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Final Report

Design Manual for Low Water Stream Crossings

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report

College of Engineering
Iowa State University

The purpose of this manual is merely to provide guidelines for low water stream crossings. Rigid criteria for determining the applicability of a LWSC to a given site are not established nor is a "cookbook" procedure for designing a LWSC presented. Because conditions vary from county to county and from site to site within the county, judgment must be applied to the suggestions contained in this manual.

This edition of the manual, dated October 1983, has received final acceptance by the Iowa Highway Research Board. The August 1983 edition with a blue cover is invalid.

Final Report

Design Manual for Low Water Stream Crossings

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1. INTRODUCTION

1.1. Purpose of the Manual

The purpose of this manual is to provide design guidelines for low water stream crossings (LWSCs). Rigid criteria for determining the applicability of a LWSC to a given site are not established since each site is unique in terms of physical, social, economic, and political factors.

Because conditions vary from county to county, it is not the intent to provide a "cook-book" procedure for designing a LWSC. Rather, engineering judgment must be applied to the guidelines contained in this manual.

1.2. Definition of a LWSC

A LWSC is a stream crossing that will be flooded periodically and closed to traffic. Carstens (1981) has defined a LWSC as "a ford, vented ford (one having some number of culvert pipes), low water bridge, or other structure that is designed so that its hydraulic capacity will be insufficient one or more times during a year of normal rainfall."

In this manual, LWSC are subdivided into these same three main types: unvented fords, vented fords and low water bridges. Within the channel banks, an unvented ford can have its road profile coincident with the stream bed or can have its profile raised some height above the stream bed.

1.3. Components of a LWSC

A LWSC consists of several components: core material(s), foreslope surface, roadway surface, pipes (if it is a vented ford), and cutoff walls or riprap for protection against stream erosion. The core can consist of earth, sand, gravel, riprap, concrete, or a combination of these materials. Erosion protection for the foreslopes can consist of turf, riprap, soil cement, gabions, or concrete. The roadway surface can be composed of similar materials with the provision that a suitable riding surface be provided. The cost and availability of these materials vary from county to county; therefore, the exact composition of the core and surfacing will depend on local conditions. Pipes can be circular, oval, or arch and made of concrete, corrugated metal (CMP), or polyvinylchloride (PVC).

Protection against stream erosion can be provided by either cutoff walls or by armoring the stream bed. Cutoff walls can be constructed of either concrete or steel. The armoring could be riprap or gabions. Again, whether steel, concrete, or rock is used will depend on local cost and availability of materials and machinery such as pile drivers.

1.4. Rationale for Using a LWSC

Most counties (and many municipalities) have bridges that are no longer adequate and, therefore, are faced with a large capital expenditure if the same size replacement structure is proposed. A LWSC may be an attractive low cost alternative to replacing a costly bridge.

Many states have used LWSCs extensively and a number have been constructed in Iowa. Numerous existing bridges are obsolete and a prudent administrator of construction funds will look to alternatives. When is an obsolete bridge location a candidate for a LWSC? The ideal situation would be to close the road but this alternative is not always available. However, if loss of access for a short time is not a problem, the site may be a candidate for a LWSC.

A classic example of a LWSC candidate would be on a primitive road serving only as a field access for local farmers. During good weather conditions, a well-designed vented ford would provide adequate facilities for any traffic using the road. In fact, a LWSC might be superior to the typical obsolete bridge found at this site. This type of bridge might be a narrow roadway wood structure built just after the turn of the century. Farmers using modern farm equipment even have problems with modern bridges. Bridges were not designed for farm equipment with widths of 18 to 20 feet, and in some cases reaching 28 feet with axle loads approaching 80,000 lbs. As a consequence, when vandals set fire to a bridge, or heavy equipment causes it to fail structurally, the farmer may be better served by the LWSC.

During dry weather periods, the primitive road is passable by most vehicles and the LWSC provides a suitable stream crossing.

During periods of significant rainfall, since the primitive or unpaved road is not passable except by farm equipment or four-wheel drive vehicles, the closing of the flooded LWSC is not a problem to the traveling public.

However, not all obsolete bridges are on a primitive road serving only as a field access. Other potential locations for LWSC which may tolerate a short loss of access are those which have:

- o no residences with sole access over the LWSC
- o no critical school bus route
- o no recreation use
- o no critical mail route

If these uses do exist, the road may still be a potential candidate for a LWSC if an alternate route is available.

The size of the drainage area also can affect the decision as to whether a LWSC should be used. During high flows on a small watershed, floodwaters rise rapidly and subside rapidly, whereas on a larger watershed, flood waters rise more slowly and flow over the LWSC for a longer time. Thus, road closures for a short time due to a LWSC on a small watershed may be tolerated, whereas at a similar LWSC on a larger watershed, closures for a longer period of time while the high water overflows the road may not be tolerable.

The same type of reasoning concerning the effect of watershed size holds during low flow periods. The equations developed for Iowa are based on flow durations over the long term. Therefore, if a crossing was designed to be closed on the average of one week per year, during a dryer year it may not be closed at all, whereas during a wetter year it may be closed for a total of a month or so.

Streams in smaller watersheds also tend to dry up sooner than those in larger watersheds. During a wet period, flows may subside in some of these smaller streams, but rainfall in other portions of the larger watershed, that these smaller streams are tributary to, keeps water flowing in the larger stream at a rate which inundates the LWSC for longer periods of time. Thus, road closures for a short time on smaller watersheds may be tolerable, whereas the longer period of time flow overtops a LWSC in a larger watershed may not be tolerable.

Traffic volume as a criterion for LWSC use can be misleading. Significant volumes of traffic identify a user demand for that particular route. Closing a LWSC temporarily increases user costs by diverting traffic to another alternate route. Perhaps, more significantly, the larger volume of traffic increases the probability that a user will take chances and cross a LWSC when flooded.

Surfacing or pavement is not necessarily a criterion for LWSC locations. Obviously, an unsurfaced road indicates a route of lesser importance. In this case, periodic closing is probably of less concern to the user. On the other hand, a high type surfacing might indicate a high users' demand for improved facilities on an important route.

Other lower cost alternatives are available for smaller drainage areas other than replacing a bridge with a LWSC. One is to use a culvert designed for the 2-, 5-, or 10-year return period discharge with riprap on the foreslopes to protect the crossing against larger discharges. The road profile may or may not have a "dip" in it depending on conditions at the site. Another alternative is available if the valley upstream of the crossing can be used to store runoff temporarily for several hours. Depending on the volume of temporary storage existing at the site, a culvert could be designed and used for the 10-, 25-, or 50-year return period without water overtopping the roadway.

A LWSC may in fact be applicable in combination with an existing obsolete bridge. Consider the situation of a wood bridge with sub-standard width and structural capability to handle farm equipment. If this bridge were posted so as to preclude all vehicles but automobiles and a "shoo-fly" vented or unvented ford was provided adjacent to the bridge as shown in Fig. 1.1, both types of users would be served. When the LWSC was overtopped preventing farm equipment, trucks, or four-wheel drive vehicles from using it, there would probably be little demand for this type of service anyhow.

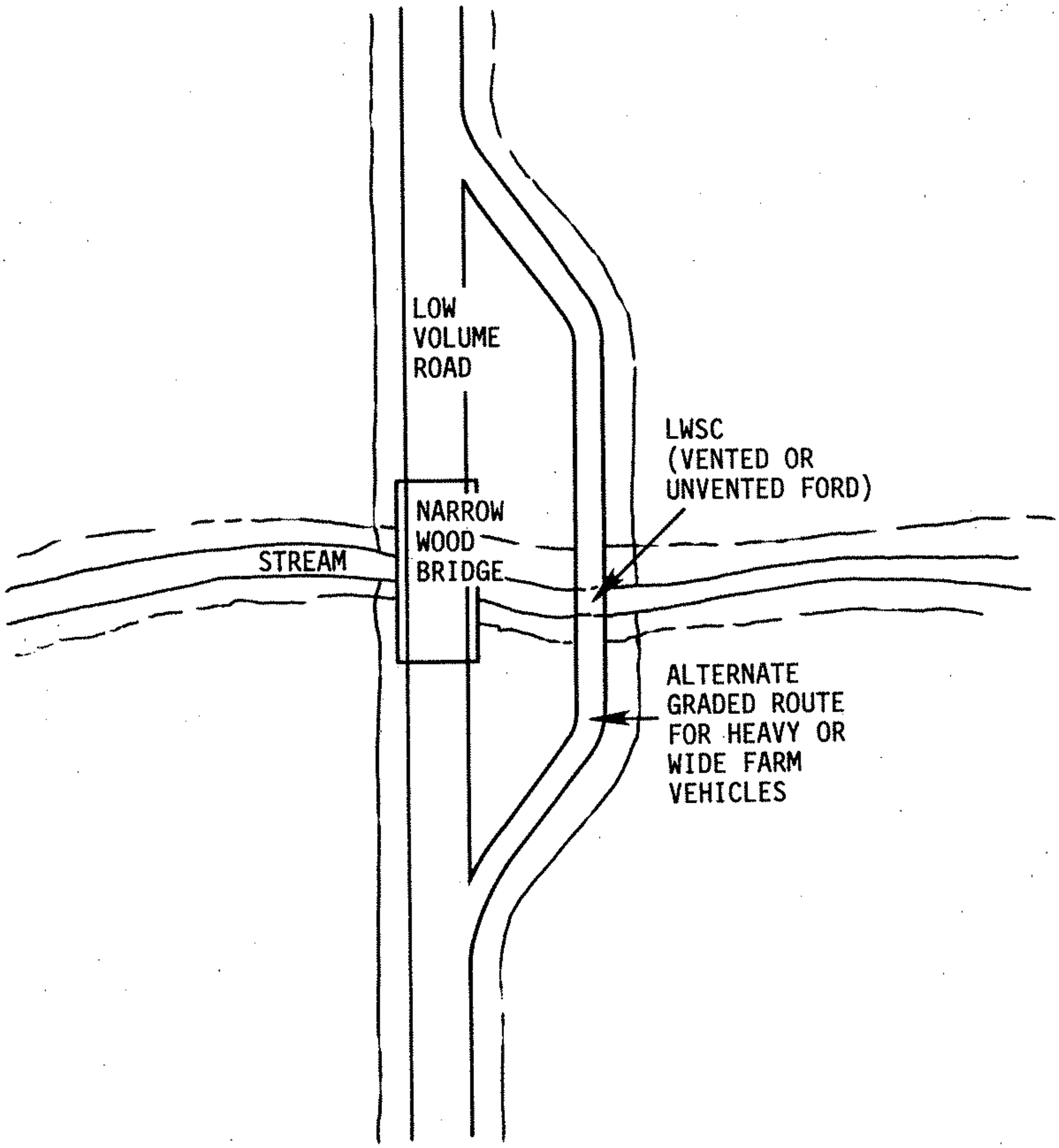


Figure 1.1. Combination obsolete bridge with alternate LWSC for farm equipment.

2. DESIGN CONCEPTS AND CRITERIA

This chapter sets forth the criteria, design concepts and data needed for the design of a LWSC. Its purpose is to provide an overview of the entire design process. Because each site is unique and each county has its own unique set of conditions, these criteria and concepts should be viewed as guidelines which lead to a well-designed, safe crossing. Each step in the design process then is discussed in detail in the following chapters.

2.1. General Criteria

1. Based on the study by Carstens (1981), with the adoption of the recommended regulatory sign and support resolution, the road will be closed when water is flowing across it. Because of this, for vented fords the headwater elevation for the selected overtopping frequency and estimated discharge must be at or slightly below the low point in the roadway. For unvented fords, a LWSC should only be used on those intermittent streams which are dry for significant portions of the year, since any time there is stream flow, water will be flowing over the roadway.
2. This overtopping discharge is based on the concept that the crossing will be closed a certain percent of the time. Since each site is unique and the decision on overtopping duration must be based on the existing physical, social, economic, and political factors present for that site and county, only general guidelines can be given for the allowable overtopping duration.

3. The assumption is made that the existing channel cross section is not altered, i.e., its width is not increased so that more pipes can be laid in the widened channel. However, the channel banks could be cut down to allow for proper approach grades.
4. The minimum depth of cover over the pipes in a vented ford is one foot.
5. Road grades, vertical curve lengths, and rideability reflect the low speeds allowed on these roads.
6. Flows overtopping the crossing should be controlled to minimize erosion so that damage is low and repair is easier. This can be done by keeping the difference between the upstream and downstream water surfaces to a minimum. One way to achieve this is to keep the difference between the low point in the roadway and the stream bed to a minimum.
7. Because alternative types of materials can be used in the construction of a LWSC, the availability and cost of these materials in different counties could lead to different decisions in these counties.
8. Based on the study by Carstens (1981), proper signing reduces the liability.
9. The type of material used to protect the LWSC from erosion could be influenced by the size and location of the county's maintenance force and the number of LWSCs in the county. Some crossings may need to be inspected for needed maintenance after a flood event. This maintenance could range from sediment and debris removal to major repairs. The time lapse between the flood event and the

road being reopened could be excessive if the number of LWSCs requiring significant maintenance is large and the maintenance force is small and located some distance away. How long a period of time is "excessive" is dependent on the site and the county's social and political climate.

2.2. Steps in Design

Figure 2.1 lists the eight general steps involved in the design of a LWSC. Each step is discussed briefly in the following paragraphs.

The location in Iowa is needed to determine in which hydrologic region the LWSC is located. The watershed size is measured in square miles. Two methods of obtaining the watershed area are given in a subsequent section. Both the hydrologic region and watershed area are used to estimate discharges and select crossing materials.

Most LWSC will be vented fords. Unvented fords could be closed much of the time because of the safety problems of driving through water. Therefore, they should be used only on those intermittent streams which are dry for the percent of time compatible with the uses of the road.

The allowable overtopping duration is a function of the several items discussed in the introduction. Each site is unique and the decision on the duration of overtopping must be based on the existing physical, social, economic, and political factors for that site and county. Once this decision is made, the overtopping discharge then can be estimated using equations developed by the U.S. Geological

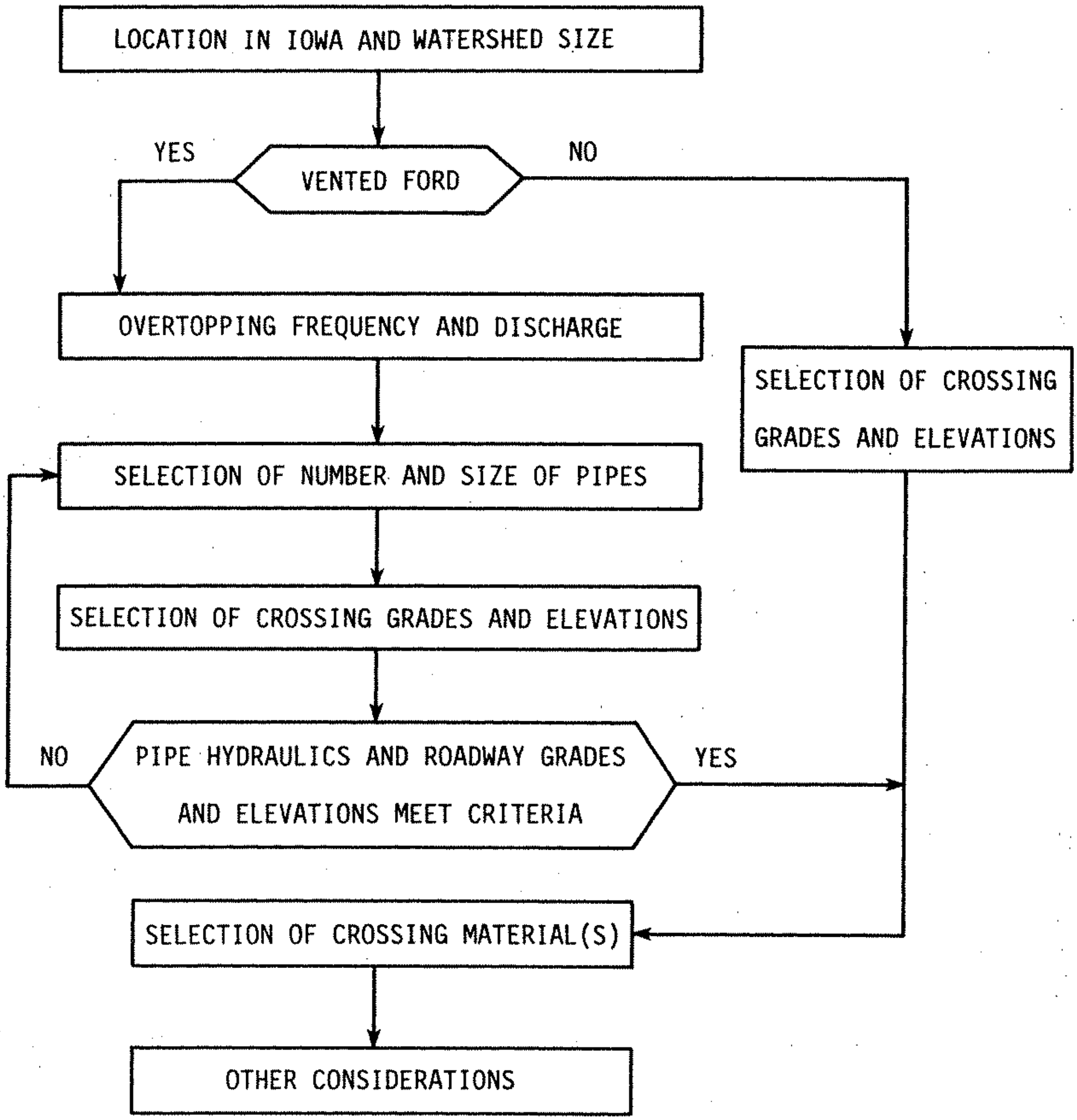


Fig. 2.1. General design steps for a low water stream crossing.

Survey for Iowa. For example, $Q_{10\%}$ means that the flow will be sufficient so that the crossing will be closed 10 percent of the time or on the average of about 37 days per year. The words "on the average" are used because these equations are based on long-term gaging station records. Heavier or lighter rainfall or snow melt events during any one year would mean that the crossing would be closed more or less than 37 days that year.

Using the overtopping discharge and the criteria listed in the previous section, the number and size of pipes and headwater depth can be determined from Herr and Bossy (1965), commonly known as HEC-5 or Bulletin 5. The pipe can be circular, oval, or arch and made of concrete, corrugated metal, or PVC. Each of these pipe shapes and materials can be analyzed using HEC-5 under both inlet control and outlet control. Field experience indicates that smaller pipe (12-inch to 18-inch) tends to clog less than larger pipe.

The crossing grades and elevations are a function of the physical characteristics of the existing channel and roadway and the overtopping discharge headwater depth. For vented fords, the low point in the roadway should be in the range of two to six feet above the stream bed, depending on the size of pipes, depth of cover over the pipes, roadway and surfacing material used, and depth of channel. Grades and lengths of curves are discussed in detail in Chapter 4.

Two criteria must be met as shown in Fig. 2.1: (1) the headwater depth for the number and size of pipes selected is at or slightly below the low point in the roadway and (2) the grades and length of the sag vertical curve must meet the rideability criterion. The possibility exists that in order to meet criterion number 2, the low point in the roadway has to be raised above the elevation needed for either the calculated headwater depth or minimum cover criteria. In this case, the possibility exists that the number and/or size of pipes could be reduced.

Material selection for the crossing foreslopes and roadway surface is a function of the channel velocity and tractive force. High flows (Q_{10} to Q_{50}) will usually govern but for large differences between headwater and tailwater depth, the velocity of the overtopping discharge ($Q_{50\%}$ to $Q_{1\%}$) plunging down the downstream foreslope could be the governing case. These materials can range from turf to concrete.

The other considerations include provisions to protect against stream erosion and seepage. This could consist of steel or concrete cutoff walls or riprap blankets. As indicated before, availability of material and equipment will vary from county to county; therefore, only general guidelines are included to indicate the items that should be taken into consideration before a decision is made.

The six general steps in the design of a vented ford are listed in Fig. 2.1. Chapter 3 contains a detailed description of steps 1, 2, and 4, the hydrologic and hydraulic portion of the design. Roadway geometrics, Step 3, is presented in Chapter 4. The last two steps are discussed in Chapter 5.

2.3. Data Requirements

2.3.1. Pipe Selection

If the LWSC will be a vented ford, the following data are needed to determine the number and size of pipes.

1. Location of site in Iowa
2. Watershed size in square miles
3. *Design overtopping duration* (Q_e)
4. Cross section and roughness coefficient (Manning's n) of existing channel at site
5. Slope of channel at site in feet per foot

The first three items are needed to estimate the overtopping discharge. The next two items are needed to determine the stage-discharge curve for the existing channel.

2.3.2. Roadway Geometry

The following are required in order to calculate the elevation of the low point of the LWSC:

1. Existing road or ground profile at site
2. *Tentative* crossing grades and elevations
3. Headwater depth

The following dimensions must be selected for the roadway cross section design at the LWSC:

1. Roadway width dimension
2. Roadway crown cross slope rate
3. Roadway foreslope rate

2.3.3. Material Selection

The following data are needed to determine the material, such as grass, riprap, and/or concrete, used to protect the roadway and foreslopes. Three methods are presented in this manual to select these materials and can be used for both vented and unvented fords. The data requirements for the first two methods are fewer since they are based on geomorphic relationships developed at existing gaging stations in Iowa.

Method 1

1. Location of site in Iowa
2. Watershed area in square miles

Method 2

1. Location of site in Iowa
2. Watershed area in square miles
3. Cross section of existing channel at site

Method 3

1. Location of site in Iowa
2. Watershed area in square miles
3. Depending on site location, profile of main channel slope from design point to watershed divide
4. Valley and channel cross section at site
5. Roughness coefficients (Manning's n) for valley and channel
6. Slope of channel at site in feet per foot
7. Final crossing grades and elevations

3. DESIGN OF A VENTED FORD

3.1. Step 1 Region and Drainage Area

The region in Iowa in which the vented ford is located is determined from Fig. 3.1. The drainage area of a stream at a specific location is that area, measured in a horizontal plane, enclosed by a topographic divide from which direct surface runoff from precipitation normally drains into the stream upstream from the specified point. For smaller watersheds, the drainage area can be determined by outlining the watershed on a 7.5 or 15 minute quadrangle map. The watershed is planimetered and the drainage area determined in square miles.

For watersheds larger than five square miles, Bulletin No. 7 (Larimer, 1957) can be used to determine the approximate drainage area. The final watershed size then can be determined by using quadrangle maps to determine the contributing area between the design point and the point shown in Bulletin No. 7.

3.2. Step 2 Flow-Duration Estimates

A flow-duration curve indicates the percent of time, within a certain period, in which given rates of flow were equaled or exceeded. An example of a flow-duration curve is shown in Figure 3.2. This curve indicates that, during the 32 year period of 1949-81, the average flow of Timber Creek near Marshalltown, Iowa, was at least 25 cubic feet per second (cfs) for 50 percent of the time. Similarly, it was at least 150 cfs for 10 percent of the time.

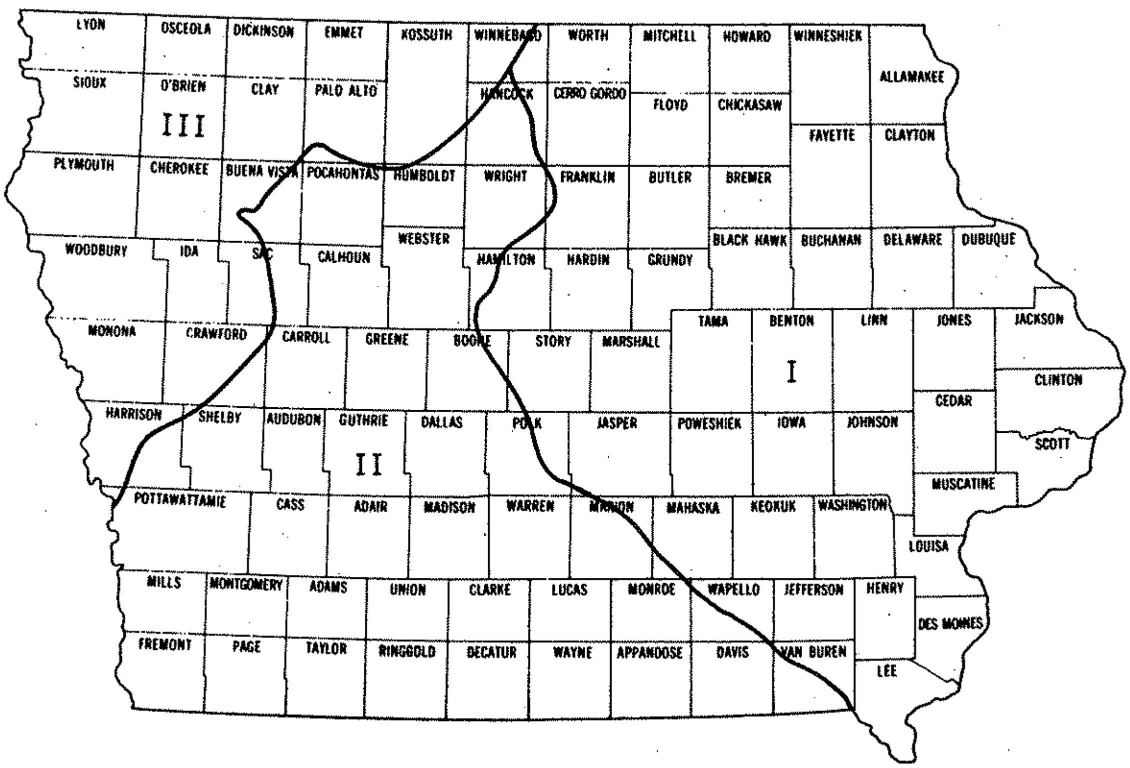


Fig. 3.1. Hydrologic regions for duration of discharge equations.

This curve was prepared by arranging the daily discharges collected during the period of 32 years in class intervals of ascending order of magnitude. Next, the percent of time during which the flow was equal to or greater than the lower limit of each class was determined. The results of these computations are summarized in tables or graphs. An example of a graphic presentation is shown in Fig. 3.2. The open circles in this figure are the estimated discharges for the 118 square mile Timber Creek watershed based on the regional equations presented below. Flow-duration information for daily flows collected at all the gaging stations in Iowa can be found in Lara (1979).

3.2.1. Flow-Duration Curves at Ungaged Sites

The preceding paragraph briefly described the preparation of a flow-duration curve at stream locations where recorded data of daily discharges are available. More frequently, flow-duration information is needed at stream crossings where no recorded data are available. The following procedure can be used to estimate flow-duration information for ungaged sites:

1. Using the map in Figure 3.1, identify the hydrologic region where your project site is located.
2. Determine the size of the drainage area at the site in square miles.
3. Select a value of e and the corresponding regression coefficients from Table 3.1, then solve the following equation.

$$Q_e = aA^b \tag{3.1}$$

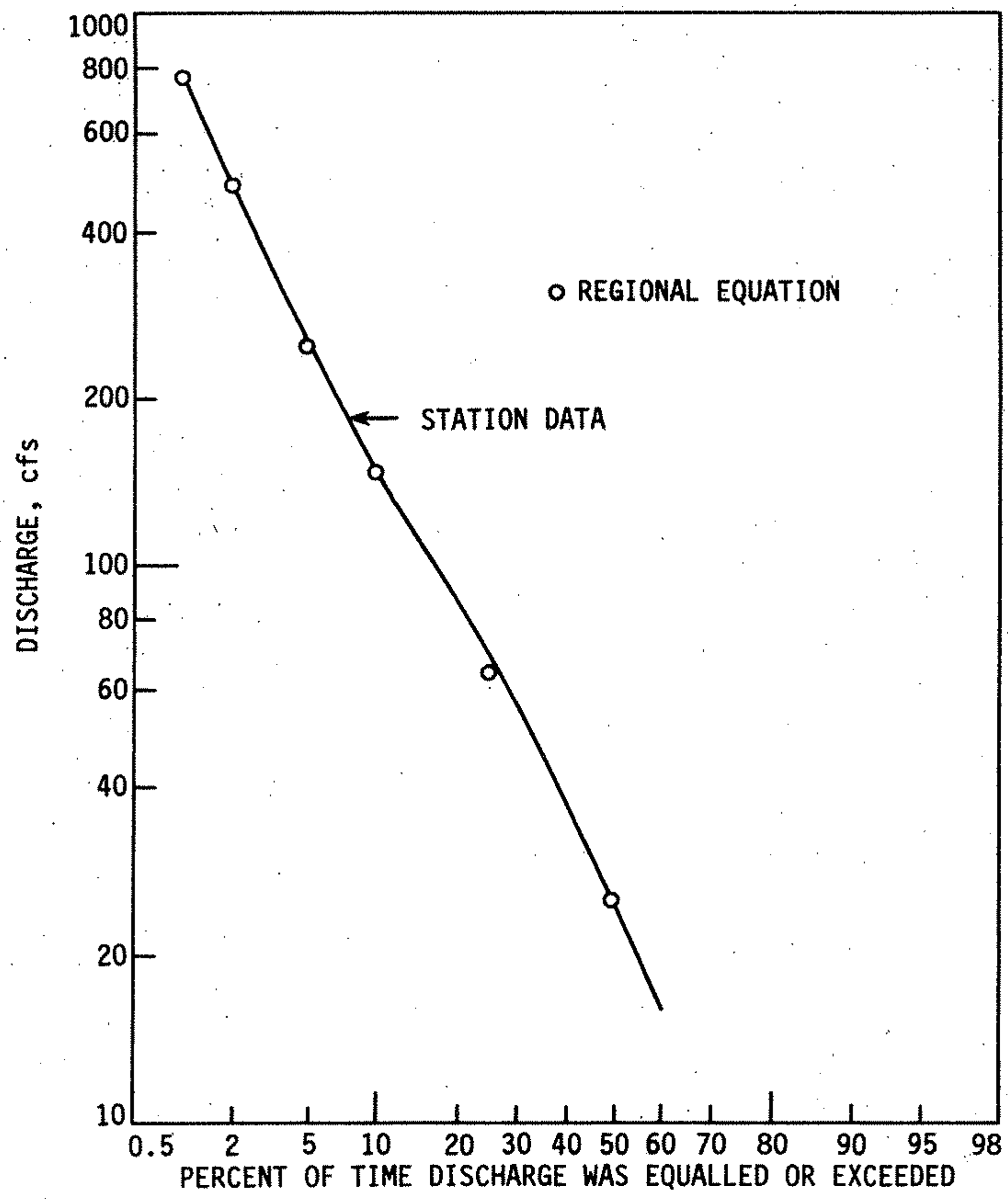


Figure 3.2. Duration curve of daily flow, Timber Creek near Marshalltown, Iowa, 1949-81.

where: Q is the discharge in cfs

e is the exceedance probability in percent

A is the drainage area in square miles

a and b are the regression coefficients. Values of a and b for each hydrologic region are listed in Table 3.1.

4. Repeat step 3 for other values of e.

Table 3.1. Regional regression coefficients for estimating duration of flows having the indicated exceedance probability.

