thus, the basic *n* will be visualized as varying only with the materials forming the sides and bottom of the channel. The minimum values of *n* shown by reported test results for the best channels in earth are in the range from 0.016 to 0.018. Practical limitations associated with maintaining smooth and uniform channels in earth for any appreciable period indicate that 0.02 is a realistic basic *n*. The basic *n*, as it is intended for use in this procedure, for natural or excavated channels, may be selected from the following table. Where the bottom and sides of a channel are of different materials, this fact may be recognized in selecting the basic *n*.

Character of channel	Basic *n*
Channels in earth	0.020
Channels cut into rock	0.025
Channels in fine gravel	0.024
Channels in coarse gravel	0.028

Step 2 Select the modifying value for surface irregularity. The selection is to be based on the degree of roughness or irregularity of the surfaces of channel sides and bottom. Consider the actual surface irregularity; first, in relation to the degree of surface smoothness obtainable with the natural materials involved, and second, in relation to the depths of

flow under consideration. Actual surface irregularity comparable to the best surface to be expected of the natural materials involved calls for a modifying value of zero. Higher degrees of irregularity induce turbulence and call for increased modifying values. This table may be used as a guide to the selection.

Degree of irregularity	Surfaces comparable to	Modifying value
Smooth	The best obtainable for the materials involved	0.000
Minor	Good dredged channels; slightly eroded or scoured side slopes of canals or drainage channels	0.005
Moderate	Fair to poor dredged channels; moderately sloughed or eroded side slopes of canals or drainage channels	0.010
Severe	Badly sloughed banks of natural channels; badly eroded or sloughed sides of canals or drainage channels; unshaped, jagged and irregular surfaces of channels excavated in rock	0.020

Step 3 Selection of modifying value for variations in shape and size of cross sections. In considering changes in size of cross sections, judge the approximate magnitude of increase and decrease in successive cross sections as compared to the average. Changes of considerable magnitude, if they are gradual and uniform, do not cause significant turbulence. The greater turbulence is associated with alternating large and small sections where the changes are abrupt. The degree of effect of size changes may be best visualized by considering it as depending primarily on the frequency with which large and small sections alternate and secondarily on the magnitude of the changes.

In the case of shape variations, consider the degree to which the changes cause the greatest depth of flow to move from side to side of the channel. Shape changes causing the greatest turbulence are those for which shifts of the main flow from side-to-side occur in distances short enough to produce eddies and upstream currents in the shallower portions of those sections where the maximum depth of flow is near either side. Selection of modifying values may be based on the following guide:

Character of variations in size and shape of cross sections	Modifying value
Changes in size or shape occurring gradually	0.000
Large and small sections alternating occasionally or shape changes causing occasional shift of main flow from side to side	0.005
Large and small sections alternating frequently or shape changes causing frequent shifting of main flow from side-to-side	0.010 to 0.015

Step 4 Selection of modifying value for obstructions. The selection is to be based on the presence and characteristics of obstructions such as debris deposits, stumps, exposed roots, boulders, fallen and lodged logs. Care should be taken that conditions considered in other steps are not re-evaluated or double-counted by this step.

In judging the relative effect of obstructions consider: the degree to which the obstructions occupy or reduce the average cross-sectional area at various stages; the character of obstructions (sharp-edged or angular objects induce greater turbulence than curved, smooth-surfaced objects); and the position and spacing of obstructions transversely and longitudinally in the reach under consideration. The following table may be used as a guide to the selection.

Relative effect of obstructions	Modifying value
Negligible	0.000
Minor	0.010 to 0.015
Appreciable	0.020 to 0.030
Severe	0.040 to 0.060

Step 5 Selection of modifying value for vegetation. The retarding effect of vegetation is probably due primarily to the turbulence induced as the water flows around and between the limbs, stems and foliage, and secondarily to reduction in cross section. As depth and velocity increase, the force of the flowing water tends to bend the vegetation. Therefore, the ability of vegetation to cause turbulence is partly related to its resistance to bending force. Furthermore, the amount and character of foliage, that is, the growing season condition versus dormant season condition is important. In judging the

retarding effect of vegetation, critical consideration should be given to the following: the height in relation to depth of flow; the capacity to resist bending; the degree to which the cross section is occupied or blocked out; and the transverse and longitudinal distribution of vegetation of different types, densities, and heights in the reach under consideration. The following table may be used as a guide to the selection:

