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Preface

The purpose of this book is to provide a professional text for agricultural engineering students. The science of soil and water conservation engineering has developed to a point where more comprehensive text material is needed. Recent research has been carried out largely by the Soil Conservation Service, Agricultural Research Service, Bureau of Reclamation, state agricultural experiment stations, and other state and federal agencies. It is important that material found in professional journals, bulletins, handbooks, technical pamphlets, books, etc., be brought together in a form suitable for classroom teaching and field use.

This book includes subject matter on the five engineering phases of soil and water conservation as well as on hydrology and soil physics. The first chapter covers the general aspects of soil and water conservation engineering; Chapters 2 through 4, hydrology; Chapter 5, soil physics; and Chapters 6 through 22, erosion and its control, earth dams, flood control, drainage, irrigation, and land clearing. Although land clearing, irrigation, and flood control have not been given as much space as erosion control and drainage, many aspects of these subjects are included in other chapters. The irrigation chapter is limited primarily to sprinkler systems because schools that wish to give additional emphasis to irrigation have adequate textbooks available.

We have assumed in writing this text that the student has taken such basic courses as surveying, mechanics, hydraulics, and soils. However, a knowledge of these subjects is not essential for understanding many portions of the text. In presenting the subject, we have attempted to emphasize the analytical approach supplemented with sufficient field data to point out practical applications. Although stressing principles rather than tables, charts, and diagrams, the text may provide considerable basic data for practicing engineers as well. Class problems and examples have been included to emphasize design principles and to facilitate an understanding of the subject matter.

Although a thorough review of the literature was made, only the more important references are included at the end of each chapter. Where specific information is cited and where material pertinent to but not included in the text is mentioned, the reference for this material is generally indicated by superscripts.

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Ames, Iowa
January, 1955

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R. K. F. T. W. E.
G. O. S. K. K. B.

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January, 1955

Contents

Abbreviations

acre-ft	acre-feet
Agron.	Agronomy
ASAE	American Society of Agricultural Engineers
ASCE	American Society of Civil Engineers
ASTM	American Society for Testing Materials
cons.	conservation
cfm	cubic feet per minute
cfs	cubic feet per second
D.C.	drainage coefficient
dhp	drawbar horsepower
dia.	diameter
exp.	experiment
fpm	feet per minute
fps	feet per second
geophys.	geophysical
gpm	gallons per minute
GPO	Government Printing Office
hp	horsepower
L.F.	load factor
iph	inches per hour
mimeo.	mimeographed
mph	miles per hour
P.C.	point of curvature
pcf	pounds per cubic foot
P.I.	point of intersection
ppm	parts per million
P.T.	point of tangency
publ.	publication
res.	research
serv.	service
SCS	Soil Conservation Service
soc.	society
t/a	tons per acre
V.I.	vertical interval

Signs and Symbols

a	cross-sectional area; constant
A	watershed area in acres
b	constant; width
b_n	width of notch
b_w	width of waterway
B_c	outside diameter
B_d	width of trench
c	cut; chord length
C	coefficient; conservation practice factor
d	diameter; depth; dry density; distance
d_c	critical depth
d_i	inside diameter
d_w	wet density
D	diameter; depth; runoff; degree of curvature

D_f	length of exposure of water surface
e	void ratio; distance; deflection angle; vapor pressure
E	efficiency; specific energy head; degree of erosion factor
f	infiltration rate; hydraulic friction factor; depth; monthly evapo-transpiration factor
F	total infiltration; fertility factor; Froude number
g	acceleration of gravity
G	specific gravity of solids
G_a	apparent specific gravity
G_f	specific gravity of fluids
h	head; wave height
H	total head; height; total head loss including friction
H_f	friction head loss
i	rainfall intensity; inflow rate
I	total rainfall; angle of intersection
I_c	impact coefficient
k	constant; permeability; time conversion factor; capillary conductivity
K	constant; permeability; evapo-transpiration coefficient; soil factor; conductivity
K_c	head loss coefficient for pipe and square conduits
K_e	entrance head loss coefficient
K_s	Scobey's coefficient of retardation
L	length
m	exponent; moisture content
M	watershed area in square miles
n	roughness coefficient; porosity
o	outflow rate
p	wetted perimeter
pF	logarithm of soil moisture potential
P	power; pressure; peak runoff rate; rainfall
q	seepage rate; sprinkler discharge rate
Q	discharge; runoff rate
r	scale ratio (prototype to model); radius
R	hydraulic radius; radius; rainfall factor; rotation factor
s	slope in feet per foot; rate of storage; distance
S	slope in per cent; storage; settlement; sprinkler spacing
t	mean monthly temperature; thickness; time; width
T	conversion time interval; concentrated surface load; tangent distance; width
T_c	time of concentration
u	monthly evapo-transpiration; volume conversion factor
U	seasonal evapo-transpiration
v	velocity; rate of capillary movement
v_t	threshold velocity
V	volume; rate of soil moisture movement
w	unit weight of soil; flow conversion factor
W	weight; top width of dam; watershed characteristics

W_c soil load on conduits
W_d dry weight of soil
W_f volume of water pumped
W_s volume of water stored in the root zone
W_t load on conduits due to a concentrated surface load
W_w wet weight of soil
X soil loss
X_a annual soil loss
y depth
z side slope ratio (horizontal to vertical); depth; height above soil surface
Z vertical distance
θ side slope angle
ρ density
μ dynamic viscosity
ϕ soil moisture potential
ψ gravitational potential

CHAPTER 1

Introduction

Soil and water conservation engineering is the application of engineering principles to the solution of soil and water management problems. The conservation of these vital resources implies *utilization without waste* so as to make possible a high level of production which can be continued indefinitely.

The engineering problems involved in soil and water conservation may be divided into the five following phases: erosion control, drainage, irrigation, flood control, and land clearing. Although soil erosion takes place even under virgin conditions, the problems to be considered are caused principally by man's removal of the protective cover of natural vegetation. Drainage is the removal of excess water from wet land; irrigation is the application of water to land having a deficiency of moisture for optimum crop growth. Flood control consists of the prevention of overflow on low land and the reduction of flow in streams during and after heavy storms. Land clearing includes the removal of trees, stumps, brush, or stones from otherwise tillable land. The two principal ways of increasing crop production are to develop new land not now in production and to improve the productivity of present cropland. The development of new land is brought about primarily by drainage, irrigation, and land clearing. However, all five phases are applicable to the improvement of land already in production.

1.1. Agricultural Engineers in Soil and Water Conservation. Sound soil and water conservation is based upon the full integration of engineering, plant, and soil sciences. The agricultural engineer because of his training in soils, plants, and other basic agricultural subjects, in addition to his engineering background, is well suited to carrying out the integration of these three sciences. To carry out this plan the engineer must have a knowledge of the soil including its physical and chemical characteristics as well as a sound over-all viewpoint. All professional groups should have an appreciation of each other's problems and should cooperate to the fullest extent since few problems can be solved within the limits of any one profession.

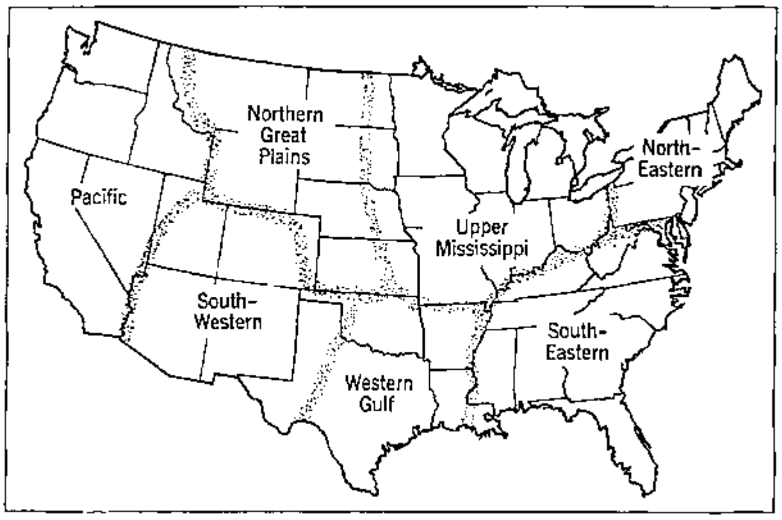

Fig. 1.1. Soil and water conservation regions of the United States.

To be fully effective in applying technical training, the agricultural engineer must also acquaint himself with the social and economic backgrounds that relate to soil and water conservation.[4,5,6,13,14] He must have a full understanding of the various governmental structures and mechanisms that have been developed to implement sound soil and water conservation programs. A number of references will provide this background material.[2,4,10,16,21,22] The agricultural engineer should also become familiar with the principles of mapping and classifying land for its use in accordance with its capabilities.[11,12,17,30]

1.2. Soil and Water Conservation Regions. For purposes of making various recommendations for conservation practices, it is desirable to subdivide the United States into seven geographical areas. These areas are shown in Fig. 1.1. It should not be concluded that climatic and soil conditions within the same area are uniform. In later sections of the book, the designation of these geographical areas refers to this subdivision.

SOIL EROSION CONTROL

The control of soil erosion caused by water and by wind is of great importance in the maintenance of crop yields. It is

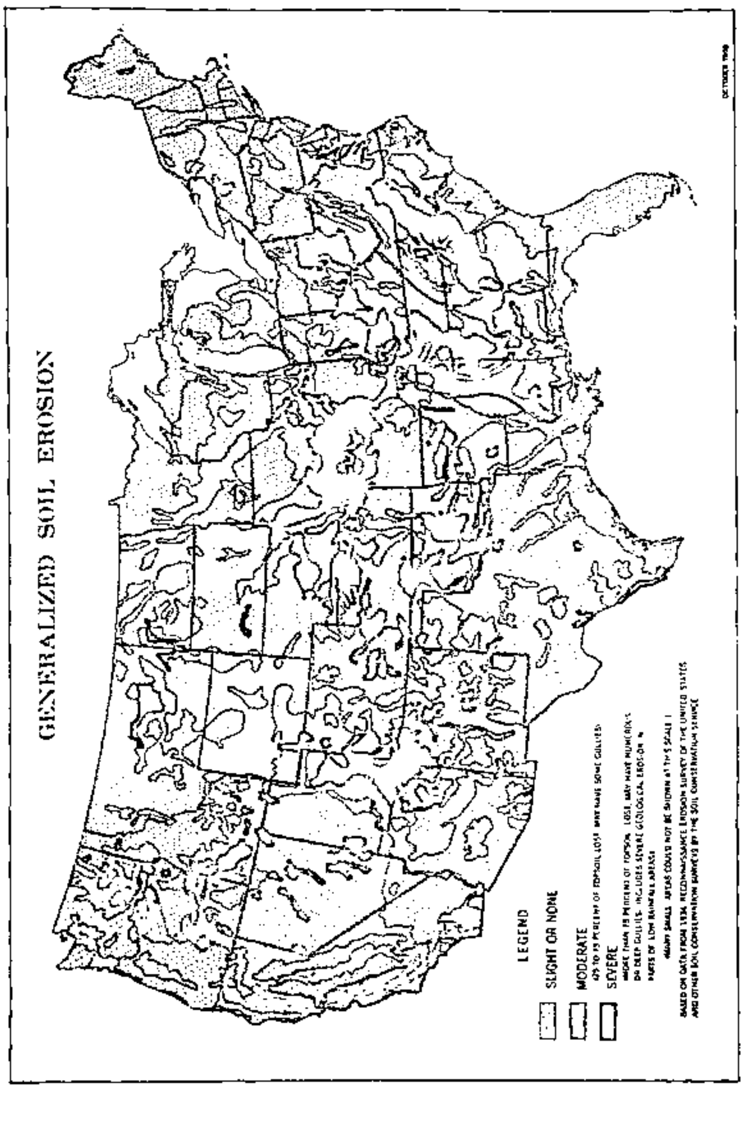

Fig. 1.2. Distribution of soil erosion in the United States. (Courtesy Soil Conservation Service.)

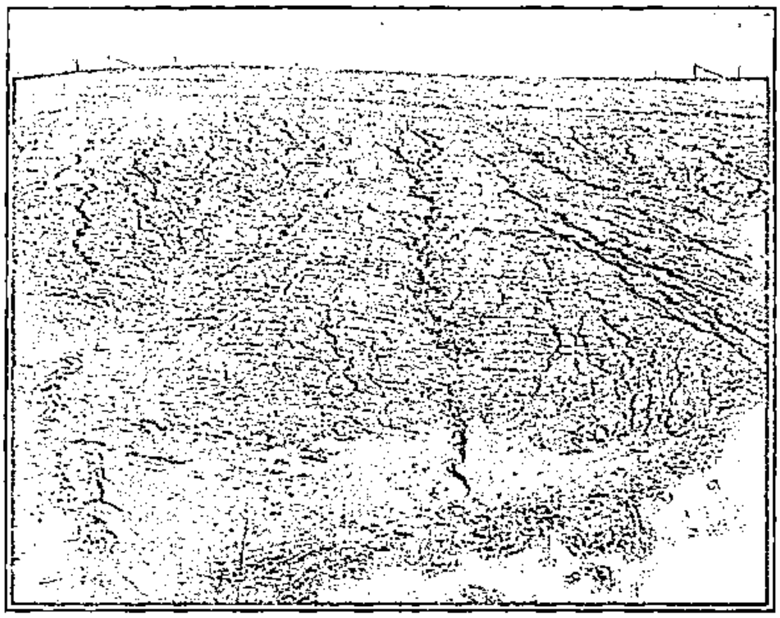

Fig. 1.3. Example of soil erosion on cropland. (Courtesy Soil Conservation Service.)

estimated from available measurements that at least 3 billion tons of soil are washed out of the fields and pastures of the nation every year.[3] Assuming a weight of 80 pcf, this quantity of soil represents a volume equivalent to a depth of 1 foot on 21,000 80-acre farms. In addition to these losses by water there are also large losses due to wind erosion. Not only is soil lost in the erosion process but also a porportionally higher percentage of plant nutrients, organic matter, and fine soil particles in the removed material is lost than in the original soil.

The relative degree of erosion and its distribution in the United States are indicated in Fig. 1.2. This map shows areas having slight, moderate, or severe erosion and does not differentiate between that caused by water and that caused by wind. Many small areas where severe erosion may occur locally cannot be shown on a map of this scale. An erosion problem is shown in Fig. 1.3.

As a result of many years of experience and the collection of much research data, the following erosion control practices are recommended where applicable: (1) performing all planting, tillage, and harvesting operations on or nearly on the contour; (2) planting close-growing and intertilled crops in alternate strips; (3) constructing cross-slope channels (terraces) to carry the water off at reduced velocities; (4) planting belts of trees or constructing other barriers for protection from wind erosion; (5) using crop residues either on the surface or incorporated in the topsoil with different tillage methods; (6) establishing permanent vegetation in waterways and other eroded areas; and (7) stabilizing gullies with suitable structures.

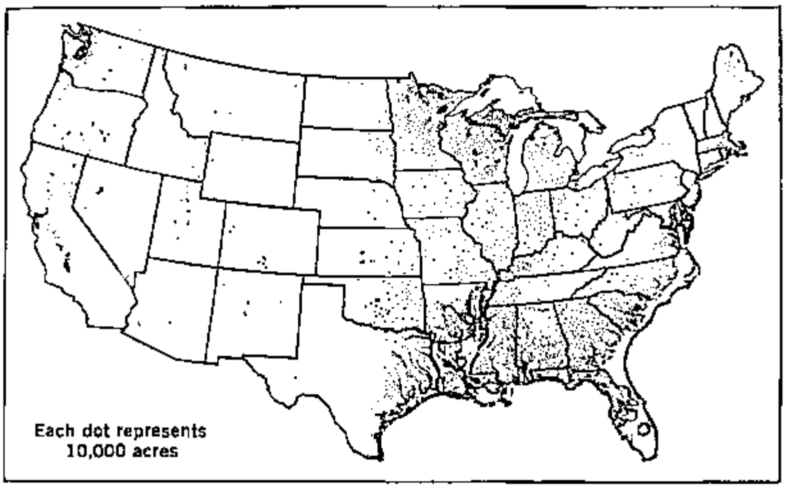

Fig. 1.4. Distribution of drainable wet land. (Courtesy U. S. Department of Agriculture.)

DRAINAGE

Probably not more than 75,000,000 acres can be drained at a cost economical for growing cultivated crops.[21] About 68 per cent[21] of this acreage requires land clearing, and some land that is too low in fertility may be more suitable for wildlife than for cultivation. Investigations show that the ultimate *equivalent* cropland acreage susceptible of development by land clearing and drainage is 31,000,000 acres.[19] The distribution of drainable wet land in the United States is shown in Fig. 1.4. About two-thirds of this wet land is located in the South, and about one-sixth is in Michigan, Wisconsin, and Minnesota.[21] An area needing drainage is shown in Fig. 1.5.

In removing excess water from the land it is usually necessary to use either surface ditches or tile drains or a combination of both. Wet land is usually flat, has high fertility, and does not have serious erosion problems. Where two or more landowners

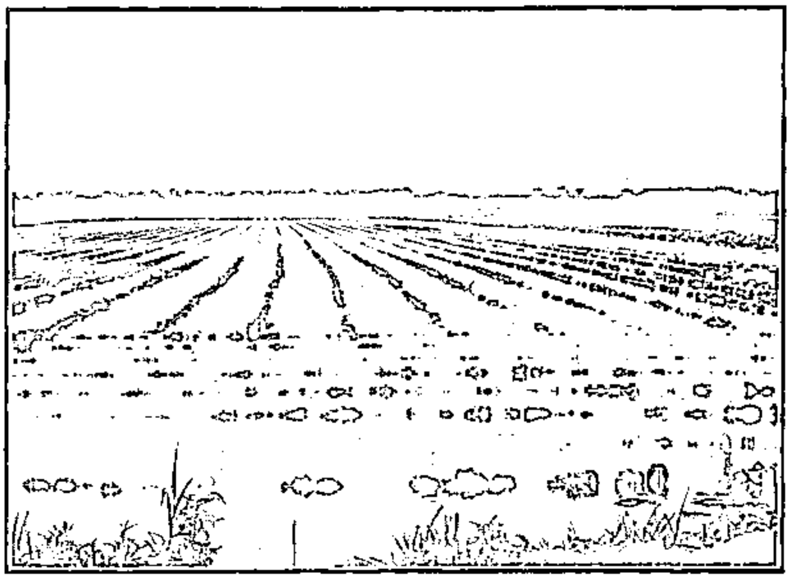

Fig. 1.5. A drainage problem. (Courtesy Soil Conservation Service.)

are involved, organized drainage districts may be formed to obtain outlets for drainage systems. Such drainage enterprises have been developed principally in (1) the prairie and level uplands of the Midwest, (2) the bottom lands of the Mississippi Valley, (3) the bottom lands in the Piedmont and hill areas of the South, (4) the coastal plains of the East and South, and (5) the irrigated areas of the West.

In 1946 the Committee on Drainage of the American Society of Agricultural Engineers[1] reported that 29,000,000 acres in organized drainage enterprises required improved drainage, 20,000,000 acres outside of drainage enterprises could be developed by organized or community drains, and 8,000,000 acres of irrigated land are in need of drainage, thus a total of 57,000,000 acres. In addition for many areas in humid regions individual farm drainage systems are required.

IRRIGATION

Irrigation provides one of the greatest possibilities for increasing potential production. Although it is most extensive in the West, more and more irrigation is being carried out in the

Fig. 1.6. A corn crop in need of irrigation. (Courtesy Agricultural Research Service.)

eastern states. Where the annual rainfall is less than 10 inches, irrigation is a necessity; where rainfall is from 10 to 20 inches, crop production is limited unless the land is irrigated; and where rainfall is more than 20 inches irrigation is often required for maximum production.

A field in need of irrigation is shown in Fig. 1.6. The develop-

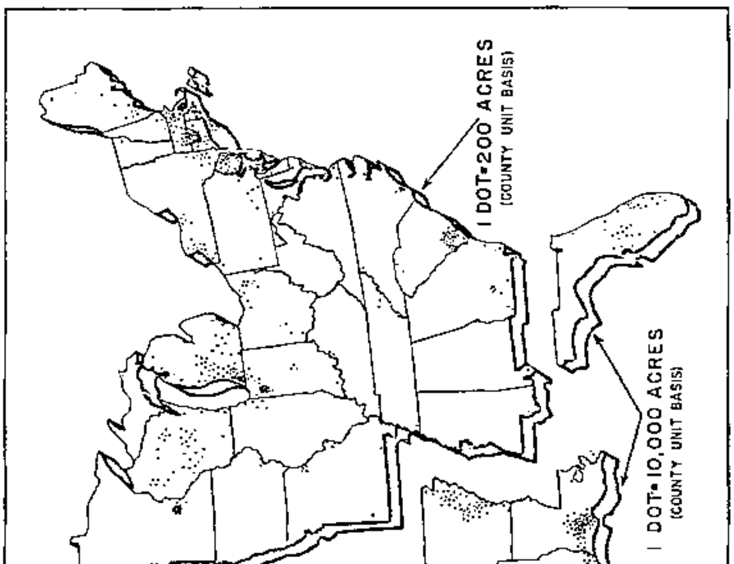

Fig. 1.7. Distribution of irrigated farmland in the United States, 1949. (Courtesy Bureau of the Census.)

ment of such land is possible only where water supplies are adequate. Reclamation of arid land often requires the removal of brush, smoothing of the surface, and provisions for draining the land at the time of development or at some future date.

Irrigation is limited largely by the available water supply. Relatively large quantities of water are required to satisfy the needs of the crop and to supply evaporation and seepage losses. In the 17 western states an average of 4.5 acre-ft of water is required at the source to irrigate each acre of land.[29] It is estimated that only about 1.5 acre-ft per acre is actually used by the plant, the remaining 3.0 acre-ft being lost by evaporation and seepage.

The distribution of irrigated land in the United States is shown in Fig. 1.7. In the 29 eastern states, excluding Arkansas and Louisiana, about 518,000 acres were irrigated in 1949. Present trends indicate that this acreage has greatly increased. Portable, light-weight sprinkling equipment is being used on much of the irrigated land in the East and on considerable acreage in the West.

FLOOD CONTROL

The agricultural engineer is primarily concerned with floods which occur in headwater areas of less than 1000 square miles. Since downstream floods on major tributaries are more spectacular and damages are more evident, floods in headwaters have too often been neglected. The total flood losses increase with the size of the drainage area, but the losses per unit area decrease. Damage from floods in headwater areas is primarily on agricultural land; downstream floods cause major damage to metropolitan areas. Flood damage to agricultural land is shown in Fig. 1.8. Damaging floods of greater or less degree occur on some streams every year.

The principal headwater flood control measures include proper watershed management and the storage of water in small reservoirs. Proper watershed control measures reduce runoff, and they also result in a corresponding decrease in soil loss. Headwater flood control programs are also concerned with such related activities as drainage, irrigation, gully and streambank erosion control, and land clearing.

Fig. 1.8. Flood damage to agricultural land.
(Courtesy Soil Conservation Service.)

Fig. 1.9. Land that may be put into production through clearing.
(Courtesy Soil Conservation Service.)

LAND CLEARING

Forest land suitable for development by clearing covers large areas. In addition to forest land many acres of idle, brush-covered, cutover, and stony land could be developed. Much of the area requiring clearing must also be drained before being put into production. An area in need of clearing is indicated in Fig. 1.9.

As an indication of the extent of land-clearing operations in the United States, 2,400,000 acres were cleared under the agricultural conservation program from 1941 to 1950.[28]

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CHAPTER 2

Precipitation

Precipitation, along with the atmospheric phenomena of heat, moisture, and air movement, is a part of the science of meteorology. This science of weather is of particular interest to those concerned with the effective use of soil and water. The weather is often the controlling factor in problems of preventing excessive movement of soil, of retaining needed moisture, of increasing the intake of surface water, of adding needed water by irrigation, and of removing excess water by drainage. Moisture, whether too much, too little, or poorly distributed, is one of the major limitations in agricultural production. Though everyone is concerned with the weather, it is thus of particular advantage to the specialist in this field to be familiar with the general principles of meteorology.[3,18]

2.1. The Hydrologic Cycle. The science of meteorology is a part of the much broader field of hydrology, which includes the study of water as it occurs in the atmosphere as well as on and below the surface of the earth. One representation of the hydrologic cycle is given in Fig. 2.1. It shows the formation of precipitation, which may occur as rain, snow, sleet, or hail. Some of this precipitation evaporates partially or completely before reaching the ground; some precipitation changes from one form to another before reaching the earth's surface. Precipitation reaching the earth's surface may be intercepted by vegetative material, it may infiltrate the surface of the ground, or it may evaporate. Evaporation may be from the surface of the ground, from free water surfaces, or from the leaves of plants through transpiration. A portion of the total rainfall moves over the earth's surface as runoff while another portion moves into the soil surface, is used by vegetation, becomes part of the deep ground water supply, or seeps slowly to streams and to the ocean.

Figure 2.1 shows also the measurements which are commonly made of those portions of the hydrologic cycle of special interest to agricultural engineers. These include the measurements of precipitation by rain or snow gages, the measurement of accumulated snow by snow surveys over established ranges, the measurement of runoff by gaging stream channels, and the measurement of ground water levels. These ground water levels may be measurements either of the deep water tables as indicated by the height the water rises in wells or of the shallower perched water tables of particular interest in analyzing drainage problems.

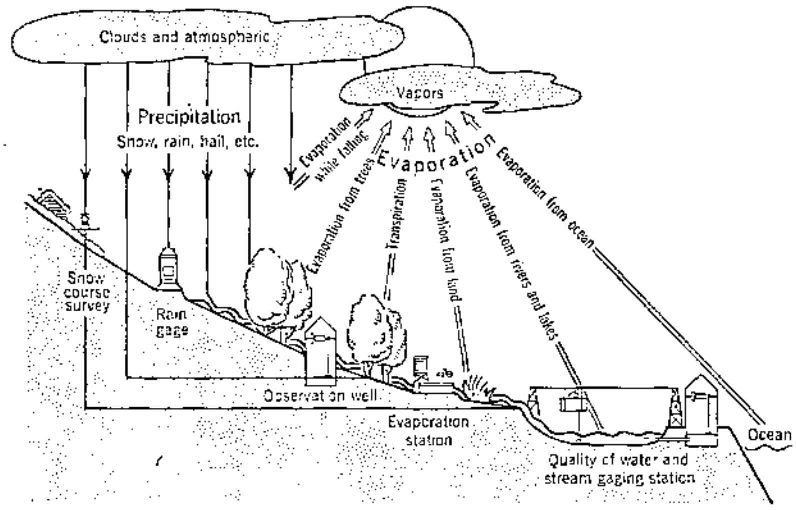


Fig. 2.1. The hydrologic cycle and its measurement. (Redrawn from Saville.[20])

OCCURRENCE OF PRECIPITATION

2.2. Forms. Precipitation may occur in any of a number of forms and may change from one form to another during its descent. The forms of precipitation consisting of falling water droplets may be classified as drizzle or rain. Drizzle consists of quite uniform precipitation with drops less than 0.5 millimeter in diameter. Rain consists of generally larger particles.

Precipitation may also occur as frozen water particles including snow, sleet, and hail. Snow is composed of a grouping of small ice crystals known as snowflakes. Sleet forms when raindrops are falling through air having a temperature below freezing; a hail stone is an accumulation of many thin layers of ice over a snow pellet. Of the forms of precipitation, rain and snow make the greatest contribution to our water supply.

Moisture at the soil surface is also made available by direct condensation and absorption from the atmosphere, commonly referred to as dew. Studies of the amounts and rates of dew formation in Ohio[6] showed that such condensation and absorption generally occurs in the evening when the ground begins to cool, and that very little moisture accumulates after midnight. Monthly averages of dew formation measured over a period of 6 years varied from a low of 0.1 inch in July to a high of 0.42 inch in December. The total average condensation absorption on rotation plots for the 6-month period from April through September inclusive was about 2.5 inches. Though most dew was evaporated by noon, it was shown that dew was effective in reducing the rate of soil moisture depletion.

2.3. Characteristics of Raindrops. Since by far the largest portion of precipitation occurs as rain, and since rainfall directly affects soil erosion, the characteristics of raindrops are of interest. Raindrops were found by Laws and Parsons[13] to include water particles as large as 7 millimeters in diameter. They found that the size distribution in any one storm covered a considerable range and that this size distribution varied with the rainfall intensity. Figure 2.2 gives the raindrop diameter for three of the intensities studied. Not only does the higher-intensity storm have more large-diameter raindrops but it also has a wider range of raindrop diameters.

Raindrops are not necessarily spherical, or even streamlined. Falling raindrops are deformed from spherical shape by unequal pressures, due to air resistance, developing over its surface. Large raindrops divide in the air, drops over 5 millimeters in diameter being generally unstable.

In studies of soil erosion the velocities of raindrops may be important. Laws[12] made observations on the velocity of different sizes of waterdrops falling from different heights. He found that the velocity of fall depended on the size of the particle, and that large drops fell more rapidly. As the height of fall was increased, the velocity increased only to a height of about 35 feet; the drops then approached a terminal velocity, which varied from about 15 feet per second for a 1-millimeter drop to about 30 feet per second for a 5-millimeter drop. Laws also compared the velocity of fall of the waterdrops with actual falling raindrops and found a good correlation.

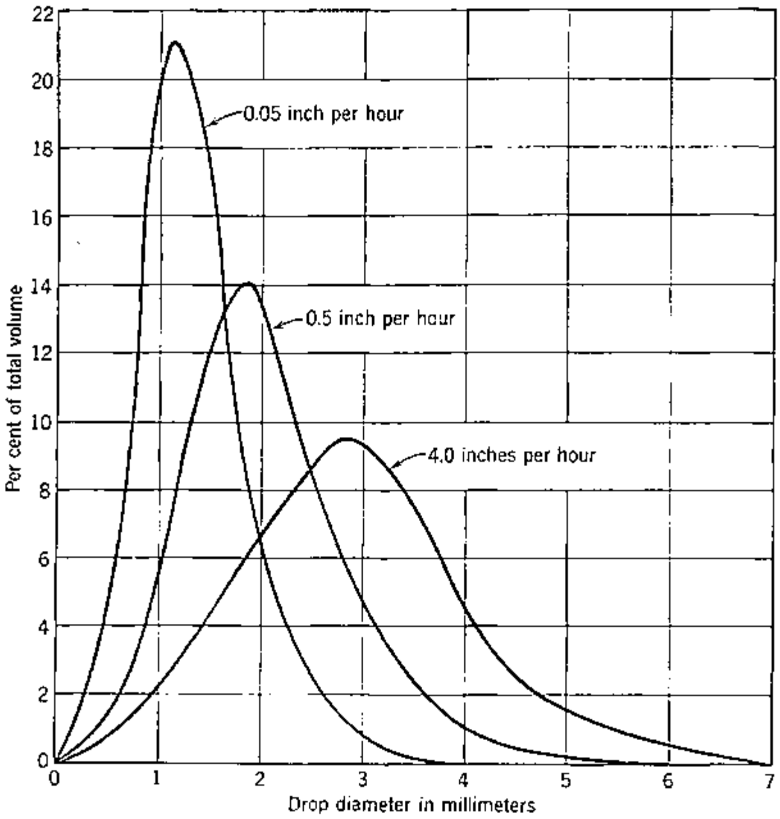


Fig. 2.2. Per cent of total volume of rainfall contributed by drops of various sizes for three rainfall rates. (Redrawn from Laws and Parsons.[13])

2.4. Air Masses. The characteristics of air masses are controlling factors in development of precipitation. Air masses are formed by continued association with specific surface and radiation conditions. Figure 2.3 gives the predominant air masses affecting the North American continent. The tropical maritime (mT) air formed over the Gulf of Mexico is subjected to considerable heating by the sun. As a result of long association with the water surface, it takes up much moisture and provides the moist warm air characteristic of the southerly winds in the central and eastern portions of the United States. By contrast the cold air masses known as polar continental (cP) air are generally formed over north-central Canada. Gigantic high-pressure centers may lie here for as long as several weeks. As these air masses have low moisture and often rest on large snow-covered areas, they have considerable negative radiation from the surface of the earth and become very cold.

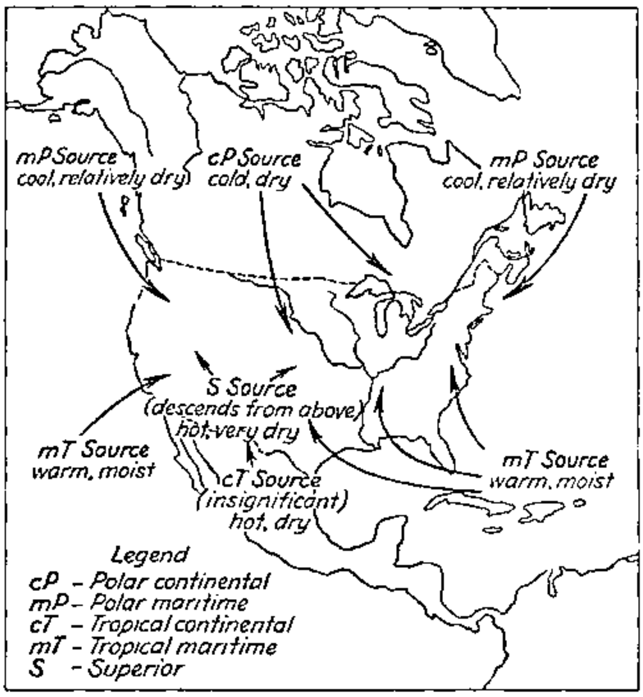


Fig. 2.3. Air masses of the North American Continent. (Redrawn from Rouse.[19])

Other important air masses in Fig. 2.3 are those denoted by S, the hot continental air formed in summer over the southwest desert areas of this country, and by mP, the polar maritime air mass, such as is formed over the northern parts of the Atlantic and Pacific oceans.

