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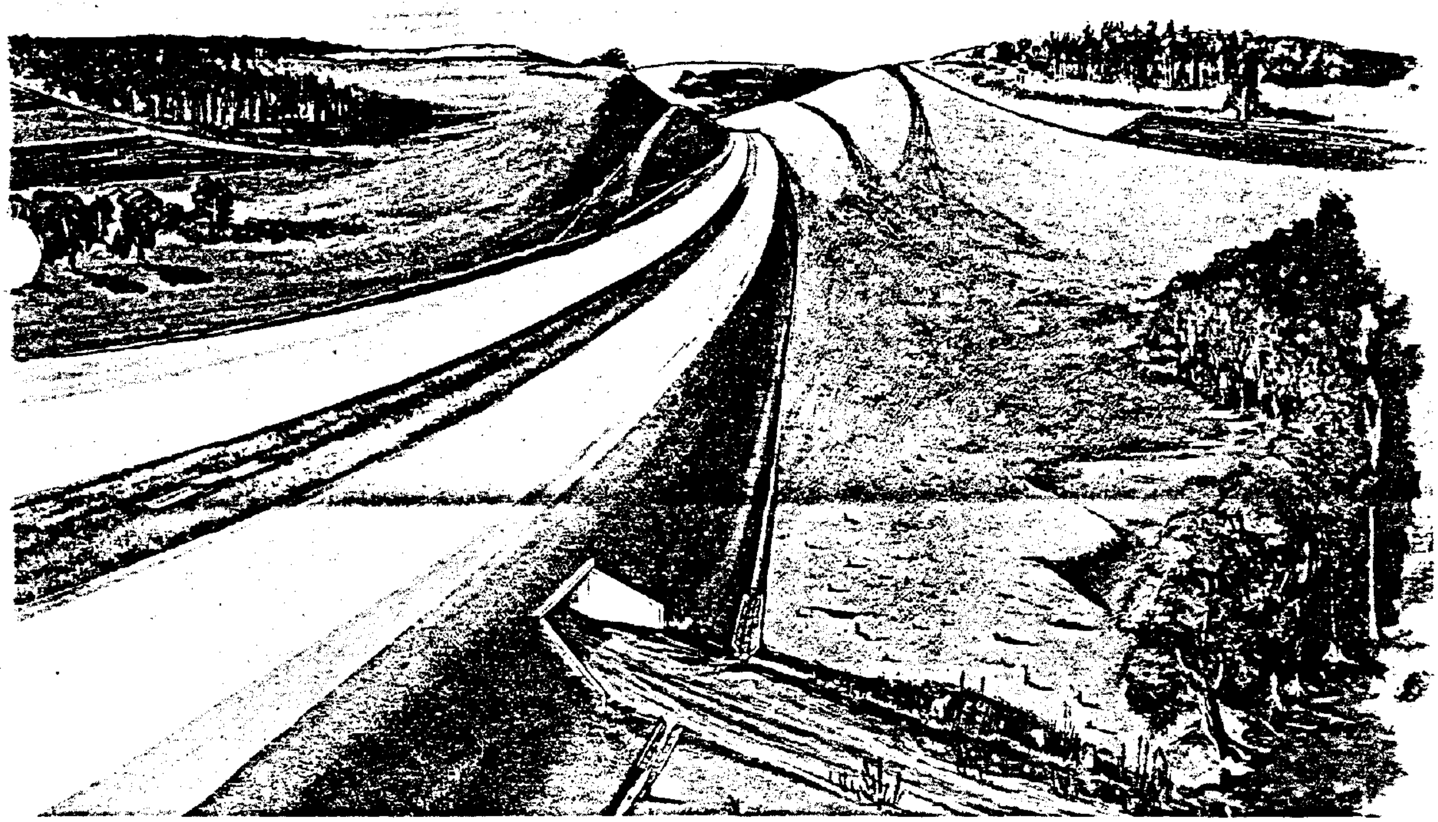
HYDRAULIC DESIGN SERIES NO. 4

DESIGN OF

ROADSIDE DRAINAGE CHANNELS



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16. Abstract

This is the fourth in a series of publications by the Federal Highway Administration on the Design of Hydraulic Drainage Structures. It addresses specific methods for the design of open channels.

This report discusses the characteristics of various types of linings and cross sections used for drainage channels; and the effects of channel alinement and grade on channel performance. It also briefly discusses the principals of hydrology and hydraulics.

A number of example problems, illustrating the use of the design procedures are included.

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Other Publications of the Hydraulic Design Series

Hydraulics of Bridge Waterways

Hydraulics of Bridge Waterways. Hydraulic Design Series No. 1, Second Edition, published in 1970 and reprinted in 1974, presents simplified methods for computing backwater caused by bridges. These methods were developed from extensive model tests and actual measurements of flow on streams with wide flood plains. The empirical curves and methods of calculation contained in the new publication have a much wider range of application than those of the first edition, published in 1960, which were based principally on hydraulic model studies. Additional field data collected during floods were available for the second edition. A considerable amount of new material has been added, including chapters on partially inundated superstructures, the proportioning of spur dikes at bridge abutments, and supercritical flow under a bridge, together with examples.

The nature of this publication is indicated by the chapter titles: computation of backwater; difference in water level across approach embankments; configuration of backwater; dual bridge; abnormal stage-discharge conditions; effects of scour on backwater; superstructures partially inundated; spur dikes; flow passes through critical depth; preliminary field and design procedures; illustrative examples; and discussion of procedures and limitations of method.

Hydraulics of Bridge Waterways is available from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402.

Peak Rates of Runoff from Small Watersheds

Peak Rates of Runoff from Small Watersheds, published in 1961 as No. 2 in the Federal Highway Administration's Hydraulic Design Series, is presently out of print. This publication will be reissued pending completion of current (1973) research to update and expand the data base and to improve the method of estimating peak discharges.

Design Charts for Open Channel Flow

Design Charts for Open Channel Flow, Hydraulic Design Series No. 3, published in 1961 and reprinted in 1973, makes generally available a group of hydraulic charts which facilitate the computation of uniform flow in open channels. Some of the charts are also useful in the design of storm drains.

The text is not intended to be a treatise on the design of open channels, although a brief discussion of the principles of flow in open channels is included. It is intended rather, as a working tool that should be of considerable service to the designer already familiar with the subject.

This publication contains charts which provide direct solution of the Manning equation for uniform flow in open prismatic channels of various cross sections; instructions for using the charts: a table of recommended values of n for use in the Manning equation; tables of permissible velocities in earth and vegetated channels; instructions for constructing charts similar to those presented; and a nomograph for use in the solution of the Manning equation. Charts are included for rectangular, trapezoidal, and triangular channels, grass-lined channels, circular pipe channels (part-full flow), pipe-arch channels, and oval concrete pipe channels.

Design Charts for Open-Channel Flow is available from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C., 20402.

DESIGN OF ROADSIDE DRAINAGE CHANNELS

Hydraulic Design Series No. 4

By the Hydraulics Branch, Bridge Division,
Office of Engineering

Reported by James K. Searcy, Hydraulic Engineer

U.S. DEPARTMENT OF TRANSPORTATION

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FIGURES

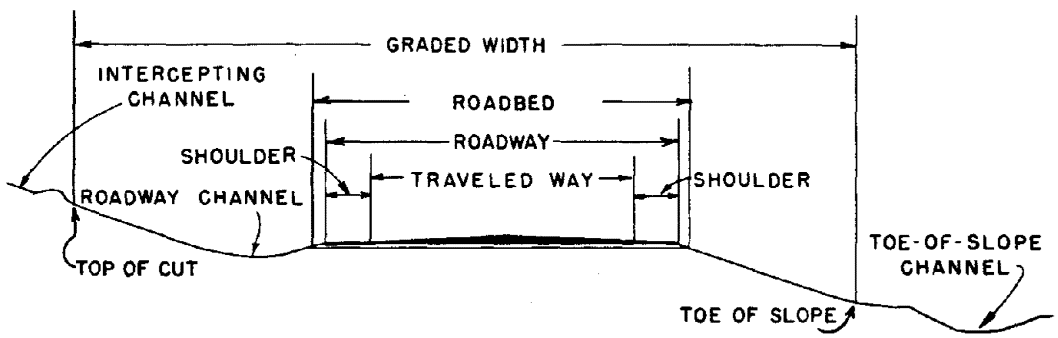


Figure 1.—Elements of the highway cross section.

LETTER SYMBOLS AND UNITS

Symbol	Units	Description
A	acres	Drainage area of a stream at a specified location
A	sq. ft.	Area of cross section of flow
B	ft.	Width of rectangular channel or conduit
b	ft.	Bottom width of a trapezoidal channel
C		Runoff coefficient in the rational formula
D	ft.	Height of conduit or diameter of circular conduit
d	ft.	Depth of flow at any section
d_c	ft.	Critical depth of flow in channel
d_m	ft.	Mean depth of flow $\left(\frac{A}{T}\right)$
d_n	ft.	Normal depth of flow in uniform channel for steady flow
F		Froude number $=\frac{V}{\sqrt{gd_m}}$
g	ft./sec.^{2}	Acceleration of gravity $=32.2$
H	ft.	Total head
H_c	ft.	Specific head at minimum energy $=d_c+V_c^2/2g$
H_o	ft.	Specific head $=d+V^2/2g$
h	ft.	Vertical drop in ditch check
h_L	ft.	Cumulative losses in a channel reach
i	in./hr.	Average rainfall intensity during the time of concentration
K		Channel conveyance $=\frac{1.49}{n}AR^{2/3}$
k	ft.	Stone diameter for bank protection (165 lb./ft.^{3} stone)
k_e		Entrance loss coefficient
k_w	ft.	Stone diameter for bank protection for stone weighing other than 165 lb./ft.^{3}
L	ft.	Length of channel reach
n		Manning roughness coefficient
Q	c.f.s.	Rate of discharge
R	ft.	Hydraulic radius $=\frac{A}{WP}$
S	ft./ft.	Slope of the energy grade line (total head line). When the Manning equation applies, $S=S_0$
S_a	ft./ft.	Slope that produces the allowable velocity for a given discharge
S_c	ft./ft.	That particular slope of a uniform channel at which normal depth equals critical depth for a given discharge
S_f	ft./ft.	Minimum slope required to overcome friction
S_0	ft./ft.	Slope of the flow line of a channel (bed slope)
T	ft.	Top width of water surface in a channel
T_c	min.	Time of concentration of a watershed
V	f.p.s.	Mean velocity of flow
V_c	f.p.s.	Mean velocity of flow in a channel when flow is at critical depth (critical velocity)
V_n	f.p.s.	Mean velocity of flow in a channel when flow is at normal depth
V_s	f.p.s.	Mean velocity of flow against stone in a rocklined channel
w	lb./ft.^{3}	Unit weight of stone used for channel linings
WP	ft.	Wetted perimeter—length of line of contact between the flowing water and the channel
Z	ft.	Elevation of bed of channel above a datum
z		Slope of sides of a channel (ratio of horizontal to vertical)

PRINCIPAL EQUATIONS

Rational formula (p. 7):

$$Q=CiA \tag{1}$$

Manning equation (p. 12):

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

$$Q=\frac{1.49}{n}AR^{2/3}S^{1/2} \tag{5}$$

$$S=\left(\frac{Vn}{1.49R^{2/3}}\right)^2 \tag{13}$$

Critical depth (p. 17):

Rectangular section:

$$d_c=0.315\sqrt[3]{\left(\frac{Q}{B}\right)^2} \tag{15}$$

Trapezoidal channel:

$$d_c=\frac{4zH_0-3b+\sqrt{16z^2H_0^2+16zH_0b+9b^2}}{10z} \tag{16}$$

Triangular channel:

$$d_c=0.574\sqrt[5]{\left(\frac{Q}{z}\right)^2} \tag{17}$$

Froude number (p. 20):

$$F=\frac{V}{\sqrt{gd_m}} \tag{26}$$

Chapter I.—INTRODUCTION

1.1 General. Roadside drainage channels perform the vital function of diverting or removing surface water from the highway right-of-way. They should provide the most efficient disposal system consistent with cost, importance of the road, economy of maintenance, and legal requirements. One standard channel will rarely provide the most satisfactory drainage for all sections of a highway, although it might be adequate for most locations. Thus, the design engineer needs procedures for designing various types of channels. This publication discusses flow in roadside drainage channels, estimation of peak discharges from small areas, prevention of channel erosion, and presents methods for the design of drainage channels required to remove runoff from the area immediately adjacent to the highway.

Drainage channel design requires, first, the determination of where surface or ground water will occur, in what quantity, and with what frequency. This is a hydrologic problem. Then structures of appropriate capacity must be designed to divert water from the highway roadway, to remove water that reaches the roadway, and to pass collected water under the roadway. This is a hydraulic problem. This publication will discuss only the part of the problem that deals with roadside drainage channels; however, many of the principles discussed also apply to natural streams.

Erosion control (*1*)[1] is a necessary part of good drainage design. Unless the side slopes and the drainage channels themselves are protected from erosion, unsightly gullies appear, maintenance costs increase, and sections of the highway may be damaged or even destroyed.

Drainage design begins with the road location. Locations that avoid poorly drained areas, unstable soil, frequently flooded areas, and unnecessary stream crossings greatly reduce the drainage problem. Data gathered in the field permit solution of the drainage problems that cannot be avoided when locating the highway. Adequate notes on farm drainage, terraces, and manmade channels are particularly important, as these channels are seldom shown on topographic maps.

The design of roadside drainage channels has been discussed by Izzard (*2*), and criteria for drainage channel design appears in the publication, "A Policy on Geometric Design of Rural Highways" (*1*). Subsurface drainage of highways is discussed in Highway Research Board Bulletin 209 (*3*) and the function of highway shoulders in surface drainage is discussed by Ackroyd (*4*).

In this publication, types and sections of drainage channels are first discussed, followed by a discussion of channel alinement, grade, and protection from erosion. Hydrologic and hydraulic principles are discussed briefly and design procedures are presented with typical charts for determining channel size. Finally, the methods described are applied in the design of the channels illustrated in figure 2. For convenience of the user, all tables are placed in appendix A.

The design procedures discussed in this publication are only tools to aid in solving the surface drainage problem—there is no standard solution. The drainage problem of each section of highway is individual and for solution requires adequate field data and an engineer experienced in highway drainage.

1.2 Factors in design. The primary purpose of roadside drainage channels is to prevent surface runoff from reaching the roadway and to efficiently remove the rainfall or surface water that reaches the roadway. To achieve this purpose, the drainage channels should have adequate capacity for the peak rates of runoff that recur with a frequency depending upon the class of road and the risk involved. On less important roads where little damage would result from overtopping of the drainage channel and where traffic would suffer only minor inconvenience, a peak runoff that recurs frequently might be satisfactory. On major highways or on minor highways where serious erosion damage would result from overtopping the drainage channels, a less frequent peak runoff might be used as the design runoff.

The frequency used depends somewhat upon the climate, the topography of the area, and the cost of the drainage system. Water standing on the edge of the traveled way for a few minutes during an intense storm might not interfere with traffic movement any more than the storm itself, but should water remain for a long time or collect in pavement sags or in underpass depressions, a traffic hazard will result. At locations where shoulder gutters protect high fills highly susceptible to erosion, the peak discharge for which the channels are designed might be greater than the class of the road would otherwise warrant.

Bureau of Public Roads policy requires that on Interstate System projects, all drainage facilities other than culverts and bridges be designed to keep the traveled ways usable during storms at least as great as that for a 10-year frequency, except that a 50-year frequency shall be used for underpass or other depressed roadways where ponded water can be removed only through the storm-drain system.

[1] Italic numbers in parentheses refer to the references on p. 51.

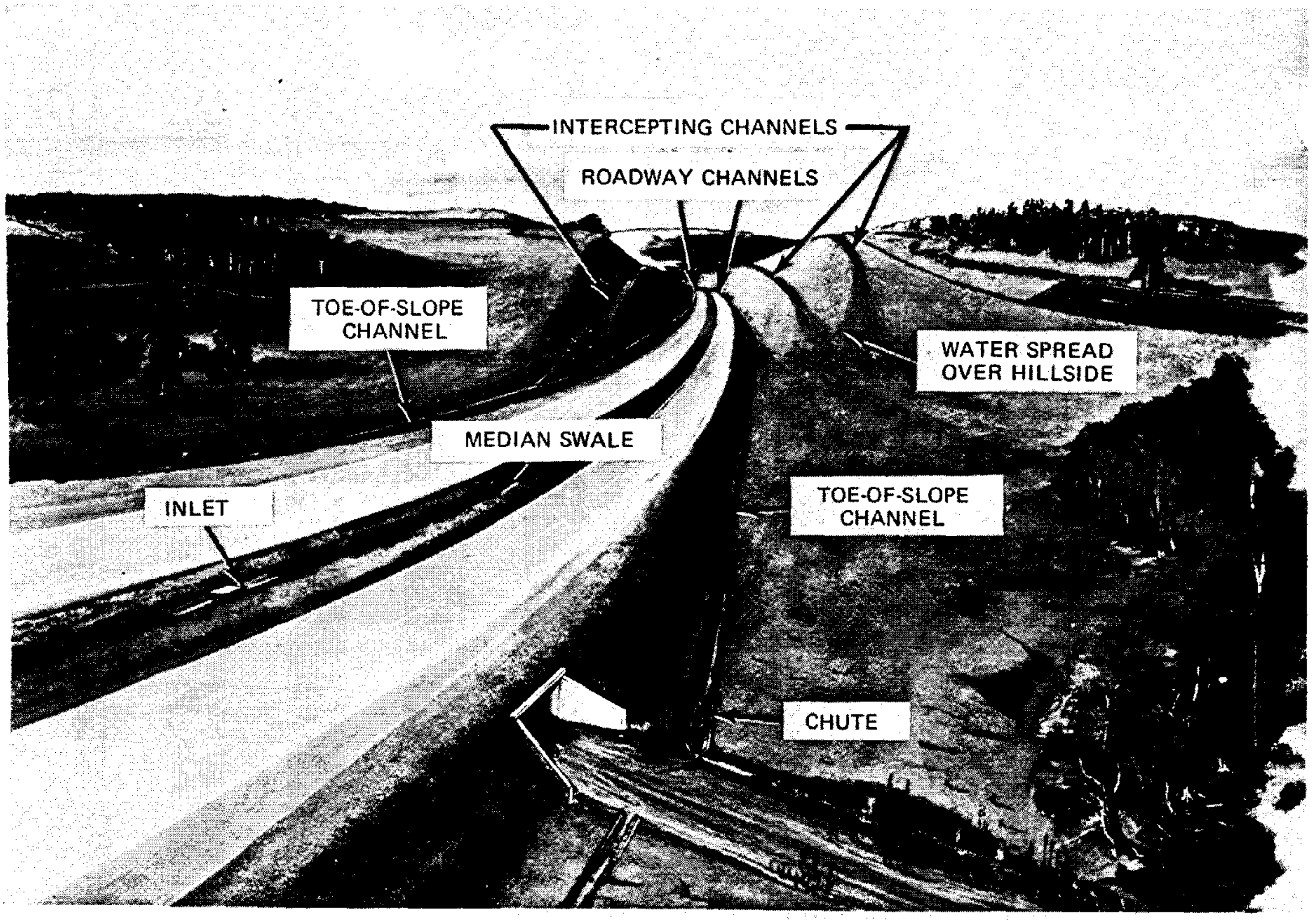


Figure 2.—Types of roadside drainage channels.

The capacity of a drainage channel carrying uniform flow depends upon its shape, size, slope, and roughness. For a given channel, the capacity becomes greater when the grade or the depth of flow is increased. The channel capacity decreases as the channel surface becomes rougher. For example, a rubble or stone gutter has only about half the capacity of a concrete gutter of the same size, shape, and slope because of the differences in channel roughness. A rough channel is sometimes an advantage on steep slopes where it is desirable to keep velocities from becoming too high.

The most efficient shape of channel is that of a semicircle, but hydraulic efficiency is not the sole criterion. In addition to performing its hydraulic function, the drainage channel should be economical to construct, and require little maintenance during the life of the roadway. Channels should also be safe for vehicles accidentally leaving the traveled way, pleasing in appearance, and dispose of collected water without damage to the abutting property. Most of these additional requirements for drainage channels reduce the hydraulic capacity of the channel. Thus, the best design for a particular section of highway is a compromise among the various requirements, sometimes with each requirement having a different influence on the design from that of another section.

1.3 Types of drainage channels. Highway drainage channels may be classified according to function as: gutters, chutes, roadway channels, toe-of-slope channels, intercepting channels, median swales, and channel changes. Figure 2 shows a typical divided highway in a humid region where several types of drainage channels are needed to drain the highway. Starting at the outer edge of the right-of-way are the intercepting channels on the natural ground outside the cut or fill slope, or on benches breaking the cut slope. Next are the roadway channels between the cut slope and the shoulder of the road and the toe-of-slope channels which take the water discharged from the roadway channels and convey it along or near the edge of the roadway embankment to a point of disposal. A shallow depression or swale drains the median. The types of drainage channels are defined and discussed in subsequent sections of this chapter.

1.4 Gutters. Gutters are the channels at the edges of the pavement or the shoulder formed by a curb or by a shallow depression. Gutters are invariably paved with concrete, brick, stone blocks, or some other structural material.

Gutters are generally used in lieu of other type channels for urban highway drainage and are sometimes used in rural areas, particularly on parkways, in mountainous regions, in sections with limited right-of-way, in areas of poor soil stability, and for special drainage problems such as traffic interchanges and underpasses.

In areas where vegetative cover cannot be used to prevent erosion damage to high fills, shoulders are designed to serve as a gutter with a curb constructed at the outer edge to confine the water to the shoulder. The water collected in the gutter is discharged down the slope through chutes. The curb may be made of earth, bituminous material, portland cement concrete, or cut stone.

1.5 Chutes. Chutes, as used in this publication, are steeply inclined open or closed channels, which convey the collected water to a lower level. Chutes are also called flumes and spillways. The most common applications in highway construction are the chutes used to convey water down cut or fill slopes. Open chutes can be metal or be paved with portland cement concrete, bituminous material, stone, or sod, depending upon the volume and velocity of the water to be removed. On long slopes, closed (pipe) chutes are generally preferable to open chutes because in an open chute the high velocity water is likely to jump out of the channel, erode the slope, and destroy the chute. Also, open chutes may interfere with machine-mowing operations on the roadway slopes. The inlet of all chutes must be adequately designed to prevent water bypassing the chute and eroding the slope. Frequently energy dissipators or other types of erosion protection are needed at the chute outlet.

1.6 Roadway channels. Roadway channels are the channels provided in the cut section to remove the runoff from rain falling on the roadway and on the cut slopes. These channels are sometimes called gutters when paved; however, in this publication paved channels separated from the traveled way will be called roadway channels.

A well-designed roadway channel removes storm water from the cut areas with the lowest overall cost, including cost of maintenance, and with the least hazard to traffic. The channel should also be pleasing in appearance. To meet these requirements, the AASHO (*1*) "Policy on Geometric Design of Rural Highways" recommends that where terrain permits, roadside drainage channels built-in earth should have side slopes not steeper than 4 horizontal to 1 vertical, and a rounded bottom at least 4 feet wide. Flatter side slopes are desirable on channels beside low fills. The depth of channel should be sufficient to remove the water without saturating the pavement subgrade.

It is unnecessary to standardize the design of roadside drainage channels for any length of highway. Not only can the depth and breadth of the channel be varied with variation in the amounts of runoff, rate of channel grade, and distance between lateral outfall culverts, but the dimensions can be varied by the use of different types of channel lining. Nor is it necessary to standardize the lateral distance between the channel and the edge of pavement. Often liberal offsets can be obtained where cuts are slight and where cuts end and fills begin.

Automobile proving ground tests reported by Stonex (*5*) show that the most important element in controlling the shock of impact when driving into a flat-sloped channel is the length of vertical curve between the side slopes and the channel bottom. (See fig. 3.) For a speed of 65 miles per hour and a striking angle of 15° with the channel centerline, on a channel with a 6:1 side slope, a vertical curve 6½ feet long is recommended and a vertical curve about 10 feet long is recommended for a channel with a 4:1 side slope.

1.7 Toe-of-slope channels. Toe-of-slope channels are located at or near the toe of a fill when it is necessary to convey water collected by the roadway channel to the point of disposal. On the downhill side of the highway, this channel can often be laid on a mild slope and the lower end flared to spread the water over the hillside. Where this practice would cause erosion or permit water to drain into the highway embankment, the toe-of-slope channel must convey the storm water to a natural watercourse.

In arid and semiarid regions, the water draining out of the roadway cut should be diverted away from the fill far enough so that it does not come back to the highway. The landowner will seldom object to receiving storm water from the highway, provided it is delivered without causing erosion.

1.8 Intercepting channels. Intercepting channels are located on the natural ground near the top edge of a cut slope or along the edge of the right-of-way, to intercept the runoff from a hillside before it reaches the roadway. Intercepting the surface flow reduces erosion of cut slopes, lessens silt deposition and infiltration in the roadbed area, and decreases the likelihood of flooding the highway in severe storms.

Intercepting surface water is particularly important in arid and semiarid regions. Intercepting dikes may be built well back from the top of the cut slope and generally on a flat grade until the water can be spread or emptied into a natural watercourse. In most cases, the owners of rangeland will permit highway departments to construct a series of contour furrows beyond the right-of-way in order to recover the water.

An intercepting channel constructed by forming a dike with borrow material is superior to an excavated channel because the latter destroys the natural ground cover and is more likely to erode. Care should be taken to avoid ponding water at the tops of slopes subject to sliding. In slide areas, storm water should be intercepted and removed as rapidly as practicable and sections of the channel crossing highly permeable soil might require lining with impermeable material.

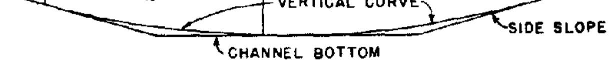


Figure 3.—Location of vertical curves in channel cross section.

1.9 Median swales. Median swales are the shallow depressed areas at or near the center of medians used to drain the median area and portions of the roadway. The depressed area or swale is sloped longitudinally for drainage, and at intervals the water is intercepted by inlets and discharged from the roadway. It is not necessary that the longitudinal slope of the swale conform to the pavement grade, particularly on flat grades. (See sec. 4.3.)

Generally, curbs are not provided on the edge of the pavement and the median swale drains part or all of the pavement area in addition to the median area. Even where curbs are provided, it is preferable to slope medians wider than 15 feet to a swale. This keeps water in the median off the pavement and prevents snowmelt water from running onto the pavement and becoming a hazard after a quick freeze. Medians less than 15 feet wide are generally crowned for drainage, and if under 6 feet in width the median is usually paved. Six feet is about the minimum width that can be mowed with mechanical equipment (see *1*).

1.10 Channel changes. Channel changes alter the alinement or cross sections of natural watercourses. Replacing a long sinuous natural channel by a shorter improved channel will increase the channel slope and usually decrease the channel roughness. Both of these changes cause an increase in the velocity of the flowing water, sometimes enough to cause damage to the highway embankment near the stream or excessive scour around footings of structures. At other times damage occurs because the stream continues to use the old channel rather than the new because adequate training works are not provided to divert the stream into the new channel. In addition to possible damage to the highway, a major channel change may be detrimental to fish and other aquatic life because of increased velocities, decreased depth of flow, and the removal of boulders and irregularities in the channel. Placing boulders at random in the new channel will aid in restoring the fish habitat.

Occasionally, small watercourses crossing the highway can be diverted into an adjacent watercourse at a lower cost than by providing separate cross drains. Channel changes in the vicinity of a highway or at a highway crossing are sometimes made to secure borrow material for the roadway embankment with the thought of incidental improvement in the channel cross section or alinement. Such a practice on a stabilized stream channel almost always results in future maintenance problems. Channel changes also incur a legal liability for damage to private property that might be brought about by the changed channel. Thus, channel changes should always be studied for their value and effect rather than made to secure borrow material or to save the cost of a cross drain.

Borrow ditches should not be placed near the toe of embankments adjacent to natural streams which overflow their banks. A ditch so located will often carry flow at high velocities with attendant damage to the embankment section during flood periods. In some locations borrow ditches on the upstream side of the fill will direct the high-velocity water back into the main channel in a manner that induces scour at bridge piers and abutments.

1.11 Alinement and grade. The width of the right-of-way usually allows little choice in the alinement or in the grade of the channel, but insofar as practicable abrupt changes in alinement or in grade should be avoided. A sharp change in alinement presents a point of attack for the flowing water, and abrupt changes in grade cause deposition of transported material when the grade is flattened or scour when grade is steepened.

A drainage channel should have a grade that produces velocities that neither erode nor cause deposition in the channel. This optimum velocity also depends upon the size and shape of channel, the quantity of water flowing, the material used to line the channel, and upon the nature of the soil and the type of sediment being transported by the stream.

Ordinarily the highway drainage channel must be located where it will best serve its intended purpose, using the grade and alinement obtainable at the location. The capacity of the channel and the degree of protection given the channel are then determined as explained in chapter 4.

The point of discharge of a drainage channel into the natural watercourse requires particular attention. The alinement of the drainage channel should not cause eddies with attendant scour in the natural watercourse or near drainage structures. In erodible soils, if the flow line of the drainage channel is appreciably higher than that of the watercourse at the point of entry, a spillway or chute should be provided to discharge the water into the watercourse in order to prevent erosion in the drainage channel. The chute should be designed to prevent being undermined and destroyed.

1.12 Protection from erosion. The need for erosion prevention is not limited to the highway drainage channels; it extends throughout the right-of-way and is an essential feature of adequate drainage design. Erosion and maintenance are minimized largely by the use of: flat side slopes, rounded and blended with natural terrain; drainage channels designed with due regard to location, width, depth, slopes, alinement, and protective treatment; proper facilities for ground water interception; dikes, berms, and other protective devices; and protective ground covers and planting (*1*).

The discussion in this publication is limited to providing erosion control in drainage channels by proper design, including the selection of an economical channel lining. Lining as applied to drainage channels includes vegetative coverings. The type of lining should be consistent with the degree of protection required, overall cost, safety requirements, and esthetic considerations. Control of erosion caused by overland or sheet flow is not discussed.

In general, when a lining is needed, the lowest cost lining that affords satisfactory protection should be used. This is often sod in humid regions, used alone or in combination with other types of linings. Thus, a channel might be grass lined on the flatter slopes and lined with more resistant material on the steeper slopes. In cross section, the channel might be lined with a highly resistant material within the depth required to carry floods occurring frequently and lined with grass above that depth for protection from the rare floods.

Ditch checks were once used extensively to prevent erosion in roadway channels. In recent years they have become unpopular in humid areas as grade-control structures in channels because they are often a hazard to vehicles driving off the highway, they are difficult to maintain, they are often unsightly, and in most locations, their job can be done better and more cheaply with a vegetative lining. Figure 4 shows what happened to a channel protected by poorly designed ditch checks.

In semiarid and arid regions, some erosion of cut and fill slopes is inevitable. Occurring infrequently at any one location, the damage can be repaired without excessive average annual cost. The roadbed is generally protected by paving or treating the road shoulders. Channels in cut sections can be protected by rubble or by paving. Channels off the roadway can best be protected by laying them on grades which will not produce velocities in excess of those permissible (see table 3) for the soil of that locality and by spreading the collected water over the hillside as soon as it can be released without reaching the roadway. When steeper channel grades cannot be avoided, vertical drops or properly designed ditch checks can be used to maintain noneroding grades between drops. If the spacing of the drops is close, their cost should be compared with the cost of paving the entire channel and providing an adequate outlet structure. High drop structures require careful design (*6*, *7*, and *8*) and though hazardous to traffic and at times unsightly, they are to be preferred to a deep gully.