Vegetation and flow conditions comparable to:	Degree of effect on *n* value	Range in modifying value
• Dense growths of flexible turf grasses or weeds, of which Bermuda and blue grasses are examples, where the average depth of flow is 2 to 3 times the height of vegetation • Supple seedling tree switches such as willow, cottonwood or salt cedar where the average depth of flow is 3 to 4 times the height of the vegetation	Low	0.005 to 0.010
• Turf grasses where the average depth of flow is 1 to 2 times the height of vegetation • Stemmy grasses, weeds or tree seedlings with moderate cover where the average depth of flow is 2 to 3 times the height of vegetation • Dormant season brushy growths, moderately dense, similar to willows 1 to 2 years old, along side slopes of channel with no significant vegetation along the channel bottom, where the hydraulic radius is greater than 2 feet	Medium	0.010 to 0.025
• Turf grasses where the average depth of flow is about equal to the height of vegetation • Dormant season, willow or cottonwood trees 8 to 10 years old, intergrown with some weeds and brush, none of the vegetation in foliage, where the hydraulic radius is greater than 2 feet • Growing season, bushy willows about 1 year old intergrown with some weeds in full foliage along side slopes, no significant vegetation along channel bottom, where hydraulic radius is greater than 2 feet	High	0.025 to 0.050
• Turf grasses where the average depth of flow is less than one half the height of vegetation • Growing season, bushy willows about 1 year old, intergrown with weeds in full foliage along side slopes; dense growth of cattails along channel bottom; any value of hydraulic radius up to 10 or 15 feet • Growing season; trees intergrown with weeds and brush, all in full foliage; any value of hydraulic radius up to 10 or 15 feet	Very high	0.050 to 0.100

A further basis for judgment in the selection of the modifying value for vegetation may be found in table 14A–1 which contains descriptions and data for actual cases where *n* has been determined. In each of the cases listed in table 14A–1, the data were such that the increase in *n* due to vegetation could be determined within reasonably close limits.

Step 6 Determination of the modifying value for meandering of channel. The modifying value for meandering may be estimated as follows: Add the basic *n* for step 1 and the modifying values of steps 2 through 5 to obtain an estimate of *n* for a straight channel, or n_s.

Let L_f= the straight length of the reach under consideration.

L_c = the meander length of the channel in the reach.

Compute modifying value for meandering in accordance with the following table.

Ratio L_c/L_f	Degree of meandering	Modifying value
1.0 to 1.2	Minor	0.000
1.2 to 1.5	Appreciable	0.15 n_s
1.5 and greater	Severe	0.30 n_s

Where lengths for computing the approximate value of L_c/L_f are not readily obtainable, the degree of meandering can usually be judged reasonably well.

Step 7 Computation of *n* for the reach. The value of *n* for the reach is obtained by adding the values determined in steps 1 through 6. An illustration of the estimation of *n* is given in example 14A–1.

Dealing with cases where both channel and floodplain flow occurs

Work with natural streams and floodways often requires consideration of a wide range of discharges. At the higher stages both channel and overbank or floodplain flow are involved. Usually the conditions are such that the channel and floodplain will have different degrees of retardance and, therefore, different *n* values. In such cases, the hydraulic computations will be improved by dividing the cross sections into parts or subdivisions having different *n* values.

The reason for and effect of subdividing cross sections is to permit the composite *n* for the reach to vary with stage above the bankfull stage. This effect is illustrated by example14A– 2. The usual practice is to divide the cross section into two parts; one subdivision being the channel portion and the other the floodplain. More than two subdivisions may be made if conditions indicate wide variations of *n*. However, in view of the practical aspects of the problem, more than three subdivisions would not normally be justified.

In estimating *n* for the channel subdivision, all of the factors described previously and all of the procedural steps would be considered. Although conditions might indicate some variation of *n* with stage in the channel, it is recommended that an average value of *n* be selected for use in the hydraulic computations for all stages.

In the case of floodplain subdivisions, the estimate of *n* would consider all factors except meandering. That is, the estimate would employ all of the procedural steps except step 6. Floodplain *n* values will normally be somewhat greater than the channel values. Agricultural floodplain conditions are not likely to indicate an *n* less than 0.05 to 0.06. Many cases will justify values in the 0.07 to 0.09 range, and cases calling for values as high as 0.15 to 0.20 may be encountered. These higher values apply primarily because of the relatively shallow depths of flow. The two factors requiring most careful consideration are obstructions and vegetation. Many agricultural floodplains have fairly dense networks of fences to be evaluated as obstructions in step 4. Vegetation probably would be judged on the basis of growing season conditions.