Exceedance Probability e, %	Region I		Region II		Region III	
	a	b	a	b	a	b
(1)	(2)	(3)	(4)	(5)	(6)	(7)
50	0.17	1.05	0.06	1.09	0.015	1.24
25	0.52	1.01	0.24	1.06	0.04	1.25
10	1.37	0.98	0.91	1.00	0.15	1.19
5	2.58	0.96	2.26	0.95	0.33	1.15
2	6.78	0.90	6.78	0.90	1.23	1.06
1	13.50	0.85	13.50	0.85	3.56	0.96

In order to demonstrate this technique, the duration data for a six square mile watershed in Dallas County will be computed using these regional equations. Solving Eq. (3.1) by inserting the proper coefficients from Table 3.1 yields the following results.

$$Q_{50\%} = 0.06(6)^{1.09} = 0.4 \text{ cfs}$$

$$Q_{25\%} = 0.24(6)^{1.06} = 1.6 \text{ cfs}$$

$$Q_{10\%} = 0.91(6)^{1.00} = 5.5 \text{ cfs}$$

$$Q_{5\%} = 2.26(6)^{0.95} = 12.4 \text{ cfs}$$

$$Q_{2\%} = 6.78(6)^{0.90} = 34.0 \text{ cfs}$$

$$Q_{1\%} = 13.5(6)^{0.85} = 61.9 \text{ cfs}$$

These discharges are interpreted as follows. If the LWSC is designed for $Q_{25\%}$, the crossing will be closed on the average of three months each year. If the LWSC is designed for $Q_{2\%}$, the crossing will be closed on the average of seven days each year.

3.2.2. Limits of Application

The estimating equations presented in this section have been developed using data for unregulated natural streams in Iowa. Therefore, they are not applicable for streams controlled by man-made structures, such as diversion or storage reservoirs. Obviously, they are applicable only to streams in Iowa. Note also, that these equations define values for exceedance probabilities ranging from 1 to 50 percent. No attempt should be made to extrapolate the curve beyond the 50 percent exceedance.

If the project watershed is located near a regional boundary, there is the possibility that the stream begins or flows across another region. In this case, there may be a need to use equations for both regions and estimate the weighted average.

3.3. Step 3 Stage-Discharge Curves

A stage-discharge curve for a channel section is determined from a combination of Manning's equation and the continuity equation. This yields Eq. (3.2).

$$Q = 1.49\ AR^{2/3}\ S^{1/2}/n \qquad (3.2)$$

where

- Q = discharge in cfs
- A = cross sectional area of flow in square feet
- R = A/WP = hydraulic radius in feet
- WP = wetted perimeter in feet
- S = channel slope at site in feet per foot
- n = Manning's roughness coefficient, dimensionless

The stage-discharge curve is developed by assuming increasing values of depth, solving Eq. (3.2) for each depth, then plotting depth vs discharge with depth as the ordinate.

The channel cross section and slope (low water surface profile) at the site are measured in the field. Field observations also are made to allow estimation of the roughness coefficient. Calculations for area and wetted perimeter are made by plotting the channel cross section as a series of straight lines, then using simple geometric shapes.

The roughness coefficient is a function of channel material, degree of irregularity in channel cross section surface, variation in cross section along the channel's length, effect of obstructions, height of vegetation, and degree of channel meandering. These factors are combined in Eq. (3.3). Values for these factors are obtained from Table 3.2, which was taken from Chow (1959). Table 3.3 is a list of roughness coefficients for various kinds of channels, which was also taken from Chow (1959).

$$n = (n_o + n_1 + n_2 + n_3 + n_4)\ M_5 \qquad (3.3)$$

The development of a stage-discharge curve is shown in the following example. Assume a channel has a flat bottom with a width of 14 feet, a depth of 5 feet and 2:1 (horizontal to vertical) side slopes. The channel slope is 14 feet per mile or 0.00265 feet per foot. The roughness coefficient is 0.035. Determine the stage-discharge curve for each one-half foot of depth.

Substituting these values into Eq. (3.2) yields Eq. (3.4).

$$Q = 1.49\ AR^{2/3}\ (0.00265)^{1/2}/0.035 = 2.19\ AR^{2/3} \qquad (3.4)$$

The area of this trapezoid is

$$A = \frac{d}{2}\ (14 + 14 + 2 \times 2d) = 14d + 2d^2 \qquad (3.5)$$

where

d is the depth of flow in feet

Table 3.2. Values for the computation of the roughness coefficient (after Chow, 1959).

Channel Conditions			Values
Material involved	Earth	n_0	0.020
	Rock cut		0.025
	Fine gravel		0.024
	Coarse gravel		0.028
Degree of irregularity	Smooth	n_1	0.000
	Minor		0.005
	Moderate		0.010
	Severe		0.020
Variations of channel cross section	Gradual	n_2	0.000
	Alternating occasionally		0.005
	Alternating frequently		0.010-0.015
Relative effect of obstructions	Negligible	n_3	0.000
	Minor		0.010-0.015
	Appreciable		0.020-0.030
	Severe		0.040-0.060
Vegetation	Low	n_4	0.005-0.010
	Medium		0.010-0.025
	High		0.025-0.050
	Very high		0.050-0.100
Degree of meandering	Minor	m_5	1.000
	Appreciable		1.150
	Severe		1.300

Table 3.3. Values of the roughness coefficient n (after Chow, 1959).

Type of Channel and Description	Minimum	Normal	Maximum
C. EXCAVATED OR DREDGED			
a. Earth, straight and uniform			
1. Clean, recently completed	0.016	0.018	0.020
2. Clean, after weathering	0.018	0.022	0.025
3. Gravel, uniform section, clean	0.022	0.025	0.030
4. With short grass, few weeds	0.022	0.027	0.033
b. Earth, winding and sluggish			
1. No vegetation	0.023	0.025	0.030
2. Grass, some weeds	0.025	0.030	0.033
3. Dense weeds or aquatic plants in deep channels	0.030	0.035	0.040
4. Earth bottom and rubble sides	0.028	0.030	0.035
5. Stony bottom and weedy banks	0.025	0.035	0.040
6. Cobble bottom and clean sides	0.030	0.040	0.050
c. Dragline-excavated or dredged			
1. No vegetation	0.025	0.028	0.033
2. Light brush on banks	0.035	0.050	0.060
d. Rock cuts			
1. Smooth and uniform	0.025	0.035	0.040
2. Jagged and irregular	0.035	0.040	0.050
e. Channels not maintained, weeds and brush uncut			
1. Dense weeds, high as flow depth	0.050	0.080	0.120
2. Clean bottom, brush on sides	0.040	0.050	0.080
3. Same, highest stage of flow	0.045	0.070	0.110
4. Dense brush, high stage	0.080	0.100	0.140
D. NATURAL STREAMS			
D-1. Minor streams (top width at flood stage <100 ft)			
a. Streams on plain			
1. Clean, straight, full stage, no rifts or deep pools	0.025	0.030	0.033
2. Same as above, but more stones and weeds	0.030	0.035	0.040
3. Clean, winding, some pools and shoals	0.033	0.040	0.045
4. Same as above, but some weeds and stones	0.035	0.045	0.050
5. Same as above, lower stages, more ineffective slopes and sections	0.040	0.048	0.055
6. Same as 4, but more stones	0.045	0.050	0.060

Table 3.3. Continued.

Type of Channel and Description	Minimum	Normal	Maximum
7. Sluggish reaches, weedy, deep pools	0.050	0.070	0.080
8. Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.075	0.100	0.150
b. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stages			
1. Bottom: gravels, cobbles, and few boulders	0.030	0.040	0.050
2. Bottom: cobbles with large boulders	0.040	0.050	0.070
D-2. Flood plains			
a. Pasture, no brush			
1. Short grass	0.025	0.030	0.035
2. High grass	0.030	0.035	0.050
b. Cultivated areas			
1. No crop	0.020	0.030	0.040
2. Mature row crops	0.025	0.035	0.045
3. Mature field crops	0.030	0.040	0.050
c. Brush			
1. Scattered brush, heavy weeds	0.035	0.050	0.070
2. Light brush and trees, in winter	0.035	0.050	0.060
3. Light brush and trees, in summer	0.040	0.060	0.080
4. Medium to dense brush, in winter	0.045	0.070	0.110
5. Medium to dense brush, in summer	0.070	0.100	0.160
d. Trees			
1. Dense willows, summer, straight	0.110	0.150	0.200
2. Cleared land with tree stumps, no sprouts	0.030	0.040	0.050
3. Same as above, but with heavy growth of sprouts	0.050	0.060	0.080
4. Heavy stand of timber, a few down trees, little undergrowth, flood stage below branches	0.080	0.100	0.120

Table 3.3. Continued.

Type of Channel and Description	Minimum	Normal	Maximum
5. Same as above, but with flood stage reaching branches	0.100	0.120	0.160
D-3. Major streams (top width at flood stage >100 ft). The n value is less than that for minor streams of similar description, because banks offer less effective resistance.			
a. Regular section with no boulders or brush	0.025	.....	0.060
b. Irregular and rough section	0.035	.....	0.100

The wetted perimeter is determined from Eq. (3.6) and is equal to the bottom width plus the length along both side slopes which is wetted. The length along the side slope is calculated using the Pythagorean Theorem.

$$WP = 14 + 2\ (d^2 + (2d)^2)^{1/2} = 14 + 4.47d \qquad (3.6)$$

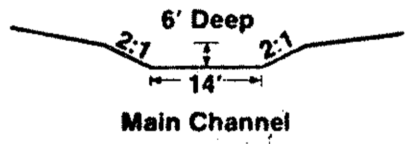
The calculations for this stage-discharge curve are shown in Table 3.4. The stage-discharge curve is obtained by plotting column 1 against column 6.

A computer program which calculates the stage-discharge curve for any type of channel or valley cross section is available from the Iowa DOT. The input and output for the above example are shown in Tables 3.5 and 3.6, respectively.

Table 3.4. Calculations for a stage-discharge curve.

d ft (1)	A sq ft (2)	WP ft (3)	R ft (4)	$R^{2/3}$ (5)	Q cfs (6)	V fps (7)
0.0	0.0	0.0	0.00	0.00	0	0.0
0.5	7.5	16.2	0.46	0.60	10	1.3
1.0	16.0	18.5	0.86	0.91	32	2.0
1.5	25.5	20.7	1.23	1.15	64	2.5
2.0	36.0	22.9	1.57	1.35	107	3.0
2.5	47.5	25.2	1.88	1.52	159	3.3
3.0	60.0	27.4	2.19	1.69	222	3.7
3.5	73.5	29.6	2.48	1.83	295	4.0
4.0	88.0	31.9	2.76	1.97	380	4.3

Table 3.5. Input for DOT stage-discharge curve program




```
logon br03/br4942

BR03 LOGON IN PROGRESS AT 13:49:32 ON APRIL 7, 1982
NO BROADCAST MESSAGES
READY
val

ENTER JOB IDENTIFICATION.
MAXIMUM OF 63 CHARACTERS
:
low water crossing example
ANY CHANGES? ENTER Y OR N. : n
ENTER NUMBER OF CROSS SECTION SHOTS.   : 4
ANY CHANGE?   : n
ENTER CROSS SECTION SHOTS, OFFSET FOLLOWED BY ELEV.
X(  1)  Y(  1)  :0 6
X(  2)  Y(  2)  :12 0
X(  3)  Y(  3)  :26 0
X(  4)  Y(  4)  :38 6
ANY CHANGES?  : n
ENTER SLOPE IN FT./MI.  : 14
ANY CHANGE? (Y OR N)  : n
ENTER NUMBER OF SECTIONS.  : 1
ANY CHANGE?  : n
ENTER SECTION DISTANCE AND N VALUE.
D(  1)  N(  1)  :38 .035
ANY CHANGES?  : n
ENTER STAGE DATA. (HIGH ELEV, LOW ELEV, AND INCREMENT)
MAKE SURE HIGH ELEV IS LOWER THAN HIGHEST CROSS SECTION SHOTS.
:
4 0 .25
ANY CHANGES?  : n

IS THERE A LOW POINT ON THE FLOOD PLAIN? (Y OR N)  : n
```

Table 3.6. Output from DOT stage-discharge curve program.

Low water crossing example		Section	Discharge	Velocity	Conveyance	N value	
Stage elev.	0.00						
		1	0 CFS	0.0 FPS	0 sq. ft.	0	0.0350
		Total	0 CFS	0.0 FPS	0 sq. ft.	0	
Stage elev.	0.25						
		1	3 CFS	0.8 FPS	4 sq. ft.	60	0.0350
		Total	3 CFS	0.8 FPS	4 sq. ft.	60	
Stage elev.	0.50						
		1	10 CFS	1.3 FPS	7 sq. ft.	191	0.0350
		Total	10 CFS	1.3 FPS	7 sq. ft.	191	
Stage elev.	0.75						
		1	20 CFS	1.7 FPS	12 sq. ft.	379	0.0350
		Total	20 CFS	1.7 FPS	12 sq. ft.	379	
Stage elev.	1.00						
		1	32 CFS	2.0 FPS	16 sq. ft.	619	0.0350
		Total	32 CFS	2.0 FPS	16 sq. ft.	619	
Stage elev.	1.25						
		1	47 CFS	2.3 FPS	21 sq. ft.	909	0.350
		Total	47 CFS	2.3 FPS	21 sq. ft.	909	
Stage elev.	1.50						
		1	64 CFS	2.5 FPS	25 sq. ft.	1247	0.0350
		Total	64 CFS	2.5 FPS	25 sq. ft.	1247	
Stage elev.	1.75						
		1	84 CFS	2.7 FPS	31 sq. ft.	1634	0.0350
		Total	84 CFS	2.7 FPS	31 sq. ft.	1634	
Stage elev.	2.00						
		1	107 CFS	3.0 FPS	36 sq. ft.	1069	0.0350
		Total	107 CFS	3.0 FPS	36 sq. ft.	1069	
Stage elev.	2.25						
		1	131 CFS	3.2 FPS	42 sq. ft.	2554	0.0350
		Total	131 CFS	3.2 FPS	42 sq. ft.	2554	
Stage elev.	2.50						
		1	159 CFS	3.3 FPS	47 sq. ft.	3087	0.0350
		Total	159 CFS	3.3 FPS	47 sq. ft.	3087	
Stage elev.	2.75						
		1	189 CFS	3.5 FPS	54 sq. ft.	3671	0.0350
		Total	189 CFS	3.5 FPS	54 sq. ft.	3671	
Stage elev.	3.00						
		1	222 CFS	3.7 FPS	60 sq. ft.	4306	0.0350
		Total	222 CFS	3.7 FPS	60 sq. ft.	4306	
Stage elev.	3.25						
		1	257 CFS	3.9 FPS	67 sq. ft.	4992	0.0350
		Total	257 CFS	3.9 FPS	67 sq. ft.	4992	
Stage elev.	3.50						
		1	295 CFS	4.0 FPS	73 sq. ft.	5731	0.0350
		Total	295 CFS	4.0 FPS	73 sq. ft.	5731	
Stage elev.	3.75						
		1	336 CFS	4.2 FPS	81 sq. ft.	6523	0.0350
		Total	336 CFS	4.2 FPS	81 sq. ft.	6523	
Stage elev.	4.00						
		1	380 CFS	4.3 FPS	88 sq. ft.	7371	0.0350
		Total	380 CFS	4.3 FPS	88 sq. ft.	7371	

3.4. Step 4 Number and Size of Pipes

Determining the number and size of pipes for a particular site is a trial and error process. Several items must be kept in mind: (1) the total width of pipes, including the spaces between them, must be less than the width of the existing channel, (2) the headwater depth controls the low point in the roadway, (3) the pipes can operate under either inlet control or outlet control, (4) pipe lengths are short, but differences in friction losses due to pipe material still could be significant, (5) a large difference between the low point in the roadway and the downstream water surface increases the erosion potential on the downstream foreslope, and (6) a large difference between the low point in the roadway and the stream bed increases the volume of material needed in the crossing and, thus, its cost.

The trial and error process begins by determining headwater depths for the estimated overtopping discharge and assumed combinations of pipe material, number, and size operating under inlet control. The results are reviewed in light of the above items and the several combinations reduced to the few best alternatives. These alternatives are checked for outlet control and the final type, size, and number of pipes selected. If the final low point in the roadway is higher than the calculated headwater depth due to roadway criteria, then the possibility exists that the number and/or size of pipes could be reduced.

The information needed to determine pipe size is available in Herr and Bossy (1965), commonly known as Hydraulic Engineering Circular No. 5 (HEC-5) or Bulletin 5. The equations needed to determine the depth of flow over the roadway are presented later in this section. It is assumed that users of this manual are familiar with Bulletin No. 5.

The following example illustrates the design process outlined above. The selected site is located in Dallas County and has a tributary area of six square miles. Based on conditions existing at the site, the decision has been made to design this LWSC to be closed about one week per year on the average, the two percent flow duration. From section 3.2.1, $Q_{2\%}$ is 34 cfs and $Q_{1\%}$ is 62 cfs. Several sizes of CMP are already on hand. Assume the pipes will be mitered to the 2:1 crossing foreslopes. The channel cross section is the one illustrated in section 3.3. Table 3.4 contains the stage-discharge curve calculations for this channel.

Arbitrarily select several sizes and number of pipes and assume that they are operating under inlet control. Then using the appropriate chart in Bulletin 5, determine the headwater depth for each combination. The results are listed in Table 3.7. Columns 1 and 2 are assumed values. The discharge flowing through each pipe is assumed to be the total discharge divided by the number of pipes. Column 3 is obtained from Chart 5 in Bulletin 5, included here as Figure 3.3. The headwater (HW) in column 4 is equal to the value in column 3 multiplied by the pipe diameter in feet.

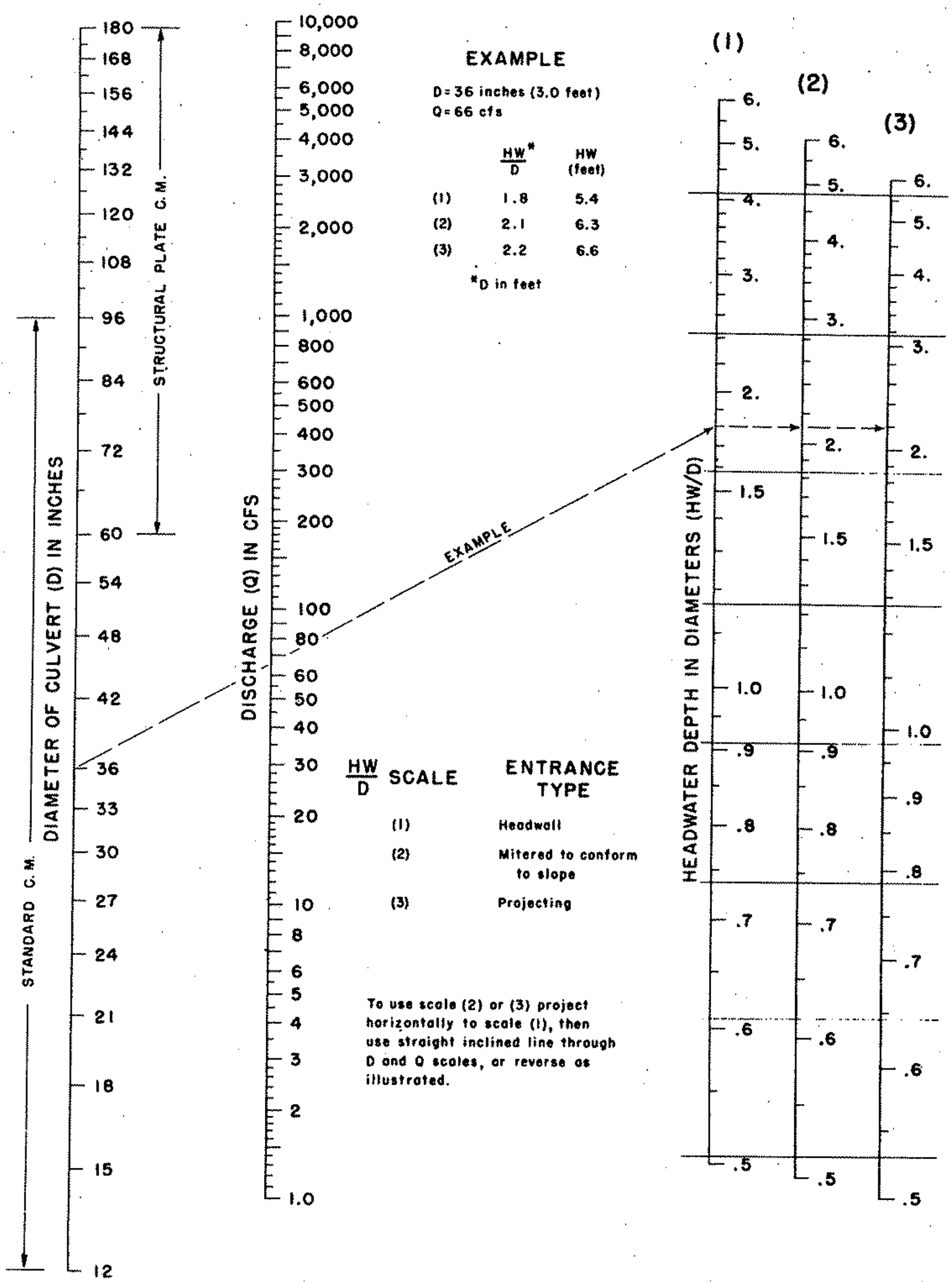


Fig. 3.3. Headwater depth for C.M. pipe culverts with inlet control.

Table 3.7. Headwater depths for various number and sizes of CMP operating under inlet control.

Diameter in. (1)	Number (2)	HW/D (3)	HW ft (4)
12	3	8.0	8.0
12	4	5.0	5.0
12	5	3.4	3.4
15	3	3.5	4.4
15	4	2.4	3.0
15	5	1.83	2.3
18	3	1.95	2.9
18	4	1.37	2.1
18	5	1.07	1.6
21	3	1.23	2.2
21	4	0.94	1.6
21	5	0.81	1.4

Since the channel width is 14 feet, all the combinations listed in Table 3.7 will work. Because the channel depth is five feet, all the 12-inch pipes and the three 15-inch pipes are eliminated since too little of the channel depth would remain above the low point in the roadway. Because the minimum depth of cover over the pipes is one foot, all the 21-inch pipes and the five 18-inch pipes are eliminated since the low point in the roadway would be too far above the design headwater depth. This leaves four alternatives to be checked for outlet control: the four and five 15-inch pipes and the three and four 18-inch pipes.

Headwater computations for outlet control are summarized in a form contained in Bulletin 5 and included here as Table 3.8. K_e is an entrance loss coefficient obtained from Table 1 in Bulletin 5. The head loss, H, is obtained from Chart 11 in Bulletin 5, included here as Figure 3.4. Critical depth, d_c, is obtained from Chart 16 in Bulletin 5 and is included here as Figure 3.5. The next column in Table 3.8 is the average of critical depth and the pipe diameter. The tailwater depth, TW, for the total discharge is taken from the channel stage-discharge curve. Figure 3.6 was drawn by plotting columns 1 and 6 of Table 3.4. H_o is the greater of $(d_c + D)/2$ and TW as explained in Bulletin 5. LS_o is the product of the length and slope of the pipe. The headwater depth for culverts operating under outlet control is computed using Eq. (3.7).

$$HW = H + h_o - LS_o \tag{3.7}$$

From the calculations shown in Table 3.8, only the four 18-inch pipes are acceptable since the others would leave too little of the channel remaining above the low point in the roadway. Note that in this example the tailwater depth does not govern.

For those users not familiar with Bulletin 5, Tables 3.9 and 3.10 provide road maps for the design of box and pipe culverts, respectively.

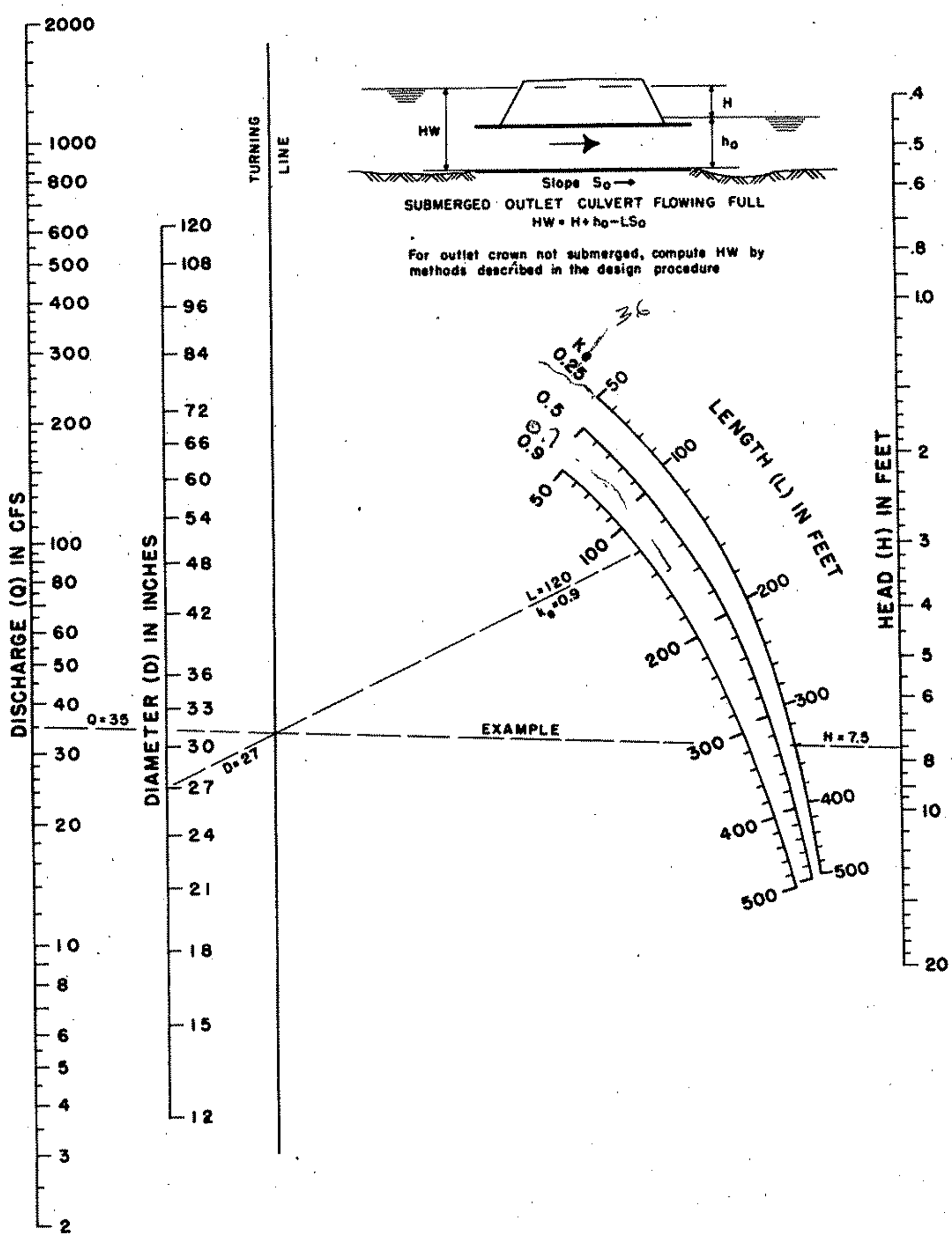


Fig. 3.4. Head for standard C.M. pipe culverts flowing full (n = 0.024).

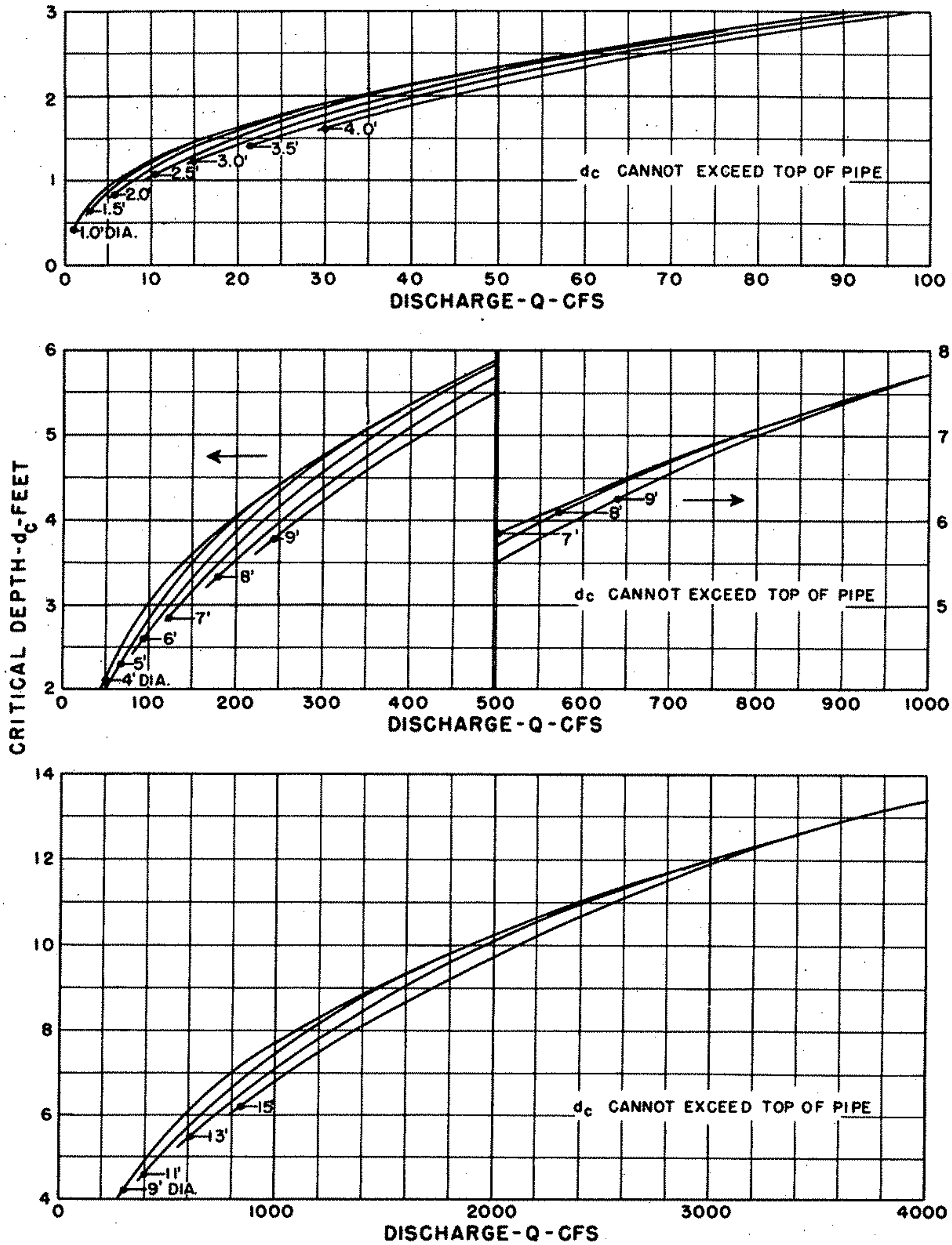


Fig. 3.5. Critical depth for circular pipes.