Systematic maintenance is essential to any drainage channel. Without proper maintenance, a well-designed channel becomes an unsightly gully. Maintenance methods should be considered in the design of drainage channels so that the channel sections will be suitable for the methods and equipment that will be used for their maintenance.

Figure 4.—Poorly designed ditch checks.

Chapter II.—ESTIMATING STORM RUNOFF FROM SMALL AREAS

2.1 General. The first step in designing a channel is to determine the quantity of water the channel is to carry. This is a far more difficult problem than computing the size of channel needed to carry the design flow and determining what protection is required to prevent channel erosion.

Many formulas have been developed (*9*) for estimating peak storm runoff, but most of the formulas were developed from data collected from a limited area and can be considered applicable only to the area and under the conditions for which the data were collected. Runoff from developed surfaces, drainage areas of comparatively uniform slope and surface characteristics, can be determined by the method described by Izzard (*10*) if there is sufficient regularity in surface to justify the work required.

Some of the storm-runoff formulas used in the past give the size of structure directly, but modern practice (*11*, p. 90) is to first calculate the anticipated discharge and then design the waterway or channel to accommodate the discharge. A formula which gives only a required waterway area ignores the great difference in capacity of structures which have the same waterway area but have different hydraulic characteristics.

In the design of storm sewers, more than 90 percent (*12*, p. 31) of the engineering offices in the United States use the rational method which has been in use since 1889. Application of this method to channel design is described in this chapter. The method is recommended for use in determining the design discharge for roadside drainage channels draining less than about 200 acres. For larger areas, the methods described by Potter (*13*), by Dalrymple (*14*), or by the Soil Conservation Service (*15*) are more applicable. There is no clearly defined line where one method should end and another method be used. The methods sometimes give results for the same area that agree quite well, and in other instances they may disagree by 50 or more percent. Discrepancies between methods approaching 50 percent may be expected because of the small sample (length and number of streamflow records) available for estimating magnitude and frequency of storm runoff by any method. However, when estimated runoff rates obtained by two methods differ by ratios of 2 to 1 or more, the relative accuracy of the methods becomes important, and must be judged by the quantity and quality of the data upon which the method is based.

The expected frequency of occurrence of the design discharge (sec. 1.2) is of concern because economy is always a factor in the design. Overdesign and underdesign both involve excessive costs on a longtime basis. A channel designed to carry a 1-year flood would have a low first cost, but the maintenance cost would be high because the channel would be damaged by storm runoff almost every year. On the other hand, a channel designed to carry the 100-year flood would be high in first cost but low in maintenance cost. Somewhere between these limits lies the design which will produce the lowest annual cost.

2.2 Storm runoff. Runoff comes from precipitation falling on the land and water surfaces of the watershed. A small part of the precipitation evaporates as it falls and some is intercepted by vegetation. Of the precipitation that reaches the ground, a part infiltrates the ground, a part fills the depressions in the ground surface, and the remainder flows over the surface (overland flow) to reach defined watercourses. The surface runoff is sometimes augmented by subsurface flow that flows just beneath the ground surface and reaches the watercourse in time to be a part of the storm runoff.

The precipitation infiltrating the ground replenishes the soil moisture and adds to the ground-water storage. Some of this underground water reaches the stream long after the storm runoff has passed and some is withdrawn by the life processes of vegetation or by man for his use.

The storm runoff which must be carried by the roadside drainage channels is thus the residual of the precipitation after losses (the extractions for interception, infiltration, and depression storage). The rate of water loss depends upon the amount of the precipitation and the rate at which it falls (intensity), upon temperature, and upon the characteristics of the land surface.

Not only does the rate of runoff vary with the permeability of the land surface and the vegetal cover, but it varies from time to time for the same surface depending upon the antecedent conditions. It would be impracticable, even if the data were available, to determine for each channel drainage area the frequency of recurrence of the numerous factors which affect the rainfall-runoff relation and thus compute the magnitude of the runoff for a given frequency rainfall.

If a long-period record of storm runoff existed for every site for which we wish to design a drainage channel, we could determine the frequency of various magnitudes of storm runoff. However, runoff records for areas of less than 200 acres are practically nonexistent and we are forced to estimate runoff frequency from frequency of rainfall by assuming the rainfall to have the same frequency as runoff when the design storm occurs. (See sec. 2.4–1.)

2.3 Rainfall intensity—duration—frequency analysis. The intensity of rainfall is the rate at which the rain falls. Intensity is usually stated in inches per hour regardless of the duration of the rainfall, although it may be stated as total rainfall in a particular period. Frequency can be expressed as the probability of a given intensity of rainfall being equaled or exceeded or it can be expressed in terms of the average interval (recurrence interval) between rainfall intensities of a given or greater amount. The frequency of rainfall intensity cannot be stated without specifying the duration of the rainfall because the rainfall intensity varies with the duration of rainfall. (See fig. 6.)

Frequency analysis of rainfall intensity is discussed by Chow (*16*) and others (*17*, *18*). Two methods are in general use for selecting the rainfall data used in frequency analyses. These methods are the annual series and the partial-duration series. The annual-series analysis considers only the maximum rainfall of each year (usually calendar year) and ignores the other rainfalls during the year. These lesser rainfalls during the year sometimes exceed the maximum rainfalls of other years. The partial-duration series analysis considers all of the high rainfalls regardless of the number occurring within a particular year. In designing drainage channels for return periods greater than 10 years, the difference between the two series is unimportant. When the return period (design frequency) is less than 10 years, the partial-duration series is believed to be more appropriate. To change the frequency curves based on the annual series to one based on the partial-duration series, multiply the annual series values by the following factors (*19*, p. 1):

2-year return period	1. 13
5-year return period	1. 04
10-year return period	1. 01
20-year or more	1. 00

The Weather Bureau has prepared a Rainfall-Frequency Atlas of the United States (*20*). This Atlas contains maps of rainfall frequency for 30-minute, 1-, 2-, 3,- 6-, 12-, and 24-hour durations in each of the return periods 1-, 2-, 5-, 10-, 25-, 50-, and 100-year. The rainfall lines on the maps in the Atlas are based on a partial-duration series analysis and represent total rainfall, in inches, for the stated duration.

Also of value to the designer of roadside drainage channels is Weather Bureau Technical Paper 25 (*19*) which contains rainfall intensity-duration-frequency curves for selected stations in the United States, Alaska, Hawaiian Islands, and Puerto Rico. The curves in this publication are based on the annual series and the preceding corrections should be used for return periods less than 10 years.

2.4 The rational method. Rainfall intensity is converted into rate of storm runoff by the rational formula:

$$Q = CiA \tag{1}$$

Where

Q = peak rate of runoff, in cubic feet per second.

C = weighted runoff coefficient (average of the coefficients assigned to the different types of contributing areas).

i = average rainfall intensity, in inches per hour, for the selected frequency and for duration equal to the time of concentration.

A = drainage area, in acres, tributary to the point under design.

The formula is not dimensionally correct; however, a 1-inch depth of rainfall applied at a uniform rate in 1 hour to an area of 1 acre will produce 1.008 cubic feet per second of runoff if there are no losses. This makes the numerical value of Q very nearly equal to the product of i and A. The coefficient C accounts for the losses (sec. 2.4-1).

The rational formula is based on the thesis that if a uniform rainfall of intensity i were falling on an impervious area of size A, the maximum rate of runoff at the outlet to the drainage area would be reached when all portions of the drainage area were contributing; the runoff rate would then become constant. The time required for runoff from the most remote point (point from which the time of flow is greatest) of the drainage area to arrive at the outlet is called the time of concentration.

Actual runoff is far more complicated than the rational formula indicates. Rainfall intensity is seldom the same over an area of appreciable size or for any substantial length of time during the same storm. If a uniform intensity of rainfall of duration equal to the time of concentration were to occur on all parts of the drainage area, the rate of runoff would vary in different parts of the area because of differences in the characteristics of the land surface and the nonuniformity of antecedent conditions. Under some conditions maximum rate of runoff occurs before all of the drainage area is contributing. (See sec. 2.4-6.) The temporary storage of storm water en route toward defined channels and within the channels themselves accounts for a considerable reduction in the peak rate of flow except on very small areas. The error in the runoff estimate increases as the size of the drainage area increases. For these reasons, the rational method should not be used to determine the rate of runoff from large drainage areas. For the design of highway drainage structures, the use of the rational method should be restricted to drainage areas less than 200 acres unless no other method is available to estimate the design discharge.

Many refinements have been suggested in the application of the rational method. A few of the discussions of the method are listed in the references (*12*, *21*, *22*, and *23*, app. A). The suggested refinements in the method probably improve the runoff estimate, but the collection of additional data required and the increased work involved do not appear warranted in the design of drainage channels.

2.4-1 Runoff coefficient. The runoff coefficient C in the rational formula is the ratio of the rate of runoff to the rate of rainfall at an average intensity i when all the drainage area is contributing. The coefficient C varies widely from storm to storm, but Horner and Flynt (*24*) state that when rainfall intensity and runoff (based on a 20-year record) were considered separately it was found that the ratio,

$$C = \frac{\text{Peak runoff rate of a given frequency}}{\text{Average rainfall intensity of the same frequency}}$$

remained reasonably constant for the various frequencies.

The range in values of C listed in table 1, appendix A, permit some allowance for land slope and differences in permeability for the same type cover. For flat slopes and permeable soil, use the lower values.

Where the drainage area is composed of several types of ground cover, the runoff coefficient is weighted (see example 1) according to the area of each type of cover present.

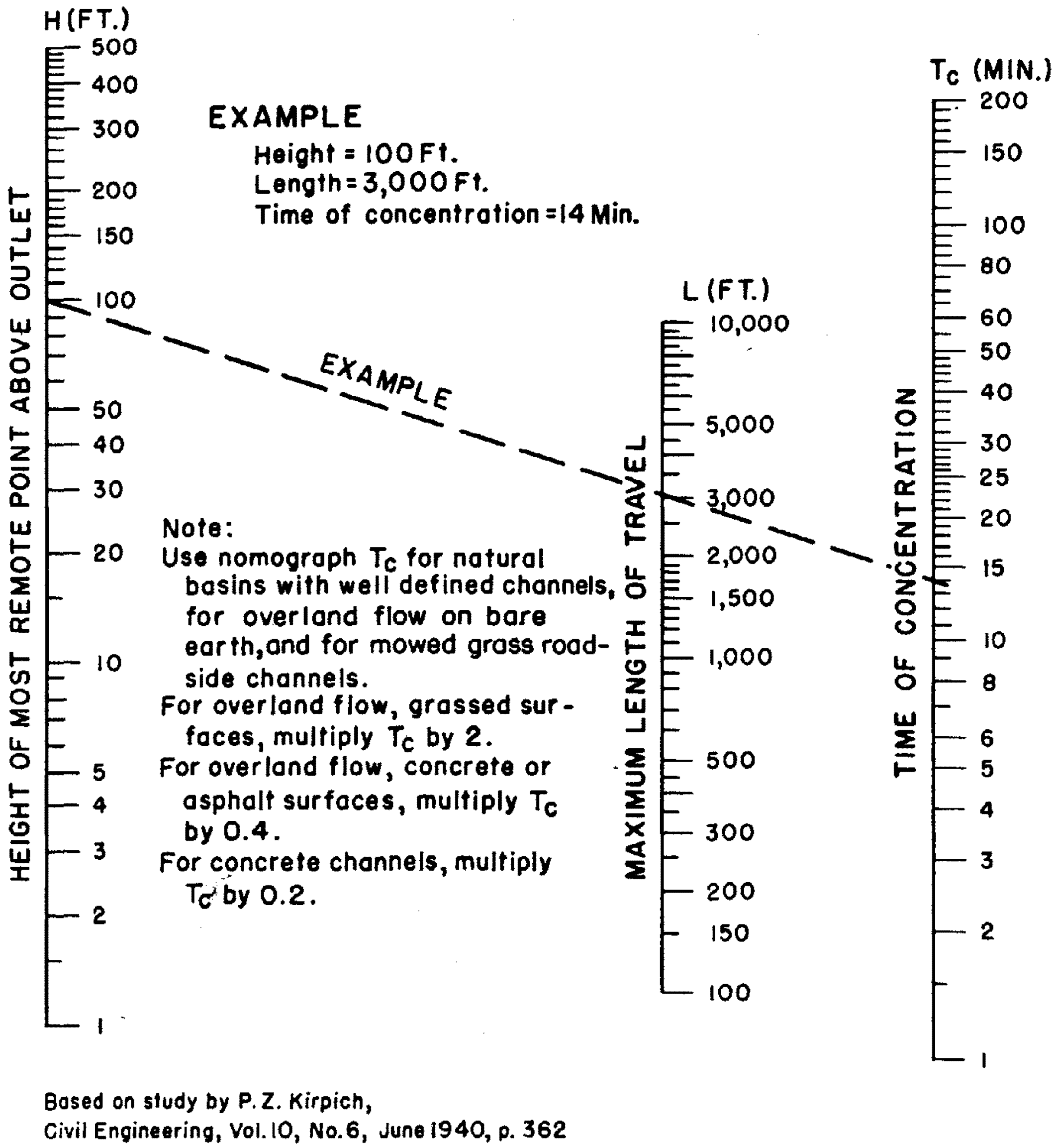


Figure 5.—Time of concentration of small drainage basins.

Example 1

Given: An area tributary to a roadway channel. The area has a fairly uniform cross section as follows: 12 ft. of concrete pavement; 26 ft. gravel shoulder, channel, and backslope; 200 ft. of grassed pasture. The length of the area is 400 ft.

Find: Runoff coefficient, C

Solution:

Area (sq. ft.)	Type of surface	C (table 1)	CA (sq. ft.)
4,800	Concrete pavement	0.9	4,320
10,400	Shoulder, channel and backslope	.5	5,200
80,000	Pasture	.3	24,000
95,200	Total		33,520

$$\text{Weighted } C=\frac{33,520}{95,200}=0.35$$

For use in the rational formula, the CA product (33,520) can be converted to acres and multiplied by i without computing the weighted C.

2.4-2 Time of concentration. The time of concentration (defined in sec. 2.4) varies with the size and shape of the drainage area, the land slope, the type of surface, the intensity of rainfall, whether flow is overland or channelized and many other factors. Extreme precision is not warranted in determining time of concentration for the design of drainage channels on rural highways. Time of concentration can be obtained from figure 5 which is based on a study by Kirpich (*22*, p. 309-318) of six watersheds which varied in size from 1.25 to 112 acres. The watersheds were all located on a single farm in Tennessee. Research is badly needed on the time of concentration of other types of watersheds. The values of T_c from figure 5 are based on meager data, and should only be used when better information is not available. A minimum time of concentration of 5 minutes is recommended for finding the intensity used for estimating the design discharge.

Use of figure 5 requires the length (L) of the drainage area measures along the principal drainage line to the most remote (longest T_c) point and the height of this point above the outlet at which the flow is to be estimated.

2.4-3 Rainfall intensity. Rainfall intensity-frequency data (sec. 2.3) are taken from Weather Bureau Atlas Technical Paper 40 (*20*) or from Weather Bureau Technical Paper 25 (*19*). A chart such as that of figure 6 (*19*, p. 9) is constructed for the project location, but the project chart need contain only the curves for the frequencies to be used for the project designs for the T_c values likely to be encountered. If the project is near a Weather Bureau station for which a rainfall intensity-duration-frequency curve is given in Technical Paper 25 (*19*), the Weather Bureau chart can be used directly or for frequencies less than 10 years convert chart values to the partial-duration series. For other areas, the design chart should be constructed from the maps in the Rainfall-Frequency Atlas of the United States (*20*). For use in the rational method, the values of total rainfall are converted to rainfall intensity by dividing the map value by the duration expressed in hours.

When the Weather Bureau Atlas is not available, approximate rainfall intensity-duration-frequency data can be obtained from figure 7 by the use of coefficients. Figure 7 is adapted from a map in the Weather Bureau Atlas by converting total rainfall for 30 minutes to 30-minute rainfall intensity (divide map value by 0.5) for the 2-year recurrence interval. The 2-year rainfall intensity for other durations is obtained by multiplying the 30-minute rainfall intensity for the project location from figure 7 by the following factors:

Duration (minutes)	Factor	Duration (minutes)	Factor
5	2.22	40	0.8
10	1.71	50	0.7
15	1.44	60	0.6
20	1.25	90	0.5
30	1.0	120	0.4

To convert 2-year recurrence interval rainfall for a given duration to other recurrence intervals, multiplying by the following factors gives acceptable results in much of the country. The use of these factors should be checked against the Weather Bureau Atlas before extensive use in a particular locality. More accurate results can be obtained by using the maps or the method given in reference 20 (using the relation of 2-year and 100-year rainfall).

Recurrence interval (years):	*Factor*
1	0.75
2	1.0
5	1.3
10	1.6
25	1.9
50	2.2

2.4-4 Drainage area. The drainage area, in acres, contributing to the point for which channel capacity is to be determined, can be measured on a topographic map or determined in the field by estimation, pacing, or a survey comparable in accuracy to the stadia-compass traverse. The data required to determine time of concentration (sec. 2.4-2) and the runoff coefficient (sec. 2.4-1) should be noted at the time of the preliminary field survey.

2.4-5 Computing the design discharge. The design discharge is computed by the formula, $Q=CiA$ (sec. 2.4) as explained in example 2.

Example 2

Given: The contributing area as described in example 1 (sec. 2.4-1). The weighted C is 0.35; the channel grade is 0.5 percent; and the outer edge of the contributing area at the crest of the hill is 4 ft. above the bottom of the channel. The location is near Washington, D.C.

Find: The discharge for a 10-year frequency rainfall at the outlet of a grassed roadside channel 400 ft. from the crest of a hill.

Solution: The time of concentration is obtained from figure 5. The distance from the channel to the ridge of the area is 210 ft. (200 ft. of grassed pasture and 10 ft.

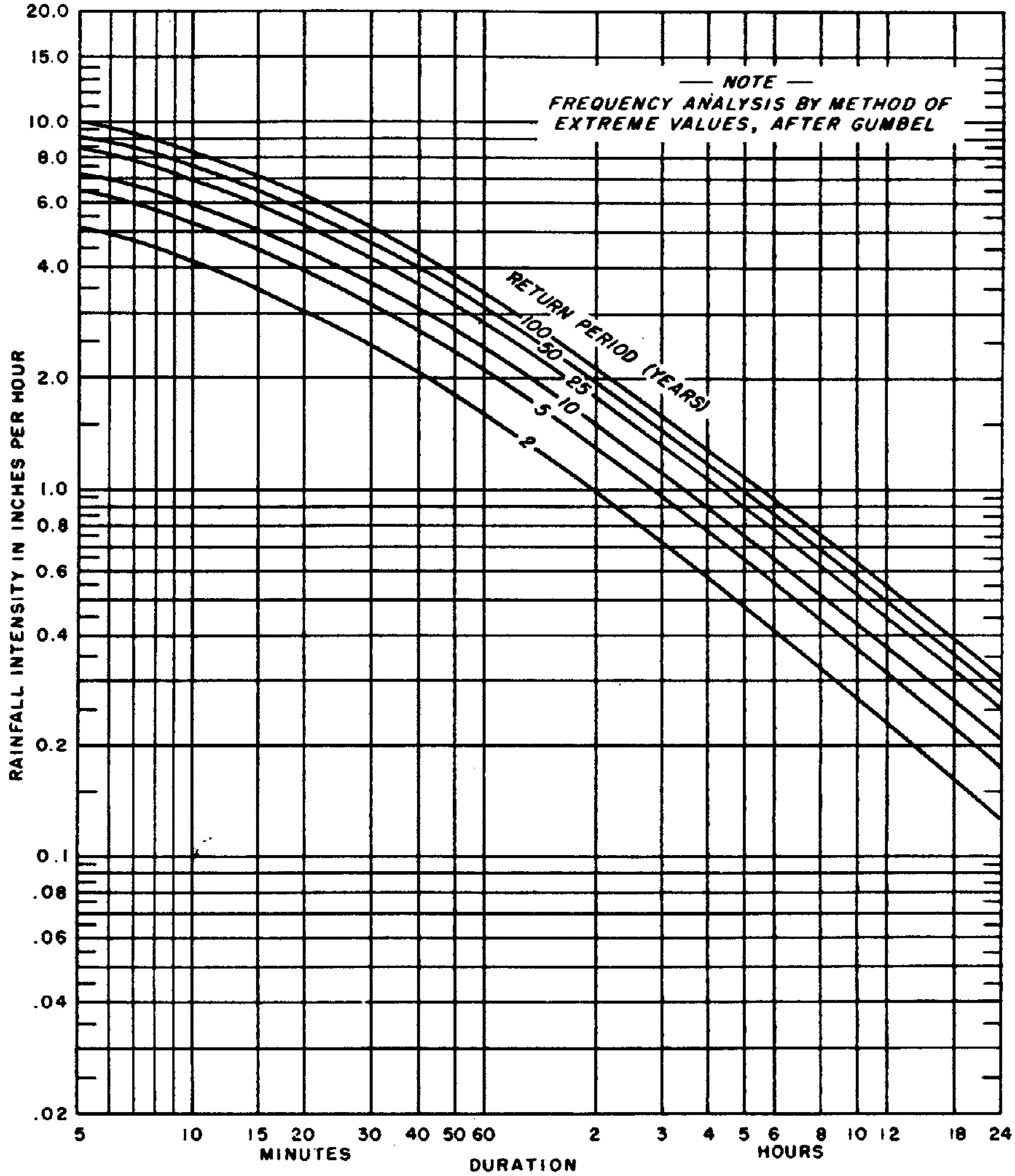


Figure 6.—Rainfall intensity—duration—frequency curve for Wash. D.C., 1896–97, 1899–1953 (U.S. Weather Bureau).

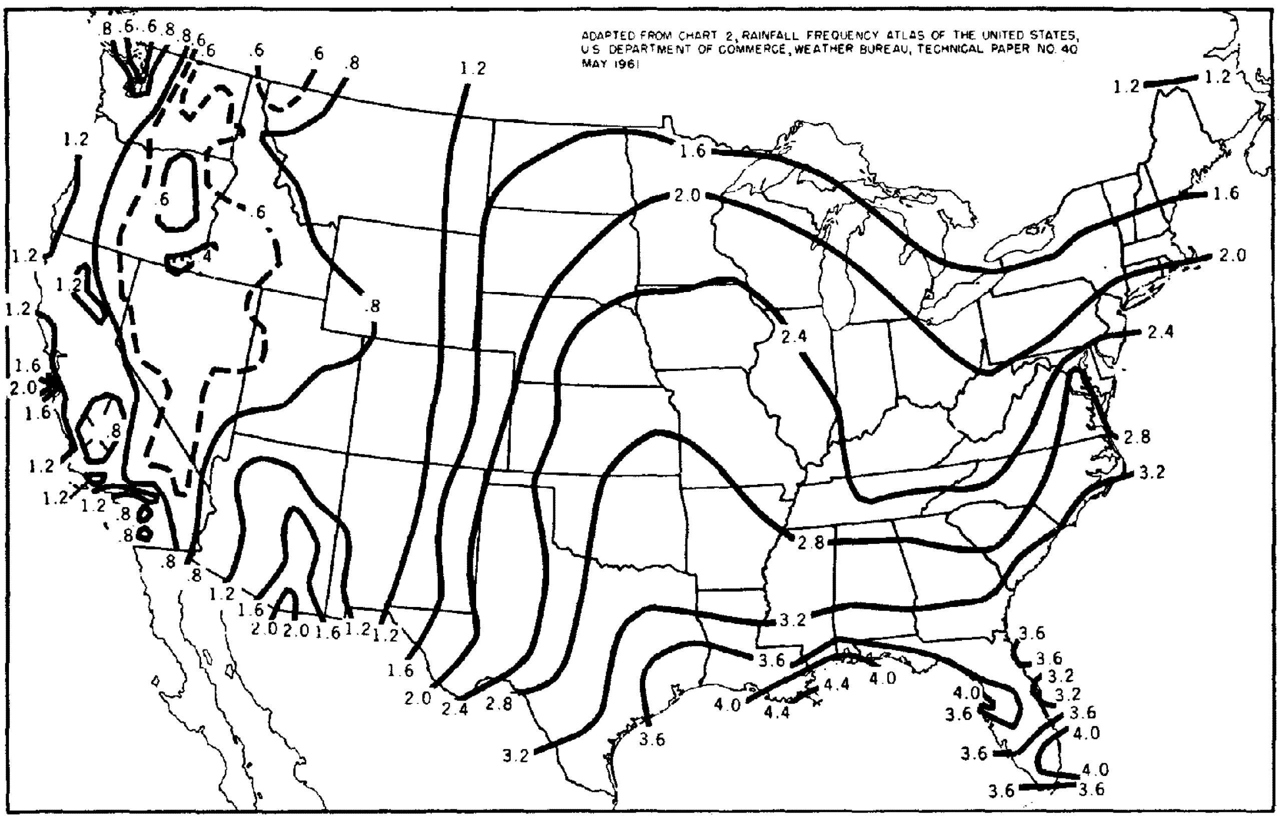


Figure 7.—Map of the contiguous United States showing 2-year, 30-minute rainfall intensity.

of ditch bank slope) and that of the channel is 400 ft., making $L=610$ ft. The height of the most remote point above the outlet $=4$ ft. $+(0.005\times400)=6$ ft. From figure 5, $T_c=7$ min.

The rainfall intensity for a 7-minute duration and 10-year return period is 6.8 in./hr. (fig. 6).

The drainage agea $=238\times400=95{,}200$ sq. ft. or

$$\frac{95{,}200}{43{,}560}=2.2 \text{ acres}$$

Then $Q=0.35\times6.8\times2.2=5.2$ c.f.s.

2.4–6 Computing the design discharge for complex drainage areas. Example 2 explains the rational method for a simple drainage area. For other points along the channel, the design discharge is computed using the longest time of travel to the point for which the discharge is to be determined. Generally, less work is required to compute the runoff from the separate areas as is done for point 5B in section 8.5 rather than obtain a weighted C for the whole area.

On some combinations of drainage areas, the maximum rate of runoff will be reached from the higher intensity rainfall for periods less than the time of concentration for the whole area, even though only a part of the drainage area is contributing. One example of such an occurrence is the discharge for point 5C, section 8.5. In this example the longest time of concentration is 14 minutes, but 20.0 acres of the total of 21.4 acres has a time of concentration of only 5 minutes. The 20 acres with a 5-minute intensity produces a higher peak discharge than the total area with a 14-minute intensity. Another combination of drainage areas that might produce the maximum rate of runoff from a partial contribution of the total area is a downstream impervious area with a high runoff coefficient and a short time of concentration. This could produce the same effect as the unbalanced area combination just described.

Extreme precision is not warranted in determining the combination of contributing areas (CA) and rainfall intensity (i) that would produce the greatest peak runoff. Unless the areas or times of concentration are considerably out of balance, as the example in the preceding paragraph, it is unnecessary to check the peak from part of the area.

The discharges in the illustrative problems of chapter 8 were also computed by the rational method, using combinations and assumptions other than those explained in this publication in order to check the sensitivity of the method. It was found that the method was stable for any reasonable assumption and that the results by different assumptions were well within the range of accuracy of the method.

Chapter III.—HYDRAULICS OF DRAINAGE CHANNELS

3.1 General. Most highway drainage channels can be designed by using the open-channel charts in Hydraulic Design Series No. 3 (*25*) or the tables and formulas presented in this publication. The tables contained in references 26 through 28 are helpful in designing channels not included in reference 25. These aids reduce the work required to design drainage channels, but they cannot replace engineering judgment and a knowledge of the hydraulics of open-channel flow. Open-channel flow also includes flow through conduits with a free water surface. The discussion of hydraulics in this chapter does not provide the knowledge of open-channel flow needed for the more difficult design problems; nevertheless, a brief review of some of the hydraulic principles governing open-channel flow should be helpful to the designer of highway drainage channels. A movie, "Introduction to Highway Hydraulics," illustrates many of the principles of open-channel flow. The movie is available on loan from the Bureau of Public Roads.

Flow in open channels is classified as steady or unsteady. Unsteady flow results from variations in the supply and will not be covered in this publication. Although the flow in most channels during the storm period is an excellent example of unsteady flow, we are usually interested in designing a channel to carry a peak flow that recurs with a selected frequency. Considering the peak flow as steady flow greatly simplifies the design of drainage channels.

Steady flow occurs when the quantity of water passing any section of the stream is constant. Steady flow is further classified as uniform if velocity and depth of flow are constant, and nonuniform or varied if velocity and depth of flow changes from section to section. Although most drainage channels are designed on the assumption of uniform flow, a knowledge of varied flow is needed to solve the more complex flow problems. Another classification of flow, subcritical (tranquil) or supercritical (rapid or shooting), will be discussed in section 3.3–2.

3.2 Uniform flow. To have uniform flow, the grade must be constant and all cross sections of flow must be identical in form, roughness, and area, necessitating a constant mean velocity. Under uniform flow conditions, the depth (d_n) and the mean velocity (V_n) for a particular discharge are said to be normal. Under these conditions, the water surface is parallel to the streambed (fig. 11). Normal depth is also defined as the depth at which uniform flow will occur when a given quantity of water flows through a long channel of uniform dimensions, roughness (n), and slope (S_0).

Uniform flow conditions are rarely attained in drainage channels, but the error in assuming uniform flow in a channel of fairly constant slope and cross section is small in comparison to the error in determining the design discharge. If the channel cross section, roughness, and slope are fairly constant over a sufficient distance to establish essentially uniform flow, equations such as that of Manning give reliable results.

3.2–1 The Manning equation. Water flows in a sloping drainage channel because of the force of gravity. The flow is resisted by the friction between the water and the wetted surface of the channel. The quantity of water flowing (Q), the depth of flow (d), and the velocity of flow (V) depend upon the channel shape, roughness, and slope (S_o). Various equations have been devised to express the flow of water in open channels. A useful equation for channel design is that named for Robert Manning, an Irish engineer. The Manning equation for velocity of flow in open channels is:

$$V=\frac{1.49}{n}R^{2/3}S^{1/2} \qquad (2)$$

Where

V = mean velocity in feet per second (f.p.s.).
n = Manning coefficient of channel roughness.
R = hydraulic radius, in feet.
S = slope, in feet per foot. When the Manning equation applies, $S=S_0$.

The value of the Manning coefficient n is determined by experiment. Some n values for various types of channels are given in table 2, appendix A.

R, the hydraulic radius, is a shape factor that depends only upon the channel dimensions and the depth of the flow. It is computed by the equation,

$$R=\frac{A}{WP} \qquad (3)$$

Where

A = cross-sectional area of the flowing water in square feet taken at right angles to the direction of flow.
WP = wetted perimeter or the length, in feet, of the wetted contact between a stream of water and its containing channel, measured in a plane at right angles to the direction of flow.

Another basic equation in hydraulics is

$$Q=AV \qquad (4)$$

or discharge (Q) is the product of the cross-sectional area (A) and the mean velocity (V).

By combining equations (2) and (4), the Manning equation can be used to compute discharge directly or

$$Q=\frac{1.49}{n}AR^{2/3}S^{1/2} \qquad (5)$$

In many computations it is convenient to group the properties peculiar to the cross section, in one term, called conveyance (K) or

$$K=\frac{1.49}{n}AR^{2/3} \qquad (6)$$

then

$$Q=KS^{1/2} \qquad (7)$$

When a channel cross section is irregular in shape such as one with a relatively narrow deep main channel and a wide shallow overbank channel, the cross section must be subdivided and the flow computed separately for the main channel and for the overbank channel. (See *29*, sec. 8.5.) The same procedure is used when different parts of the cross section have different roughness coefficients. In computing the hydraulic radius of the subsections, the water depth common to two adjacent subsections is not counted as wetted perimeter.