2.5. Sources of Moisture in Precipitation. The air mass that contributes the largest amounts of moisture to the central and eastern portions of the United States is the tropical maritime (mT) which moves warm moist air in from the Gulf of Mexico.

On the west coast most of the precipitation comes from the polar maritime (mP) air which carries moisture in from the Pacific Ocean. The portion of precipitation moisture originating from continental evaporation is very small.[15] That this local evaporation cannot increase local moisture has been shown.[8] For local evaporation to have this effect would require complete stagnation of air, something that seldom, if ever, occurs. Instead, most of the möisture removed by evaporation is carried away by cool, dry continental air masses.

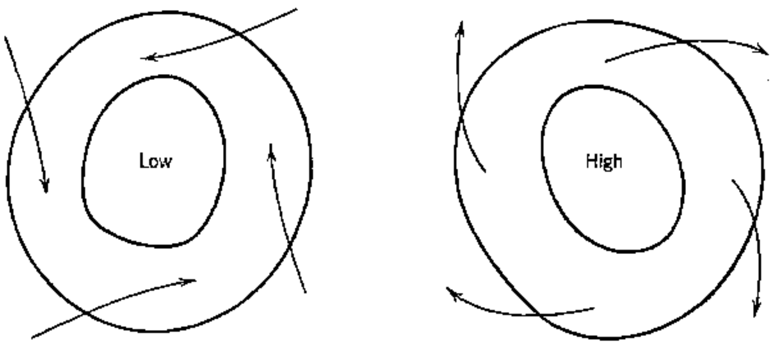


Fig. 2.4. Circulation around low- and high-pressure centers.

2.6. Influence of Land and Water Masses. Land and water masses are important in their effects on weather. Since land cools much more rapidly than water, the air masses passing over it are not so drastically changed. Also in winter the formation of snow provides an excellent opportunity for negative radiation and the resultant formation of very cold air masses. Uneven distribution of land and water areas on the earth's surface also causes deviations from the expected circulation patterns because water surfaces offer less resistance to air movement than do the generally rougher land surfaces.

2.7. Areas of High and Low Pressure. The distribution of atmospheric pressure on the earth's surface is indicated on weather maps (Fig. 2.10) by means of lines of constant pressure called isobars. In general the wind blows nearly parallel to the isobars with the friction between the moving air and the earth's surface giving the surface winds a slight component toward the area of low pressure.[3,18] Thus high- and low-pressure centers have wind directions as shown in Fig. 2.4. The wind blows counterclockwise and slightly toward the center of low-pressure areas, giving an upward movement near the center which cools the air and increases the tendency for precipitation in these low-pressure centers. Conversely the air blows clockwise and slightly out of the center of high-pressure areas, giving a downward movement in the center of the high which warms the air and tends to cause clearer skies near the high-pressure center.

STORMS

In general, there are three major types of storm precipitation. The first of them is known as a frontal storm and is associated with the conflict between the cold and warm air masses. The second is the air-mass thunderstorm, which is a result of convection over either land or water surfaces. The third is the orographic storm, in which precipitation results from the cooling of the air due to the upward movement of the air mass, especially as over a range of mountains.

2.8. Frontal Storms. Frontal storms occur at boundaries of warm moist air and relatively dry cold air. The most common such combination in central and eastern United States is between tropical maritime air from the Gulf of Mexico and polar continental air from central Canada. The greatest precipitation occurs in low-pressure centers which move along the air mass boundary. The formation of these waves or storm centers is a gradual process, and is followed by the eventual disappearance of the low-pressure area. These storm centers often develop over continental North America and frequently do not disappear until they have passed far into continental Europe.

The process of the development and eventual disappearance of a frontal storm center is illustrated diagrammatically in Fig. 2.5. The opposite wind direction at the air-mass boundary in Fig. 2.5*a* often results in a circular motion, shown in Fig. 2.5*b*. Gradually this circulation increases, and the warm moist air rides over the heavier colder air. Gradually the warm and cold fronts develop as in Fig. 2.5*c*. The cold front occurs where the cold air is able to push the warm air back, and the warm front occurs where the warm air pushes the cold air before it. Since the cold front moves more rapidly than the warm front, it over-

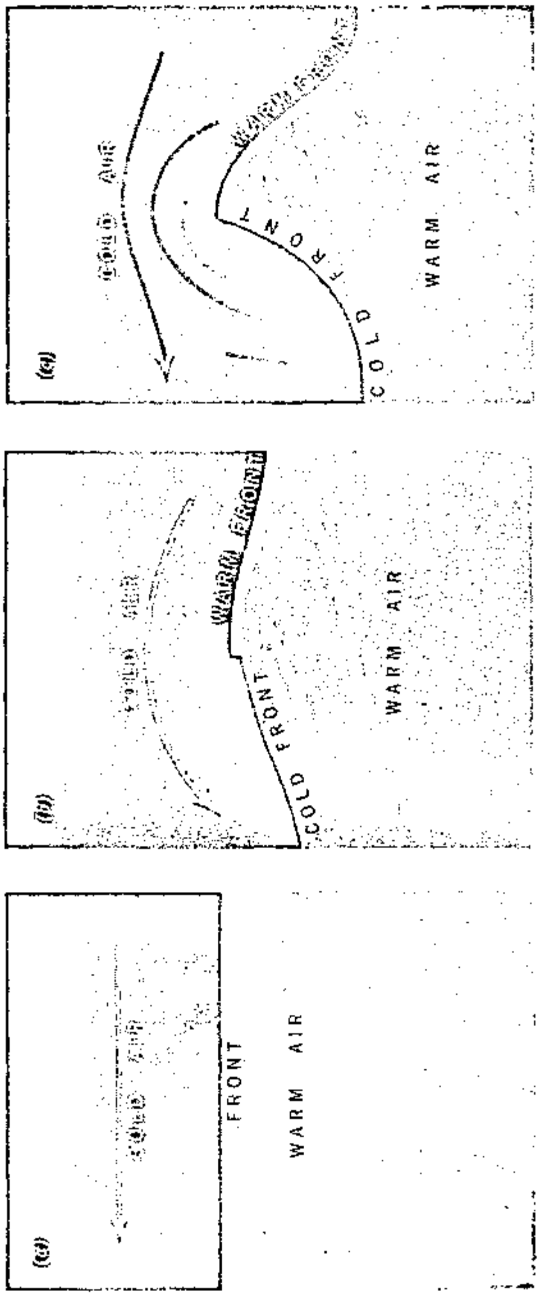


Fig. 2.5. Development and disappearance of a frontal storm center. (From reference 24.)

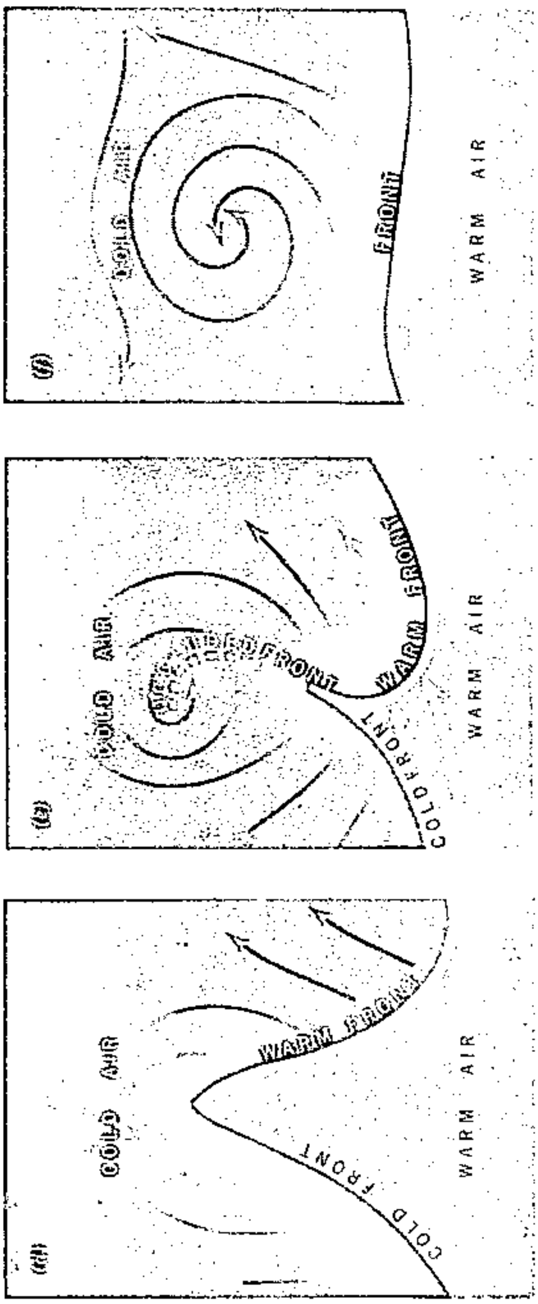


Fig. 2.5. Development and disappearance of a frontal storm center. (From reference 24.)

takes the warm front (Fig. 2.5*d*), trapping the warm air aloft and forming the occlusion illustrated in Fig. 2.5*e*. Finally, as the wave dies, merely a swirl of air (Fig. 2.5*f*) persists where the wave occurred.

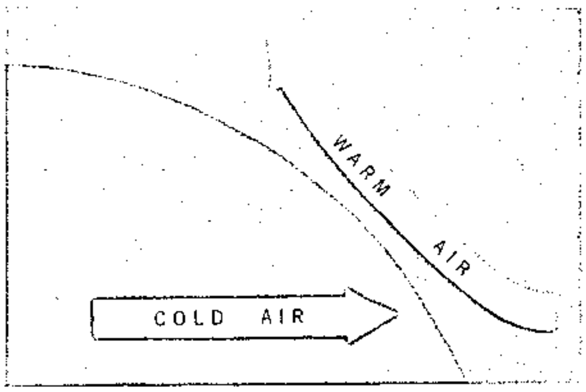


Fig. 2.6. Cold front. (From reference 24.)

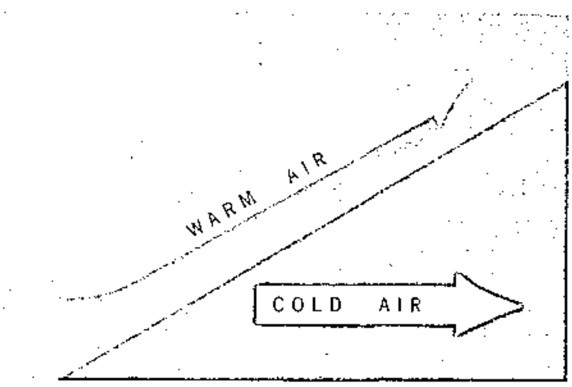


Fig. 2.7. Warm front. (From reference 24.)

Cross sections of cold, warm, and occluded fronts are shown in Figs. 2.6, 2.7, and 2.8, respectively. The slope of the cold front is steeper than that of the warm front. Thus the cold front causes a more rapid rise of the warm moist air and develops the violent thunderstorm type of precipitation. The more gentle slope of the warm front results in more uniform precipitation varying from light drizzles to heavy rains. This warm-front precipitation is generally of longer duration and covers larger areas than does cold-front precipitation.

In some cases an additional type of front known as the stationary front forms, allowing the warm air to continually push over the cold air and causing continued precipitation. In many cases waves, causing large areas of precipitation, form and move along stationary fronts.

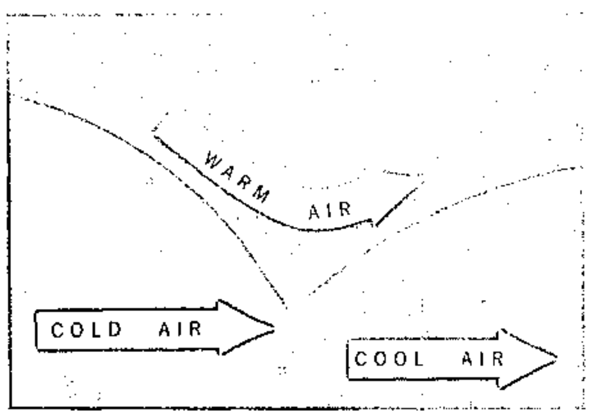


Fig. 2.8. Occluded front. (From reference 24.)

Both the paths followed and the velocity of movement of these storm centers are variable. Some of the paths commonly followed by these centers of low-pressure areas are given in Fig. 2.9. In general they move from 200 to 500 miles in a 24-hour period, or from 8 to 20 miles per hour.

2.9. Air Mass Storms. During the summertime the contrast between the polar continental and the tropical maritime air is not so striking. Since during these warmer months the temperature variations across frontal boundaries may be only a few degrees as compared with 10 to 30 degrees during the fall, winter, and spring months, frontal storms contribute only a small part of the summertime precipitation. During this period the air in the central and eastern states has a high moisture content and, especially near the surface, is subjected to considerable radiation heating. The heated air moves upward, being cooled both by the surrounding air and by the expansion process. When it is cooled to its condensation point, it forms a cloud of the convective type that may develop into a thunderstorm. Any source of ground heating, even a large fire, can set off circulation of this type. Whether such a convective circulation develops into a thunderstorm depends upon the variation of temperature with height and the moisture conditions in the atmosphere. Though these storms generally cause precipitation only over small areas, they may result in very intense precipitation and are particularly important causes of floods on small watersheds.

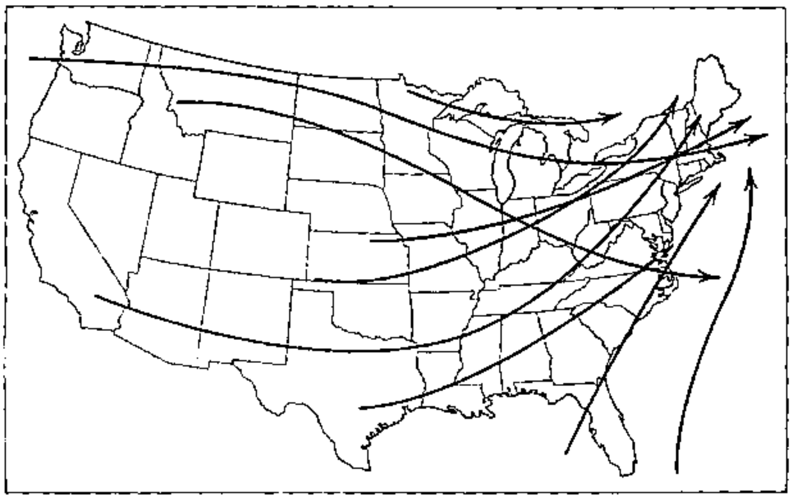


Fig. 2.9. Paths followed by storm centers moving over continental United States.

2.10. Orographic Storms. The influence of topography on precipitation is especially important. As air masses move over high elevations, such as mountain ranges, the air is pushed upward, cooled, and oftentimes reaches the condensation point. Thus much of the precipitation occurs on the upslope side of the mountain range. This process causes the highest annual precipitation found in North America. Conversely as the air moves downslope it is warmed and, having had most of the moisture squeezed from it, deposits little precipitation. Thus the mountain ranges along the western coast are largely responsible for the arid areas further inland. As the central states are reached, this lack of moisture is compensated by the air movement from the Gulf of Mexico, which gradually builds up the precipitation as the east coast is approached.

2.11. Weather Maps. The weather picture is commonly depicted by weather maps showing the position of the isobars, the ground position of the fronts, and the areas of precipitation. Such a weather map is shown in Fig. 2.10, along with the idealized picture of how air masses, if visible, might look to an observer viewing continental United States from a distance. The function of the weather forecaster is to prognosticate the movement of these frontal areas, their development, and the probable precipitation. As weather maps are now commonly included in daily newspapers, the individual is provided with an opportunity to practice forecasting, and to compare his predictions with those of the weather bureau.

MEASUREMENT OF PRECIPITATION

Since most estimates of runoff rates are based on precipitation data, information regarding the amounts and intensity of precipitation is of great importance.

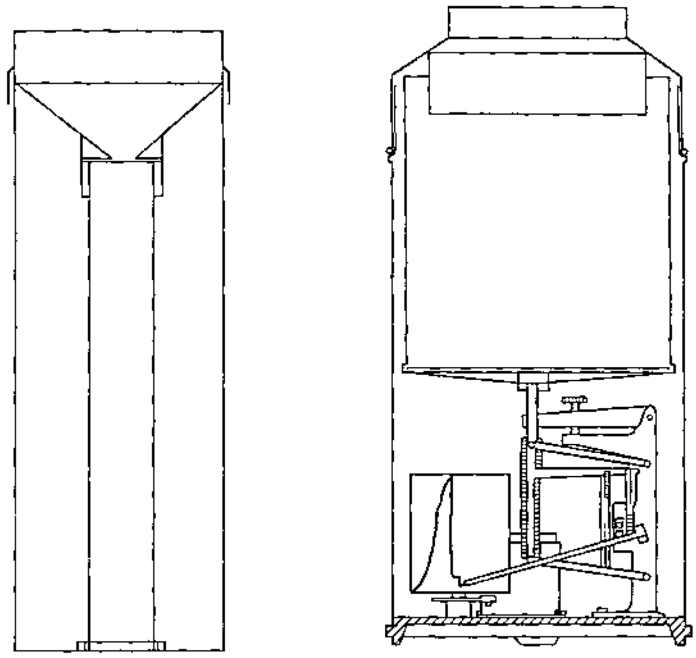
2.12. Gaging Rainfall. The purpose of the rain gage is to measure the depth and intensity of rain falling on a flat surface without considering infiltration, runoff, or evaporation. The many problems of measurements with gages include effects of topography and nearby vegetation as well as the design of the gage itself. Rain gages generally used in the United States are vertical, cylindrical containers with top openings 8 inches in diameter. A funnel-shaped hood is inserted to minimize evaporation losses.

Rain gages may be classified as recording or nonrecording. Nonrecording rain gages, such as shown in Fig. 2.11*a*, are economical, require servicing only after rains, and are relatively free of maintenance. The gage illustrated here is the Weather Bureau type. The water is funneled into an inner cylinder one-tenth of the cross section of the catch area. This provides a magnification of ten times the depth of the water and makes it possible to measure to the nearest one-hundredth of an inch.

Recording rain gages may be of several types. The type shown in Fig. 2.11*b* is the Fergusson weighing rain gage. Water is caught in a bucket placed upon a scale mechanism. The

Fig. 2.10. Frontal system over central United States with a corresponding weather map. (From reference 24.)

weight of the water compresses the spring. The amount of compression is recorded through an appropriate linkage on a chart placed on a clock-driven drum. The recording mechanism shown, which allows the needle to reverse itself three times for the full



(a) Standard weather bureau gage

(b) Fergusson weighing rain or snow gage

Fig. 2.11. (a) U. S. Weather Bureau nonrecording and (b) Fergusson weighing rain gage. (Redrawn from Middleton.[17])

compression of the spring, gives a large vertical scale and makes possible a more accurate reading of the chart. Some weighing gages do not have the reversing mechanism. There are other types of recording rain gages, most of them using either the tipping bucket or the float-and-syphon principle.[17]

2.13. Measuring Snowfall. Since the water content of freshly fallen snow varies from less than 0.5 inch to over 5 inches of water per foot of snow, snowfall is much more difficult to measure than rainfall. While this wide variation in density makes it hazardous to indicate the amount of snow by simple depth measurements, an equivalent depth of 1.2 inches of water per foot of snow is a commonly accepted mean. Water content of compacted snow, however, is often 4 to 6 inches of water per foot of depth, and in snow compacted to glacial ice there may be over 10 inches of water per foot of snow.[14]

Snowfall measurements are often made with regular rain gages, the evaporation hood having been removed. A measured quantity of some noncorrosive, nonevaporative, antifreeze material is generally placed in the rain gage to cause the snow to melt upon entrance. Since errors due to wind are more serious in measuring snowfall than in measuring rain, specially designed shields[17] are sometimes used.

Another method of measuring snowfall is by determining the depth of snow by a snow survey.[9] Such surveys are particularly common in mountainous areas where the snowfall provides valuable storage for irrigation water to be used the following summer. These snow courses consist of ranges which are sampled at specified intervals. The sampling equipment consists of specially designed tubes which take a sample of the complete depth of the snow. The sample is then weighed and the equivalent depth of water recorded. By measuring these snow courses for a period of years and comparing the equivalent water depth with the observed runoff from the snow field, one can make predictions of the amount of runoff. These predictions are of particular value in planning for the most effective use of the quantities of irrigation water available during the following summer, as well as in forecasting the probability of spring floods.

2.14. Errors in Measurement. Many errors in measurement result from carelessness in handling the equipment and in analyzing data. These errors may either increase or decrease the rainfall measurements. Errors characteristic of the nonrecording rain gage of the Weather Bureau type include the water creeping up on the measuring stick and the denting of the cans. Also, there is a 2 per cent correction[14] that may be allowed for the volume of water displaced by the measuring stick. Errors of this type are cumulative, always adding to the amount of rainfall observed.

Another class of errors is due to such obstructions as trees, buildings, and uneven topography. These errors can be minimized by proper location of the rain gages. The gages are normally placed with the opening about 30 inches above the surface of the ground. They should be located so as to minimize turbulence in the wind passing across the gage. A rule sometimes used is to have a clearance of 45 degrees from the vertical center line through the gage, but a safer rule is to be sure that the distance from the obstruction to the gage is equal to at least 2 times the height of the obstruction.

The wind velocity also affects the amount of water caught; stronger winds cause less water to enter the gage than actually falls as precipitation. Whenever possible the gage should be located on level ground as the upward or downward wind movement often found on uneven topography may easily affect the amount of precipitation caught.

2.15. The Gaging Network in the United States. Precipitation records have been kept in this country ever since it has been settled. However, only since about 1890 have recording rain gages, giving the intensity of precipitation, been used. Rain gages have steadily increased in number until the gaging network in the United States consists of about 9500 nonrecording and 3200 recording instruments. Many of the nonrecording gages are serviced by volunteer personnel; most of the recording equipment is either connected with Weather Bureau installations or used as a part of various research investigations. The results of these extensive gaging activities are given in the various publications of the United States Weather Bureau and in reports of other federal and state agencies.

ANALYSIS OF PRECIPITATION DATA

Rainfall data are of interest both in a specific locality and over considerable areas. Since a rain gage gives the precipitation at a given point, it is easier to make a point rainfall analysis than to study rainfall over an area.

2.16. Rainfall Intensity and Duration. One of the most important rainfall characteristics is rainfall intensity, usually expressed in inches per hour. Very intense storms are not necessarily more frequent in areas having a high total annual rainfall. Storms of high intensity generally last for fairly short periods and cover small areas. Storms covering large areas are seldom of high intensity and may last for several days.

The infrequent combination of relatively high intensity and long duration gives large total amounts of rainfall. These storms do much erosion damage and may cause devastating floods. These unusually heavy storms are generally associated with warm-front precipitation. They are most apt to occur when the rate of frontal movement has decreased, when other fronts may pass by at close intervals, when stationary fronts persist in an area for a considerable period, or when tropical cyclones move into the area.

2.17. Recurrence Interval. Intense rainstorms of varying duration occur from time to time over almost all portions of the United States. However, the probability of these heavy rainfalls varies with the locality. The first step in designing a water control facility is to determine the probable recurrence of storms of different intensity and duration so that an economical size of structure can be provided. For most purposes it is not feasible to provide a structure that will withstand the greatest rainfall that has ever occurred. It is often more economical to have a periodic failure than to design for a very intense storm. Where human life is endangered, however, the design should handle runoff from storms even greater than have been recorded. For these purposes data providing recurrence intervals of storms of various intensities and durations are essential. This recurrence interval, sometimes called frequency, can be defined as the period of years during which one storm of a given duration and intensity can be expected to occur.

The most widely used compilation of recurrence interval data for the United States was developed by Yarnell,[25] who prepared a series of maps of the country with isohyetals (lines of constant rainfall) which might be expected for different durations and recurrence intervals. A set of these maps for storms of 1-hour duration is given in Appendix A. These data are based on observations for 206 recording rain gages operating for an average period of 29.5 years.

In order to obtain data readily for durations other than 1 hour, Hathway[7] developed the chart given in Appendix A. The curves indicating the different durations on the chart were obtained from a systematic analysis of Yarnell's data for durations other than 1 hour.

To use the chart, the 1-hour intensity is first determined from the appropriate map in Appendix A, and this 1-hour intensity in inches per hour is converted to the rainfall intensity for the duration indicated by the appropriate curve. Thus the rainfall intensity for the geographical location, the duration, and the selected recurrence interval may be determined.

Other recurrence interval relationships based on local observations have been developed for many localities. Since Yarnell's

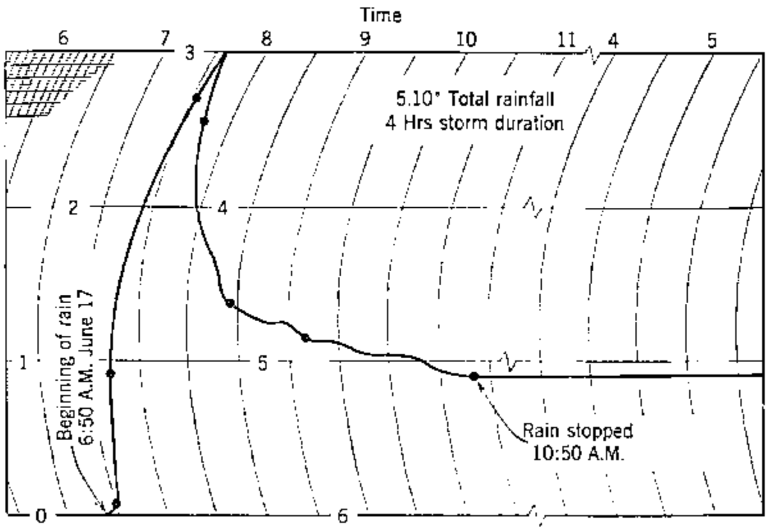


Fig. 2.12. Rain gage chart for rain gage of the reversible, recording type.

data have not been revised since 1935, such local data, when available, may be preferable. Procedures for development of such data are given in hydrology textbooks.[5,11,16]

2.18. Point Rainfall Analysis. A typical recording rain gage chart is given in Fig. 2.12. The line on the chart is a cumulative rainfall curve, the slope of the line being proportional to the intensity of the rainfall. The peak is the point of reversal of the recording gage. To analyze the chart, the time and amount of rain should be selected from representative points where the rainfall rate changes so that the data will represent the curve on the chart. These points may be tabulated as in Table 2.1, with cumulative rainfall and intensity for various periods of time also being recorded.

Table 2.1 Rain Gage Chart Analysis

Time, A.M.	*Time Interval,* min	*Cumulative Time,* min	*Rainfall during Interval,* in	*Cumulative Rainfall,* in	*Rainfall Intensity for Interval,* iph
6:50					
	10	10	0.05	0.05	0.30
7:00					
	10	20	0.41	0.46	2.46
7:10					
	5	25	0.42	0.88	5.04
7:15					
	20	45	1.82	2.70	5.46
7:35					
	10	55	0.74	3.44	4.44
7:45					
	40	95	1.20	4.64	1.80
8:25					
	45	140	0.24	4.88	0.32
9:10					
	100	240	0.22	5.10	0.13
10:50					

To determine the recurrence interval for different periods of time, the maximum amount of rain that fell during the selected interval should first be determined. By referring to Appendix A and finding the equivalent 1-hour precipitation, the appropriate recurrence interval can be determined by selecting the map in Fig. A.1 that matches the equivalent 1-hour precipitation or by interpolating for the appropriate geographical location.

Mass rainfall curves, required for some types of analyses, may be obtained by plotting the cumulative rainfall against time as in Fig. 2.13*a*. It is also often convenient to plot the rainfall intensity for increments of time as illustrated in Fig. 2.13*b*.

2.19. Classification of Storms. Since no two rainstorms have exactly the same time-intensity relationships, it is often convenient to group storms with regard to some of their characteristics. The most common characteristics used in such groupings are the intensity of the storm and the pattern of the rainfall intensity histogram.

Storms may be divided into intensity classes based upon the portion of a storm occurring within specified ranges of intensities. Such a division, making it possible to group storms expected to produce critical results under different watershed conditions, has been proposed.[21]

The pattern of a storm is determined by the arrangement of the rainfall intensity histogram. Storm patterns are important because they are one of the factors determining the runoff hydrograph. Horner and Jens[9] arbitrarily selected the four storm

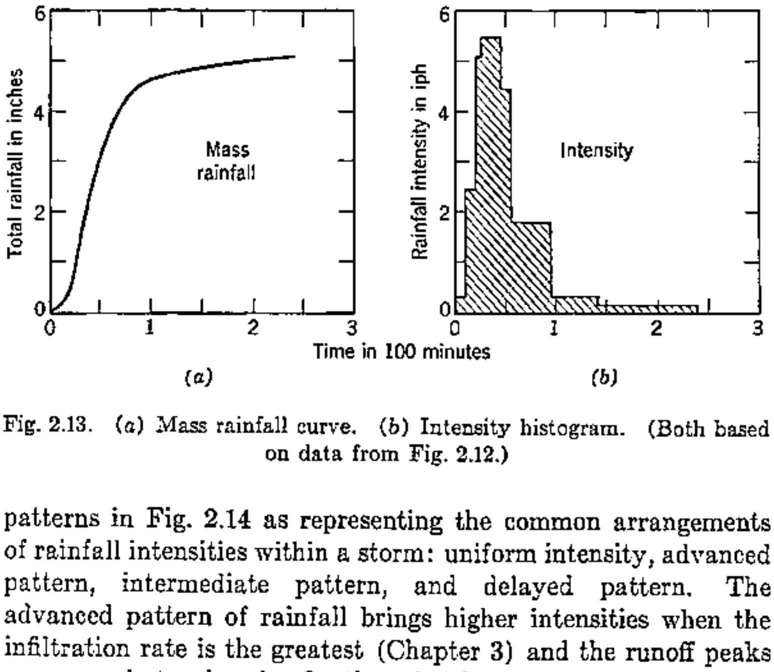


Fig. 2.13. (a) Mass rainfall curve. (b) Intensity histogram. (Both based on data from Fig. 2.12.)

patterns in Fig. 2.14 as representing the common arrangements of rainfall intensities within a storm: uniform intensity, advanced pattern, intermediate pattern, and delayed pattern. The advanced pattern of rainfall brings higher intensities when the infiltration rate is the greatest (Chapter 3) and the runoff peaks are somewhat reduced. On the other hand, the delayed pattern causes higher runoff peaks, as the high intensities occur when the infiltration is at a minimum and depression storage has been largely satisfied. In general, the cold front produces a storm of an advanced type, and the warm front a uniform or intermediate pattern. In Ohio,[21] in a study of 1-hour storms of all intensity classes, the advanced pattern was found to be the most common.

2.20. Average Depth over Area. Of the several methods of determining the average depth of precipitation over an area, where one rain gage is used, the rainfall is applied over the entire area. Where more than one gage is used, the simplest

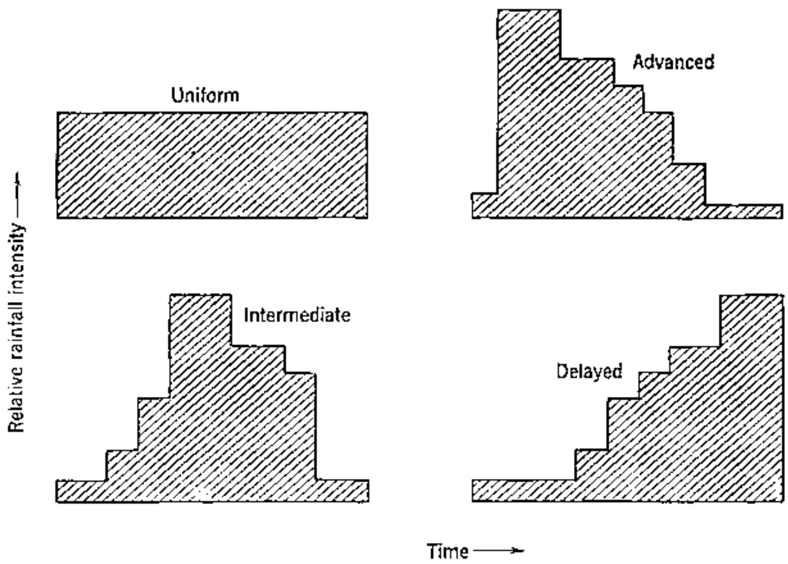


Fig. 2.14. Rainfall intensity patterns. (After Horner and Jens.[9])

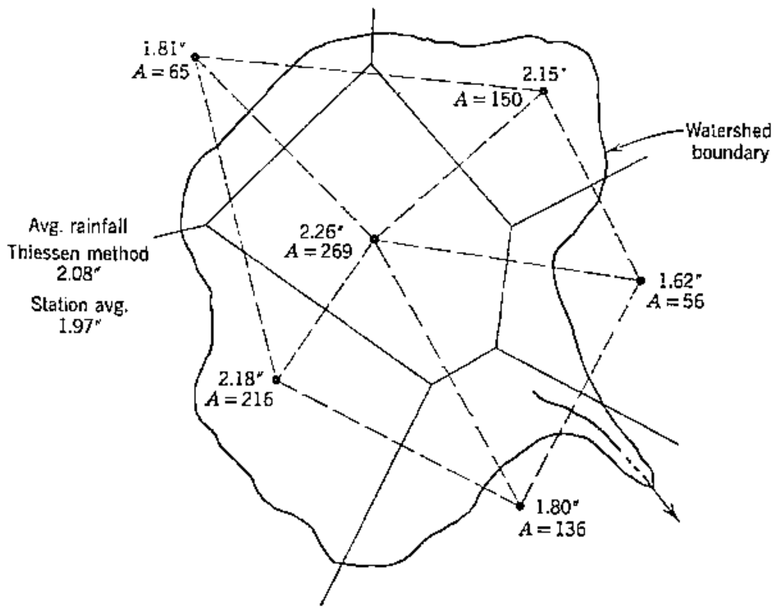


Fig. 2.15. Thiessen network.

method is to take the arithmetic mean of the rainfall in the gages. Since each gage may not represent equal areas, other methods often give greater accuracy.