Field and office work

It is suggested that field parties record adequate notes on field conditions pertinent to the five factors affecting *n* at the time cross section surveys are being made. The actual estimates of *n* may then be made in the office. This will require training of both field and office personnel. The conditions to be covered by field notes and considered in the estimate of *n* apply to a reach of channel and floodplain. It is not adequate to consider only those conditions in the immediate vicinity of a cross section. Note the sketch on figure 14A–l. With cross sections located as shown, field notes should describe the channel and floodplain conditions through the reach indicated as a basis for estimating the *n* values (assuming subdivided sections) to be incorporated in the hydraulic computations at section 2.

Table 14A–1 Examples of effect of vegetation on *n*

Example no.	Names and descriptions of channels Names, plates, and tables Refer to USDA SCS (1963); USDA (1929)	Range in mean velocity	Range in hydraulic radius	Average value *n*	Modifying value
1	Kaskaskia Mutual Dredged Channel near Bondville, Illinois; page 1. Channel shape, approximately trapezoidal, approximate bottom width 10 feet and more than 8-foot depth. An estimated *n* for the channel without vegetation is 0.025.	0.89–1.15	1.36–4.35		
	a. Condition: badly obstructed by trees 1 to 6 inches in diameter on side slopes and edges of bottom; some weeds, but practically no grass; no foliage.			0.049	0.024
	b. Condition: as described in a, but with summer foliage and water weed on bottom along one-tenth of course.			0.067	0.042
2	Cummins Lake dredged channel near Gould, Arkansas; pages 19 and 20. Average cross section of channel resembles a parabola. At bankfull stage depth about 13 feet, top width about 75 feet.	0.53–1.82	2.41–6.23		
	a. Side slopes moderately irregular from erosion and sloughing; estimated *n* for channel without vegetation 0.035.				
	b. Dormant season. Willows about 1 year old and 6 to 10 feet high continuous along side slopes except for about the upper third of sides. No growth in a strip about 20 feet wide along bottom. No foliage.			0.056	0.021
	c. Growing season, otherwise vegetation same as above. Willows and some weeds in full foliage. No vegetation along bottom.			0.072	0.037
3	Natural channel of Embarras River near Charleston, Illinois, page 24. Channel shape, approximately trapezoidal, approximate bottom width 100 feet and depth 19 feet.	2.09–2.94	6.52–11.72		
	a. An estimate of *n* for this channel with no growth on the banks would be 0.025.				
	b. Condition: channel bottom comparatively clean, and smooth, upper part of side slope covered with large trees, natural channel.			0.032	0.007
4	Ditch No. 18 of Cypress Creek drainage district near Arkansas City, Arkansas; page 3. Average cross section is approximately triangular; at bankfull stage depth about 13 feet, top width about 70 feet.	0.47–1.08	1.91–4.99		
	a. Dredged channel about 8 years old. Side slopes moderately irregular. Estimated *n* for the channel without vegetation 0.035.				
	b. Dormant season. Practically the entire reach covered with trees, mostly willows and cottonwoods. Some dry weeds and brush. No foliage.			0.061	0.026
	c. Growing season. Vegetation described under b, in full foliage.			0.102	0.067

Table 14A–1 Examples of effect of vegetation on *n*—Continued

Example no.	Names and descriptions of channels Names, plates, and tables Refer to USDA SCS (1963); USDA (1929)	Range in mean velocity	Range in hydraulic radius	Average value *n*	Modifying value
5	Lake Fork special dredged channel near Bement, Illinois; pages 14 and 15. Average cross section is approximately parabolic; at bankfull stage depth about 13 feet, top width about 65 to 70 feet.	0.76–1.65	2.6–7.33		
	a. Dormant season. Channel cleared; practically no vegetation of any type in channel.			0.031	
	b. Growing season. Densely growing, bushy willows continuous along side slopes; some poplar saplings scattered among willows; no growth in a strip 20 to 30 feet wide along bottom. No foliage.			0.062	0.031
	c. Growing season. Vegetation described under b, in full foliage.			0.092	0.061
6	Ditch No. 1 of Little River drainage district near Chaffee, Missouri, page 4. Average cross section trapezoidal, side slopes about 1:1, bottom width about 10 feet, depth about 8 feet.	0.68–1.51	2.00–4.26		
	a. Channel newly cleared, practically no vegetation.			0.029	
	b. Dormant season. Dense, bushy willows continuous along side slopes; no foliage. No vegetation along bottom of channel.			0.071	0.042