Conveyance can be computed and a curve drawn for any channel cross section. The area and hydraulic radius are computed for various assumed depths and the corresponding value of K is computed from equation (6). The values of conveyance are plotted against the depths of flow and a smooth curve connecting the plotted points is the conveyance curve. If the section was subdivided, the conveyance of each subsection (k_a, k_b, . . . k_n) is computed and the total conveyance of the channel is the sum of the conveyances of the subsections or $K=\Sigma k_a+k_b \ldots +k_n$. Discharge can be computed using equation (7). The discharge in each subsection can also be computed with equation (7), using the conveyance of the subsection.

The concept of channel conveyance is useful when computing the distribution of overbank flood flows in the stream cross section and the distribution through the openings in a proposed stream crossing (*29*). The discharge through each opening can be assumed to have the same ratio to the total discharge as the ratio of conveyance of the opening bears to the total conveyance of the channel.

Discharges computed by the Manning equation do not have the accuracy to which the computation can be carried. Results of the discharge computations are generally rounded off to avoid an inference of great accuracy.

3.2-2 Aids in solution of the Manning equation. The examples in this section show the solution of the Manning equation and the use of the computation aids mentioned in section 3.1. Figure 9, used in example 4, is typical of the open-channel charts in reference 25. Nomographs such as that in appendix B provide a graphical solution of the Manning equation (see example 3). Tables in references 26–28 contain the channel properties (A, R) of many channel sections and tables of velocity for various combinations of slope and hydraulic radius. Their use is explained in example 3. In addition to tables of channel properties, reference 28 contains tables for the direct solution of the Manning equation. (See sec. 4.6–2.) Charts in reference 30 can be used to compute flow in rectangular and trapezoidal sections and circular sections with free water surface.

The Manning equation can be readily solved for a given channel when the normal depth (d_n) is known. The nomograph, appendix B, can be used to solve the equation graphically as explained in example 3.

Example 3

Given: A trapezoidal channel, of straight alinement and uniform cross section in earth, bottom width 2 ft., side slopes 1 to 1, slope 0.003, and normal depth 1 ft.

Find: Velocity and discharge.

Solution:

1. In table 2, appendix A, for a weathered earth channel in fair condition, n is 0.020.
2. The cross-sectional area (A) of the channel is 3.0 sq. ft. and the hydraulic radius (R) is 0.6 ft. (A and R can be computed or obtained from tables such as that on p. 121 of reference 26.)
3. Using the nomograph of appendix B, lay a straightedge between the outer lines at the values of $S=0.003$ and $n=0.02$. Mark the point where the straightedge intersects the turning line.
4. Then place the straightedge so as to line up the point on the turning line and the hydraulic radius of 0.6 ft.
5. Read the velocity, 2.9 f.p.s. on the velocity line.
6. The discharge, $Q=AV$, is 3.0 sq. ft. times 2.9 f.p.s. or 8.7 c.f.s. The velocity can also be obtained from the table on page 388 of reference 26.

The preceding example showed the solution of the Manning equation for the velocity and discharge in a channel of given dimensions when the normal depth is known. The more common problem in channel design is to find the size of channel required to carry the design discharge on the available slope and to compute the velocity in the channel in order to determine what protection is needed to prevent channel erosion. Unless channel charts (*25*) or special tables (*28*) are available, this problem requires a trial-and-error procedure illustrated by example 4.

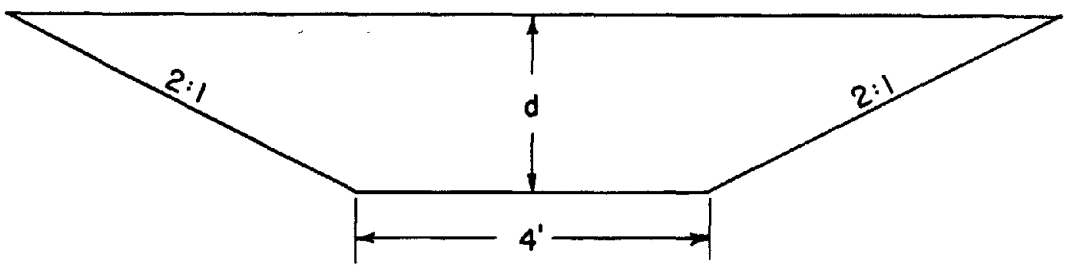


Figure 8.—Trapezoidal channel.

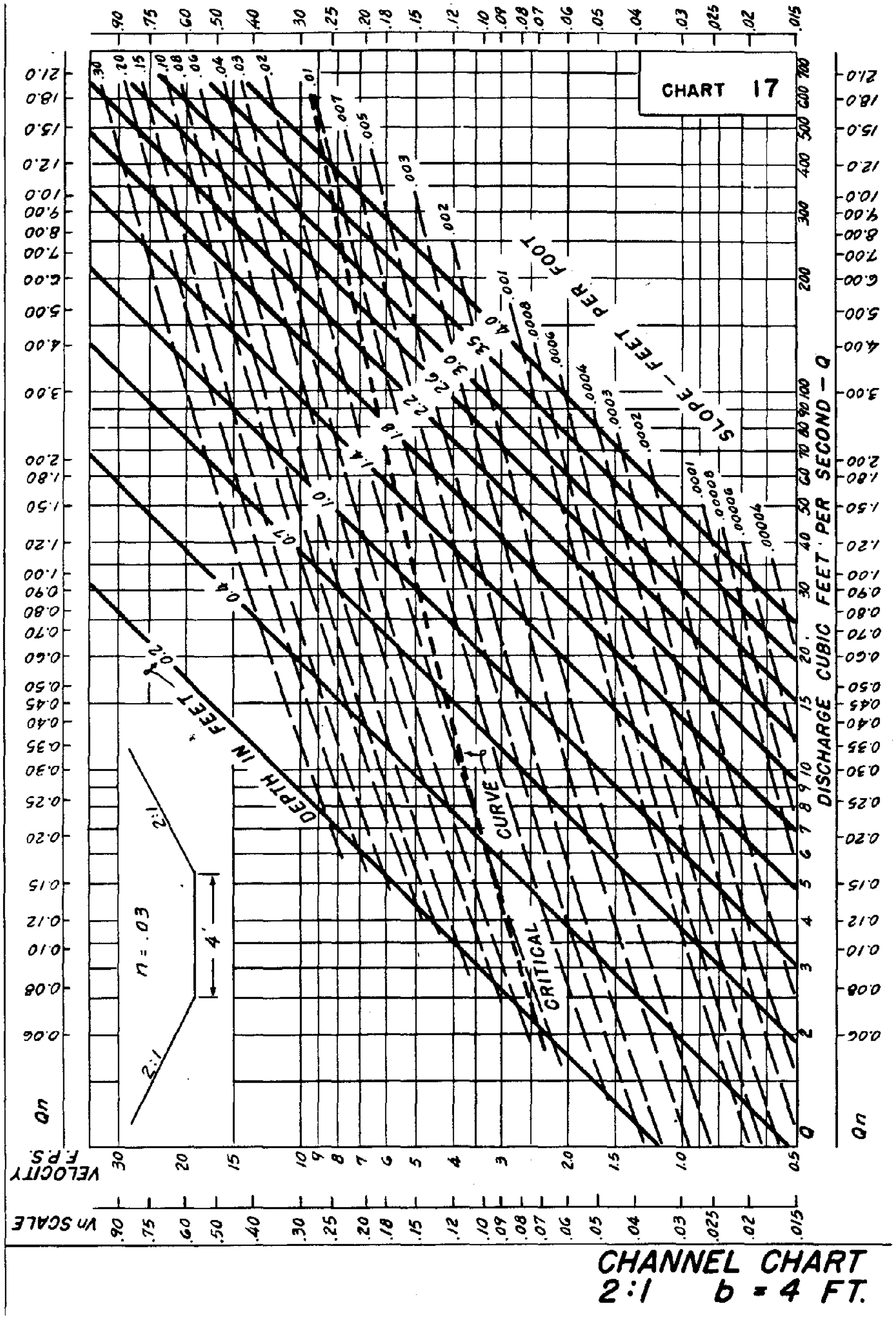


Figure 9.—Open-channel chart (from design charts for open-channel flow, hydraulic design series No. 3).

Example 4

Given: A trapezoidal channel (fig. 8) in stiff clay, bottom width 4 ft., side slopes 2:1, $n=0.03$, slope 0.005, and discharge 100 c.f.s., water carries fine silt.

Find: Depth, d, and velocity, V.

Solution:

1. General solution of area and hydraulic radius.

$A=(4+2d)d$

$WP=4+2d\sqrt{5}=4+4.47d$

$R=\frac{A}{WP}=\frac{(4+2d)d}{4+4.47d}$

2. Try: $d=3$ ft.

$A=(4+6)\ 3=30$ sq. ft.

$WP=4+13.4=17.4$ ft.

$R=1.72$ ft.

From the nomograph of appendix B, $V=5.1$ f.p.s.

$Q=30\ (5.1)=153$ c.f.s. too high

3. Try: $d=2.5$ ft.

$A=(4+5)\ (2.5)=22.5$ sq. ft.

$WP=4+(4.47)\ (2.5)=15.2$ ft.

$R=1.48$ ft.

From the nomograph, $V=4.6$ f.p.s.

$Q=22.5\ (4.6)=104$ c.f.s. too high

4. Try: $d=2.4$ ft.

$A=(4+4.8)\ 2.4=21.1$ sq. ft.

$WP=4+4.47\ (2.4)=14.7$ ft.

$R=1.44$ ft.

From the nomograph, $V=4.5$ f.p.s.,

$Q=4.5\ (21.1)=95$ c.f.s. too low

The last two trials ($d=2.5$ and 2.4 ft.) are about equally high and low, thus, the solution is $d=2.45$ ft., $V=4.6$ f.p.s.

Where design charts are available, the problem is greatly simplified; for example, see figure 9 taken from reference 25. For $Q=100$ c.f.s. and $S=0.005$, the value of d is about 2.45 ft. and $V=4.6$ f.p.s. See also section 4.6–2 for a direct solution using King's table.

Table 3, appendix A, shows a velocity of 5.0 f.p.s. can be permitted in a channel in stiff clay. Thus the natural material will withstand erosion.

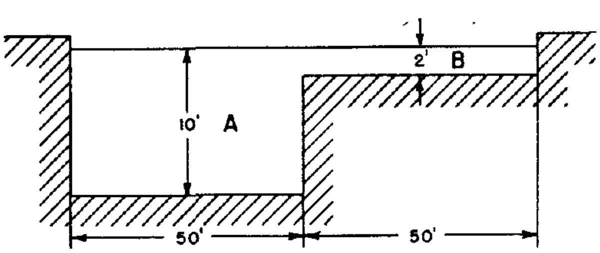


Figure 10.—Irregular-channel section.

The discharge in an irregular section is computed by dividing the channel into subsections, each approximately regular in shape. See example 5 for a two-subsection channel.

Example 5

Given: The channel in figure 10, $n=0.02$, and slope 0.005.

Find: Discharge, Q. The average velocity has little meaning in a channel of this shape.

Solution:

1. Subsection A:

$A=10\ (50)=500$ sq. ft.

$WP=10+50+8=68$ ft.

$R=\frac{500}{68}=7.35$

The velocity by formula (2) is:

$V=\frac{1.49}{0.020}\ (7.35)^{2/3}\ (0.005)^{1/2}=19.9$ f.p.s.

The velocity could have been obtained from the nomograph, appendix B, or page 388 of reference 26.

$Q=19.9\ (500)=9{,}950$ c.f.s.

2. Subsection B:

$A=2\ (50)=100$ sq. ft.

$WP=50+2=52$ ft.

$R=\frac{100}{52}=1.92$ ft.

$V=8.1$ f.p.s.

$Q=100\ (8.1)=810$ c.f.s.

3. For the entire channel:

$A=500+100=600$ sq. ft.

$Q=9{,}950+810=10{,}800$ c.f.s.

If the channel had been considered as a whole without subdividing, the following results would have been obtained.

$A=600$ sq. ft.

$WP=52+68=120$ ft.

$R=\frac{600}{120}=5.00$ ft.

$V=15.4$ f.p.s.

$Q=15.4\ (600)=9{,}240$ c.f.s.

The discharge (9,240 c.f.s.) computed for channel considered as a whole is less than the discharge computed separately for subsection A (9,950 c.f.s.). This paradox is brought about by the effect of the hydraulic radius on the computations. It also illustrates the necessity of subdividing highly irregular sections whether irregular in shape or in roughness and shows the difference in mean velocities in a shallow section as compared with the mean velocity of the main channel. The true discharge of this channel is perhaps less than 10,800 c.f.s., but it is much closer to 10,800 than to 9,240 c.f.s.

3.3 Nonuniform or varied flow. Varied steady flow occurs when the quantity of water remains constant, but the depth of flow, velocity, or cross section changes from section to section. The relation of all cross sections will be:

$$Q = A_1 V_1 = A_2 V_2 \qquad = A_n V_n \tag{8}$$

Equation 8 is sometimes called the equation of continuity.

Velocity of uniform flow in open channels can be computed by the Manning equation, using the slope of the channel bed as the slope of the energy line but nonuniform steady flow computations require other methods.

The hydraulic design engineer needs a knowledge of varied flow in order to determine the behavior of the flowing water when changes in channel resistance, size, shape, or slope occur. A discussion of varied flow properly begins with a discussion of energy of the flowing water.

3.3–1 Energy of flow. Water flowing in an open channel possesses energy of two kinds—potential energy and kinetic energy. The potential energy is due to the position of the water above some datum, and kinetic energy is due to the velocity of the flowing water. In channel problems, energy is conveniently expressed in terms of head. Thus, a column of water 20 feet high has a potential (static) head of 20 feet with respect to the bottom of the column. Flowing water has both *potential head* and *velocity head,* the velocity head being equal to

$$\frac{V^2}{2g}$$

Where

V = the mean velocity in feet per second.

g = acceleration of gravity or 32.2 feet per second per second.

A useful hydraulic concept of the energy of the flowing water within one vertical cross section of the channel is that of specific head (also called *specific energy*).

$$\text{Specific head } (H_0) = d + \frac{V^2}{2g} \tag{9}$$

If the potential head is related to some datum (fig. 11), at or below the bed of the channel at the outlet, energy can be expressed in terms of total head. If Z is the elevation of the channel bottom, total head at any section is:

$$\text{Total head } (H) = d + \frac{V^2}{2g} + Z \tag{10}$$

The energy losses due to friction, channel contractions, changes in alinement, and other factors are termed *head losses* (h_L). The law of conservation of energy (Bernoulli's theorem) states that the total head at any section is equal to the total head at any section downstream, plus intervening head losses or for the channel in figure 11, is equal to the total head at section 2, plus head loss between sections 1 and 2, or

$$d_1 + \frac{V_1^2}{2g} + Z_1 = d_2 + \frac{V_2^2}{2g} + Z_2 + h_L \tag{11}$$

In figure 11, the head loss, in a channel of uniform cross section, equals the change in Z or $(Z_1 - Z_2)$. Thus, the water surface is parallel to the streambed, and

$$d_1 + \frac{V_1^2}{2g} = d_2 + \frac{V_2^2}{2g}$$

The flow is uniform and can be computed by the Manning equation. The head loss

$$(Z_1 - Z_2) = LS_0 \tag{12}$$

Where

L = horizontal distance between section 1 and section 2

S_0 = channel slope or $\frac{Z_1 - Z_2}{L}$

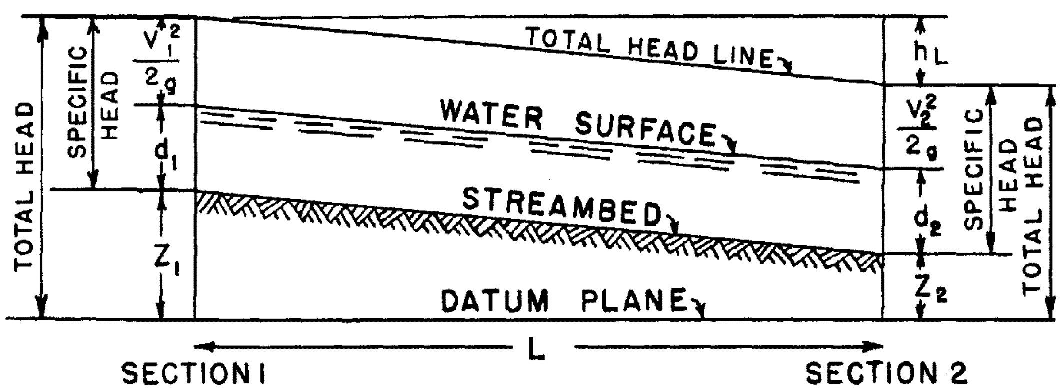


Figure 11.—Water-surface profile of channel with uniform flow.

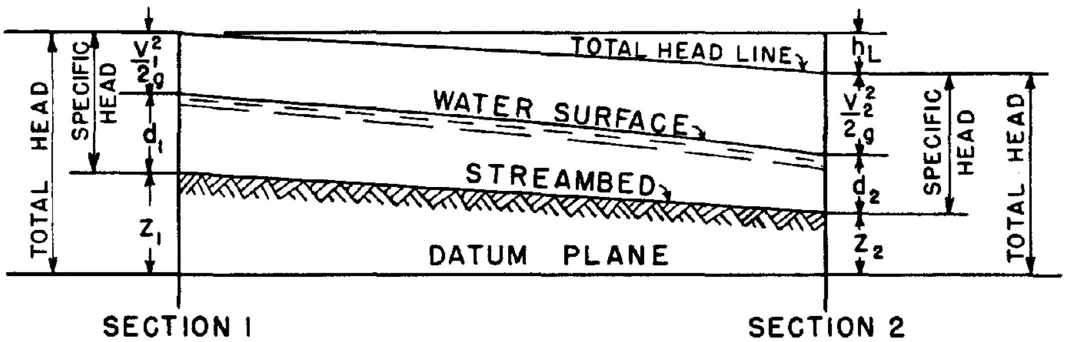


Figure 12.—Water-surface profile of channel with nonuniform flow.

S_o in uniform flow is sometimes called the friction slope. For uniform flow, the Manning equation (sec. 3.2–1) can be computed for $S(=S_o)$

$$S=\left(\frac{Vn}{1.49R^{2/3}}\right)^2 \tag{13}$$

When the head loss does not equal the change in Z, nonuniform flow occurs and the depth of flow either increases or decreases in a uniform channel. In figure 12, flow takes place with decreasing depth.

Between sections 1 and 2, the velocity is increasing and the rate of losing energy is, therefore, not constant. This condition could be caused by a channel slope steeper than that needed to overcome frictional resistance or by a change in channel cross section. Thus, the total head line (also called the energy line or energy gradient) is not a straight line. The water surface line in an open channel is sometimes called the hydraulic grade line.

3.3–2 Critical Flow. With a constant discharge passing a cross section, changing the depth of flow causes a different specific head for each depth. If specific head is plotted against depth of flow, the result is a specific-head (energy) diagram. (See fig. 13.)

The specific-head curve is asymptotic to the line representing the energy due to depth and the vertical line of zero depth. Examination of figure 13 reveals several important facts. Starting at the upper right of the curve with a large depth and small velocity, the specific head decreases with decrease in depth, reaching a minimum value at depth d_c, known as *critical depth*, sometimes called the depth of the minimum energy content. Further decrease in depth results in rapid increase in specific head. For any value of specific head except that corresponding to critical depth, there are alternate depths at which the flow could occur. These alternate depths are sometimes referred to as equal energy depths.

When the flow occurs at depths greater than critical depth (velocity less than critical), the flow is called *subcritical or tranquil*. When the flow occurs at depths less than critical depth (velocities greater than critical), the flow is called *supercritical, rapid, or shooting*. The change from supercritical to subcritical flow is often very abrupt, resulting in the phenomenon known as *hydraulic jump*.

Flow at the critical depth is called *critical flow* and the velocity at critical depth is the *critical velocity*. The channel slope which produces critical depth and critical velocity for given discharge is the *critical slope*.

Critical depth for a particular discharge is dependent on channel size and shape only and is independent of channel slope and roughness. Critical slope depends upon the channel roughness, the channel geometry and the discharge. For a given critical depth and critical velocity, the critical slope for a particular roughness can be computed by the Manning equation (sec. 3.2–1).

Supercritical flow is difficult to control because abrupt changes in alinement or in cross section produce waves which travel downstream, alternating from side to side, and sometimes cause the water to overtop the channel sides. Changes in channel shape, slope, or roughness cannot be reflected upstream except for very short distances (upstream control). Supercritical flow is common in steep flumes, and in mountain streams. Pulsating flow (*30*) can occur at depths as great as 8 feet.

Subcritical flow is relatively easy to control. Changes in channel shape, slope, and roughness affect the flow for some distances upstream (downstream control). Subcritical flow is characteristic of the streams located in the plains and valleys regions where stream slopes are relatively flat.

Critical depth is important in hydraulic analyses because it is always a hydraulic control. The flow must pass through critical depth in going from one type of flow to the other. Typical locations of critical depth are:

(1) At abrupt changes in slope when a flat (subcritical) slope is sharply increased to a steep (supercritical) slope.

(2) At a channel constriction such as a culvert entrance under some conditions.

(3) At the unsubmerged outlet of a culvert or flume on a subcritical slope, discharging into a wide channel or with a free fall at the outlet.

(4) At the crest of an overflow dam or weir.

Distinguishing between the types is important in channel design, thus the location of critical depth and the determination of critical slope for a cross section of given shape, size, and roughness becomes necessary. When flow occurs at critical depth

$$\frac{A^3}{T}=\frac{Q^2}{g} \tag{14}$$

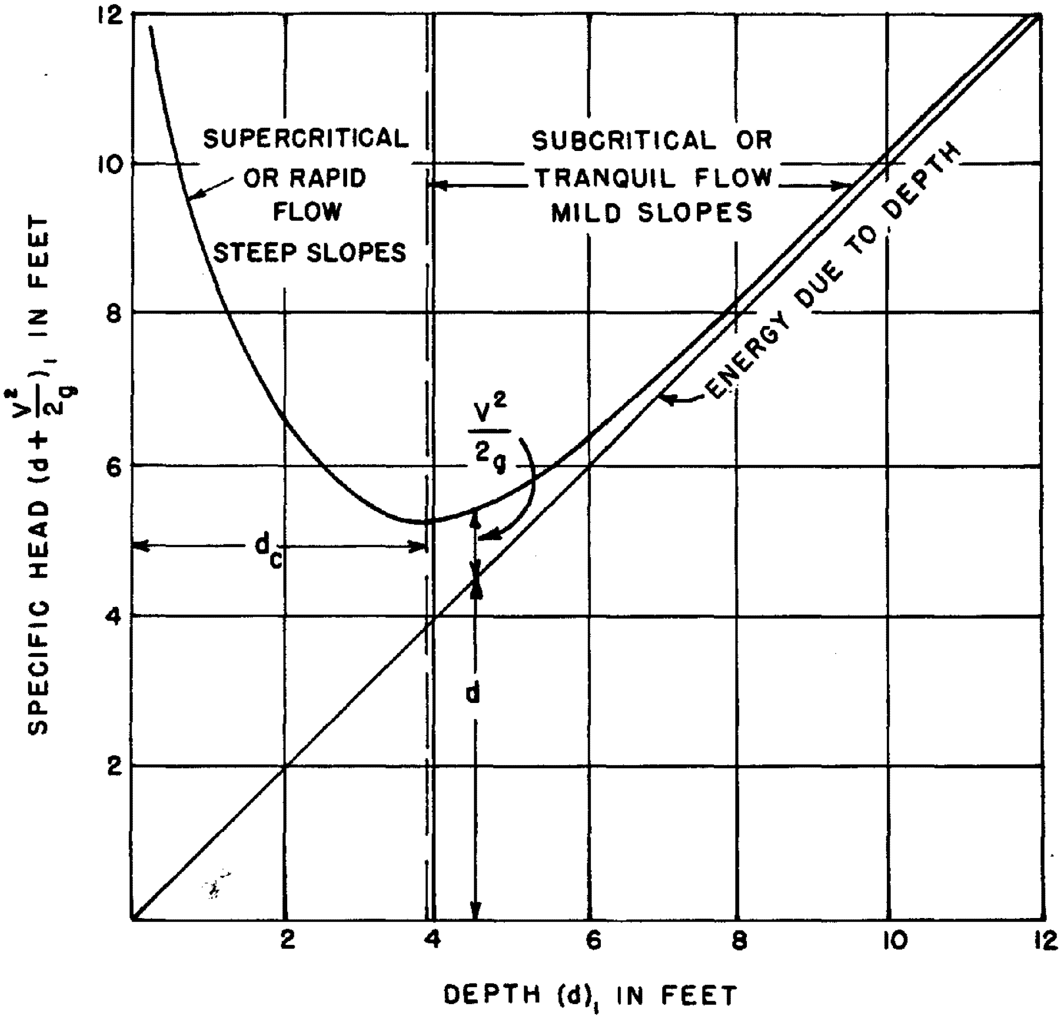


Figure 13.—Specific head diagram for constant Q.

Critical depth (d_c) can be found from the design charts in reference 25 or computed for various channel cross sections (see *28*, p. 8–7) by the following equations:

Rectangular sections

$$d_c=0.315\sqrt[3]{\left(\frac{Q}{B}\right)^2} \qquad (15)$$

Trapezoidal sections

$$d_c=\frac{4zH_0-3b+\sqrt{16z^2H_0^2+16zH_0b+9b^2}}{10z} \qquad (16)$$

The tables in King's Handbook (*28*) provide a much easier solution for critical depth than equation (16).

Triangular sections

$$d_c=0.574\sqrt[5]{\left(\frac{Q}{z}\right)^2} \qquad (17)$$

Circular sections, approximate solution (*31*)

$$d_c=0.325\left(\frac{Q}{D}\right)^{2/3}+0.083D \qquad (18)$$

Accurate only when $\frac{d_c}{D}$ lies between 0.3 and 0.9.

Where

A = area of cross section of flow, in square feet.
B = the width of a rectangular channel, in feet.
b = bottom width of a trapezoidal channel, in feet.
D = diameter of circular conduit, in feet.
g = acceleration of gravity, 32.2 feet per second².
H_0 = specific head in section, in feet (equation 9).
Q = rate of discharge, in cubic feet per second.
T = top width of water surface, in feet.
V = mean velocity of flow, in feet per second.
z = slope of sides of a channel (horizontal to vertical).

For irregular sections, critical depth may be found by a trial-and-error solution using equation (14). An expression for the critical velocity (V_c) in channels of any cross section is:

$$V_c = \sqrt{g d_m} \tag{19}$$

where d_m = the mean depth of flow $\left(\frac{A}{T}\right)$

In a given channel when the velocity head $\left(\frac{V^2}{2g}\right)$ is less than one-half the mean depth, the flow is subcritical; if the velocity head is equal to one-half (½) the mean depth, the flow is critical; and if the velocity head is greater than one-half (½) the mean depth, the flow is supercritical.

Uniform flow within about 10 percent of critical depth (*30*) is unstable and should be avoided in design. The reason for the unstable flow can be seen by referring to figure 13. As the flow approaches the critical depth from either limb of the curve, a very small change in energy is required for the depth to abruptly change to the alternate depth on the opposite limb of the specific-head curve. If the unstable flow region cannot be avoided in design, the least favorable type of flow should be assumed for the design.

3.3–3 Problems in nonuniform flow. Problems in nonuniform flow include computing the water surface profile, design of channel transitions, and dissipation of energy of the flowing water. All of these problems are beyond the scope of this publication; however, two cases of nonuniform flow are discussed briefly, subcritical flow around bends and a case that must be considered in the design of chutes. (See sec. 4.15.) The latter is a case of a sudden change in channel grade from one less than critical to one greater than critical. (See fig. 14.)

The depth of flow at section 1 can be computed using the Manning equation. The flow at section 2 (which for practical purposes can be assumed to occur at the change in grade) passes through critical depth (d_c). If the channel grade downstream from section 2 is equal to the critical slope, the flow will become uniform at a depth equal to d_c. However, when a drainage channel discharges into a chute, the chute grade is steeper than critical slope and the flow is nonuniform and accelerating. Section 2 becomes the control section for both the flow in the channel (downstream control) and the flow in the chute (upstream control).

Knowing the specific head (H_o) in the approach channel, the capacity of the chute entrance, such as section 2 (fig. 14), for a rectangular channel can be computed by a weir formula:

$$Q = 3.09\ k_e B H_0^{3/2} \tag{20}$$

For a trapezoidal channel the capacity can be computed by the formula (*23*, p. 79):

$$Q = 8.03 k_e (H_0 - d_c)^{1/2} (d_c)(b + z d_c) \tag{21}$$

Where

d_c = critical depth at section 2.
H_0 = specific head at section 1.
k_e = coefficient which represents the entrance loss—varies from 1.0 for perfect entrance of smooth curves and gradual transitions to 0.82 for rectangular shaped structure with square corners.
z = slope of sides of a channel (horizontal to vertical).

The critical depth, d_c, is computed by equations (15–19) of section 3.3–2.

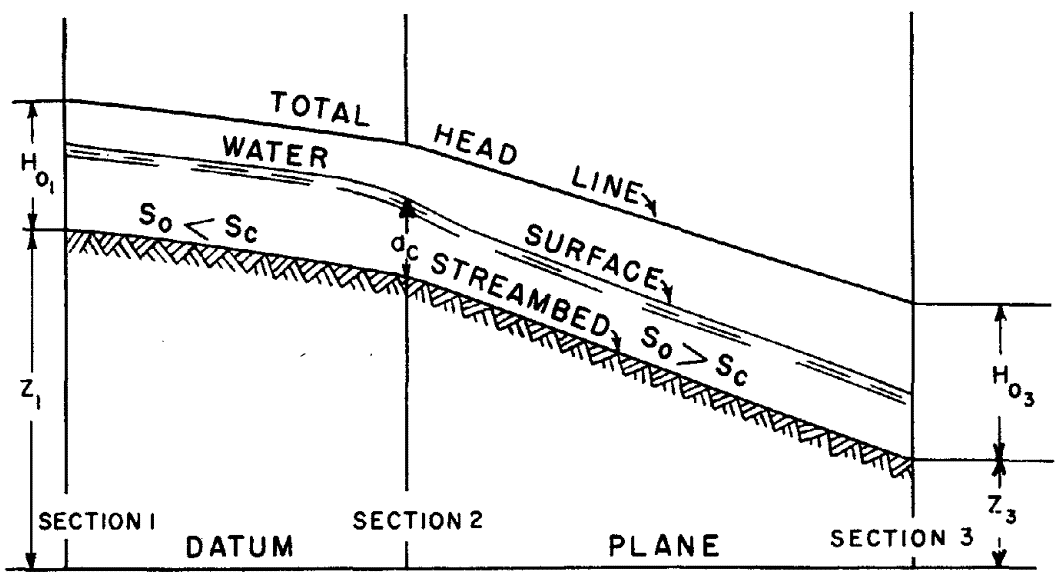


Figure 14.—Water-surface profile of channel with sudden change in grade.

The usual problem is to determine the size of chute channel required to carry a given discharge. The bottom width at the chute entrance (sec. 2) can be computed by equation (22) for a rectangular channel,

$$B=\frac{0.324Q}{k_e H_0^{3/2}} \qquad (22)$$

and approximately by equation (23) for a trapezoidal channel (*23*, p. 84).

$$b=\frac{0.324Q}{k_e H_0^{3/2}}-0.7zH_0 \qquad (23)$$

where the symbols are the same as those in equations (20) and (21) from which equations (22) and (23) were derived.