2.21. Thiessen Method. The use of the Thiessen method[22] is illustrated in Fig. 2.15. The location of the rain gages is plotted on a map of the watershed. Straight lines are then drawn between the rain gages. Perpendicular bisectors are then constructed on these connecting lines in such a way that the bisectors enclose areas referred to as Thiessen polygons. All points within one polygon will be closer to its rain gage than to any of the others. The rain recorded is then considered to represent the precipitation within the appropriate area.

Some difficulty may be encountered in determining which connecting lines to construct in forming the sides of the polygon. Though in general the shorter lines are used, the proper lines can best be determined by a trial-and-error procedure. Since only one set of Thiessen polygons generally needs to be drawn for a given watershed and set of rain gage locations, this procedure does not present a serious limitation. The average precipitation over a watershed can be determined by using the following equation:

$$P = \frac{A_1P_1 + A_2P_2 + \cdots A_nP_n}{A} \tag{2.1}$$

where P represents the average depth of rainfall in a watershed of area A and $P_1, P_2, \ldots P_n$ represent the rainfall depth in the polygon having areas $A_1, A_2, \ldots A_n$ within the watershed.

Example 2.1. A storm on the watershed illustrated in Fig. 2.15 produces rainfall at the various gage locations as indicated. Compare the average precipitation as determined by the average depth and by the Thiessen methods.

Solution. By the average depth method the arithmetic mean is 1.97 inches.

By the Thiessen method, the areas represented by the various rain gages are determined with a planimeter and substituted in equation (2.1):

$$P = \frac{(65)(1.81) + (150)(2.15) + (269)(2.26) + (216)(2.18) + (56)(1.62) + (136)(1.80)}{892}$$

$P = 2.08$ inches

2.22. Isohyetal Method. The isohyetal method consists of plotting the depth of rainfall at the location of the various rain gages and plotting isohyetals (lines of equal rainfall) by the method used in drawing topographic maps. The area between isohyetals may then be planimetered and the average rainfall determined by the above equation.

The choice of the method of analysis will depend partly upon the area of the watershed, the number of rain gages, the distribution of the rain gages, and in some situations, the character of the rainstorm. Depth-area curves, where needed, can be constructed from isohyetal maps.

DISTRIBUTION OF PRECIPITATION IN THE UNITED STATES

2.23. Time Distribution. *Diurnal.* The time of the day in which precipitation may be expected to occur will depend upon the type of precipitation. Frontal storms are not much influenced by diurnal effects. Storms of the convective type, since they are due to surface heating, are much more apt to occur in the afternoon.

Seasonal. That rainfall be distributed throughout the growing season is important. A considerable difference in the seasonal distribution of precipitation throughout the United States is shown in Fig. 2.16. Even in the areas of the west coast where annual precipitation is high, summertime precipitation is generally very low, making irrigation necessary. In the Middle West and South the monthly summertime precipitation is generally somewhat higher than the monthly average, and in the Eastern portion of the United States there is little difference between summer and winter precipitation.

Annual. The annual rainfall over the United States is shown in Fig. 2.17. Annual rainfall amounts vary from less than 5 inches to over 100 inches in some mountainous areas. Annual precipitation is not in itself a good index of the amount of moisture available for plant growth because evaporation, seasonal distribution, and water-holding capacity of the soil vary with geographical locations.

Cycles. That precipitation occurs in cycles has often been suggested. However, as yet there has been no statistical proof that such cycles exist or that there is any relationship between such cycles and other natural phenomena.

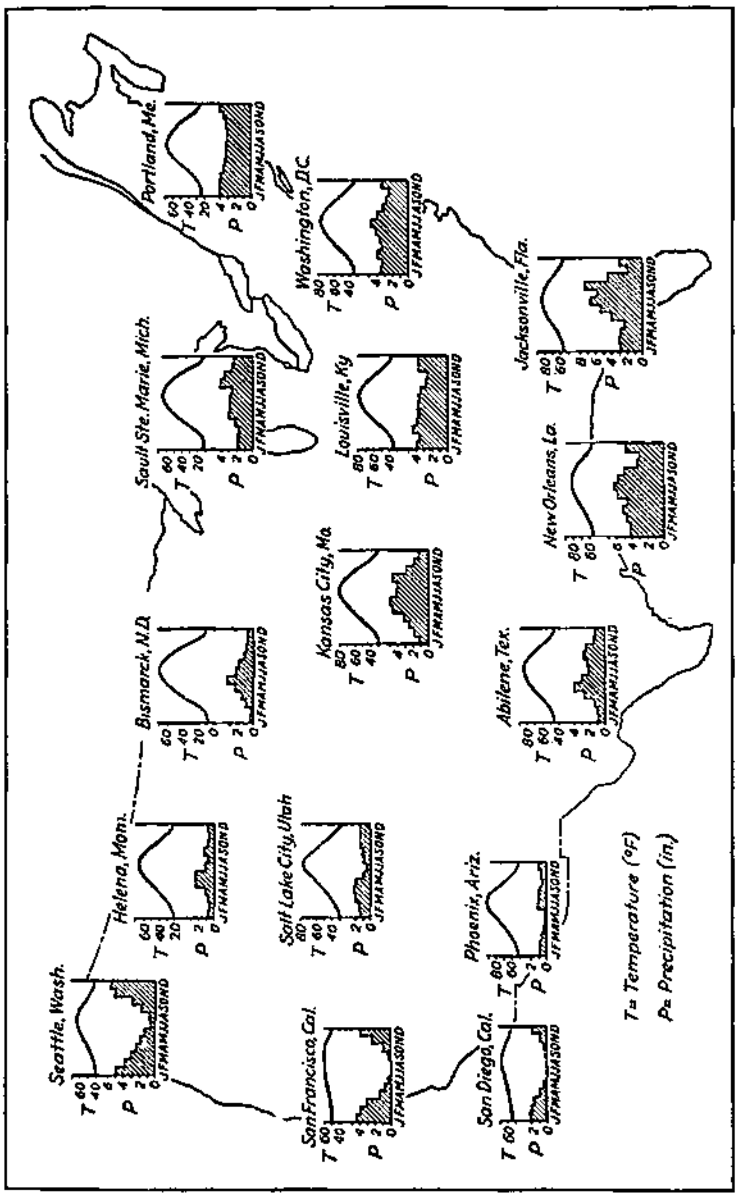


Fig. 2.16. Monthly precipitation and mean temperature for selected locations in the United States. (From Rouse.[19])

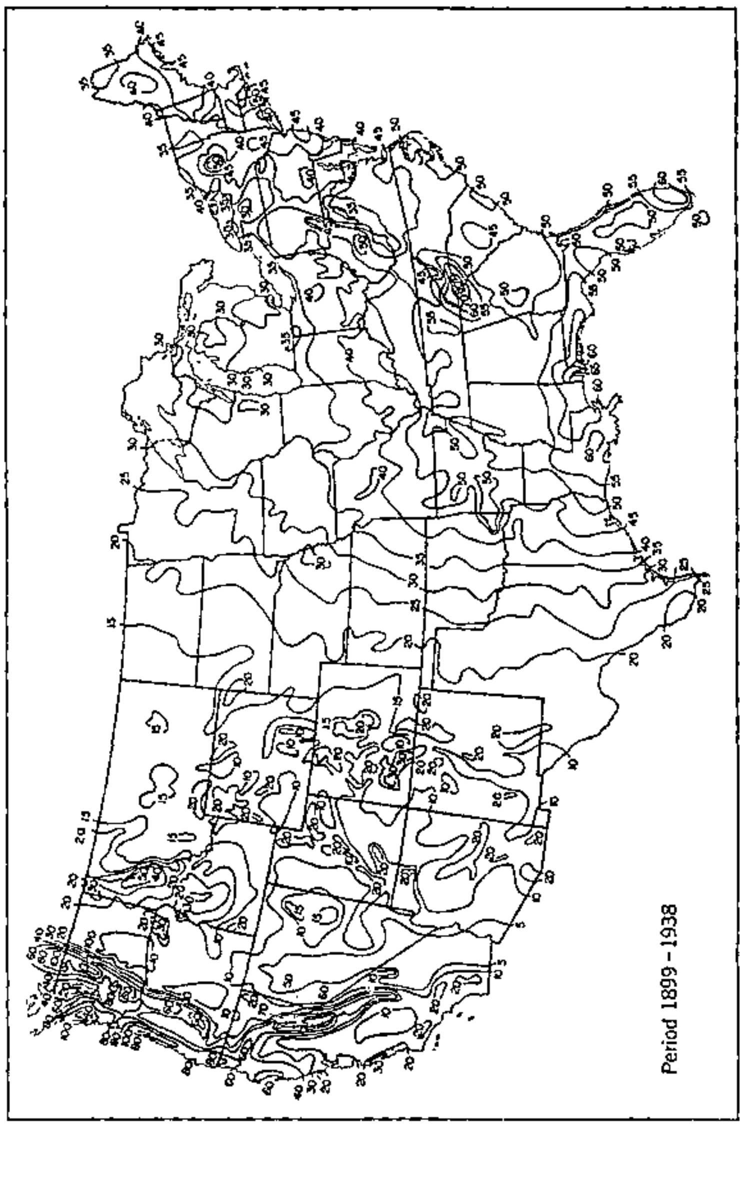


Fig. 2.17. Average annual rainfall in the United States in inches. (From reference 23.)

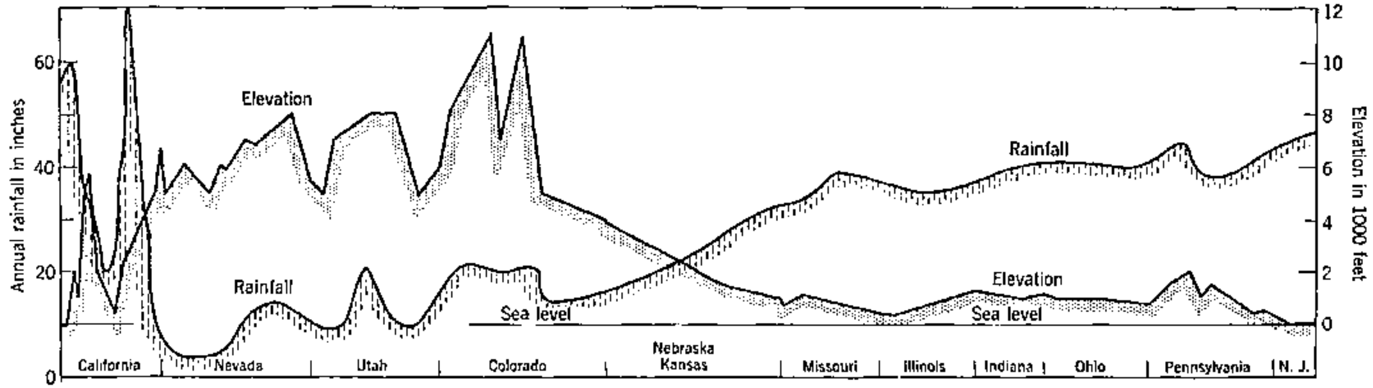


Fig. 2.18. Section of the United States along the 40th parallel, showing elevation and average annual rainfall.

2.24. Geographical Distribution. The geographical distribution of rainfall over the United States is largely determined by the location of large bodies of water, by the movement of the major air masses, and by changes in elevation. Figure 2.18 illustrates the effect of elevation and of moist air-mass movement on annual rainfall. It presents a section of the United States along the fortieth parallel. Moving from west to east, one notes that the highest rainfall occurs as the air is first pushed up by the mountains, with lesser rises as the dryer air is pushed to higher elevations. As the air moves down the mountain slopes, lower annual rainfall is generally observed. The rainfall does not increase until the effects of the maritime tropical air moving up from the Gulf of Mexico become apparent. Then the rainfall gradually increases as the eastern boundary of the United States is approached, with the effect of the Appalachian mountains again apparent.

2.25. Moisture Deficiency. In determining whether irrigation may be desirable, it is often convenient to be able to predict how often droughts may be expected. Though data of this type are not available for the country as a whole, Blumenstock[2] has prepared a series of curves in which the recurrence interval of droughts is plotted against the amounts of precipitation that can be expected. That the seasons of the year in which droughts are more apt to occur vary with different parts of the country can be surmised from Fig. 2.16.

A 10-year study[4] of drought recurrence in 7 states of the Midwest showed that on the average there are 6 dry periods of 1 to 2 weeks' duration and 1 dry period of 2 to 3 weeks' duration during the growing season. Rainless periods of 3 or more weeks occurred less frequently. Other studies in humid regions showed similar results.

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PROBLEMS

2.1. Determine the total rainfall to be expected once in 5, 25, and 100 years for a 60-minute storm at your present location (see Appendix A).

2.2. Determine the maximum rainfall intensity to be expected once in 10 years for storms of durations of 5, 30, 120, and 480 minutes, respectively, at your present location.

2.3. From rainfall data given in Table 2.1 determine the maximum rainfall intensity for a 5-, 30-, and 240-minute period. Determine the recurrence interval for these intensities if the storm occurred at your present location.

2.4. Compute the average rainfall for a given watershed by the Thiessen method from the following data. How do the weighted average and the station average compare?

Rain Gage	*Area in Acres*	*Rainfall in Inches*
A	34.6	2.30
B	11.2	1.60
C	13.2	2.02
D	12.1	1.71

CHAPTER 3

Infiltration, Evaporation, and Transpiration

Three phases of the hydrologic cycle of particular interest to agricultural engineers are infiltration, evaporation, and transpiration. Infiltration is the passage of the water into the soil surface and is distinguished from percolation, which is the movement of water through the soil profile. Evaporation is the process by which moisture is returned to the air by the change of the moisture from a liquid to a gaseous state. Transpiration is the process by which water as water vapor is transferred to the atmosphere by plants. Although there are wide regional variations, about three-fourths of the total precipitation on the land areas of the world returns to the atmosphere through evaporation or transpiration. Most of the balance returns to the ocean as surface or subsurface flow. Evaporation and transpiration are difficult to separate and are often considered together as evapo-transpiration.

Infiltration is of particular interest because, if water is to be conserved in the soil and made available to plants, it must first pass through the soil surface. Also, if a high infiltration rate is maintained, less water passes over the soil surface and erosion is thereby reduced. In this way not only are runoff quantities and peaks reduced (Chapters 4 and 13), but a measure of gully and flood control is provided if infiltration is increased over sufficiently wide areas.

Evaporation, which may occur either from the water surface or from the surface of soil particles, is of interest wherever moisture conservation is a factor. When evaporation is reduced, more moisture remains available for plant growth. Evaporation is also an important factor in determining the requirements for irrigation as well as in predicting the amount of water that may be available from farm ponds.

The importance of transpiration is based largely upon its effect on the moisture requirements of crops. Crops with high transpiration rates require larger amounts of moisture. This in turn affects water requirements as furnished by irrigation or natural rainfall. The removal of excess water by transpiration is an aid to drainage.

INFILTRATION

The movement of water into the soil by infiltration may be limited by any restriction to the flow of water through the soil profile. Although such restriction often occurs at the soil surface, it may occur at some point in the lower ranges of the profile. The most important items influencing this rate of infiltration have to do with the physical characteristics of the soil and the cover on the soil surface, but such other factors as soil moisture, temperature, and rainfall intensity are also involved.

3.1. Soil Factors. Soil functions essentially as a pervious medium which provides a large number of passageways for water to move into the surface. The effectiveness of the soil as an agent for transporting water depends largely upon the size and permanency of these channels. In general the size of the passageways and the infiltration into the soil is dependent upon:[19] (1) the size of the particles that make up the soil, (2) the degree of aggregation between the individual particles, and (3) the arrangement of the particles and aggregates. In general the larger the pore size that can be maintained, the greater is the resulting infiltration rate.

The importance of maintaining permanent channels, particularly at the soil surface, has been shown by a number of investigators. Duley[4] points out that the rapid reduction in the rate of intake of water through the surface is accompanied by the formation of a thin compact layer on the surface (see Fig. 8.5). This layer is a result of severe breakdown of structure due in part to the beating action of the raindrops and in part to an assorting action of the water flowing over the surface, fitting the fine particles around the larger ones to form a relatively impervious seal and to give the surface of the soil a slick appearance.

This surface-sealing effect can largely be eliminated when the soil surface is protected by mulch or by some other permeable mechanical protection. The effectiveness of such protection is illustrated in Fig. 3.1, which first shows the constant infiltration rate of soil covered by straw. After 40 minutes of infiltration at a constant rate, the straw was removed and the infiltration rate dropped to about one-sixth of its original value. The

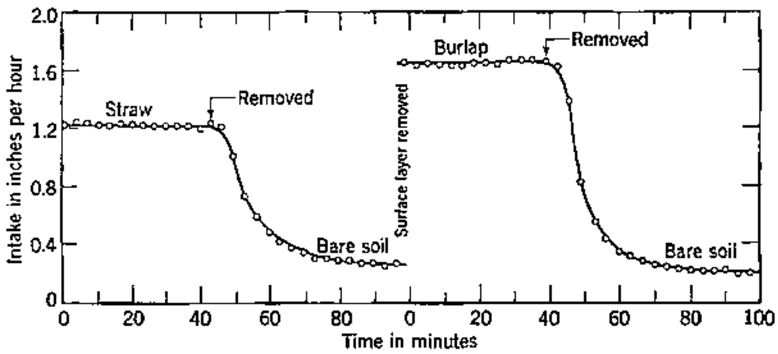


Fig. 3.1. Effect of protective cover on infiltration. (Redrawn from Duley.[4])

straw had protected against the formation of the impervious surface layer, and when the straw was removed the impermeable layer developed quickly through the beating action of the raindrops. By removing the puddled surface layer of soil and protecting the newly exposed soil surface with a layer of burlap, the infiltration rate increased to a new high value. When after 40 minutes the burlap was again removed, the soil surface puddled, and the infiltration rate fell to a new low.

3.2. Vegetation. Surface sealing can be greatly reduced by vegetation.[5,7,25] In general vegetative cover and surface condition have more influence on infiltration rates than do the soil type and texture.[7] The protective cover may be grasses or other close-growing vegetation as well as mulches. It has been shown that, when infiltration rates are determined for soil protected by vegetation and the vegetation is removed, surface sealing occurs and infiltration drops much as illustrated in Fig. 3.1. Figure 3.2 gives a number of infiltration rates for unprotected soil and for several surface cover conditions as determined for three South Carolina soils.

3.3. Other Factors. Other factors affecting infiltration include antecedent soil moisture, soil slope, water temperature, and the factor of the soil being frozen. The effect of slope on rate of infiltration has generally been shown to be small, and to be more important on slopes less than 2 per cent than on steeper gradients.[7] Some investigators feel that the effect of slopes steeper than 2 per cent on infiltration is not significant.

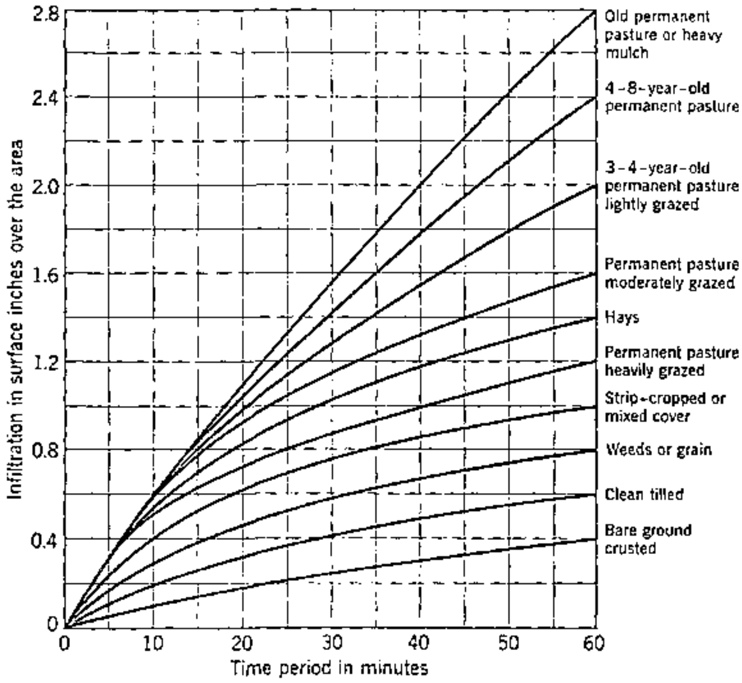


Fig. 3.2. Typical mass infiltration curves. (Redrawn from Holtan and Kirkpatrick.[14])

Soil moisture generally reduces or limits the infiltration rate. The reduction is due in a large part to the fact that moisture causes some of the colloids in the soil to swell, and thereby to reduce both the pore space and the rate of water movement. Consequently, in making infiltration runs with cylinders and infiltrometers, it is customary to make both a dry run and a wet run, often 24 hours later. Design is usually based on the minimum values obtained.

The effect of water temperature on infiltration is not significant,[6] perhaps because of the probabilities that the soil changes the temperature of the entering water and that the size of the pore spaces may change with temperature changes. Although freezing of the soil surface greatly reduces its infiltration rate, freezing does not necessarily render the soil surface impervious.[1]

3.4. Soil Additives. The physical characteristics of the soil, including the infiltration capacity, can be changed by adding chemical materials to it. In general these additives are of one of two types. The first type consists of materials that add to the permanency of the soil aggregate formations, and thereby generally improve the soil structure.[12,13] This improved structure causes considerable increases in both the infiltration and percolation rates. The second type of additive is essentially a wetting agent which does not change the soil but instead changes the angle of contact of the soil water with the soil surface and thereby the rate at which water can move through the soil.[9] It therefore affects water movement at depths greater than the zone of application. In general it may be necessary to reapply these wetting agents periodically, as they leach out with continued water application.

3.5. Methods of Measuring Infiltration. Infiltration measurements may be made either by observing runoff from real or simulated rainfall or by observing the rate of fall of ponded water. The most suitable method of measuring infiltration will depend upon the intended application of data as well as the equipment and other resources available.

Infiltrometers using simulated rainfall cover areas varying from less than 2 to nearly 500 square feet. One of the more common types[27] consists of a rectangular metal barrier surrounding the soil area to be studied. Water is applied through a set of sprinklers expressly designed to simulate actual rainfall conditions. A protecting tent may be used to minimize the effect of wind. The rate of water application is determined by collecting all the water falling on the area in a given time. Water is then applied directly to the soil at the same rate, and runoff is measured at convenient time intervals. The runoff rate and the water application rate may then be plotted against time as in Fig. 3.3. The difference in the ordinates is then plotted as the infiltration rate. Infiltration is also sometimes estimated by determining the rate of water application at a point in a sprinkler irrigation pattern where there is slight ponding but no runoff.

The effective infiltration rate of an entire watershed may be estimated by an analysis of rainfall and runoff records.[16,24] The procedure is essentially that of subtracting runoff rates from rainfall rates with appropriate correction for surface and channel detention (Chapter 4). If the infiltration is to be representative of any soil, crop, or topographic condition, the entire watershed must be uniform in these respects.

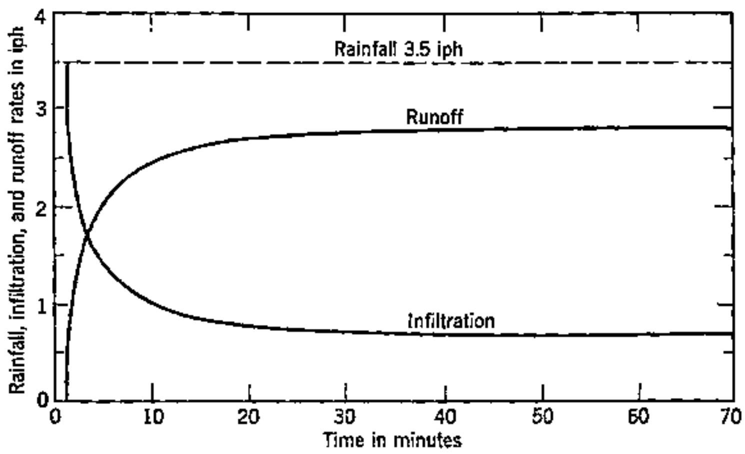


Fig. 3.3. Typical infiltration curve developed from infiltrometer data.

Infiltration measurements with ponded water may be made by inserting cylinders into the soil, applying water, and measuring the rate of fall of the water level.[8,10,19] To provide the buffer area around the space where the measurement is being made and to insure vertical movement, concentric cylinders with the same water level maintained in both the inside and the outside cylinder are often used. The measurements are taken only on the inside cylinder. Ponded water infiltration determinations are also made by measurements of the rate of fall in ponds formed by ridges as in check irrigation.

Infiltrometers utilizing simulated rainfall have been found[21] to give lower values of infiltration more nearly representative of cover conditions and of natural rainfall on a watershed. On the other hand, ponded water infiltration measurements gave values more nearly representative of soil profile characteristics.

3.6. Expressing Infiltration Data. Infiltration data are commonly expressed graphically with inches per hour as the ordinate and time as the abscissa. Figure 3.3 presents a typical infiltration curve as determined by the infiltrometer method. Here, as usual, the potential infiltration capacity at first exceeds the rate of water application. However, as the soil pores fill with water, and as surface sealing takes place, the rate of water intake gradually drops. It then normally approaches a constant value which may be taken as the infiltration rate of the soil.

EVAPORATION

Evaporation is the transfer of liquid water into the atmosphere. The water molecules, both in the air and in the water, are in rapid motion. Evaporation occurs when a larger number of the moving molecules break through the water surface and escape into the air as vapor than the number that break through the water surface from the air and become entrapped in the liquid. The factors affecting the rate of evaporation are the nature of the evaporating surface and the vapor pressure differences as affected by temperature, wind, atmospheric pressure, and the quality of water.

That the rate of evaporation increases with the rise in temperature of the water surface is to be expected as vapor pressure increases with increases in temperature. The effect of air temperatures on evaporation is not clearly established, but in general there is a decrease in evaporation at the lower temperatures found at high latitudes.[20] It has been shown that mean monthly air temperatures do not alone provide a satisfactory means of predicting mean monthly evaporation.[19]

Wind increases the rate of evaporation, particularly as it disperses the moist layer found directly over the evaporating water surface under stagnant conditions. Because of this mixing, characteristics of the atmosphere above the surface are of interest. As might be expected, from the decreased concentration of water molecules, evaporation increases with decreased barometric pressure. Likewise, if other conditions are unchanged, there is greater evaporation at higher elevations. Also the rate of evaporation has been found to decrease with increases in the salt content of the water.

3.7. Methods of Measuring and Predicting Evaporation. Evaporation can be predicted by formulas based either on atmospheric conditions or on the transformation of energy. It can be measured by evaporating pans, by measuring the evaporation from larger water areas, or by atometers.

Most evaporation formulas are based upon Dalton's law:

$$E = C(e_s - e_d) \tag{3.1}$$

where e_s is the saturated vapor pressure at the temperature of the water surface, e_d is the saturated vapor pressure of the air at its dew point, E is the rate of evaporation, and C is a constant based on the other variables affecting the rate of evaporation. Meyer[20] and Rohwer[22] have essentially proposed means of evaluating C in Dalton's basic equation. For instance, Rohwer's empirical formula for evaporation is obtained when the constant, C, in Dalton's equation is equal to $(0.44 + 0.118W)$. When W is the surface wind velocity in miles per hour and when the vapor pressures are measured in inches of mercury, the evaporation E is obtained in inches per 24 hours. A correction factor for changes in altitude may be applied.[22]

Since heat is required to vaporize or evaporate water, it has been proposed that evaporation should be predicted by formulas based on energy transformation. Such formulas have been suggested[19] but have severe limitations because equipment required to make the necessary measurements is not generally available.

Evaporation measurements from free water surfaces are commonly made using evaporation tanks or pans. The Class A pan, accepted as standard by the U. S. Weather Bureau, is 4 feet in diameter, 10 inches deep, and requires a water depth of between 7 and 8 inches. The pan is supported about 6 inches above the ground, so that the air may circulate under it, and the materials and color of the pan are specified. This pan is the most widely used in the United States. Descriptions of other styles of pans and correction coefficients for converting evaporation data from a pan of one type to that of another are available and are given elsewhere.[19] These small pans have higher rates of evaporation than do larger free water surfaces, a factor of about 0.7 being recommended[19] in converting the observed evaporation rates to those of large surface areas.

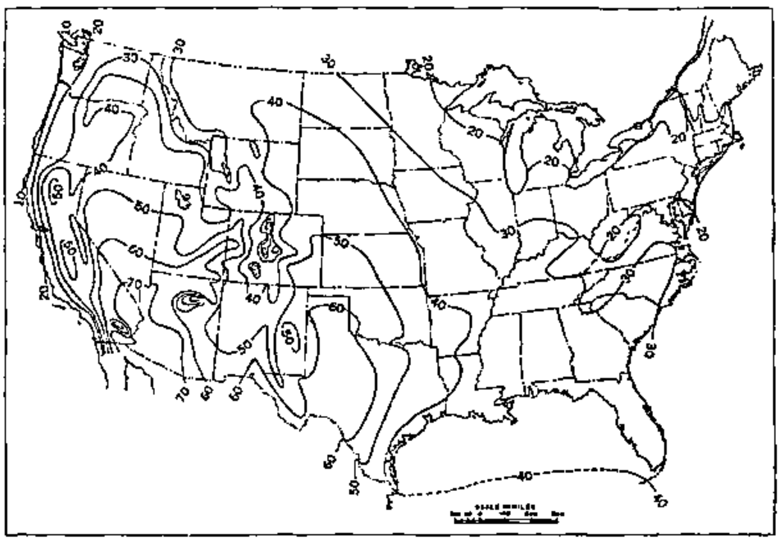
3.8. Evaporation from Land Surfaces. Because of differences in soil te[illegible] l moisture movement, it is difficult to generalize on the amounts of evaporation from soil surfaces. For saturated soils, the evaporation may be expected to be essentially the same as from open free water surfaces. As the water table drops, however, the evaporation rate will decrease greatly. Evaporation from soil surface is generally unimportant at moisture levels below field capacity, as soil moisture movement is very slow when the soil is relatively dry (Chapter 5). Mulches are effective[23] for a few days after a rain. The mulch restricts air movement and maintains a high air vapor pressure, which in turn reduces evaporation. Also freezing of a bare soil surface causes the surface to become wet, and greatly increases the evaporation rate.[2]

3.9. Geographical Distribution. The geographical distribution of annual and seasonal evaporation has been determined by two methods. The first[20] is based upon Meyer's evaporation formula with evaporation for different locations in the United States calculated from Weather Bureau wind, temperature, and moisture data. The second[15] is based upon the evaporation actually observed in Class A evaporation pans and in other evaporation pans with the values corrected to Class A pans by accepted coefficients. The calculated evaporation rates are those considered to be directly applicable to ponds and lakes; those from the evaporation pan can be used for large free water surfaces only with the appropriate area correction factor. Figure 3.4 presents the mean summer (April to October) evaporation as calculated by Meyer's formula.

TRANSPIRATION

The amount of water that passes through plants by the transpiration process is often a substantial portion of the total moisture available during the growing season. It can vary from practically nothing to as much as 25 inches on a given land area, depending largely upon the moisture available, the kind and density of plant growth, the amount of sunshine, and the soil fertility and structure. Less than 1 per cent is actually retained by the growing organisms.

3.10. Transpiration Ratio. The effectiveness of the plant's use of water in producing dry matter is often given in terms of its transpiration ratio. This is the ratio of the weight of water



NOTE

Evaporation from large deep lakes and reservoirs, particularly in arid regions, will be substantially less in spring and summer, greater in fall and winter, and less for the year than the values here shown.

Evaporation from the surfaces of soil and vegetation immediately after rains or irrigation will begin at greater rates and diminish rapidly with the supply of available moisture.

Great local differences in topography and climate in mountainous regions cause large local differences in evaporation not adequately shown here, particularly in the western states.

Fig. 3.4. Mean summer evaporation from shallow lakes and reservoirs from April to October in inches. (Redrawn from Meyer.[20])

transpired to the weight of dry matter in the plant. It, therefore, varies with the same factors as does transpiration. Approximate transpiration ratios for several common plants[11] are: 250 for sorghum, 350 for corn, 450 for red clover, 500 for wheat, 640 for potatoes, and 900 for alfalfa.

3.11. Evapo-Transpiration. For convenience evaporation and transpiration are combined into evapo-transpiration, often referred to as consumptive use. Various methods[18] for determining evapo-transpiration include (1) tank and lysimeter experiments; (2) field experimental plots where the quantity of water applied is kept small to avoid deep percolation losses. and surface runoff is measured; (3) soil moisture studies, a large number of moisture samples being taken at various depths in the root zone; (4) analysis of climatological data; (5) integration methods where the water used by plants and evaporation from the water and soil surfaces are combined for the entire area involved; and (6) inflow-outflow method for large areas where yearly inflow into the area, annual precipitation, yearly outflow from the area, and the change in ground water level are evaluated.

Blaney and Criddle[3] have developed a method that is receiving considerable use for determining evapo-transpiration from climatological and irrigation data. The procedure is to correlate existing evapo-transpiration data for different crops with the monthly temperature, per cent of daytime hours, and length of growing season. The correlation coefficients are then applied to determine the evapo-transpiration for other areas where only climatological data are available.

The monthly evapo-transpiration can be computed by the empirical formula:

$$u = \frac{ktp}{100} = kf \tag{3.2}$$

where u = monthly evapo-transpiration in inches.
k = monthly evapo-transpiration (consumptive use) coefficient (determined for each crop from experimental data).
t = mean monthly temperature in degrees Fahrenheit.
p = monthly per cent of daytime hours of year.
$f = \frac{(tp)}{100}$ = monthly evapo-transpiration (consumptive use) factor.