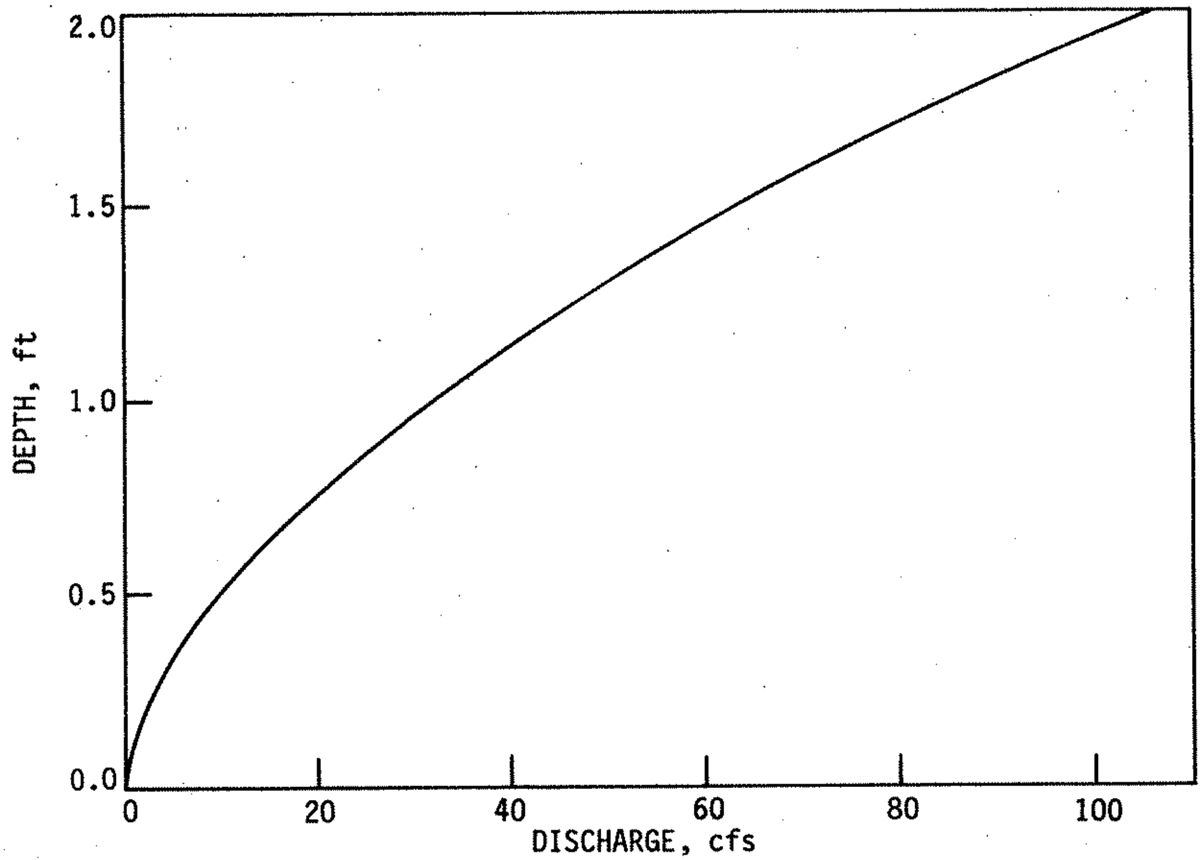


Figure 3.6. Stage-discharge curve.

Table 3.8. Computation form for culvert design.

PROJECT: LWSC

DESIGNER: Rossmiller

DATE: ______

HYDROLOGIC AND CHANNEL INFORMATION

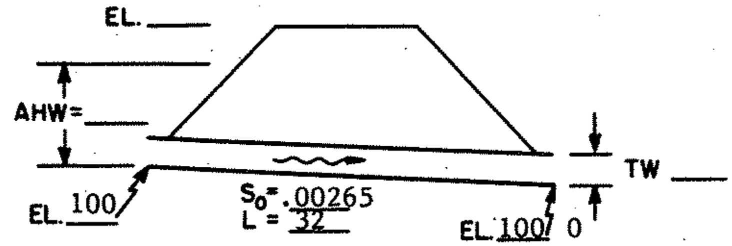
Q_1 = ______ TW_1 = ______

Q_2 = ______ TW_2 = ______

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

SKETCH

STATION: ______



MEAN STREAM VELOCITY = ______

MAX. STREAM VELOCITY = ______

CULVERT DESCRIPTION (ENTRANCE TYPE)	Q	SIZE	HEADWATER COMPUTATION											CONTROLLING HW	OUTLET VELOCITY	COST	COMMENTS
			INLET CONT.		OUTLET CONTROL $HW=H+h_o-LS_o$												
			$\frac{HW}{D}$	HW	K_e	H	d_c	$\frac{d_c+D}{2}$	TW	h_o	LS_o	HW					
CMP-mitered	8.5	15			0.7	3.4	1.2	1.2	1.1	1.2	0.1	4.5				
CMP-mitered	6.8	15			0.7	2.2	1.0	1.1	1.1	1.1	0.1	3.2				
CMP-mitered	11.3	18			0.7	2.3	1.3	1.4	1.1	1.4	0.1	3.6				
CMP-mitered	8.5	18			0.7	1.3	1.1	1.3	1.1	1.3	0.1	2.5				

SUMMARY & RECOMMENDATIONS:

Table 3.9. Guidelines for the design of box culverts using Herr and Bossy (1964).

PROJECT: LWSC

DESIGNER: Rossmiller

DATE: ______

HYDROLOGIC AND CHANNEL INFORMATION

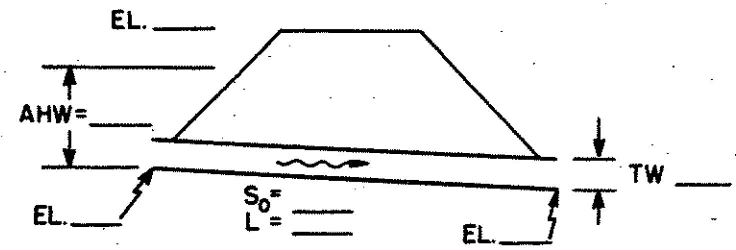
Q_1 = ______ TW_1 = ______

Q_2 = ______ TW_2 = ______

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

SKETCH

STATION : ______



MEAN STREAM VELOCITY= ______

MAX. STREAM VELOCITY= ______

CULVERT DESCRIPTION (ENTRANCE TYPE)	Q	SIZE	HEADWATER COMPUTATION										CONTROLLING HW	OUTLET VELOCITY	COST	COMMENTS
			INLET CONT.		OUTLET CONTROL HW=H + h_o - LS_o											
			$\frac{HW}{D}$	HW	K_e	H	d_c	$\frac{d_c+D}{2}$	TW	h_o	LS_o	HW				
Box	Assigned	Trial	Chart 1 Page 5-21	Compute D X $\frac{HW}{D}$	from Table 1 Page 5-49	Chart 8 Page 5-31	Chart 15 Page 5-38 in manual	Compute	from Stage-Discharge Curve	Larger of TW or $\frac{d_c + D}{2}$ Page 5-10	Compute (length X slope)	Compute: HW = H + H_o - LS_o	Larger value of inlet or outlet control Page 5-11, 5-12			

SUMMARY & RECOMMENDATIONS:

Table 3.10. Guidelines for the design of pipe culverts using Herr and Bossy (1964).

PROJECT: LWSC

DESIGNER: Rossmiller

DATE: ______

HYDROLOGIC AND CHANNEL INFORMATION

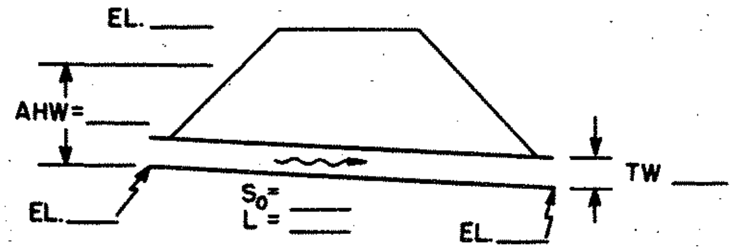
Q_1 = ______ TW_1 = ______

Q_2 = ______ TW_2 = ______

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

SKETCH

STATION: ______



MEAN STREAM VELOCITY = ______

MAX. STREAM VELOCITY = ______

CULVERT DESCRIPTION (ENTRANCE TYPE)	Q	SIZE	HEADWATER COMPUTATION: INLET CONT. $\frac{HW}{D}$	INLET CONT. HW	OUTLET CONTROL $HW=H+h_o-LS_o$: K_e	H	d_c	$\frac{d_c+D}{2}$	TW	h_o	LS_o	HW	CONTROLLING HW	OUTLET VELOCITY	COST	COMMENTS
Pipe	Assigned	Trial	Charts 2-7 use one Pp. 5-22 to 5-27	Compute $D \times \frac{HW}{D}$	from Table 1 Page 5-49	Charts 9-14 use one Pp. 5-32 to 5-37	Charts 16-20 use one Pp. 5-39 to 5-43	Compute	from Stage-Discharge Curve	larger of TW or $\frac{d_c+D}{2}$ Page 5-10	Compute (length $\times$ slope)	Compute: $HW = H + h_o - LS_o$	larger value of inlet or outlet control Page 5-11, 5-12			

SUMMARY & RECOMMENDATIONS:

4. ROADWAY GEOMETRICS

4.1. Crossing Profile

4.1.1. General Concepts

Low water stream crossings are designed for occasional overtopping with floodwater and as a consequence have an inherent vertical "dip" characteristic. The approach roadway is at or above the normal ground level on the stream banks, whereas the low point of the crossing may be very close to the normal water flow surface as shown in Figure 4.1.

This sudden "dip" in the vertical alignment is not consistent with drivers' expectations of a public highway profile. Proper signing is essential to alert the driver to a condition that can not be traversed at the higher speeds associated with tangent alignments and flat grades.

In some cases the stream width may be wide and the banks low so that a relatively flat approach grade eases the transition into the low water crossing as shown in Figure 4.2.

In other cases the stream may be narrow with high banks so that steep grades are necessary on the approaches as indicated in Figure 4.3. This condition is more common at sites suitable for low water crossings.

Figure 4.4a shows the usual configuration of the crossing will be a symmetrical sag vertical curve. However, if one stream bank has a significantly higher elevation than the other side, unequal tangent grades or an asymmetrical vertical curve may result as indicated in Figure 4.4b. Conditions may be such that a wide stream crossing results in independent vertical curves with a tangent across the bottom as shown in Figure 4.4c.

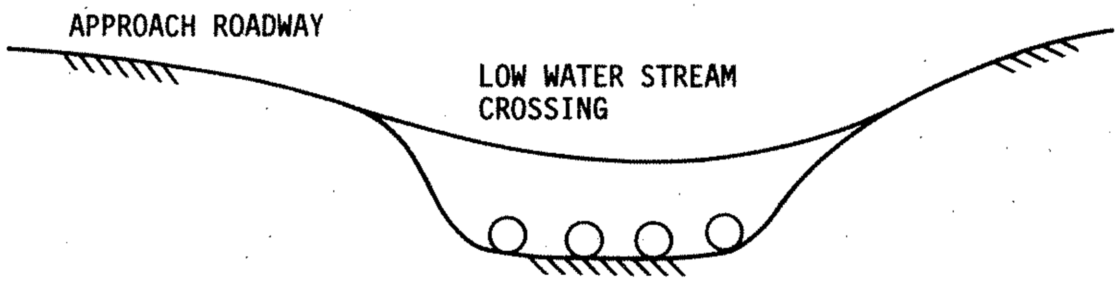


Fig. 4.1. Inherent roadway dip in low water crossing design.

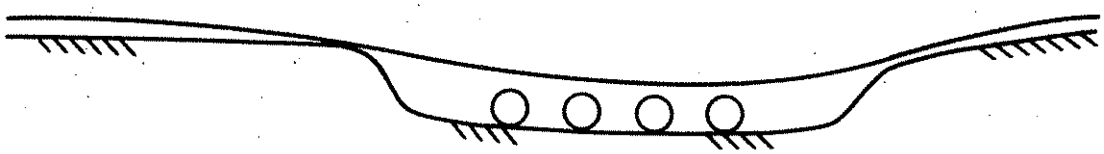


Fig. 4.2. Wide stream with low banks and relatively flat approach grades.

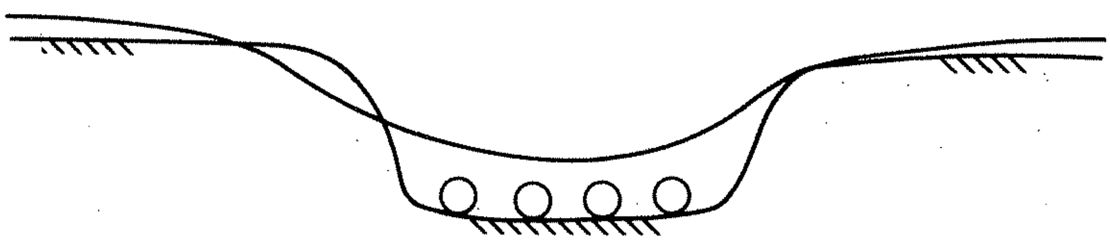


Fig. 4.3. Narrow stream with high banks and steep approach grades.

4.1.2. Selecting Tangent Grades

The variables of concern in the design of the stream crossing profile are the tangent grades, the length of sag vertical curve, and crest vertical curve lengths at the stream edges.

The selection of tangent grade lines will be dependent on the height of the stream banks and the slope of the terrain adjacent to the stream banks, as well as the amount of cut allowed into the stream bank. If minimal grading is desired, steep grades will result. In general, a grade of 12 percent could provide a surface suitable for driving when wet and muddy, but only at very low speeds. This arbitrary maximum may in fact be increased without undue concern, if the users are farm equipment and four-wheel drive vehicles and speeds are very low. Steep grades significantly increase the stopping distance and consequently, reduce the allowable speed.

The use of flat grades that cause a cut back into the stream bank can result in a maintenance problem as shown in Figure 4.5. When high water causes overtopping of the crossing, the flood water spreads onto these flat approach grades, wider than the normal stream width, and subsequently deposits debris and mud on the crossing roadway. The steeper grades may be self cleaning but have the disadvantage of a more abrupt change in vertical alignment with subsequent reduced speed requirements.

4.1.3. Criteria for Selecting the Length of Vertical Curves

A number of criteria are recognized in the design of a profile. Stopping sight distance is the usual criterion for selecting the length of crest vertical curves, whereas headlight sight distance, driver com-

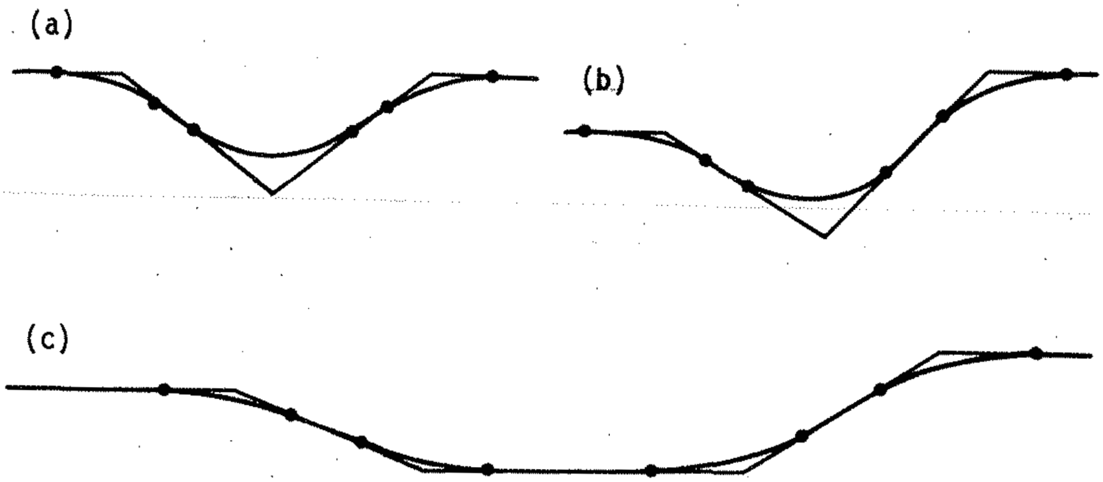


Fig. 4.4. Types of sag vertical curves.

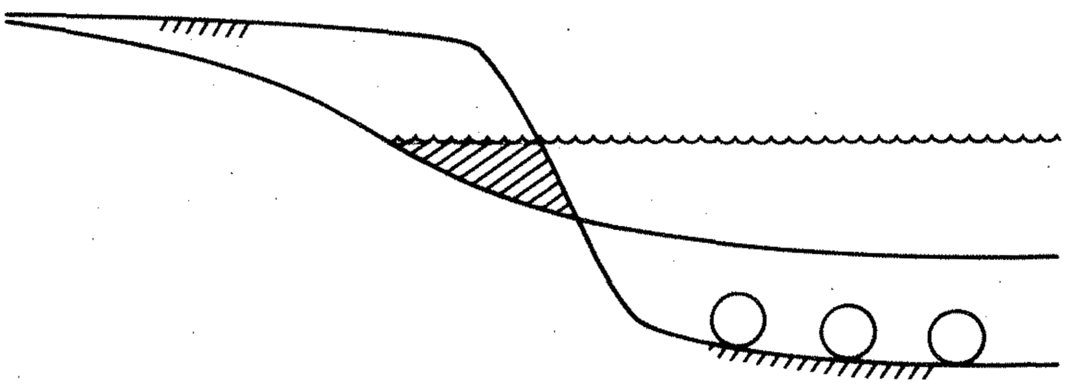


Fig. 4.5. Effect of flat approach grades on debris deposit.

fort, and appearance may be used for sag vertical curve length determination as shown in Figure 4.6.

The normal procedure for designing a crest vertical curve is to provide a length of vertical curve such that a driver may bring the vehicle to a stop after discerning an object six inches high on the roadway ahead. The normal procedure for designing a sag vertical curve is to provide a length of vertical curve such that a driver may bring the vehicle to a stop after the headlights illuminate an object on the roadway ahead.

4.1.4. Sight Distance Criteria

It should be noted that other criteria could be selected for crest vertical curve design. Figure 4.7 presents three alternatives. Location 1 in Figure 4.7 is not related to the shape of the vertical curve and is not appropriate. Location 2 could be used but would require a plotted profile to evaluate each site since the geometric shape would be difficult to describe mathematically. Location 3 would provide a more restrictive design than current AASHO (1965) policy since the height of object has been reduced to zero.

Current accepted minimum crest vertical curve design practices are based on AASHO (1965) for stopping sight distances. Stopping sight distance is the distance traveled from the first sighting of an object until the vehicle reaches this object. The length of vertical curve selected must provide a shape such that the driver may bring the vehicle to a stop in the stopping sight distance for the initial speed and related design assumptions.

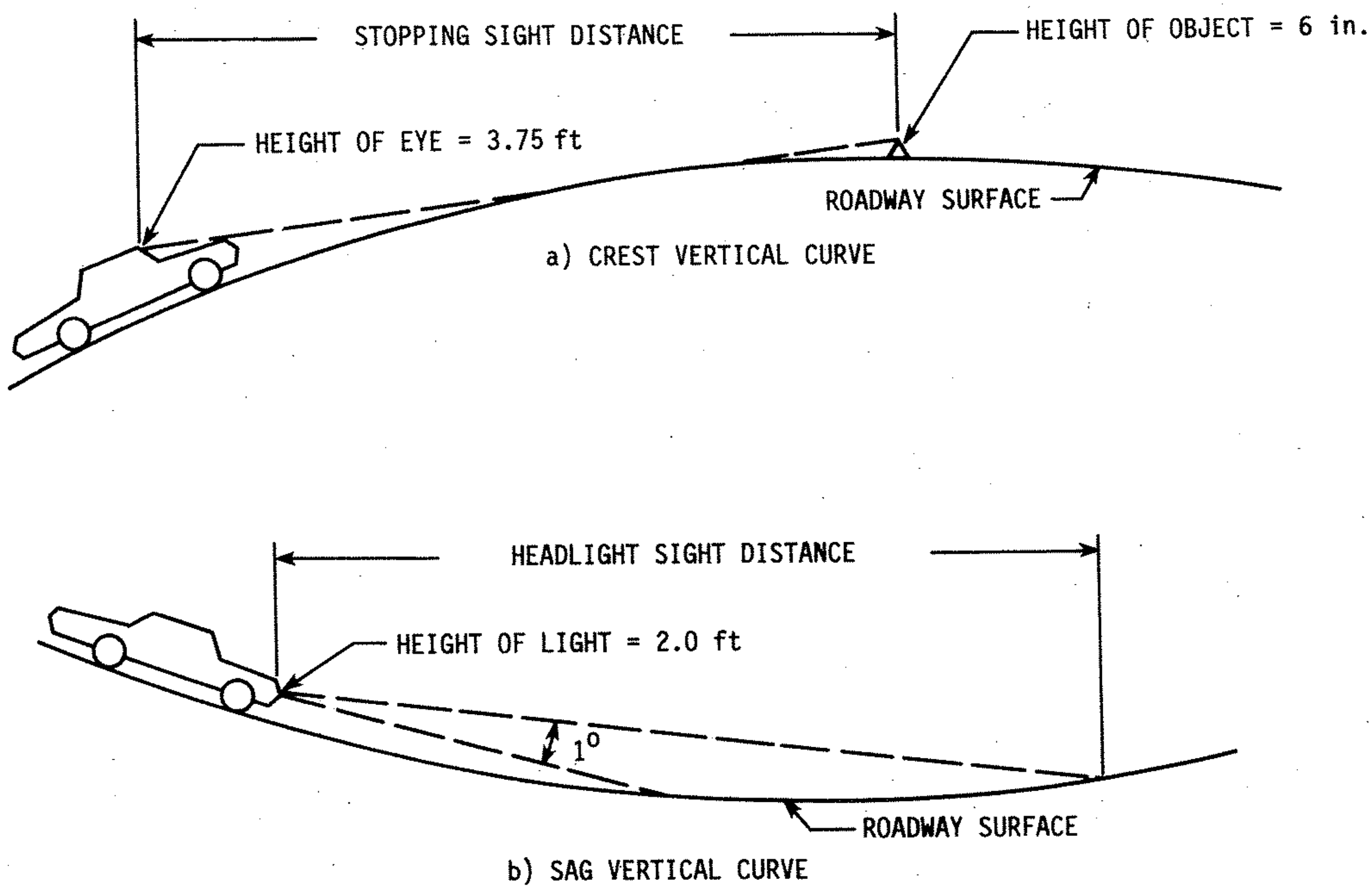
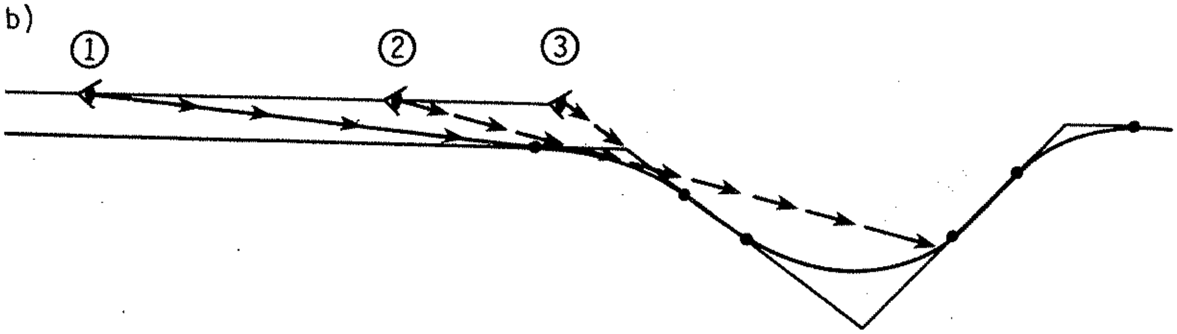


Fig. 4.6. Design of crest and sag vertical curves.

b)



LOCATION 1: DRIVER APPROACHING A LWSC NOTES THE PAVEMENT HAS DISAPPEARED.

LOCATION 2: DRIVER APPROACHING A LWSC SEES THE OPPOSITE BANK OF THE LWSC AT ABOUT MID HEIGHT.

LOCATION 3: DRIVER APPROACHING A LWSC SEES THE ENTIRE SURFACE OF THE LWSC SAG VERTICAL CURVE.

Fig. 4.7. Alternative sight distance criteria for selecting crest vertical curve lengths.

The stopping sight distance formula as presented in The American Association of State Highway Officials (AASHO) (1965) is:

$$d = 1.47\ V(t) + \frac{V^2}{30(f \pm G)} \tag{4.1}$$

where

d = stopping distance in feet

V = speed in miles per hour

t = perception reaction time = 2.5 sec

f = coefficient of friction

G = grade in percent divided by 100

Assume f may be equal to 0.20 due to wet surface (slick) conditions on an unpaved road and G may average ten percent.

Based on these criteria, Table 4.1 has been prepared for the stopping sight distances to be used in LWSC vertical curve calculations.

Table 4.1. Stopping sight distances for LWSCs.

V mph (1)	Perception and brake reaction distance ft (2)	Braking distance ft (3)	Stopping distance ft (4)
5	18.4	8.3	27
10	36.8	33.3	70
15	55.1	75.0	130
20	73.5	133.3	210
25	91.8	208.3	300
30	110.3	300.0	410

4.1.5. Crest Vertical Curves

The calculations for a crest vertical curve length are based on the following formula as presented in AASHO (1965).

for d < L

$$L = \frac{Ad^2}{100((2h_1)^{0.5} + (2h_2)^{0.5})^2} \tag{4.2}$$

for d > L

$$L = 2d - \frac{200((h_1)^{0.5} + (h_2)^{0.5})^2}{A} \tag{4.3}$$

where

L = length of crest vertical curve in feet

A = algebraic difference in grades in percent

h_1 = height of driver's eye

h_2 = height of object

If the normal AASHO (1965) practice for crest vertical curve design was used, the height of eye would be 3.75 feet and the height of object would be six inches. It should be noted, however, that a change in AASHO design criteria is imminent. A new "Policy on Geometric Design for Highways and Streets" will cause the height of eye criterion to be reduced to 3.50 feet. No change is anticipated in the height of object criterion.

Using a height of eye of 3.5 ft (h_1) and a height of object of six inches (h_2), Equations (4.2) and (4.3) can be reduced to:

for $d < L$

$$L = Ad^2/1329 \tag{4.4}$$

for $d > L$

$$L = 2d - \frac{1329}{A} \tag{4.5}$$

The minimum length of crest vertical curve may be calculated from Equation (4.4) or (4.5), based on a given speed (and determining d from Table 4.1) and the tangent grades selected (A).

For a given algebraic difference in grades (A) and a vertical curve selected to fit the terrain, the determination of the controlling speed is not as readily calculated from Equations (4.1) and (4.4) or (4.5). Designers generally use the reciprocal of the rate of change of grade or $K = L/A$ as a measure of curvature in determining speeds for a given crest vertical curve design. Table 4.2 is presented as a design aid and is based on Equation (4.4), where $d < L$, or:

$$K = \frac{L}{A} = \frac{d^2}{1329} \tag{4.6}$$

Table 4.2. Minimum crest vertical curve design criteria for LWSCs.

V (mph)	d (ft)	K (Length in feet per percent A) Calculated	Rounded
15	130	12.71	12.5
20	210	33.18	33.0
30	410	126.49	125.0

A more common procedure for determining minimum length of crest vertical curves is to plot (A) and (L) for various speeds. Figure 4.8 is a design chart for selecting a length of LWSC crest vertical curve, or conversely, having selected a suitable length of vertical curve to fit the terrain, Figure 4.8 may be used to determine the speed for that design.

The minimum vertical curve lengths in Figure 4.8 are based on a value of three times the speed in feet per second.

4.1.6. Sag Vertical Curves

In the design of a sag vertical curve for normal street and highway design practice, the concept of headlight sight distance determines the length of vertical curve. A suitable length of sag vertical curve allows the roadway ahead to be illuminated so that a vehicle could stop in accordance with the stopping sight distance criteria. The design of a sag vertical curve using headlight stopping sight distance formula is:

for $d < L$,

$$L = \frac{Ad^2}{400 + 3.5d} \qquad (4.7)$$

for $d > L$,

$$L = 2d - \frac{400 + 3.5d}{A} \qquad (4.8)$$

where:

L = length of sag vertical curve, in feet

d = headlight beam distance, in feet

A = algebraic difference in grades, percent

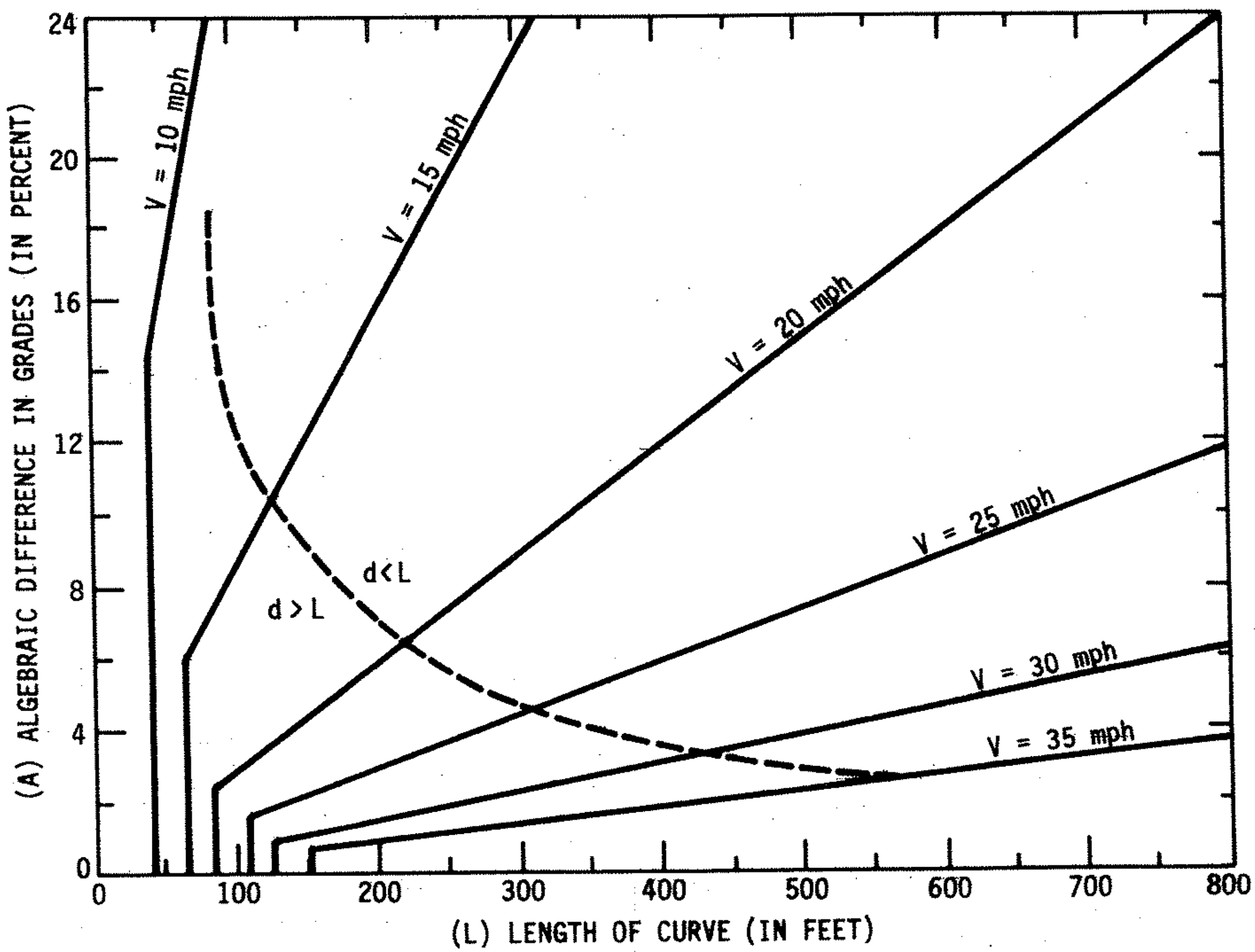


Fig. 4.8 Minimum length of crest vertical curve for LWSCs (based on height of eye = 3.5 ft, height of object = 6 in., and stopping sight distance from Table 4.1).