The flow through the chute must satisfy equation (11). If the head loss (h_L) through the chute is solely from friction, it can be expressed in terms of the hydraulic properties at each end of the chute, the roughness coefficient (n), and the length of chute (L) or

$$h_L=\frac{n^2}{4.41}\left[\left(\frac{V_1}{R_1^{2/3}}\right)^2+\left(\frac{V_2}{R_2^{2/3}}\right)^2\right]L \qquad (24)$$

If the flow is accelerating ($h_L < LS_0$), the cross section of a large chute can be gradually reduced in order to provide a more economical section, by the method described on page 8.41 of reference 28.

3.3–4 Subcritical flow around curves. When the flow is subcritical, the water surface is elevated on the outside of the curve and lowered on the inside of the curve. The approximate difference in elevation (ΔE) between the water surface along the sides of the curved channel can be found by the equation

$$\Delta E=\frac{BV^2}{rg} \qquad (25)$$

Where

B=width, in feet, of a rectangular channel.
g=acceleration of gravity, 32.2 feet per second per second.
r=mean radius, in feet, of curvature.
V=mean velocity, in feet per second.

The equation gives values of ΔE somewhat lower than will occur in the natural channel because of assumption of uniform velocity and uniform curvature but the computed value will be generally less than twenty percent in error (*22*, p. 523).

Other problems introduced by curved alinement of channels with subcritical flow include spiral flow, changes in velocity distribution, and increased friction losses within the curved channel as contrasted with the straight channel. Flow around bends is discussed by Chow (*32*) and others. (See references in ch. 16 of reference *32*).

3.3–5 Supercritical flow around curves. Changes in alinement of supercritical flow are difficult to make. The water traveling at supercritical velocities around bends of smooth channels builds up waves which may climb out of the channel and set up wave action continuing for some distance downstream. Changes in alinement, whenever possible, should be made near the upper end of the section—before the supercritical velocity has developed. If a change in alinement is necessary in a channel carrying supercritical flow, the channel should be rectangular in cross section, preferably enclosed and satisfy equation 31. Changes in alinement of open channels should be designed to reduce the wave action, resulting from changing the direction of flow. (See ch. 8, reference *22* and reference *33* and *34*.) Many designs involving supercritical flow should be model tested to develop the best design.

3.4 The Froude number. A useful parameter of flow is the Froude number, one form of which is

$$F=\frac{V}{\sqrt{gd_m}}=\frac{V}{V_c} \qquad (26)$$

Where

d_m=mean depth of flow in feet. In the general expression any characteristic dimension of flow might be used.
g=acceleration of gravity=32.2 f.p.s.^{2}
V=mean velocity in feet per second.
V_c=critical velocity for the channel and discharge.

The Froude number uniquely describes the flow pattern when gravity and inertia forces are the dominant factor in the flow. For example, in figure 13 each point on the specific-head curve has a single value of the Froude number, although two values on the curve can be found for a particular value of specific head. The Froude number of critical flow is one; values greater than one indicate supercritical flow and values less than one indicate subcritical flow.

Chapter IV.—DESIGN PROCEDURE

4.1 General. Highway drainage channels provide surface drainage for the highway right-of-way. They must be placed where they will adequately perform their drainage function. The principles are simple, but the actual layout and design of the drainage system requires an experienced hydraulic engineer who is familiar with both the construction and the maintenance problems involved.

After the highway has been located, the topography and other factors largely fix the location, alinement, and grade of the channels and determine the quantity of surface water entering the channels. Design of the channel then consists of determining a suitable channel section and specifying the type of channel lining needed to protect the channel from erosion.

4.2 Layout of the drainage system. The layout of the drainage system should preferably be made on a topographic map which contains the location of the highway, the location of all drainage structures, and the accentuated ridge and drainage lines. (See fig. 25.) The edges of the right-of-way and of the roadway are drawn on the map. Then the drainage channels necessary to intercept the water before it reaches the roadbed are sketched in, followed by the channels required to remove the water that cannot be intercepted before reaching the roadway. The quantities of water which must be removed when the design storm occurs are estimated for a few points along the drainage channels using the method explained in chapter II for areas smaller than 200 acres. It is rarely necessary to compute the incremental additions to the flow along the channels.

When the highway location cuts across tilled farmland, the highway drainage plan should be coordinated with the farmer's drainage system. Some farms are terraced and plowed on contour lines. Thus, the overland flow is collected and concentrated at points where the highway intersects the farm drainage channels, rather than occurring in a more or less uniform sheet over the hillside. Adequate notes by the locating party describing the farm drainage channels in the vicinity of the location center line are essential to the design of facilities for handling the storm runoff.

4.3 Channel grade. The approximate grade of the channel is computed from the topographic map. To prevent deposition of sediment, the minimum gradient for earth and grass-lined channels should be about 0.5 percent. The channel grade should be kept constant or increasing in the downstream direction, insofar as practicable, to avoid deposition. The grade affects both the size of the channel required to carry a given flow and the velocity at which the flow occurs. The flow should be subcritical whenever possible in order to avoid the adverse characteristics of supercritical flow.

4.4 Channel alinement. Changes in channel alinement should be as gradual as the width of right-of-way and terrain permits. Whenever practicable, changes in alinement should be made in the reaches of the channel which have the flatter slopes, particularly if the flow becomes supercritical on the steeper slopes.

4.5 Channel section. In general, open channels adjacent to the roadway should have a section with side slopes not steeper than 4:1 (horizontal to vertical) and a rounded bottom at least 4 feet wide (*1*). The rounding (vertical curves) at the junctions of the side slopes and the bottom do not appreciably affect the capacity, but they do add greatlv to the safetv and appearance of the channel.

Stonex (*5*, p. 26) states that for roads designed for speeds above 65 miles per hour, the minimum channel section required for safety should have side slopes 6:1, a bottom width of 6.5 feet, and vertical curves 6.5 feet long connecting sides and bottom. The depth of the channel need onlv be deep enough to carry the flow with a freeboard of from 0.3 to 0.5 ft. on flat slopes (*2*), except where subdrainage requires a deeper channel.

Safety considerations are not as important for drainage channels inaccessible to highway traffic; however, shallow, wide channels disturb the natural surface least and are much easier to maintain. Intercepting channels formed by dikes of borrow material do not disturb the existing topsoil and thus are less susceptible to erosion than are excavated channels. In most soils, the channel side slope should be no steeper than 1:1; but the limit will vary with the type of material, from vertical in rock and 1/2:1 in stiff clay to 2:1 or flatter in loam and sandy soils. Side slopes of vegetative-lined channels should be flat enough for easy maintenance and have suitable slopes for growing grass in the particular soil type and climate.

On some highways, storm water is removed through inlets and underground storm drains. The design of these facilities is beyond the scope of this publication.

4.6 Channel capacity. Determination of channel capacity was explained in section 3.2–2. Generally, the discharge to be carried is estimated for several points along the channel. The points selected should include a section immediately above sharp breaks in grade and the points of entry of concentrated flow. Channel charts (*25*) or King's tables (*28*) provide a direct solution of the size of channel needed to carry the flow. Without these aids a trial size of channel must be selected and the depth of channel required to carry the flow computed. The trial size is adjusted until a size is found that will carry the design discharge and the need for protection is determined (sec. 4.7). Freeboard is added to the required depth.

Where a standard channel section has been adopted as a minimum section, a capacity table for various grades and types of lining could be prepared as a guide to the adequacy of the standard channel section for a particular site.

The capacity of the trial channel can be increased by increasing the grade, the bottom width, the depth, or by decreasing the resistance of the channel through the use of a smoother lining. Increasing the bottom width has the least effect on the velocity of the flow and is generally the desirable way to increase capacity for a given depth and type of channel when the velocity is near the permissible limit.

4.6–1 Use of channel charts. Charts such as that in figure 9, taken from reference 25, greatly facilitate the design of drainage channels. Construction of the chart is explained in appendix B of reference 25. Each chart provides a direct solution to the Manning equation for a channel of given shape and roughness, but auxiliary scales make the charts applicable to other values of n. The abscissa scale is discharge, in cubic feet per second, and the ordinate scale is velocity, in feet per second. The chart contains a series of heavy solid lines for depths of flow (normal depth, d_n) and another series of lighter dashed lines for channel slope. Given any two of the conditions for flow, the other two elements of the flow can be read from the chart. A heavy dashed line shows the position of the critical curve. For channels having the same value of n as that for which the chart was constructed, values above the critical curve indicate supercritical flow and steep slopes, while values below the line indicate subcritical flow and mild slopes. Use of the channel charts for channels having the same value of n as that for which the charts were constructed is explained by example 6. Use of the charts with values of n other than that for which the chart was constructed is explained by example 7.

Example 6

Given: A trapezoidal channel in erosion-resistant soil; bottom width 4 ft.; side slopes 2:1; grade 0.5 percent; $n=0.030$; and discharge 30 c.f.s. The channel is to be lined with a mixed grass sod.

Find: Depth, velocity, critical slope, and adequacy of the channel lining.

Solution:

1. On figure 9 at the point of intersection of $Q=30$ c.f.s. and the slope line 0.005, the depth (without freeboard) is 1.4 ft. and the velocity is 3.2 f.p.s. Flow is subcritical, since the point of intersection is below the dashed critical curve. The maximum permissible velocity from table 4 is about 5 f.p.s.

2. Critical slope is read or interpolated from the slope line at the intersection of the design Q and the critical curve, 0.017 in this example.

The channel charts can be used for values of n other than that for which the chart was constructed by using the Q_n and V_n scales. The design discharge is first multiplied by the design value of n. Next the chart is entered with the value of Q_n on the Q_n scale and at the intersection of the Q_n value with the slope line read the value of V_n. V is then the V_n value divided by n. The value of the critical depth is read at the intersection of the Q line (not Q_n) and the critical curve and the critical velocity is the V value for this point. Critical slope varies with channel roughness and when the value of n is other than that for which the chart was constructed, proceed as shown in example 7.

Example 7

Given: The same cross section and design discharge (30 c.f.s.) as in example 6, but with a concrete lining (n is 0.015).

Find: Depth, velocity, and critical depth, velocity and slope.

Solution:

1. $Qn=30(0.015)=0.45$, and from the chart (fig. 9), Vn is 0.08 and $d=0.9$ ft. Then $V=0.08/0.015=5.3$ f.p.s.

2. Critical depth is 1.0 ft. and critical velocity is 5 f.p.s. (both read at intersection of Q and critical curve). Flow is in the supercritical range, since V is greater than V_c and d is less than d_c.

3. The critical slope is found by first finding the critical depth (1.0 ft.). The critical slope is then read or interpolated from the intersection of this depth line and the Qn value on the Qn scale. In this example the critical slope is 0.004, a lesser slope than in example 6 because of the reduced roughness of the channel.

4.6–2 Use of King's tables. "Handbook of Hydraulics" (*28*) contains tables for the direct solution of open-channel flow problems. The use of these tables to find the depth of channel required for a given discharge is explained by examples 8 and 15. The references to King's Handbook applies to section 7 of the 4th edition, but the corresponding references to chapter 7 of the 3d edition follows, in parentheses.

King's solution for depth of flow requires the computation of a discharge factor, K', by his equation 37 (40), which is

$$K'=\frac{Qn}{b^{8/3}S^{1/2}} \tag{27}$$

Where

Q=discharge, in cubic feet per second.
n=Manning coefficient of channel roughness.
b=bottom width of a trapezoidal channel, in feet.
S=slope of channel, in feet per foot.

King's table 95 (111) contains values of numbers to the 8/3 power which facilitate the preceding computation.

Then in table 97 (113), for a trapezoidal channel, find the value of $\frac{d_n}{b}$ (King's Handbook uses D for d_n) for the computed value of K' and the given channel side slopes.

The depth of flow is then b multiplied by the tabular value of $\frac{d_n}{b}$.

Example 8

Given: A trapezoidal channel; bottom width 4 ft.; side slopes 2:1; grade 1 percent; $n=0.03$; discharge 100 c.f.s.

Find: Depth and mean velocity.

Solution:

1. Compute:

$$K'=\frac{Qn}{b^{8/3}S^{1/2}}=\frac{100(0.03)}{(4)^{8/3}(0.01)^{1/2}}=\frac{3}{(40.3)(0.1)}=0.74$$

2. In King's table 97 (113) for $K'=0.74$ and side slopes $2{:}1, \frac{d_n}{b}=0.52$. Then $d_n=0.52(4)=2.1$ ft.

3. The mean velocity is found from equation (4), $Q=AV$, after computing the area which is

$$A=d(b+zd)=2.1[4+2(2.1)]=17.2 \text{ sq. ft.}$$

$$V=\frac{Q}{A}=\frac{100}{17.2}=5.8 \text{ f.p.s.}$$

4.6–3 Significance of the roughness coefficient (Manning *n*). Table 2 (app. A) lists n values for channels with various degrees of roughness. A value about midway in the range of values is ordinarily used for design. When new, the carrying capacity of the channel will be greater than that for which the channel is designed. This is ordinarily good, but when the design velocities are near the permissible velocity for the lining of the channel, damage might occur before the channel reaches the design condition. This is illustrated in example 9.

Example 9

Given: A trapezoidal channel in easily eroded soil; bottom width 4 ft.; side slopes 4:1; $d=1.0$ ft.; grade 2 percent; to be lined with Bermuda grass, kept mowed to 2 in.; $n=0.035$ (table 2).

Find: Adequacy of channel lining if seeding is used to establish the grass.

Solution: As designed, the channel would carry 36 c.f.s. at a velocity of 4.5 f.p.s. If the channel received the design flow (36 c.f.s.) before the grass had taken hold (when $n=0.02$), the velocity would be 6.7 f.p.s. The channel would probably be damaged. (See table 4.) If the risk of a damaging discharge occurring before the grass is established is great, sod or temporary protection should probably be used. (See sec. 5.4.)

Another type of problem is the determination of the more economical of two suitable linings of different roughness.

Example 10

Given: A trapezoidal channel in easily eroded soil; bottom width 3 ft.; side slopes 3:1; grade 2 percent; and design discharge 40 c.f.s.

Find: Which is the more economical lining for the channel—concrete ($n=0.015$) or stone ($n=0.030$)?

Solution: Using the tables of reference 26 or King's tables (see 4.6–2), the comparative channel characteristics (without freeboard) are:

	Concrete	*Stone*
A (area)	4. 32	7. 00
d (depth)	. 8	1. 1
Q (discharge $=VA$)	40. 0	38. 5
V (velocity)	9. 3	5. 5

With the required cross sections for the two linings computed, allowance for freeboard is added and the cost of excavation and lining is computed for each type, thus determining the more economical channel lining.

A third type of problem (example 11) involves using a rougher channel lining to decrease the velocity from supercritical to subcritical.

Example 11

Given: A trapezoidal channel paralleling the highway; bottom width 4 ft.; side slopes 3:1; discharge 225 c.f.s., and grade 1.0 percent. At the lower end of the channel, the flow must be turned and passed under the roadway.

Find: A suitable type of channel lining.

Solution:

1. For a concrete-lined channel ($n=0.015$) the required data from p. 201 of reference 26 are:

$A=20.0$ sq. ft.
$d=2.0$ ft.
$R=1.2$ ft.
$T=16.0$ ft.

and from p. 370 of reference 26, the velocity for a hydraulic radius of 1.2 ft. and slope 0.01 is 11.2 f.p.s. The discharge is computed to be 224 c.f.s. which checks the design discharge. However, water flowing at 11.2 f.p.s. is supercritical, since from equation (19) $V_c=\sqrt{(32.2)\ \frac{20.0}{16.0}}=6.3$ f.p.s. Supercritical flow will be difficult to turn.

2. If the channel is lined with grass ($n=0.040$), the required data from reference 26 are:

$A=41.3$ sq. ft.
$d=3.1$ ft.
$Q=223$ c.f.s.
$R=1.75$ ft.
$T=22.6$ ft.
$V=5.4$ f.p s

The grass-lined channel requires a much larger cross section, but the water flowing at 5.4 f.p.s. is subcritical ($V_c=7.7$ f.p.s.) and will be much easier to handle than the same quantity of water flowing at supercritical velocity of over 11 f.p.s. in the concrete-lined channel.

4.7 Channel protection. Maximum permissible velocities for channels in various soil types are given in table 3, appendix A. If the mean velocity at the design flow exceeds the permissible velocity for the particular soil type, the channel should be protected from erosion. Channel protection can be provided by linings of grass, concrete, bituminous material, stone, fiber glass, or a preformed material such as metal or wood fiber impregnated with pitch. Generally, the lowest cost (including maintenance cost) lining that provides adequate protection should be used. The type of channel lining might vary along the length of the channel, using a low-cost lining such as grass on the flatter slopes and a high-cost lining such as concrete on the steeper slopes. The capacity of the channel varies with the roughness (n value) of the lining, thus the channel dimensions must often be changed when the channel lining is changed.

Research on vegetative linings (*35, 36*) by the Soil Conservation Service and others has demonstrated the value of grass linings for drainage channels in regions where grass can be grown. Minor erosion damage to grass linings often "heals" itself where rigid-type linings progressively deteriorate unless repaired. Maximum permissible velocities for channels lined with various vegetal covers are given in table 4, appendix A.

Grass linings are particularly suitable for use in combination with other types of paving. Figure 15 shows one

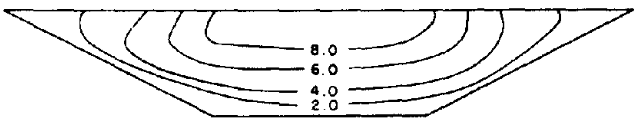


Figure 15.—Approximate distribution of velocities in a straight trapezoidal channel.

reason why this is true. Velocities in a straight, uniform channel are generally greatest in the upper part of the middle portion. Velocities decrease toward the channel sides and bottom approximating the distribution shown in figure 15. Although the mean velocity might exceed the permissible value for a grass lining and thus require a higher cost lining, the mean velocity in the triangular section embracing the upper edge of the bank slope might be low enough for grass. The most economical solution would probably be the combination of a rigid-type lining in the lowest part of the channel and grass lining on the upper bank slopes.

Combination linings are also used where the channel bottom requires protection which could be furnished by a grass lining, but low flows of long duration, from snow melt or seepage, retard the growth of grass. In such a situation, the channel could be paved with a rigid-type lining to carry the low flow and with grass above the elevation of the continued low flow.

If grass is to be seeded, a strip of grass sodding should be placed along the edge of rigid-type linings (at the time of construction) to prevent undermining of the grass lining.

4.8 Grass-lined channels. The method presented in this publication for the design of grass-lined channels is based principally upon the experiments of the Soil Conservation Service (*35, 36*). The Manning equation can be used to determine the capacity of a grass-lined channel, but the value of n varies with the type of grass, the development of the grass cover, the depth of flow, and the velocity of flow.

The permissible velocity in a grass-lined channel depends upon the type of grass, the condition of the grass cover, the texture of the soil comprising the channel bed, the channel slope, and to some extent upon the size and and shape of the drainage channel. To guard against overtopping, the channel capacity should be computed for taller grass than is expected to be maintained, while the velocity used to check the adequacy of the protection should be computed assuming a lower grass height which will likely be maintained.

The variable value of n complicates the solution of the Manning equation. The depth and velocity of flow must be estimated and the Manning equation solved using the n value (table 2, app. A) which corresponds to the estimated depth and velocity. The trial solution provides better estimates of the depth and velocity for a new value of n and the equation is again solved. The procedure is repeated until a depth is found that carries the design discharge.

Three methods will be explained for designing a grass-lined channel. The first method (example 12) uses table 2 to obtain a trial value of n for solving the Manning equation. The second method (example 13) follows more closely the method used by the Soil Conservation Service (*36*) and can be used for values outside the range of table. The third method (example 14) provides a direct solution through the use of channel charts (*25*). The third method is preferred if a chart is available for the type of channel to be used. Construction of channel charts for grass-lined channels is explained in appendix B of reference 25.

Example 12

Given: A trapezoidal channel; bottom width 4 ft.; side slopes of 4:1; grade 1 percent; lined with Bermuda grass kept mowed to a height of 2 to 4 inches; in easily erodible soil.

Find: Depth required to carry the design flow of 100 c.f.s. and the adequacy of the grass lining.

Solution:

1. Assume a depth of 2 ft. and a velocity of 4 f.p.s.

2. From section IV of table 2, appendix A, the value of n for tall grass is about 0.04. The increased depth (2 as compared with 1.5 ft.) compensates for the lesser velocity of flow (5 as compared with 6 ft. per sec.). For 2-inch grass, the value of n is about 0.035. The solution of the Manning equation is the same as for an unlined ditch.

3. The following data are taken from p. 241 of reference 26 (King's tables, sec. 4.6–2 could be used):

$$n=0.04$$
$$A=24.0 \text{ sq. ft.}$$
$$d=2.0 \text{ ft.}$$
$$R=1.17 \text{ ft.}$$
$$T=20.0 \text{ ft.}$$

and from p. 450 of reference 26

$$V=4.1 \text{ f.p.s.}$$

Then $Q=98$ c.f.s. which nearly equals the design discharge.

4. For computing the adequacy of the lining ($n=0.035$), the velocity is 4.6 f.p.s. and d is 1.9 ft.

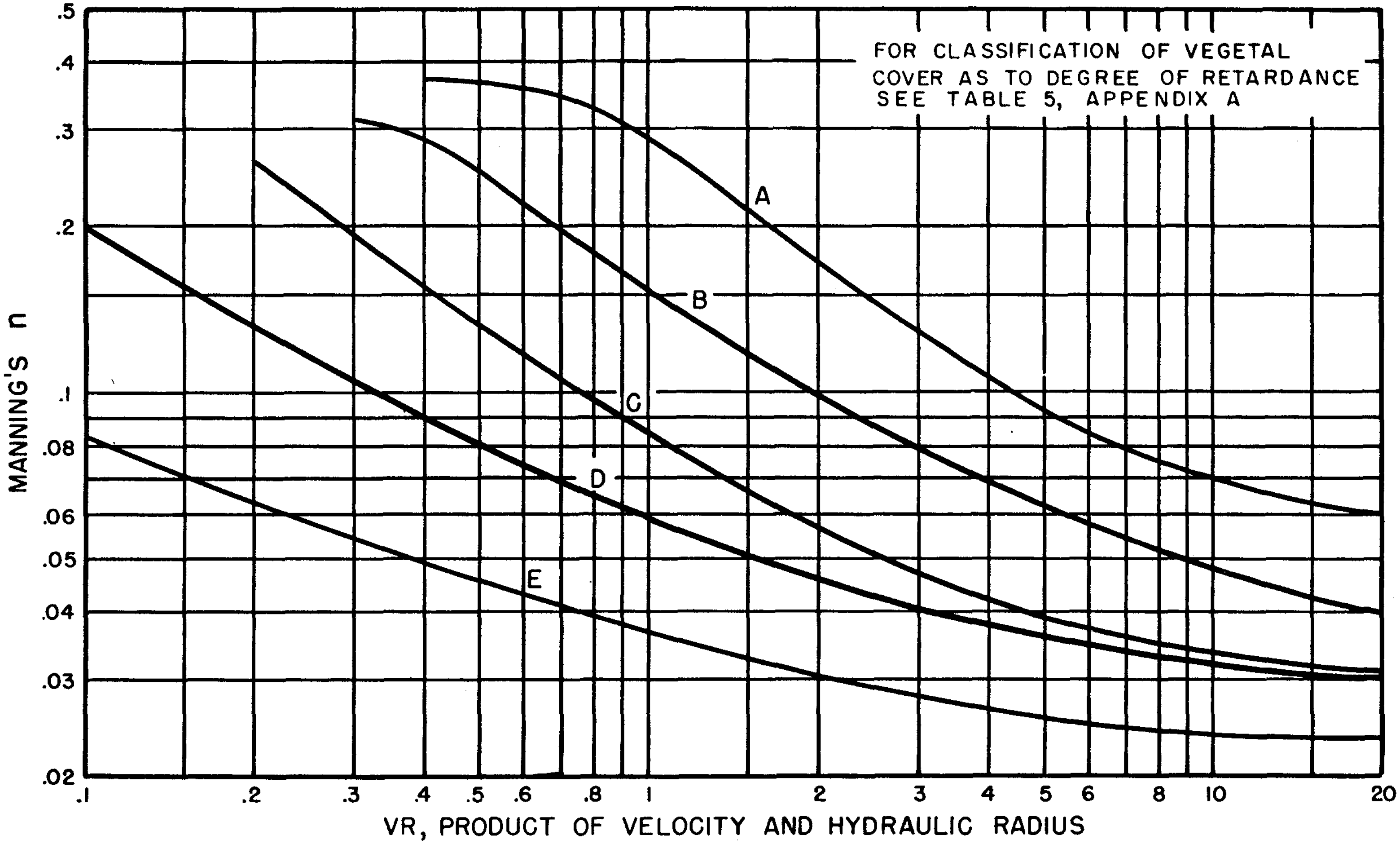


Figure 16.—Manning* n *for vegetal-lined channels (from handbook of channel design for soil and water conservation SCS–TP–61 revised June 1964).

Table 4, appendix A, gives the allowable velocity as 6 f.p.s. which exceeds the computed value. The Bermuda grass will provide adequate protection.

A more general solution for example 12 for values that lie outside the range of table 2 is: First classify the retardance of the grass cover from the information given in table 5, appendix A. Then from the product of the estimated velocity and the estimated hydraulic radius obtain the estimated values and repeat the procedure until a depth is found that carries the design discharge. See example 13.

Example 13

Given: The same data as in example 12.

Find: Depth and adequacy of grass lining.

Solution:

1. In the example, the retardance in Class C (table 5), using slightly taller grass than 4 inches, and the hydraulic radius is 1.17 ft. for an assumed depth of 2 ft. The assumed velocity is 5 f.p.s.

2. From figure 16, the Manning *n* for a VR product of 5.85 is about 0.04 (.035 for Class D, 2.5-inch grass). It will be noted that the value of *n* could have been read to greater refinement, but this is sufficiently accurate for practical problems.

3. The problem now becomes the same as that illustrated in example 12 and the 2-ft. depth (before allowing for freeboard) is satisfactory. The Bermuda grass provides adequate protection.

Example 14

Given: The same data as in example 12.

Find: Depth and adequacy of grass lining.

Solution:

1. Channel charts (Nos. 30 to 33, ref. *25*) have been prepared for trapezoidal channels with bottom width 4 feet and various side slopes, having retardance classifications, C and D. Charts for median swales are also supplied (No. 34, ref. *25*).

2. To use the channel charts, first find the retardance classification in table 5, appendix A, or table 5, appendix A, of reference 25, and select the appropriate chart in reference 25 for the bottom width, side slope, and retardance classification. Enter the chart with the design discharge and channel slopes and read the depth and velocity from the graph.

3. In this example, the retardance classification for capacity is *C* (table 5); and the appropriate chart is No. 31 (fig. 17). Entering the bottom graph of figure 17 with $Q=100$ c.f.s. and $S=0.01$, the depth is 2 ft. and the velocity is 4.1 f.p.s. For allowable velocity (retardance D), the depth is 1.9 ft. and the velocity is 4.6 f.p.s.

4.9 Concrete-lined channels. Concrete linings are generally used for protection against erosion on steep slopes; however, they are sometimes used on very flat slopes to increase the velocity of flow to a nonsilting velocity, to more efficiently remove water from ponded areas, or to reduce the size of channel needed to carry the design discharge. The capacity of concrete-lined channels can be computed as described for channels in sections 3.2–2 and 4.6. Values of the Manning coefficient for concrete linings are given in table 2, appendix A.

For economy of construction, the side slopes of concrete-lined trapezoidal sections should not be so steep (no steeper than 1½:1 and preferably no steeper than 2:1) that surface forms would be needed. Other construction details are discussed in section 5.5.

Velocities in concrete-lined channels on the steeper longitudinal grades are usually supercritical. At high velocities air entrainment occurs. This produces a bulking effect which increases the depth of flow. Air entrainment also causes a reduction in channel friction with a resulting increase in velocity over that computed using the Manning *n* from table 2. An *n* of about 0.008 is recommended (*37*, p. 289) for computing velocity and specific energy in concrete-lined channels carrying supercritical flow.

To allow for the bulking effect of the entrained air, the depth of channel required to carry the air-water mixture must be greater than that computed by Manning's equation using the *n* value from table 2. One method (*37*, p. 289) is to use an *n* of 0.018 for computing normal depth. Another method for computing the depth of the air-water mixture in a rectangular channel is adapted from a formula given on page 547 of reference 22 in which:

$$d=0.11(Q/B)^{2/3} \qquad (28)$$

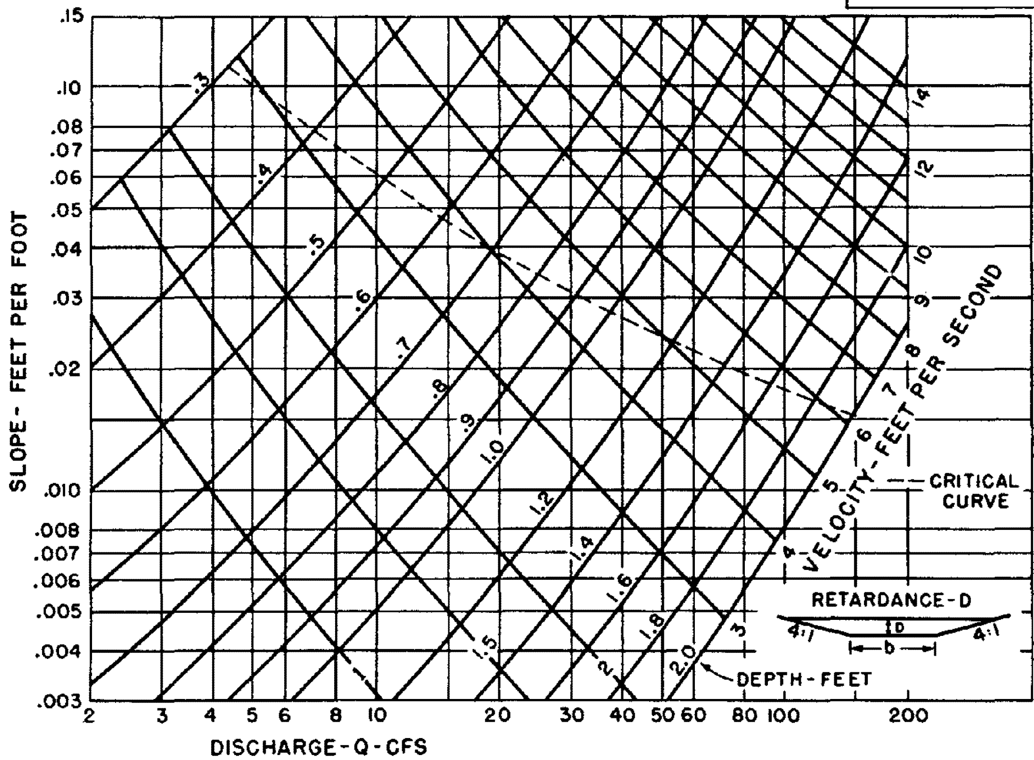
Both methods of allowing for air entrainment are rough approximations and do not include an allowance for freeboard.