For the entire growing season the following equation is more convenient:[13]

$$U = KF = \sum kf \tag{3.3}$$

where U and K correspond to u and k in equation 3.2 and F = sum of the monthly evapo-transpiration (consumptive use) factors f for the period. Mean monthly temperatures and per cent of daytime hours for each month can be determined from Weather Bureau records or from other data for the locality.

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CHAPTER 4

Runoff

The agricultural engineer in soil and water conservation is called upon continually to design structures and channels that will handle natural flows of water. These flows are usually runoff from rainfall or melting snow. The runoff constitutes the hydraulic "load" which the structure or channel must withstand.

4.1. Definition. Runoff is that portion of the precipitation that makes its way toward stream channels, lakes, or oceans as surface or subsurface flow. When the term "runoff" is used alone, surface runoff usually is implied. The engineer designing channels and structures to handle natural surface flows is concerned with peak rates of runoff, with runoff volumes, and with temporal distribution of runoff rates and volumes.

4.2. The Runoff Process. Before runoff can occur the precipitation must satisfy the demands of evaporation, interception, infiltration, surface storage, surface detention, and channel detention.

Interception may be so great as to prevent a light rain from wetting the soil. Interception by dense covers of forest or shrubs commonly amounts to 25 per cent of the annual precipitation.[10] A good stand of mature corn will have a net interception storage capacity of 0.02 inch.[17] Trees, such as willows, may intercept nearly 0.5 inch from a long, gentle storm.[10] Interception also has a detention storage effect, delaying the progress of precipitation that reaches the ground only after running down the plant or dropping from the leaves.

Runoff will occur only when the rate of precipitation exceeds the rate at which water may infiltrate into the soil (see Chapter 3). After the infiltration rate is satisfied water begins to fill the depressions, small and large, on the soil surface. As the depressions are filled overland flow begins. The depth of water builds up on the surface until the head is sufficient to result in runoff in equilibrium with the rate of precipitation less infiltration and interception. The volume of water involved in the head build-up is in surface detention. As the flow moves into defined channels there is a similar build-up of head with a volume of water in channel detention. The volume of water in surface and channel detention is returned to runoff as the runoff rate subsides. The water in surface storage eventually goes into infiltration or is evaporated.

FACTORS AFFECTING RUNOFF

The factors affecting runoff may be divided into those factors associated with the precipitation and those factors associated with the watershed.

4.3. Rainfall. Rainfall duration, intensity, and areal distribution (see Chapter 2) influence the rate and volume of runoff. Total runoff for a storm is clearly related to the duration for a given intensity. It has been noted (Chapter 3) that infiltration capacity may decrease with time in the initial stages of a storm. Thus a storm of short duration may produce no runoff, whereas a storm of the same intensity but of long duration will result in runoff.

Rainfall intensity influences both the rate and the volume of runoff. An intense storm exceeds the infiltration capacity by a greater margin than does a gentle rain; thus the total volume of runoff is greater for the intense storm even though total precipitation for the two rains is the same. The intense storm actually may decrease the infiltration rate because of its destructive action on the structure of the soil surface.

Figure 4.1 shows relationships of runoff to rainfall intensity and to total rainfall per storm. The data summarize 8 years of record on bare plots at Statesville, N. Carolina. In this study it was found that 28.7 per cent of the total precipitation occurred in storms of 0 to 1 inch, whereas only 22.5 per cent of the total runoff resulted from such storms. At the other extreme, 9.9 per cent of the precipitation occurred in storms of over 3 inches, but 12.8 per cent of the runoff resulted from such storms. Also, 23.9 per cent of the rain fell at intensities greater than 3 inches per hour and resulted in 36.9 per cent of the runoff. Only 26.4 per cent of the runoff resulted from the 43.7 per cent of the precipitation which fell at intensities less than 1.5 inches per hour.

Rate and volume of runoff from a given watershed are influenced by the distribution of rainfall and of rainfall intensity over the watershed. Generally the maximum rate and volume of runoff occurs when the entire watershed contributes. However, an intense storm on one portion of the watershed may result in greater runoff than a moderate storm over the entire watershed.

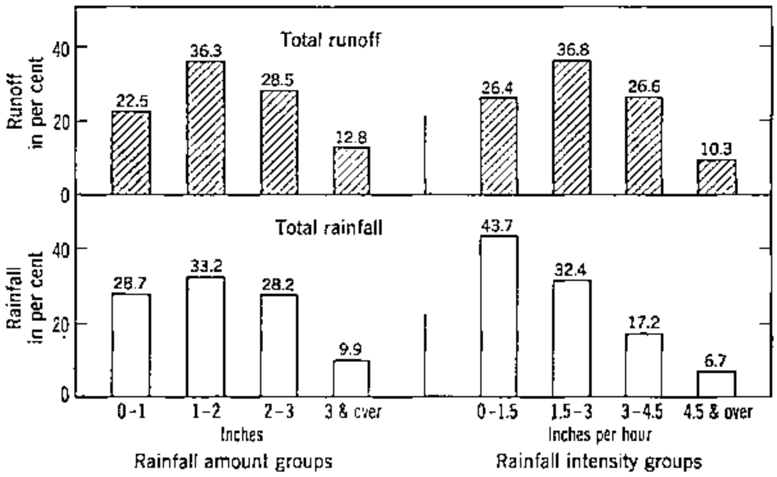


Fig. 4.1. Total storm runoff related to rainfall amounts and intensities. (Redrawn from Copley and others.[2])

4.4. Watershed. Watershed factors affecting runoff are size, shape, orientation, topography, geology, and surface culture. Both runoff volumes and rates increase as watershed size increases. However, both rate and volume per unit of watershed area decrease as the runoff area increases. Watershed size may determine the season at which high runoff may be expected to occur. Harrold[5] has observed that on watersheds in the Ohio River basin 99 per cent of the floods from drainage areas of 1 square mile occur in May through September; 95 per cent of the floods on drainage areas of 100,000 square miles occur in October through April. Some type of seasonal runoff relationships may be found to exist in other geographic regions.

Long, narrow watersheds are likely to have lower runoff rates than more compact watersheds of the same size. The runoff from the former does not concentrate as quickly as it does from the compact areas, and long watersheds are less likely to be covered uniformly by intense storms. When the long axis of a watershed is parallel to the storm path, storms moving upstream cause a lower peak runoff rate than storms moving downstream. In the former case runoff from the lower end of the watershed is diminished before the peak contribution from the headwaters arrives at the outlet. However, a storm moving downstream causes a high runoff from the lower portions coincident with high runoff arriving from the headwaters.

Topographic features, such as slope of upland areas, the degree of development and gradients of channels, and the extent and number of depressed areas, affect rates and volumes of runoff. Watersheds having extensive flat areas or depressed areas without surface outlets have lower runoff than areas with steep, well-defined drainage patterns. The geologic or soil materials determine to a large degree the infiltration rate and capacity, and thus have their effect upon runoff. Vegetation and the practices incident to agriculture and forestry also influence infiltration (see Chapter 3). Vegetation retards overland flow and increases surface detention to reduce peak runoff rates. Works of man, such as dams, levees, bridges, and culverts, all influence runoff rates.

METHODS OF PREDICTING RUNOFF

The engineer concerned with the design of hydrologic structures must obtain quantitative estimates of runoff rates, volumes, and temporal distribution. From the above discussion of the runoff process it is seen that accurate prediction is a difficult job. Methods of runoff estimation necessarily neglect some factors and make simplifying assumptions regarding the influence of others. Methods presented here are commonly used in problems of soil and water conservation.

ESTIMATION OF DESIGN RUNOFF RATES

The capacity to be provided in a structure that must carry runoff may be termed the design runoff rate. Structures and channels are planned to carry runoff which occurs with a specified recurrence interval. Vegetated controls and temporary structures are usually designed for a runoff that may be expected to occur once in 10 years; expensive, permanent structures will be designed for runoffs expected only once in 50 or 100 years. Selection of the design recurrence interval depends upon the economic balance between the cost of periodic repair or replacement of the facility and the cost of providing additional capacity to reduce the frequency of repair or replacement. In some instances the possibility of downstream damage potentially resulting from failure of the structure may dictate the choice of the design recurrence interval.

4.5. Rational Method. The rational method of predicting a design peak runoff rate is expressed by the equation

$$Q = CiA \tag{4.1}$$

where Q is the design peak runoff rate in cubic feet per second, C is the runoff coefficient, i is the rainfall intensity in inches per hour for the design recurrence interval and for a duration equal to the "time of concentration" of the watershed, and A is the watershed area in acres. The time of concentration of a watershed is the time required for water to flow from the most remote (in time of flow) point of the area to the outlet. It is assumed that, when the duration of a storm equals the time of concentration, all parts of the watershed are contributing simultaneously to the discharge at the outlet. Appendix B contains a graph for estimating the time of concentration. The runoff coefficient C is defined as the ratio of the peak runoff rate to the rainfall intensity and is dimensionless. Appendix B gives a table (Table B.2) for use in estimating the value of C.

Equation 4.1 may not appear to be dimensionally correct. Although i is specified in inches per hour, 1 iph is 1.008 cfs per acre, and in using the equation the two are taken to be numerically equal.

The rational method is recognized to have a number of weaknesses in the light of modern knowledge of runoff mechanics. It is a great oversimplification of a complicated process. However, the method is considered sufficiently accurate for runoff estimation in the design of relatively inexpensive structures where the consequences of failure are limited. Application of the rational method as presented here is normally limited to watersheds of less than 5 square miles (3200 acres).

The rational method is developed from the assumptions that: (1) rainfall occurs at uniform intensity for a duration at least equal to the time of concentration of the watershed, and (2) rainfall occurs at a uniform intensity over the entire area of the watershed. If these assumptions were fulfilled, the rainfall and runoff for the watershed would be presented graphically by Fig.

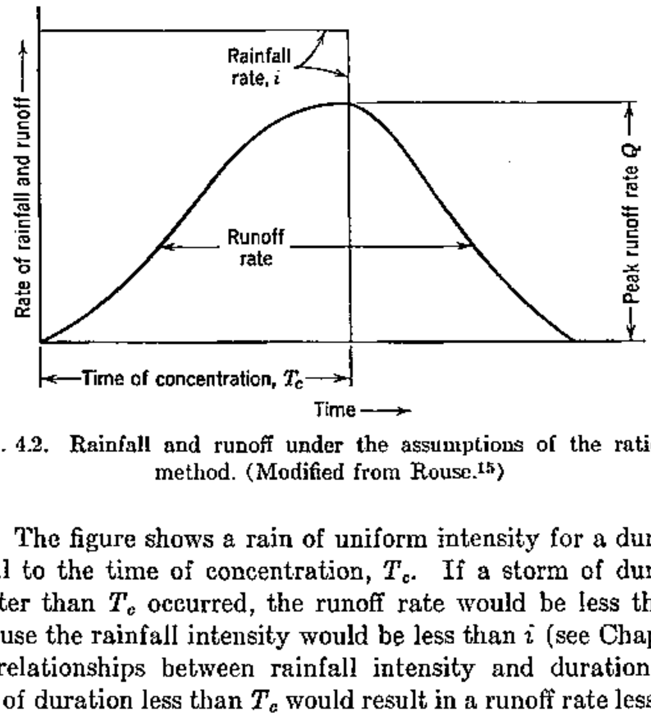


Fig. 4.2. Rainfall and runoff under the assumptions of the rational method. (Modified from Rouse.[15])

4.2. The figure shows a rain of uniform intensity for a duration equal to the time of concentration, T_c. If a storm of duration greater than T_c occurred, the runoff rate would be less than Q because the rainfall intensity would be less than i (see Chapter 2 for relationships between rainfall intensity and duration). A rain of duration less than T_c would result in a runoff rate less than Q because the entire watershed would not contribute simultaneously to the discharge at the outlet.

The rational method is illustrated by the following problem.

Example 4.1. Determine the design peak runoff rate for a 50-year recurrence interval from an area containing 60 acres of flat, cultivated clay loam and 40 acres of rolling (5–10 per cent) sandy loam woodland. The maximum length of flow is 1700 feet and the fall along this path is 6.8 feet. The watershed is located in southern Illinois.

Solution. Enter Fig. B.1 with $K = \sqrt{L^3/H} = \sqrt{1700^3/6.8} = 26{,}900$, and find T_c as 20 minutes. From Appendix A.1 the 1-hour rainfall for a 50-year recurrence interval in southern Illinois is 3.00 inches. Using Fig. A.2 convert the 1-hour intensity to a 20-minute intensity of 5.4 iph. From Appendix B, determine a weighted value of C.

60 acres of flat, cultivated clay loam $C = 0.50$
40 acres of rolling sandy loam woodland $C = 0.25$
Average $C = (60/100)\,0.5 + (40/100)\,0.25 = 0.40$
$Q = CiA = 0.40(5.4)100 = 216$ cfs

4.6. Cook's Method. A different approach[4] to estimation of runoff from small agricultural areas was developed by Cook. By this method the runoff characteristics of a watershed are examined under the four categories of relief, soil infiltration, vegetal cover, and surface storage. Through observation of peak floods from agricultural areas, other investigators[18] have assigned numerical values to the various conditions of relief, infiltration, vegetal cover, and surface storage which may be present. Description of these conditions and their numerical values are presented in Appendix B.

The sum, $\sum W$, of the numerical values assigned to the watershed characteristics is obtained. Runoff curves presented in Appendix Fig. B.2 are then entered with the drainage area and the $\sum W$, and a value of peak runoff for a 10-year recurrence interval is obtained. This peak runoff value is modified for recurrence interval and geographic rainfall characteristics by the formula

$$Q = PRF \tag{4.2}$$

where Q is the peak runoff for a specified geographic location and recurrence interval, P is the peak runoff from Fig. B.1, R is the geographic rainfall factor from Fig. B.2, and F is the recurrence interval factor from Fig. B.1.

Example 4.2. Estimate the peak runoff for a 50-year recurrence interval from the watershed of Example 4.1.

Solution. Determine the $\sum W$ to be:

Relief	14
Soil infiltration	12
Vegetal cover	11
Surface storage	15
$\sum W =$	52

Entering Fig. B.1 with a drainage area of 100 acres and a $\sum W$ of 52, $P = 155$ cfs. From Fig. B.2, $R = 0.95$,

$$Q = (155)(0.95)(1.4) = 206 \text{ cfs}$$

4.7. Other Methods. Many other methods have been proposed for estimating flood runoff.[3,8,10,14] One method, flood frequency analysis, depends upon the existence of a number of years of record from the basin under study. These records then constitute a statistical array which defines the probable frequency of recurrence of floods of given magnitudes. Extrapolation of the frequency curves enables the hydrologist to predict flood peaks for a range of recurrence intervals.[3,10] The depth-discharge method[7] evaluates infiltration and detention storage rates and deducts them from the design rainfall to estimate the design runoff.

A number of empirical formulas have been developed to describe the magnitude of extreme floods. These formulas take the form: $Q = KA^x$, where Q is the magnitude of the peak runoff, K is a coefficient dependent upon various characteristics of the watershed, and A is the watershed area; x is determined from field observations.

ESTIMATION OF RUNOFF VOLUME

It is often desirable to predict the total volume of runoff which may come from a watershed during a design peak flood. This estimation requires a knowledge of the rainfall and the disposition of the rainfall for the design storm.

4.8. Mass Rainfall-Infiltration Method. The simplest method of estimating total runoff is to plot mass rainfall and mass infiltration curves as shown in Fig. 4.3. Mass rainfall data can be obtained from Appendix A. The infiltration rate is assumed to be constant and at the lowest value that might result from high antecedent moisture conditions. The maximum runoff volume is then represented by the largest value of rainfall minus infiltration. This occurs where dI/dT equals the infiltration rate. The total volume of runoff is obtained from the equation

$$V = A(I - F)_{maximum} \tag{4.3}$$

where V is the total flood volume, A is the watershed area, I is mass rainfall, and F is mass infiltration.

Example 4.3. Determine the estimated maximum volume of runoff for a 50-year recurrence interval which may be expected from the watershed of Example 4.1.

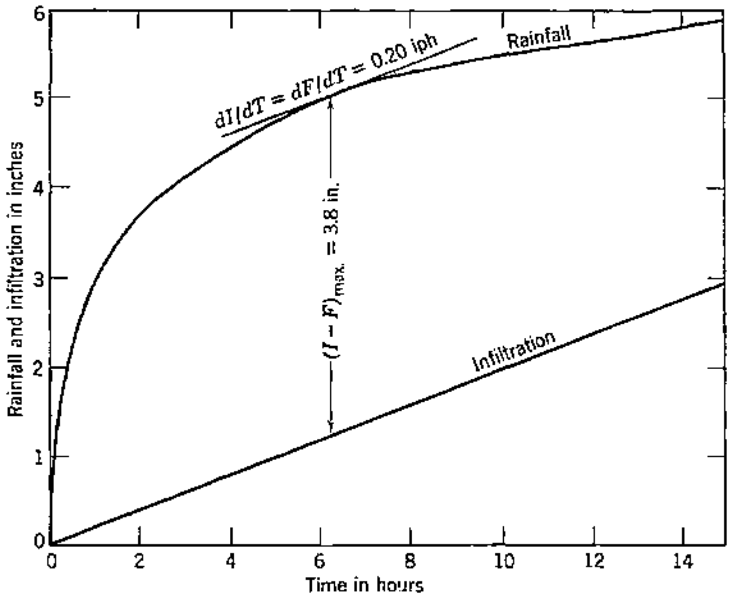


Fig. 4.3. Mass rainfall and mass infiltration curves used in estimating flood volume.

Solution. From Appendix A plot the mass rainfall curve for a 50-year recurrence interval in southern Illinois. The infiltration rate is estimated as 0.20 iph. These data are plotted in Fig. 4.3. The maximum $(I - F)$ occurs at 6.2 hours, and the total runoff is 3.8 inches.

$$V = \frac{(100)(3.8)}{12} = 31.6 \text{ acre-ft}$$

DEVELOPMENT OF RUNOFF HYDROGRAPHS

A hydrograph is a graphical or tabular representation of run-off rate against time. Figure 4.4 gives the hydrograph of the discharge from a 309-acre agricultural watershed. This discharge resulted from a storm of 1.71 inches in 30 minutes.

4.9. Basic Hydrograph. Records of streamflow from which hydrographs may be developed[16] are generally not available for watersheds of interest to agricultural engineers. The basic hydrograph method is used in the development of design runoff hydrographs for such watersheds. The basic hydrograph given in Fig. 4.5 has a form typical of the runoff hydrographs from most watersheds. The basic hydrograph is plotted over 100 arbitrary units of flow and 100 arbitrary units of time.

To develop the design hydrograph it is necessary to estimate the peak flow and runoff volume for the design recurrence interval. With the peak flow and runoff volume estimated, the design runoff hydrograph can be determined from the basic

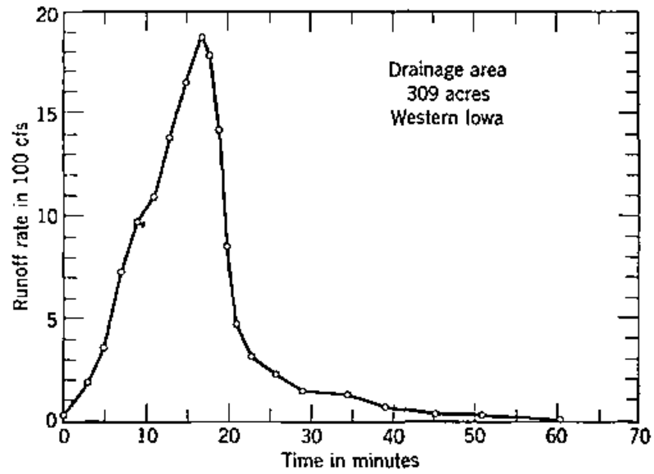


Fig. 4.4. Runoff hydrograph for the upper Theobold Watershed, Woodbury County, Iowa, June 15, 1950.

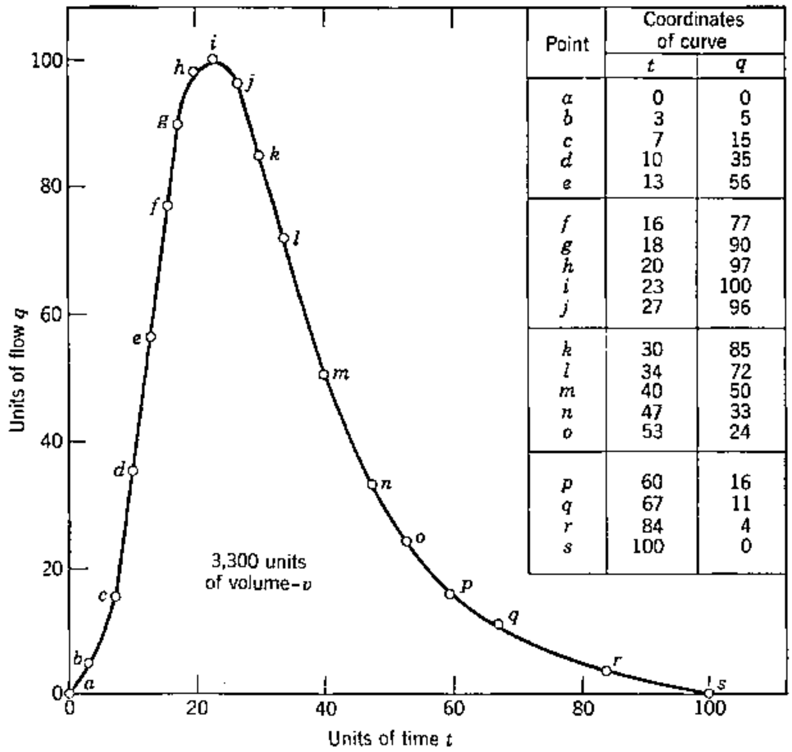
hydrograph by use of conversion factors u, w, and k. The factor u is the ratio of the total runoff volume to the area under the basic hydrograph. The area under the basic hydrograph is 3300 square units. Thus, each square unit under the basic hydrograph has a value of

$$u = V/3300 \tag{4.4}$$

for the design storm having a total runoff volume V. The factor w is the ratio of peak runoff for the design storm to the peak flow of 100 on the basic hydrograph. Each unit of flow on the basic hydrograph has a value of

$$w = \frac{Q}{100} \tag{4.5}$$

in the hydrograph of the design storm. The factor k is the value that each unit of time on the basic hydrograph represents in the design hydrograph. On the design hydrograph $\frac{1}{100}$ of the peak flow times $\frac{1}{100}$ of the duration of runoff must equal $\frac{1}{3300}$ of the flood volume just as it does on the basic hydrograph. Since w is equal to $\frac{1}{100}$ of the design peak flow, k must be equal



Point	Coordinates of curve t	Coordinates of curve q
a	0	0
b	3	5
c	7	15
d	10	35
e	13	56
f	16	77
g	18	90
h	20	97
i	23	100
j	27	96
k	30	85
l	34	72
m	40	50
n	47	33
o	53	24
p	60	16
q	67	11
r	84	4
s	100	0

Fig. 4.5. Basic flood hydrograph. (Modified from references 1 and 18.)

to $\frac{1}{100}$ of the design duration, and u is $\frac{1}{3300}$ of the design flood volume

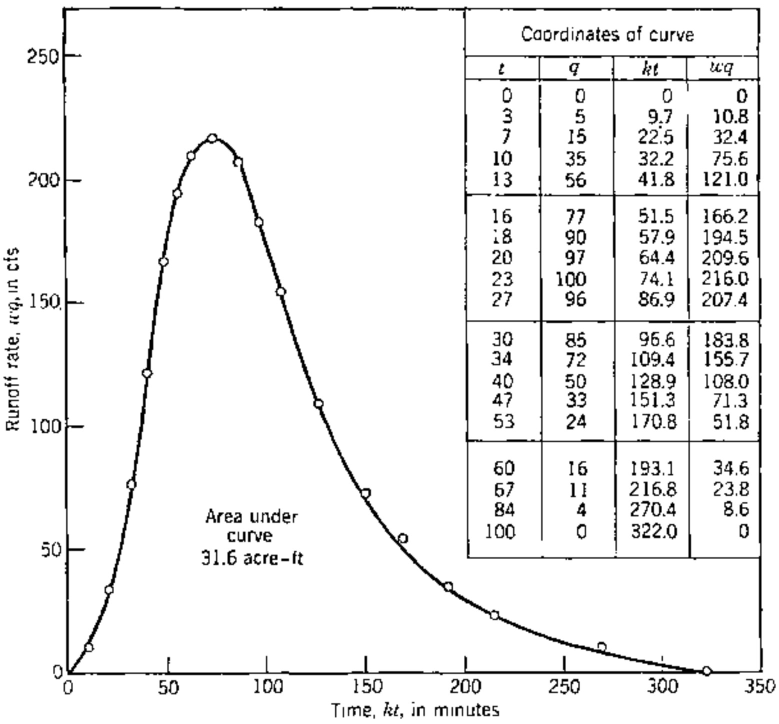
$$wk = u$$

and

$$k = u/w \tag{4.6a}$$

When runoff rate is measured in cubic feet per second, runoff volume is measured in acre-feet, and time is measured in minutes,

$$k = \frac{u(\text{acre-ft}) \times 43{,}560(\text{ft}^2/\text{acre})}{w(\text{cfs}) \times 60(\text{sec/min})} = 726\,\frac{u}{w} \tag{4.6b}$$



Coordinates of curve

t	q	kt	wq
0	0	0	0
3	5	9.7	10.8
7	15	22.5	32.4
10	35	32.2	75.6
13	56	41.8	121.0
16	77	51.5	166.2
18	90	57.9	194.5
20	97	64.4	209.6
23	100	74.1	216.0
27	96	86.9	207.4
30	85	96.6	183.8
34	72	109.4	155.7
40	50	128.9	108.0
47	33	151.3	71.3
53	24	170.8	51.8
60	16	193.1	34.6
67	11	216.8	23.8
84	4	270.4	8.6
100	0	322.0	0

Fig. 4.6. A design hydrograph developed by the basic hydrograph method.

The coordinates of the design hydrograph are obtained by multiplying the ordinates and abscissas of the basic hydrograph by w and k, respectively.

Example 4.4. Develop a runoff hydrograph for a design recurrence interval of 50 years for the watershed of Examples 4.1 and 4.3.

Solution. The peak runoff is 216 cfs, and the flood volume is 31.6 acre-ft. From equations 4.4, 4.5, and 4.6*b*

$$u = \frac{31.6}{3300} = 0.00957 \text{ acre-ft/unit}$$

$$w = \frac{216}{100} = 2.16 \text{ cfs/unit}$$

$$k = 726\,\frac{0.00957}{2.16} = 3.22 \text{ min/unit}$$

Ordinates and abscissas of the design hydrograph are obtained by multiplying the values of q and t from Fig. 4.5 by w and k, respectively. The calculated coordinates and a plot of the hydrograph are given in Fig. 4.6.

METHODS OF MEASURING RUNOFF

The principles discussed earlier in the chapter have been developed from or supported by actual field measurements of runoff. To appreciate the limitations of such measurements and to supplement the existing data with additional measurements, the agricultural engineer should understand the basic methods of runoff measurement.

Measurement of flow in open channels is based upon the relationship

$$Q = av \tag{4.7}$$

where Q is the flow rate through a section of cross-sectional area a and mean velocity v.

4.10. Current Meter. The current meter is widely used in measurement of mean velocity of flow in open channels. Figure 4.7 shows a typical current meter with accessories. The essential part of the meter is a wheel so arranged that it revolves when suspended in flowing water. An electrical circuit is incorporated so as to indicate the speed of revolution of the wheel. The revolving wheel actuates a set of breaker points in the electrical circuit and the revolutions are indicated by clicks in the earphone. The meter may be suspended by a cable for deep streams or attached to a rod in shallow streams. When supported by a cable a streamlined weight holds the meter against the current. A vane attached to the rear of the meter keeps the wheel headed into the stream. The revolutions are counted for a known period of time and the velocity of the current is determined from a calibration curve or equation for the particular meter.

When the mean velocity of a stream is determined with a current meter, the cross section of flow is divided into a number of subareas. Width of the subareas may be from 2 to 20 feet, depending on the size of the stream and the precision desired. This is illustrated in Fig. 4.8. The subareas may be indicated by marks on a tape or cable stretched across the stream or by marks on a bridge railing or other convenient structure. The average velocity at each station across the section is determined with the current meter. It has been found that the average of readings taken at 0.2 and 0.8 of the depth below the surface

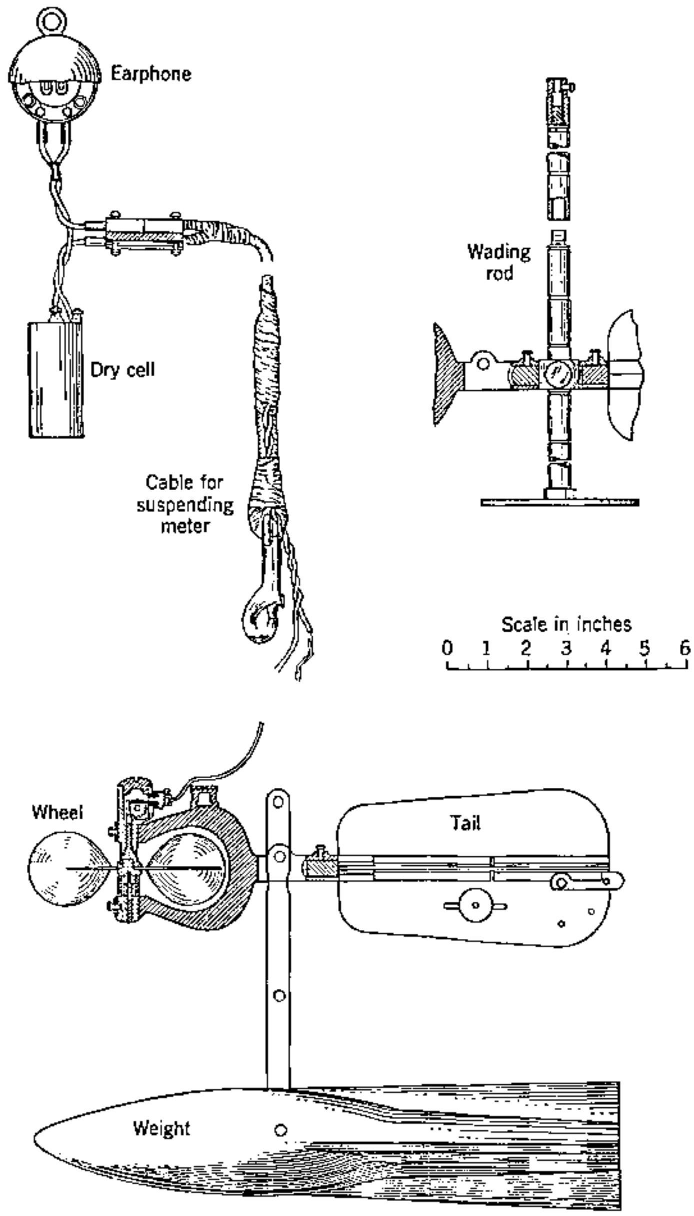


Fig. 4.7. Price current meter and attachments.

is an accurate estimate of the average velocity in the vertical. Where the stream is so shallow as to prevent the taking of a reading at 0.8 of the depth, the velocity at 0.6 of the depth below

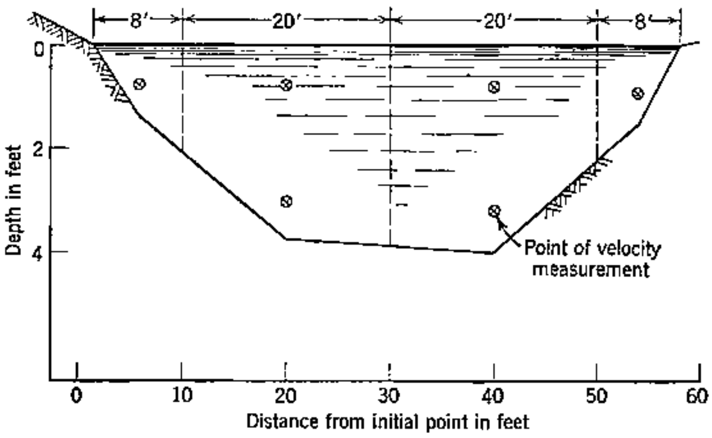


Fig. 4.8. Subdivision of a stream cross section for current meter measurements.

the surface may be taken as the average velocity. The area of the cross section may be determined by sounding with the current meter or other convenient device. Table 4.1 gives the calculation of the discharge for the section shown in Fig. 4.8.

4.11. Float. A crude estimate of the velocity of a stream may be made by determining the velocity of an object floating with the current. A straight uniform section of stream several hundred feet long should be selected and marked by stakes or range poles on the bank. The time required for an object floating on the surface to traverse the marked course is measured and the velocity calculated. The average surface velocity is determined by averaging float velocities measured at a number of distances from the bank. Mean velocity of the stream is often taken as 0.8 to 0.9 of the average surface velocity.

Floats consisting of a weight attached to a floating buoy are sometimes used to measure directly mean velocity in a vertical. The weight is submerged to the depth of mean velocity, and the buoy marks its travel downstream. The float method has the

Table 4.1 CALCULATION OF DISCHARGE FROM CURRENT METER MEASUREMENTS

Gaging of Skunk River at Ames, Iowa Date April 4, 1949
Meter No. SC5514394 Measurement began at 1:15 P.M.
Gage height 2.92 ft Measurement ended at 2:30 P.M.
Gaging made by DeHart and Storm

Distance from Initial Point ft	Width ft	Depth ft	Observation Depth ft	Revolutions	Time seconds	Velocity fps		Area ft²	Discharge cfs
						At Point	Mean in Vertical		
2		0							
6	8	1.33	0.6	5	42	0.28	0.28	10.6	3.0
20	20	3.75	.8	20	41	1.10	1.18	75.0	88.5
			.2	25	45	1.26			
40	20	4.00	.8	15	46	0.74	0.87	80.0	69.6
			.2	20	45	1.00			
54	8	1.50	.6	5	57	0.21	0.21	12.0	2.5
								Total	163.6

advantage of giving an estimate of velocity with a minimum of equipment. The method is, however, obviously lacking in precision.