Figure 14A–1 Selection of Manning's *n* for a cross section

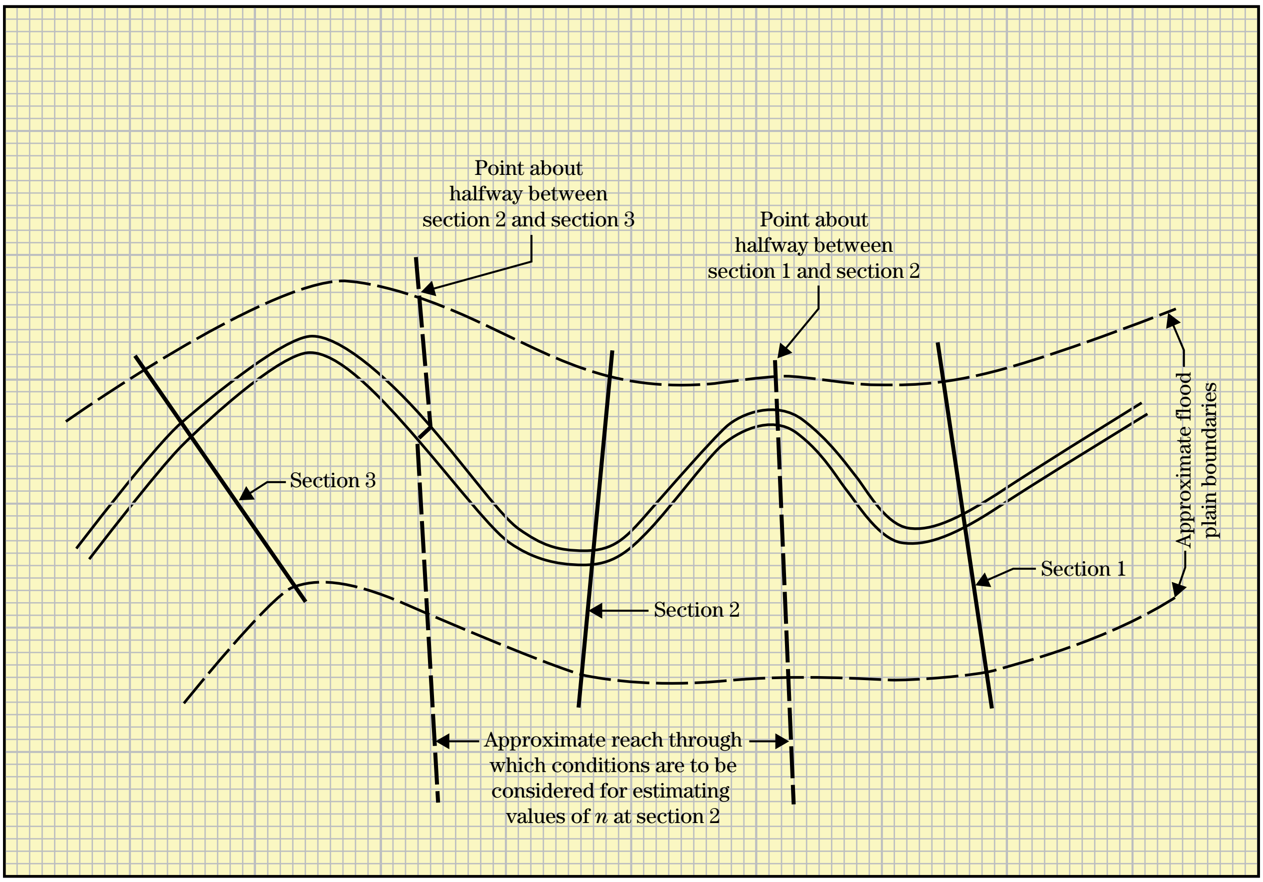


Figure 14A–2 shows a sample set of notes that illustrate the type of field information to be recorded as a basis for estimating *n*. Field staff should be trained to recognize and record in brief statements those conditions that are necessary for realistic evaluation of the five factors discussed under procedural steps 1 to 6.

Example 14A–1 Estimation of *n* for a reach

This example is based on a case where *n* has been determined so that comparison between the estimated and actual *n* can be shown.