It should be noted that other criteria could be used for the design of LWSC sag vertical curves, such as comfortable ride. However, because of the potential for flooding and subsequent deposits of debris on the roadway, the minimum design should not be less than the headlight sight distance criterion. For safety reasons, the light beam distance is set equal to the safe stopping distance as discussed in Section 4.1.4 and in Table 4.1.

Table 4.3 presents a K factor for design where K = L/A. For combinations of grade and speed where d < L, the length of vertical curve can be calculated as L = KA.

Figure 4.9 is the sag vertical curve design chart. It may be used to select the length of sag vertical curve for a specific set of grades and speed condition, or having selected a trial sag vertical curve, the speed associated with that design may be determined. The minimum values on Figure 4.9 are based on three times the speed in feet per second.

Table 4.3. Minimum sag vertical curve design criteria for LWSCs.

V (mph)	d (ft)	K (Length in feet per percent A) Calculated	Rounded
10	70	7.6	8
15	130	19.8	20
20	210	38.9	39
30	410	91.6	92

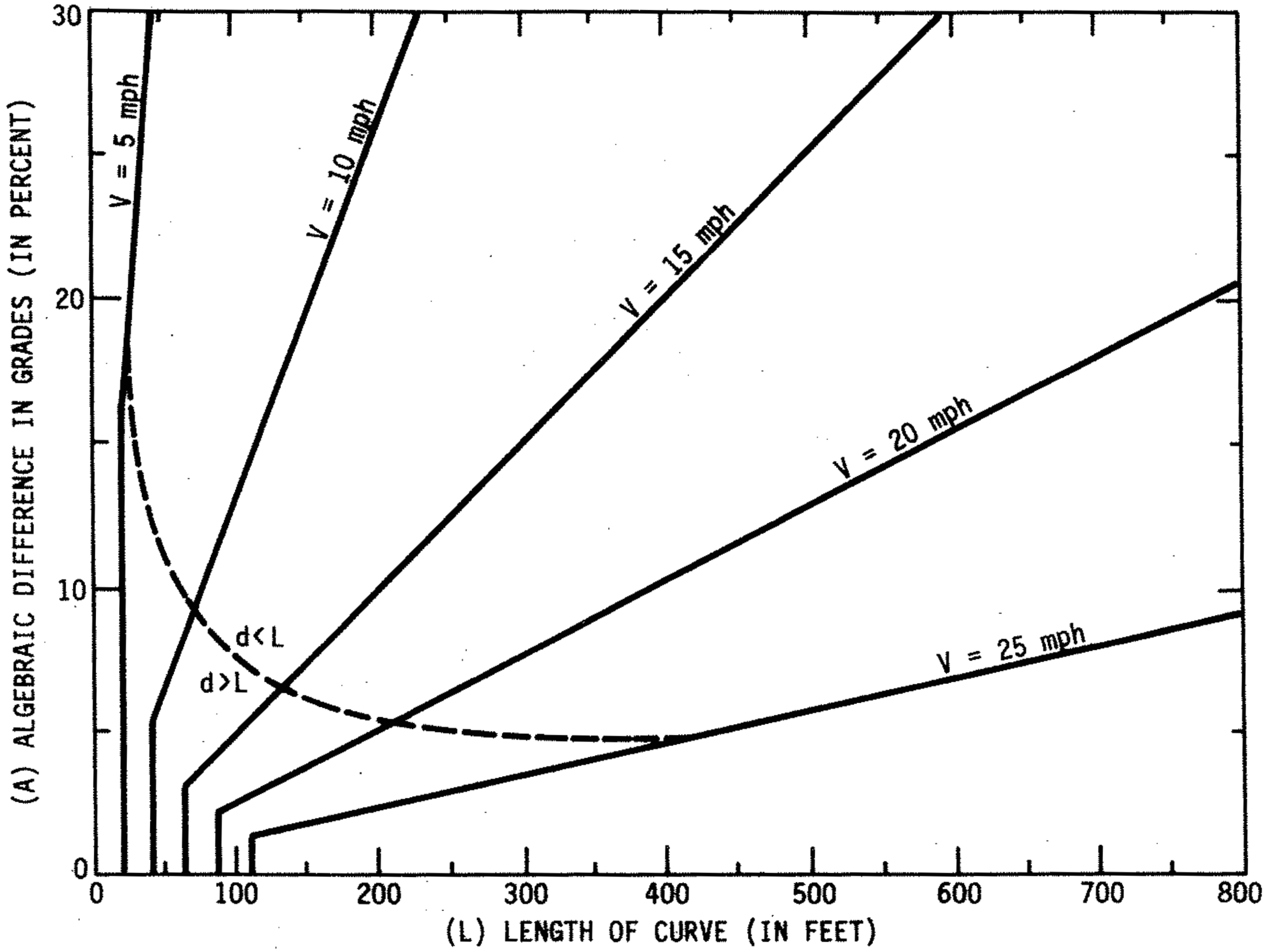


Fig. 4.9 Minimum length of sag vertical curve for LWSCs
(based on stopping sight distances from Table 4.1).

4.2. Cross Section

The function of the cross section is to accommodate vehicles on the roadway and to allow periodic higher stream flows to cross the roadway.

The roadway width must accommodate the vehicles using the road. Although one-way traffic flow can be assumed in most cases, the design selected probably will accommodate two passenger vehicles on the top of the LWSC as a factor of safety. Passenger vehicles are in the range of 6.0 feet to 6.5 feet in width. Pick-up trucks are common on these types of roads and are in the range of eight feet in width.

Farm vehicles of much wider dimensions commonly use these types of roads and may legally do so. In fact, on a farm field access road, one of the advantages of using a LWSC instead of a bridge, is the unrestricted farm vehicle width that can be accommodated. Old bridges with guard rails on the approaches present problems for wide farm vehicles.

Farm vehicles in common use have transport widths of 18 to 20 feet. In fact, some vehicles may reach 28 feet in transport width. One farm vehicle made in Iowa has a rear axle loaded weight of 74,000 pounds.

A minimum width to accommodate an eight foot tread width vehicle, with clearances on both sides for safety and operational weaving (e.g. shoulders), can be used if no handrails, delineator posts, or other appurtenances appear on the outside edges of the roadway. Such a cross section width will allow over width farm vehicles to negotiate the crossing with the extra width extending beyond the edges of the roadway.

For design purposes a 16 feet top width would be minimal, with a 20 foot or greater top width desirable. The roadway should be crowned to cause water to run off and reduce ponding on the roadway. A crown will even cause dirt to migrate to the edges under traffic conditions. As periodic overtopping of the roadway occurs, a crown of 0.02 feet per foot from the upstream side to the downstream side will tend to be more self-cleaning than a crown symmetrical about the centerline. Also, the pavement should have transverse grooves for traction. Transverse cross slopes of 0.04 or 0.06 may be suitable.

Low water stream crossings have been constructed with vertical sides as well as with battered side slopes. Also, the pipes may protrude or be flush with the foreslopes of the cross section. The major disadvantage of a vertical foreslope is the debris, erosion problem. It has been reported that vented fords have been washed out when the pipes have plugged with debris. A 2:1 foreslope with smoothly trimmed pipes may be self-cleaning on the upstream side. Such a configuration provides a more hydraulically efficient design. The use of curtain walls on both the upstream and the downstream edges is common to reduce erosion and undercutting.

Based on the above discussion, the cross section shown in Fig. 4.10 is recommended.

Where low stream banks occur, or at least where relatively flat grades are used, a V-shaped cross section has been used in place of a sag vertical curve. The bump associated with this abrupt grade change

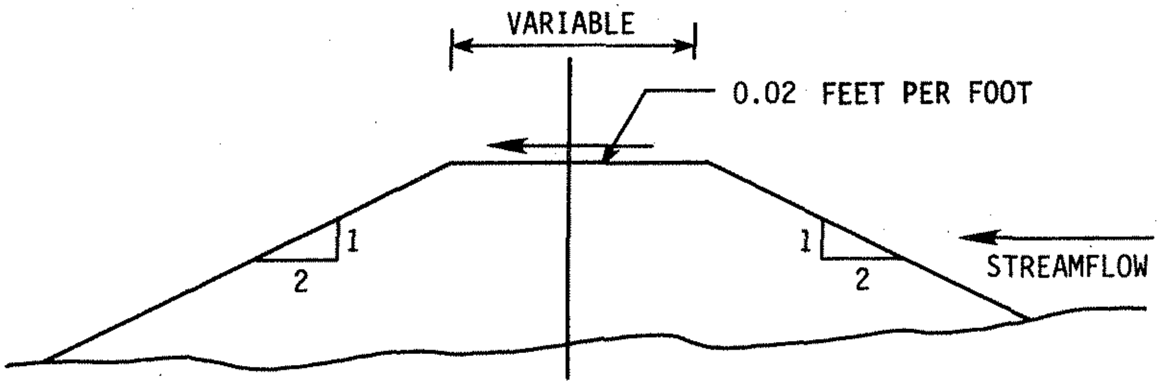


Fig. 4.10. Typical LWSC cross section.

may be tolerated by farm vehicles or others moving at very low speeds. Very close control of the speed would be required. Such a cross section would have the advantage of confining overtopping flows to a narrow width.

The use of streamflow gages, edge identification posts, or other such vertical projections tend to catch debris and are considered objectionable. The use of raised blocks on the downstream edge, with a taper on the upstream side have been used effectively to aid in defining the edge of roadway. Small indentations in the pavement at the edge lines can also be used to identify edges effectively and not catch debris.

Whatever cross section is used, it is important that observations be made after high water to assure proper maintenance.

4.3. Traffic Control

A low water stream crossing has two unique characteristics not associated with a traditional bridge that may create a potential for accidents and subsequent liability claims. The vertical profile at the crossing is usually restricted to low speeds and the pavement surface is subject to periodic flooding. It is imperative that adequate warning of these conditions be transmitted to the user.

The recommendations contained herein are based on the recent research by Carstens (1981).

4.3.1. Application of LWSC

In Carstens' survey of LWSC use in the U.S., 61 percent of the respondents reported they were used only on unpaved roads. Because paved highways have geometric design and traffic control conducive to higher speeds, drivers' expectations are not consistent with the vertical profile encountered at LWSCs. Also, because unpaved roads are limited to low traffic volumes, the use of LWSCs on these roads would involve a lower exposure to traffic. Carstens does not recommend the use of LWSCs on paved roads in Iowa.

The use of a LWSC design is based on an acceptance of periodic flooding. If flooding isolates a place of human habitation, either an alternate design should be considered or an alternate emergency access route should be developed.

4.3.2. Approach Signing

As previously noted, these signing recommendations are based on Carstens' research which was subsequently adopted by the Iowa DOT as recommended practice. The recommendation is shown in Figure 4.11.

According to Carstens, the intent of the regulatory sign "DO NOT ENTER WHEN FLOODED" is to preclude travel across the LWSC when the roadway is covered with water. Such a regulatory sign requires a resolution by the Board of Supervisors. The adoption of this sign in effect significantly reduces the applicability of an unvented ford.

4.3.3. Supplemental Signing

If the location of the LWSC is not apparent from a point approximately 1,000 feet in advance of the crossing, a supplementary distance plate may be used. The message "700 feet" would be displayed with the

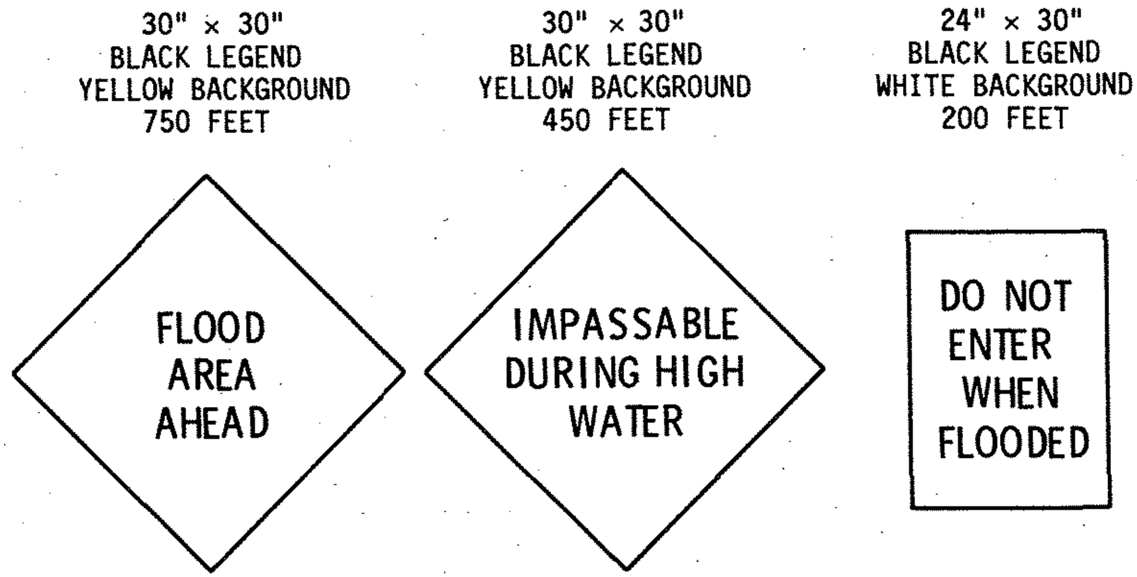


Fig. 4.11. Signs recommended for installation at low water stream crossings.

FLOOD AREA AHEAD sign. It would be 24 inches x 18 inches with black legend on a yellow background.

An advisory speed plate may be used if the maximum recommended speed at the LWSC is less than the speed limit in effect, which is the usual case. If used, the advisory speed plate is installed in conjunction with the FLOOD AREA AHEAD sign. However, if a supplemental distance plate is used (as noted above) the advisory speed plate is installed in conjunction with the IMPASSABLE DURING HIGH WATER sign.

4.3.4. Controls at the LWSC

Various controls have been used to delineate the edges of the traveled way at the LWSC. Curbs are generally unacceptable because water flow over the roadway tends to deposit mud and debris on the roadway. Attempts to create a series of small raised curb blocks with tapered blocks to provide for smooth laminar flow exist at a few locations. The use of any projections above the normal roadway surface will have an adverse effect on the self-cleaning aspect of the smooth cross section, however, observations on existing applications, or further research in this area, is needed.

Editor's Note: Pages 65-74 are not included in this publication due to a late revision.

5. SELECTION OF CROSSING MATERIALS

The surfacing material of any ford can be determined by using one of three methods which allow an estimation of tractive force and velocity. These values then can be compared with critical values for various materials. The first method presumes that the design engineer only has a knowledge of the size of the drainage area upstream of the proposed crossing site. With this watershed size and the design charts which relate watershed size to tractive force and velocity, the engineer then can select appropriate materials to be used. The rationale for these design charts is described in Appendix B.

The second design method is slightly more involved than the first. In this method it is presumed that the engineer also has detailed information about the channel's cross-sectional geometry. Using these data and the design charts presented, the engineer then can select appropriate materials.

These first two methods rely on geomorphic relationships developed from flow gaging stations in Iowa. The third method uses only physical data collected at the site to determine a velocity and tractive force. Then these are used to select the appropriate materials.

Various materials which might be used in the construction of any ford are described in terms of their suitability under different flow conditions and different site conditions. Recommendations regarding their use under these different conditions are made.

5.1. Method I

5.1.1. Step 1

Two items of data are needed for Method I: region in Iowa and watershed area in square miles. The region is obtained from Figure 5.1. Methods for obtaining the drainage area were described in Section 3.1.

5.1.2. Step 2

Step two involves the use of Figures 5.2 and 5.3 for Region I and Figures 5.4 and 5.5 for Region II. To use these figures, enter the value of drainage area in either figure, then read off a value of either the estimated tractive force (τ_t) from Figures 5.2 and 5.4, or velocity (V_t) from Figures 5.3 or 5.5. These values of tractive force (τ_t) and velocity correspond to flood flows with return periods of 10-, 25- and 50-years. In fact they are upper limit values of tractive force and velocity which provide an inherent factor of safety in the selection process. The reason why they are upper limit values is explained in Appendix B. The designer can select a return period which is appropriate to his or her particular site. Alternatively, the designer can select values for all three return periods and determine the variation in construction material, if any, which results and use this information in the decision-making process.

5.1.3. Step 3

The recommended value that grass is capable of resisting is a velocity of three feet per second. Section 5.4 gives more information on the resistance of vegetation to velocity.

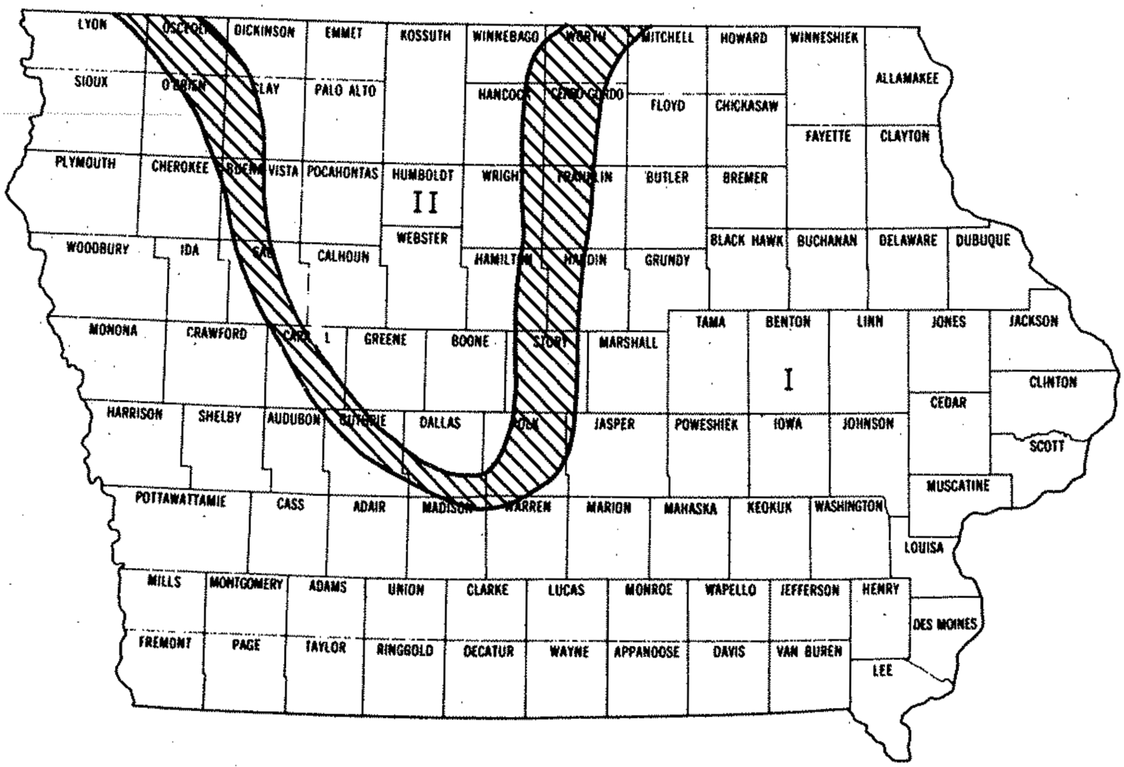


Fig. 5.1. Hydrologic regions of Iowa for flood-frequency estimates.

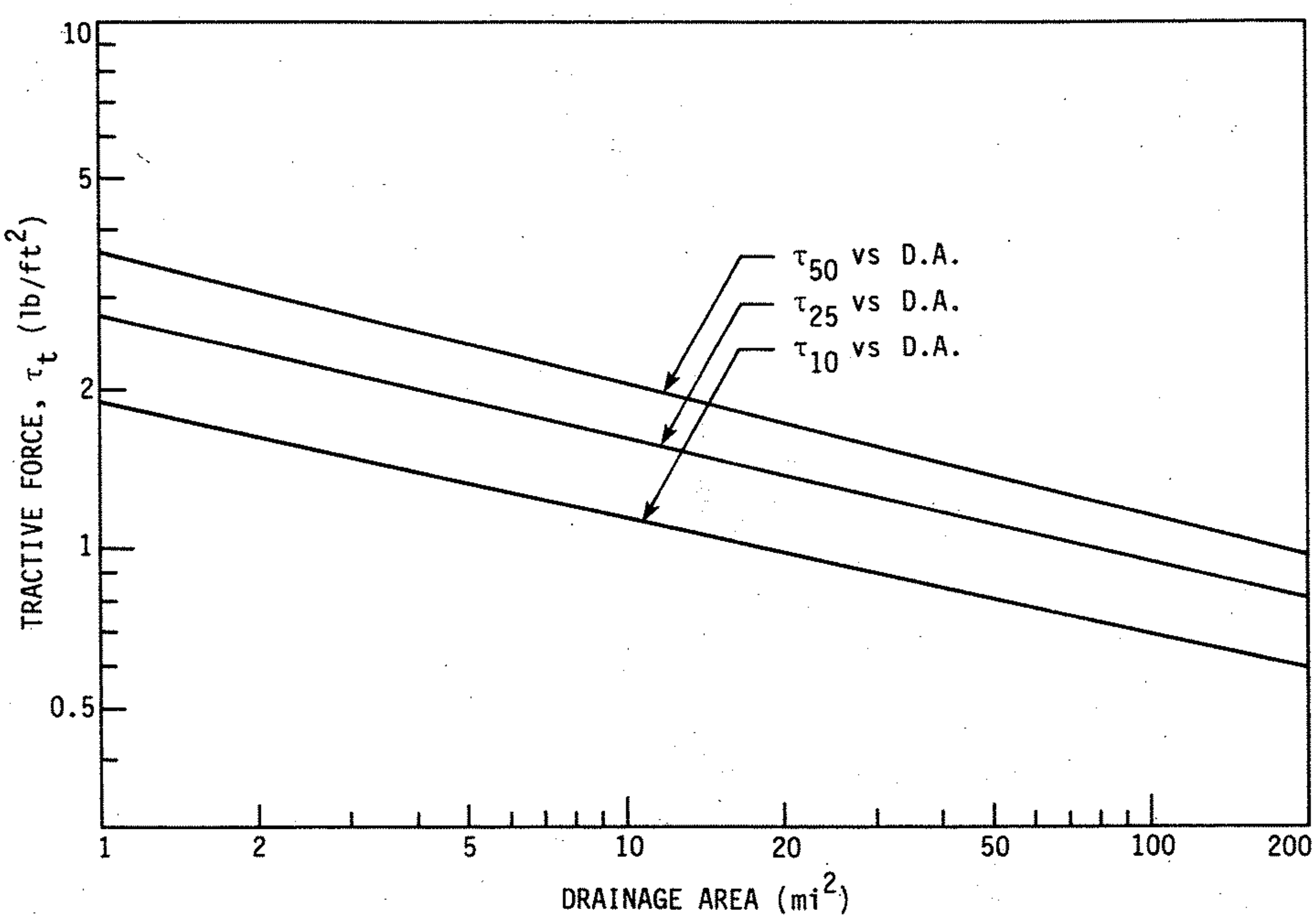


Fig. 5.2. Tractive force (τ_t) vs drainage area, Region I.

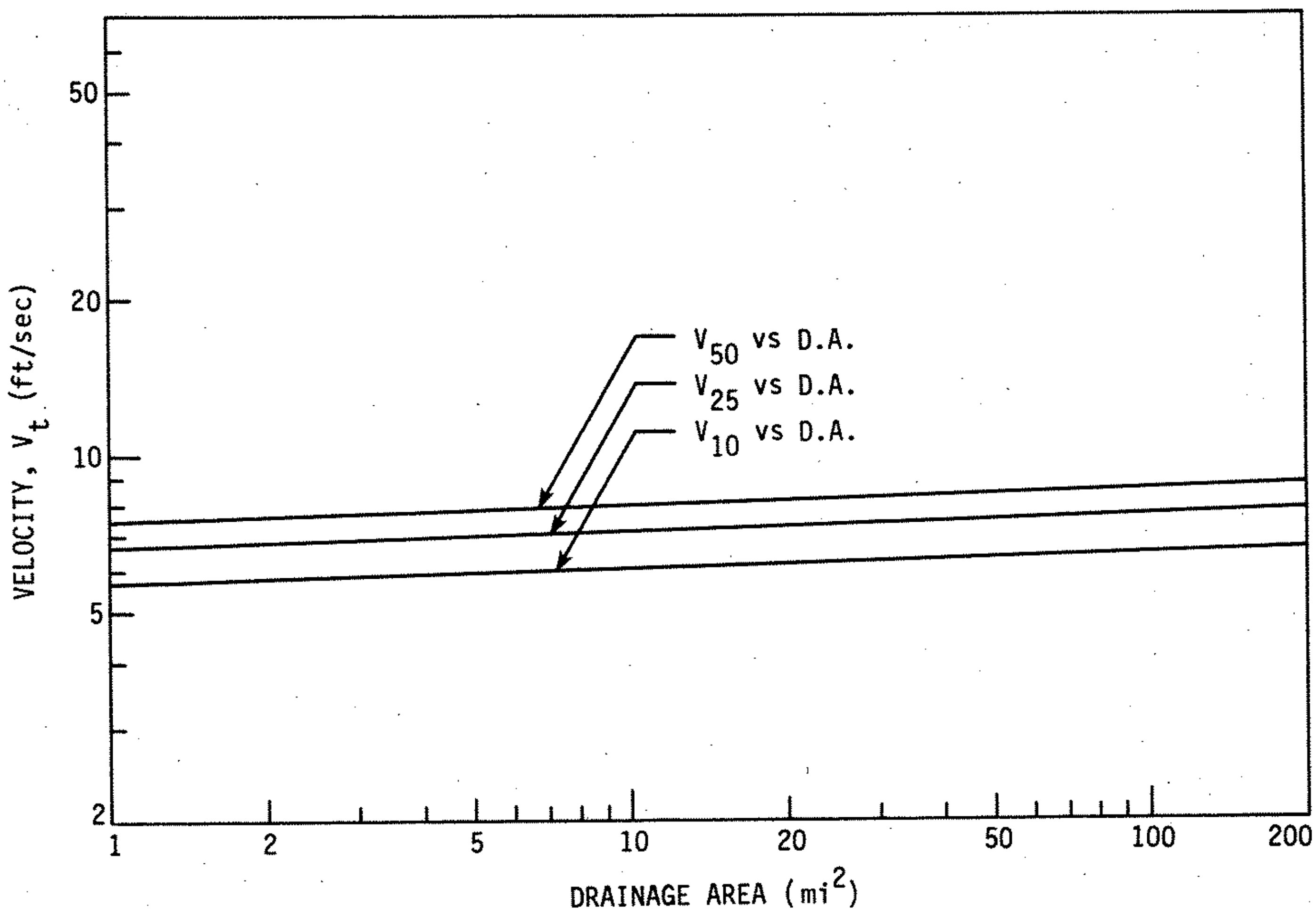


Fig. 5.3. Velocity (V_t) vs drainage area, Region I.

The tractive forces given in Table 5.1 correspond to the critical tractive force (τ_c) which the various sizes of riprap are capable of resisting. Using Table 5.1 the engineer can select a riprap size which will be capable of resisting the τ_t values obtained under step 2. Anderson (1973) and Austin (1982) discuss in full the design of riprap.

Table 5.1. Critical tractive force values for different sizes of riprap.

Material[1] (1)	Critical tractive force, lb/ft[2] (2)
Riprap D_{50} = 6"	2.0
Riprap D_{50} = 15"	5.0
Riprap D_{50} = 27"	7.3
Riprap D_{50} = 30"	10.0

[1]D_{50} is the size of riprap sample, 50 percent of which is finer by weight.

The engineer can use soil cement, gabions, Fabriform, and Portland cement concrete as construction materials for values of velocity and tractive force greater than the values given above. Considerations involved in the use of these materials also are explained in section 5.4.

5.2. Method II

5.2.1. Step 1

Determine the region in Iowa and watershed area as described in section 5.1.1.

5.2.2. Step 2

Use Figures 5.6 and 5.7 for Regions I and II, respectively, with the drainage area determined in Step 1 and obtain the slope of the channel bed, S.

5.2.3. Step 3

Use Figures 5.6 or 5.7 depending on the particular region in which the crossing is located and obtain the depth of flow, d_t, for the design flood, with a 10-, 25- or 50-year return period. Alternately, all three d_t may be obtained for comparative purposes.

5.2.4. Step 4

Draw a valley cross section along the centerline of the proposed crossing. Then by plotting a horizontal line a distance d_t above the bed of the channel, the cross-sectional area of flow, A_t, within the channel itself can be determined as illustrated in Figure 5.8. Using the shaded area in Figure 5.8, the wetted perimeter, WP, can also be determined.

5.2.5. Step 5

Calculate the velocity of flow in the channel, V_t, by using Manning's equation as described in section 3.3 and repeated here as Eq. (5.1).

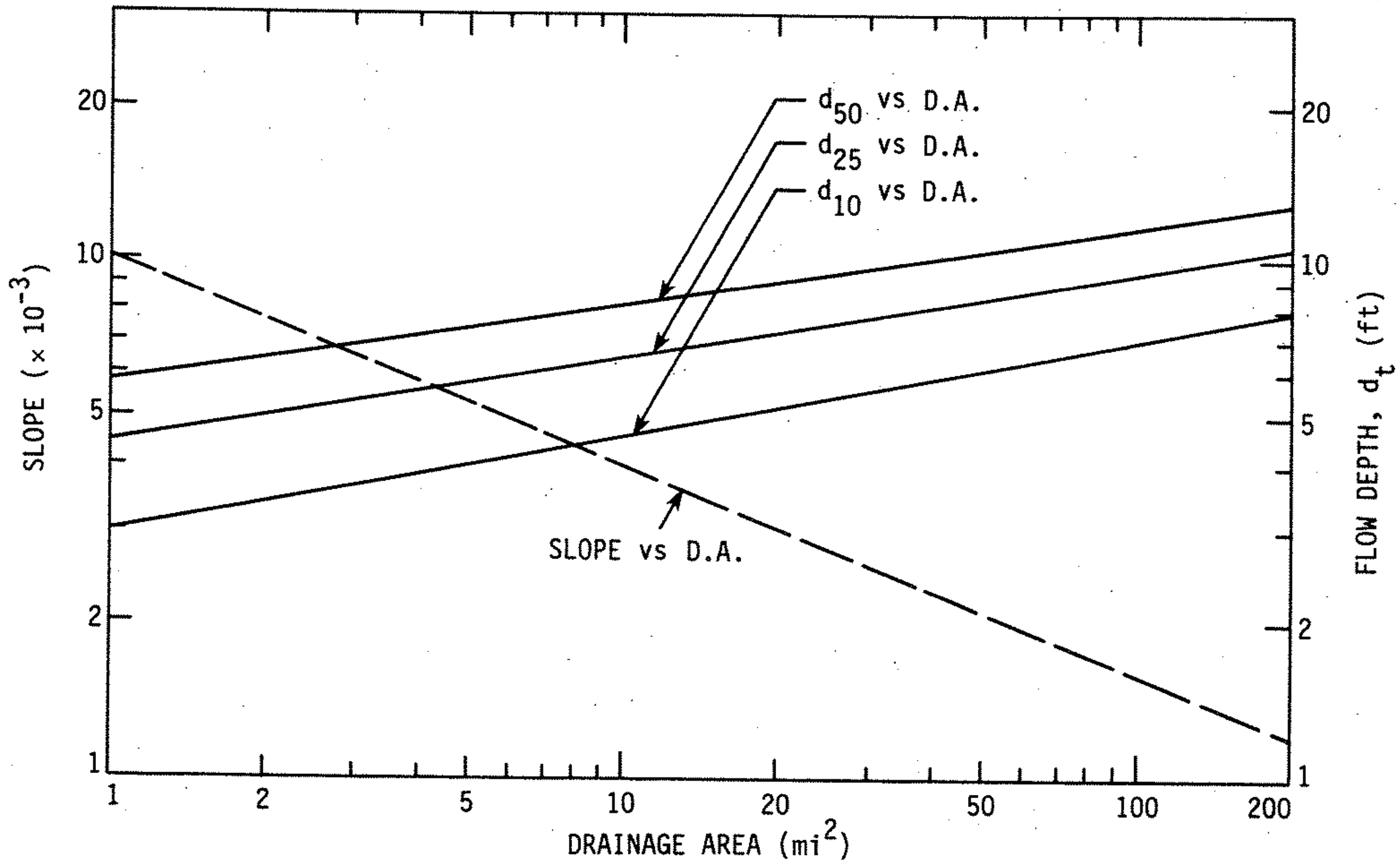


Fig. 5.6. Slope and flow depth (d_t) vs drainage area, Region I.