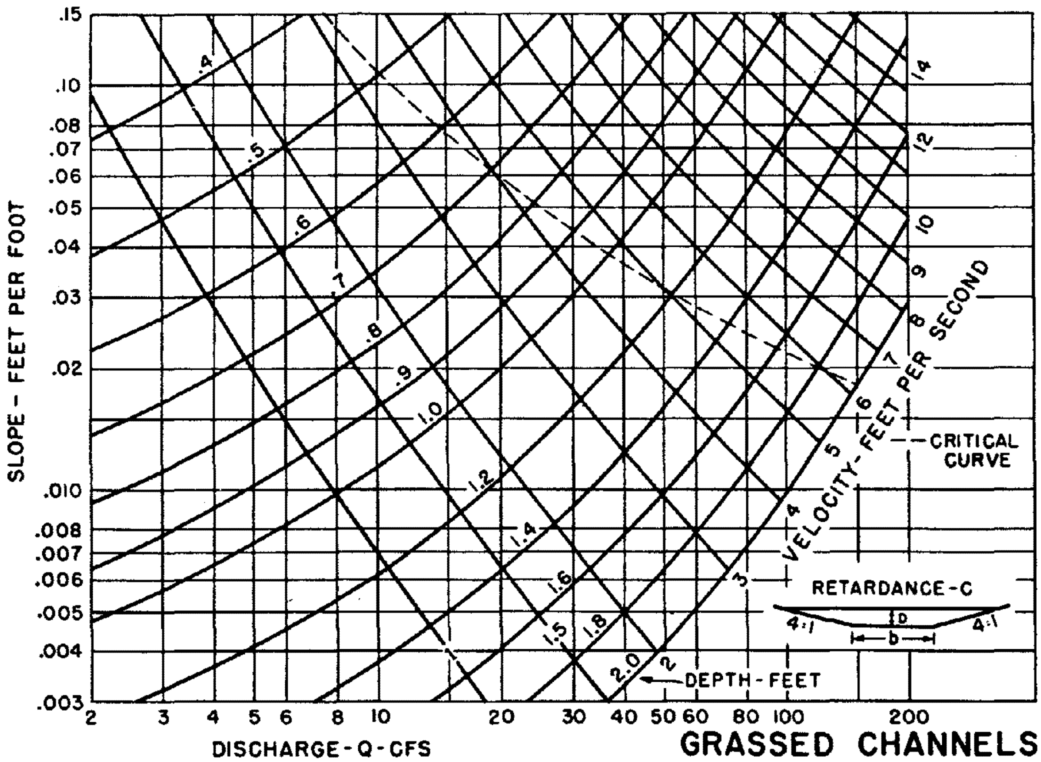
The effect of the high velocity flow at the channel exit must be considered and some provision made to dissipate the excess energy. Otherwise, erosion might occur at the channel outlet, resulting in damage to embankment slopes or in undermining of the channel outlet. Design of stilling basins and energy dissipators is discussed in references 38 and 39.

High-velocity flow can damage the lining itself if projections of the lining occur as a result of faulty design or construction or through unequal settlement of the supporting soil. Offsets at construction or expansion joints cause negative pressures beneath the joint and loss of the supporting soil. To avoid offsets, transverse joints should be made so that the upstream edge of the lower slab cannot heave without moving the downstream edge of the upper slab a like amount. The edge of the lower slab should be constructed about one-half inch lower than the edge of the upper slab. Keyed joints are unsatisfactory because the abutting slabs are subject to differential movement which might result in high stresses on the keys or keyways and cause them to spall. An unkeyed joint with slip dowels is the preferred type for large concrete channels (*37*, p. 340).

Concrete-lined channels built on steep slopes should be anchored to the subgrade by cutoff walls at both the upper and lower ends of the channel. On channels longer than about 50 feet, intermediate anchor walls may be necessary. The cutoff wall also minimizes erosion underneath the lining. The absence of a cutoff wall at the outlet often results in the loss of part of the concrete lining. The cutoff wall at the outlet is particularly necessary where a hydraulic jump occurs. Cutoffs on pipe chutes usually take the form of collars bolted to the pipe.

CHART 31





GRASSED CHANNELS
4:1 b = 4 ft.

Figure 17.—Open-channel chart for grass-lined channel (from design charts for channel flow, hydraulic design series No. 3).

Another frequent cause of failure of concrete-lined channels is overtopping of the channel when the freeboard is insufficient to contain the waves generated by disturbance of the high-velocity flow. (See sec. 4.15.)

4.9-1 Buoyancy of empty channels. In saturated soils, empty channels with rigid linings may float or break up because of the uplift water pressure. The total upward force is equal to the weight of the water displaced by the channel or 62.4 times the volume in cubic feet of the portion of channel cross section which lies below the water table.

The uplift pressure is resisted by the total weight of the lining. When the weight of the lining is less than the uplift pressure, the channel is unstable in saturated soils. The lining should then be increased in thickness to add additional weight or if the flow is subcritical, weep holes may be placed at intervals in the channel bottom to relieve the upward water pressure in the channel. The diameter of weep holes varies from 2 inches to 4 inches and the spacing depends upon soil conditions. When flow is supercritical, subdrainage should be used rather than weep holes to reduce uplift pressure.

A companion problem in northern regions is frost heave which is probably the greatest factor in the destruction of concrete linings in regions with subfreezing temperatures. Unless the subgrade is free draining, subdrainage might be required. All rigid linings require a firm, well-compacted subgrade. (See sec. 5.5.)

4.10 Channels with combination linings. At some locations grass alone does not provide sufficient protection to the channel because of high-velocity flow or the inability of grass to become established in the bottom of the channel. The best solution for some of these locations might be a combination channel where the lower part of the channel cross section is lined with stone or a rigid type of lining and the upper part of the channel cross section is grassed.

Another application of combination channels might be on slight slopes where a smooth rigid lining is used to maintain nonsilting velocities at low flows and grass provides protection to the upper part of the channel.

The top of a rigid lining should be protected by a strip of sod at the time of construction and the edge of the lining finished as described in section 5.5. The design of channels with combination linings is illustrated by example 15.

Example 15

Given: A trapezoidal channel; bottom width 4 ft.; side slopes 3:1; grade 0.5 percent; in sandy loam; carrying flood water containing fine silts and a spring flow which reaches 30 c.f.s.

Find: A suitable design for the design flood of 170 c.f.s. (including spring flow).

Solution:

A. *Unlined Channel*

1. Table 3 gives a permissible velocity of 2.5 f.p.s. for an unprotected channel. The n value of the unprotected channel (table 2) is 0.020. The channel will be designed using King's tables. (See sec. 4.6-3.) The tables referred to in the computations following are from section 7, reference 28, but the corresponding table in chapter 7 of the 3d edition of King's Handbook is given in parentheses. Compute K' from equation (27), section 4.6-2.

$$K'=\frac{Qn}{b^{8/3}S^{1/2}}=\frac{170(0.020)}{(4)^{8/3}(0.005)^{1/2}}=1.19$$

King's table 95 (111) contains 8/3 power of numbers and his table 92 (108) contains 1/2 power of decimal numbers.

2. In King's table 97 (113), for $K'=1.19$ and side slopes 3:1, $\frac{d_n}{b}=0.59$. Then $d_n=0.59(4)=2.4$ ft.

3. The velocity is computed from formula (4), $Q=AV$ where A for a trapezoidal channel$=d(b+zd)=2.4[4+3(2.4)]=26.9$ sq. ft. Then

$$V=\frac{170}{26.9}=6.3 \text{ f.p.s.}$$

A channel 2.4 ft. deep (2.7 ft. with freeboard) will carry the design discharge, but the velocity of 6.3 f.p.s. exceeds that allowable. Therefore the channel should be protected.

B. *Concrete-Lined Channel*

1. For a concrete-lined channel ($n=0.015$). The computations are similar to part A of this example.

2. $K'=\frac{170(0.015)}{(4)^{8/3}(0.005)^{1/2}}=0.89$ or

$$K'_B=K'_A\frac{n_B}{n_A}=1.19\,\frac{(0.015)}{(0.020)}=0.89$$

3. $\frac{d_n}{b}=0.52$ and $d_n=(4)(0.52)=2.1$ ft.

4. $A=2.1[4+3(2.1)]=21.6$ sq. ft.

$$V=\frac{170}{21.6}=7.9 \text{ f.p.s.}$$

5. The Froude number (sec. 3.4) of the flow is 0.96, which indicates that the flow is in the unstable range just below critical flow. The flow should be considered as supercritical in allowing for freeboard or in attempting to change the channel alinement.

C. *Combination Lining*

1. A more economical lining than that of part B might be a combination lining with a concrete-lined channel designed to carry the spring flow of 30 c.f.s. and with the channel slopes above the concrete lined with Bermuda grass mowed to 2 inches. This design will have the cross section shown in figure 18. The depth (without freeboard) of the concrete-lined channel is first determined.

2. $K'=\frac{30(0.015)}{(4)^{8/3}(0.005)^{1/2}}=0.16$

3. $\frac{d_n}{b}=0.22 \qquad d_n=(4)(0.22)=0.9$ ft.

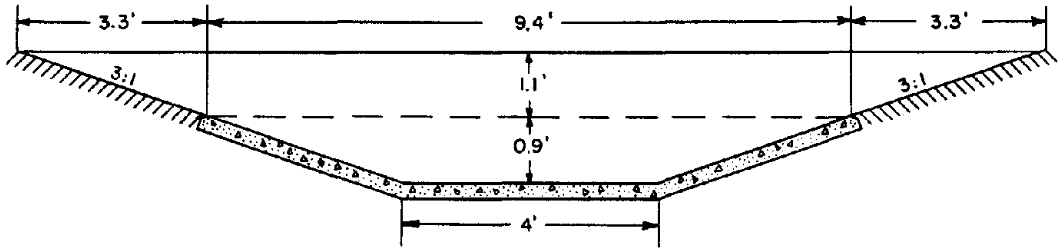


Figure 18.—Trapezoidal channel with combination lining.

4. Area $=0.9[4+3(0.9)]=6.0$ sq. ft.

$$V=\frac{30}{6.0}=5.0 \text{ f.p.s.}$$

Top width $=b+2zd=4+(2)(3)(0.9)=9.4$ ft.

A concrete-lined channel 0.9 ft. deep will satisfactorily carry the design flow of 30 c.f.s.

The next step is to estimate the total depth of the channel. As a guide, a rectangular channel of top width 9.4 ft. can be estimated to carry the flow. The velocity will be somewhat higher than the 5.0 f.p.s. velocity in the 0.9 ft. depth of the concrete-lined section, say 9 f.p.s. The discharge per foot of depth is 9×9.4 or 85 c.f.s. The trial depth is then 170/85 or 2 ft. (1.1 ft. above concrete lining).

For a trial depth of 2 ft. the computation for the section above the concrete lining is:

$A=6.0+(9.4\times1.1)=16.4$ sq. ft.

$WP=9.7$ ft.

$$R=\frac{A}{WP}=\frac{16.4}{9.7}=1.69$$

$V=10.0$ f.p.s. (from nomograph, app. B for $S=0.005$, $n=0.015$, and $R=1.69$)

$Q=16.4(10.0)=164$ c.f.s.

The water carried in one of the triangular sections above the grass would be computed as follows:

$$A=\frac{1}{2}(3.3\times1.1)=1.81 \text{ sq. ft.}$$

$$WP=\sqrt{(3.3)^2+(1.1)^2}=3.5 \text{ ft.}$$

$$R=\frac{1.81}{3.5}=0.52$$

The value of n from table 2 is 0.050 for a depth of between 0.7 and 1.5 ft. and estimated velocity of 2 f.p.s. Then $V=1.25$ f.p.s. (app. B) and $Q=1.25(1.81)=2.3$ c.f.s.

The water carried by the two triangular grassed sections will be 4.6 c.f.s. and the total capacity of the combination ditch (without freeboard) is 164+4.6, or 169 c.f.s., which nearly equals the design discharge.

The small quantity of water (4.6 c.f.s. in this example) and low mean velocities carried in the few feet of channel near each bank is a powerful argument for using combination ditches rather than lining the entire channel section with a rigid lining. In this example concrete paving throughout the slope would have increased the discharge by only 12.5 c.f.s., but would have required an additional 7.0 sq. ft. of concrete lining per linear foot of channel.

4.11 Bituminous linings. Capacity of channels lined with bituminous material can be computed as described in sections 3.2–2 and 4.6. The Manning roughness coefficient (table 2, app. A) for bituminous linings is about the same as that of portland cement concrete linings, thus the channel size and the velocities developed in the channel are about equal for the two types of lining. Channel side slopes for asphalt concrete-lined channels less than 10 feet deep should be no steeper than 1½:1.

Bituminous linings are more flexible than portland cement linings and can more readily adjust to minor subgrade settlement, but the bituminous lining has less strength and weight to resist uplift from frost action or hydrostatic pressure. (See sec. 4.9–1.)

Weeds and other plants are a potential hazard to bituminous linings when conditions are favorable to their growth. Soil sterilization of the lining subgrade is sometimes required.

Additional information on bituminous linings for channels is contained in The Asphalt Institute's Manual No. 12 (*40*).

4.12 Channels lined with stone. Stone channel linings can be constructed (sec. 5.7) of dumped stone, hand-placed stone, or grouted stone. The channel bed and slopes can be lined throughout the area that will be in contact with the design flood or stone can be used in a combination channel with grass or concrete. The size of stone used ranges from gravel size to large stone several feet in diameter. The size of stone needed to protect a particular channel from erosion may be calculated as explained in sections 4.12–1 to 4.12–3. Bank and shore protection is discussed in section 4.16.

A dumped stone lining is the most flexible of the three types and will more readily adjust itself to uneven bank settlement (sec. 7.5). In areas where stone is plentiful, dumped stone is generally the least costly type.

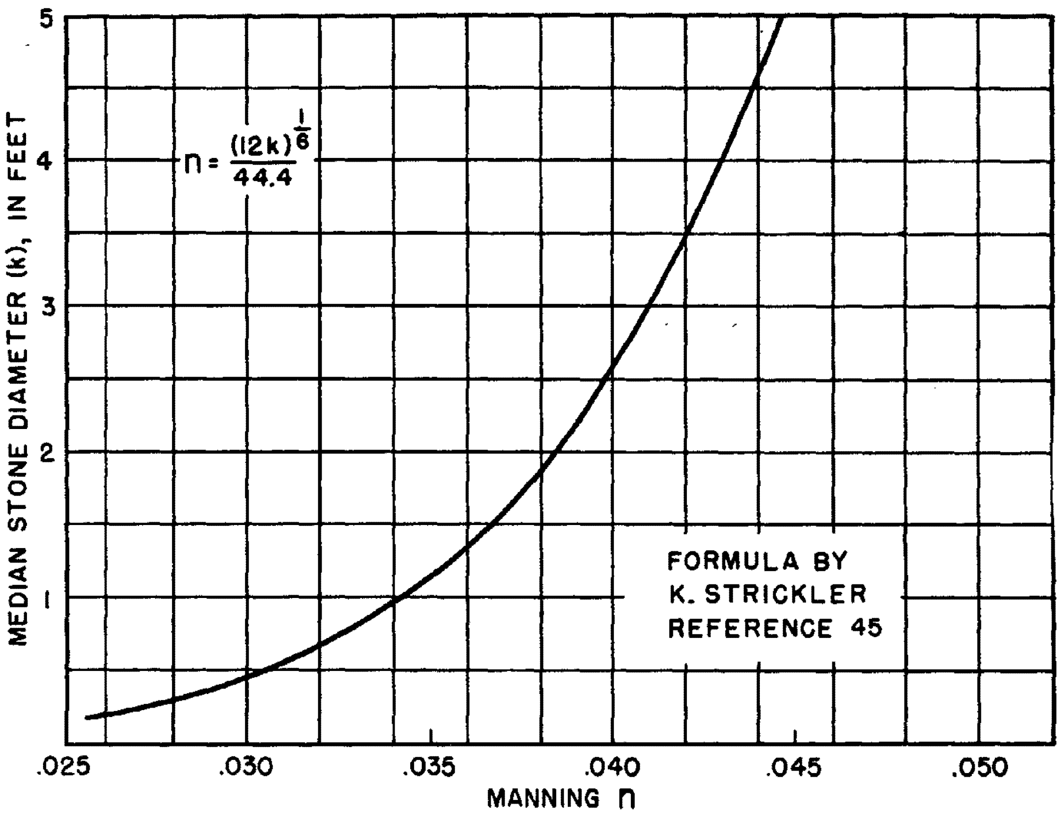


Figure 19.—Relation of Manning n *to size of stone.*

Hand-placed stone linings require less material than dumped stone linings and might be used where the cost of stone is exceptionally high. Hand placement is generally restricted to larger size stone and includes both manually and machine placed stone. (See sec. 5.7.) The finished appearance of the hand-placed lining is somewhat more uniform than dumped stone but its performance is less satisfactory. (See sec. 4.12–2.)

Grouted stone is seldom used except in small channels carrying high-velocity flow or where the available stone is not large enough to withstand the expected velocities. Even here the use of wire mesh baskets (*43*) filled with stone might be preferable to grouted stone.

All types of stone linings should be laid on a filter blanket of gravel or crushed stone unless the gradation of the natural soil is such that it will not filter up through the stone lining. The design of the filter blanket is discussed in section 4.12–4.

The procedures for selection of stone size which follow are based largely upon the report of the Subcommittee on Slope Protection of the Committee on Earth Dams of the Soil Mechanics and Foundations Division, American Society of Civil Engineers (*41*). The size of stone needed for slope protection (see fig. 21) is based on the Ishbash formula. This formula is compared with all available (through 1958) experimental data on Corps of Engineers Chart 712–1, reference 42.

The California Division of Highways discusses the design of stone slope protection (*43*, pp. 110–126) in their bulletin on Bank and Shore Protection. The use of stone sausages for highway fill protection is discussed by Posey (*44*). Reference 39 (p. 209) gives a curve showing the maximum stone size recommended by the Bureau of Reclamation for protection downstream from stilling basins. This curve would lie slightly to the left of the curve for a 1:1 slope in figure 21.

When the cost of stone linings is great, alternate means of channel protection by reducing the velocity should be investigated.

4.12–1 Dumped-stone linings. The resistance of stone to displacement by moving water depends upon—

1. Weight, size, and shape of the individual stones.
2. The gradation of the stone.
3. Depth of water over stone lining.
4. The steepness of the protected slope.
5. The stability and efficiency of the filter blanket and the embankment on which the stone is placed.
6. The velocity of the flowing water against the stone.

The size of stone required is determined by first computing the mean velocity of the water and the size of the channel required to carry the design discharge. Next, a

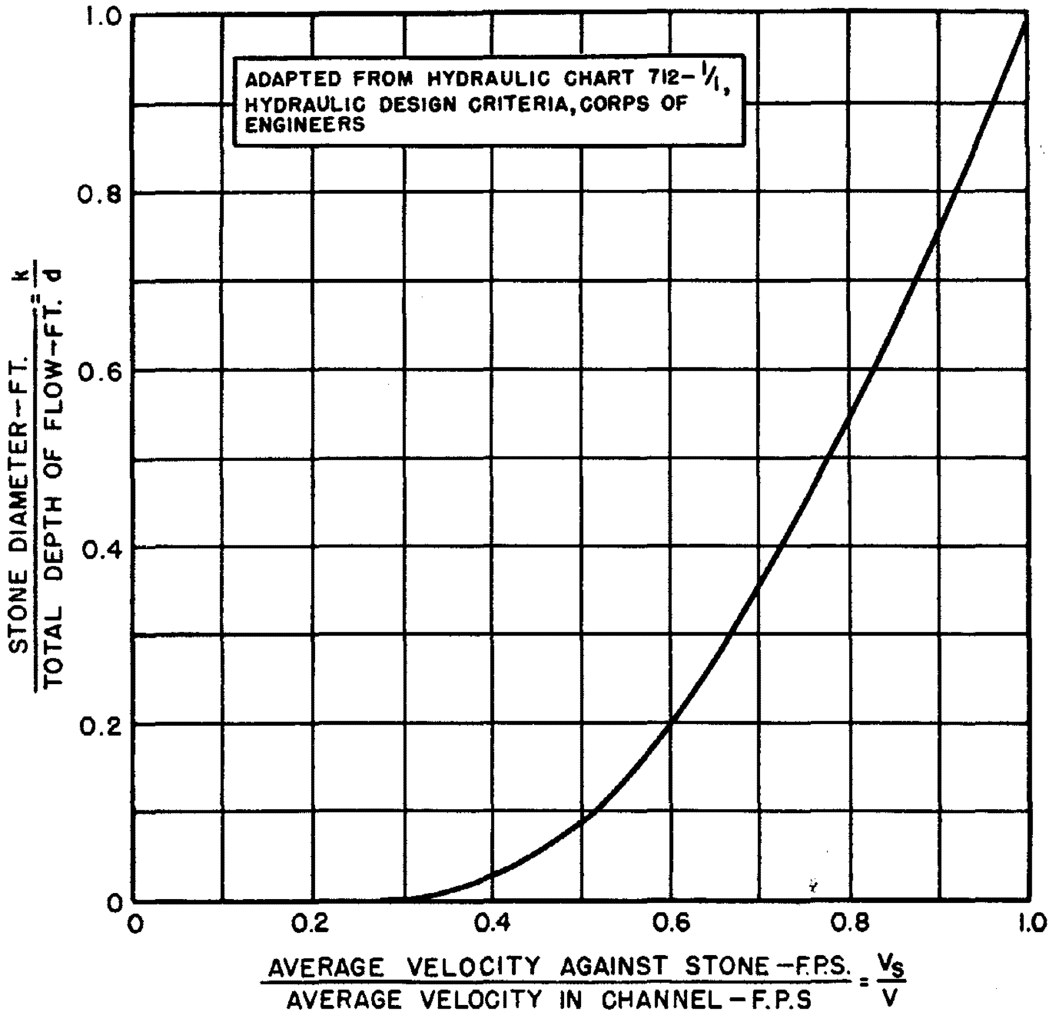


Figure 20.—Average velocity against stone on channel bottom.

size of stone (k) is selected that will withstand the expected velocity. A direct solution of the problem is impracticable, unless charts similar to the charts (Nos. 30–34) for grassed channels in reference 25 are constructed, because the value of the Manning n increases with increase in the size of stone used; this requires a trial and error procedure. (See example 16.) The steps in the procedure are as follows:

1. Select a trial value of n from figure 19 corresponding to the estimated size of stone to be used. Figure 19 applies to a stone lining on both sides and bottom of channel; when only the channel sides are lined, the n value might require weighting when the bottom width exceeds 4 times the depth of flow. The value of the Manning n also varies with the ratio of stone size to the hydraulic radius (*45*). The effect of this variation is generally minor in the determination of stone size.

2. Compute a size of channel, using the Manning equation, that will carry the design discharge.

3. Divide the assumed stone diameter (k), in feet, by the computed depth of flow in the channel (d) to obtain the $\frac{k}{d}$ ratio.

4. Enter figure 20 with this ratio to obtain the V_s/V ratio.

5. Multiply the computed mean value of V by the V_s/V ratio from figure 20 obtain the value of V_s.

6. Enter figure 21 with the value of the V_s and read the stone size, in feet at the intersection of the V_s and the curve corresponding to the channel side slope.

7. If the estimated stone size (step 1) is smaller or much greater than the required size (step 6), select a different size stone and repeat steps 1 through 6, until the estimated size agrees with the required size.

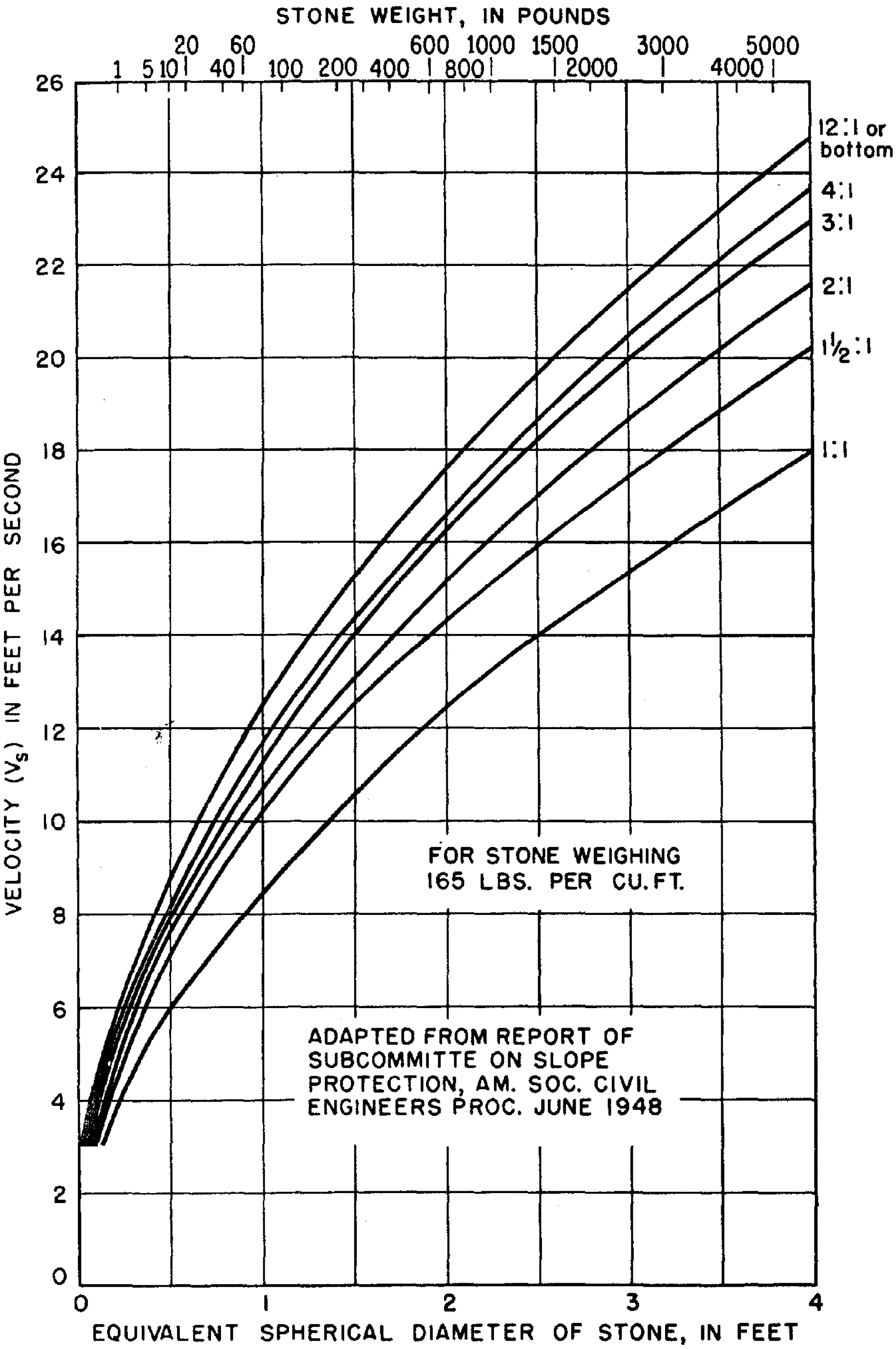


Figure 21.—Size of stone that will resist displacement for various velocities and side slopes.

Determining the size of stone placed within about 50 B of the downstream end of a chute or culvert carrying high-velocity flow requires a modification of steps 4 and 5 of the foregoing procedure. The average velocity at the end of the chute should be used to enter figure 21 rather than V_s from figure 20 which applies to fully developed boundary flow.

The size of stone required to resist displacement from direct impingement of the current as might occur with a sharp change in alinement is probably greater than the value obtained from figure 21, although research data are lacking on just how much larger the stone should be. The California Division of Highways (*43*) recommends doubling the velocity against the stone as determined for straight alinement (step 5) before entering figure 21 for stone size. Lane (*45*) recommends reducing the allowable velocity by 22 percent for very sinuous channels, or for determining stone size the velocity is increased. The factor applied to the velocity V_s for increasing the stone size at the point of impingement would vary from 1 to 2 depending upon the severity of the attack by the current. The area where the larger stone is placed should be determined by the conditions at each site. It is obvious that protection is not required on a bank where deposition occurs.

The size of stone (k) taken from figure 21 is for stone with a unit weight of 165 pounds per cubic foot. The stone size (k_w) for stone of other weights can be computed from Creager's equation (discussion of reference 41):

$$k_w=\frac{102.5k}{w-62.5} \qquad (29)$$

The American Society of Civil Engineers Subcommittee (*41*) recommends the following rules as to the gradation of the stone:

1. Stone equal to or larger than the theoretical size from figure 21 with a few larger stones, up to about twice the weight of the theoretical size tolerated for reasons of economy in the utilization of the quarried rock, should make up 50 percent of the rock by weight.
2. The gradation of the lower 50 percent should be selected to satisfy the filter requirements (sec. 4–12–4) between the stone and the upper layer of the filter blanket.
3. The depth of the stone should accommodate the theoretically sized stone with a tolerance in surface elevations designed to permit the oversized stones mentioned in rule 1. (This requires a tolerance of about 30 percent of the thickness of the stone.)
4. Within the preceding limitations, the gradation from largest to smallest sizes should be quarry run. (See fig. 24 for good gradation curve.)

Tests made at the U.S. Army Engineer Waterways Experiment Station (*46*) show the importance of gradation as compared with maximum size of stone. Two gradations failed under the same conditions although one gradation had maximum pieces 36 inches in equivalent diameter (2,300 pounds), as opposed to 24 inches (700 pounds) for the other gradation. The 50-percent size of each gradation was 16 inches (200 pounds). Two other gradations failed under the same conditions, one with maximum pieces 24 inches (700 pounds) in equivalent diameter and the other 16 inches (200 pounds) in equivalent diameter. However, 75 percent of each of these gradations consisted of stone 10 inches (50 pounds) in equivalent diameter or smaller. In the model tests, the large pieces were dislodged by undercutting resulting from the removal of the smaller pieces. Murphy and Grace (*46*) concluded that pieces of stone larger than those which represented some critical size (the 60- to 65-percent size in these tests) do not increase the effectiveness of the particular gradation.

Example 16 shows a sample calculation of stone size for a roadside channel. Bank protection of stream channels is briefly discussed in section 4.16 and placing of riprap is discussed in section 5.7.

Example 16

Given: A trapezoidal channel in easily eroded soil; bottom width 4 ft.; side slopes 3:1; grade 2 percent; and design discharge 75 c.f.s.

Find: Depth of channel and design a dumped stone lining.

Solution:

1. The channel without lining ($n=0.020$) would have a depth of 1.15 ft. and $V=8.8$ f.p.s. With rougher stone lining, the velocity will be reduced to approximately $\frac{0.02}{0.03}$ $(8.8)=6$ f.p.s. The velocity against the stone (fig. 20) would be about 0.8 (estimating $k=0.5$ ft.) of this, or 5 f.p.s. This velocity (fig. 21) would require stone of about 0.25-ft. diameter. The trial value of n (for $k=0.25$) from figure 19 is 0.028.

2. From p. 201, reference 26, for a trial depth of 1.35 ft., $A=10.9$ sq. ft., $R=0.86$ ft.

On p. 430 for $n=0.030$, $V=6.3$ f.p.s., and for $n=0.028$, $V=\frac{0.030}{0.028}(6.3)=6.8$ f.p.s. Then $Q=10.9\ (6.8)=74$ c.f.s., which checks the design Q.

3. $\frac{k}{d}=\frac{0.25}{1.35}=0.18.$

4. $\frac{V_s}{V}$ ratio from figure 20 is 0.58.

5. $V_s=0.58V$, $V=0.58\ (6.8)=3.9$ f.p.s.

6. In figure 21 for $V_s=3.9$ and slope 3:1, the size of stone is about 0.1 ft. which checks the assumed size of 0.25 ft. (step 1). Use 2-inch stone.

The depth of the channel after lining is 1.35 ft. (without freeboard allowance). The minimum thickness of the linings is 2 inches with a tolerance of about 1 inch in surface elevation. Stone of equivalent spherical diameter, equal to or larger than 2 inches with a few larger stone not exceeding 3 inches, should make up 50 percent of the rock by weight.