4.12. Slope Area. The basic equations for velocity of open channel flow may be applied to stream flow measurements. The equation most commonly accepted is the Manning formula,

$$v = \frac{1.486}{n} R^{2/3} s^{1/2} \tag{4.8}$$

where v is the mean velocity of flow, n is the roughness coefficient, R is the hydraulic radius defined by a/p with a the cross-sectional area in square feet and p the wetted perimeter in feet, and s is the gradient of flow. Calculation of velocity from the Manning formula is given in Appendix C. Application of the formula to estimation of flow in open channels requires measurement of the slope of the water surface and measurement of the properties of the cross section of flow. The reach of channel selected should be uniform and if possible as much as 1000 feet long. The value of the roughness coefficient must be estimated and this is difficult to do accurately. Appendix C gives values of n which are helpful in arriving at such estimates.

The slope-area method is sometimes used in estimating the discharge of past flood peaks. Cross-sectional area and flow gradient are measured from high-water marks along the channel. However, the cross section of the channel at the time of such observations may be quite different from that during the peak flood flow. This method must be regarded as giving only a rough approximation of the peak flow.

4.13. Weirs and Flumes. For accurate measurement of flow in open channels it is desirable to install structures of known hydraulic characteristics. Flow through such structures has a consistent relationship between head and discharge.

Weirs. A weir consists of a barrier placed in a stream to constrict the flow and cause it to fall over a crest. The basic equation for flow through such a structure is

$$Q = CLh^m \tag{4.9}$$

where Q is the discharge, C is a coefficient dependent on the nature of the crest and the approach conditions, L is the length of the crest, h is the head on the crest, and the exponent m is dependent upon the shape of the weir opening. Weir openings may be rectangular, trapezoidal, or triangular in cross section, or they may take special shapes to give desired head-discharge relationships. Consult standard hydraulic handbooks and references for detailed discussion of weirs.[9,13,15] A typical temporary weir for measuring stream flow is illustrated in Fig. 4.9.

Flumes. Specially shaped and stabilized channel sections may also be used to measure flow. Such a section is termed a flume. Flumes are generally less inclined to catch floating debris and sediment than are weirs, and for this reason they are particularly suited to measurement of runoff. One common type of measuring device is the Parshall flume.[12] This flume is illustrated in Fig. 4.10. The Parshall flume has the advantage of requiring a very low head loss for operation. Discharge tables for all sizes of flumes are available.[13] When the head at H_b (see Fig. 4.10) is less than 0.7 H_a, the flow for flumes of

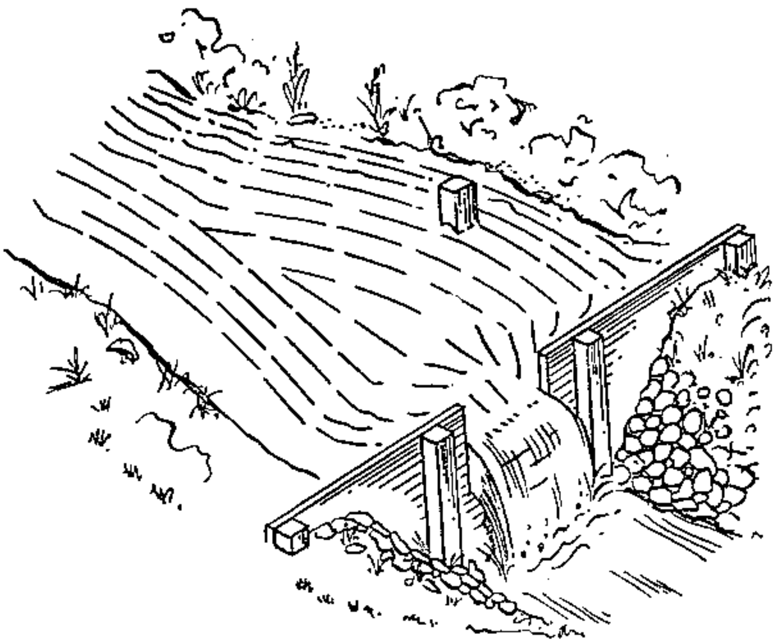


Fig. 4.9. Rectangular weir for measurement of flow in a small stream.

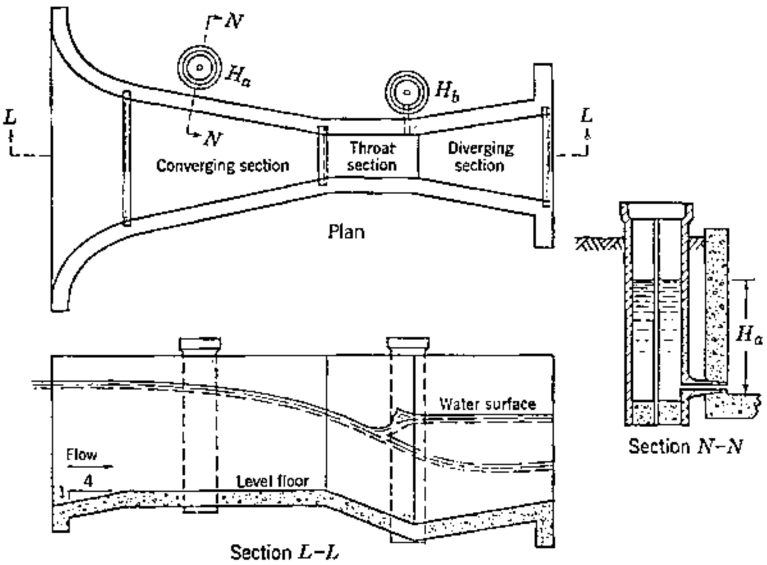


Fig. 4.10. The Parshall measuring flume. (Redrawn from Parshall.[13])

1 to 8 ft throat width is[12]

$$Q = 4WH_a^{1.522W^{0.026}} \tag{4.10}$$

When H_b is greater than $0.7H_a$, both H_a and H_b must be considered in determining the discharge, and reference should be made to the calibration tables.[13]

The Parshall flume is inaccurate at low flows and is therefore not entirely satisfactory for measurement of widely fluctuating runoff. A flume particularly adapted to runoff measurement is

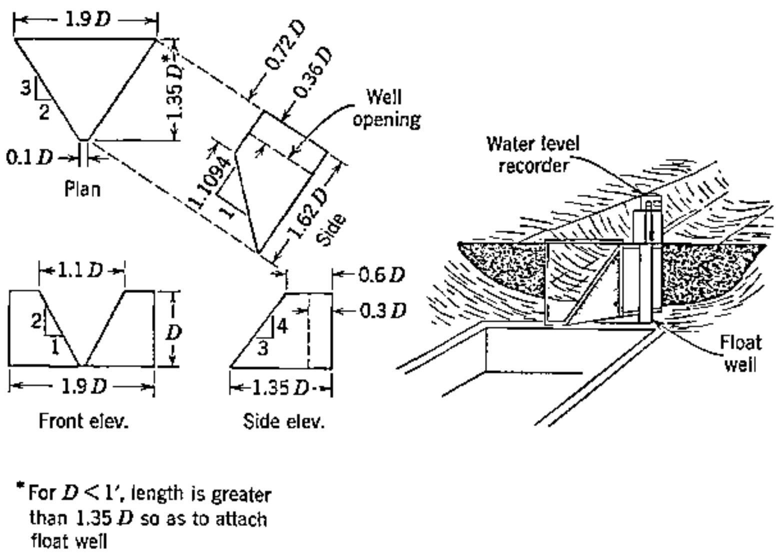


Fig. 4.11. Type H flume. (From Harrold and Krimgold.[6])

shown in Fig. 4.11. Known as the Type H flume, this device has a V-shape which gives accuracy at low flows as well as providing high capacity. Dimensions of the flume are given in Fig. 4.11, and the capacity ratings are given in Table 4.2.

4.14. Water Level Recording Equipment. Stream gaging stations, weirs, and flumes are often equipped with continuous water level recording devices. Such a device is shown in Fig. 4.12. The float rests on the water in a float well which is connected to the main channel by a pipe or trench. As the flow

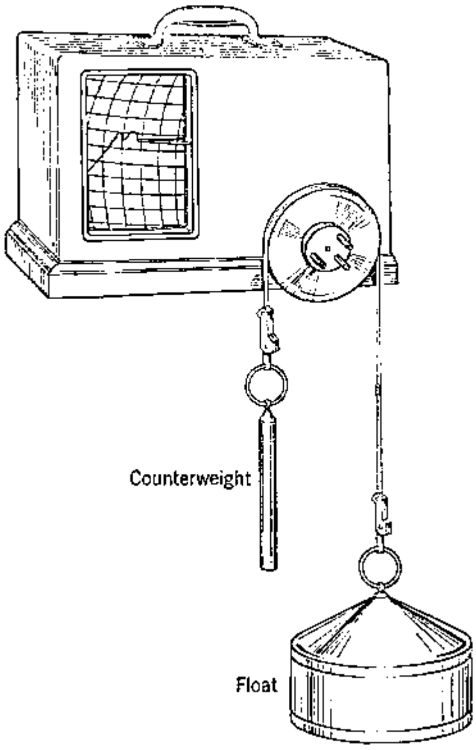


Fig. 4.12. Continuous water level recording device. (Bendix Friez, Model FW-1.)

rises or falls, the float actuates a pen which records the level on a clock-driven chart.

Table 4.2 Rate of Flow through Type H Flumes in cfs*

Flume Depth D in Feet	*Water Depth in Feet*							
	0.1	0.4	0.8	1.0	1.5	2.0	2.5	3.0
0.5	0.0101	0.204						
1.0	0.0150	0.244	1.16	1.96				
1.5	0.0200	0.283	1.27	2.09	5.41			
2.0	0.0248	0.323	1.38	2.25	5.65	11.1		
2.5	0.0298	0.363	1.49	2.41	5.91	11.5	19.4	
3.0	0.0347	0.403	1.61	2.58	6.24	11.9	20.1	31.0

* From Harrold and Krimgold.[6]

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PROBLEMS

4.1. By the rational method, determine the design runoff for a 10-year recurrence interval from 110 acres of rolling cultivated land (10 per cent slopes) located in your area. The soil is loam and one-third of the area is in rotation meadow. Maximum length of travel for the water is 3700 feet, and the difference in elevation between the outlet and most remote point is 20 feet. The entire watershed has well-defined drainageways.

4.2. Determine the runoff for Problem 4.1, using Cook's method.

4.3. What is the weighted runoff coefficient for 10 acres of clay and silt loam hilly timberland, 28 acres of rolling meadow, and 7 acres of urban land, moderately steep with 50 per cent impervious?

4.4. Compute the data for a basic runoff hydrograph for the watershed in Problem 4.1. Assume a uniform infiltration rate of 0.15 iph.

4.5. A stream 24 feet wide is gaged with a current meter at the midpoint of 4-, 8-, 8-, and 4-foot intervals. The depth of the stream and velocities at 0.2 and 0.8 of the depth for each station, respectively, are 4.6 feet, 2.8 and 2.0 fps; 5.8 feet, 4.2 and 2.2 fps; 6.2 feet, 3.5 and 1.7 fps. Record and tabulate data as shown in Table 4.1.

4.6. Determine the discharge of a stream having a cross-sectional area of 200 square feet by the float method. Trial runs for surface floats to travel 300 feet were 122, 128, 123, 124, and 128 seconds.

4.7. Determine the discharge of a stream having a cross-sectional area of 100 square feet and a wetted perimeter of 30 feet using the slope-area method. The channel has some weeds and stones with straight banks and is flowing at full stage. The difference in elevation of the water surface at points 400 feet apart is 0.28 feet.

4.8. Determine the capacity of a Parshall flume having a throat width W of 1.25 feet for $H_a = 1.30$ feet and $H_b = 0.90$ foot.

4.9. Select the smallest type H flume to carry 10 cfs. What is the depth of flow?

CHAPTER 5

Soil Physics

Soil physics is the science dealing with the mechanical behavior of the soil mass, or simply the physical properties of the soil. Soil physics includes mechanical, thermal, electrical, optical, and acoustical properties of the soil. Though the term soil physics has been accepted in agricultural work, soil mechanics is commonly used in engineering practice. There need be no confusion since the above definition of soil physics includes both terms. The application of the principles of soil mechanics is sometimes referred to as soil engineering. There are many excellent up-to-date textbooks on soil mechanics and soil physics.[2,27,29,30,31]

5.1. Agronomic and Engineering Aspects of Soils. Agricultural engineers deal with both the agronomic and engineering aspects of soils. They are concerned primarily with soil properties that influence the engineering phase of tillage, erosion, drainage, and irrigation. From the engineering aspect soil properties are important as they affect the design of such facilities as dams, conservation structures, drainage systems, and distribution works in irrigation.

The agronomist is concerned principally with the physical and chemical properties of topsoil from the standpoint of crop growth, drainage, and erosion. For crop production, the porosity, soil moisture, size and amount of aggregates, and absorption of plant nutrients are most important.

Of greatest interest in relation to earthwork construction are such properties as density, particle size distribution, porosity, and consistency. In engineering the soil is considered as a structural material or as a body on which forces may act. Thus, the science of soil engineering is often referred to as soil mechanics. Soils associated with high bearing capacity and good engineering structural properties in general have poor tilth and are generally low in productivity. The study of soils in engineering is not limited to the portion of the soil in which plants grow, but includes rock, sand deposits, and material at great depths in the earth.

MECHANICAL PROPERTIES OF SOILS

5.2. Simple Soil Properties. The soil is made up of three types of material: solid, gas, and liquid. The relative percentages of these materials in a typical soil are shown in Fig. 5.1.

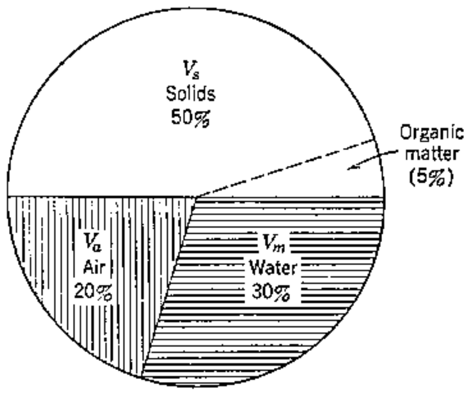

Fig. 5.1. Volume composition of a typical silt loam soil.

Definitions of a few simple soil properties will be given and several formulas will be presented which show the relationship among many of the following symbols:

V = the total volume of soils including solids, liquids, and gases.
V_s = volume of solids including organic matter.
V_e = volume of voids including liquids and gases.
V_a = volume of air.
V_m = volume of liquid.
e = void ratio.
n = porosity in per cent on a volume basis.
n_s = percentage of solids.
G = true specific gravity of the particles.
G_a = apparent or bulk specific gravity or volume weight.
m = moisture content in per cent.
d = dry density in pounds per cubic foot.
d_w = wet density in pounds per cubic foot.
W_d = dry weight of soil.
W_w = wet weight of soil.

The volume relationships shown in Fig. 5.1 may be expressed by the formulas:

$$V = V_s + V_e = V_s + V_a + V_w \tag{5.1}$$

$$e = V_e/V_s \tag{5.2}$$

$$n = (V_e/V)\,100 \tag{5.3}$$

Specific Gravity. Specific gravity is defined as the ratio of the unit weight of a substance to the unit weight of water. True specific gravity refers to the weight of mineral particles and usually ranges from 2.55 to 2.75. Apparent specific gravity is the ratio of the dry weight of a unit volume of soil as it exists in place to the unit weight of water. The following formulas show the relationship of porosity and void ratio to apparent and true specific gravity:

$$G_a = G\left(1 - \frac{n}{100}\right) \tag{5.4}$$

$$G = G_a(1 + e) \tag{5.5}$$

Soil Density. Soil density is defined as the weight per unit volume of soil. Wet density refers to the weight of soil plus water; dry density refers only to the soil. High density indicates high bearing and shearing strength and low permeability. The following equations indicate the relationship of total volume, soil weight, and moisture content to soil density:

$$d = W_d/V \tag{5.6}$$

$$d_w = W_w/V \tag{5.7}$$

There is an optimum moisture content at which maximum density occurs for a given amount of energy applied during the compaction process. A standard test developed for disturbed soils is known as the Proctor density test.[1] A typical Proctor density curve is shown in Fig. 5.2. In conducting a test the soil is placed in a container and compacted in layers with a weight dropped from a certain height for a definite number of times. The moisture content after each compaction is determined, and then water is added for the next test. After several of these tests a curve can be drawn as shown in Fig. 5.2. The density increases with moisture content up to a certain point, above which the density decreases. The maximum or Proctor density occurs at the optimum moisture content for compaction. This test is widely used in engineering construction and has been adopted by the American Association of State Highway Officials.

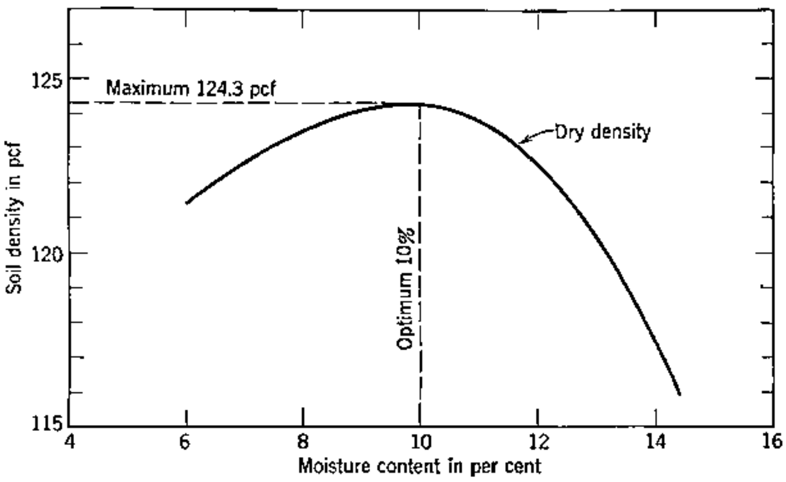

Fig. 5.2. A typical Proctor density curve.

5.3. Textural Classification. The primary soil particles are designated by textural groups as gravel, sand, silt, and clay. Two classification systems commonly used in agricultural work are those developed by the U. S. Department of Agriculture and by Atterberg as shown in Fig. 5.3. It should be noted that the sand fraction is further divided into subgroups.

Of the two methods of describing the textural gradation of a soil, the first distinguishes soil types by textural names. The second method involves a summation percentage of various sizes known as a particle size distribution curve.

A textural classification chart for 12 classes ranging from clay to sand is given in Fig. 5.4. To illustrate, a soil composed of 65 per cent sand, 15 per cent clay, and 20 per cent silt would be classified as sandy loam.

Because it is frequently desirable to estimate the soil texture in the field, the feel of the soil can be tested between the thumb and the finger or in the palm of the hand. If the soil is wet, sand particles feel gritty, silt has a rather smooth and floury feeling, and clay is plastic or sticky.

In engineering practice a common method of showing graphically the textural characteristics of a soil is by means of a particle size distribution curve. Such curves are shown in

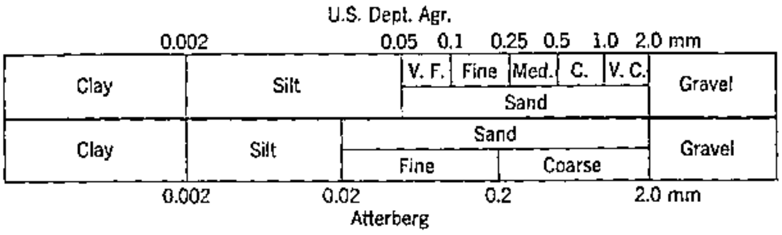

Fig. 5.3. Size limitations of soil fractions for the U. S. Department of Agriculture and Atterberg classification systems. (Data from reference 33.)

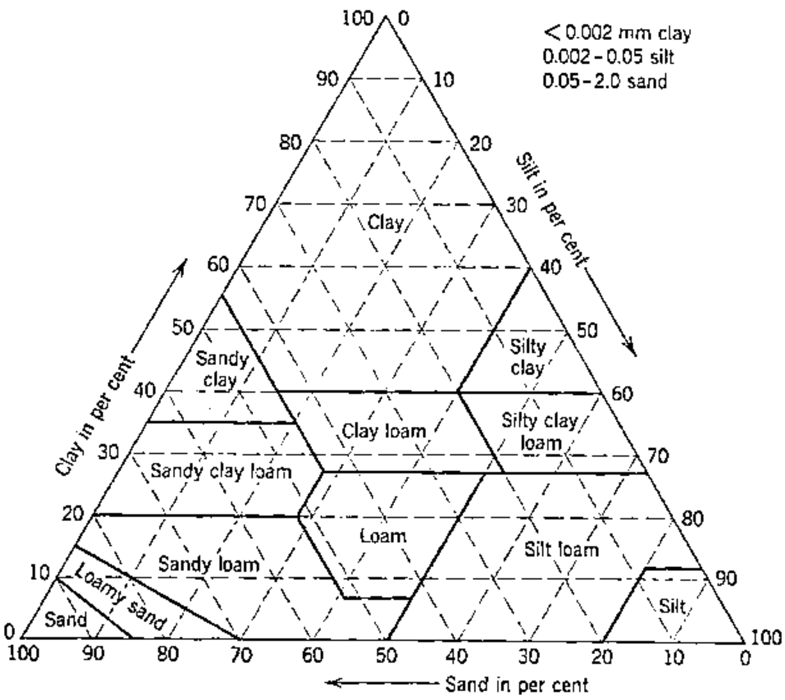

Fig. 5.4. U. S. Department of Agriculture textural classification chart. (Redrawn from reference 33.)

Fig. 5.5 for a well-graded and a uniformly graded soil. The shape of the curve shows at a glance the general composition of the soil. Further description can be obtained from such curves by determining the effective size and the uniformity coefficient. The effective size is defined as the maximum diameter of the smallest 10 per cent by weight of the soil particles; the uniformity coefficient is the maximum diameter of the smallest 60 per cent by weight divided by the effective size. These terms are

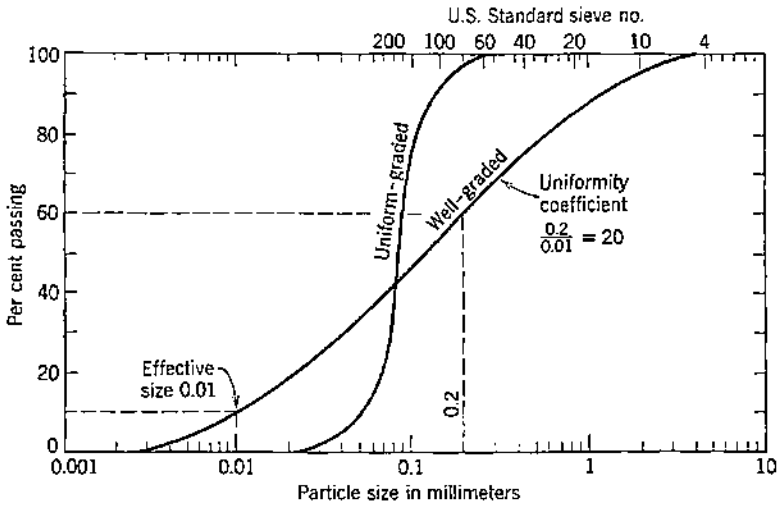

Fig. 5.5. Particle size distribution curve.

arbitrary and serve as a guide in describing a soil, particularly in connection with soil consistency characteristics to be discussed later. A high value for the effective size indicates a high percentage of small-sized fractions; a high uniformity coefficient indicates a well-graded soil. A uniformity coefficient of unity indicates all particles of the same size.

5.4. Porosity. Soil voids are divided arbitrarily into aeration porosity and capillary porosity. Aeration porosity, sometimes referred to as noncapillary porosity, is the percentage of pore space filled with air after the soil has drained to field capacity (see Art. 5.9). Capillary porosity is the percentage of pore space that may be occupied by capillary water.

Aeration porosity influences plant growth, permeability, and apparent density. The quantity of air in the soil is continually changing because of such factors as climate, tillage, tramping of livestock, plant roots, and biological activity. These changes are more pronounced in the topsoil than in the lower horizons. The distribution of aeration and capillary porosity in two soil profiles is shown in Fig. 5.6. The most striking thing about these two profiles is the difference in aeration porosity. In Marshall soil, capillary and aeration porosity are about equally divided; in Shelby soil, capillary porosity is much greater than aeration porosity. Marshall soil has adequate aeration, high infiltration and permeability, and high water-holding capacity; however,

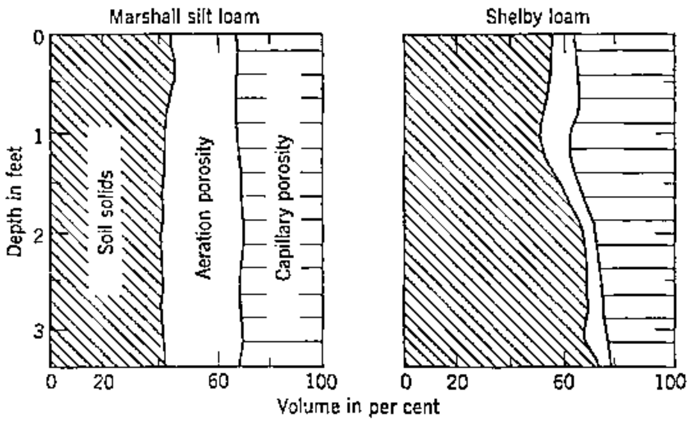

Fig. 5.6. Porosity distribution in the soil profile. (Redrawn from Baver.[2])

the opposite is true for Shelby soil. As the depth increases, total porosity for Shelby soil decreases, indicating characteristics not ideal for plant growth.

In general, porosity changes with soil texture and structure. For example, sand and organic soils have high aeration porosity and clay is low; however, clay is high in total porosity. Soil aggregation has much the same effect on aeration porosity as if the soil were of coarser texture. Aggregates greater than 0.5 millimeter are especially effective in increasing aeration porosity.

5.5. Soil Structure. Soil structure refers to the degree to which individual soil particles are grouped together to form aggregates. Aggregation has a pronounced effect on such soil properties as erodibility, porosity, permeability, infiltration, and water-holding capacity. In general, the greater the aggregation of a soil the lower its erodibility. The presence of large aggregates increases the amount of pore space in the soil, particularly the aeration porosity. Although aggregation is desirable in agricultural soils, it is generally objectionable for construction purposes.

5.6. Soil Consistency. Soil consistency describes the evident characteristics of the soil at various moisture contents when influenced by the physical forces of cohesion and adhesion.

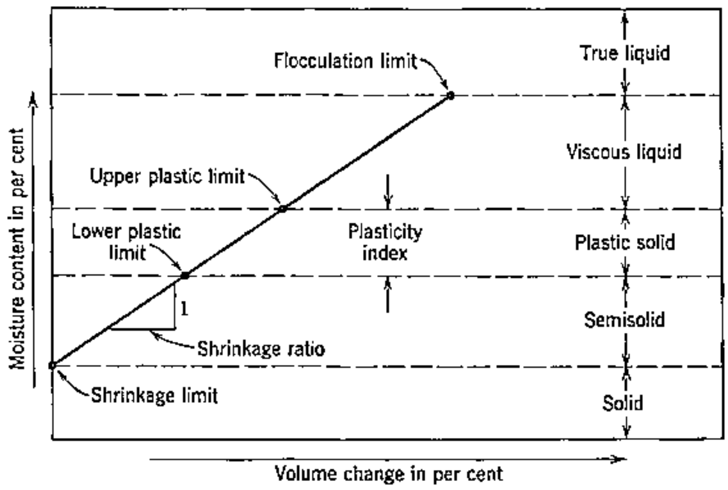

Fig. 5.7. Diagrammatic representation of soil moisture-volume change relationships. (By permission from Hogentogler,[12] *Engineering Properties of Soils*, Revised, McGraw-Hill Book Co., New York, 1937.)

Soil consistency varies with texture, structure, organic matter, percentage of colloidal material, and the shape and type of clay mineral. Several consistency terms express the various conditions of the soil mass, such as plasticity, hardness, and friability. In the application of soils for engineering purposes, consistency limits and mechanical analysis form the basis for separating soils into various categories. For example, each classification of highway subgrade material has maximum or minimum limits for size and plasticity. Soil consistency influences tillage operations and to a certain extent physical properties such as permeability.

The consistency of soil-water mixtures can be illustrated by plotting the volume change of the soil against moisture content as in Fig. 5.7. The physical significance of the following consistency terms is evident from this drawing. Although these relationships are primarily applicable for plastic soils, nonplastic soils such as sand having a small amount of clay follow the same general pattern given in Fig. 5.7. However, such a soil does not go through the plastic solid state and does not form a solid material if the moisture content is reduced to the shrinkage limit.

Flocculation Limit. Soil-water relationships may be visualized by assuming a thin mixture with the soil highly dispersed. This condition represents a true liquid. If the water from such a mixture is evaporated, the moisture content is reduced until the solution is changed from a true liquid to a viscous liquid. The minimum moisture content at which this change takes place is known as the flocculation limit.

Upper Plastic Limit. The upper plastic limit is the minimum moisture content at which the soil-water mixture changes from a viscous liquid to a plastic solid. Between the flocculation limit and the upper plastic limit the mixture is known as a viscous liquid and has all the properties of a true liquid except buoyancy. As the moisture content is decreased below the flocculation limit, the size of the capillary openings and the total porosity of the soil are reduced and thus shrinkage occurs. The upper plastic limit is frequently referred to as the liquid limit; more specifically it is the moisture content at which the soil will barely flow under an applied force.[1]

Lower Plastic Limit. The lower plastic limit is the minimum moisture content at which the soil-water mixture changes from a plastic solid to a semisolid. This is sometimes referred to as the plastic limit and is further defined as the moisture content at which the soil can be rolled into a small cylinder about ⅛ inch in diameter without breaking. The lower plastic limit represents the minimum moisture content at which puddling is possible and the maximum moisture content at which the soil is friable. The lower plastic limit generally represents the point of maximum cohesion in the soil.

Plasticity Index. The plasticity index is the difference in moisture content between the upper and lower plastic limits. This index, sometimes known as plasticity number, is defined as the range of moisture content in which the soil has the properties of a plastic solid.

Shrinkage Limit. The shrinkage limit is the moisture content at which the soil changes from a semisolid to the solid state. At this point further reduction in the moisture content does not effect a change in the volume. The bearing capacity of a soil is greatly increased near the shrinkage limit, and the soil mass acts more like a solid than a plastic material.

Shrinkage Ratio. As shown in Fig. 5.7, the shrinkage ratio is the volume change per unit change of moisture content.

Other Terms. Other soil consistency terms, such as friability, hardness, bulking, and slaking are frequently used. Friability is associated with moisture conditions that are optimum for tillage and usually implies the ease with which the soil can be crumbled. Soil hardness refers to the difficulty of penetration and is related to the denseness or compaction of a soil. Measurement of soil hardness is accomplished by means of a tapered pin dropped from a definite height or by some other type of penetrometer. Bulking is a term applied to cohesionless soils and is the swelling effect due to the adsorption of water on the soil particles. An increase in the volume of sand by adding water is an example. Slaking is the process by which a dry soil mass disintegrates or crumbles upon wetting. The breakdown of clods after a rain is an example.

SOIL MOISTURE

Soil and water conservation engineering is largely concerned with the control of soil moisture. Irrigation, drainage, and erosion control require a knowledge of soil moisture and soil moisture movement. The agricultural engineer working with tillage and planting implements finds that soil moisture conditions play an important part in determining machine performance. To perform their functions to best advantage all agricultural engineers must have a fundamental appreciation of the nature of soil moisture and of the laws that govern its retention and movement in the soil.

5.7. Classification of Soil Moisture. The simplest classification of soil moisture includes three categories:[6]

1. *Hygroscopic moisture:* Water held tightly to the surface of soil particles by adsorption forces.

2. *Capillary moisture:* Water held by forces of surface tension as continuous films around particles and in the capillary spaces.

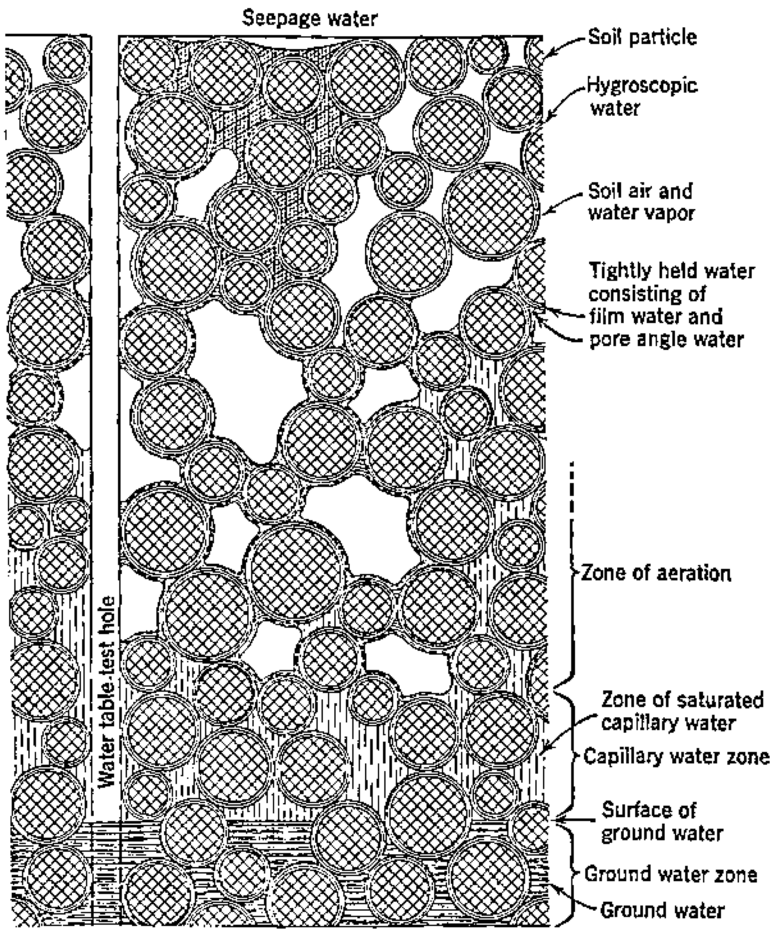

Fig. 5.8. Zunker's classification of soil moisture. (Redrawn from Zunker.[37])

3. *Gravitational moisture:* Water that moves freely in response to gravity and drains out of the soil.