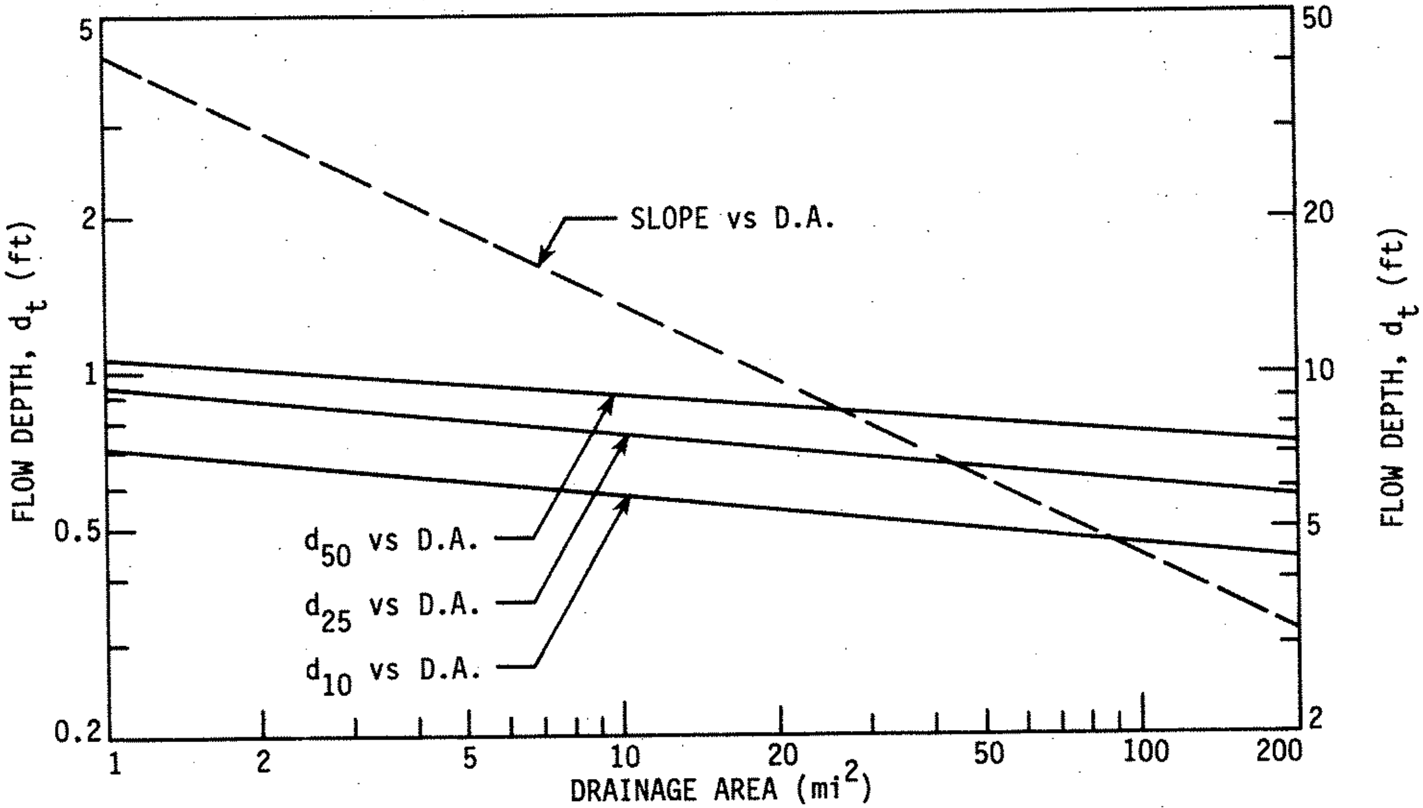


Fig. 5.7. Slope and flow depth vs drainage area, Region II.

$$V_t = \frac{1.49\ R^{2/3}\ S^{1/2}}{n} \tag{5.1}$$

where

V_t = velocity of flow in feet per second for some return period, t

R = hydraulic radius in feet = A_t/WP_t

S = channel slope in feet per foot

n = Manning's roughness coefficient

V_t can be calculated for the 10-, 25-, or 50-year flood return period or for all three.

5.2.6. Step 6

Calculate the tractive force in the channel (τ_t) using Eq. (5.2).

$$\tau_t = 62.4Sd_t \tag{5.2}$$

Again, this can be done for one or all of the three flood return periods.

5.2.7. Step 7

Using the values of V_t and τ_t calculated above, suitable riprap can be selected using Table 5.1 or other materials can be selected by considering the properties described in section 5.4. The designer can use one return period or, alternatively, can select values for all three return periods and determine the variation in construction material, if any, which results and use this information in the decision-making process.

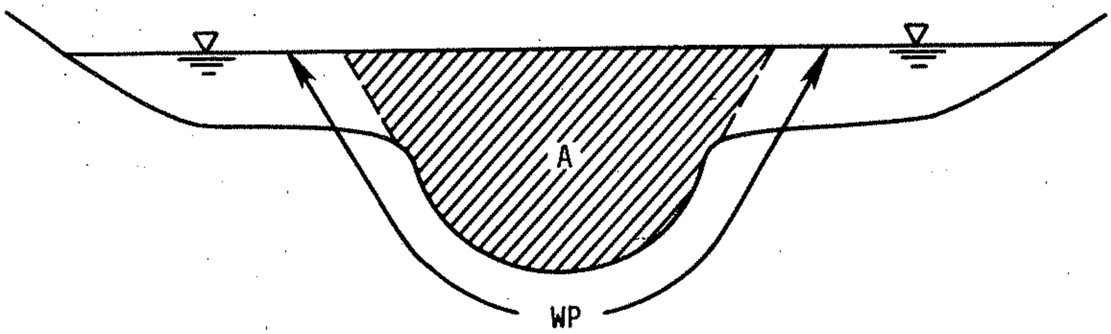


Fig. 5.8. Determination of cross sectional area of flow within the channel.

$$S = \frac{1{,}038 \text{ ft.} - 974 \text{ ft.}}{(0.75)(2.73 \text{ mi})} = \frac{64}{2.05} = 31.2 \text{ ft./mi.}$$

5.3.3. Step 3

Determine the peak discharges for one or more return periods as described in the following excerpt from Lara (1973), commonly known as Bulletin 11.

Flood characteristics at any location on Iowa streams with a drainage area of 2.0 square miles or more are computed by solving regional regression equations, which relate floods of given return periods to basin parameters. The state is divided into two hydrologic regions as shown in Figure 5.1. Region I covers about 68 percent of the state. Region II covers most of that area known as the Des Moines lobe. The regression models within each hydrologic region have the following form:

$$\text{Region I} \quad Q_t = c_t(A)^{x_t} \qquad \text{Model I} \tag{5.4}$$

$$Q_t = c_t(A)^{x_t}(S)^{y_t} \qquad \text{Model II} \tag{5.5}$$

$$\text{Region II} \quad Q_t = c_t(A)^{x_t} \tag{5.6}$$

where

Q_t is the discharge for a t-year return period

A is the drainage area in square miles

S is the main-channel slope in feet per mile, determined from the elevations at points 10 percent and 85 percent of the distance along the channel from the design point to the divide

c, x and y are the regression coefficients. Values of c, x and y for the three models are listed in Tables 5.2, 5.3 and 5.4.

Table 5.2. Regression coefficients for Region I, Model I.

t years (1)	c_t (2)	x_t (3)
2	197	0.535
5	439	0.501
10	667	0.480
25	1,040	0.455
50	1,390	0.437
100	1,800	0.421

Table 5.3. Regression coefficients for Region I, Model II.

t years (1)	c_t (2)	x_t (3)	y_t (4)
2	31.2	0.701	0.490
5	82.5	0.651	0.445
10	143	0.618	0.410
25	262	0.579	0.367
50	394	0.551	0.335
100	571	0.524	0.305

Table 5.4. Regression coefficients for Region II.

t years (1)	c_t (2)	x_t (3)
2	41.9	0.672
5	77.0	0.666
10	106	0.661
25	144	0.653
50	177	0.647
100	212	0.642

The recommended model to use in Region I is Model II; however, if a good determination of the main channel slope cannot be made, then Model I can be used. Both models yield approximately the same answer for basins larger than about 10 square miles. For basins smaller than 10 square miles down to 2 square miles, Model II should be used.

The use of these equations is illustrated in the following example. Estimate the 25-year and 50-year flood return periods for the Tarkio River at a bridge in Montgomery County located near the northeast corner of Sec. 28, T.73N, R.37W.

1. Figure 5.1 indicates that the watershed is located in Region I.
2. The drainage area equals 10.7 square miles as determined from the topo map.

3. The main channel slope equals 18.0 feet per mile as determined from a topo map.
4. Use Model II in Region I.
5. Using the regression coefficients from Table 5.3 and substituting in the model

$$Q_{25} = 262\ (10.7)^{0.579}\ (18.0)^{0.367} = 2{,}980 \text{ cfs}$$
$$Q_{50} = 394\ (10.7)^{0.551}\ (18.0)^{0.335} = 3{,}830 \text{ cfs}$$

These discharges are interpreted as follows. A discharge of 2,980 cfs has a 4 percent chance of being equaled or exceeded in any one year. A discharge of 3,830 cfs has a 2 percent chance of being equaled or exceeded in any one year.

If the project watershed is located near a region boundary, the selection of the proper set of equations becomes a matter of "judgment." At this point, the user might keep in mind that the outstanding characteristic of Region II is its flat topography and poorly developed drainage network. If part of the stream begins in or flows across another region, there may be a need to use equations for both regions and estimate a weighted average.

The designer also should endeavor to interpret computed floods in light of site experience, physiographic variations, etc. For example, at the confluence of two fairly equal drainage areas, there may be a need for analyzing coincidental peak discharges from each area; adding the two peaks, then comparing it with the overall peak discharge computed for the entire area as one unit.

5.3.4. Step 4

Develop a stage-discharge curve for the valley cross section using the method described in section 3.3. The calculations should be extended so that the largest discharge exceeds Q_t. Plot this stage-discharge curve.

5.3.5. Step 5

Develop a stage-channel velocity curve in the following manner. From the calculations made in step 4, plot depth on the ordinate vs the channel velocity.

5.3.6. Step 6

Use the stage-discharge plot with the design flood, Q_t, and obtain the corresponding depth, d_t.

5.3.7. Step 7

Use the stage-velocity plot with the flow depth, d_t, to obtain the channel velocity, V_t.

5.3.8. Step 8

Substituting the values of slope, S, and depth, d_t, into Eq. 5.2, determine the tractive force, τ_t.

5.3.9. Step 9

Using the values of V_t and τ_t calculated under steps 7 and 8, respectively, riprap can be selected using Table 5.1 or other materials can be selected by considering the properties described in Section 5.4. The designer can use one return period or, alternatively, can select values for all three return periods and determine the variation in construction material, if any, which results and use this information in the decision-making process.

5.4 Material Review

5.4.1 Design Considerations

The New York Soil Mechanics Bureau (1971) and Keown (1977) outline considerations in the selection of a suitable material for channel erosion protection. A summary of these considerations, relevant to the design of low water stream crossings, is presented below.

1. The forces causing possible failure of the material, whether they be expressed in terms of velocity or tractive force, must be evaluated for each particular material. The specifications of the type or quality of suggested material will depend on the chosen design flood return period.
2. The channel geometry in terms of bed slope and bank slope at a particular crossing location will need to be evaluated in order to calculate the forces acting on bank protection.
3. Non-uniform settlement due to soft foundations and settlement due to scouring are important considerations in design of nonflexible structures such as concrete or Fabriform.
4. Environment may have an effect on the material; this includes ice-action on riprap and sunlight on Fabriform.
5. Economic considerations such as cost of materials, labor, and maintenance will be an important factor. Low initial cost alternatives might require expensive maintenance, whereas low maintenance structures might present an overly high construction cost.

6. Aesthetic considerations are considered to be largely unimportant as the locations will generally be in relatively remote areas; however, in "State Parks" this might be an important consideration.

5.4.2 Vegetative Protection

There are two basic types of vegetation which may be used as protective materials for stream banks: grasses and woody plants. Woody plants take longer to establish than grasses but have the advantage of being more robust and having a greater retarding effect on the stream velocity. This means that they are more suitable for higher velocities. Chow (1959) presents data produced by the U.S. Soil Conservation Service on their velocity resistance and retardance characteristics. These data are given in Tables 5.5 and 5.6, respectively. The maximum design velocity permitted for the use of grass is three feet per second and is below that which most grasses are capable of resisting. The retardance effect is beneficial as it can reduce velocities close to the bank by up to 90%, thereby greatly reducing the eroding power of the flow. However, it has been found that those grasses with the largest degree of retardance also need the best growing conditions.

Environmental conditions for the successful use of grasses are very important. Table 5.5 reveals the importance of the sideslope angles and the effect of the erodibility of the soil upon which the grasses will be planted. Furthermore, grasses cannot be used in situations where they will be subjected to anything other than short, periodic flows.

Table 5.5. Permissible velocities for various types of grass (after U.S. Soil Conservation Service, 1954).

Cover (1)	Slope Range Percent (2)	Permissible Velocity, fps[1] Erosion-Resistant Soils (3)	Permissible Velocity, fps[1] Easily Eroded Soils (4)
Bermuda grass	0-5	8	6
	5-10	7	5
	>10	6	4
Buffalo grass, Kentucky blue-grass, smooth brome, blue grama	0-5	7	5
	5-10	6	4
	>10	5	3
Grass mixture	0-5	5	4
	5-10	4	3
	Do not use on slopes steeper than 10%		
Lespedeza sericea, weeping love grass, ischaemum (yellow blue-stem), kudzu, alfalfa, crabgrass	0-5	3.5	2.5
	Do not use on slopes steeper than 5% except for side slopes in a combination channel		
Annuals--used on mild slopes or as temporary protection until permanent covers are estab-listed, common lespedeza, Sudan grass	0-5	3.5	2.5
	Use on slopes steeper than 5% is not recommended		

[1]The values apply to average, uniform stands of each type of cover. Use of velocities exceeding 5 fps only where good cover and proper maintenance can be obtained.

The retardance coefficient for grass erosion protection is equal to Manning's roughness coefficient, n. The value of n for a particular grass varies with channel slope and shape; however, a relationship exists between n and the product of mean velocity, V, and hydraulic radius, R, which is practically independent of channel slope and shape. Using curves of n versus VR developed for various degrees of retardance, it is possible to design adequate vegetative protection. Tables 5.5 and 5.6 give ranges of permissible velocities and retardance, respectively, for various grasses. The designer is referred to Chow (1959) for a detailed explanation of the design process for grass erosion protection.

A vegetative lining would probably present the most aesthetically pleasing protection measure. Vegetation is cheap in material costs ($500-600 per acre at 1976 prices), flexible, and not subject to failure by the action of undermining or settlement.

Temporary initial protection of the vegetation by the use of jute mesh can provide only a minimum of protection and the best method to afford early protection is the use of sods of vegetation held in place by pins or stakes.

5.4.3 Riprap

5.4.3.1 Rock Riprap

There are three basic types of riprap: dumped, hand-placed, and grouted. The dumped or hand-placed stones constitute a protective lining made up of more than one layer of stones resting on the foundation soil or a bedding layer. This multiplicity of layers ensures that the underlying soil is not exposed if settlement should occur or if individual rock particles are dislodged by ice or debris.

Table 5.6. Classification of degree of retardance for various kinds of grasses (after U.S. Soil Conservation Service, 1954).

Retardance (1)	Cover (2)	Condition (3)
A Very high	Weeping love grass	Excellent stand, tall (av 30 in.)
	Yellow bluestem ischaemum	Excellent stand, tall (av 36 in.)
B High	Kudzu	Very dense growth, uncut
	Bermuda grass	Good stand, tall (av 12 in.)
	Native grass mixture (little bluestem, blue grama, and other long and short mid-west grasses)	Good stand, unmowed
	Weeping love grass	Good stand, tall (av 24 in.)
	Lespedeza sericea	Good stand, not woody, tall (av 19 in.)
	Alfalfa	Good stand, uncut (av 11 in.)
	Weeping love grass	Good stand, mowed (av 13 in.)
	Kudzu	Dense growth, uncut
	Blue grama	Good stand, uncut (av 13 in.)
C Moderate	Crab grass	Fair stand, uncut (10-48 in.)
	Bermuda grass	Good stand, mowed (av 6 in.)
	Common lespedeza	Good stand, uncut (av 11 in.)
	Grass-legume mixture-summer (orchard grass, redtop, Italian rye grass, and common lespedeza)	Good stand, uncut (6-8 in.)
	Centipede grass	Very dense cover (av 6 in.)
	Kentucky bluegrass	Good stand, headed (6-12 in.)
D Low	Bermuda grass	Good stand, cut to 2.5 in. ht.
	Common lespedeza	Excellent stand, uncut (av 4.5 in.)
	Buffalo grass	Good stand, uncut (3-6 in.)
	Grass-legume mixture-fall, spring (orchard grass, redtop, Italian rye grass, and common lespedeza)	Good stand, uncut (4-5 in.)
	Lespedeza sericea	After cutting to 2 in. height, very good stand before cutting
E Very low	Bermuda grass	Good stand, cut to 1.5 in. ht.
	Bermuda grass	Burned stubble

The durability and flexibility of riprap decreases from dumped to hand-placed to grouted. Although grouted riprap is the most rigid, it is most susceptible to failure by undermining. Berg (1980) suggests that dumped rock riprap is the least vulnerable to impact damage.

In terms of cost, the best alternative is dumped riprap which involves less labor costs than hand-placed and less labor and material costs than the grouted type.

Anderson (1973) performed experiments that indicate the best type of rock is well-graded with stone sizes ranging from a size equal to the thickness of the protection lining down to one inch pebbles. The advantage of well-graded over uniform-graded riprap is that well-graded riprap acts as its own filter layer thus saving the cost of a special filter layer, and preventing outwash of the underlying soil. A well-graded riprap protection can be thinner and hence, cheaper than a uniformly graded riprap.

Table 5.7 defines the minimum thickness for layers of various stone sizes in terms of D_{50}, the average stone size.

Another advantage of riprap is its coarse surface which dissipates the energy of stream flow, thus reducing the chance of bed or bank erosion downstream of the protective lining. Table 5.7 shows this property of energy dissipation expressed in terms of Manning's roughness coefficient, n, for various stone sizes.

Keown (1977) makes recommendations on the shape and texture of the riprap particles used for channel protection measures. Block shaped rather than elongated shaped rocks, with sharp rather than smooth edges provide better interlocking and stability. Generally,

Table 5.7. Critical tractive force for various weights and sizes of stone.

W lb. (1)	D_{50} inches (2)	D_{50} feet (3)	τ_c lb/ft^2 (4)	n (5)
2000	30	2.5	10.00	0.047
700	22	1.83	7.32	0.044
250	15	1.25	5.00	0.042
16	6	0.50	2.00	0.036

stones with a length to width ratio less than 3 and aggregates containing less than 25% of the stones with a length to width ratio greater than 2.5 are preferred. A limit on side slopes of 1 vertical to 2 horizontal for dumped riprap and 1 to 1.5 for hand-placed stone is recommended. The thickness should range from at least 1 to 2 times the diameter of the largest stone.

Table 5.7 gives the values of critical tractive force, τ_c, for various sizes and weights of rock riprap. The specific gravity is assumed to be 2.65 for these calculations.

The weight of a rock in terms of its D_{50} in feet is determined by Eq. (5.7), which was obtained from Mark Looschen of the Iowa DOT.

$$W = 0.762 \times G \times 62.4 \times (D_{50})^3 \qquad (5.7)$$

where

W = weight in pounds

G = specific gravity of material

62.4 = unit weight of water in pounds per cubic foot

0.762 = adjustment factor

D_{50} = average stone size in feet

The adjustment factor of 0.762 results from the relative volumes of a sphere and a cube whose nominal dimension is given by D_{50}.

5.4.3.2 Soil Cement Riprap

Soil cement can be used as a riprap substitute. This is especially useful in areas where appropriate aggregate is not available and expensive hauling costs are involved.

Wade (1982) cites details of a soil cement project carried out by the U.S. Army Corps of Engineers. A 15 cm layer of soil cement was compacted on a sandbar adjacent to the proposed bank stabilization site. The layer was scored at 15 cm intervals, covered with sand, and cured for seven days. When the soil cement was moved to the site, it fractured along the predetermined planes of weakness caused by the scoring. To this date, the material has proved to be successful in terms of ease of construction and endurance.

The most advantageous characteristics of soil cement riprap are its ease of replacement if individual particles are lost and its good interlocking capabilities due to its blockiness and sharp edges. However, its uniform grading and lower specific gravity (1.65 vs 2.7) require a thicker layer of larger blocks than rock riprap.

5.4.4 Soil Cement

Soil cement consists of a mixture of soil and Portland cement in varying quantities. Usually the soil is obtained at the job site but sometimes imported sand is required. This is because the finer the texture of the soil, the greater the percentage of cement required to give sufficient erosion resistance and freeze-thaw durability.

Soil cement has been used in highway construction as a sub-base and in hydraulic engineering as erosion protection for a number of years. The most famous example of the use of soil cement for erosion protection is the Bonney Reservoir in Colorado where soil cement was used instead of riprap to protect against wave action.

Investigation of the erosion resistance of soil cement after freeze-thaw action has been carried out by several investigators including Litton (1982).

The Portland Cement Association (PCA) (1976) has recommendations for soil cement design for water protection measures based on highway design criteria of freeze-thaw tests and wet-dry tests. Generally, a cement content 2 percent greater than that suggested for the highway application is recommended.

Litton's recommendations for permissible velocities for various soil cement mixtures based on water jet tests are shown in Table 5.8. As can be seen, the velocities encountered in both Regions I and II of Iowa fall below these permissible velocities in the majority of instances, indicating the possibility of using soil cement as a protection material.

However, Wade (1982) points out that due to the degree of compaction required, the construction of a soil cement structure needs to be done in a dry location. This means that the stream must either be diverted or relocated during construction. Also, shrinkage cracking and low

Table 5.8. Permissible flow velocities for soil cement mixtures (after Litton, 1982).

Soil Mixture (1)	Maximum Allowable Velocity for Listed Cement Content 5% fps (2)	7% fps (3)	9% fps (4)	11% fps (5)	13% fps (6)
Alluvium	3.9	4.9	8.9	14.2	15.2
Alluvium-25% sand	9.9	17.2	>24.7	>24.7	>24.7
Alluvium-40% sand	13.5	>24.7	>24.7	>24.7	>24.7
Alluvium-55% sand	21.4	>24.7	>24.7	>24.7	>24.7
Sand	>24.7	>24.7	>24.7	>24.7	>24.7

flexural strength of the soil cement may require a filter cloth to prevent scour at shrinkage or settlement cracks.

5.4.5 Gabions (and Reno Mattresses)

Gabions are wire baskets filled with stone. Reno mattresses are elongated and flattened forms of this basic basket construction. The baskets and/or mattresses can be stacked upon one another or layed adjacent to one another and wired together in a variety of geometric sequences to give a multitude of structures. Figure 5.10 is an example of a gabion structure. The wire used can be either galvanized or plastic-coated for corrosion protection. In either case, it is twisted in such a way as to prevent a general unravelling if a wire should break.

The advantages of gabions are: they are flexible, thus making them less prone to failure from settlement or undermining; they fill up with

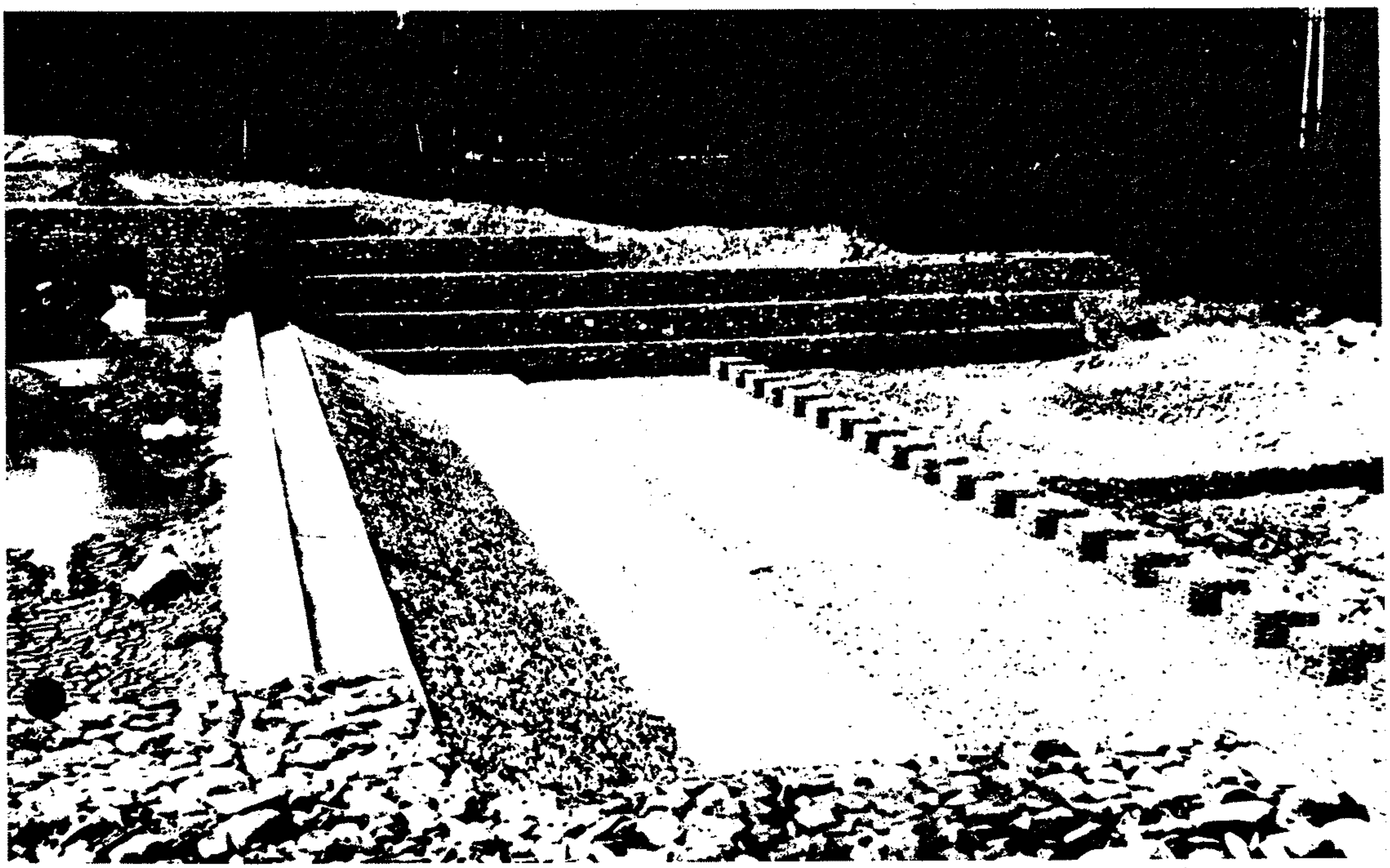


Fig. 5.10. Gabion formed weir.

silt quickly and allow the establishment of natural vegetation giving a more aesthetically pleasing look; and are 20-30% cheaper than rigid materials such as concrete. They also can be one-third as thick as an equivalent riprap protection.

However, they are labor intensive and a suitable filter material is required to prevent scouring of the underlying soil (Wade, 1982). Also, suitable rock must be available of a size large enough to prevent it being washed through the mesh (10-20 cm is a generally accepted size).

5.4.6 Fabriform

Fabriform is a nylon fabric form system to contain pumped concrete. It has been used for the construction of erosion protection on canals, reservoirs, rivers and lakes throughout the United States.

As shown in Figure 5.11, the system consists of two layers of woven nylon fabric interconnected by regularly spaced fibers and "filter points" which gives a smooth or rough hydraulic surface depending on requirements. Typical values of Manning's n for this system are 0.023 to 0.030 for the 8" filter point fabric.

The concrete mix used to fill the Fabriform has a high water cement ratio in order to give the best workability for pumping. The high water cement ratio does not result in low compressive strength because the fabric allows the excess water to bleed off. Flexural strength is somewhat limited due to the lack of longitudinal reinforcing; however, on slopes of less than 45° which are not subject to differential settlement this is not a problem. Where undercutting occurs, the system is highly vulnerable to cracking and failure, thus adequate cut-off protection against scour is vital.

Typical Mortar Volume and Weight per Square Foot (may vary with field conditions)

Fresh Mortar	Varies	0.40 ft³	0.25 ft³	0.38 ft³	0.55 ft³
Hardened Mortar	Varies	0.33 ft³	0.20 ft³	0.30 ft³	0.50 ft³
Weight	Varies	44 lbs	26 lbs	41 lbs	70 lbs
Max Thickness	Varies	4.0 in	3.3 in	5.0 in	8.0 in
Average Thickness	24 in.	3.8 in	2.2 in	3.5 in	6.0 in

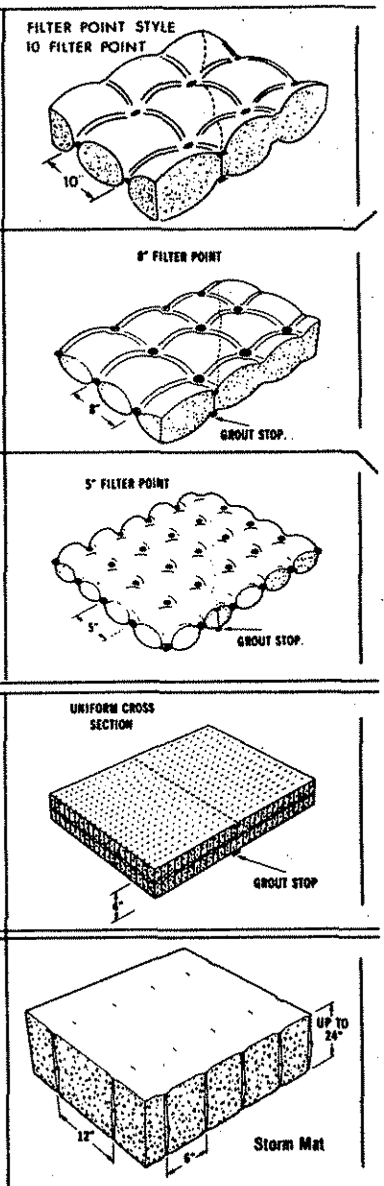


Fig. 5.11. Types of Fabriform.