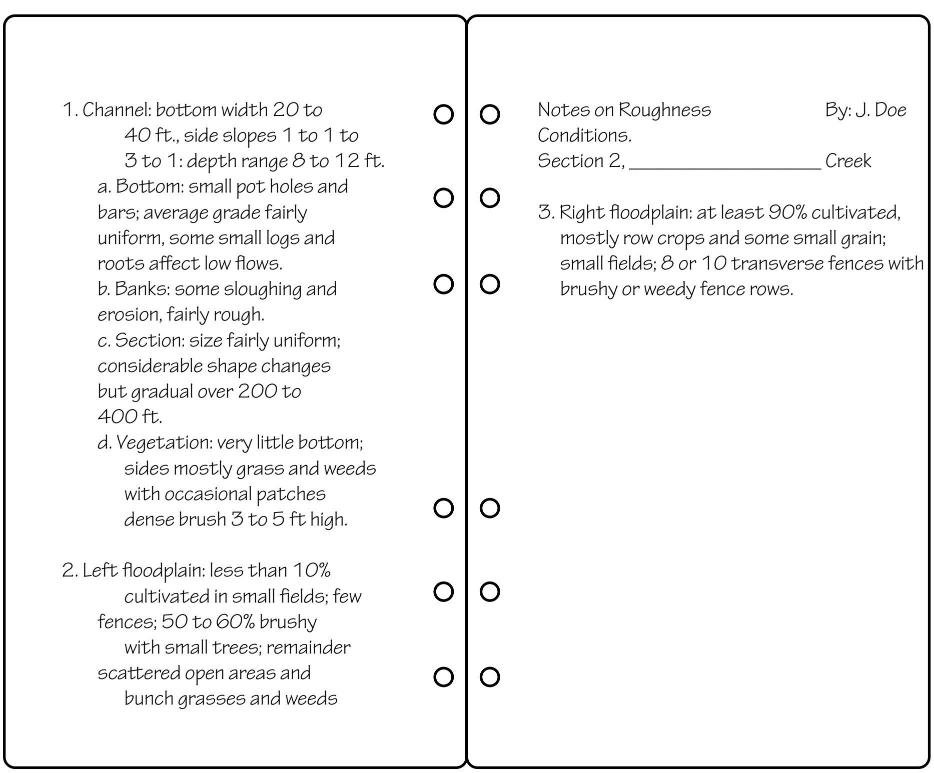
Channel: Camp Creek dredged channel near Seymour, Illinois; see USDA SCS (1963).

Description: Course straight; 661 feet long. Cross section, very little variation in shape; variation in size moderate, but changes not abrupt. Side slopes fairly regular, bottom uneven and irregular. Soil, lower part yellowish gray clay; upper part, light gray silty clay loam. Condition, side slopes covered with heavy growth of poplar trees 2 to 3 inches in diameter, large willows and climbing vines; thick growth of water weed on bottom; summer condition with vegetation in full foliage.

Average cross section approximates a trapezoid with side slopes about 1.5 to 1 and bottom width about 10 feet. At bankfull stage, average depth and surface width are about 8.5 and 40 feet, respectively.

Step	Remarks	Modifying values
1	Soil materials indicate minimum basic *n*	0.02
2	Description indicates moderate irregularity	0.01
3	Changes in size and shape judged insignificant	0.00
4	No obstructions indicated	0.00
5	Description indicates very high effect of vegetation	0.08
6	Reach described as straight	0.00
	Total estimated *n*	0.11

Figure 14A–2 Example of field notes describing roughness conditions


1. Channel: bottom width 20 to 40 ft., side slopes 1 to 1 to 3 to 1: depth range 8 to 12 ft.
   a. Bottom: small pot holes and bars; average grade fairly uniform, some small logs and roots affect low flows.
   b. Banks: some sloughing and erosion, fairly rough.
   c. Section: size fairly uniform; considerable shape changes but gradual over 200 to 400 ft.
   d. Vegetation: very little bottom; sides mostly grass and weeds with occasional patches dense brush 3 to 5 ft high.

2. Left floodplain: less than 10% cultivated in small fields; few fences; 50 to 60% brushy with small trees; remainder scattered open areas and bunch grasses and weeds

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Section 2, ________________ Creek

3. Right floodplain: at least 90% cultivated, mostly row crops and some small grain; small fields; 8 or 10 transverse fences with brushy or weedy fence rows.