4.12–2 Hand-placed stone linings. Hand-placed stone linings consist of stones placed by hand following a more or less definite pattern with the voids in the larger stones filled with smaller stones and the surface kept relatively even. The stone should be placed on a filter blanket of graded gravel or crushed stone. (See sec. 4.12–4.) Enough voids should be left in the surface of the stone lining to properly vent the subsurface. The depth and size of the stone required to resist displacement is generally specified as one-half (½) of that required for dumped stone (*41*, p. 857), but the experience of the Corps of Engineers (*47*) is that a half thickness of hand-placed stone is not satisfactory as a substitute for dumped stone. (See sec. 7.5.) Although data are lacking for a definite conclusion, it is probable that hand-placed stone is inferior to an equal thickness of dumped stone because it is not as flexible as dumped stone and does not have the structural strength to bridge over local bank failures.

4.12–3 Grouted-stone linings. Grouted-stone linings are either hand-placed or dumped-stone linings with the voids filled with portland cement mortar. The stone is placed on a gravel or crushed stone filter blanket and the grout is broomed and rodded into the voids. Some of the holes or joints are left ungrouted in order to avoid uplift from hydrostatic pressure, but care should be exercised to avoid openings that permit escape of the bank or filter material. Grouted stone is seldom used except for lining high-velocity channels, although its use might be justifiable where stones large enough for dumped stone linings are not available or when the stone available is not adaptable to hand placing.

4.12–4 Filter blankets. A filter blanket is often needed beneath the stone lining to prevent the bank material from passing through the voids in the stone blanket and escaping. The loss of bank material leaves cavities behind the stone blanket and a failure of the blanket might result. Whether a filter blanket is needed will depend upon the gradation of the bank material and the openings or voids in the riprap cover. In general, a filter ratio of 5 or less between successive layers will result in a stable condition. The filter ratio (*48*) is defined as the ratio of the 15-percent particle size (D_{15}) of the coarser layer to the 85-percent particle size (D_{85}) of the finer layer. An additional requirement for stability is that the ratio of the 15-percent particle size of the coarse material to the 15-percent particle size of the fine material should exceed 5 and be less than 40. These requirements can be stated as follows:

$$\frac{D_{15}\text{ (of coarser layer)}}{D_{85}\text{ (of finer layer)}}<5<\frac{D_{15}\text{ (of coarser layer)}}{D_{15}\text{ (of finer layer)}}<40$$

If a single layer of filter material will not satisfy the filter requirements, one or more additional layers of filter material should be used. The filter requirement applies between the bank material and the filter blanket; between successive layers of filter blanket material, if more than one layer is used; and between the filter blanket and the stone lining. In addition to the filter requirements, the grain size curves for the various layers should be approximately parallel to minimize the infiltration of the fine material into the coarse material. The filter material should contain not more than 5 percent of material passing the No. 200 sieve.

The thickness of the filter blanket ranges from 6 inches to 15 inches for a single layer, or from 4 inches to 8 inches for individual layers of a multiple layer blanket. The thicker layer is used where the gradation curves of adjacent layers are not approximately parallel.

An example of filter design follows:

Example 17

Given:

Material	Particle size (mm)	
	D_{15}	D_{85}
Riprap ("B" curve, fig. 24)	90	308
Streambank	.006	.10
Sand	.14	2.4
Gravel	4.0	50

Find: Design filter, if needed.

Solution:

1. Is filter required?

$$\frac{D_{15}\text{ (riprap)}}{D_{85}\text{ (streambank)}}=\frac{90}{0.10}=900>5 \qquad \text{Yes}$$

2. Can a single layer of gravel be used?

$$\frac{D_{15}\text{ (riprap)}}{D_{85}\text{ (gravel)}}=\frac{90}{50}=1.8<5 \qquad \text{OK}$$

$$\frac{D_{15}\text{ (gravel)}}{D_{85}\text{ (streambank)}}=\frac{4.0}{0.10}=40>5 \qquad \text{No}$$

3. Can a layer of sand and a layer of gravel be used?

1st requirement:

$$\frac{D_{15}\text{ (sand)}}{D_{85}\text{ (streambank)}}=\frac{0.14}{0.10}=1.4<5 \qquad \text{OK}$$

$$\frac{D_{15}\text{ (gravel)}}{D_{85}\text{ (sand)}}=\frac{4.0}{2.4}=1.7<5 \qquad \text{OK}$$

$$\frac{D_{15}\text{ (riprap)}}{D_{85}\text{ (gravel)}}=\frac{90}{50}=1.8<5 \qquad \text{OK}$$

2d requirement:

$$\frac{D_{15}\text{ (sand)}}{D_{15}\text{ (streambank)}}=\frac{0.14}{0.006}=23<40 \qquad \text{OK}$$

$$\frac{D_{15}\text{ (gravel)}}{D_{15}\text{ (sand)}}=\frac{4.0}{0.14}=29<40 \qquad \text{OK}$$

$$\frac{D_{15}\text{ (riprap)}}{D_{15}\text{ (gravel)}}=\frac{90}{4.0}=22<40 \qquad \text{OK}$$

If the gradation of the sand and the gravel is satisfactory and adequate placing methods are to be used, two minimum thickness layers (4 or 5 inches) can be used, one of sand and one of gravel.

4.13 Ditch checks. In humid areas ditch checks (grade-control structures) are seldom used in the roadway and toe-of-slope channels because they are a hazard to vehicles driving off the road, they are often unsightly, they hamper the use of power mowing equipment, and in most locations their jobs can be done better with protective cover. (See example 18.) In channels not accessible to vehicles or in arid and semiarid regions, ditch checks or drop structures may be used to maintain noneroding velocities in the channel section. Their use, however, should be limited because of cost and vulnerability to damage at times of unusual flooding.

The purpose of ditch checks is to reduce the channel grade to a series of parallel grades which will keep the velocity within allowable limits (table 3) for the material of the channel or for the channel lining. The difference between the allowable grade and the existing grade is taken up in the drop at the ditch checks. The drop at each check in a roadway channel is preferably from 7 to 12 inches. Ditch checks may be constructed of local stone, concrete, timber, or other suitable material. The ditch check should be well anchored in the channel side slopes, have a cutoff wall, and have an apron with a sill or lip at the lower end forming a shallow pool to dissipate the energy of the falling water. The crest of the check is ordinarily built as a trapezoidal weir, the same cross section as the channel, and with the weir crest on level with the bottom of the upper channel. The weir capacity (sec. 3.3–3) should be sufficient to prevent overtopping of the wings which should extend above the design depth of flow by the amount of the freeboard.

The spacing of ditch checks, in feet, is computed from the equation,

$$\text{spacing}=\frac{100\,h}{S-S_a} \tag{30}$$

Where

h=vertical drop, in feet.

S=slope of channel without checks, in percent.

S_a=slope of channel, in percent, for allowable velocity.

The size of the channel is computed for the design discharge and slope of channel as described in section 3.2–2. If the velocity exceeds that allowed (table 3) for the soil type or channel lining, ditch checks may be considered. Reducing the velocity also reduces the channel capacity and thus requires an increase in channel ize. (See example 18.)

Example 18

Given: A trapezoidal channel; bottom width 4 ft.; side slopes 3:1; grade 0.3 percent; and discharge 65 c.f.s. The channel is in firm loam and carries clear water.

Find: Dimensions of a channel to carry the discharge using ditch checks if needed.

Solution:

A. *Unprotected Channel*

1. In appendix A from table 2, n=0.020, and from table 3, the allowable velocity (V_a)=2.5 f.p.s.
2. From reference 26, p. 201 and 378:

d=1.7 ft.	R=1.05 ft.
T=14.2 ft.	V=4.2 f.p.s.
A=15.5 sq. ft.	Q=65 c.f.s.

3. The channel selected has adequate capacity, but the velocity (4.2 f.p.s.) exceeds the allowable velocity (2.5 f.p.s.)

B. *Ditch Checks*

If 6-inch (0.5 ft.) ditch checks are to be used to reduce the velocity to a noneroding velocity, what is the spacing of checks and what channel section will be required with the checks?

1. On p. 387 of reference 26, a velocity of 2.5 f.p.s. requires a slope of 0.10 percent, with n of 0.020 and R of 1.1 ft.

The spacing by equation (30) is:

$$\text{Spacing, in feet}=\frac{100\ (0.5)}{0.3-0.10}=250\text{ ft.}$$

2. The new channel on a slope of 0.10 percent and a velocity of 2.5 f.p.s. will require about 65 c.f.s./2.5 f.p.s. =26 sq. ft. of cross-sectional area, provided R is kept at about 1.1 ft. From p. 243 and 387 of reference 26: for a bottom width of 14 ft. and side slopes 4:1:

Trial 1	Trial 2	Trial 3	Units
d=1.4	d=1.5	d=1.45	ft.
T=25.2	T=26.0	T=25.6	ft.
A=27.4	A=30.0	A=28.7	sq ft.
R=1.07	R=1.14	R=1.10	ft.
V=2.2	V=2.3	V=2.25	f.p.s.
Q=60	Q=69	Q=65	c.f.s.

The data for d=1.45 ft. are interpolated, but are sufficiently accurate to show that this depth channel meets the requirements. Various other combinations of channel section and check spacing could be used.

C. Grass-Lined Channel

The larger channel required with ditch checks often eliminates their use particularly in a humid section when any sod-forming grass would adequately protect the channel (table 3). Changing the drop at the check changes the spacing of the checks but does not affect the size of channel required to carry a stated discharge.

For example, the channel dimensions with a sod-forming grass lining are depth, 2.35 ft.; bottom width, 6 ft.; side slopes 3:1. This channel has about same capacity as the channel with 6-inch checks; depth, 1.45 ft.; bottom width, 14 ft.; and side slopes 4:1. The same freeboard allowance would be added to either design.

4.14 Drop structures. Drop structures are sometimes used to convey water from a higher to a lower elevation. One application of drop structures is conveying water from a drainage channel into a watercourse that is at a lower elevation. The design of drop structures is beyond the scope of this publication but information on their design may be found in references 6, 7, 8, and 49.

4.15 Chutes. Chutes (see sec. 1.5) are characterized by their steep grade and short length. They carry water at supercritical velocity (see sec. 4.9) and unless excavated in rock, they will require lining to prevent channel erosion. Concrete linings are frequently used, often without providing for the high velocities at the outlet. When the

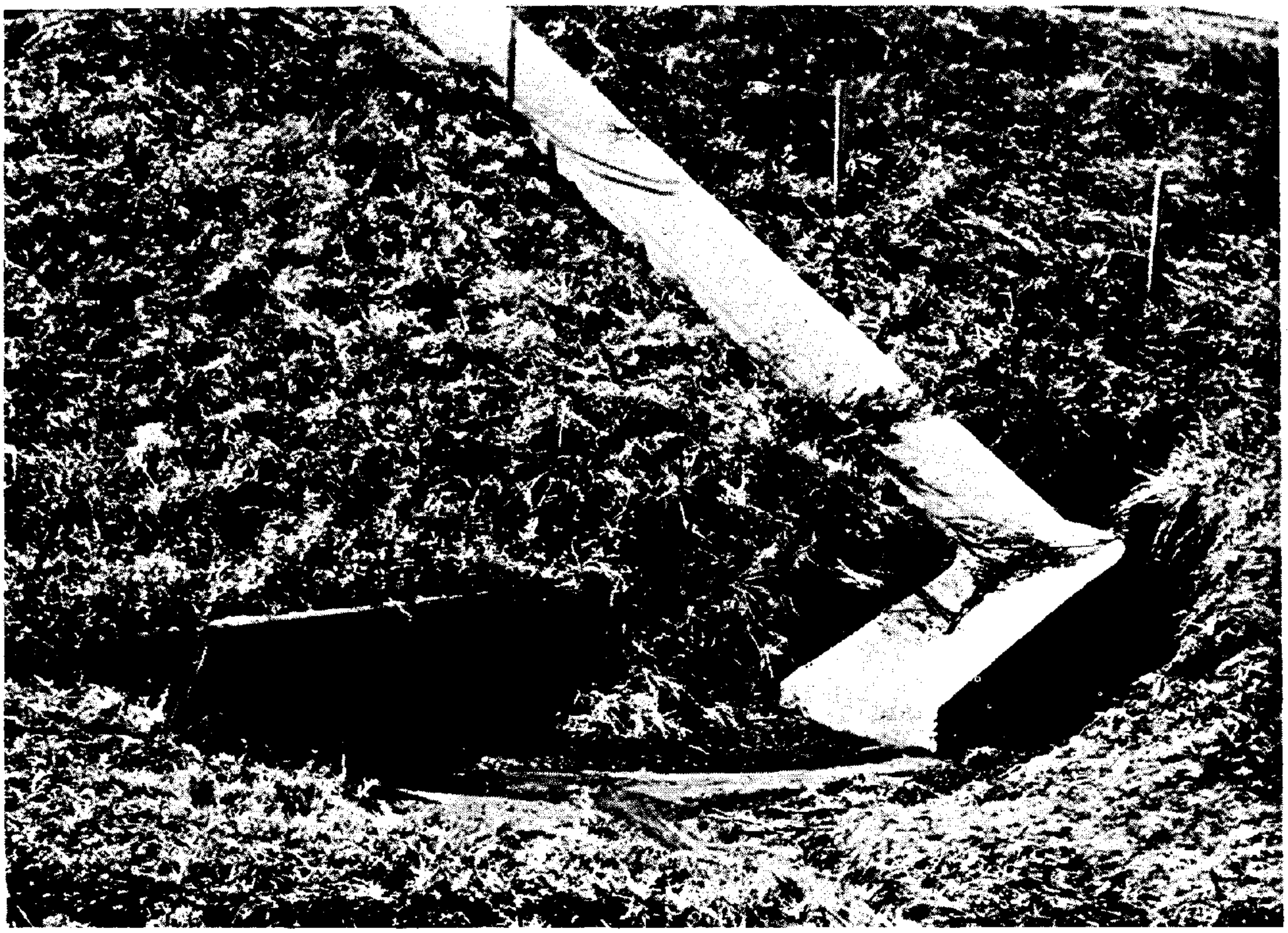


Figure 22.—Chute with sharp change in alinement. Note erosion due to sharp curvature.

quantity of water to be carried is small, sodded flumes may be used; however, the sod must generally be held in place by stakes and chicken wire until the sod has time to become established. Stone linings (sec. 4.12) not only furnish protection from erosion but they also reduce the velocity of flow. For long cut or embankment slopes, pipes are generally preferred to open channels because the flow is confined and the water cannot spill out to cause erosion under the sides. The design of pipe drains and outlets for shoulder drains will be discussed in a later publication.

The channel section after lining must be of sufficient size to carry the design flow, but a section far larger than needed to carry the design flow or required for easy maintenance is wasteful. A common design is a grass-lined channel on a flat slope discharging into a concrete-lined chute of the same cross section. The carrying capacities of the two sections are often in a ratio of more than 10 to 1. In such cases the channel section should be tapered to a section of equivalent capacity when changing type of lining.

The freeboard allowance on an open chute is greater than that used for channels on mild slopes because of the wave action in chute channels and the greater damage caused by the high-velocity water. Freeboard allowances vary, depending upon the amount of wave action and turbulence expected in the chute, but the minimum freeboard should be about 1 foot for small chutes.

Changes in channel alinement or in section must be made smoothly in order to avoid violent wave action and resultant overtopping of the channel sides. Figure 22 shows the effect of too sharp a change in alinement. Large chutes require careful hydraulic design. (See *22*, ch. 8 and *37*, ch. 8.) On small chutes, experiments have shown (*37*, p. 290) that an angular variation of the flow boundaries should not exceed that produced by the equation:

$$\tan \alpha = \frac{1}{3F} \qquad (31)$$

where F is the Froude number (sec. 3.4) and α is the angular variation.

The chute outlet is always a problem. The energy of the high-velocity flow must be dissipated to prevent erosion. Often this requires an apron or a stilling basin at the chute outlet. Design of stilling basins and energy dissipators suitable for this purpose is discussed in reference 39 and in an ASCE symposium by Bradley and Peterka and others (*38*). A bibliography containing other references to spillway design is included with paper 1406 (*38*).

When the velocity in the chute is supercritical, the chute capacity is determined by the cross section and conditions at the inlet (upstream control). If the chute is long, its cross section can often be reduced to maintain economic proportions. (See sec. 3.3–3.)

Equations are given in section 3.3–3 for computing the discharge of chutes with inlet channel in line with the chute. The design of chutes discharging flow from a channel perpendicular to the chute axis is discussed on pages 283–288 of reference 37. The application of these equations to the design of a small concrete chute is explained by example 19.

Example 19

Given: The channel in example 4 (sec. 3.2–2) discharging into a concrete chute, 120 ft. long (horizontally) on a 50-percent grade.

Find: The chute dimensions for a trapezoidal section with side slopes 2:1 ($z=2$) and entrance loss coefficient=1.

Solution:

A. *Entrance*

1. From example 4, at section 1 (see fig. 14),

$$Q=100 \text{ c.f.s.}$$

$$d_1=2.45 \text{ ft.}$$

$$V_1=4.6 \text{ f.p.s.}$$

2. The specific head (H_o) (sec. 3.3–1, equation 9) just upstream from entrance of the chute is $2.45+\frac{(4.6)^2}{2(32.2)}=$ 2.78 ft. Substituting in equation 23 (sec. 3.3–3):

$$b=\frac{0.324(100)}{(2.78)^{3/2}}-0.7(2.0)(2.78)=3.09 \text{ ft., use } 3.0 \text{ ft.}$$

3. The depth at the entrance will be critical, or by equation 16 (sec. 3.3–2).

$$d_c=\frac{\left[\begin{array}{l}4(2)(2.78)-3(3)\\ +\sqrt{16\ (2)^2(2.78)^2+16(2)(2.78)(3)+9(3)^2}\end{array}\right]}{10(2)}$$
$$=2.12 \text{ ft.}$$

4. The mean velocity (V_c) at the entrance section is the discharge, 100 c.f.s. divided by the flow area of the entrance section ($b=3$ ft., and $d=2.12$ ft.), 15.4 sq. ft. or $V_c=6.5$ f.p.s., $R=1.23$ ft. (*26*, p. 161).

The value of d_c and V_c could have been read directly from chart 16, reference 25 by noting the coordinates of the intersection of the critical flow line and the discharge, 100 c.f.s.

B. *Chute*

The minimum depth at which the flow in the chute will occur can be computed by the Manning equation, using King's tables (sec. 4.6–2), or following the method in example 4 (sec. 3.2–2), the final trial is as follows (*26*, p. 161):

$$d=0.7 \text{ ft.} \qquad S=0.5$$

$$A=3.10 \text{ sq. ft.} \qquad S^{1/2}=0.707$$

$$R=0.50 \text{ ft.} \qquad n=0.018 \text{ (sec. 4.9)}$$

$$R^{2/3}=0.63 \text{ ft.}$$

$$V=\frac{1.49}{n}R^{2/3}S^{1/2}=\frac{1.49}{0.018}(0.63)(0.707)=36.9 \text{ f.p.s.}$$

$$Q=3.10(36.9)=114 \text{ c.f.s.}$$

This checks the design discharge 100 c.f.s. A freeboard of at least 1 foot should be provided and a stilling basin or energy dissipator will be needed in most situations to dissipate the energy of the high velocity flow.

C. *Check of Fall in Chute*

Had the n value (0.015) from table 2 been used, the calculated depth would have been 0.6 ft., $R=0.44$ and $V=40.5$. These values are used to determine if the fall in chute (60 ft. in this example) is sufficient to develop the specific head required at normal depth plus the friction loss. Specific head is computed by equation (9).

$$H_0=d+\frac{V^2}{2g}=0.6+\frac{(40.5)^2}{64.4}=26.0 \text{ ft.}$$

The friction loss (h_L) is computed by equation (24).

$$h_L=\frac{n^2}{4.41}\left[\left(\frac{V_1}{R_1^{2/3}}\right)^2+\left(\frac{V_2}{R_2^{2/3}}\right)^2\right]L$$

$$h_L=\frac{(0.015)^2}{4.41}\left[\left(\frac{6.5}{(1.23)^{2/3}}\right)^2+\left(\frac{40.5}{(0.44)^{2/3}}\right)^2\right]120$$

$$h_L=30.3 \text{ ft.}$$

Total head required is $26.0+30.3=56.3$ ft. The available head (60 ft.) is sufficient to develop the specific head plus the friction loss.

Fully developed air-entrained flow on a slope of 0.5 would have developed a velocity of over 80 f.p.s. (using $n=0.008$, sec. 4.9) and would require a head in excess of 100 ft.

Reducing the cross section of this chute will provide little saving. For larger chutes, see section 3.3–3.

4.16 Bank and shore protection. The discussion in sections 4.12 to 4.12–4 on stone channel linings is intended to apply to roadside channels where both banks and streambed are protected by a blanket of stone. At times, however, highways encroach on rivers or are built along the seacoast or beside large bodies of water, thus exposing roadway embankments to attack by current velocity or wave action. Stream crossings are unavoidable, but most crossings expose highway embankments to attack by the stream. Wave action can be a more severe form of attack than water flowing parallel to the embankment. Protection from wave action is discussed in references 37, 41, 43, and 50.

Problems involving scour of streambanks vary for many reasons and standard solutions cannot be specified even for types of problems. Sometimes the best solution is to relocate the highway and avoid the hazard or to move the attacking water away from the embankment by a channel change (sec. 1.10).

Usually the severity of the attack and the availability and cost of materials dictate the type and the extent of protection. Where vegetal cover will grow and velocities of flood waters are moderate, grass (sec. 4.8), shrubs, and vines are low in cost and give adequate protection after they become established. Where high velocities and wave action are encountered, stone provides suitable protection (secs. 4.12 to 4.12–4).

The method given in section 4.12–1 for computing stone size might require modification for deep swift streams When the depth, d, is great the $\frac{k}{d}$ ratio becomes small and figure 20 shows a low velocity against the stone. This results in a size of stone (fig. 21) which while adequate at the total depth, may not provide sufficient protection for the bank near the water surface. The depth used in figure 20 should be the depth of most severe attack rather than the total depth.

The most common cause of failure in protective covers is the undermining of the toe or the termini of the protection. All protective covers of structural materials should be firmly anchored in the protected bank at the upstream and downstream termini, and at the toe of the embankment (sec. 5.7). If the protective cover is long, intermediate anchorages might be required to reduce the hazard of complete failure. The upper vertical limit of the protective cover should extend to an elevation that provides protection from floods and waves having a recurrence interval consistent with the importance of the highway, initial investment, and replacement costs. Sod placed above more rigid protection will provide considerable protection from unusual floods.

The beginning of the protection should extend above and below the point of reverse curvature on the outside of a curved channel. Bank protection is usually not required on the inside of the curve unless return of overbank flow creates a scour problem. On a straight channel, bank protection should begin and end at a stable feature in the bank if practicable. Such features might be outcroppings of erosion-resistant materials, trees, vegetation, or other evidence of stability.

4.17 Drainage structures in channel. Where driveways or access roads cross a drainage channel, culverts or other structures are placed in the channel to carry the flow underneath the crossing. These structures affect the capacity of the drainage channel and at some locations create problems. Silt and debris may be deposited in the channel upstream from structure and scour may occur at the culvert outlet. The design of the culvert (*51*) and the channel should be coordinated to insure the proper functioning of both.

Chapter V.—CONSTRUCTION

5.1 General. Construction methods depend upon the equipment used and the experience of the contractor and are outside the scope of this publication. This chapter will discuss a few of the procedures that should be followed to construct satisfactory highway drainage channels.

Drainage channels should be constructed early in the grading operations and the necessary channel protection should be provided before erosion damage occurs. Effective drainage during construction frequently eliminates costly delays as well as later failures that might result from a saturated subgrade. Slopes should be protected from erosion as early as practicable in order to minimize damage and lessen the discharge of eroded soil into existing and newly constructed channels or into drainage structures.

Elements of farm drainage systems, such as terrace outlets and tile drains, that are cut by the highway should be properly drained to avoid ponding or erosion on adjacent property or on the nighway.

5.2 Supervision. Proper design of drainage channels will not produce an adequate drainage system without careful supervision during construction. Supervision of drainage structure construction means no' only seeing that the construction complies with the plans and specifications but that any omissions in the plans are corrected. The channel drainage work should be shown on the construction plans together with sufficient hydraulic design data, such as drainage area and design discharge, so that the construction engineer has the necessary information to solve unforeseen drainage problems. Some guides for showing drainage information on project plans are given in reference 52.

5.3 Excavation. Channels are usually dug from the outlet toward the higher end so that the channel will drain during construction. Dikes for intercepting channels are preferably built from material excavated from the adjacent cuts without disturbing the natural soil at the channel location.

5.4 Grass-lined channels. Grass lining can best be attained by sodding. The upper parts of the channel may be sprigged or seeded if the cost of sod makes tnis necessary, but the time of the year, and the likelihood of damaging rains occurring before the seedlings become established, should be considered. A type of grass should be selected that is adapted to the locality and to the site conditions (*53*). Sod grasses are preferred to bunch grasses because of their superior performance in resisting erosion.

Seeding can be protected by mulch, temporary cover grasses, jute or fiber bagging, and fiber glass. At some locations, sod strips 18 inches by 10 feet or more can be laid perpendicular to the channel centerline at 10-foot intervals to protect the intervening seeded area. These strips have proved very effective in some locations. Their use is restricted to channels designed for velocities between 2 feet and 4 feet per second.

Sod can be used alone or in combination with seeding. Sod might be used in the channel bottom and up part of the side slope for immediate protection with the remainder of channel slope seeded. In some States sod chutes have been used successfully at the downhill end of bridges to conduct the runoff from the end of the bridge to natural ground level. The sod is held in place by chicken wire and staked to the embankment.

A strip of sod should be placed along the edges of all concrete, riprap, and similar linings, and along the edges of open chutes to prevent erosion and undermining the linings. If sod cannot be maintained as in semiarid or arid regions, adequate freeboard should be provided in the lined channel.

5.5 Concrete-lined channels and chutes. Concrete channel linings can be cast-in-place, shotcrete, or precast. Soil-cement linings have been successful at some locations. The construction of concrete channel linings as applied to irrigation canals is discussed in reference 54. State highway specifications usually give requirements for constructing concrete drainage channels.

Concrete lining must be placed on a firm well-drained foundation to prevent cracking or failure of the lining. Soils of low density should be thoroughly compacted or removed and replaced with suitable material. Where the soil is deep loess, concrete or other type rigid linings might not be suitable. Expansive clays are extremely hazardous to rigid-type linings because their movement buckles the linings as well as producing an unstable support. Reference 54, page 30, discusses methods of controlling or reducing damage to linings which must be placed on expansive clays.

The Bureau of Reclamation found (*37*, p. 535) that when placing an unformed slab on a slope, a tendency exists to use a stiff concrete mix that will not slough. Drill cores showed that placement of such low-slump concrete without thorough vibration usually results in considerable honeycombing on the underside. To avoid such results the concrete should not be stiffer than a 2½-inch slump. Concrete of this consistency will barely stay on a steep slope. After spreading, the concrete should be thoroughly vibrated. Reference 37 (p. 536) shows the use of a steel-faced slipform screed.

The linings of channels that carry high-velocity flow should be poured as nearly monolithic as possible, without expansion joints or weepholes, and using as few construction joints as possible. Construction joints should be made watertight. Longitudinal and transverse reinforcing steel should be used throughout to control cracking with

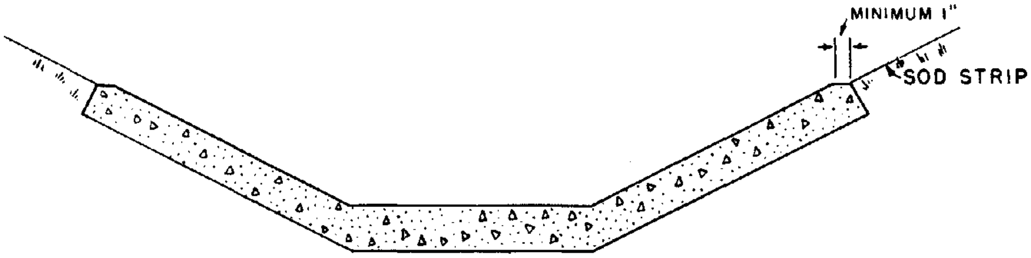


Figure 23.—Section of concrete-lined channel showing treatment of edges.

the longitudinal steel carried through the construction joints. The lining should be anchored to the slope as necessary by reinforced cutoff walls to prevent sliding.

Proper curing of the concrete lining is important, particularly in warm, dry, windy weather, to prevent the early drying of corners, edges, and surfaces. A well-moistened subgrade and wet burlap in contact with the exposed concrete surfaces is excellent for curing purposes.

The edges of newly constructed channels should be protected by a strip of sod at the time of construction. The design of the lining edges should allow enough depth of soil to permit the growth of grass, and the channel side corners of the top edge should be beveled as shown in figure 23.

5.6 Bituminous-lined channels. The construction of bituminous-lined channels is described in reference 40. A well-drained subgrade is necessary beneath a bituminous lining because the strength and weight of the lining is not sufficient to withstand high hydrostatic uplift pressure. Weed control (*40*, p. 12) measures are sometimes necessary before the lining is placed. These measures consist of careful grubbing of the subgrade followed by the application of a soil sterilant (*40*, p. 13).

5.7 Stone-lined channels. All stone used for channel linings or bank protection should be hard, dense, and durable. Most of the igneous and metamorphic rocks, many of the limestones, and some of the sandstones make excellent linings. Shale is not suitable and limestones and sandstones that have shale seams are undesirable. Quarried stones, angular in shape, are preferred to rounder boulders or cobbles. If rounded stones are used, the size of stone and thickness of the blanket, determined by the methods of section 4.12–1, should be increased, particularly if the rounded stones are relatively uniform in size (*37*, p. 207). Neither breadth or thickness of a single stone should be less than one-third of its length. Figure 24 shows two sample gradation curves which were found to be very satisfactory in tests at the U.S. Army Engineer Waterways Experiment Station (*46*). Curves approximately parallel to these curves and passing through the theoretical size (sec. 4.12–1) at the 50-percent point should make an acceptable gradation.

The stone lining or bank protection should be placed on a filter blanket of gravel or crushed stone meeting the requirements of section 4.12–4 unless the natural soil meets the requirements of a filter blanket.

The stones should be placed on the filter blanket or prepared natural slope in a manner which will produce a reasonably well-graded mass of stone with the minimum practicable percentage of voids. Stone protection should be placed to its full course thickness at one operation and in such a manner as to avoid displacing the underlying material. Placing of stone protection in layers or by dumping into chutes or by similar methods likely to cause segregation should not be permitted. The larger stones should be well distributed and the entire mass of stones should roughly conform to the gradation specified. The stone protection should be so placed and distributed as to avoid large accumulations or areas composed largely of either the larger or smaller sizes of stone. The mass should be fairly compact, with all sizes of material placed in their proper proportions. Hand placing or rearranging of individual stones by mechanical equipment may be required to the extent necessary to secure the results specified above. The desired distribution of the various sizes of stone throughout the mass might be obtained by selective loading at the quarry, by controlled dumping of successive loads during placing, or by a combination of these methods. Ordinarily the stone protection should be placed in conjunction with the construction of the embankment, with only sufficient delay in construction of the stone protection as may be necessary to prevent mixture of embankment and stone.

Bank protection, where the channel is composed of sand or silt, should extend a minimum vertical distance of 5 feet below the streambed on a continuous slope with the embankment. Where the streambed is of material other than sand or silt, the bank protection should be terminated in a rock toe at the level of the streambed to prevent undermining the bank protection. The toe should have a minimum base width of 6 feet and a minimum thickness of 3 feet with 1 on 1½ side slopes. On large rivers, or tidal estuaries, having a considerable depth of flow at low water stages, the stone protection need be carried down only 5 feet vertically below mean low water and the toe can be omitted.