Uplift pressures are allowed for in the filter point fabric where the filter points act as pressure dissipators. The manufacturer recommends that a geotextile be used when the soil to be protected is a silty one.

5.4.7 Reinforced Concrete

The use of reinforced concrete should be considered as the most elaborate form of LWSC construction, as it is the most expensive of all the materials considered and the strongest.

However, design of a structure from this material is probably the most complicated in terms of overall specifications and safety considerations. For instance, it is vital that adequate protection or allowance for scour around the structure be provided. Otherwise, an expensive structure might be made unusable due to undermining. Additionally, sufficient reinforcing must be provided to guard against failure due to differential settlement. Consideration of depth to reinforcement with regard to the destructive effect of freeze-thaw action, and debris or ice impact, must be made.

5.5 Other Considerations

5.5.1 Erosion Considerations

Previous sections have discussed the erosion of the crossing itself. This section discusses erosion at the site adjacent to the structure. When selecting the site for a crossing, the designer should select a location where the stream channel is stable. If evidence of aggradation, degradation, or lateral migration is present at the proposed location, the designer should attempt to relocate the crossing or provide remedial measures.

Evidence of channel degradation includes newly exposed sediments in the stream bank, exposed piling and/or abutments, and large scale mass movements of the bank. If the crossing must be located in the reach of active degradation, the crossing itself may serve as a grade stabilization structure; however, downstream cutoffs or a stilling basin should be provided to avoid undercutting of the structure.

Channel aggradation is evidenced by sediment covering structures or vegetation. If the crossing cannot be relocated in this situation, the extent of future aggradation should be estimated and the elevation of the crossing and the size and location of the pipes should be adjusted to accommodate the future stream profile.

Any crossing situated in the bend of a river may be subjected to lateral migration with erosion occurring on the convex side of the bend and deposition occurring on the concave side of the bend. If the crossing must be located at such a site, appropriate bank protection measures must be employed to stabilize the channel. It should be recognized that if a low water crossing is proposed for a site where either degradation, aggradation, or lateral migration is occurring, a bridge may be a more economical alternative.

Once the site for the crossing is selected, the designer must make provision for erosion which may occur adjacent to the structure. In order to protect against this, erosion resistant material or cutoff walls should be provided. The exit velocity, depth of scour, and length of stilling basin can be estimated from relationships given in Corry, et al. (1978).

5.5.2 Seepage Considerations

Two potential problems can arise as the result of subsurface seepage beneath hydraulic structures: excessive uplift pressures and piping. The probability of these problems increases with increasing head difference between the upstream and downstream sides of the crossing. In vented fords it is unlikely that the head difference will exceed several inches whereas in the case of a ford, head differences more than two feet might occur. A flow net analysis was done using typical ford geometries and sediment properties for a two foot head difference. This analysis indicated that without any cutoff for seepage control, the possibility for problems of excessive uplift pressures and high exit gradients is unlikely and cutoffs for seepage control are not necessary. However, if the designer anticipates unusual conditions, a flow net analysis should be conducted to evaluate both pore pressure distribution and exit gradients for conditions of no cutoff and various cutoff geometries. Lambe and Whitman (1979) provide clear and concise examples of appropriate analyses.

Although a cutoff may not be justifiable as a means of seepage control, it may be necessary as protection against scour. The presence of a cutoff wall on the downstream side of a low water crossing will have the effect of decreasing seepage quantities and decreasing exit gradients relative to a condition of no cutoff. However, the cutoff will have a tendency to increase uplift pressures on the downstream side of the crossing. Therefore, it is recommended that if a cutoff is designed for scour control, the structure should be analyzed with a flow net to ensure that the pore pressures are not excessive.

5.5.3 Minimum Soil Cover Over Pipes

In certain situations the soil cover over the pipes may be so low that the surface loads will cause excessive deformations of the conduit. For flexible conduits, the rule of thumb is that the minimum depth of soil cover shall be one-eighth the conduit diameter but not less than one foot (Watkins, 1975). For all practical purposes in the case of low water crossings, the minimum cover will be one foot. In the case of rigid conduits, the strength of the conduit is based upon the three edge bearing tests (American Concrete Pipe Association, 1970). The test load is more severe than a wheel load and, therefore, the three edge strength is conservative even for zero soil cover and a factor of safety of two would be sufficient for impact effects and other uncertainties.

6. DESIGN EXAMPLE

6.1 Site Data

The site is located in western Iowa: Region II for flow-duration estimates and Region I for flood return period estimates. Its drainage area is 40 square miles. The stream slope at the site is ten feet per mile or 0.0019 feet per foot. No information is available concerning the main channel slope between the site and the watershed divide. Figure 6.1 shows the cross section of the main channel at the site. The section is deep and wide due to the loess soils in the area.

Manning's roughness coefficient for both the channel and overbank area is 0.04. The overbank area slopes toward the channel on both sides at a one percent slope as does the existing road at the site. A determination has been made that the road could be closed two percent of the time or about seven days per year on the average.

6.2 Discharge Estimates

The regression coefficients to be used in Eq. (3.1) for the flow-duration estimates were taken from Table 3.1. The regression coefficients to be used in Eq. (5.4) for the flood return period estimates were taken from Table 5.2. These values and the discharges estimated from these equations for various durations and return periods are listed in Table 6.1. The discharges have been rounded off to two significant figures.

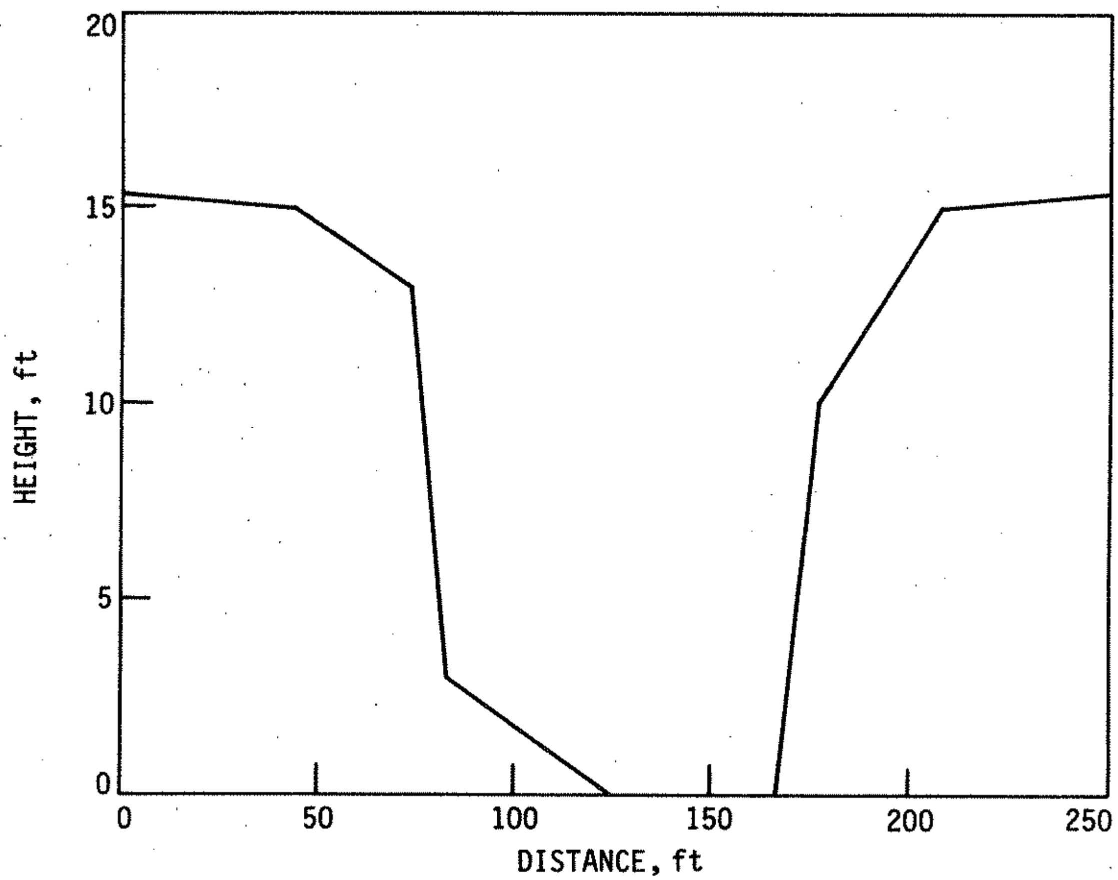


Fig. 6.1. Channel cross section at example site.

Table 6.1. Discharge estimates for the example site.

e, %, or R.I., yr (1)	Coefficient (2)	Exponent (3)	Discharge cfs (4)
$D_{10\%}$	0.15	1.19	12
$D_{5\%}$	0.33	1.15	23
$D_{2\%}$	1.23	1.06	61
$D_{1\%}$	3.56	0.96	120
Q_{10}	667	0.480	3,900
Q_{25}	1040	0.455	5,600
Q_{50}	1390	0.437	7,000

6.3 Stage-Discharge Curve

The calculations for the stage-discharge curve for the cross section shown in Figure 6.1 are contained in Table 6.2. These calculations are based on Manning's formula, Eq. (3.2). Substituting the values from section 6.1 into this equation, the following equation is obtained.

$$Q = 1.49\ AR^{2/3}\ (0.0019)^{1/2}/0.04 = 1.62\ AR^{2/3} \qquad (6.1)$$

The stage-discharge curve, columns 1 and 6 of Table 6.2, for the low flow is depicted in Figure 6.2. Figure 6.3 is the stage-discharge curve for the higher flows. Figure 6.4 is the stage-velocity curve.

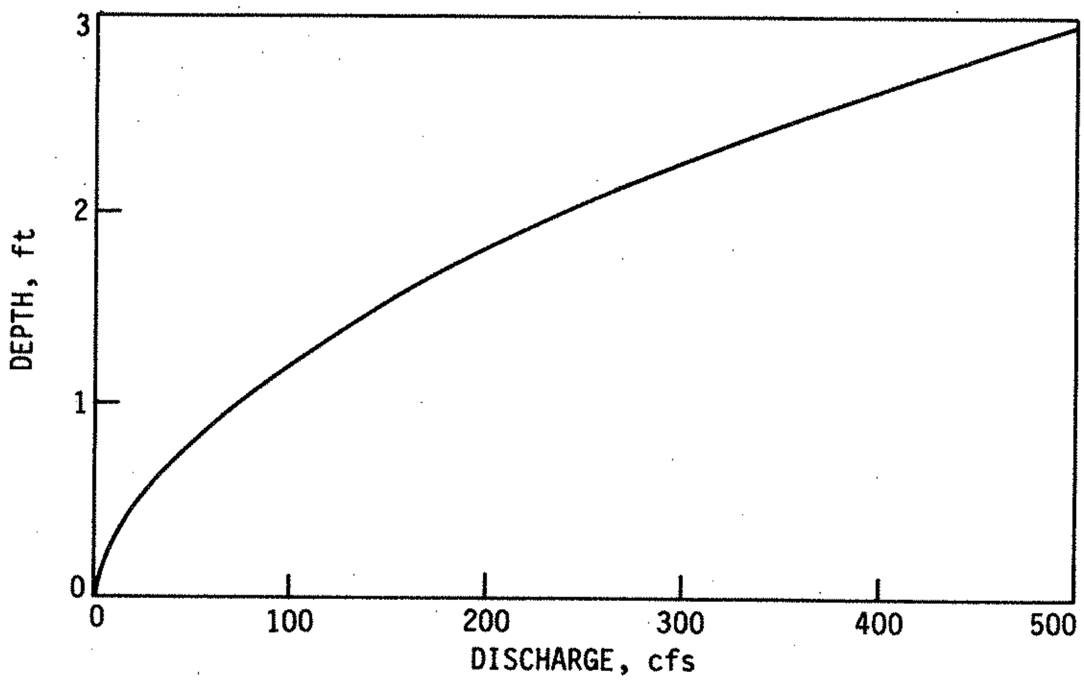


Fig. 6.2 Low flow stage-discharge curve.

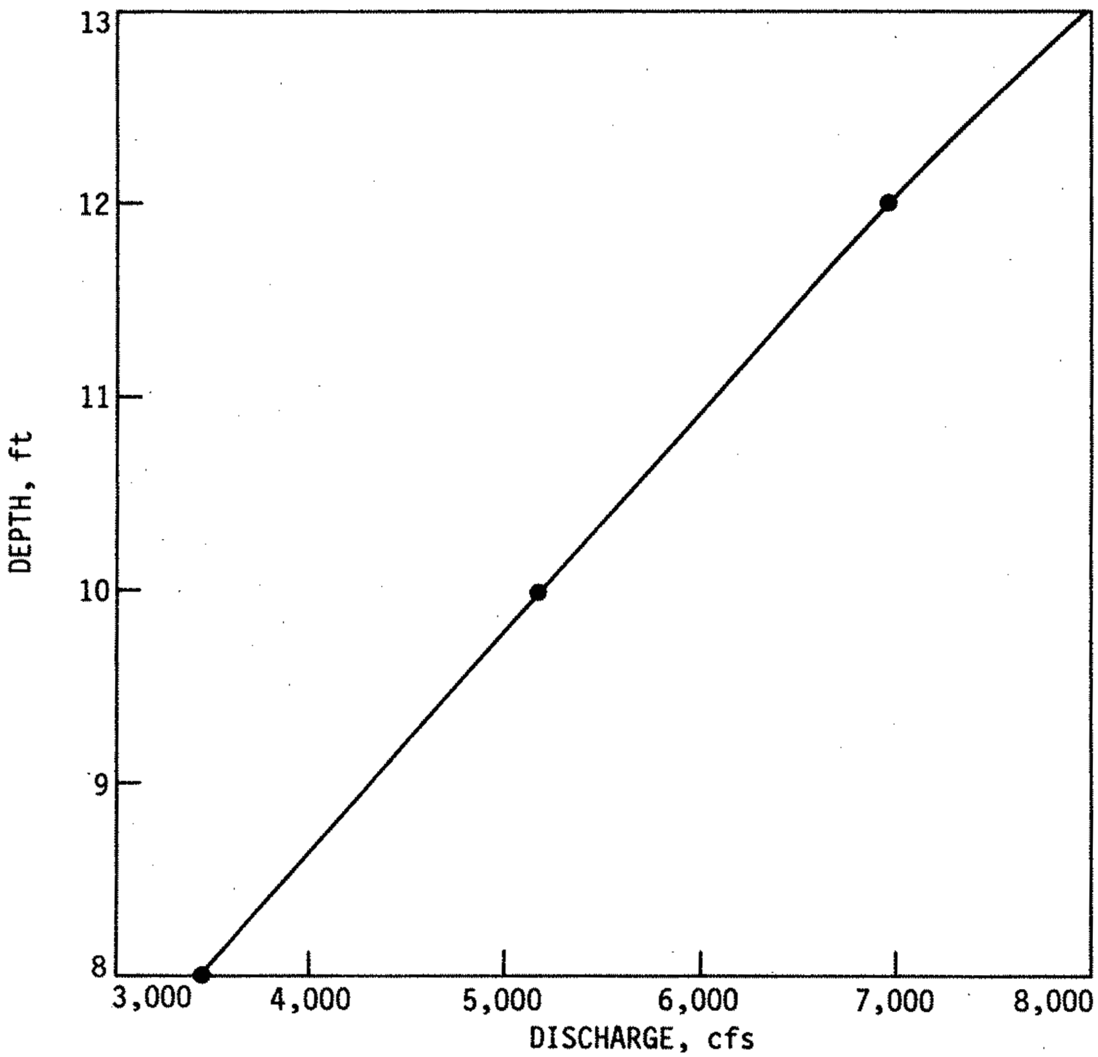


Fig. 6.3. High flow stage-discharge curve.

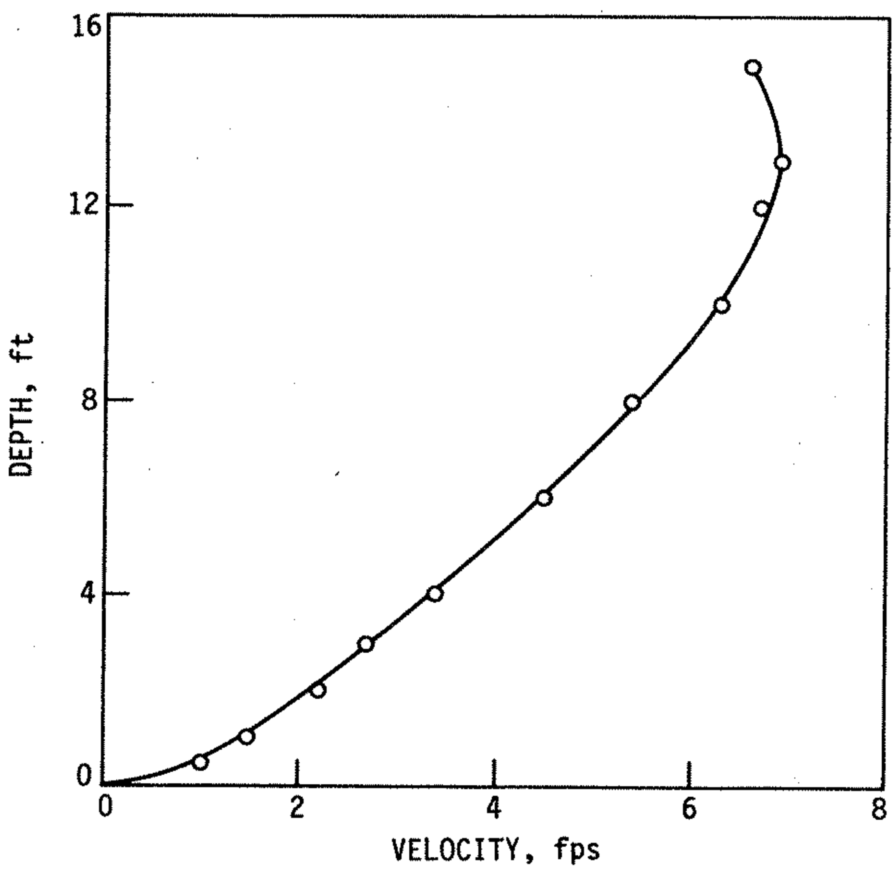


Fig. 6.4. Stage-channel velocity curve.

Table 6.2. Stage-discharge curve calculations for example problem.

D ft (1)	A sq ft (2)	WP ft (3)	R ft (4)	$R^{2/3}$ (5)	Q cfs (6)	V fps (7)
0.0	0	0	0.00	0.00	0	0.0
0.5	23	50	0.46	0.60	22	1.0
1.0	49	57	0.86	0.90	72	1.5
1.5	78	65	1.20	1.13	143	1.8
2.0	112	73	1.53	1.33	241	2.2
2.5	149	81	1.84	1.50	362	2.4
3.0	189	88	2.14	1.66	510	2.7
4.0	274	91	3.01	2.08	926	3.4
6.0	450	97	4.64	2.78	2,030	4.5
8.0	634	103	6.16	3.36	3,450	5.4
10.0	826	108	7.65	3.88	5,190	6.3
12.0	1,036	123	8.42	4.14	6,950	6.7
13.0	1,152	131	8.79	4.26	7,950	6.9
15.0	1,432	173	8.28	4.09	9,490	6.6

6.4 Number and Size of Pipe

As suggested in section 3.4, several combinations of number and sizes of CMP were assumed and headwater depths determined using the appropriate chart in Bulletin 5. The results are shown in Table 6.3. The discharge of 61 cfs from Table 6.1 for $D_{2\%}$ was assumed to be equally divided between the pipes. A few alternatives were rejected

Table 6.3. Headwater depths for various number and sizes of CMP pipe operating under inlet control.

Diameter inches (1)	Number (2)	HW/D (3)	HW feet (4)
12	10	3.4	3.4
12	12	2.6	2.6
12	14	2.1	2.1
15	7	2.5	3.1
15	9	1.8	2.2
15	11	1.4	1.8
18	4	3.0	4.5
18	6	1.8	2.7
18	8	1.2	1.8

because the headwater depth was too great. Two were rejected because the depth of cover over the pipe was less than one foot.

Three alternatives were selected for further review because a headwater depth of 3 to 4 feet seemed "reasonable" for this site. These results for outlet control are shown in Table 6.4. Note that in Tables 6.3 and 6.4 of this example, outlet control governs for all three pipe sizes. All three sets of pipe will fit in the existing channel. Use the nine 15-inch CMP. The low point in the roadway should be set 3.5 feet above stream bed.

Table 6.4. Headwater depths for pipes operating under outlet control.

PROJECT: Example

DESIGNER: Rossmiller

DATE: ______

HYDROLOGIC AND CHANNEL INFORMATION

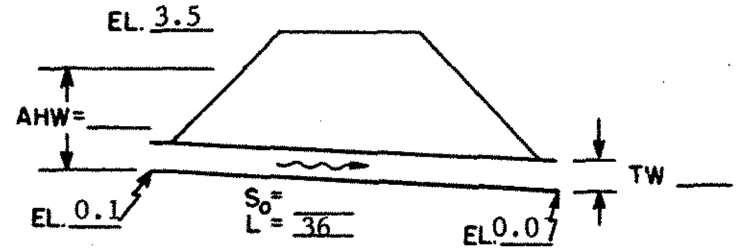
Q_1 = ______ TW_1 = ______

Q_2 = ______ TW_2 = ______

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

SKETCH

STATION: ______



MEAN STREAM VELOCITY= ______

MAX. STREAM VELOCITY= ______

CULVERT DESCRIPTION (ENTRANCE TYPE)	Q	SIZE	HEADWATER COMPUTATION										CONTROLLING HW	OUTLET VELOCITY	COST	COMMENTS
			INLET CONT.		OUTLET CONTROL $HW = H + h_o - LS_o$											
			$\frac{HW}{D}$	HW	K_e	H	d_c	$\frac{d_c+D}{2}$	TW	h_o	LS_o	HW				
CMP-mitered	4.4	12			0.7	2.5	0.9	1.0	0.9	1.0	0.1	3.4				14 pipes
CMP-mitered	7.6	15			0.7	2.2	1.1	1.2	0.9	1.2	0.1	3.3				9 pipes
CMP-mitered	10.2	18			0.7	2.2	1.3	1.4	0.9	1.4	0.1	3.5				

SUMMARY & RECOMMENDATIONS:

6.5 Roadway Profile and Cross Section

Figure 6.5 shows the profile selected for this site. The dashed line is the existing channel. This profile was designed in the following manner. As stated in section 6.1, the existing road slopes toward the channel from both sides at a one percent grade. Because of the width and depth of the existing channel, ten percent grades were sketched in and looked "reasonable." Twelve percent grades might also have been used, but speeds would have been reduced somewhat from the ten percent grades.

The station and elevation of the PVI for the sag vertical curve were determined in the following manner. The station was set midway between the channel banks so that the sag vertical curve would be symmetric with the channel. Thus, when flow depths are five feet and greater, water will flow from bank to bank over the crossing with the minimum turbulence possible. The disadvantage of this arrangement is that the low flow channel is offset to the right side of the channel. This makes the pipe at the center of the total channel about five feet shorter than the pipe nearest the channel bank because of the difference in roadway elevations at these points.

This disadvantage is minor compared to the situation depicted in Figure 6.6. Here the station of the PVI has been shifted 20 feet to the right. Although the pipe lengths now are more or less equal because of the small differences in roadway elevation, flow over the crossing is concentrated towards the right bank. Flow near the left bank must move towards the right because the roadway elevation is 2.5 feet higher

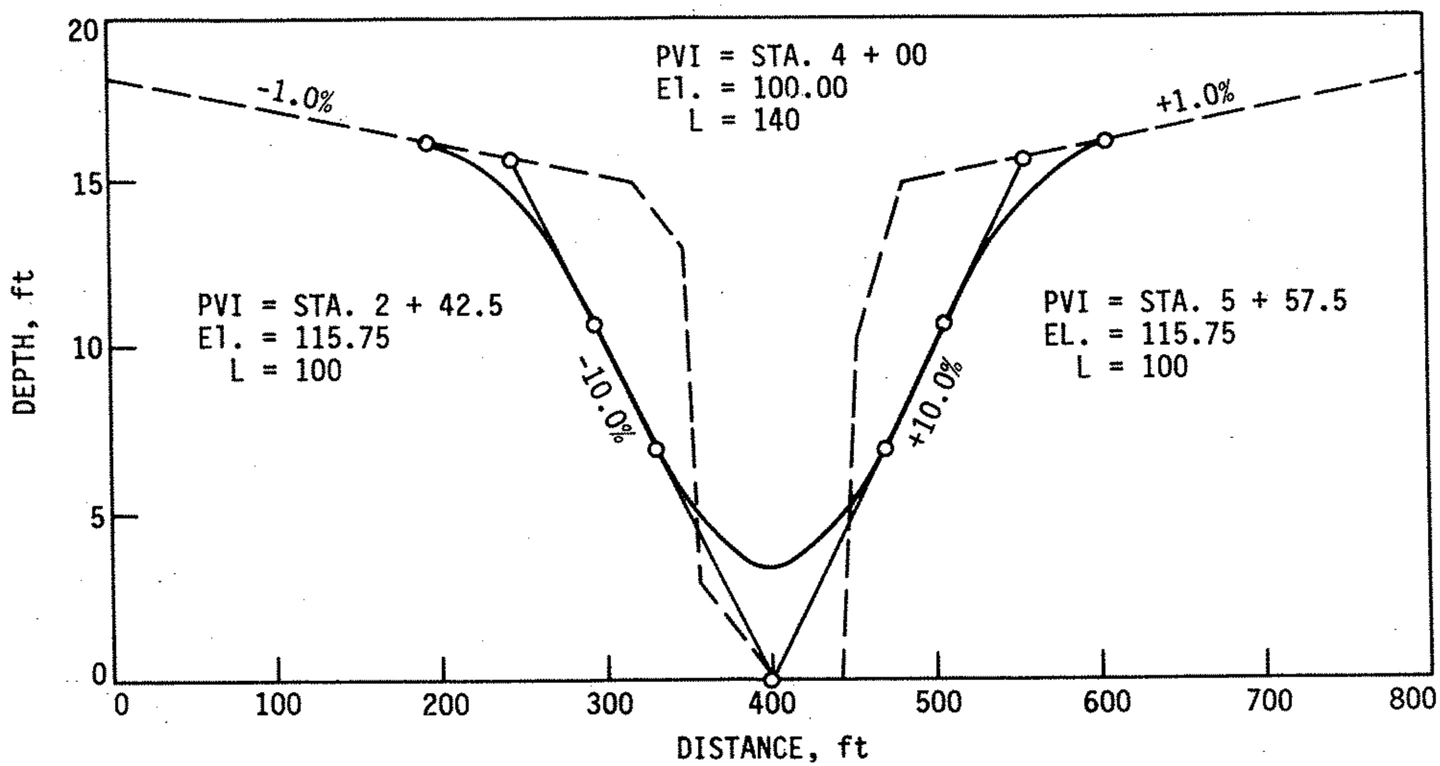


Fig. 6.5. Roadway profile for example problem.

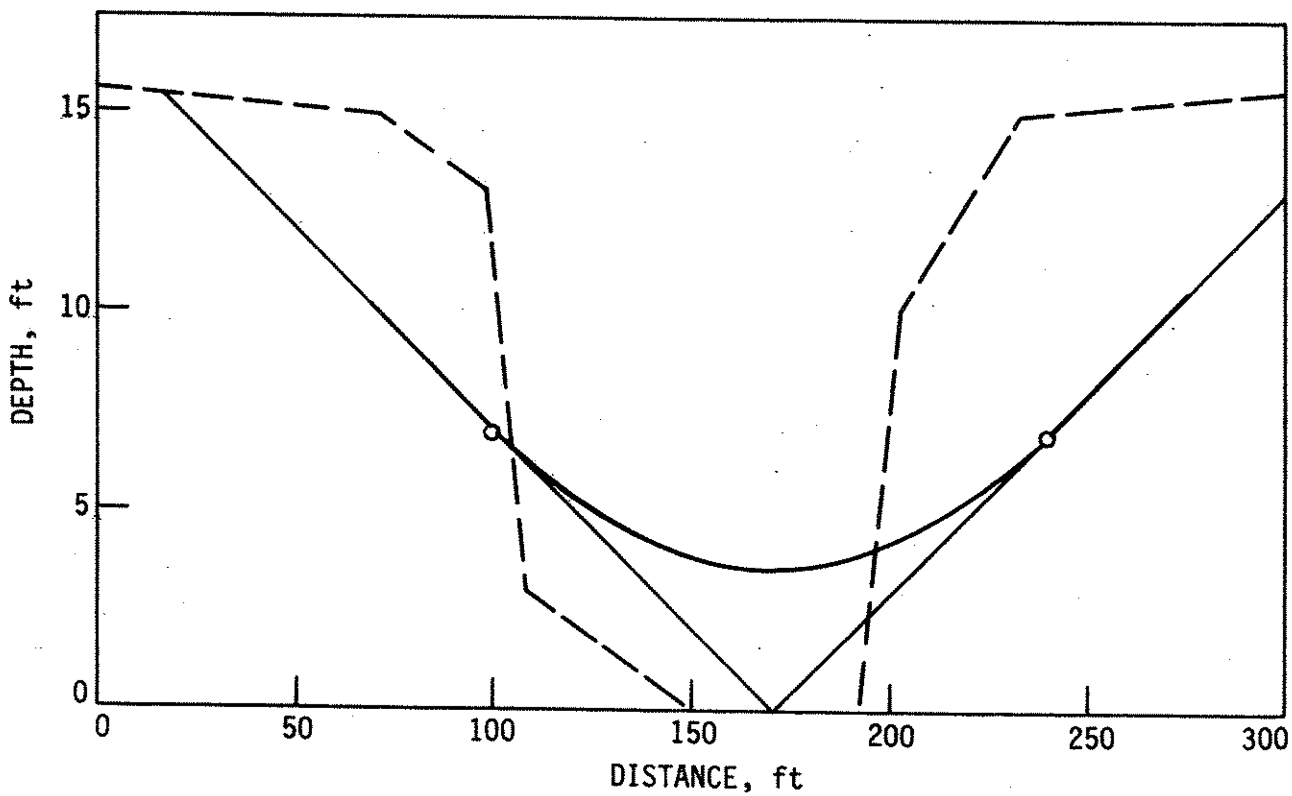


Fig. 6.6. Alternate location of PVI for sag vertical curve.

near the left bank. The additional turbulence in this situation could cause erosion of the right bank.

The elevation of the PVI for the sag vertical curve was set in the following manner. As a first trial, the tangent offset at the midordinate (MO) was selected as 3.5 feet and the length of curve determined as shown below.

$$MO = AL/8 \tag{6.2}$$

$$L = 8MO/A = 8 \times 3.5/20 = 1.4 \text{ stations} \tag{6.3}$$

From Figure 6.5, this elevation (stream bed elevation) and length of vertical curve (140 feet) looked "reasonable" for this site and they were adopted. This decision was based on the design concern that the vertical dimension from stream bed to low point in the roadway could be optimized. Too large a value will result in excessive fill, concrete required, and extended culvert lengths. Too small a value will result in inadequate fill over the top of the pipes.