A more comprehensive classification of soil moisture was developed by Zunker[37] and has been presented by Baver[2] as:

1. *Osmotic water*—in cells of organic matter (bacteria, etc.).
2. *Hygroscopic water*—amount of water in soil when it shows no heat of wetting. Attracted on surface of particles by free surface energy forces.
3. *Capillary water*—held by capillary forces in fine soil pores that are connected with the ground water.

4. *"Held" water* (*Haftwasser*)—water held by surface tension forces on soil particles, under normal pressures and capable of movement, but not in union with the ground water.
   (*a*) *Film water*—water on soil particles as a skin.
   (*b*) *"Pore angle" water*—water held in the angles formed by the points of contact of particles.
   (*c*) *Capillary "held" water*—water held by capillaries not connected with ground water.
5. *Gravitational water*—water found in downward or horizontal movement within the zone of aeration.
   (*a*) *Capillary gravitational water*—water that moves downward and laterally in the capillary pores by means of gravity and capillarity.
   (*b*) *Downward gravitational water*—water that moves by gravity through the noncapillary pores to the ground water.
6. *Ground water*—water that fills the tension-free pore space.
7. *Water vapor*—water in vapor form in the soil pores.

This classification gives a most complete picture of the forms in which soil moisture may exist. The various types of moisture are illustrated in Fig. 5.8.

5.8. Energy Concept of Soil Moisture. If a dry column of soil is placed in contact with free water, moisture will rise into the soil. Figure 5.9 represents a homogeneous column of soil that has reached isothermal moisture equilibrium. The forces acting upon each element of moisture in the soil column are in balance. The downward force of gravity is balanced by an upward force which may be termed the moisture potential field force. A potential is defined as the work required to move a unit mass from a point where the potential is zero to the point in question. The work that must be done against the moisture potential field force is the soil moisture potential. The work required to move a unit mass of water, against the moisture potential field force, from the free water surface to point A in Fig. 5.9 is equal numerically but opposite in sign to the work required to move the unit mass of water against the force of gravity. Assume that h in Fig. 5.9

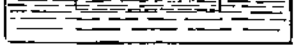

Fig. 5.9. Diagram of soil column at moisture equilibrium with a free-water surface. (Redrawn from Russell.[23])

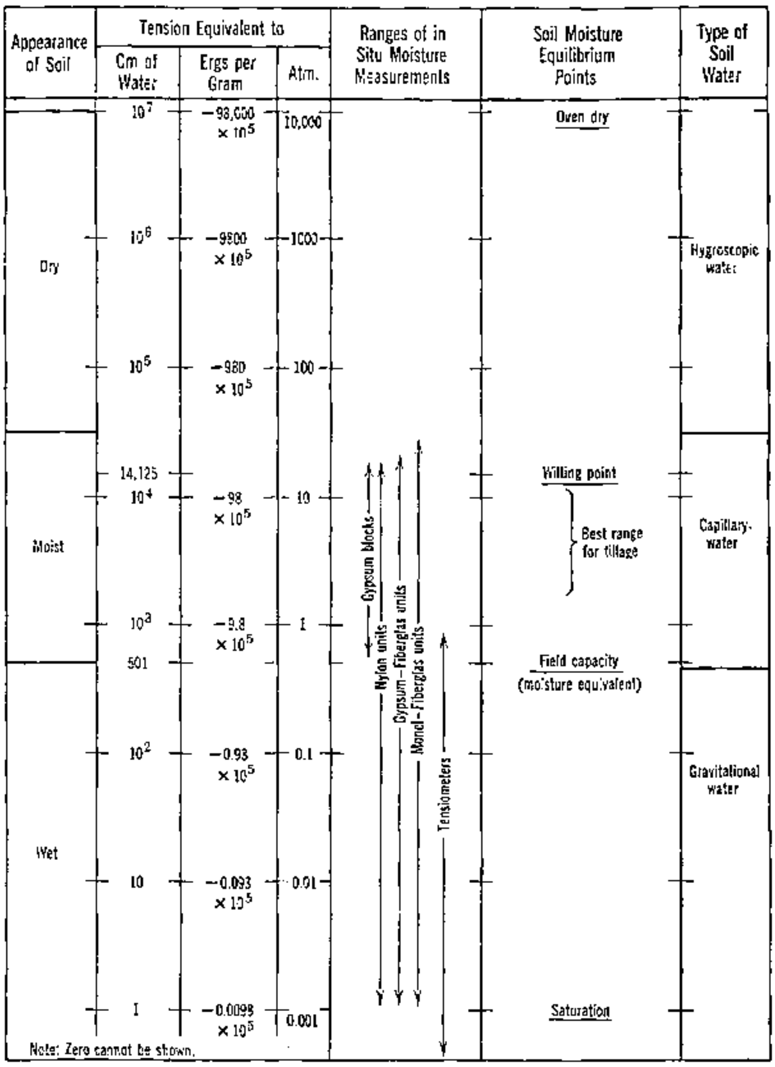

Fig. 5.10. Soil moisture relationships. (Modified from Kohnke.[16])

is 68 centimeters. The work involved in raising water to point A against gravity has been 66,640 ergs/gram, and the work moving the water against the potential field force has been $-66{,}640$ ergs/gram. Thus the moisture potential at A is $-66{,}640$ ergs/gram.

The movement of water from the free water surface to point *A* has been introduced here to give a physical picture of the energy relationships involved. Actually, soil moisture potential

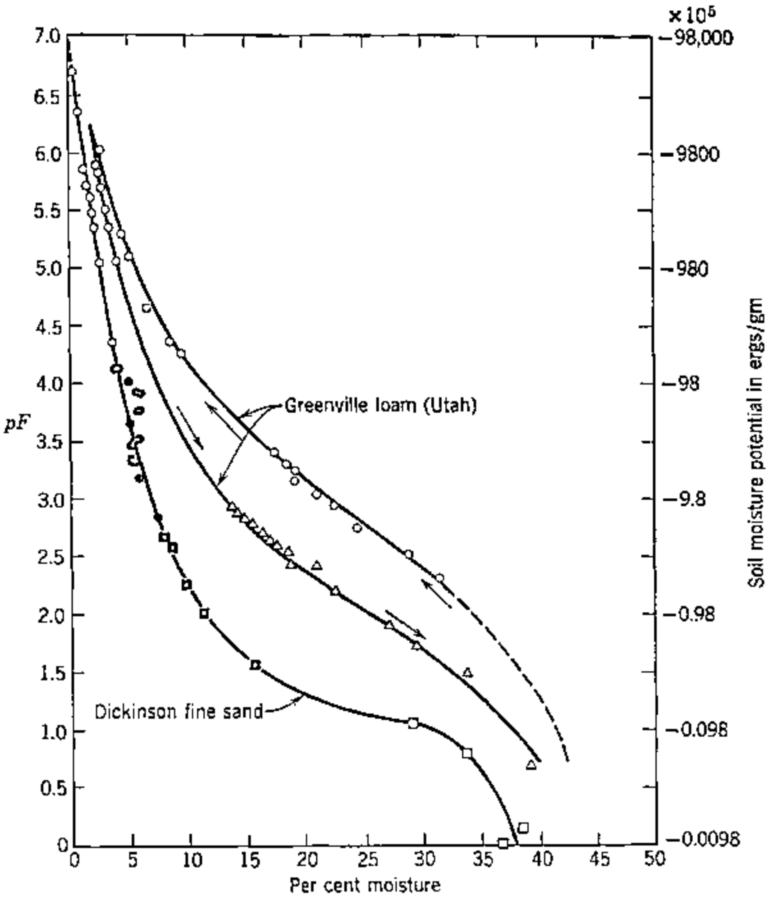

Fig. 5.11. Relationship between soil moisture potential and moisture content for two soils. (Modified from Baver.[2])

is a function of moisture film curvature, which depends upon moisture content and pore size. Any soil identical in properties with the soil in the column considered would have a moisture potential of −66,640 ergs/gram when wetted to the same moisture content as exists at point *A*.

Soil moisture potential has the dimensions of work per unit mass; however, it is commonly expressed in terms of the height of the column of water that could be held by the soil at the given potential. In the above example, the $-66{,}640$ ergs/gram would be indicated by describing the potential as equal to a tension of 68 centimeters. Moisture potential is often expressed in atmospheres. One atmosphere is equivalent to a tension of 1035 centimeters of water. The relationships among atmospheres, centimeters of tension, and ergs per gram as measures of soil moisture potential are given in Fig. 5.10.

The soil moisture potential often is referred to as the capillary potential, because in the high moisture range the forces involved are primarily capillary forces. At tensions of 1000 centimeters or more it is likely that the forces are primarily of molecular origin at the solid-liquid interfaces; at lower tensions surface-tension forces at the air-liquid interfaces are dominant. Regardless of the nature of the forces involved the relationship between moisture potential and moisture content appears as a continuous function. This relationship for two soils is given in Fig. 5.11. The figure shows that a coarse-textured soil holds less moisture at a given potential than is held by a fine-textured soil at the same potential. It should also be noted that the two curves for Greenville loam show a difference of potential at a given moisture content for wetting compared to drying.

5.9. Soil Moisture Equilibrium Points. Certain soil moisture potential levels have particular significance in relation to the water-holding capacity of the soil and to plant growth. The points of most practical importance are saturation, field capacity, moisture equivalent, wilting point, and oven dryness. These five points are marked on the scales of Fig. 5.10. At saturation, all pore space in the soil is filled with water and the potential is zero. The field capacity[35] is defined as the moisture content of the soil after downward movement of water has "materially decreased." As may be judged by the definition, this is the least definite of the equilibrium points. In mineral soils it usually occurs at less than 200 centimeters tension.[22] The moisture equivalent is defined as the soil moisture content held against a force of 1000 times gravity in a specially designed centrifuge. This correlates closely with ⅓ atmosphere tension[22] and is often taken as an approximation of the field capacity.

The wilting point occurs at 15 atmospheres.[22] At this moisture level the potential of the plant root to absorb moisture is balanced by the moisture potential of the soil, and thus soil moisture is not available to the plant. Plants will be permanently wilted if the moisture in the root zone falls to the wilting point. Oven-dry soil has a moisture potential of 10,000 atmospheres.

5.10. Available Moisture. Soil moisture potential and the moisture equilibrium points come into their greatest practical usefulness through the concept of available moisture. The quantity of water present between the field capacity and the wilting point is the moisture available to plants. Knowledge of the soil's ability to hold available moisture is of particular importance in planning and operating irrigation systems. The available moisture may be expressed in per cent but is most useful when expressed as inches of available water that may be held per foot of soil. Soils differ greatly in their available moisture-holding capacity. A sandy soil may hold less than 0.5 inch of available moisture per foot of depth; a clay loam may hold 2 inches of available moisture per foot of soil.

5.11. Measurement of Soil Moisture. *Direct Weighing.* The basic method of soil moisture measurement is to weigh the moist soil, place it in an oven at 105° C until all moisture is driven off as evidenced by no additional loss of weight with additional time in the oven, and then weigh the oven-dry soil. Soil moisture content is usually expressed as per cent by weight, dry basis. For example, 115 grams of moist soil placed in an oven and dried to a constant weight of 95 grams would have a moisture content of

$$100 \frac{115 - 95}{95} = 21.05\%$$

All other methods of measuring soil moisture are calibrated against the direct method.

Gravimetric Plug.[26] The gravimetric plug apparatus consists of a hollow gypsum point embedded in the soil at the level of moisture measurement, a gypsum plug that fits inside the point, and a tube connected to the point and stoppered to prevent evaporation from the tube. The point and the removable plug gain or lose moisture to remain in equilibrium with the soil moisture. The soil moisture is determined by removing and weighing the gypsum plug. The method is not affected by salt concentrations, but it is slow to reach equilibrium with changes in soil moisture.[14]

Electric Resistance. The most commonly used electric resistance method was developed by Bouyoucos.[4,5] Two metal electrodes are embedded in a small gypsum block or in a nylon "sandwich," and the unit is placed in the soil at the level desired for moisture readings. Leads from the electrodes are brought to the surface, where they are attached to a specially designed Wheatstone bridge. A variation of this type of unit consists of a gypsum block containing layers of Fiberglas surrounding the electrodes.[36] Another electrical resistance unit consists of a Monel metal screen Fiberglas sandwich used with a battery-powered ohmmeter.[8]

The resistance units maintain a moisture content in equilibrium with the adjoining soil, and their resistance is affected by their moisture content. The units have the disadvantage of being influenced by salt concentrations in the soil. The gypsum blocks tend to dissolve in the soil water and may not be suited to more than one season's use. They operate satisfactorily in the available moisture range. The nylon unit is sensitive to moisture levels from saturation to the wilting point, as is the gypsum-Fiberglas unit. The Monel-Fiberglas unit is sensitive in a range from saturation to tensions somewhat beyond the wilting point. Figure 5.10 shows the range of application of these units. All of them must be calibrated against oven determinations for the particular soil under study if accuracy is desired.

Tensiometers. The tensiometer is a simple and reliable instrument for determination of soil moisture content in a range of 0 to 850 centimeters tension.[2] A porous clay cup and a manometer are interconnected by a tube filled with water. The cup is filled with water and placed in the soil. If the soil is not saturated, water moves from the cup into the soil until the manometer is drawn to a tension balancing the soil moisture potential. The equilibrium tension is read directly from the manometer. At moisture tensions above 850 centimeters the water column breaks so that the readings are no longer valid. Since these instruments are usable only in the high moisture range, they are most useful under irrigated conditions and particularly on soils having a pore size distribution such that most of the available water is held at tensions less than 850 centimeters.

Other Methods. The variation of thermal characteristics of the soil with moisture content,[25] the influence of moisture content on the dielectric properties of the soil,[9] and neutron scatter by the hydrogen in soil moisture[11] have been proposed as the basis for soil moisture measurement.

SOIL MOISTURE MOVEMENT

Movement of soil moisture takes place in response to a potential gradient and may be expressed by the formula

$$V = -K \frac{d\Phi}{ds} \tag{5.8}$$

which states that the rate of movement V is proportional to the potential gradient $d\Phi/ds$. Φ is the potential, s is distance along the path of greatest change in potential, and K is the conductivity. The negative sign is introduced because movement is in the direction of decreasing potential. The nature of the potential, Φ, and of the conductivity, K, depends upon the soil moisture range in which movement is occurring. The formula is seen to be analogous to Ohm's law for flow of electricity and to Fourier's law for flow of heat.

MOISTURE MOVEMENT UNDER UNSATURATED CONDITIONS

5.12. Capillary Movement. In Art. 5.8 the relationship between soil moisture potential and moisture content was discussed. The dryer the soil, the lower is the potential if other soil properties are constant. As heat flows from high- to low-temperature potentials, moisture flows from high- to low-moisture potentials in accordance with Eq. 5.8. For example, flow will take place from a point where the potential is —98,000 ergs/gram (tension 100 centimeters) toward a point where the potential is —980,000 ergs/gram (tension 1000 centimeters). In a homogeneous soil the first point will have a higher moisture content than the second. However, suppose that a fine sand at 20 per cent moisture is in contact with a loam at 25 per cent moisture. The sand may be at a potential of —63,000 ergs/gram (64 centimeters tension), and the loam at —110,000 ergs/gram (112 centimeters tension) (see Fig. 5.11). The moisture would move from the 20 per cent moisture content sand to the 25 per cent moisture content loam because of the lower potential in the loam.

In capillary movement the K in equation 5.8 is termed the capillary conductivity. Capillary conductivity is a function of soil moisture content as well as number, size, and continuity of soil pores. At moisture contents below the field capacity, capillary conductivity is so low that capillary movement is of little or no significance in relation to plant growth.[22] Many investigations have shown that capillary rise from a free water table can be an important source of moisture for plants only when free water is within 2 or 3 feet of the root zone.

For capillary moisture movement the potential Φ in Eq. 5.8 is the soil moisture potential ϕ plus the gravitational potential ψ. In horizontal movement, only ϕ applies. Under the conditions of downward movement, capillary and gravitational potential act together to produce downward movement. In upward capillary movement ϕ and ψ oppose one another. Letting k be specific for capillary conductivity and v specific for rate of capillary flow,

$$v = -k \frac{d(\phi + \psi)}{ds} \tag{5.9}$$

The direction of s is the path of greatest change in $(\phi + \psi)$.

5.13. Vapor Movement. At moisture contents in the range of very low capillary conductivity there can be little movement of water in the liquid form. At these low moisture levels water movement may occur largely in the vapor phase. The potential causing movement in the vapor phase is the vapor pressure of the soil air. Factors influencing this vapor pressure are the moisture content or vapor pressure of the soil and the temperature of the soil and soil air. Vapor pressure differences resulting from temperature differentials are the most important cause of vapor movement.[2,22] It has been found in one soil in Russia that 66.2 millimeters of water moved from deep horizons to the surface during the winter months (October 26 to March 1) when the surface was cooler than the lower horizons.[17]

MOISTURE MOVEMENT UNDER SATURATED CONDITIONS

5.14. Darcy's Law. A specific form of Eq. 5.8 which applies to unidirectional flow of water through soils under saturated conditions is

$$v = -K\frac{h}{L} \tag{5.10}$$

This form is known as Darcy's law, and v is the apparent velocity of flow, K is the soil permeability, and h/L is the potential gradient. Soil permeability is frequently referred to as hydraulic conductivity. Reference to Fig. 5.12 will clarify

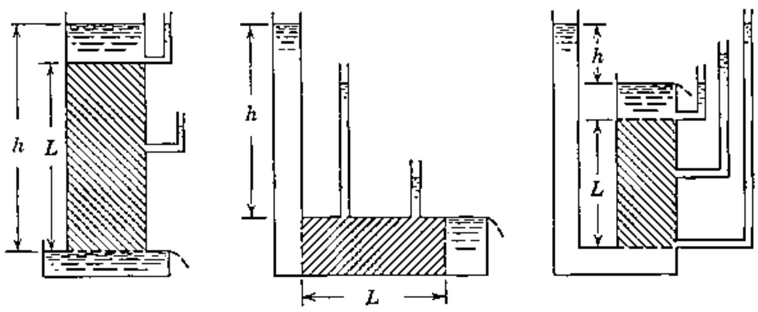

Fig. 5.12. Applications of Darcy's law.

the equation. In Fig. 5.12*a* the direction of flow is downward through a vertical soil column, in Fig. 5.12*b* it is horizontal through the soil column, and in Fig. 5.12*c* it is upward through the soil column. In each illustration piezometers (manometer tubes for measurement of hydraulic head) are attached to the soil column to show the distribution of potential. Discharge from the soil columns would be obtained by multiplying v by the cross-sectional area normal to the direction of flow to give

$$Q = -Ka\frac{h}{L} \tag{5.11}$$

Darcy's law applies so long as the velocity of flow and the size of soil particles are such that the Reynolds number, defined as $\rho\, dv/\mu$, is less than 1 where ρ is the density of the fluid, d is the mean diameter of soil particles, and μ is the dynamic viscosity of the fluid. An example of a simple flow problem that may be solved by direct application of Darcy's law is given in Fig. 5.13.

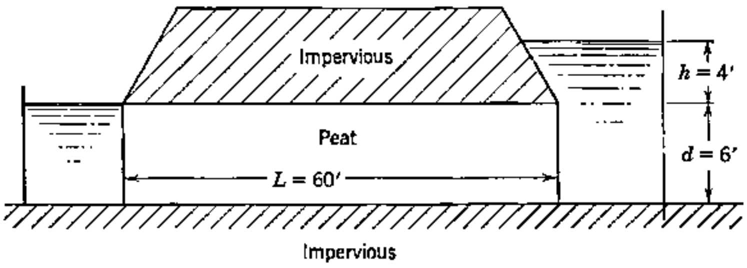

Fig. 5.13. Application of Darcy's law to a simple flow problem.

Example 5.1. A roadway fill impervious to flow rests on peat which in turn rests upon impervious clay (Fig. 5.13). At one side of the roadway a drainage ditch is filled with water to a level 4 feet above the peat, and on the other side of the roadway a ditch is filled with water to the surface of the peat. If the permeability of the peat is 0.8 iph determine the seepage per day under ½ mile of road.

Solution.

$$h = 4 \text{ ft} \qquad a = (6 \text{ ft})\left(\frac{5280 \text{ ft}}{2}\right) = 15{,}840 \text{ sq ft}$$

$$L = 60 \text{ ft} \qquad K = 0.8 \text{ iph}$$

By equation 5.11,

$$Q = \frac{0.8 \times 24}{12} \times 15{,}840 \times \frac{4}{60} = 1690 \text{ ft}^3/\text{day}$$

5.15. LaPlace's Equation. Application of Darcy's law and the equation of continuity to three-dimensional flow of an incompressible fluid through a porous medium results in the derivation of LaPlace's equation:

$$\frac{\partial^2 \Phi}{\partial x^2} + \frac{\partial^2 \Phi}{\partial y^2} + \frac{\partial^2 \Phi}{\partial z^2} = 0 \tag{5.12}$$

The equation states that the second partial derivatives of the potential with respect to x, y, and z sum to zero. This equation also applies to steady-state flow of heat and electricity. Solutions of this differential equation have been developed for many flow problems, some of which will be discussed in a later chapter. Reference should be made to textbooks on applied mathematics and physics for derivation and applications of LaPlace's equation.[7,20]

5.16. Flow Nets. A flow net is a plot of lines of equal potential and paths of flow. These are referred to, respectively, as equipotential lines and streamlines. Flow nets are conventionally drawn so that the flow between any two adjacent streamlines is equal to the flow between any other two adjacent streamlines. Figure 5.14 shows a flow net for the problem of

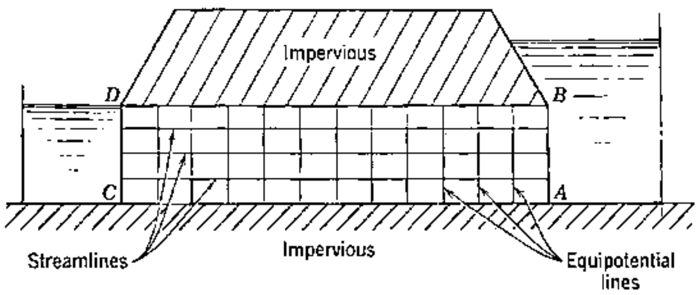

Fig. 5.14. Flow net for one-dimensional flow.

Fig. 5.13. The flow net for a more complex problem is given schematically in Fig. 5.15. In this situation parallel sheet piling has been driven into a saturated soil to form a canal above the soil surface. Water impounded in the channel seeps down between the piling and back up to the soil surface outside the piling. The piezometers show the level at which water would stand in tubes having their open ends on the various equipotential lines. The permeability is uniform over the flow area, but its value is not specified. The appearance of the flow net is independent of permeability in a homogeneous soil. In a nonhomogeneous soil the flow net is influenced by the relative permeabilities of various layers but not by the absolute values of the permeability. Streamlines always intersect equipotential lines at right angles. Just as surface water flows downhill in the direction of greatest slope, i.e., perpendicular to contour lines, seepage water flows in the direction of greatest hydraulic slope, i.e., perpendicular to the equipotential lines.

5.17. Methods of Determining Flow Patterns. In some flow problems the flow net may be plotted directly from an

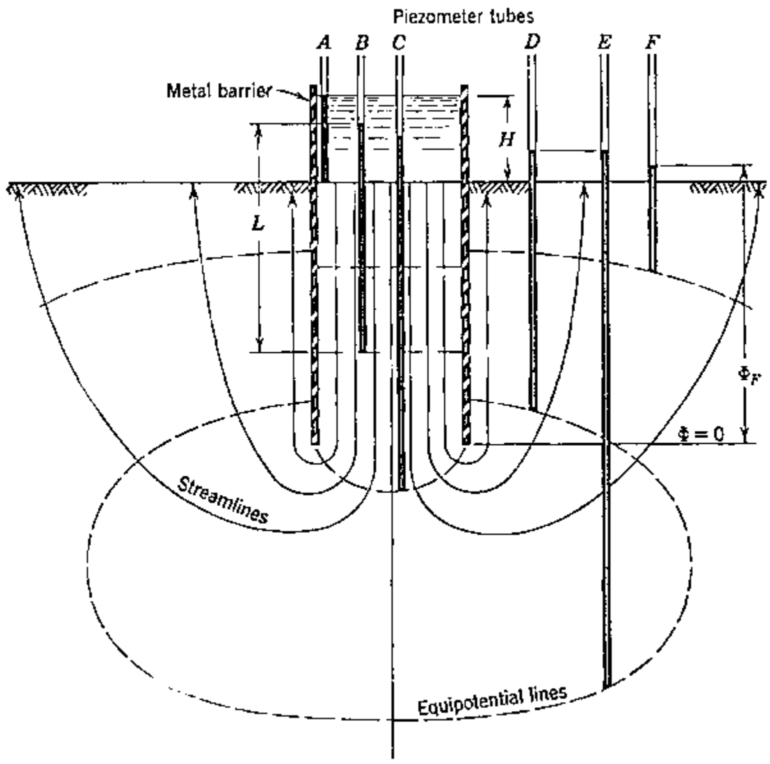

Fig. 5.15. Streamlines and equipotential lines.

analytical solution of equation 5.12. Where analytical solutions cannot be made it is often possible to arrive at a solution by application of numerical approximation (iteration) methods for solutions of potential flow problems.[18,28] Solutions may also be obtained by the method of trial sketching.[27]

Flow nets are often determined by study of models and analogues. Sand tank models may be constructed with glass sides. Dyes injected into the flow trace out the streamlines, and equipotential lines may be determined from piezometers inserted into the model. Since equation 5.12 also applies to flow of electricity, flow nets may be determined with electric circuits that are geometrically similar to the actual flow problem.[10,24] A tank of electrolyte is arranged to have the same geometry of flow boundaries as the problem under study. An electrical potential is impressed upon the system to correspond to the hydraulic potential in the actual flow problem. Lines of equal potential are located by measurement of the distribution of electrical potential in the electrolyte. The electrical analogue method is particularly valuable for three-dimensional flow problems.

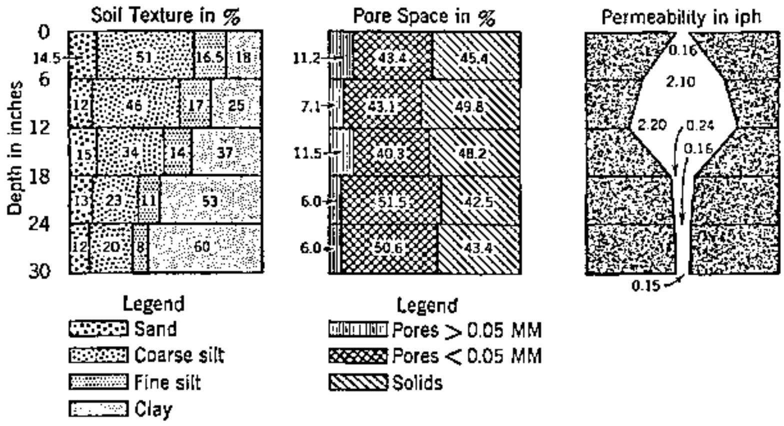

Fig. 5.16. Physical characteristics of Shelby loam, Harrison Co., Mo. (Redrawn from Uhland and O'Neal.[32])

Flow nets may also be developed from field measurements.[15] Piezometers are placed in the flow area, and the hydraulic potential is measured and plotted on a diagram of the flow area. Equipotential lines may then be drawn according to the same procedure employed in drawing contour lines on a topographic map.

5.18. Permeability. Permeability was defined mathematically by equation 5.11. In specific application to soil moisture movement, permeability is the hydraulic conductivity of saturated soil. It is usually measured in inches per hour or in inches per day. The permeability is the apparent velocity of flow in response to a unit hydraulic gradient. Soil permeability is related to the size, total volume, and size distribution of soil pores. Permeability varies with different horizons in a given soil profile, and it may have different values in the horizontal and the vertical direction. Figure 5.16 shows the variation of permeability and other physical properties of one soil profile.

MEASUREMENT OF SOIL PERMEABILITY

Practical applications of knowledge of the nature of saturated moisture flow in soils depend upon evaluation of the permeability of the soil material under study. Such measurements are difficult to obtain accurately. Methods have been developed for measurement of permeability in the laboratory and in the field. Field methods have the advantage of measurement of permeability with the soil in an undisturbed state and with water as it exists under field conditions. Field methods as yet are unfortunately limited by the necessity of having saturated soil conditions existent where measurements are desired. Laboratory methods are more independent of field conditions but involve disturbance of the soil.

5.19. Laboratory Methods of Measuring Permeability. Laboratory measurement of permeability is accomplished by direct application of Darcy's law. Core samples obtained in the field are placed in a permeameter to produce a geometry of flow similar to any of those illustrated in Fig. 5.12. Discharge rate through the core is measured, and the permeability is calculated directly from equation 5.11. Details of laboratory permeability methods are given in references on soil mechanics[27] and ground water flow.[20]

5.20. Field Methods of Measuring Permeability. Field methods of permeability measurement involve observation of the rate of ground water flow into holes penetrating the saturated soil. One method employs a 4-inch diameter auger hole bored to a depth of 12 to 30 inches below the water surface. Water is pumped from the auger hole. As the hole refills the rate of rise of water is observed, and the permeability is determined from this rate. A second method uses a hole lined with thin-walled pipe and having an unlined section 4 inches long below the end of the pipe. Rise of water in the pipe is observed, and permeability is computed as before. Details of these methods are given in reference 13. These methods of measurement of soil permeability are based upon analytical solutions of equation 5.12 with certain constants evaluated through use of three-dimensional electric analogues.

Field techniques measure flow through a relatively large volume of soil as compared to the core sample method. If the volume of a core 2 inches in diameter and 2 inches long is taken as unity, the sample sizes used in permeability measurement by a 1-inch pipe, an 8-inch pipe, and a 4-inch auger hole are 1, 35, 270, and 1400, respectively.[21]

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PROBLEMS

5.1. Determine the U.S.D.A. textural classification of a soil containing 70 per cent clay, 10 per cent silt, and 20 per cent sand. One containing 10 per cent clay, 30 per cent silt, and 60 per cent sand.

5.2. By plotting on semilog paper the particle size distribution curve, determine the effective size and the uniformity coefficient for soils A and B.

Per Cent Passing—Sieve Size in Millimeters

Soil	0.42	0.25	0.105	0.074	0.050	0.005	0.001
A	100	95	85	80	54	25	5
B		100	99	98	95	9	6

5.3. During the construction of an earth dam 10.28 pounds of soil with a specific gravity of 2.68 were removed from a hole whose volume was 0.1 cubic foot. If the soil contained 10.2 per cent moisture, what is the wet density of the soil? Dry density? Void ratio? Porosity?

5.4. If 8000 cubic yards of soil with a void ratio of 1.20 are removed from a borrow area, how many cubic yards of fill with a void ratio of 0.80 could be constructed, using sheep's foot rollers? If the soil has a specific gravity of 2.65 and contains 15 per cent moisture, how many tons of material will have to be moved? How many tons of water?

5.5. If the moisture content of a soil at the upper and at the lower plastic limit is 30 per cent and 22 per cent, respectively, what is the plasticity index? What is the shrinkage ratio, if the volume change between these two limits is 4 per cent?

5.6. Convert a moisture potential of 130 centimeters of water to ergs per gram. To atmospheres of tension.

5.7. If the moisture potential at point A in a uniform soil is 8640 centimeters of water and the moisture potential at point B is 40 centimeters, will moisture move from A to B or from B to A? Is the moisture content suitable for tillage at A and at B?

5.8. Determine the permeability of a soil sample 7.5 centimeters in diameter and 10 centimeters in length, using a permeameter similar to Fig. 5.12*a*. With 2 centimeters of water above the soil surface, 885 cubic centimeters passed through the soil in 45 minutes.

CHAPTER 6

Soil Erosion Principles

The two major types of erosion are geological erosion and accelerated erosion. Geological erosion includes soil-forming as well as soil-eroding processes which maintain the soil in a favorable balance, suitable for the growth of most plants; accelerated erosion includes the deterioration and loss of soil as a result of man's activities. Though both types of soil removal are recognized, only accelerated erosion is considered in conservation activities.

6.1. Geological Erosion. Geological erosion, sometimes referred to as natural or normal erosion, is found when the soil is in its natural environment under the protective cover of native vegetation. This type of erosion has contributed to the formation of our soils and their distribution on the surface of the earth. This long-time eroding process caused most of our present topographic features, such as canyons, stream channels, and valleys.

Practically all geological erosion takes place as a result of the action of water, wind, gravity, and glaciers. Water causes erosion through sheet runoff, stream flow, wave action, and ground water flow. Wind picks up and transports soil particles, thus causing a general mixing of the soil at the surface. Gravity causes mass movement, such as soil creep, rock creep, mudflow, rock slide, and subsidence of the soil surface.

6.2. Accelerated Erosion. Accelerated erosion is soil loss in excess of geological erosion. It is normally associated with changes in natural cover or soil conditions and is caused primarily by water and wind. The forces involved in accelerated erosion are (1) attacking forces which remove and transport the soil particles and (2) resisting forces which retard erosion. Hereafter, accelerated erosion will be referred to as soil erosion or simply erosion.

Erosion is one of the most important agricultural problems in the United States and in the world. In addition to soil losses there are large losses of plant food elements which are removed in the runoff.