Hand-placed stone should be carefully laid to produce a more or less definite pattern with a minimum of voids and with the top surface relatively smooth. Joints should be staggered between courses. The stone used for hand-placed protection should be of better quality than the minimum quality suitable for dumped stone protection (*37*, p. 208). Stones that are roughly square and of fairly uniform thickness are much easier to place than irregular stones. Stones of a flat stratified nature should be placed with the principal bedding planes normal to the slope. Openings to the subsurface should be filled with rock fragments; however, enough voids or openings should be left to vent the subsurface properly.

5.8 Ditch checks. Ditch checks must be firmly anchored into the banks of the drainage channels. The choice of material determines the methods used, but all checks should have a suitable apron at the toe of the drop and a cutoff wall at the downstream end of the apron. The apron should have a depression or a sill at the downstream edge so that a pool will be created to dissipate the energy of the falling water. If clay is available, local stone can be laid up in a rich clay mortar. This makes the check almost watertight and results in less maintenance than if the stones are laid up loose with the expectation that the check will become impermeable in time.

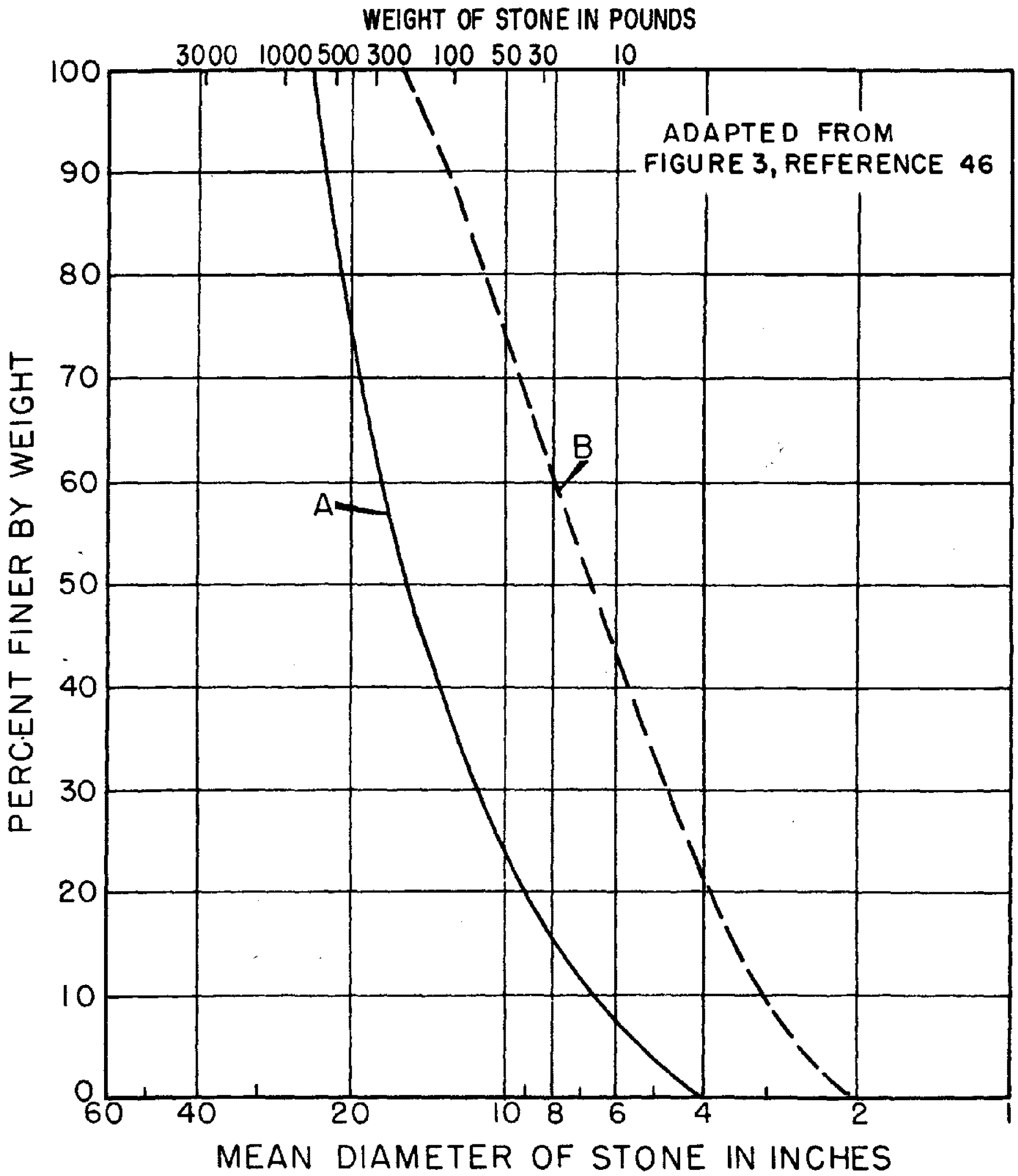


Figure 24.—Gradation curves for dumped-stone protection.

Chapter VI.—MAINTENANCE

6.1 General. Drainage channels rapidly lose their effectiveness unless they are adequately maintained. Thus, a good maintenance program is of equal importance with the proper design and construction of the channels. In fact, a knowledge of the equipment to be used in maintenance and the methods to be employed is a prerequisite of proper design.

Maintenance of vegetative cover on slopes and in drainage channels requires continued attention. The original treatment applied during construction will not last forever. Repeated applications of fertilizer, lime, or organic material at intervals are as necessary on the highway roadside as on the home lawn. Areas often need reseeding or resodding to restore the vegetative cover. This should be done before serious erosion occurs.

Minor erosion damage within the highway right-of-way should be repaired immediately after it occurs and action taken to prevent a recurrence of the damage. The damage caused by light storms reveals the points of weakness in the drainage system. If these weaknesses are corrected when repairing the damage itself, the drainage system will likely carry the design discharge without damage. Deficiencies that are found in the drainage systems and the corrective action taken should be reported to the hydraulic or design engineer so that similar troubles will not occur on future construction. Reports on drainage works that function well during severe storms are equally valuable to the designer.

6.2 Effect of maintenance on channel capacity. Maintenance of highway channels includes repairing erosion damage, mowing grass, and removing any sediment or debris deposited in the channel. All these measures keep the capacity of the channel at the design level. If the channel cross section contains brush, sediment, or debris, the channel cannot carry the flow for which it was designed. In addition, the sediment and debris may kill the vegetative lining with subsequent erosion damage during higher flood flows. In some situations, sediment traps and debris barriers might be constructed in the channel in order to collect the objectionable material for easy removal.

The effect of weed growth in the channel can be seen by reference to table 2, appendix A. For example, if a channel section 1.5 feet deep is designed for a velocity of 6 feet per second with a grass lining, mowed to a length of 2 inches, the value of n is 0.035. If the channel is not maintained and dense weeds 2 feet high are allowed to grow, the value of n becomes 0.01. The effect of the weed growth is that the channel will only carry $\frac{0.035}{0.01}$, or about one-third the flow for which it was designed. The remainder of the design flow must overflow the channel and cause flooding or possible erosion.

Chapter VII.—ECONOMICS OF DRAINAGE CHANNELS

7.1 General. Providing adequate drainage is essential to the existence of the highway. Economical drainage design is achieved through doing an adequate job at the lowest cost.

The lowest cost adequate channel maintains proper balance between first cost, flood damage, and maintenance cost, and has the capacity and protection to carry the runoff for which it was designed. Selection of the frequency (see sec. 1.2) of the design runoff is a matter of economics, while estimating the magnitude of the storm runoff for a selected frequency belongs in the field of hydrology. (See ch. II.)

In this chapter some of the factors to be considered in the economic selection of highway drainage channels are discussed.

7.2 Frequency of the design storm. The average annual cost of a drainage channel is the sum of (1) the first cost divided by the expected life of the channel, plus (2) the average annual maintenance cost, plus (3) the annual charge for possible damage from runoff exceeding the channel capacity. Computation of first cost is dicussed briefly by Hewes and Oglesby (*55*, p. 59). The average annual maintenance cost might also include the annual charge for flood damage if a flood exceeding the channel capacity has occurred. It is probably better to separate these costs. The damage to a channel designed to carry a 10-year runoff from a chance occurrence of, say, a 200-year runoff in a few years' record of maintenance expenditures would distort the average annual maintenence cost. The annual charge for possible flood damage should consider the frequency of the design storm and equals the cost of damage from runoff exceeding the channel capacity divided by the return period, in years, of the design storm.

If these costs could be evaluated for various combinations of the component costs, the most economical channel could be determined as the channel with the lowest average annual cost. The optimum frequency of the design storm would then be the frequency associated with the storm runoff that, in combination with other costs, produced the lowest average annual cost. The three cost items are interrelated and are difficult to evaluate, particularly the item of damage by runoff that exceeds the design runoff. The cost of storm damage includes the cost of traffic interruption by floodwaters or washed-out highway, as well as the cost of repairing the damage to the highway and drainage channels and the additional damage to the abutting property directly attributable to the presence of the highway. A further complication is the variation in damage due to the magnitude by which the runoff exceeds the design runoff.

Individual analysis for each small channel is impractical if not impossible. The best solution appears to be a study of average conditions and selection of the frequency of design runoff to be used for various drainage structures according to the class of the highway. The designated frequencies might vary from State to State or even within a State composed of areas differing widely in topography or population density.

Individual variations in the designated frequency of the design storm might be needed at locations where damage by flooding would be great and the cost of a channel large enough to carry a less frequent storm is moderate.

7.3 Effect of channel section. Roadway channels built in earth (sec. 1.6) should have side slopes not steeper than 4:1, and rounded bottoms at least 4 feet wide. Table 6 gives a comparison of trapezoidal channels of the same capacity by showing the ratios of area, wetted perimeter, and velocity for the various channels to those for the 4-foot bottom, 4:1 side slope channel. The depth of channel in table 6 is equal to the depth of flow. When the side slope remains 4:1, changing the bottom width from 2 to 5 feet changes excavation by 3 percent (compare ratio *A*); changes amount of lining by about 10 percent (compare ratio *WP*); and changes the velocity by 3 percent (compare ratio *V*). The depth of flow changes from 1.03 feet to 0.80 foot.

Table 6.—Properties of trapezoidal channels of the same capacity, with $n=0.03$ and slope $=0.01$

[Denominator of ratio is corresponding characteristic of a 4-foot bottom channel with side slopes 4:1]

Side slope	Property	Bottom widths			
		2	3	4	5
4:1	*d*, ft	1.03	0.94	0.87	0.80
	A, sq. ft	6.32	6.39	6.54	6.56
	R, ft	.60	.59	.58	.57
	V, f.p.s	3.52	3.48	3.44	3.40
	Q, c.f.s	22.2	22.2	22.5	22.5
	WP, ft	10.5	10.8	11.2	11.6
	Ratio *A*	.97	.98	1.00	1.00
	Ratio *WP*	.94	.96	1.00	1.04
	Ratio *V*	1.02	1.01	1.00	.99
3:1	*d*, ft	1.11	1.02	0.91	0.84
	A, sq. ft	5.96	6.19	6.15	6.34
	R, ft	.66	.65	.63	.61
	V, f.p.s	3.75	3.71	3.63	3.56
	Q, c.f.s	22.4	22.9	22.3	22.6
	WP, ft	9.0	9.5	9.8	10.3
	Ratio *A*	.91	.94	.94	.97
	Ratio *WP*	.80	.85	.88	.92
	Ratio *V*	1.09	1.08	1.06	1.03

Changing the side slopes to 3:1 has a greater effect on channel costs than changing the bottom width. Excavation ranges from 91 to 97 percent of that of the recommended channel; lining required ranges from 80 to 92 percent of that of the recommended channel; and velocities range from 3 to 9 percent greater. Depth of flow ranges from 0.04 to 0.08 foot deeper than that of the recommended channel.

The comparison of channel area in table 6 is only the area occupied by water without freeboard. In a cut section, the quantity of excavations would increase with increase in bottom width of the roadway channel because of the added width of the cut section. The percentages would change with change in channel slope. Change in the value of n would change both Q and V, for the same depth, but the velocity ratio would remain the same.

7.4 Effect of topography. In deep-cut sections the added excavation necessary for a trapezoidal roadway channel is oftentimes prohibitive in cost. A more economical drainage system might be a gutter of small capacity at the toe of the cut slope, with frequent inlets connected to a storm sewer under the roadbed. A similar system is sometimes used on parkways in combination with curbs and gutters; however, this choice is often made for appearance rather than economy.

7.5 Effect of channel lining. The choice in channel lining is limited to linings which can withstand the expected channel velocities. (See tables 3 and 4, app. A.) In comparing the cost of channel linings, the effect of channel roughness should be considered (sec. 4.6–3, example 10) so that the comparison is made among channels of equal capacity rather than that of equal size.

In regions where grass grows readily, grass-lined channels are generally the lowest in cost. In some areas, however, salt used for ice removal makes the maintenance of grass linings difficult and such factors should be considered when selecting the type of lining.

Where a choice must be made between concrete and stone for lining, a survey by the U.S. Corps of Engineers (*37*, p. 204) of the performance of materials in upstream slope protection of earth dams is interesting although not directly applicable to channel linings. The survey of approximately 100 dams, from 5 to 50 years old, located in various sections of the United States, with a wide variety of climatic conditions and wave severity, showed that—

(1) dumped riprap failed in 5 percent of the cases where used, failures being attributed to improper size of stones;

(2) hand-placed riprap failed in 30 percent of the cases where used;

(3) concrete pavement failed in 36 percent of the cases where used.

This comparison of the performance of concrete and stone suggests that when hand-placed riprap or concrete pavement is used for channel linings, greater support is needed from the subgrade than when dumped stone is used. Perhaps more attention should be given to the drainage and preparation of the subgrade for a rigid type of lining than for a more flexible type of lining.

When hand-placed stone is used, a better quality of stone is required than the minimum quality suitable for dumped stone (*37*, p. 208). However, hand-placed stone should not be used where settlement or heavy ice action is anticipated. Settlement is also detrimental to concrete linings.

In making a comparison of channel lining costs, the designer should consider the velocity of the water in the channel and include the cost of any measures required to dissipate the energy of flow that would not be required with an alternate type of lining.

7.6 Drop structures versus chutes. Either drop structures or chutes are frequently needed to convey water down steep slopes or into natural watercourses with incised channels. An economic comparison of several types of such structures for specific conditions indicated a wide range in cost. The types studied were a sodded channel on the maximum permissible grade; a corrugated metal pipe culvert with flared inlet without stilling basin, a reinforced concrete drop structure; and a gravel-lined channel on the maximum permissible grade. The results of this study for structures with drops of 4-, 6-, and 10-feet designed for a discharge of 10 cubic feet per second are given in table 7, following. The comparison is given by the ratio of the cost of the structure to the cost of a sodded channel with the same drop.

Table 7.—Cost comparison of drop structures

Type of structure	Cost ratio drop, in feet		
	4	6	10
Sodded channel	1.00	1.00	1.00
C.M. pipe culvert on a 5-percent grade	5.98	4.33	2.33
C.M. pipe culvert on a 20-percent grade	2.03	1.33	.66
Reinforced concrete drop	6.50	3.84	2.17
Gravel-lined channel	3.43	3.44	3.38

The variation in cost of the sodded channel with the fall, based on the 4-foot fall, are: 4-foot fall, 1.00; 6-foot fall, 2.20; 10-foot fall, 7.12.

The ratios in table 7 apply only to the specific conditions of this analysis and might vary locally with the availability of materials. The costs used were the average of several 1960 bid prices in 10 widely dispersed states. Except for the 10-foot drop, the sodded channel is the lowest cost means of conveying 10 cubic feet per second of water down steep slopes. The apparent advantage of the pipe culvert on the steeper grades might be offset at some locations by the added cost of dissipating the energy of water at the outlet. On the other hand, the use of a sodded channel requires that sufficient length be available to place the channel on a noneroding grade.

Chapter VIII.—ILLUSTRATIVE PROBLEM IN CHANNEL DESIGN

8.1 General. The procedures given in this publication are used to design several of the drainage channels shown in figures 2 and 25. The site of the highway is assumed to be in the vicinity of Washington, D.C., and the runoff from a 10-year storm has been selected for the design discharge. Typical cross sections of the roadway are not shown, but the maximum depths of cut and fill, at the centerline, are 50 ft. and 26 ft., respectively.

The design of drainage channels is in three phases: (1) laying out the drainage system (sec. 4.2), (2) estimating storm runoff (ch. II), and (3) designing the channels (ch. IV). For convenience, the design will be completed for each drainage channel, combining the estimate of runoff and the computation of size of channel before proceeding to the next channel.

8.2 Layout of the drainage system. Figure 25 is a plan view of figure 2. The roadbed and right-of-way lines have been drawn and the drainage divides are shown by heavy dashed lines, but other roadway details have been omitted in this problem. The drainage channels are indicated by short dashed lines with arrows to indicate direction of flow and are numbered as shown in figure 26. The points for which channel sizes are computed are indicated by letters. The numbers are for identification in this problem; however, a similar system might be helpful in an actual design problem. The channels are numbered on the assumption that the road project began at Station 45 and the stream at Station 55 is the third drainage channel in the project.

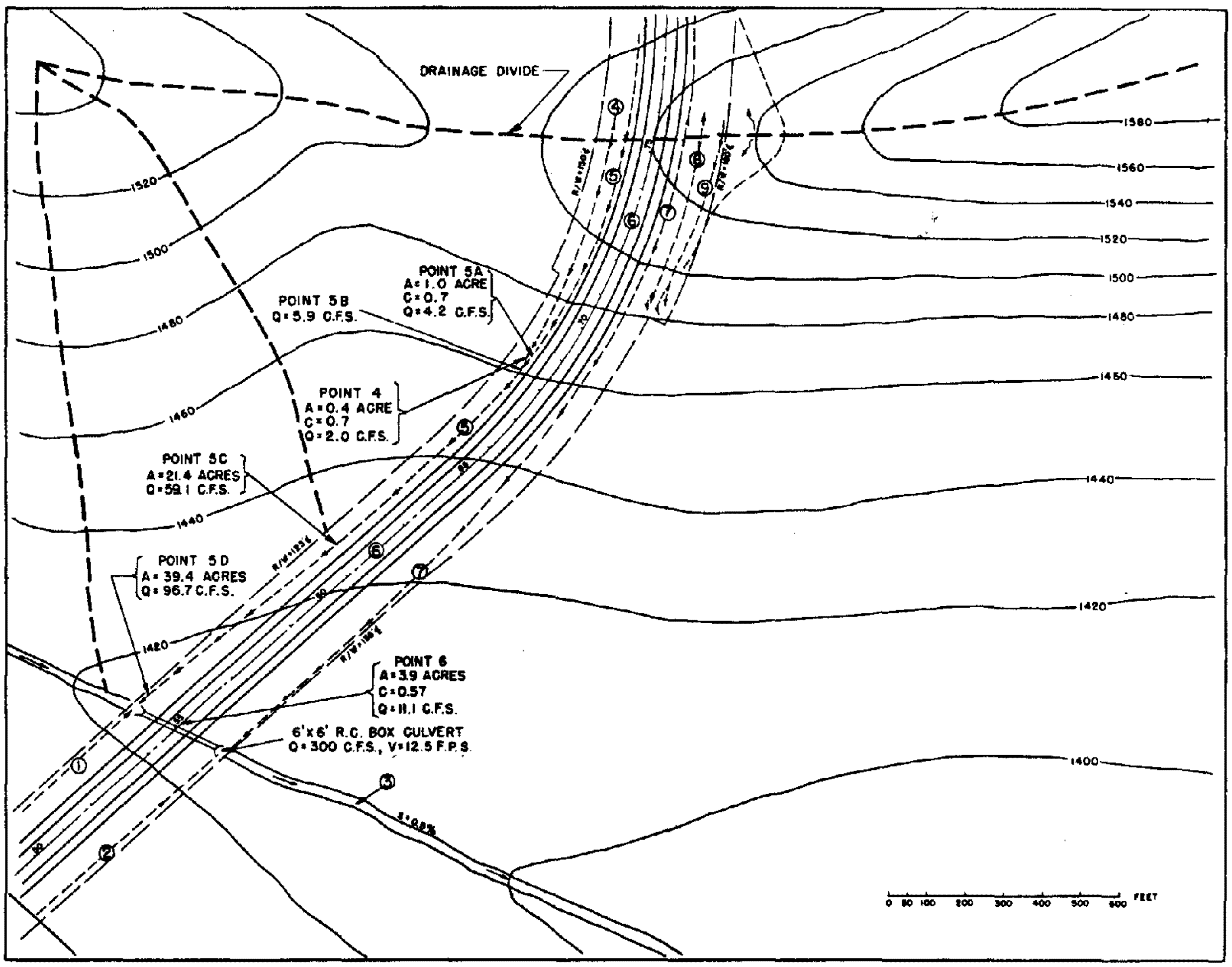


Figure 25.—Layout of drainage system for illustrative problem.

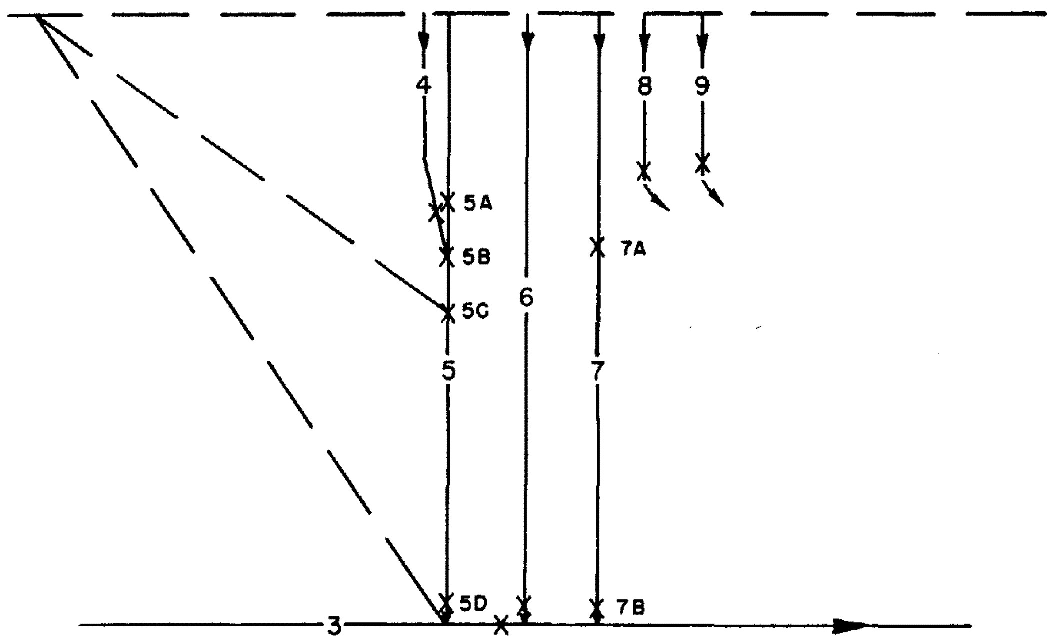


Figure 26.—Identification of drainage channels.

In figure 25, roadway channels (Nos. 5 and 7) and the median swale (No. 6) drain the roadway cut. The water from the roadway channels must be carried to the natural watercourse through the toe-of-slope channels which continue to carry the channel number. Intercepting channels (Nos. 4 and 8) are placed on the cut slopes to illustrate their design. The depth of cut in this problem would seldom require intercepting channels located on the side slope. Note that the exit of channel 8 is flared and the flow is spread over the hillside rather than brought into channel 7, while the water in channel 4 is conducted to channel 5.

The intercepting channel (No. 9) at the top of the cut on the right prevents storm water from running down the cut slope; its flow is distributed over the hillside. This type of disposal is preferable wherever the quantity of water intercepted is small and the topography and land use permit. On the left side of the cut, the natural ground slopes away from the cut edge and an intercepting channel is not required.

8.3 Channel 3. The grade of the small stream is 0.5 percent and the design discharge is 300 c.f.s. Using the charts of Hydraulic Engineering Circular No. 5 (*51*) a 6-ft. by 6-ft. reinforced-concrete box culvert is selected. The outlet velocity is 12 f.p.s. and the depth of flow at the outlet is 4.2 ft. when carrying the design discharge. The specific head at the culvert outlet, computed by equation (9), is 6.4 ft. The channel downstream from the culvert outlet is to be trapezoidal with bottom width 6 ft. and side slopes 2:1.

In going from the rectangular culvert section to the larger open-channel section, the flow expands with some loss in energy, accompanied by a decrease in depth and an increase in velocity. The depth and velocity at the expanded section can be approximated by assuming a loss in the expansion of 20 percent of the velocity head. This determination of expansion loss is somewhat arbitary and is based on an analogy to pipe flow. See any text on hydraulics. In this problem the approximate loss is $0.2\,\frac{(12)^2}{64.4}=0.5$. The specific head at the expanded section is then $6.4-0.5=5.9$ ft.

The depth and velocity at the expanded section can be computed by finding a combination of depth and velocity which will produce the specific head at the section (5.9 ft. in this problem). The computation is a trial-and-error procedure which can be facilitated by the tables in reference 26 or the charts in reference 25. A good starting depth for the trial computations is about one-half the depth at the culvert outlet, or 2.0 ft. Using table 10 on page 162 of reference 26, the area of the trapezoidal channel can be found. The discharge is divided by the area to obtain the velocity. The velocity is squared and divided by 64.4 (2g) to obtain the velocity head, H_v. H_v is added to the depth to obtain the specific head, H_0 The data for this problem are:

Quantity	Trial 1	Trial 2	Units
d	2.0	1.9	ft.
A	20.0	18.6	sq. ft.
V	15.0	16.1	f.p.s.
H_v	3.5	4.0	ft.
H_0	5.5 too low	5.9 OK	ft.

A velocity of 16.1 f.p.s. will require a reduction in velocity (stilling basin) or protection of the channel in most materials. Dumped stone will be used in this problem. When the stone size becomes too large, a stilling basin at the culvert outlet might be considered. A concrete-lined channel would only move the erosion problem downstream.

To compute the size of stone (see sec. 4.12–1), assume a stone size $(k)=2.25$ ft.

$$\frac{k}{d}=\frac{2.25}{1.9}=1.2 \qquad \text{thus } \frac{V_s}{V}=1.0 \text{ and } V_s=16.1 \text{ f.p.s.}$$

From figure 21, the required stone size is 2.25 ft., which checks the assumed size.

The increased roughness of the stone lining will reduce flow velocities and change the depth of flow from that at the beginning of the channel. The new channel depth will be 4.3 ft., computed as follows:

From figure 19, for 2.25-ft. stone, $n=0.04$, and from page 162 and page 448, of reference 26:

$d=4.3$ ft.
$A=62.8$ sq. ft.
$R=2.49$ ft.
$V=4.8$ f.p.s. for $n=0.04$ and $S=0.5$ percent
$Q=301$ c.f.s.

The channel will be lined with 2¼ ft. equivalent diameter stone (see sec. 5.7 for gradation) with a minimum lining thickness of 3.0 ft. The lined channel will have a depth of 6 ft. (including about 1½ ft. of freeboard) at the culvert outlet. The depth of the lined channel will be gradually decreased to 4.7 ft. (0.4 ft. freeboard) at the edge of the right-of-way. The stone will be dumped on a filter blanket, designed after a soil analysis of the bank and bed material. The side slope above the stone lining would be grassed lined.

The freeboard allowance is somewhat arbitrary. The larger freeboard at the culvert outlet is to allow for the turbulence and the hydraulic jump in the reach in which some of the energy is being dissipated. The velocity of flow in the channel at a depth of 4.3 ft. is computed to be 4.8 f.p.s. This is only an approximation because the reach of channel is too short to develop the normal velocity. Stone 2¼ ft. in diameter would not be required at such a low velocity. The depth after the hydraulic jump is about 4.3 ft.

8.4 Channel 4. This channel is an intercepting channel extending from Station 68 to Station 75 on the left side of the highway. The design runoff need only be estimated at the downstream end of the channel.

Estimating Runoff (See Ch. II)

The channel drains the grassed cut slope which ranges in width from 0 to 50 ft. and is 700 ft. long.

The components of the formula $Q=CiA$ are:

$C=0.7$ (table 1).
i read from fig. 6 after computing T_c.
Computation T_c (sec. 2.4–2).

	Cut slope	Channel	Total	Units of reference
$H=$	33	28	61	ft.
$L=$	50	700	750	ft.

$T_c=3.3$ min. (fig. 5). Use 5 min.
i (fig. 6) for 10-year return period and T_c of 5 min. $=7.1$ in. per hr.

$$A=\frac{50}{2}\times\frac{(700)}{(43{,}560)}=0.4 \text{ acre.}$$

Then $Q=CiA=0.7\times7.1\times0.4=2.0$ c.f.s.

Computing Channel Section (See Sec. 4.8)

For a 2-ft. bottom, grass-lined channel, 2:1 side slopes, $Q=2.0$ c.f.s., $n=0.10$ (table 2), $S=0.04$. Chart 15 reference 25 can be used to find d and V.

$Qn=2.0\ (0.10)=0.20$.
$d\ =0.5$ and $Vn=0.14$.

$$V=\frac{0.14}{0.10}=1.4 \text{ f.p.s.}$$

Note that a high value of n (0.10) is used to compute the size of channel because the grass might be allowed to grow taller than anticipated. To check the adequacy of the lining, a low n value (0.05), based on the channel being maintained at design conditions, is used. If the lower value of n is effective, the velocity in the channel from chart 15 ($Qn=0.10$, $Vn=0.115$) is 2.3 f.p.s. This velocity would not endanger the grass lining of the channel. The freeboard needed for this channel is negligible, as the capacity was computed for the downstream point on the channel and little damage would result from an overtopping of the channel.

8.5 Channel 5. This channel is the left roadway channel from Station 68 to Station 75 and a toe-of-slope channel from Station 55 to Station 68. The design runoff will be estimated (see fig. 26) just before (5A) and just after (5B) the entrance of channel 4; and below the entry of runoff from each of the two large areas on the hillside to the left (5C and 5D).

Estimating Runoff (See ch. II)

Point 5A (Sta. 68) above junction

The section is on a curve and the channel (Sta. 68 to Sta. 75) must carry the runoff from the left lane (24 ft.×700 ft.=16,800 sq. ft., $C=0.9$), the shoulder and ditch section and the runoff from the triangular section of cut slope not intercepted by channel 4 $\left(\frac{80}{2}\text{ ft.}\times700\text{ ft.}=28{,}000\text{ sq. ft.}, C=0.6\right)$

The components of the formula $Q=CiA$ are:

Weighted C (see sec. 2.4–1).	*Sq. ft.*	C
pavement=	16,800×0.9=	15,120
shoulder ditch and slope=	28,000×0.6=	16,800
	44,800	31,920
Weighted C		=0.7

i is read from figure 6 after computing T_c.
Computation T_c (sec. 2.4–2).

	Cut slope	Channel	Units or reference
H=	21	10.5	ft.
L=	32	700	ft.
T_c=	1 (approx.)	6	min.

T_c for pavement about 4 minutes (estimated).
Total $T_c=4+6=10$ min.

Note that the 1 min. T_c for the side slope is concurrent with the 4 min. T_c for the pavement area and only the longer time is used, i (fig. 6) for a 10-year return period and T_c of 10 min.=6.0 in. per hr.

$$A=\frac{44{,}800}{(43{,}560)}=1.0 \text{ acre.} \quad \text{(See calculation of } C.)$$

Then $Q=CiA=0.7\times6.0\times1.0=4.2$ c.f.s.