Having selected a sag vertical curve of 140 feet, the next step was to determine the speed for this roadway design. Using Figure 4.9, for A = 20 and L = 140, a speed of approximately 10 mph was indicated.

In selecting the crest vertical curves, determining the tangent lengths available after the sag vertical curve has been established is the initial step. A maximum of 175 feet is available at either crest if no tangent distance is to be used. In this example, a 100 foot curve was selected at each crest.

Figure 4.8 was used to determine the speed for the crest vertical curves. Both curves have A = 9 and L = 100 values and a speed of 15 mph was indicated.

The advisory speed plate should be for 10 mph since the sag vertical curve controls this LWSC design speed. If the design speed were to be changed, the sag vertical curve could be lengthened (with subsequent impact on the fill quantities and length of pipes) and/or the tangent grades reduced.

The roadway will have a 24-foot top width sloped at a two percent grade in the direction of flow with 2:1 foreslopes as depicted in Figure 4.10.

6.6 Material Selection

The material used to protect the crossing itself from erosion was selected using all three methods described in Chapter 5. Three return periods, the 10-, 25- and 50-year floods, were used in each of the methods.

Method I is described in section 5.1. The site is located in Region I and has a drainage area of 40 square miles, based on the data listed in section 6.1. The results obtained from Method I are shown in Table 6.5. Column 1 is the assumed return periods. The tractive forces in column 2 were obtained from Figure 5.2. The velocities in column 3 were obtained from Figure 5.3. These velocities are too

Table 6.5. Velocity and tractive force using Method I.

Return Period years (1)	Tractive Force lb/ft^2 (2)	Velocity fps (3)
10	0.8	6.3
25	1.2	7.5
50	1.5	8.3

high for vegetation to be used. Comparing the tractive forces in column 2 with Table 5.1, riprap with D_{50} equal to six inches is adequate for this site for all three return periods.

Method II is described in section 5.2. The results obtained by using this method are shown in Table 6.6. Column 1 is the assumed return periods. The slope in column 2 was obtained from Figure 5.6. The depths listed in column 3 were also obtained from Figure 5.6. The velocities shown in column 4 were obtained from Figure 6.4 using the depths listed in column 3. The tractive forces in column 5 were calculated using Eq. (5.2) with the slope and depths shown in columns 2 and 3, respectively. The velocities in column 4 are too high for vegetation to be used. Comparing the tractive forces in column 5 with Table 5.1, riprap with D_{50} equal to six inches is adequate for this site for all three return periods.

Method III is described in section 5.3. The results obtained by using this method are shown in Table 6.7. Column 1 is the assumed return periods. The peak discharges in column 2 were calculated

Table 6.6. Velocity and tractive force using Method II.

Return Period years (1)	Slope ft/ft (2)	Depth feet (3)	Velocity fps (4)	Tractive Force lb/ft^2 (5)
10	0.0011	5.8	4.4	0.30
25	0.0011	8.2	5.5	0.56
50	0.0011	10.1	6.3	0.69

using Eq. (5.4) and the regression coefficients listed in Table 5.2. The depths in column 3 were obtained from Figure 6.3. The velocities shown in column 4 were obtained from Figure 6.4 using the depths listed in column 3. The tractive forces listed in column 5 were calculated using Eq. (5.2) with the slope given in section 6.1 and the depths shown in column 3. The velocities in column 4 are too high for vegetation to be used. Comparing the tractive forces in column 5 with Table 5.1, riprap with D_{50} equal to six inches is adequate for this site for all three return periods.

Table 6.7. Velocity and tractive force using Method III.

Return period years (1)	Discharge cfs (2)	Depth ft (3)	Velocity fps (4)	Tractive Force lb/ft^2 (5)
10	3,900	8.5	5.7	0.58
25	5,600	10.5	6.4	0.72
50	7,000	12.0	6.7	0.82

All three methods yield the same results, riprap with D_{50} equal to six inches. Any size riprap, six inches or larger, or gabions or soil cement or concrete can be used, depending on the availability and cost of these materials in the county. In many counties, the cost of larger size riprap can be the same or less than the cost of smaller sizes; therefore, the use of larger riprap can give added protection against erosion without any increase in cost.

As stated in section 6.1, this site is located in western Iowa with loess soils. The crossing will act as a grade control structure to prevent further degradation upstream. Both a cutoff wall and riprap blanket should be used on the downstream side of the crossing to protect it as the channel continues to degrade downstream. The depth of the cutoff wall and the size of the blanket are dependent on site conditions.

7. CONSTRUCTION DETAILS

7.1. General Concepts

A general document with detailed construction procedures and techniques is not practical because of the wide range of construction materials and variations in design. The intent of this chapter is to review the various elements of a LWSC in terms of design and construction and to suggest alternatives and the ramifications associated with certain decisions.

7.2. Vented Fords

The construction of a vented ford consists of six general components: core, pipes, riding surface, sidewalls and cutoff walls, upstream and downstream erosion protection, and approaches. These components are shown in Figure 7.1. Because of the wide range in designs, materials selected, and maintenance practices at a given site, an overview of current practice is desirable.

7.2.1. Core

The core material will normally consist of earth, sand, gravel, rubble, broken concrete, or combinations. The construction procedures of placing and compacting at a given site are dependent on the core material selected. The design phase will have investigated velocity erosion potential due to overtopping, undermining, and seepage based on the core cover protection and cutoff walls.

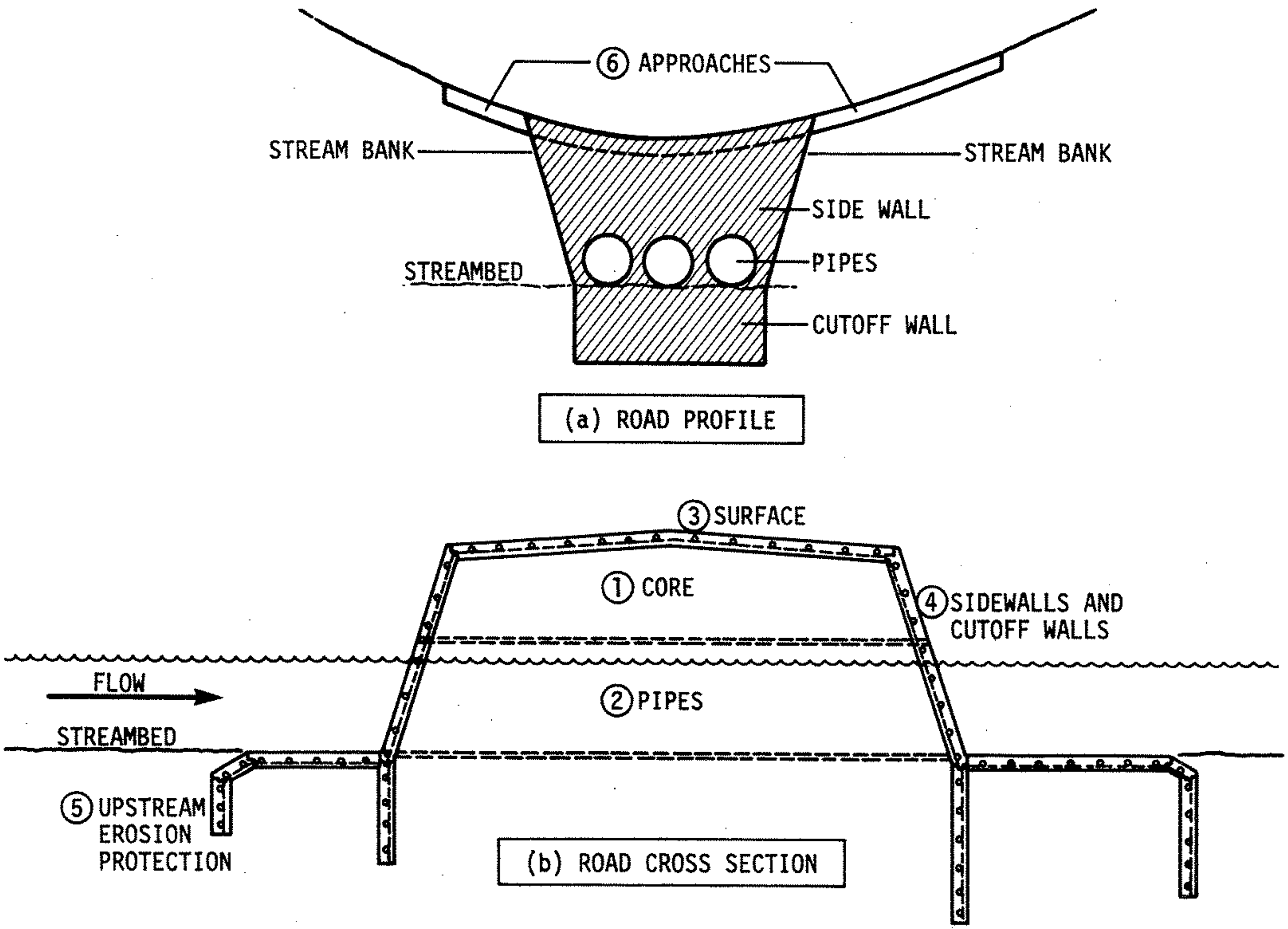


Fig. 7.1 Six (construction) components of a LWSC.

7.2.2. Pipes

Corrugated metal, PVC, and precast concrete pipes are commonly used for LWSCs. The details of assembling and placing are dependent on the normal practices for the material selected.

For smoother hydraulic operation, and to reduce the potential for clogging, both ends should be mitered to fit the sidewall slope. Diaphrams commonly are used to reduce seepage. Some designs utilize one or more cables anchored to an upstream piling and tied to the pipe or diaphrams to hold the pipe in place in case of a wash out of the core material. See Figure 7.2.

7.2.3. Surface

The surface material of the roadway normally will consist of gravel, rubble, hot or cold asphaltic materials, or Portland Cement (PC) concrete. The selection of material at a specific site is based on a design analysis considering erodibility from overtopping and rideability for the anticipated traffic. If concrete is used, a prominent texture is required to increase traction following overtopping and the subsequent deposits on the surface. A crown should be constructed to assure drainage and to preclude ponding on the surface. Surfaces other than the rigid type should have a steeper crown.

If curbs, buttons, or other edge identifying elements are used, care should be taken that the surface will drain completely after overtopping and that the shape is self cleaning. Some roadway surfaces will require maintenance after every overtopping.

Joints in PC concrete should be tied to reduce the problem of opening and stream intrusion with subsequent core material erosion.

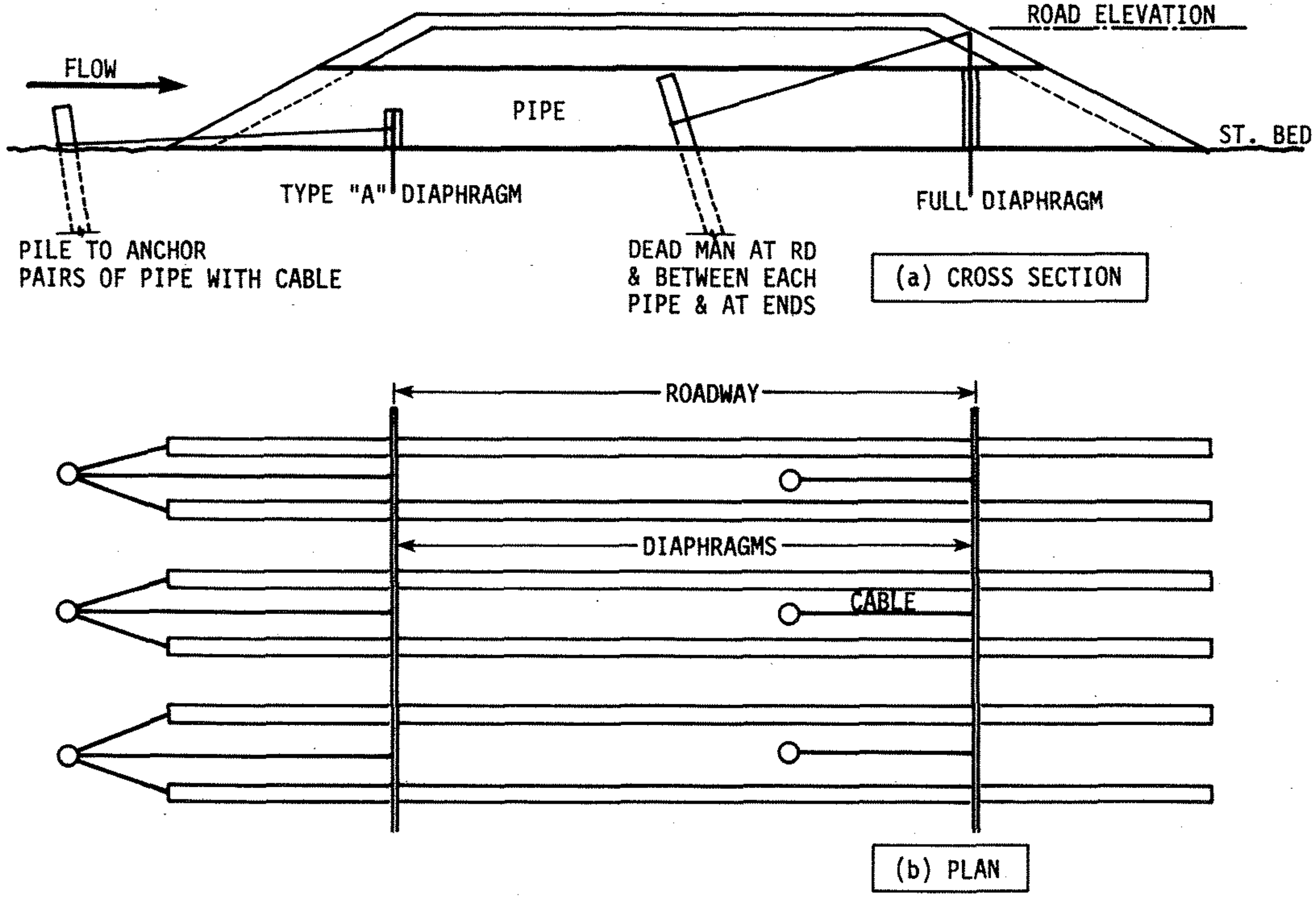


Fig. 7.2 Cable anchor details.

The use of a geotextile fabric may be appropriate based on the materials selected.

7.2.4. Sidewalls and Cutoff Walls

The function of the LWSC sidewalls (roadway foreslopes) is to protect the edges of the structure and prevent erosion of the core material. Sidewalls also serve as a support for the roadway surface and if a vehicle leaves the roadway they are of concern from a safety standpoint. A slope of at least 2:1 is recommended for safety reasons and to improve the self-cleaning aspects and flow in the pipes. A vertical side-wall is not recommended.

If the sidewalls are constructed of concrete, the joints should be tied to reduce intrusion of stream flow. If rip-rap is used, the pieces should be selected and placed to minimize the openings and subsequent access to the core material. Geotextiles also may be appropriate in this application.

If the sidewalls are not tied into bedrock or a firm foundation of non-erodible material, cutoff walls may be necessary to protect against scouring. If cutoff walls are required, they normally will be required both upstream and downstream. Cutoff walls can be concrete, rubble, or sheet piling. See Figure 7.3.

7.2.5. Upstream and Downstream Erosion Protection

Because the LWSC is designed for overtopping on a relatively regular basis, consideration for stream bed erosion protection is desirable. Horizontal aprons extending upstream and downstream will reduce the scour in erodible channels as shown in Figure 7.4. These aprons will reduce the potential for the high water flows to create

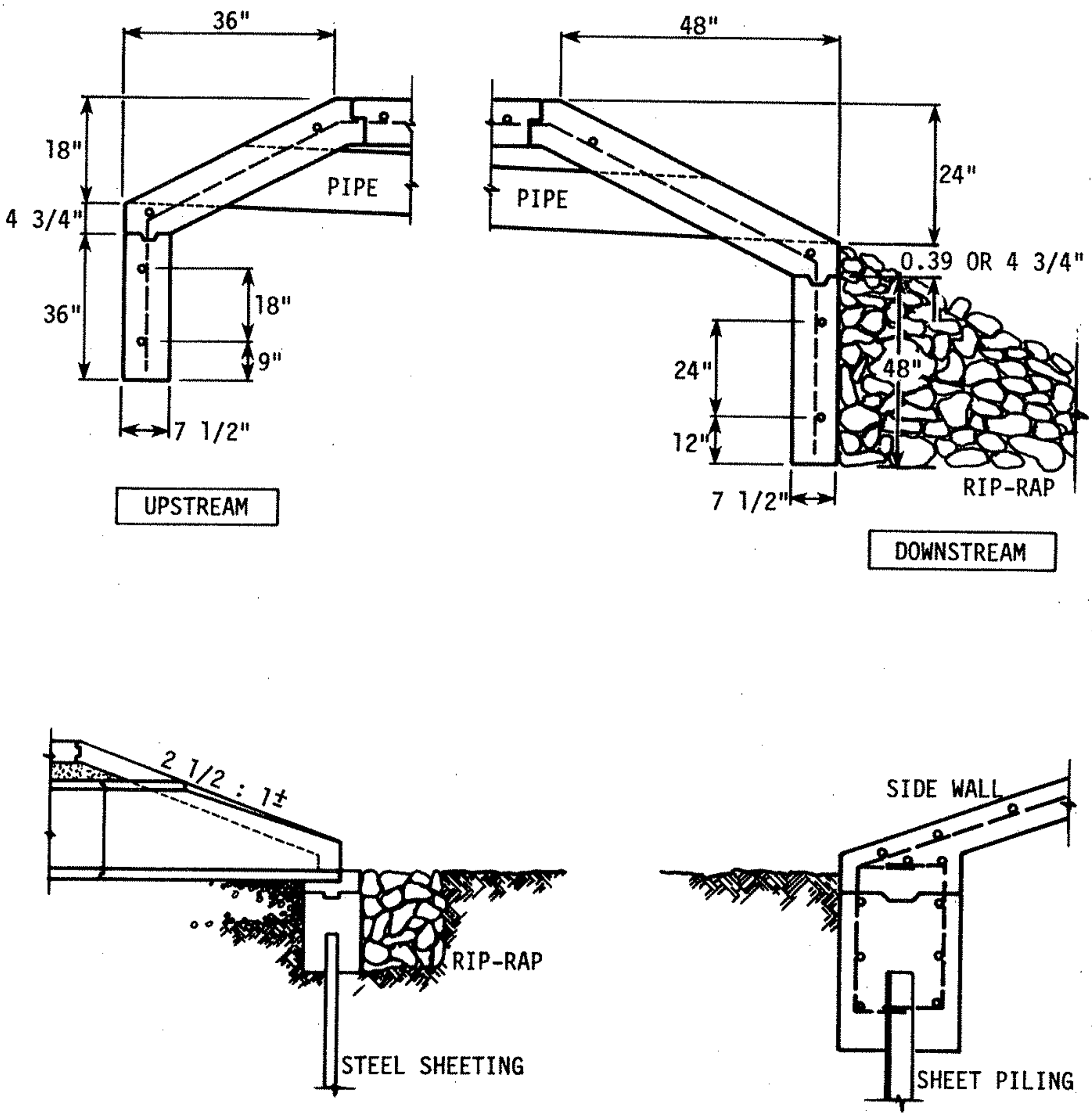


Fig. 7.3. Typical sidewall and cut off wall sections.

scour pools with subsequent undermining of the sidewalls. Aprons may be constructed of concrete or riprap.

7.2.6. Approaches

The LWSC roadway surfacing material should be extended in each direction away from the structure in order to reduce problems of erosion and sediment deposit associated with overtopping flows. The surfacing material used on the LWSC should be extended outside the limits of a 10-year return period as indicated in Figure 7.5.

7.3. Unvented Fords

The simplest form of LWSC is the unvented ford as illustrated in Figure 7.6. Construction may be in one of the following forms: (a) the roadway surface coincides with the stream bed, (b) the roadway surface has been excavated below the stream bed, and (c) the roadway surface has been raised above the stream bed. In any case the construction should assure a stable tractive surface suitable for the vehicles using the facility and protect the LWSC from erosion.

When the LWSC is on a stream bottom that is stable, such as bed rock or coarse gravel, case (a) may be applicable. In some unique cases, the stream bed may be utilized as the roadway surface and the vehicles simply follow the roadway alignment on each side of the stream to identify the crossing location. In most locations in Iowa, the stream bed material is not suitable for a stable tractive roadway surface. Because of this situation, excavation below the stream bed must occur so that a gravel, rubble, or in some cases concrete surface can be placed.

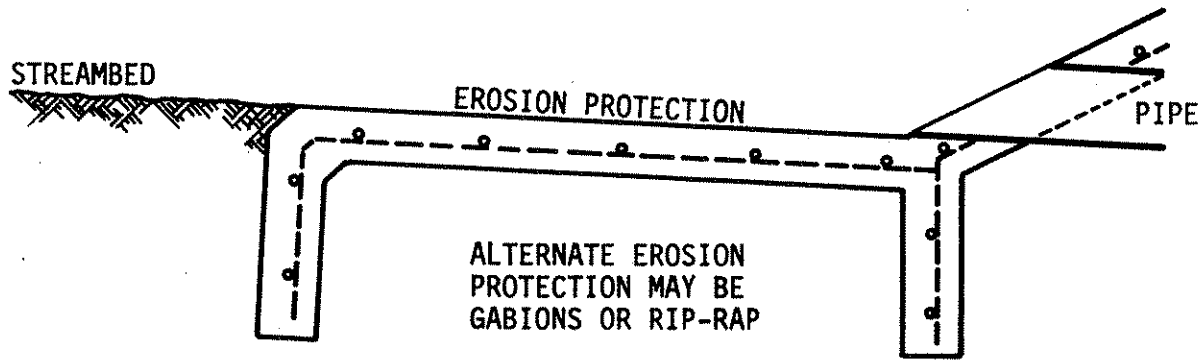


Fig. 7.4. Typical erosion protection.

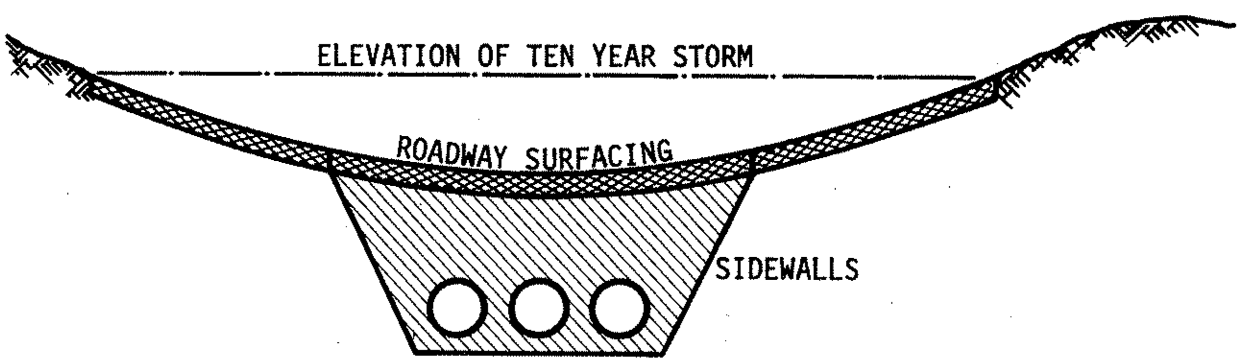


Fig. 7.5. Minimum limits of LWSC roadway surfacing.

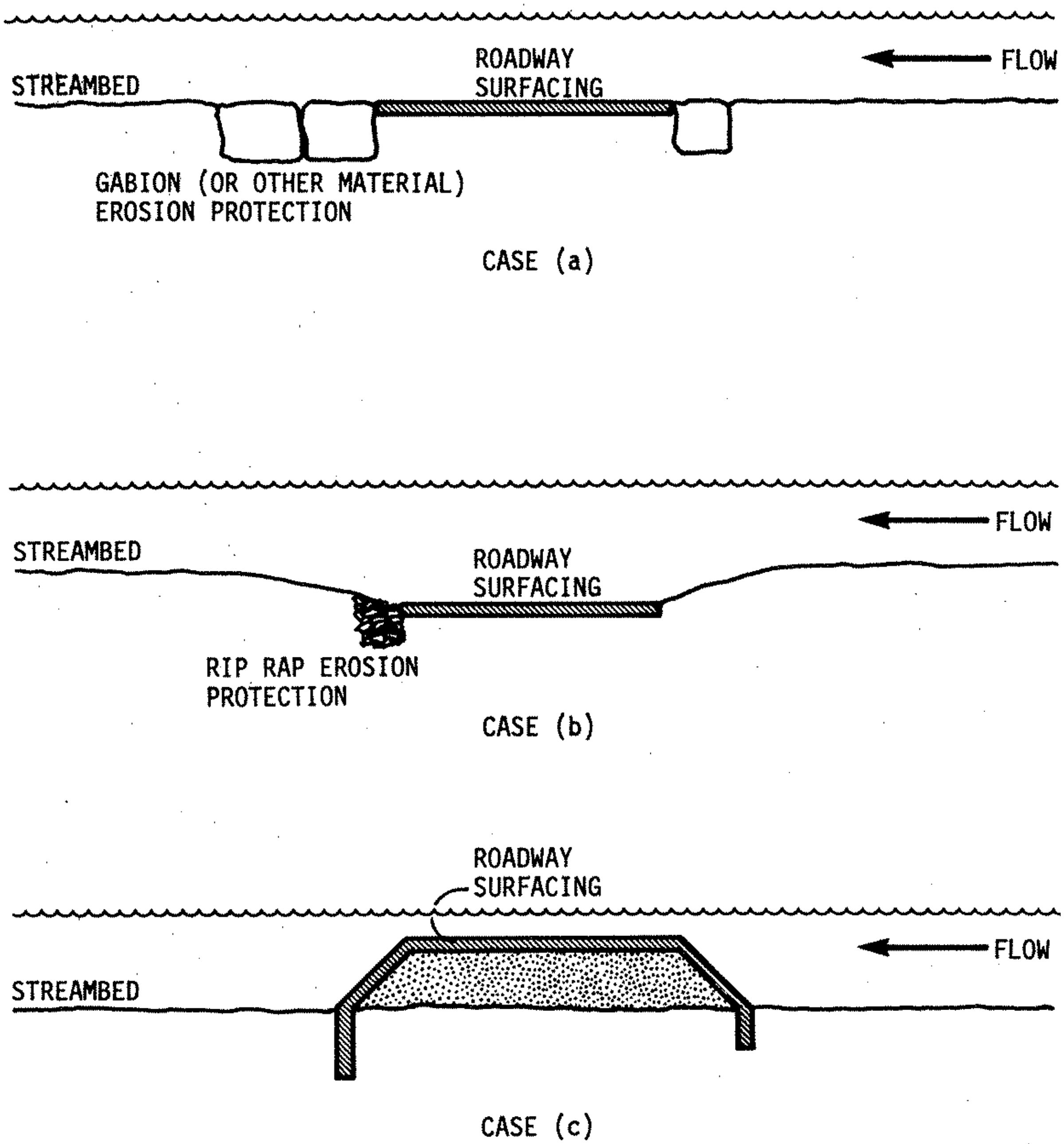


Fig. 7.6. Typical fords--roadway cross section.

If the stream bed is of a readily erodible material during higher flows, then the crossing, as depicted in case (b), may be applicable. This form of LWSC allows for some stream degradation with minimal impact on the crossing roadway. In the case of a flood event, the LWSC is not washed out and all that is necessary to place it back in operation is removing any deposited material on the roadway. The surfacing material may be any material used in case (a).

If the stream has high banks, so that approach grades preclude the use of case (a) or (b) crossings, it may be necessary to raise the LWSC above the stream bed as depicted in case (c). However, since all flow must overtop the LWSC, there must be protection of the fill material. An encasement of the core material, including surface and sidewalls, may be necessary if the core material is erodible. Also, in an erodible stream, sidewalls, cutoff walls, and upstream and downstream erosion protection may be necessary to reduce scour and wash out. This design could be similar to the vented fords previously discussed but with no pipes.

Edge of the roadway surface protection may be necessary in all cases. A variety of endwall treatments have been used ranging from boulders, rubble, rip rap, gabions, and poured concrete cutoff walls. This treatment to reduce scour and undermining is used on the downstream side and may be required on the upstream side.

APPENDIX A

NOMENCLATURE

Symbol	Description
	Hydrology
A	Drainage area in square miles
a	Regression coefficient
b	Regression coefficient
c_t	Regression coefficient for a t-year return period
e	Exceedance probability in percent
Q_e[1]	Discharge in cubic feet per second for some exceedance probability
Q_t[1]	Discharge in cubic feet per second for some return period
S	Main channel slope in feet per mile, determined from the elevations at points 10 percent and 85 percent of the distance along the channel from the design point to the divide
t	Return period in years
x_t	Regression coefficient for a t-year return period
y_t	Regression coefficient for a t-year return period

[1]The difference between Q_t and Q_e is as follows. Q_{50} (Q_t) is the magnitude of flood, measured in cubic feet per second, which has a two percent chance of being equaled or exceeded in any one year, i.e., 1/50 = 0.02 = 2 percent. $Q_{50\%}$ (Q_e) is the magnitude of low flow, measured in cubic feet per second, which will be equaled or exceeded 50 percent of the time, i.e., if a LWSC were designed for $Q_{50\%}$, the road would be overtopped on the average of six months each year. On the other hand, a flood equal to Q_{50} would be experienced on the average of only once every 50 years.