USDA SCS (1963) gives the following determined values for n for this channel: for average depth of 4.6 feet $n = 0.095$; for average depth of 7.3 feet $n = 0.104$.

Example 14A–2 Effect of subdividing cross sections

The purpose of this example is to illustrate the effect of subdividing sections on the value of n for the complete section. It is not an illustration of hydraulic computations for determining water surface profiles or stage discharge relationships.

This illustration is based on the following:

- An actual stream cross section for which curves showing depth versus area and depth versus hydraulic radius for the channel and floodplain subdivisions and for the complete section are plotted on figure 14A–3. Values of n are: for the channel subdivision 0.04; for the floodplain subdivision 0.08.
- The conditions of uniform, steady flow are assumed.

Notation:

Q = discharge, ft³/s
A = cross section area, ft²
R = hydraulic radius, ft
S_o = channel slope, ft/ft
n = roughness coefficient

$$Q = \frac{1.486}{n} AR^{\frac{2}{3}} S_o^{\frac{1}{2}} \qquad \text{(eq. 14A–1)}$$

$$\text{Let Kd} = \frac{1.486}{n} AR^{\frac{2}{3}} \qquad \text{(eq. 14A–2)}$$

then:

$$Q = \text{Kd } S_o^{\frac{1}{2}} \qquad \text{(eq. 14A–3)}$$

Assume the conditions are such that it is desirable to recognize more than one subdivision, each having a different n. Let subscripts 1, 2, and 3 refer to the section subdivisions and subscript t to the total section.

From equation 14A–3:

$$Q = \left(\text{Kd}_1 + \text{Kd}_2 \ldots \text{Kd}_n\right) S_o^{\frac{1}{2}} = \sum \text{Kd } S_o^{\frac{1}{2}} \qquad \text{(eq. 14A–4)}$$

Table 14A–2 shows the computations for example 14A–2, and figure 14A–3 shows a plot of cross section properties for the complete section versus depth.

In natural streams, n normally shows a minor decrease as stage increases up to, or somewhat above, the bankfull stage, then appreciably increases as overbank stage increases. When n is significantly different for different parts of the cross section, subdivision of the cross section, as a basis for making the computations, automatically causes n_t to vary with stage above the bankfull stage. This is true although n_t is not computed in methods for determining water surface profiles. Note on figure 14A–3 that n_t, which has been computed in example 14A–2 for illustrative purposes, shows considerable increase with stage above the 10-foot depth and that this increase is automatically recognized by subdivision of the cross section.

The plot of hydraulic radius on figure 14A–3 illustrates a typical characteristic of natural streams. Note that the hydraulic radius for the complete section increases up to bankfull depth, then decreases through a limited range of depth, and again increases as depth of overbank flow increases.

This example also illustrates that recognition of high retardance for floodplain subdivisions by the use of relatively high n values does not cause n for the complete section, n_t, to be unreasonably high. In this case, the channel and floodplain are assigned n values of 0.04 and 0.08. The value of n_t ranges up to 0.072 as shown by table 14A–2 and figure 14A–3.