WATER EROSION

Water erosion is the removal of soil from the land's surface by running water, including runoff from melted snow and ice. Water erosion is subdivided into raindrop, sheet, rill, gully, and stream channel erosion.

6.3. Factors Affecting Erosion by Water. Since, as discussed later, there is a direct relationship between total runoff and soil loss from agricultural areas, the factors affecting runoff also influence soil loss. The major variables affecting soil erosion are climate, soil, vegetation, and topography. Of these the vegetation and to some extent the soil may be controlled. The climatic factors and the topographic factors, except slope length, are beyond the power of man to control.

Climate. Climatic factors affecting erosion are precipitation, temperature, wind, humidity, and solar radiation. Temperature and wind are most evident through their effects on evaporation and transpiration. However, wind also changes raindrop velocities and the angle of impact. Humidity and solar radiation are somewhat less directly involved in that they are associated with temperature.

The effect of precipitation on soil and water losses is illustrated in Fig. 6.1. Soil loss and runoff are here compared for different amounts and intensities of rain. Total soil loss varies with the total runoff for the different rainfall amount groups. The soil loss per inch of runoff, however, decreases as the size of the storms increases. When storms are grouped by rainfall intensities, the soil loss per inch of runoff increases as the intensity increases, even though the total rainfall decreases. In many instances one or two high-intensity rains may cause as much soil loss as all other storms during a season or during several seasons.

Soil. Physical properties of soil affect the infiltration capacity and the extent to which it can be dispersed and transported. These properties which influence erosion include soil structure, texture, organic matter, moisture content, and density or compactness, as well as chemical and biological characteristics of the soil. As yet no one soil characteristic or index provides a satisfactory means of predicting erodibility.

Vegetation. The major effects of vegetation in reducing erosion are (1) interception of rainfall by absorbing the energy of the raindrops and thus reducing runoff, (2) retardation of erosion by decreased surface velocity, (3) physical restraint of soil movement, (4) improvement of aggregation and porosity

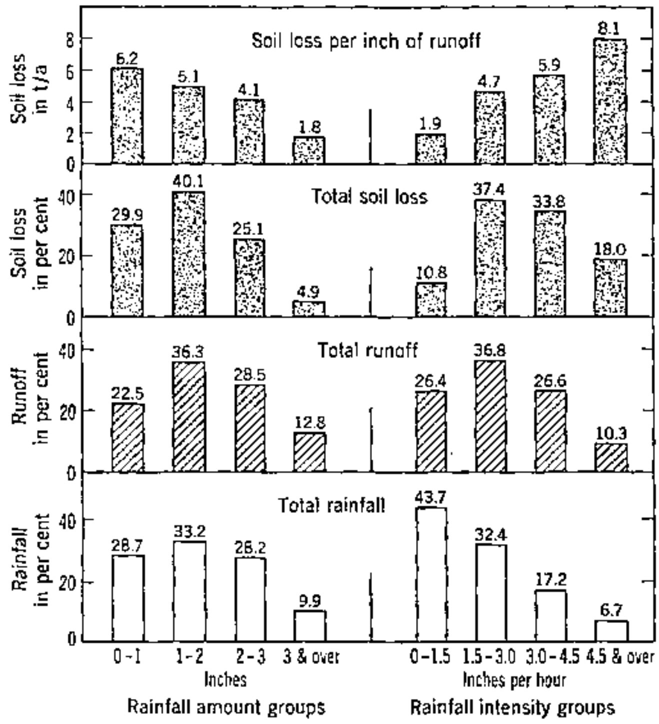

Fig. 6.1. Effect of rainfall and runoff on soil loss from a sandy clay loam soil in North Carolina. (Redrawn from Copley and others.[5])

of the soil by roots and plant residue, (5) increased biological activity in the soil, and (6) transpiration, which decreases soil moisture, resulting in increased storage capacity. These vegetative influences vary with the season, crops, degree of maturity, soil, and climate, as well as with the kind of vegetative material, namely, roots, plant tops, and plant residues.

Topography. Topographic features that influence erosion are degree of slope, length of slope, and size and shape of the watershed. On steep slopes high velocities cause serious erosion by scour and by sediment transportation.

RAINDROP EROSION

Raindrop erosion is soil splash resulting from the impact of water drops directly on soil particles or on thin water surfaces. Although the impact on water in shallow streams may not splash soil, it does cause turbulence, providing a greater sediment-carrying capacity.

Tremendous quantities of soil are splashed into the air, most of it more than once. The amount of soil splashed into the air as indicated by the splash losses from small elevated pans was found to be 50 to 90 times greater than the washoff losses.[11] On bare soil it is estimated that as much as 100 tons of soil per acre are splashed into the air by heavy rains.[7]

6.4. Characteristics of Raindrops. Characteristics of raindrops affecting erosion are drop size and velocity (see Chapter 2). Large drops may increase the sediment-carrying capacity of runoff as much as 12 times.[15] The velocity of raindrops greatly affects soil splash and erosion. From a study of four soils Ellison[8] found that the relative soil loss for a 30-minute rainfall period varied as the 4.33 power of the velocity.

6.5. Soil Detachment and Transportation. The process of soil erosion involves soil detachment and soil transportation. The corresponding soil characteristics that describe the ease with which soil particles may be detached and transported are soil detachability and soil transportability. In general, soil detachability increases as the size of the soil particles increase, and soil transportability increases with a decrease in particle size. That is, clay particles are more difficult to detach than sand, but clay is more easily transported.

On level land raindrop splash is not serious, but on sloping fields considerably more soil is splashed downhill than uphill. This may account to a large extent for serious erosion on short, steep slopes. Sheet and rill erosion increase with the length of slope and are more serious at the lower end of the field, whereas raindrop erosion occurs over the entire area.

The effect of a raindrop as it strikes a thin water surface is shown in Fig. 6.2. The effect of a single drop as it splashes is shown in the left photograph; the illustration on the right shows the splash pattern of many raindrops during a rainstorm.

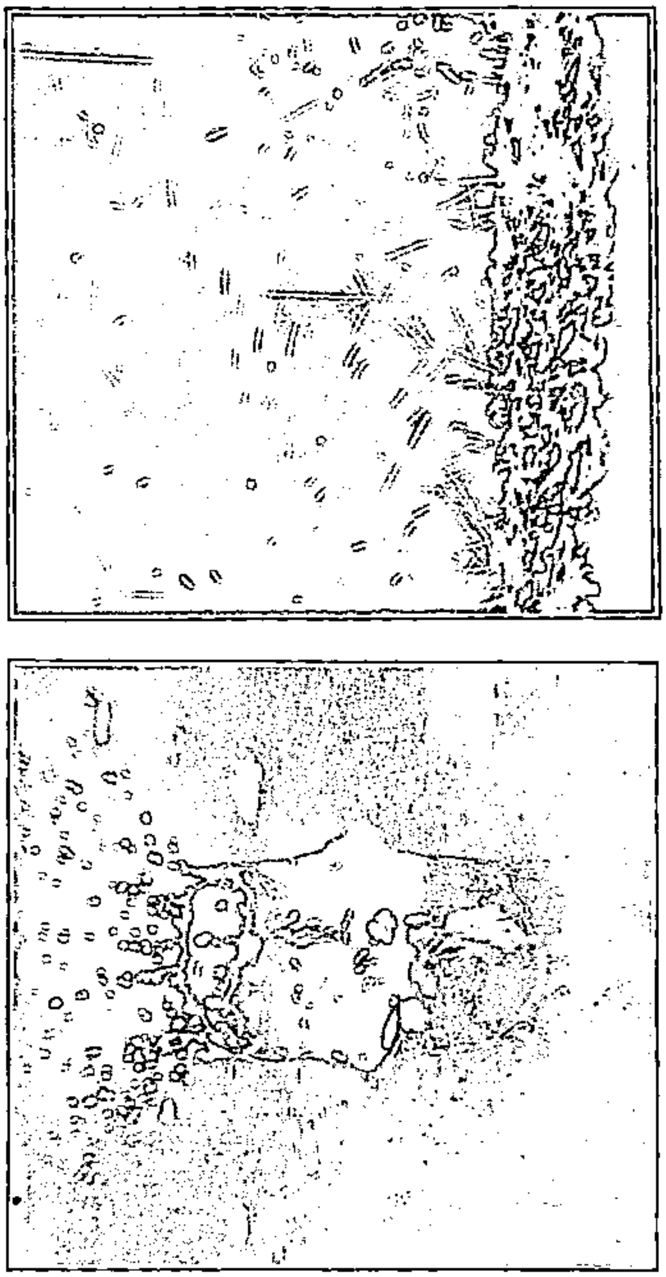

Fig. 6.2. Raindrop splash. *Left:* Impact from a single drop. *Right:* Splash pattern during a storm. (Courtesy Soil Conservation Service.)

Splashed particles may move more than 2 feet in height and more than 5 feet laterally on level surfaces.[7]

Although no mathematical analysis has yet been developed, the relationship between soil detachment and erosion is close. Detachment causes damage because (1) the soil particles are removed from the soil mass and thus easily transported, (2) the fine material and plant nutrients are removed, and (3) seeds may be separated and washed out of the soil. The washing-out of fine materials results in so-called erosion pavement, which is the result of an accumulation of coarse particles or rock fragments at the surface. This effect may take place with either of the principal erosive agents, water or wind. Soil detachment increases with an increase in the height of plant cover and decreases with the density of cover.

Factors affecting the direction and distance of soil splash are slope, wind, surface condition, and such impediments to splash as vegetative cover and mulches. On sloping land the splash moves farther downhill than uphill not only because the soil particles travel further, but also because the angle of impact causes the splash reaction to be in a downhill direction. Components of wind velocity up or down the slope have an important effect on soil movement by splash. Surface roughness and impediments to splash tend to counteract the effects of slope and wind. Contour furrows and ridges break up the slope and cause more of the soil to be splashed uphill. If raindrops fall on crop residue or growing plants, the energy is absorbed and thus soil splash is reduced. Raindrop impact on bare soil not only causes splash but also decreases aggregation and causes deterioration of soil structure.

SHEET EROSION

Sheet erosion is the uniform removal of soil in thin layers from sloping land. This results from sheet or overland flow, the runoff from the surface in thin layers. Although important, sheet erosion is often unnoticed because it occurs gradually.

The beating action of raindrops combined with surface flow causes the major portion of sheet erosion. From an energy standpoint raindrop erosion is far more important because raindrops have velocities of about 20 to 30 fps, whereas overland flow velocities are about 1 to 2 fps. Raindrops cause the soil particles to be detached and the increased sediment reduces the infiltration rate by sealing the soil pores. Areas where loose, shallow topsoil overlies a tight subsoil are most susceptible to sheet erosion. The eroding and transporting power of sheet flow are functions of the depth and velocity of runoff for a given size, shape, and density of soil particle or aggregate.

RILL EROSION

Rill erosion is the removal of soil by water from small but well-defined channels or streamlets when there is a concentration of overland flow. There is no sharp line of demarcation where sheet erosion ends and rill erosion begins. Rills are small enough to be easily removed by normal tillage operations. Although rill erosion is more apparent than sheet erosion, it likewise is often overlooked. Detachability and transportability are both greater in rill erosion than in sheet erosion because of higher velocities. Rill erosion is most serious where intense storms occur on soils having high runoff-producing characteristics and loose, shallow topsoil.

GULLY EROSION

Gully erosion produces channels larger than rills. These channels carry water during and immediately after rains, and, as distinguished from rills, gullies cannot be obliterated by tillage. Thus, gully erosion is an advanced stage of rill erosion much as rill erosion is an advanced stage of sheet erosion.

6.6. Principles of Gully Erosion. The rate of gully erosion depends primarily on the runoff-producing characteristics of the watershed; the drainage area; soil characteristics; the alignment, size, and shape of the gully; and the slope in the channel.

A gully develops by processes that may take place either simultaneously or during different periods of its growth. These processes are (1) waterfall erosion at the gully head, (2) channel erosion caused by water flowing through the gully or by raindrop splash on unprotected soil, (3) alternate freezing and thawing of the exposed soil banks, and (4) slides or mass movement of soil in the gully. Four stages of gully development are generally recognized:

Stage 1. Channel erosion by downward scour of the topsoil. This stage normally proceeds slowly where the topsoil is fairly resistant to erosion.

Stage 2. Upstream movement of the gully head and enlargement of the gully in width and depth. The gully cuts to the *C* horizon, and the weak parent material is rapidly removed. A waterfall often develops where the flow plunges from the upstream segment to the eroded channel below.

Stage 3. Healing stage with vegetation beginning to grow in the channel.

Stage 4. Stabilization of the gully. The channel reaches a stable gradient, gully walls reach a stable slope, and vegetation begins to grow in sufficient abundance to anchor the soil and permit development of new topsoil. The healing stage is a necessary prelude to stabilization, and the one stage grades into the other.

During the two latter stages the gully head has progressed toward the upper end of the watershed, and the rate of runoff into the gully head decreases because the drainage area is reduced. The remainder of the runoff enters at many points along the length of the gully.

6.7. Classification of Gullies. Of the several systems of gully classification, the one given in Table 6.1 is based on an

Table 6.1 Description of Gullies and Drainage Areas

Description	*Gully Depth, ft*	*Drainage Area, acres*
Small	3 or less	5 or less
Medium	3 to 15	5 to 50
Large	15 or more	50 or more

arbitrary classification of gully sizes and drainage areas as small, medium, and large.

Another system classifies gullies with respect to their cross sections. Gully cross sections may be V- or U-shaped, depending upon soil and climatic conditions, age of the gully, and type of erosion. U-shaped gullies may be found in loessial regions and alluvial valleys where both the surface soil and the subsoil are easily eroded. Under such conditions gullies tend to develop vertical walls which result from undermining and collapse of the banks. The scouring of soil by concentrated runoff in unprotected depressions results in V-shaped gullies having sloping heads. Such gullies may develop where the subsoil is resistant to erosion. Both V- and U-shapes are commonly found in the same channel. More precise classifications based on gully shape have been proposed.[13]

As gullies work upstream, the most active portion is near the upper end or at the gully head, whereas the most stable section of the gully is generally at the lower end. Active gullies are gullies that continue to enlarge. They may be identified by the presence of bare soil exposed on the side slopes.

STREAM CHANNEL EROSION

Stream channel erosion consists of soil removal from stream banks or soil movement in the channel. Stream channel erosion and gully erosion are distinguished primarily in that channel erosion applies to the lower end of headwater tributaries and to streams that have nearly continuous flow and relatively flat gradients whereas gully erosion generally occurs in intermittent streams near the upper ends of headwater tributaries.

6.8. Stream Bank Erosion. Stream banks erode either by runoff flowing over the side of the stream bank or by scouring and undercutting below the water surface. Stream bank erosion, less serious than scour erosion, is often increased by the removal of vegetation, by overgrazing, or by tilling too near the banks. Scour erosion is influenced by the velocity and direction of flow, depth and width of the channel, and soil texture. Poor alignment and the presence of obstructions such as sandbars increase meandering, the major cause of erosion along the bank.

6.9. Sediment Movement in Channels. Sediment in streams is transported by suspension, by saltation, and by bed load movement. Although many theoretical and empirical relationships have been developed between the sediment-transporting capacity of a stream and certain parameters of flow, it is not now possible to predict sediment loads with any degree of accuracy. It is, however, possible to estimate sediment loads by resurveying reservoir bottoms and by sampling the flow of streams.[16] Variables affecting sediment movement include velocity of flow; turbulence; size distribution, diameter, cohesiveness, and specific gravity of transported materials; channel roughness; obstructions to flow; and the availability of materials for movement.

Suspension. Suspended sediment is that which remains in suspension in flowing water for a considerable period of time without contact with the stream bed. Formulas have been derived to express the concentration of sediment at any point in the

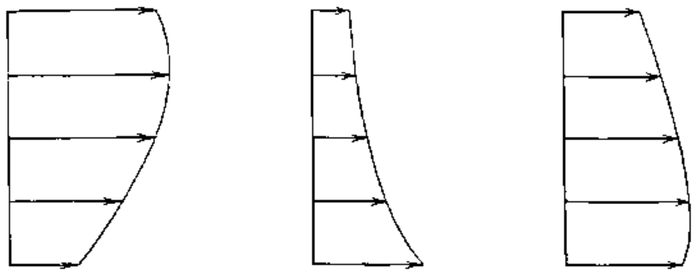

Fig. 6.3. Velocity, sediment concentration, and suspended sediment discharge distribution in a straight natural stream. (Redrawn from reference 22.)

vertical direction when the concentration at some reference level is known.

A typical relationship among velocity, sediment concentration, and sediment discharge for different depths at the center of a straight natural channel is shown in Fig. 6.3. The velocity and concentration of sediment vary with depth as shown in (*a*) and (*b*); the quantity of sediment discharge is a product of velocity and concentration, as illustrated in (*c*). Although the velocity is a minimum at the channel floor, the high concentration of sediment near the bed accounts for the maximum sediment discharge at a point just above the bottom of the channel. The distribution of fine sediment is more nearly uniform with depth than is that of coarse material.

Saltation. Sediment movement by saltation occurs where the particles skip or bounce along the stream bed. The height of bounce, expressed in mathematical form, is directly proportional to the ratio of particle density to fluid density. Particles in water rise only about 1/1000 of that in air (Fig. 6.4), or, for most practical conditions, a few particle diameters.[14] In comparison to total sediment transported, saltation is considered relatively unimportant.

Bed Load. Bed load is sediment that moves in almost continuous contact with the stream bed, being rolled or pushed along the bottom by the force of the water. Although none of the many theoretical formulas developed to express the rate of bed load movement have been entirely satisfactory, laboratory studies have shown that the critical threshold velocity (competent velocity) required to initiate movement of particles in the bottom of a stream is expressed by the empirical equation:[17]

$$v_t = \tfrac{1}{2}d^{4/9}(G - 1)^{1/2} \tag{6.1}$$

where v_t = threshold velocity in fps.
d = particle diameter in millimeters.
G = specific gravity of the particles.

This equation was developed for unigranular materials ranging in diameter from 0.35 to 5.7 millimeters and in specific gravity from 1.83 to 2.64.

Though saltation and bed load are distinct types of movement, saltation is usually included in bed load when sediment transportation is described. Although considerable data have been gathered on sediment transported by streams, most measurements are limited to the collection of suspended sediment since bed load is more difficult to determine. Bed load estimates normally range from about 10 to 50 per cent of the total sediment and vary from time to time even in the same stream. For example, on the Colorado River, bed loads varied from 12 to 50 per cent of the total load and the percentage by weight of suspended material in the water ranged from 0.13 to 2.81 per cent based on monthly averages over a period of 15 years.[10]

WIND EROSION

Wind erosion, unlike water erosion, cannot be divided into types, as wind erosion varies only by degree. For example, there may be only a slight disturbance on the surface covering a small area or there may be a huge dust storm covering several states. Wind erosion takes place at a slow, geologic rate under natural vegetation and normal soil conditions, whereas under cultivation serious erosion may result.

6.10. Factors Affecting Erosion by Wind. The major factors affecting erosion by wind are climate, soil, and vegetation. Topography appears to be relatively unimportant in the wind erosion process, but the length of the eroding surface greatly influences soil movement.

Climate. The climatic factors influencing wind erosion are precipitation; temperature; wind; and humidity, viscosity, and density of the air. The amount and distribution of rainfall and its effect on soil moisture are of primary concern. The dispersing and mellowing action of freezing and thawing may be of considerable importance, especially in the spring. Evaporation and transpiration as influenced by temperature, wind, and humidity deplete soil moisture, thus making the soil more subject to wind erosion. The principal characteristics of wind affecting erosion are velocity, direction, duration, and turbulence.

Soil. The soil factors affecting wind erosion are texture, structure, density of particles, density of soil mass, organic matter, moisture content, and surface roughness. The moisture content is especially significant because only a relatively dry soil is subject to wind erosion. Surface roughness changes with tillage practices such as listing and plowing. Surface crusts, although decreasing surface roughness, have a retarding influence on soil movement.

Vegetation. The vegetative factors are the height and density of cover, type of vegetation, and seasonal distribution. Living plant roots and tops are more effective in retarding erosion than crop residues.

6.11. Types of Soil Movement. Suspension, saltation, and surface creep in wind erosion are comparable to suspension, saltation, and bed load, respectively, in sediment movement by water. These three distinct types of movement usually occur simultaneously. Investigations have shown that the major portion of soil movement takes place near the surface at heights not greater than approximately 3 feet. Above this height the only movement is normally by suspension, whereas all three types of movement occur near the surface. In laboratory studies 55 to 72 per cent of the soil was moved by saltation, 3 to 38 per cent by suspension, and 7 to 25 per cent by surface creep.[3]

Movement of soil particles by saltation is caused by the pressure of the wind on the soil particle and its collision with other particles. The characteristic path of a soil particle moved by saltation together with final and initial velocity vectors is shown in Fig. 6.4. As the soil particle leaves the surface, it moves nearly in a vertical direction. The horizontal distance through which the particle continues to rise is about one-fifth to one-

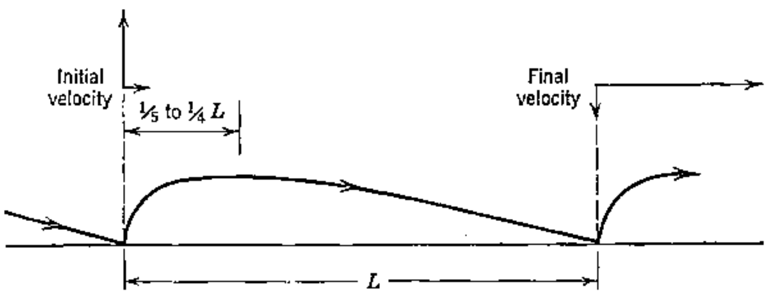

Fig. 6.4. Characteristic path of a soil particle moved in air by saltation. (Redrawn from Bagnold.[1])

fourth of the distance L. As the particle descends to the surface, it travels in a straight line with an angle of descent of about 6 to 12 degrees.[3]

6.12. Mechanics of Wind Erosion. Wind erosion takes place in three distinct phases, initiation of movement, transportation, and deposition.

Initiation of Movement. Soil movement is initiated as a result of turbulence and velocity of the wind. The fluid threshold velocity is defined as the minimum velocity required to produce soil movement by direct action of the wind, whereas the impact threshold velocity is the minimum velocity required to initiate movement from the impact of soil particles carried in saltation. Except very near the surface and at very low (less than about 2 mph[23]) velocities, the surface wind is always turbulent.

Transportation. The quantity of soil moved is influenced by the particle size, gradation of particles, wind velocity, and distance across the eroding area. Winds, being quite variable in velocity and direction, produce gusts with eddies and cross currents which lift and transport soil. The quantity of soil moved varies as the cube of the excess wind velocity over and above the constant threshold velocity, directly as the square root of the particle diameter, and increases with the gradation of the soil.

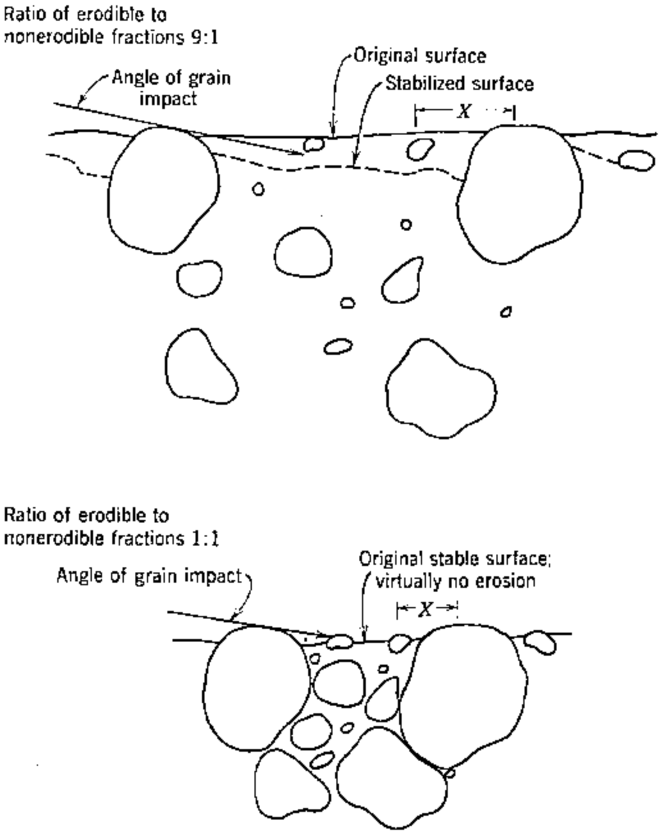

Fig. 6.5. Erosion on soils with different ratios of erodible to nonerodible fractions. (Redrawn from Chepil.[4])

A diagrammatic representation of wind erosion with two different proportions of erodible to nonerodible fractions is shown in Fig. 6.5. The distance between the centers of nonerodible particles is shown as X. In the upper drawing the erodible soil between the widely separated nonerodible particles has been removed, but no erosion is evident where the nonerodible particles were close together. For any stabilized surface the ratio of the height of the particles above the surface to the distance between them is a constant, regardless of the particle size.[4] However, the constant varies with wind velocity and with the size and specific gravity of the particles.

The rate of soil movement increases with distance from the windward edge of the field or eroded area. Fine particles drift and accumulate on the leeward side of the area or pile up in dunes. Increased rates of soil movement with distance from the windward edge of the area subject to erosion are the result of increasing amounts of erosive particles, thus causing greater abrasion and a gradual decrease in surface roughness. The rate of erosion varies for different soils (Chapter 7), some soils being as much as 10 times more erosive than others.

The atmosphere has a tremendous capacity for transporting soil, particularly those soil fractions less than 0.1 millimeter in diameter. It is estimated that the potential carrying capacity for 1 cubic mile of the atmosphere is up to 126,000 tons of soil, depending on the wind velocity.[21] As much as 200 pounds of soil per acre were deposited in Iowa in 1937 from a dust storm originating in the Texas Panhandle.

Deposition. Deposition of sediment occurs when the gravitational force is greater than the forces holding the particles in the air. This generally occurs when there is a decrease in wind velocity.

SOIL LOSSES

Soil losses vary considerably with the type of erosion. Often both wind and water erosion are present in the same area. For example, in the Texas Panhandle region on slopes of 2 per cent or less 89 per cent of erosion was caused by wind and 11 per cent by water.[9] Though several methods have been proposed for estimating erosion by water, no suitable method has been found for predicting wind erosion losses.

The importance of soil losses is indicated by the effect of topsoil depth on crop yield, as shown in Fig. 6.6. Since the data have been compiled from many areas, the average curve and the limits of variation do not represent any one condition. At greater depths of topsoil, yields are reduced at a slower rate than at the shallower depths. For example, the average yield at topsoil depths of 14, 7, and 0 inches are 100, 70, and 25 per cent, respectively. In other words, yields were reduced 45 per cent for the lower 7 inches and only 30 per cent for the top 7 inches. On some soils these yield decreases can be largely overcome by proper fertilization.

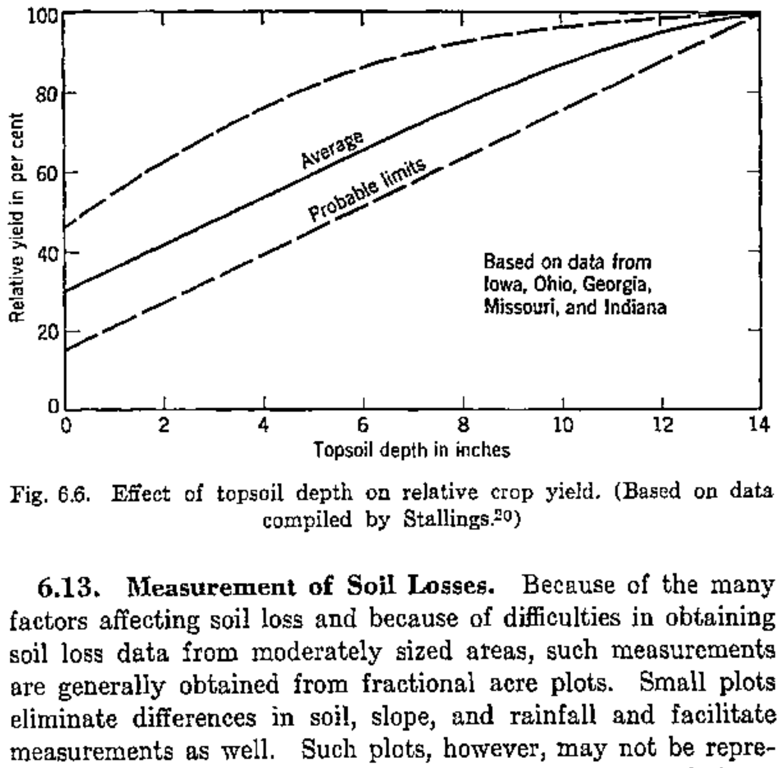

Fig. 6.6. Effect of topsoil depth on relative crop yield. (Based on data compiled by Stallings.[20])

6.13. Measurement of Soil Losses. Because of the many factors affecting soil loss and because of difficulties in obtaining soil loss data from moderately sized areas, such measurements are generally obtained from fractional acre plots. Small plots eliminate differences in soil, slope, and rainfall and facilitate measurements as well. Such plots, however, may not be representative of tillage conditions in the field, and suitable techniques for applying the data to larger watersheds have not yet been developed. The results of these plot studies do provide a basis for comparing the effects of such factors as soil, slope, slope length, rotation, and tillage practice.

Several small watershed studies have been established to furnish a means of observing soil and water losses under field conditions. These areas vary in size from less than an acre to several hundred acres. From watershed studies in Ohio the annual and monthly soil losses for the years that the area is in corn are shown in Fig. 6.7. These areas were approximately 1.5 acres in size and were located on slopes from 6 to 15 per cent. The soil losses from watersheds with recommended conservation practices are compared to the soil losses from the corresponding check watersheds representing typical Ohio prac-

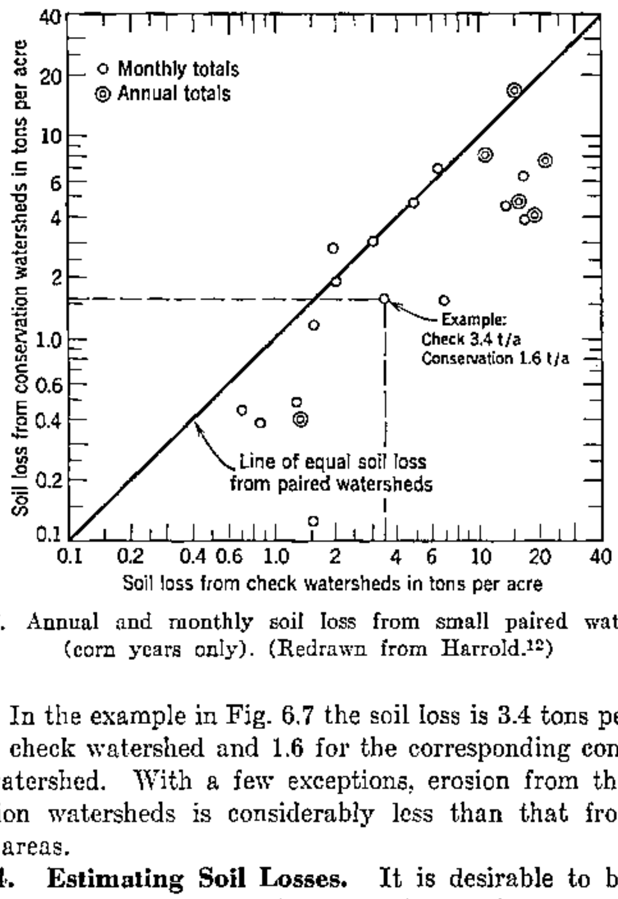

Fig. 6.7. Annual and monthly soil loss from small paired watersheds (corn years only). (Redrawn from Harrold.[12])

tices. In the example in Fig. 6.7 the soil loss is 3.4 tons per acre on the check watershed and 1.6 for the corresponding conservation watershed. With a few exceptions, erosion from the conservation watersheds is considerably less than that from the check areas.

6.14. Estimating Soil Losses. It is desirable to be able to predict soil losses for a given set of conditions in order to determine the adequacy of conservation measures in farm planning. Any method for estimating soil loss should show the relationship of each of the factors and should be easily applied to field conditions.

The annual soil loss by water erosion may be expressed as a

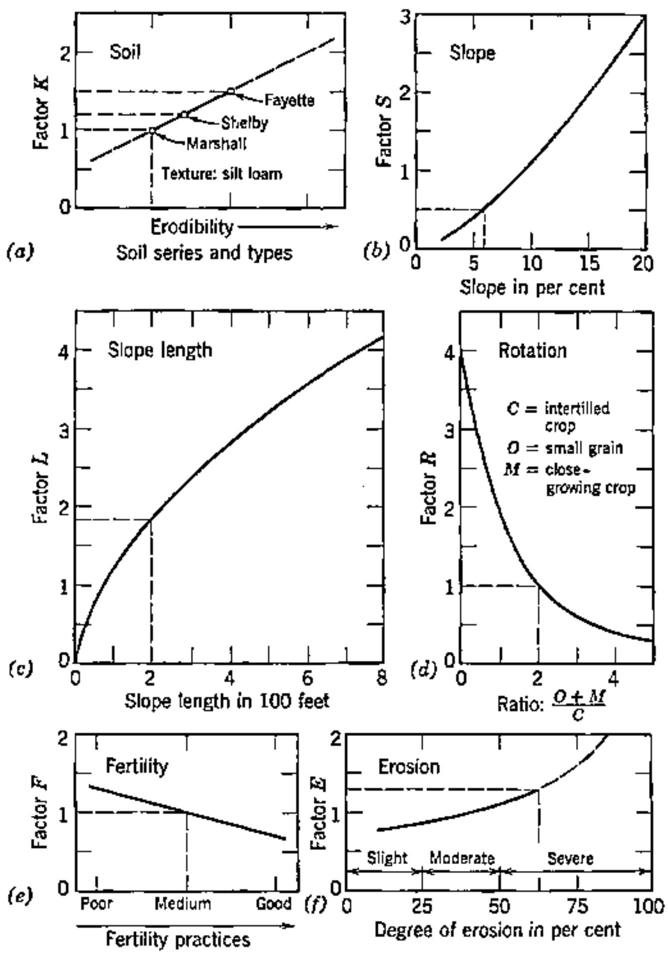

Fig. 6.8. Soil erosion factors for soil type, slope, slope length, rotation, fertility, and degree of erosion. (Based on data by Browning.[2])

functional equation in which

$$X_a = f(K,S,L,R,F,E,C,P) \tag{6.2}$$

where X_a = annual soil loss in tons per acre.
K = soil factor.
S = degree of slope factor.
L = length of slope factor.