Symbol	Description
	Hydraulics
A	Cross-sectional area of flow in square feet
B	Width of a box culvert in feet
B_r	Breadth of roadway, shoulder to shoulder, in feet
C	Coefficient of discharge in weir formula
D	Diameter of pipe in feet
D_b	Height of box culvert in feet
d	Depth of flow in feet
d_c	Critical depth in feet
g	Acceleration due to gravity
H	Total head loss in feet between inlet and outlet of culvert
HW	Headwater depth in feet at entrance of a culvert
H_w	Total head on a weir in feet
h	Head on weir in feet, equal to depth of flow above crest
h_o	Height above culvert invert at the outlet in feet, equal to tailwater depth or height above invert of the equivalent hydraulic grade line, $(d_c + D)/2$
L	Length of culvert in feet
L_f	Length of flow section along the roadway, normal to the direction of flow, in feet
M_5	Degree of channel meandering, component of Manning's n
n	Manning's roughness coefficient
n_o	Material involved, component of Manning's n
n_1	Degree of irregularity in channel cross section surface, component of Manning's n
n_2	Variation in channel cross section along its length, component of Manning's n

Symbol	Description
n_3	Relative effect of obstructions, component of Manning's n
n_4	Relative height of vegetation, component of Manning's n
P	Difference between stream bed elevation and elevation of the low point in the roadway in feet
Q	Discharge in cubic feet per second
R	Hydraulic radius in feet, equal to A/WP
S	Channel slope at the site in feet per foot
S_o	Culvert slope in feet per foot
V	Mean velocity of flow in feet per second
W	Width of channel in feet
WP	Wetted perimeter in feet
Z	Channel side slope, horizontal to vertical

Geometrics

Symbol	Description
A	Algebraic difference in grades ($G_1 - G_2$) in percent
a	Vertical radial acceleration in feet per second^2
d	Minimum stopping sight distance in feet
e	Depth of water over crossing in feet
f	Coefficient of friction (braking)
G	Highway grade tangent in percent
g	Highway grade in percent (at a specific location)
h_1	Height of driver's eye in feet
h_2	Height of object in feet
K	Length per percent A in feet
L	Length of vertical curve in feet
l	Length of spread of water on crossing in feet

Symbol	Description
R	Rate of change in grade in percent per station
t	Perception reaction time in seconds
V	Motor vehicle speed in miles per hour

Material Selection

Symbol	Description
A_t	Cross-sectional area of flow for a t-year return period in square feet
a	Regression coefficient
b	Regression coefficient
c_t	Regression coefficient for a t-year return period
D_a	Drainage area in square miles
D_{50}	Size of riprap sample, 50 percent of which is finer by weight
d_t	Depth of flow for a t-year return period in feet
G	Specific gravity of a material
n	Manning's roughness coefficient
P	Difference between stream bed elevation and elevation of the low point in the roadway in feet
Q_t	Discharge for a t-year return period in cubic feet per second
R	Hydraulic radius in feet
r	Correlation coefficient
S	Bed slope of channel in feet per foot
t	Flood return period in years
V_t	Velocity of flow for a t-year return period in feet per second
W	Weight of rock in pounds
W_t	Flow surface width for a t-year return period in feet
WP_t	Wetted perimeter for a t-year return period in feet

Symbol	Description
x_t	Regression coefficient for a t-year return period
y_t	Regression coefficient for a t-year return period
τ_c	Critical tractive force in pounds per square foot
τ_t	Tractive force for a t-year return period in pounds per square foot

APPENDIX B

RATIONALE FOR SELECTION METHOD GIVEN IN SECTION 5.1

This appendix presents a detailed explanation of the process by which Figures 5.2, 5.3, 5.4, and 5.5 were developed.

This is done in a series of nine steps. The reader is referred to the list of nomenclature included in Appendix A.

Step 1 Determination of Various Flood Magnitudes (Q_t)

The first step in the development of the selection method given in section 5.1 was the calculation of the magnitude of the various floods (Q_t) in cubic feet per second. This was accomplished using the procedure recommended by the United States Water Resources Council (1977).

Step 2 Determination of Flow Depths (d_t) Corresponding to the 10-, 25- and 50-year Floods (Q_t)

Lara (1976) gives stage-discharge data for the gaging stations considered in this report. The gage readings represent the water surface elevation above an arbitrary datum. This means that a special procedure had to be adopted in order to determine the depth for a given return period. The procedure used was as follows.

Knowing discharge from step one, the corresponding stage was determined from the station data. Then assuming that the stage for zero flow would represent approximately the elevation of the channel bed above the unknown datum, this stage was calculated. Hence, by subtracting the zero flow stage from the appropriate flood stage, the depth of flow, d_t, corresponding to the t-year return period flood was obtained.

Step 3 Determination of the Flow Width, W_t, Corresponding to Various Floods, (Q_t)

The calculations for the flood flow widths, W_t, were based on three assumptions. First, it was assumed that the channel was rectangular. This assumption was checked by calculating flow widths based on a trapezoidal channel with 2:1 side slopes for several cases. The differences in the values of W_t obtained for the different channel geometries were negligible and so calculations were made on the basis of a rectangular channel. The second assumption was that the flow was entirely contained by the channel. Of course, some water will flow out onto the flood plain in many instances, however, the velocities outside the channel will be severely reduced by the increased roughness of the flood plain as compared to the channel. The exact percentage of the total flow occurring in the channel will depend on local conditions and can only be accurately determined by measuring the cross-sectional profile of the channel and flood plain and calculating a stage-discharge relationship. Assuming that the flow is contained within the channel does enable the calculation of the worst case in terms of tractive force (τ_t) and flow velocity (V_t).

The third assumption made was the value of Manning's roughness coefficient, n. Henderson (1966) suggests n = 0.035 for a winding channel with pools and shoals. This value was used for the existing channels under consideration.

Manning's equation gives an expression for the velocity of flow, V, in a channel as:

$$V = \frac{1.49\ R^{2/3}\ S^{1/2}}{n} \tag{B.1}$$

where

R = hydraulic radius = $\frac{A}{P} = \frac{\text{cross-sectional area of flow}}{\text{wetted perimeter of channel}}$

S = bed slope

n = Manning's roughness coefficient.

The discharge, Q, is a function of flow velocity and cross-sectional area:

$$Q = VA \tag{B.2}$$

and hence combining Eqs. (B.1) and (B.2) yields Eq. (B.3).

$$Q = \frac{1.49\ AR^{2/3}\ S^{1/2}}{n} \tag{B.3}$$

When expressions for the cross-sectional area, A, and the hydraulic radius, R, in terms of flow depth, d, and surface width, W, are substituted in Eq. (B.3), it takes the form:

$$A = Wd, \quad P = W + 2d$$

$$R = \frac{A}{P} = \frac{Wd}{W + 2d}$$

therefore

$$Q = \frac{1.49}{n} \cdot \frac{(Wd)^{5/3}}{(W + 2d)^{2/3}} \cdot S^{1/2} \qquad (B.4)$$

Using Eq. (B.4) and n = 0.035, values of the surface flow width corresponding to the various return period flood flows obtained under step one and flow depths obtained under step two were calculated by an iterative approach.

Step 4 Plotting of Flow Depth, d_t, and Flow Width, W_t, Against Q_t

Once the values of flow depth, d_t, and flow width, W_t, have been calculated, regression analyses to determine the relationship between d_t, W_t, and Q_t were carried out for Regions I and II. The results of these analyses are given in Table B.1 and the curves that the regression equation represent are shown in Figure B.1 for Region I and Figure B.2 for Region II. The regression analyses were carried out using the method of least squares.

Step 5 Determination of Regional Relationships Between Discharge and Drainage Area, D_a

The statistical model used for the calculation of the various return period in step one provides the best answer for a particular gaging station. However, this report is concerned with the whole of Iowa and thus it was considered better to use Lara's (1973) regional

Table B.1. Results of regression analyses.

Relationship (1)	Region I a (2)	Region I b (3)	Region I Correlation Coefficient (4)	Region II a (5)	Region II b (6)	Region II Correlation Coefficient (7)
$d_{10} = aQ_{10}^{b}$	0.2872	0.3643	0.4993	13.7831	-0.1401	-0.5402
$W_{10} = aQ_{10}^{b}$	0.8535	0.5827	0.5075	0.0041	1.4145	0.9813
$Q_{10} = aD_{a}^{b}$	623.7631	0.5029	0.9572	106*	0.661*	1.000*
$d_{25} = aQ_{25}^{b}$	0.3991	0.3495	0.5085	18.8075	-0.1397	-0.6490
$W_{25} = aQ_{25}^{b}$	0.5587	0.5911	0.5583	0.0040	1.3357	0.9810
$Q_{25} = aD_{a}^{b}$	964.9194	0.4758	0.9450	144*	0.655*	1.000*
$d_{50} = aQ_{50}^{b}$	0.4900	0.3412	0.5216	18.737	-0.1087	-0.6047
$W_{50} = aQ_{50}^{b}$	0.4371	0.5941	0.6008	0.0051	1.2562	0.9855
$Q_{50} = aD_{a}^{b}$	1.0566×10^{-7}	2.2286	0.9675	177*	0.647*	1.000*
$S = aD_{a}^{b}$	0.0101	-0.4013	-0.7959	0.0041	-0.4835	-0.8936

*Values obtained from Lara (1973).

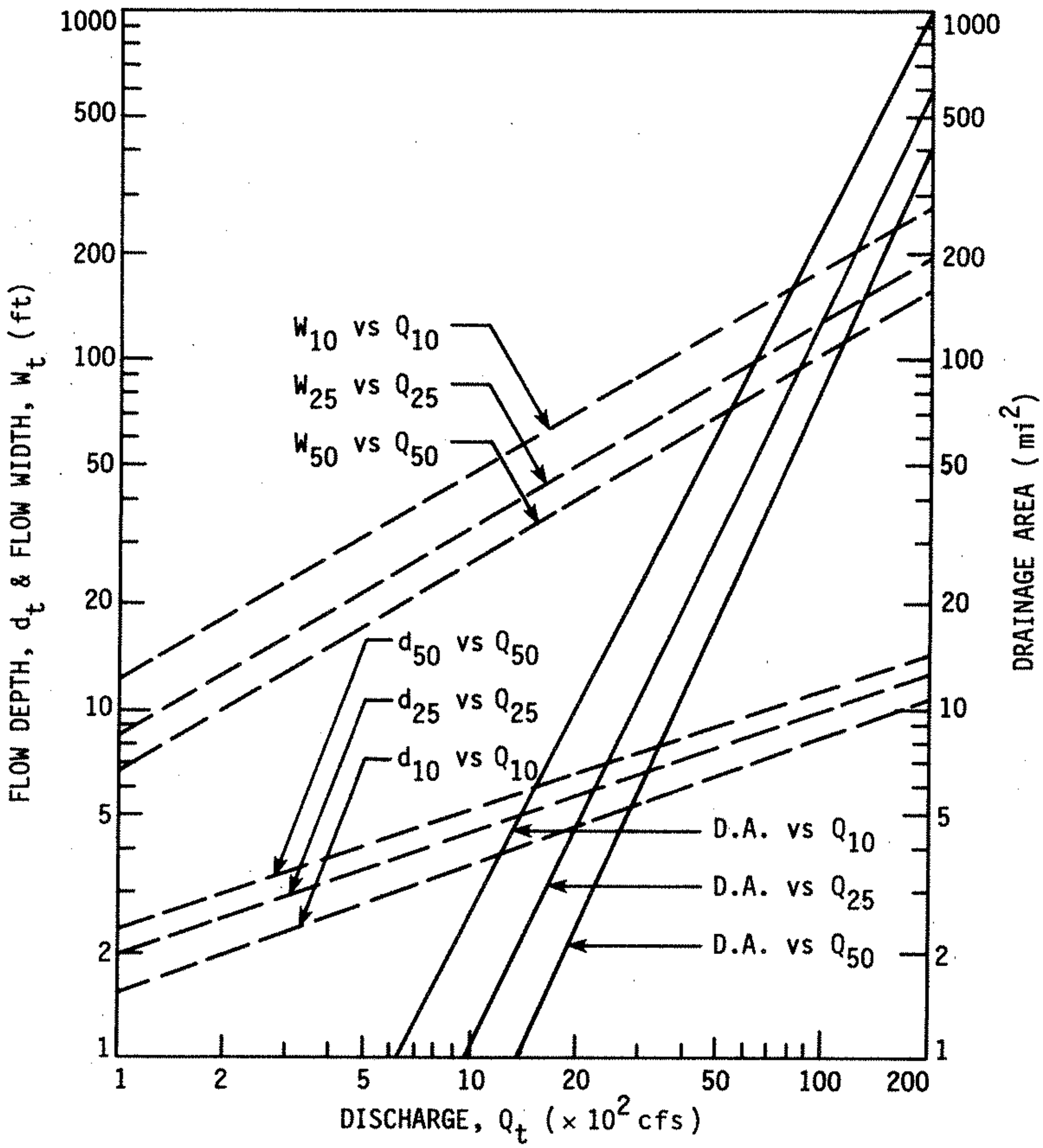


Fig. B.1. Geomorphological relationships, Region I.

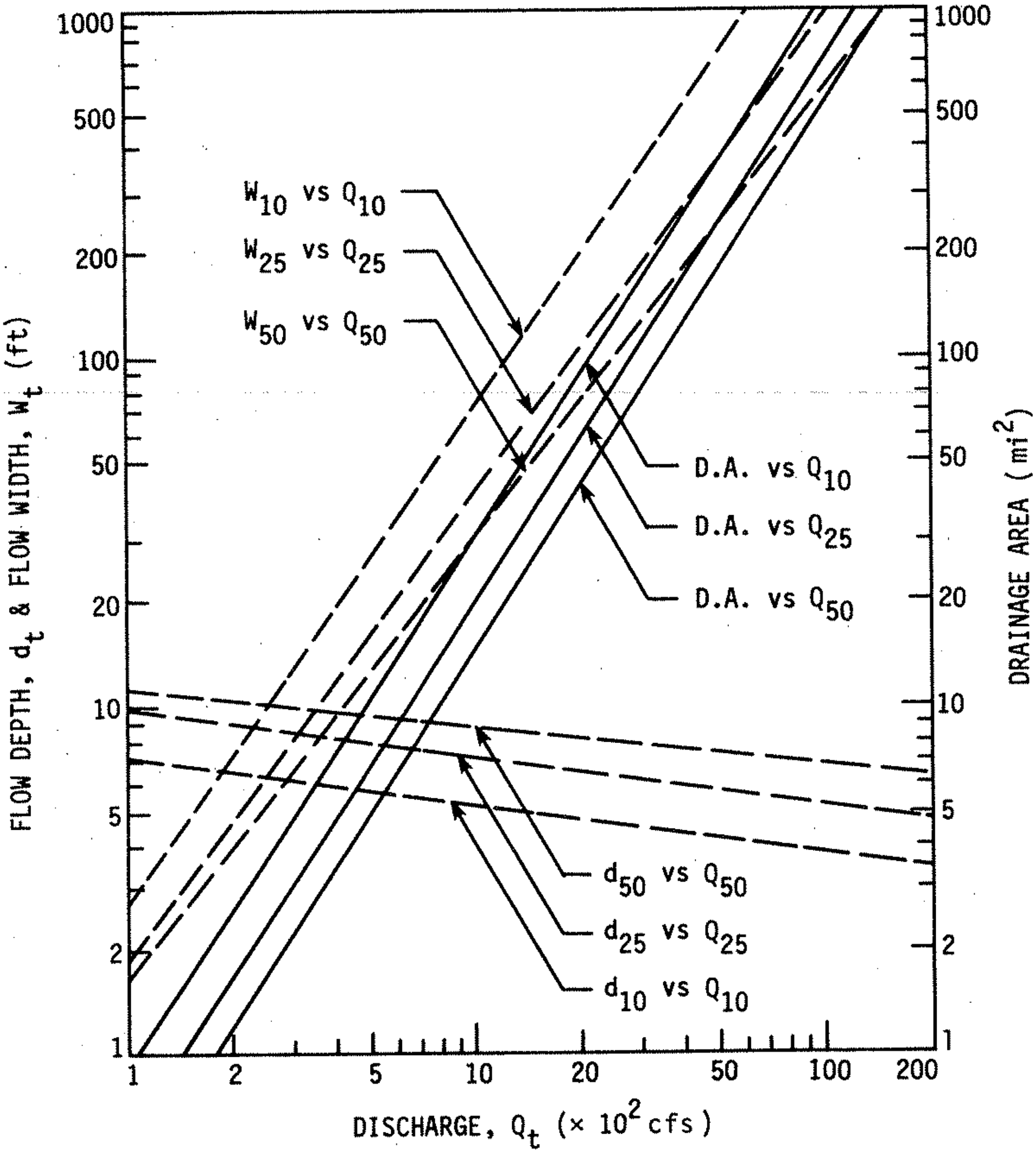


Fig. B.2. Geomorphological relationships, Region II.

equations relating discharge to drainage area for Region II and discharge to drainage area and channel slope for Region I, as the basis for the discharge versus drainage area curves shown in Figures B.1 and B.2.

Lara's equation for Region I is:

$$Q_t = c_t \, (D_a)^{x_t} \, (S)^{y_t} \qquad \text{(B.5)}$$

where

t = return period in years

S = channel slope in ft/mile between 10% and 85% points

D_a = drainage area in square miles

c, x, and y are tabulated coefficients depending on the value of t.

Lara's equation for Region II is:

$$Q_t = c_t \, (D_a)^{x_t} \qquad \text{(B.6)}$$

Table B.1 shows the results of least squares regression analyses carried out on Lara's regional equations. For Region II this correlation coefficient is 1.0 as Lara's Region II equation was obtained by a regression analysis of flow and drainage area. The correlation coefficient for the Region I Q_t versus D_a relatonship is less than one because in Lara's original regression equation, discharge was a function of both drainage area and bed slope. In this analysis, bed slope values for the gaging stations concerned were obtained from Lara (1976) and were substituted in Lara's original equation for Region I to obtain the relationship of Q_t as a function of drainage area alone.

Step 6 Determination of Bed Slope, S, Versus Drainage Area, D_a, Relationship

Figures B.3 and B.4 show the relationship between bed slope and drainage area for Regions I and II, respectively. The data for these figures were obtained from Lara (1973). Regression analyses were carried out on the data and the results are shown in Table B.1.

Step 7 Determination of Q_t, d_t and W_t for Given Drainage Areas, D_a

Figure B.5 is an example of how, for a given value of drainage area, values of flow depth, d_t, flow width, W_t, and a t-year return period flood, Q_t, were obtained from Figures B.1 and B.2.

Tables B.2 for Region I and B.3 for Region II show the results of this procedure for various values of drainage area.

Tables B.2 and B.3 also show the values of bed slope, S, corresponding to the various drainage areas. These values of S were obtained from Figure B.3 for Region I and Figure B.4 for Region II.

Step 8 Calculation of Tractive Force, τ_t, and Velocity, V_t, Corresponding to the t-year Return Period Flood

This step uses the values of Q_t, d_t, W_t, and S obtained under step seven. The tractive force corresponding to each return period was obtained using Eq. (B.7).

$$\tau_t = 62.4\ d_t\ S \qquad \text{(B.7)}$$

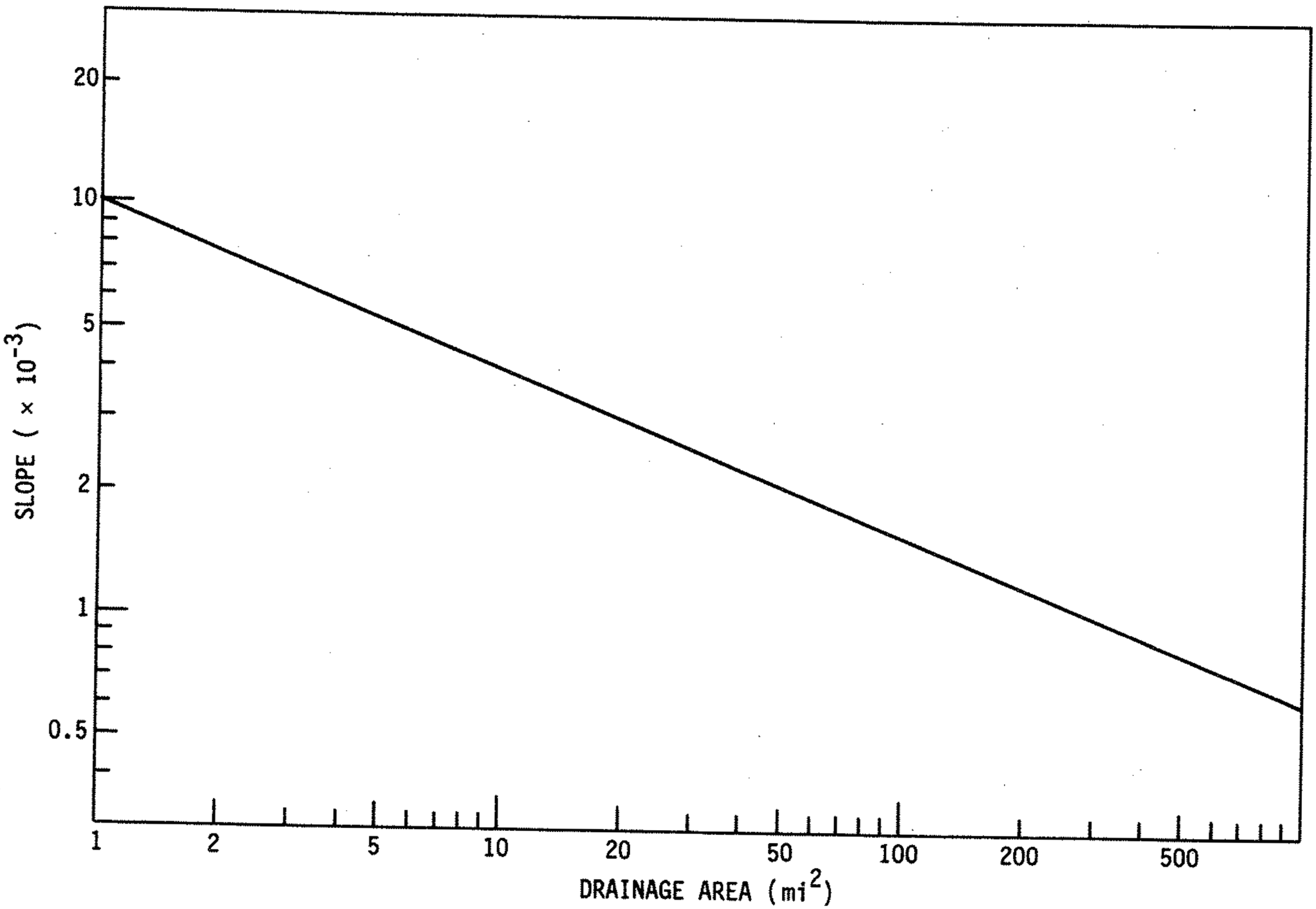


Fig. B.3. Slope vs drainage area, Region I.

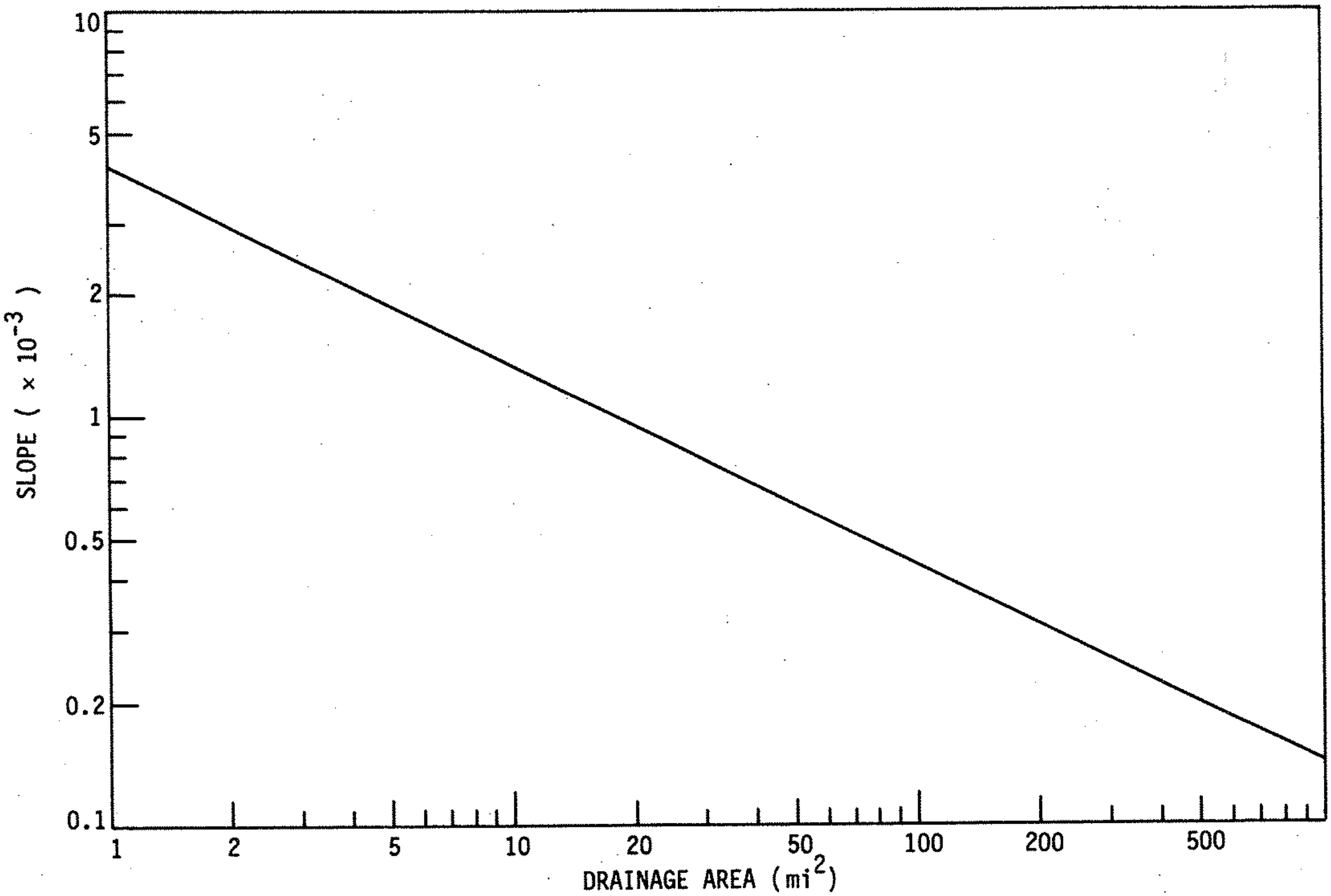


Fig. B.4. Slope vs drainage area, Region II.

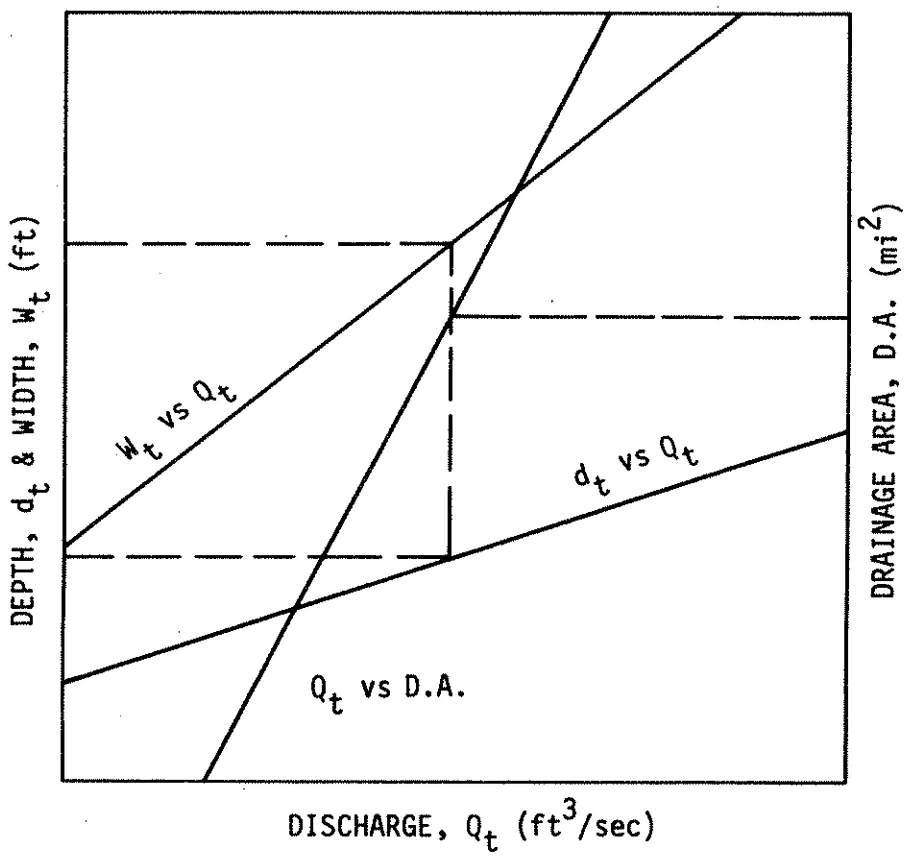


Fig. B.5. Showing how d_t, W_t and Q_t values corresponding to a given drainage area D_a were obtained.

Table B.2. Values of d_t, W_t, S, τ_t, Q_t, and V_t for various values of drainage area, D_a, and return period, t, for Region I.

Flood Return Period, t, Years (1)	Drainage Area D_a mi^2 (2)	Flow Depth d_t ft (3)	Flow Width W_t ft (4)	Slope S ft/ft (5)	Tractive Force τ_t lb/ft^2 (6)	Flood Discharge Q_t ft^3/s (7)	Mean Velocity V_t ft/s (8)
	1	3.0	36.0	0.0100	1.841	625	5.89
	5	4.0	58.0	0.00530	1.323	1400	6.03
10	15	4.9	80.0	0.00344	1.052	2450	6.25
	40	5.9	107.0	0.00234	0.861	4000	6.34
	100	6.9	140.0	0.00160	0.680	6400	6.63
	1	4.4	33.0	0.0100	2.746	975	6.71
	5	5.7	51.0	0.00530	1.885	2100	7.22
25	15	6.8	70.0	0.00344	1.460	3530	7.42
	40	8.2	93.0	0.00234	1.197	5700	7.47
	100	9.4	118.0	0.00160	0.938	8600	7.75
	1	5.7	31.0	0.0100	3.557	1350	7.640
	5	7.3	48.0	0.00530	2.414	2770	7.905
50	15	8.6	65.0	0.00344	1.846	4550	8.140
	40	10.0	85.0	0.00234	1.460	7000	8.235
	100	11.5	108.0	0.00160	1.148	10600	8.535

Table B.3. Values of d_t, W_t, S, τ_t, Q_t, and V_t for various values of drainage area, D_a, and return period, t, for Region II.

Flood Return Period, t, Years (1)	Drainage Area D_a mi^2 (2)	Flow Depth d_t ft (3)	Flow Width W_t ft (4)	Slope S ft/ft (5)	Tractive Force τ_t lb/ft^2 (6)	Flood Discharge Q_t ft^3/s (7)	Mean Velocity V_t ft/s (8)
	1	7.3	2.9	0.00441	2.009	105	4.96
	5	6.3	13.5	0.00188	0.739	305	3.59
10	15	5.6	38.0	0.00112	0.391	640	3.01
	40	5.2	97.0	0.00068	0.221	1230	2.44
	100	4.7	230.0	0.00044	0.129	2250	2.08
	1	9.3	3.1	0.00441	2.559	145	5.03
	5	8.0	12.5	0.00188	0.938	415	4.15
25	15	7.3	33.0	0.00112	0.510	850	3.53
	40	6.6	77.0	0.00068	0.280	1620	3.19
	100	6.2	173.0	0.00044	0.170	2950	2.75
	1	10.7	3.4	0.00441	2.944	177	4.865
	5	9.5	12.6	0.00188	1.114	560	4.177
50	15	8.8	30.9	0.00112	0.615	1020	3.751
	40	8.2	68.7	0.00068	0.348	1925	3.417
	100	7.7	144.0	0.00044	0.211	3480	3.139

where

62.4 = specific weight of water in lb/ft^3

Equation (B.7) was evaluated for a number of different d_t and S values corresponding to different drainage areas. The results are shown in Table B.2 for Region I and in Table B.3 for Region II.

The calculation of the velocity corresponding to each return period was carried out using Eq. (B.8) for different Q_t, d_t, and W_t corresponding to different drainage areas.

$$V_t = \frac{Q_t}{A_t} = \frac{Q_t}{d_t \cdot W_t} \qquad (B.8)$$

The results are shown in Table B.2 for Region I and Table B.3 for Region II. Figure 5.2 for Region I and Figure 5.4 for Region II show the relationship between τ_t and drainage area. Figure 5.3 and Figure 5.5 show the relationship between V_t and drainage area for Regions I and II, respectively.

Step 9 Construction of Table 5.1

The construction of Table 5.1, which was the last step in the development of the selection method given in section 5.1, is described in section 5.4.3.1.

APPENDIX C

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