Figure 14A–3 Hydraulic properties of a subdivided cross section, example 14A–2

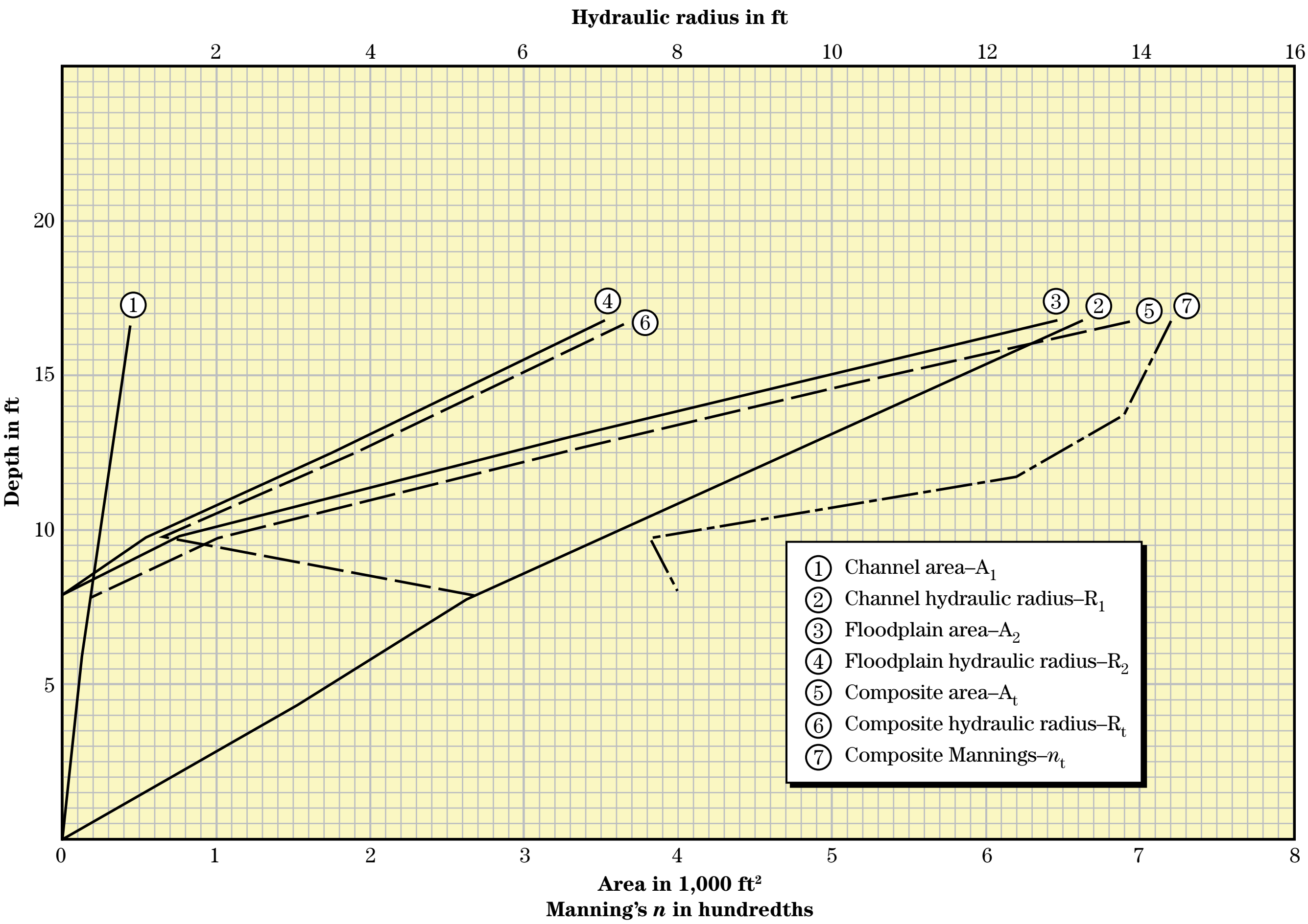

Table 14A–2 Computations for example 14A–2

Depth (ft)	A_1 (ft^2)	R_1 (ft)	$R_1^{2/3}$	Kd_1	A_2 (ft)	R_2 (ft)	$R_2^{2/3}$	Kd_2	ΣKd	A_t (ft^2)	R_t (ft)	$R_t^{2/3}$	K	n_t
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
0.0	0	0.0	0.0	0	0	0.0	0.0	0	0	0	0.0	0.0	0	0.0
4.7	90	3.33	2.23	7,456	0	0.0	0.0	0	7,456	90	3.33	2.23	298	0.04
7.8	180	5.29	3.036	20,302	0	0.0	0.0	0	20,302	180	5.29	3.036	812	0.04
9.7	240	7.06	3.68	32,813	750	1.06	1.04	14,493	47,296	990	1.31	1.197	1,761	0.037
11.7	300	8.82	4.269	47,576	2,238	2.88	2.024	84,149	131,726	2,538	3.14	2.144	8,087	0.061
13.7	360	10.59	4.822	64,495	3,853	4.58	2.758	197,381	261,876	4,213	4.82	2.854	17,864	0.068
16.7	450	13.22	5.591	93,467	6,488	7.08	3.687	444,353	537,821	6,938	7.30	3.763	38,797	0.072

$$Kd_1 = \frac{1.486}{0.04} A_1 R_1^{\frac{2}{3}} = 37.15 A_1 R_1^{\frac{2}{3}}$$

$$Kd_2 = \frac{1.486}{0.08} A_2 R_2^{\frac{2}{3}} = 18.58 A_2 R_2^{\frac{2}{3}}$$

$$K = 1.486 A_t R_t^{\frac{2}{3}}$$

$$n_t = \frac{K}{\sum Kd}$$