Point 5B (Sta. 68) below junction

The discharge below the junction of channel 4 and the channel above 5A is computed from the longer time of concentration (10 min.) of the two areas. The intensity (fig. 6) for a 10-year return period and $T_c=10$ min. is 6.0 in. per hr. The discharge for point 5B is:

Q (above 5A) $=0.7\times6.0\times1.0=4.2$ c.f.s.
Q (channel 4) $=0.7\times6.0\times0.4=1.7$ c.f.s.

Total Q -------------- 5.9 c.f.s.

Point 5C (Sta. 61)

This channel must carry the runoff from above point 5B plus the contribution from the triangular area to the left of the channel ($A=19$ acres, $H=110$ ft., $L=1{,}200$ ft.), plus the runoff from the roadway area ($A=1$ acre, weighted $C=0.66$) between channel 5 and the center of the left lane. The T_c for the roadway area is obviously much less than that of the larger area to the left of the channel and need not be computed as it runs off concurrently. T_c for the runoff above point 5B was computed as 10 minutes. This runoff must flow down to point 5C in the toe-of-slope channel ($L=700$ ft., $H=24.5$ ft.). This requires 4 minutes (fig. 5) or a total T_c of 14 min. T_c for the 19-acre area is:

$H=100$ ft.
$L=1{,}200$ ft.
$T_c=5$ min. (fig. 5)

The runoff from the two areas is concurrent and ordinarily the longer T_c would be used. i (fig. 6) $=5.1$ in. per hr. for a 10-year return period and $T_c=14$ min.

The conventional computation is made by recomputing the discharge of the separate areas using an i corresponding to a T_c of 14 min. and adding the discharges rather than computing a weighted C for the entire area above point 5C. This discharge is:

Above point 5B:
At point 4, $Q=0.7\times5.1\times0.4=1.4$ c.f.s.
At point 5A, $Q=0.7\times5.1\times1.0=3.6$ c.f.s.

5.0 c.f.s.

Between points 5B and 5C:
$Q=0.4\times5.1\times19.0=38.8$ c.f.s.
$Q=0.66\times5.1\times1.0=3.4$ c.f.s.

42.2 c.f.s.

Total Q for point 5C=47.2 c.f.s.

The discharge estimated by the conventional method is probably low because 19.0 acres of the total 21.4 acres has a T_c of 5 minutes and 4 minutes are required for the flow to reach point 5C from point 5B (1.4 acres, $T_c=10$ minutes). It appears likely that the peak at point 5C will be reached before the total area is contributing because this would take 14 minutes.

Extreme precision is not warranted in determining the combination of contributing area (CA) and rainfall intensity (i) that would produce the greatest peak runoff. Neither the data available for a particular problem nor the rational method are of sufficient accuracy to justify a trial-and-error solution to the problem. In this problem we will assume that the 5-minute time of concentration for the 19-acre area will produce the greatest peak runoff. In this 5 minutes ($i=7.1$ in. per hr.) runoff will arrive from all of the 19-acre area and from the roadway area (1 acre) between points 5C and 5B, plus that portion of the area above point 5B that can arrive in 5 minutes. The contributing portion of the area above point 5B can be estimated closely enough in most problems by assuming that the contributing area bears the same relation to the total area as the times of concentration of the respective areas. In this problem after allowing 4 minutes for channel flow time, only that portion of the area (total $T_c=10$ min.) that can reach point 5B in 1 minute will contribute to the peak discharge. This area is assumed to be one-tenth of 1.4 acres=0.1 acre.

The discharge at point 5C for part of the area contributing ($T_c=5$ min.) is:

$Q=0.4\times7.1\times19.0=53.9$ c.f.s.
$Q=0.66\times7.1\times1.0=4.7$ c.f.s.
$Q=0.7\times7.1\times0.1=0.5$ c.f.s.

Total Q ------ 59.1 c.f.s.

The design discharge is 59.1 c.f.s., the peak from the partial area because it is larger than the peak of 47.2 c.f.s. from the total area.

Point 5D (Sta. 55)

The area to the left of the channel between point 5C and 5D is 17.0 acres ($H=128$, $L=1{,}600$ ft., $C=0.4$) and that to the right is 1.0 acre (weighted $C=0.65$).

T_c for the 17-acre area is:

$H=128$ ft.
$L=1{,}600$ ft.
$T=6$ min. (fig. 5).

The T_c for the area above point 5C is 5 minutes plus the time required for the water to flow from point 5C to point 5D (L=600 ft., H=8.5 ft., T=5.5 min.). The total T_c=5 min.+5.5 min.=10.5 min. and from figure 6, i=5.9 in. per hr. The design discharge is:

Between points 5C and 5D:

Q (left side) = 0.4 × 5.9 × 17.0 = 40.1 c.f.s.
Q (right side) = 0.65 × 5.9 × 1.0 = 3.8 c.f.s.

43.9 c.f.s.

Above point 5C:

Q=0.4 × 5.9 × 19.0 = 44.8 c.f.s.
Q=0.66 × 5.9 × 1.0 = 3.9 c.f.s.
Q=0.7 × 5.9 × 0.1 = 4.1 c.f.s.

52.8 c.f.s.

Total Q=96.7 c.f.s.

It will be noted that computations of weighted C, and area once determined are used in successive computations. It is interesting to note that the size of the channel at point 5C is computed for a discharge (59.1 c.f.s.) different from the discharge (52.8 c.f.s.) contributed by the same area to the design discharge of point 5D.

Computing Channel Section (See Sec. 4.8)

The following computations are for depth of water area and do not include freeboard. The channel section at points 5B, 5C, and 5D can be designed with chart 17 (using the Qn scale), reference 25, with much less work than the method used here.

Point 5A (Sta. 68) above junction

Q=4.2 c.f.s., S=0.015, n=0.09, for grass-lined channel (table 2). The channel section is trapezoidal with a 4-ft. bottom and side slopes 4:1 on the pavement side and 1½:1 on the cut-slope side. Data from reference 26 for the final trials are:

d = 0. 6	0. 8	ft.
A = 3. 39	4. 96	sq. ft.
R ≐ 0. 46	. 57	ft.
V = 1. 18	1. 37	f.p.s.
Q = 4. 0	6. 8	c.f.s.

A channel with a depth of 0.7 ft. will have a capacity of about 5.4 c.f.s. which will carry the design discharge. The data listed cannot be found directly in reference 26 because of the unequal side slopes and the absence of a table of velocities for n=0.09. However, the values of A and R for a particular value of d can be taken from the table for a channel with a 4-ft. bottom and 1½:1 side slopes, and averaged with value of A and R for the some value of d taken from table for a channel with a 4-ft. bottom and 4:1 side slopes. The velocity was taken from the table for n=0.045 and divided by 2 to obtain the value for n=0.09.

Point 5B (Sta. 68) below junction

Q=5.9 c.f.s., S=0.065, n=0.08 (table 2).

The channel is at a fill section at point 5B and will have a grass-lined trapezoidal section with 4-ft. bottom width and side slopes 2:1. Data for the final trials from reference 26 are:

d = 0. 6	0. 4	ft.
A = 3. 12	1. 92	sq. ft.
R = 0. 47	. 33	ft.
V = 2. 85	2. 24	f.p.s.
Q = 8. 9	4. 3	c.f.s.

A channel with a depth of 0.5 ft. will have a capacity of about 6.6 c.f.s., which will carry the design discharge at a velocity which will not erode the grass lining (table 4). Caution should be observed in selecting the value of n for shallow depths in grass-lined channels. The value of n changes with changes in either depth of flow or velocity. It will be noted in table 2 that the worst condition (tall grass) was used to compute the channel capacity. (See sec. 4.8.) The best condition, mowed grass, should be used to compute the velocity for checking the ability of the lining to withstand the flow. The velocities in this channel are well within the allowable velocity for grass and the check is not necessary.

Point 5C (Sta. 61)

Q=59.1 c.f.s., S=0.035, n=0.05, grass-lined trapezoidal section with a 4-ft. bottom and side slopes 2:1. Data for the final trials from reference 26 are:

d=1.4	1. 5	ft.	
A=9.52	10. 52	sq. ft.	
R=0.93	. 98	ft.	
V=5.3	5. 5	f.p.s.	
Q=51	57. 8	c.f.s.	OK

For checking permissible velocity of the channel lining, n=0.035 (table 2). The velocity with this n is approximately $\frac{0.05}{0.035}$ (5.5)=7.9 f.p.s. This velocity is less than that permitted, 8 f.p.s., in erosion-resistant soil (table 4). The 7.9-f.p.s. velocity is for the same depth of flow. The velocity for the same discharge and an n=0.035 is 7.3 f.p.s.

Point 5D (Sta. 55)

Q=96.7 c.f.s., S=0.01, n=0.03, grass-lined trapezoidal section with 4-ft. bottom, and side slopes 2:1. Data for the final trial from reference 26 are:

d= 2. 1 ft.
A=17. 24 sq. ft.
R= 1. 29 ft.
V= 5. 9 f.p.s.
Q=102 c.f.s.

For checking permissible velocity of the channel lining, n=0.025 (table 2). The velocity with this n is approximately $\frac{0.03}{0.025}$ (5.9)=7.1 f.p.s. This velocity is less than that permitted, 8 f.p.s., in erosion-resistant soil (table 4).

Channel 5 from Sta. 68 to Sta. 55

The toe-of-slope channel can be kept at the same bottom width (4 ft.) and side slopes (2:1) and the depth progressively increased between the points for which computations were made. A freeboard of from 0.3 to 0.5 ft.

should be sufficient and a Bermuda grass lining is satisfactory throughout the length of the channel. The berm between the edge of the channel and the toe of the fill slope should be sloped to drain into the channel.

8.6 Channel 6. The capacity of the median swale and the velocity of flow will be checked at Station 55 to determine the feasibility of bringing the runoff from the top of the hill (Sta. 75+30) to Station 55.

Estimating Runoff (See Ch. II)

The components of the formula $Q=CiA$ are:

Weighted C (see sec. 2.4–1.)	Ft.	C
pavement=	24×0.9	=21.6
shoulders=	12×0.6	= 7.2
swale=	48×0.4	=19.2
	84	48.0
Weighted C		=0.57

Note that the width rather than the area was used to weight C, as the section was uniform throughout the length. After computing T_c, i is read from figure 6.

Computation T_c (sec. 2.4–2):

	Cross slope	Channel	Total	Units or reference
H=	3.3	30.4	33.7	ft.
L=	54	2,030	2,084	ft.

T_c=14 min. (fig. 5).

i (fig. 6) for 10-yr. return period and T_c of 14 min. =5.1 in. per hr.

$$A=\frac{(84)(2030)}{43,560}=3.91 \text{ acres.}$$

Then $Q=CiA=0.57\times5.1\times3.91=11.3$ c.f.s.

Computing Channel Section (See Sec. 4.8)

The median swale has a bottom width of 4 ft.; side slopes, 6:1; grade, 1.5 percent; Bermuda grass lining; and a depth of 3.3 ft. below the shoulder edge.

From table 2, for an estimated depth of 0.8 ft., and estimated velocity of 2 f.p.s., n=0.05 for 2-in. grass, and 0.06 for 6-in. grass. The final trial is for an 0.8 ft. depth (n=0.06).

A= 7.04 sq. ft.
WP=13.6 ft.
R= 0.52 ft.
V= 1.9 (reference 26).
Q=13.4 c.f.s.

The computed depth of flow, 0.8 ft., is less than the available depth, 3.3 ft., and the approximate velocity, 2.3 f.p.s. (1.9×0.06/0.05), is below the permissible velocity (table 4) for Bermuda grass. Thus, the design runoff can be safely carried to an inlet at Station 55 and dropped into the culvert at that location.

Note that this channel could have been designed by chart 32 of reference 25 with much less work than was required by the trial solution shown here, particularly since the area, wetted perimeter, and hydraulic radius had to be computed for each trial depth.

8.7 Channels 7, 8, and 9. Design of channels 7, 8, and 9 will not be shown because they do not introduce additional design problems.

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Appendix A.—TABLES

Table 1. Values of Runoff Coefficients (*C*) for Use in the Rational Method

Type of surface	Runoff coefficient (C)[1]
Rural Areas	
Concrete or sheet asphalt pavement	0.8–0.9
Asphalt macadam pavement	0.6–0.8
Gravel roadways or shoulders	0.4–0.6
Bare earth	0.2–0.9
Steep grassed areas (2:1)	0.5–0.7
Turf meadows	0.1–0.4
Forested areas	0.1–0.3
Cultivated fields	0.2–0.4
Urban Areas[2]	
Flat residential, with about 30 percent of area impervious	0.40
Flat residential, with about 60 percent of area impervious	0.55
Moderately steep residential, with about 50 percent of area impervious	0.65
Moderately steep built up area, with about 70 percent of area impervious	0.80
Flat commercial, with about 90 percent of area impervious	0.80

[1] For flat slopes or permeable soil, use the lower values. For steep slopes or impermeable soil, use the higher values.
[2] See reference 12, pp. 48–49 for more detailed data.

Table 2.—Manning Roughness Coefficients, *n*[1]

	Manning n range[2]
I. Closed conduits:	
A. Concrete pipe	0.011–0.013
B. Corrugated metal pipe or pipe arch:	
1. 2⅔- by ½-in. corrugation (riveted pipe)[3]	
a. Plain or fully coated	0.024
b. Paved invert (range values are for 25 and 50 percent of circumference paved):	
(1) Flow full depth	0.021–0.018
(2) Flow 0.8 depth	0.021–0.016
(3) Flow 0.6 depth	0.019–0.013
2. 3- by 1-in. corrugation	0.027
3. 6- by 2-in. corrugation (field bolted)	0.032
C. Vitrified clay pipe	0.012–0.014
D. Cast-iron pipe, uncoated	0.013
E. Steel pipe	0.009–0.011
F. Brick	0.014–0.017
G. Monolithic concrete:	
1. Wood forms, rough	0.015–0.017
2. Wood forms, smooth	0.012–0.014
3. Steel forms	0.012–0.013
H. Cemented rubble masonry walls:	
1. Concrete floor and top	0.017–0.022
2. Natural floor	0.019–0.025
I. Laminated treated wood	0.015–0.017
J. Vitrified clay liner plates	0.015
II. Lined open channels:[4]	
A. Concrete, with surfaces as indicated:	
1. Formed, no finish	0.013–0.017
2. Trowel finish	0.012–0.014
3. Float finish	0.013–0.015
4. Float finish, some gravel on bottom	0.015–0.017
5. Gunite, good section	0.016–0.019
6. Gunite, wavy section	0.018–0.022
B. Concrete bottom float-finished, sides as indicated:	
1. Dressed stone in mortar	0.015–0.017
2. Random stone in mortar	0.017–0.020
3. Cement rubble masonry	0.020–0.025
4. Cement rubble masonry, plastered	0.016–0.020
5. Dry rubble (riprap)	0.020–0.030
C. Gravel bottom, sides as indicated:	
1. Formed concrete	0.017–0.020
2. Random stone in mortar	0.020–0.023
3. Dry rubble (riprap)	0.023–0.033
D. Brick	0.014–0.017
II. Lined open channels—Continued	
E. Asphalt:	
1. Smooth	0.013
2. Rough	0.016
F. Wood, planed, clean	0.011–0.013
G. Concrete-lined excavated rock:	
1. Good section	0.017–0.020
2. Irregular section	0.022–0.027
III. Unlined open channels:[4]	
A. Earth, uniform section:	
1. Clean, recently completed	0.016–0.018
2. Clean, after weathering	0.018–0.020
3. With short grass, few weeds	0.022–0.027
4. In gravely, soil, uniform section, clean	0.022–0.025
B. Earth, fairly uniform section:	
1. No vegetation	0.022–0.025
2. Grass, some weeds	0.025–0.030
3. Dense weeds or aquatic plants in deep channels	0.030–0.035
4. Sides, clean, gravel bottom	0.025–0.030
5. Sides, clean, cobble bottom	0.030–0.040
C. Dragline excavated or dredged:	
1. No vegetation	0.028–0.033
2. Light brush on banks	0.035–0.050
D. Rock:	
1. Based on design section	0.035
2. Based on actual mean section:	
a. Smooth and uniform	0.035–0.040
b. Jagged and irregular	0.040–0.045
E. Channels not maintained, weeds and brush uncut:	
1. Dense weeds, high as flow depth	0.08–0.12
2. Clean bottom, brush on sides	0.05–0.08
3. Clean bottom brush on sides, highest stage of flow	0.07–0.11
4. Dense brush, high stage	0.10–0.14
IV. Highway channels and swales with maintained vegetation[5,6] (values shown are for velocities of 2 and 6 f.p.s.):	
A. Depth of flow up to 0.7 foot:	
1. Bermuda grass, Kentucky bluegrass, buffalo grass:	
a. Mowed to 2 inches	0.07–0.045
b. Length 4 to 6 inches	0.09–0.05
2. Good stand, any grass:	
a. Length about 12 inches	0.18–0.09
b. Length about 24 inches	0.30–0.15
3. Fair stand, any grass:	
a. Length about 12 inches	0.14–0.08
b. Length about 24 inches	0.25–0.13
B. Depth of flow 0.7–1.5 feet:	
1. Bermuda grass, Kentucky bluegrass, buffalo grass:	
a. Mowed to 2 inches	0.05–0.035
b. Length 4 to 6 inches	0.06–0.04
2. Good stand, any grass:	
a. Length about 12 inches	0.12–0.07
b. Length about 24 inches	0.20–0.10
3. Fair stand, any grass:	
a. Length about 12 inches	0.10–0.06
b. Length about 24 inches	0.17–0.09
V. Street and expressway gutters:	
A. Concrete gutter, troweled finish	0.012
B. Asphalt pavement:	
1. Smooth texture	0.013
2. Rough texture	0.016
C. Concrete gutter with asphalt pavement:	
1. Smooth	0.013
2. Rough	0.015
D. Concrete pavement:	
1. Float finish	0.014
2. Broom finish	0.016
E. For gutters with small slope, where sediment may accumulate, increase all above values of n by	0.002
VI. Natural stream channels:[7]	
A. Minor streams[8] (surface width at flood stage less than 100 ft.):	
1. Fairly regular section:	
a. Some grass and weeds, little or no brush	0.030–0.035
b. Dense growth of weeds, depth of flow materially greater than weed height	0.035–0.05
c. Some weeds, light brush on banks	0.04–0.05
d. Some weeds, heavy brush on banks	0.05–0.07
e. Some weeds, dense willows on banks	0.06–0.08
f. For trees within channel, with branches submerged at high stage, increase all above values by	0.01–0.10
2. Irregular sections, with pools, slight channel meander; increase values in 1 a–e about	0.01–0.02
3. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stage:	
a. Bottom of gravel, cobbles, and few boulders	0.04–0.05
b. Bottom of cobbles, with large boulders	0.05–0.07

Footnotes to Table 2 appear on p. 54.

VI. Natural stream channels—Continued	Manning n range [2]
B. Flood plains (adjacent to natural streams):	
1. Pasture, no brush:	
a. Short grass	0.030–0.035
b. High grass	0.035–0.05
2. Cultivated areas:	
a. No crop	0.03–0.04
b. Mature row crops	0.035–0.045
c. Mature field crops	0.04–0.05
3. Heavy weeds, scattered brush	0.05–0.07
4. Light brush and trees: [9]	
a. Winter	0.05–0.06
b. Summer	0.06–0.08
5. Medium to dense brush: [9]	
a. Winter	0.07–0.11
b. Summer	0.10–0.16
6. Dense willows, summer, not bent over by current	0.15–0.20
7. Cleared land with tree stumps, 100–150 per acre:	
a. No sprouts	0.04–0.05
b. With heavy growth of sprouts	0.06–0.08
8. Heavy stand of timber, a few down trees, little undergrowth:	
a. Flood depth below branches	0.10–0.12
b. Flood depth reaches branches	0.12–0.16
C. Major streams (surface width at flood stage more than 100 ft.): Roughness coefficient is usually less than for minor streams of similar description on account of less effective resistance offered by irregular banks or vegetation on banks. Values of n may be somewhat reduced. Follow recommendation of note 7 if possible. The value of n for larger streams of most regular sections, with no boulders or brush, may be in the range of from	0.028–0.033

Footnotes to Table 2

[1] Estimates are by Bureau of Public Roads unless otherwise noted and are for straight alinement. A small increase in value of n may be made for channel alinement other than straight.

[2] Ranges for secs. I through III are for good to fair construction. For poor quality construction, use larger values of n.

[3] *Friction Losses in Corrugated Metal Pipe*, by M. J. Webster and L. R. Metcalf, Corps of Engineers, Department of the Army; published in Journal of the Hydraulics Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. HY 9, September 1959, Paper No. 2148, pp. 35–67.

[4] For important work and where accurate determination of water profiles is necessary, the designer is urged to consult the following references and to select n by comparison of the specific conditions with the channels tested: *Flow of Water in Irrigation and Similar Canals*, by F. C. Scobey, U.S. Department of Agriculture, Technical Bulletin No. 652, February 1939. *Flow of Water in Drainage Channels*, by C. E. Ramser, U.S. Department of Agriculture, Technical Bulletin No. 129, November 1929.

[5] *Handbook of Channel Design for Soil and Water Conservation*, prepared by the Stillwater Outdoor Hydraulic Laboratory in cooperation with the Oklahoma Agricultural Experiment Station, published by the Soil Conservation Service, U.S. Department of Agriculture, Publ. No. SCS-TP-61, March 1957, rev. June 1954.

[6] *Flow of Water in Channels Protected by Vegetative Linings*, by W. O. Ree and V. J. Palmer, Division of Drainage and Water Control, Research, Soil Conservation Service, U.S. Department of Agriculture, Tech. Bull. No. 967, February 1949.

[7] For calculations of stage or discharge in natural stream channels, it is recommended that the designer consult the local District Office of the Surface Water Branch of the U.S. Geological Survey, to obtain data regarding values of n applicable to streams of any specific locality. Where this procedure is not followed, the table may be used as a guide. The values of n tabulated have been derived from data reported by C. E. Ramser (see footnote 4) and from other incomplete data.

[8] The tentative values of n cited are principally derived from measurements made on fairly short but straight reaches of natural streams. Where slopes calculated from flood elevations along a considerable length of channel, involving meanders and bends, are to be used in velocity calculations by the Manning formula, the value of n must be increased to provide for the additional loss of energy caused by bends. The increase may be in the range of perhaps 3 to 15 percent.

[9] The presence of foliage on trees and brush under flood stage will materially increase the value of n. Therefore, roughness coefficients for vegetation in leaf will be larger than for bare branches. For trees in channels or on banks, and for brush on banks where submergence of branches increases with depth of flow, n will increase with rising stage.

Table 3.—Maximum permissible velocities in erodible channels, based on uniform flow in continuously wet, aged channels [1]

Material	Maximum permissible velocities for—		
	Clear water	Water carrying fine silts	Water carrying sand and gravel
	F.p.s.	*F.p.s.*	*F.p.s.*
Fine sand (noncolloidal)	1.5	2.5	1.5
Sandy loam (noncolloidal)	1.7	2.5	2.0
Silt loam (noncolloidal)	2.0	3.0	2.0
Ordinary firm loam	2.5	3.5	2.2
Volcanic ash	2.5	3.5	2.0
Fine gravel	2.5	5.0	3.7
Stiff clay (very colloidal)	3.7	5.0	3.0
Graded, loam to cobbles (noncolloidal)	3.7	5.0	5.0
Graded, silt to cobbles (colloidal)	4.0	5.5	5.0
Alluvial silts (noncolloidal)	2.0	3.5	2.0
Alluvial silts (colloidal)	3.7	5.0	3.0
Coarse gravel (noncolloidal)	4.0	6.0	6.5
Cobbles and shingles	5.0	5.5	6.5
Shales and hard pans	6.0	6.0	5.0

[1] As recommended by Special Committee on Irrigation Research, American Society of Civil Engineers, 1926, for channels with straight alinement. For sinuous channels multiply allowable velocity by 0.95 for slightly sinuous, by 0.9 for moderately sinuous channels, and by 0.8 for highly sinuous channels (*45*, p. 1257).

Table 4.—Maximum permissible velocities in channels lined with uniform stands of various grass covers, well maintained [1] [2]

Cover	Slope range	Maximum permissible velocity on—	
		Erosion-resistant soils	Easily eroded soils
	Percent	*f.p.s.*	*f.p.s.*
Bermudagrass	0–5	8	6
	5–10	7	5
	Over 10	6	4
Buffalograss Kentucky bluegrass Smooth brome Blue grama	0–5	7	5
	5–10	6	4
	Over 10	5	3
Grass mixture	0–5 [3]	5	4
	5–10 [3]	4	3
Lespedeza sericea Weeping lovegrass Yellow bluestem Kudzu Alfalfa Crabgrass	0–5 [4]	3.5	2.5
Common lespedeza [5] Sudangrass [5]	0–5 [4]	3.5	2.5

[1] From *Handbook of Channel Design for Soil and Water Conservation*. (See footnote 5, table 2.)

[2] Use velocities over 5 f.p.s. only where good covers and proper maintenance can be obtained.

[3] Do not use on slopes steeper than 10 percent.

[4] Use on slopes steeper than 5 percent is not recommended.

[5] Annuals, used on mild slopes or as temporary protection until permanent covers are established.

Table 5.—Classification of vegetal covers as to degree of retardance [1]

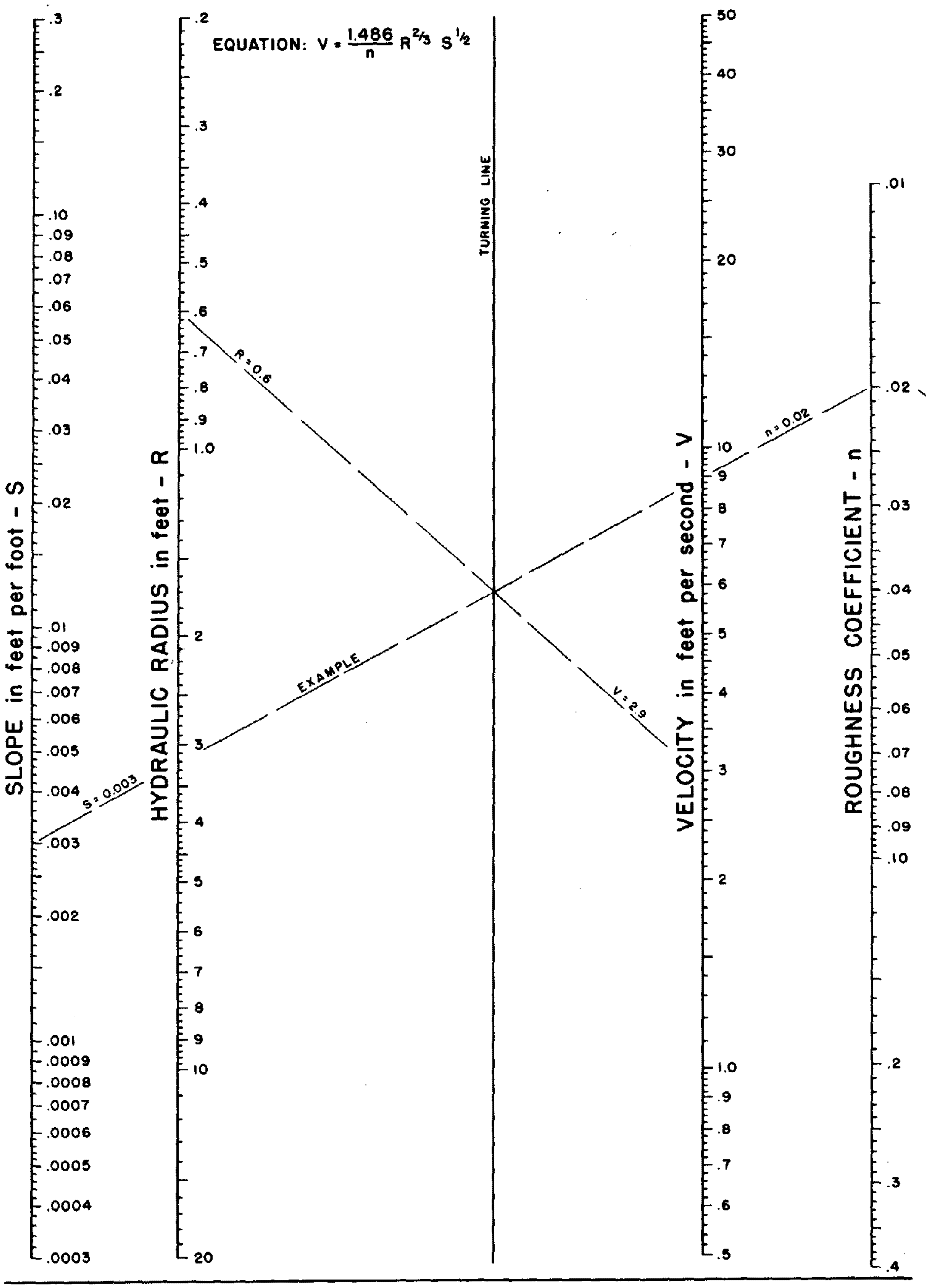
[Note: Covers classified have been tested in experimental channels. Covers were green and generally uniform]

Retardance	Cover	Condition
A	Weeping lovegrass	Excellent stand, tall (average 30 inches).
	Yellow bluestem Ischaemum.	Excellent stand, tall (average 36 inches).
B	Kudzu	Very dense growth, uncut.
	Bermudagrass	Good stand, tall (average 12 inches).
	Native grass mixture (little bluestem, blue grama, and other long and short midwest grasses).	Good stand, unmowed.
	Weeping lovegrass	Good stand, tall (average 24 inches).
	Lespedeza sericea	Good stand, not woody, tall (average 19 inches).
	Alfalfa	Good stand, uncut (average 11 inches).
	Weeping lovegrass	Good stand, mowed (average 13 inches).
	Kudzu	Dense growth, uncut.
	Blue grama	Good stand, uncut (average 13 inches).
C	Crabgrass	Fair stand, uncut (10 to 48 inches).
	Bermudagrass	Good stand, mowed (average 6 inches).
	Common lespedeza	Good stand, uncut (average 11 inches).
	Grass-legume mixture—summer (orchard grass, redtop, Italian ryegrass, and common lespedeza).	Good stand, uncut (6 to 8 inches).
	Centipedegrass	Very dense cover (average 6 inches).
	Kentucky bluegrass	Good stand, headed (6 to 12 inches).
D	Bermudagrass	Good stand, cut to 2.5-inch height.
	Common lespedeza	Excellent stand, uncut (average 4.5 inches).
	Buffalograss	Good stand, uncut (3 to 6 inches).
	Grass-legume mixture—fall, spring (orchardgrass, redtop, Italian ryegrass, and common lespedeza).	Good stand, uncut (4 to 5 inches).
	Lespedeza sericea	After cutting to 2-inch height. Very good stand before cutting.
E	Bermudagrass	Good stand, cut to 1.5 inches height.
	Bermudagrass	Burned stubble.

[1] From *Handbook of Channel Design for Soil and Water Conservation.* (See footnote 5, table 2.)

Appendix B

Nomograph for solution of Manning Equation



(See sec. 3.2-2)

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PREFACE

This publication, the fourth in a series published by the Federal Highway Administration on the hydraulic design of highway drainage structures, contains methods of open-channel design including determination of the size of channel and protection required to prevent erosion. Principles and procedures are explained but no set of rules can be furnished that will apply to all of the many diverse combinations of topography, soil, and climate that exist where highways are built. Design of roadside drainage channels will continue to require an engineer well versed in hydraulic theory and in highway drainage practice. The open-channel flow charts of Hydraulic Design Series No. 3 and hydraulic tables such as those of reference 28 will greatly reduce the work of computing channel capacity.

This reprint is identical with the 1965 edition of Hydraulic Design Series No. 4 with the exception of a redesigned cover, revised preface, and updated references.