R = rotation factor.
F = fertility factor.
E = degree of erosion factor.
C = conservation practice factor.
P = rainfall factor.

In an area where the amount and intensity of rainfall is nearly uniform, P may be omitted as a variable. By evaluating the remaining factors from plot and watershed studies, curves may

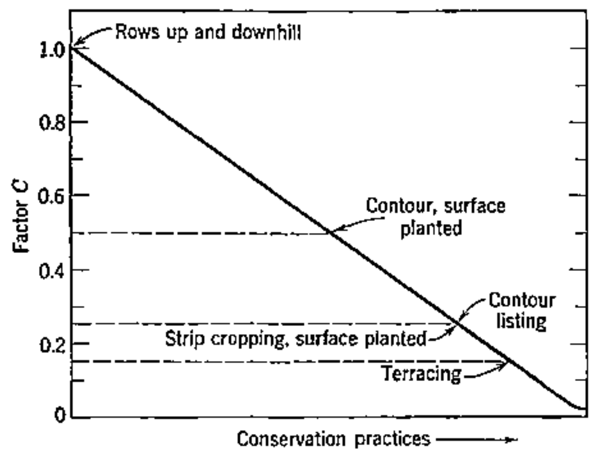

Fig. 6.9. Effect of conservation practices on soil erosion. (Plotted from data by Browning.[2])

be developed for each factor as in Figs. 6.8 and 6.9. By using the numerical value for each factor taken from such curves, the soil loss may be computed by the equation:

$$X_a = 10(KSLRFEC) \tag{6.3}$$

The data in Figs. 6.8 and 6.9 have been developed primarily from soil loss data on Fayette, Marshall, and Shelby soils at LaCrosse, Wisconsin; Clarinda, Iowa; and Bethany, Missouri, experiment stations, respectively. Other sets of curves need to be developed in order to use this method in areas having other soil, crop, and climatic conditions.

Example 6.1. Determine the soil loss for the following conditions: Marshall silt loam soil, slope 6 per cent, slope length 200 feet, rotation

COM, medium fertility practices, 63 per cent of topsoil eroded, and strip cropping (surface planted).

Solution. From Figs. 6.8 and 6.9, the soil loss factors are determined as follows: $K = 1.0$, $S = 0.5$, $L = 1.8$, $R = 1.0$, $F = 1.0$, $E = 1.3$, and $C = 0.25$. Substituting in equation 6.3,

$$X_a = 10(1.0 \times 0.5 \times 1.8 \times 1.0 \times 1.0 \times 1.3 \times 0.25)$$
$$= 2.9 \text{ tons per acre}$$

The estimated soil loss should be less than the permissible soil loss which is 5 tons per acre for Marshall soil.[6] By properly

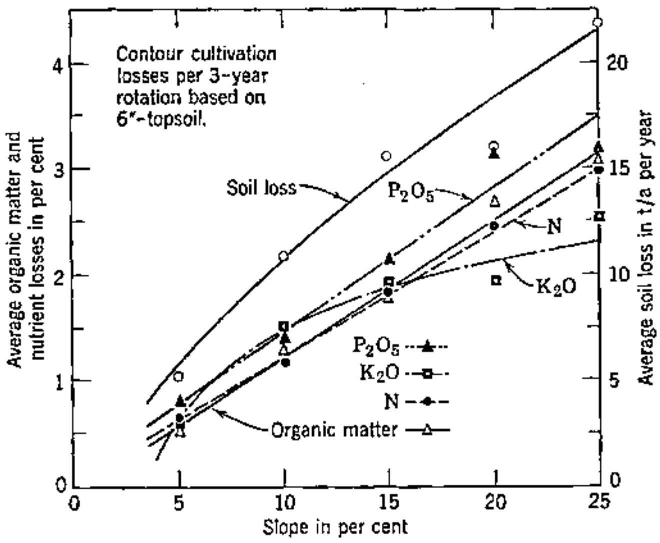

Fig. 6.10. Annual soil and plant nutrient losses from topsoil on various slopes. (Plotted from data by Moody.[18])

selecting rotations, fertility practices, and conservation practices soil losses can be reduced to the permissible value or less.

PLANT FOOD AND ORGANIC MATTER LOSSES

The loss of plant nutrients from the soil is at least as important as the loss of the soil itself. Since the erosion process causes the finer soil particles and organic matter to be removed, and since such material furnishes most of the base exchange capacity of the soil, providing storage for plant food, the removal of these smaller and lighter particles greatly decreases the fertility of the soil. Some authorities base erosion control practices on fertility losses but others prefer to use soil loss as the basis. It is well to give consideration to each because both are important in maintaining productivity.

The effect of slope on annual soil and plant nutrient losses is shown in Fig. 6.10. The loss of nutrients and soil increases as the slope increases. Plant nutrients and organic matter losses are given as a percentage of the total in the top 6 inches of soil. Except for potassium, nutrient losses varied linearly with the slope and increased with the soil loss. The data are from plots about 60 feet in length and farmed on the contour. An average of 93 per cent of the losses shown in Fig. 6.10 occurred during the years that the plots were in corn.

The effect of wind on loss of plant nutrients is likely to be many times greater during the first few years after cultivation than later when the fines have been removed. In this way wind erosion may be serious even when the total soil loss is not in itself excessive.

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PROBLEMS

6.1. If the specific gravity of moving sediment is 2.5 and the velocity of the stream along its bed is 0.5 fps, what is the maximum size of soil particle that can be moved?

6.2. Determine the soil loss in tons per acre for Shelby soil, slope 5 per cent, slope length 300 feet, *COMM* rotation, medium fertility practices, 40 per cent of topsoil eroded, and uphill and downhill farming.

6.3. What practice or combination of practices will reduce the soil loss in Problem 6.2 to a permissible value of 3 t/a?

6.4. If the soil loss for a given set of conditions is 2 t/a for a 200-foot length of slope, what soil loss could be expected for an 800-foot slope length?

CHAPTER 7

Wind Erosion Control

In the arid and semiarid regions of the United States, large areas are affected by wind erosion. The Great Plains region, an area especially subject to soil movement by wind, represents about 20 per cent of the total land area in the United States. Wind erosion not only removes soil but also damages crops, fences, buildings, and highways.

Contrary to popular opinion many humid regions are also damaged by wind erosion. The areas most subject to damage are the sandy soils along streams, lakes, and coastal plains and the organic soils. Peats and mucks comprise about 25,000,000 acres[12] located in 34 states. Although the extent of sandy areas is not available, it probably is as much or more than the acreage of peats and mucks.

The two major types of wind erosion control measures consist first of those that reduce surface wind velocities and secondly of those that affect soil characteristics. Many measures, such as permanent grass and contouring, provide both types of control. For example, vegetation not only retards surface winds but also increases soil aggregation.

CONTROLLING SURFACE WIND VELOCITY

The three principal methods of reducing surface wind velocities are vegetative measures, tillage practices, and mechanical methods. In each, the degree of protection is influenced by the height and spacing of obstructions, the breaking effect on the wind, and the resistance of the soil to movement.

Although wind is a climatic phenomenon, daily and seasonal variations in direction and velocity have been determined and are helpful in planning and carrying out control measures. In the Great Plains region in the late winter and spring months high-velocity winds, aided by a breakdown in soil structure from freezing and thawing, produce conditions conducive to severe erosion. Intensity-recurrence interval curves similar to those for rainfall can be developed for winds of different duration and for different seasons of the year. Such data have been collected for winds in Kansas.[19]

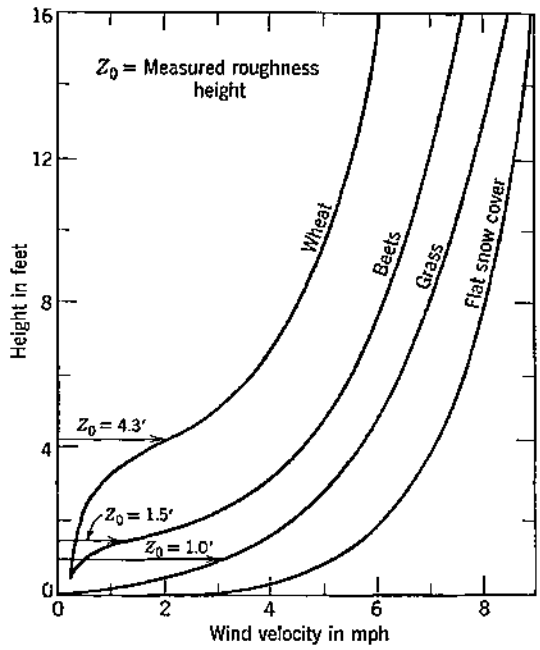

Fig. 7.1. Wind velocity distribution over different types of plant cover and soil surfaces. (Redrawn from Geigher.[11] Original from W. Paeschke.)

VEGETATIVE METHODS

Vegetation is generally the most effective means of wind erosion control. When used for this purpose, vegetation is grouped into intertilled crops, close-growing crops, and woody plants, such as shrubs and trees. Crop residues are also of considerable value in preventing soil blowing.

Vegetation has considerable effect on the magnitude of wind velocities near the surface. As an indication of this effect the wind velocity distribution with height over several types of vegetation and a flat snow surface is shown in Fig. 7.1. The velocity near the ground for beets and wheat changes only slightly at first, then increases rather rapidly, until the roughness height is reached. This roughness height is the height above which the velocity increases as an exponential function. The roughness height varies from zero for a flat snow cover to 4.3 feet for wheat. The wind distribution over flat snow cover is similar to that for smooth bare soil (not shown). These curves may vary widely, depending on the height and density of the vegetation and the roughness of the surface.

7.1. Intertilled Crops. Intertilled crops, such as corn, cotton, and vegetables, offer some protection. The extent of protection provided depends not only on the row direction but also on the width of rows, density of plants in the row, kind of crop, degree of maturity, soil, and climatic conditions. Semi-close-growing crops in the Texas Panhandle gained soil, but intertilled crops lost soil by wind erosion.[8] The best practice is to seed the crop normal to prevailing winds. In the Great Plains region the rows are then usually in an east-west direction. A good crop rotation that will maintain soil structure and conserve moisture should be followed. Crops adapted to soil and climatic conditions and providing as much protection against blowing as practical are recommended. For instance, in the Great Plains region broomcorn, cane, and sudan grass are often grown, as they are quite resistant to drought and are effective in preventing wind erosion. Stubble mulch farming and cover crops between intertilled crops in more humid regions aid in controlling blowing until the plants become established. In some dry regions emergency crops with low moisture requirements are often planted on summer fallow land before seasons of high intensity winds. In muck soils where vegetable crops are grown, miniature windbreaks consisting of rows of small grain are sometimes planted.

7.2. Close-Growing Crops. In general, close-growing crops are more effective for erosion control than intertilled crops. The effectiveness of such crops as legumes, grasses, and small grains is dependent quite largely on the stage of growth and the density of cover. Pasture or meadow tends to accumulate soil if there is a good growth of vegetation, the soil coming from neighboring cultivated fields or being deposited by sedimentation. Good management practices such as rotation grazing are important.

Stabilization of sand dunes with vegetation may be accomplished by establishing grasses and then reforesting. Vegetation used should have the ability to grow on sandy soil, the ability to grow in the open, firmness against the wind, and long life. It should also provide a dense cover during critical seasons, provide as uniform an obstruction to the wind as possible, reduce the surface wind velocity, and form an abundance of crop residue.[14]

7.3. Shrubs and Trees. The U. S. Soil Conservation Service in 1945 estimated that over 2,000,000 acres of land were in need of protection by shelterbelts and windbreaks.[17] Although a windbreak is defined as any type of barrier for protection from winds, it is more commonly associated with mechanical or vegetative barriers for buildings, gardens, orchards, and feed lots. A shelterbelt is a longer barrier than a windbreak, usually consisting of shrubs and trees, and is intended for the conservation of soil and moisture and for the protection of field crops. Approximately 120,000 miles of windbreaks and shelterbelts have been planted in the United States since the middle of the past century.[15] Not only are windbreaks and shelterbelts valuable for wind erosion control but also they save fuel, increase livestock gains, reduce evaporation, prevent firing of crops from hot winds, catch snow during the winter months, provide better fruiting in orchards, and make spraying of trees for insect control more effective as well as provide farm woodlots and wildlife refuge.

The relative wind velocity at a height of 1.3 feet above the soil surface near a windbreak is shown in Fig. 7.2. The data were obtained for a slat fence barrier 19 times longer than its height and having an average density of 50 per cent but more open in the lower than in the upper half. Similar results could be expected from a tree shelterbelt with an equivalent density. In Fig. 7.2 the wind velocity is affected for a distance of about 8 times the windbreak height on the windward side and 24 times on the leeward side. It is common practice to express the effectiveness of a shelterbelt in tree height units since the velocity distribution pattern is similar for trees of varying heights. In general, wind velocities are reduced on the windward side from 5 to 10 tree heights from the protecting shelterbelt; on the leeward side the protection varies from 10 to 30 times its height. Thus, trees about 50 feet in height would protect an area approximately one-quarter mile in width. Since the shelterbelt must filter the wind and at the same time lift it from the surface, vegetative density is important. Narrow shelterbelts which are extremely dense in the upper portion and relatively open near the ground should be avoided since they may produce blow areas on the leeward side.

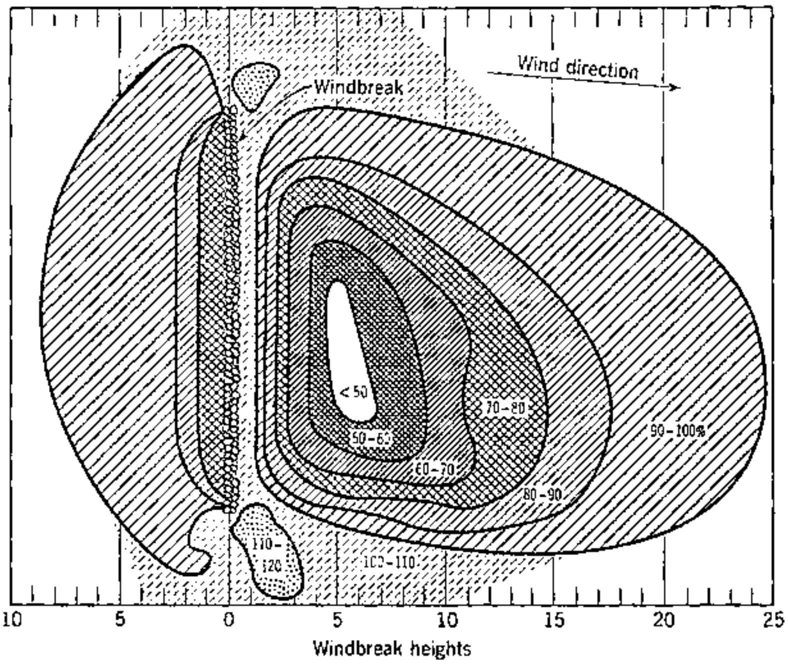

Fig. 7.2. Percentage of normal wind velocity near a windbreak having an average density of 50 per cent. (Redrawn from Bates.[1])

Since the wind velocity at the ends of the belt as given in Fig. 7.2 is as much as 20 per cent greater than velocities in the open, it is evident that long shelterbelts are more effective than short ones. An opening or break in an otherwise continuous belt results in a similar increase in velocity and will reduce the area protected. Roads through shelterbelts should therefore be avoided, and, when essential, they should cross the belt at an angle or they should be curved. In establishing the direction of shelterbelts, records of wind direction and velocity, particularly during vulnerable seasons, should be considered, and the

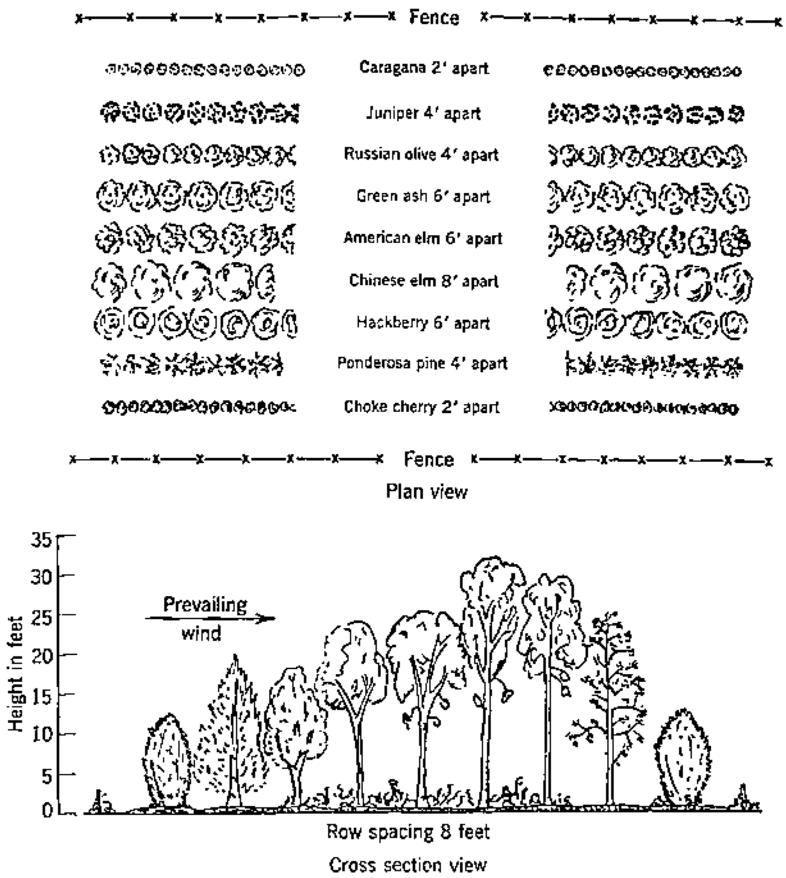

Fig. 7.3. Typical shelterbelt for the plains states. (By permission from *Soil Conservation*, by Bennett,[2] McGraw-Hill Book Co., 1939.)

barrier should be oriented as nearly as possible at right angles to the prevailing direction of winds.

A typical shelterbelt for the plains states, in Fig. 7.3, is a planting consisting of several rows of trees and shrubs of varying heights. A tight row of shrubs on the windward side is desirable, and, when combined with conifers, and low, medium, and tall deciduous trees, the shelterbelt provides a compact and rather dense barrier. Such an extensive shelterbelt may not always be required. The fence-row type consisting of only 1 or 2 rows of trees may give satisfactory results in some areas.

Where suitable, conifers are preferred over deciduous trees in the narrow belts. In the northern Great Plains a 5-row shelterbelt is the minimum width recommended since it is the narrowest planting under which forest conditions will develop.[16] Because of costs and area requirements it is not always economically feasible to use wide belts to provide all the protection required. A more common practice is to space the wide shelterbelts at one-half-mile intervals and to supplement these with intermediate belts of 1 to 3 rows of trees.

TILLAGE PRACTICES

Although all cultural methods of wind erosion control are temporary, tillage practices can be helpful. Whenever possible cultural practices should be used before blowing starts, wind erosion being much easier to prevent than to stop. In general, long periods of drought will obliterate the effects of cultural treatments.

7.4. Field and Contour Strip Cropping. Field and contour strip cropping consists of growing alternate strips of clean-cultivated and close-growing crops in the same field. Field strip cropping is laid out parallel to a field boundary or other guide line, whereas ordinary strip cropping operations are on the contour. In some of the plains states strips of fallow and grain crops are alternated. The chief advantages of strip cropping are: (1) physical protection against blowing, provided by the vegetation, (2) soil erosion limited for a distance equal to the width of strip, (3) greater conservation of moisture, particularly from snowfall, and (4) the possibility of earlier harvest. The chief disadvantages are: (1) machine problems in farming narrow strips and (2) greater number of edges to protect in case of insect infestation.

The strips should be of sufficient width to be convenient to farm, yet not so wide as to permit excessive erosion. The width of strips depends on the intensity of wind, row direction, crops, and erodibility of the soil. The relative rate of erosion for soils of different texture and length of the eroding surface is shown in Fig. 7.4. The data are for specific soil series and are not necessarily representative of all soils of the same texture. For a field width of 300 feet the rate of soil loss for fine sandy loam is over 10 times that for a loam soil. Strip cropping is effective because the length of the eroding surface is reduced to the width of the strip. For example, four strips each 300 feet in width would reduce wind erosion for a loam soil to about 15 per cent of that for a 1200-foot field before strip cropping. Because of the many variables involved, it is not practical to make general recommendations.

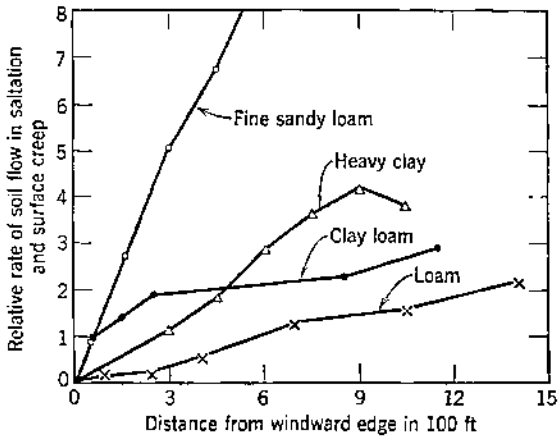

Fig. 7.4. Effect of length of eroding surface on relative rate of soil movement. (Redrawn from Chepil.[4])

7.5. Primary and Secondary Tillage. The objective of primary and secondary tillage for wind erosion control is to produce a rough, cloddy surface. In order to obtain maximum roughness the land normally should be cultivated as soon after a rain as possible. Large clods as well as a high percentage of large aggregates are desirable.

Small ridges normal to the direction of prevailing winds are effective in wind erosion control. The effect of such ridges on wind movement is shown in Fig. 7.5. Very low velocities were observed between the ridges and approximately 1 inch below the crest. In the furrow the direction of movement was opposite to that of the wind. The decrease in wind velocity and change in direction between the ridges causes soil deposition. For a height of about 6 inches above the ridges there is an area of considerable turbulence.

Tillage may be quite effective as an emergency control measure. Soil blowing usually starts in a small area where the soil is less stable or is more exposed than in other parts of the field. Where the entire field starts to blow, the surface should be put in a rough and cloddy condition as soon as practicable. This

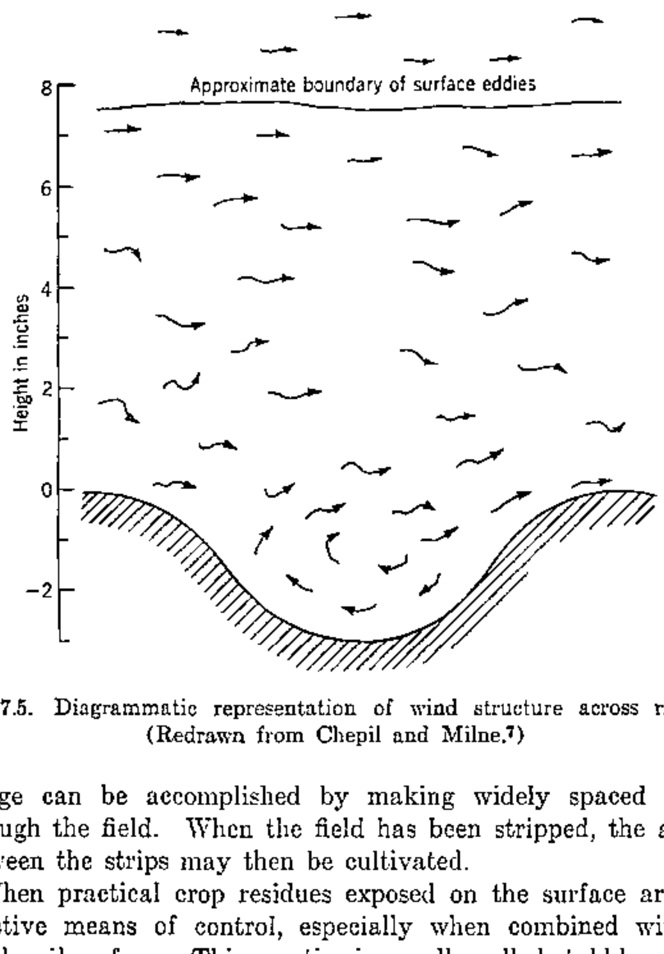

Fig. 7.5. Diagrammatic representation of wind structure across ridges. (Redrawn from Chepil and Milne.[7])

tillage can be accomplished by making widely spaced trips through the field. When the field has been stripped, the areas between the strips may then be cultivated.

When practical crop residues exposed on the surface are an effective means of control, especially when combined with a rough soil surface. This practice is usually called stubble mulch tillage. The effectiveness of various soil treatments on wind velocity and soil erosion are shown in Fig. 7.6. These experiments were carried out in Canada, with a wind tunnel for a smooth bare soil, for 0.5 ton of wheat straw of which one-half was worked into the soil, for ridges 1.25 inches high and 7 inches wide normal to the wind, and for ridges plus a straw mulch. All surfaces were exposed to the same wind velocity as measured at a height of 12 inches. The reduction in wind velocity was greatest for the ridges plus straw. The effect of the straw mulch

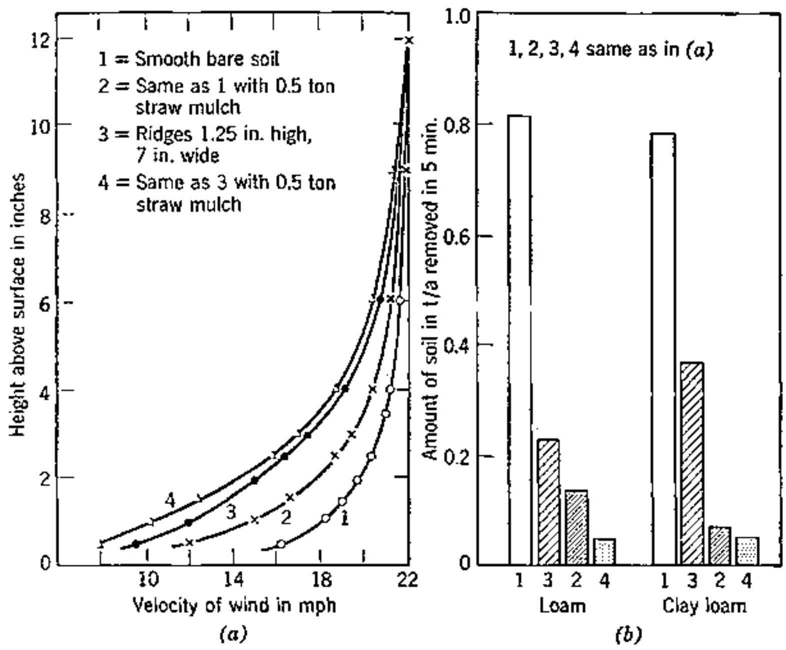

Fig. 7.6. Influence of surface treatment on wind velocity (*a*) and soil erosion (*b*). (Redrawn from Chepil.[5])

in reducing soil erosion was greater than for ridges alone. The combined effect of straw and ridges was appreciably greater than the effect of either straw or ridges. As compared to bare soil, 0.5 ton per acre of wheat straw reduced the amount of erosion by 83 and 88 per cent for loam and clay loam soils, respectively.

Crop residues act in two ways: they reduce wind velocity and trap eroding soil. Short stubble is less effective than long stubble. A mixture of straw and stubble on the surface provides more protection against erosion than equivalent amounts of straw or stubble alone.[6] The higher the wind velocity the greater the quantity of crop residue required.

The effect of various quantities of crop residue on soil loss for the soil aggregate fraction less than 0.42 millimeter in diameter is shown in Fig. 7.7. The data show that the soil loss varies inversely as the 0.8 power of the weight of plant residue.

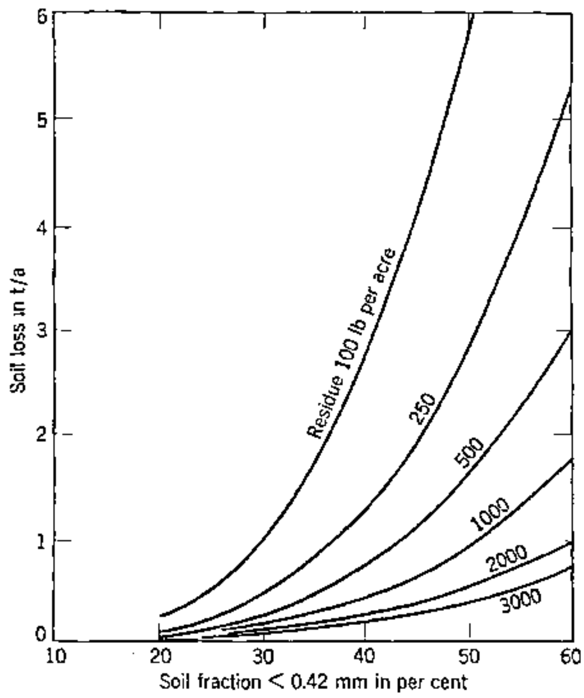

Fig. 7.7. Effect of crop residue on soil loss for soils with varying amounts of erodible fractions. (Redrawn from Englehorn and others.[10])

With 60 per cent of the soil fractions less than 0.42 millimeter, the soil loss may be reduced from 5 to 1 ton per acre by increasing the residue from 250 to 2000 pounds per acre, respectively.

MECHANICAL METHODS

Mechanical barriers such as windbreaks are of little importance for field crops, but they are frequently employed for the protection of farmsteads and small areas. Some of the mechanical methods of control include slat or brush fences, board walls, and vertical burlap or paper strips, as well as surface protection, such as brush matting, rock, and gravel. These barriers may be classed as semipermeable or impermeable. Semipermeable windbreaks are usually more effective than impermeable structures because of diffusion and eddying effects on the leeward side of the barrier. A slat snow fence is a good example of this action. Slat fences, picket fences, and vertical burlap or paper strips are sometimes used for the protection of vegetable crops in organic soils. Brush matting, debris, rock, and gravel may be suitable in stabilizing sand dune areas.

Terraces have some effect on wind erosion. In the Texas Panhandle, terraces lost less soil than the interterraced area and in some instances gained soil. Most of the soil that was lost from the interterraced area was collected in the terraces.[8] Where vegetation was growing on the ridge and in the channel, terraces were still more effective.

CONTROLLING SOIL FACTORS

The principal soil factors influencing wind erosion control include the conservation of moisture to improve vegetative growth and conditioning of the surface soil to improve aggregation.

7.6. Conserving Soil Moisture. The conservation of moisture, particularly in arid and semiarid regions, is of utmost importance for wind erosion control as well as for crop production. The means of conserving moisture fall into three categories: increasing infiltration, reducing evaporation, and preventing unnecessary plant growth. In practice these can be accomplished by such practices as level terracing, contouring, mulching, and selecting suitable crops.

The greater the amount of mulch on the surface the greater the quantity of moisture conserved. The effect of crop residue and tillage practices on the conservation of moisture is shown in Fig. 7.8. These data show the percentage of rainfall conserved from April to September. Where the straw was disked or plowed in, some of the residue was left on the surface, and, where the soil was plowed and disked, no straw was present. Although there was no runoff for the basin-listed area, the quantity of moisture conserved was about half of that for the treatment with 2 tons of straw on the surface, probably owing to greater evaporation. For the same reason disking and plowing were less effective than any of the mulch treatments.

The time of seedbed preparation in some regions has considerable effect on the conservation of moisture. This effect is shown in Fig. 7.9 for winter wheat at Hays, Kansas. Late plow-

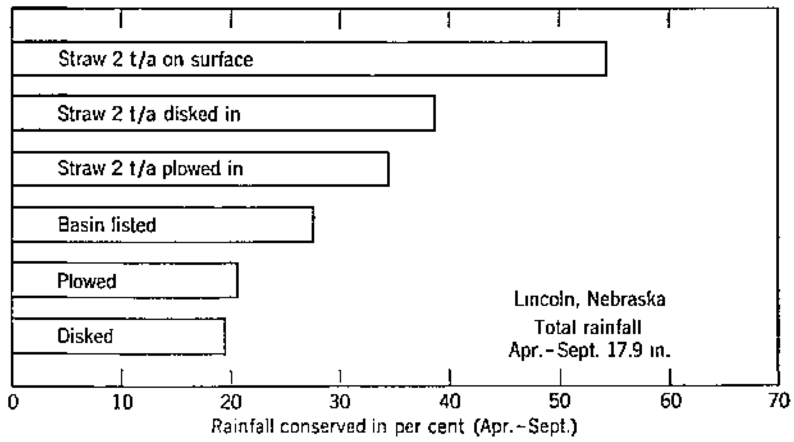

Fig. 7.8. Effect of straw and tillage practices on storage of soil moisture. (From data by Duley and Russel.[9])

ing (September-October) resulted in a decrease in available moisture as compared to early plowing (July) or summer fallow. Wheat yields for late and early plowing were 37 and 65 per cent, respectively, of that for summer fallow.

Organic soils do not blow appreciably if the soil is moist. Where the subsoil is wet, rolling the soil with a heavy roller will increase capillary movement and moisten the surface layer. Controlled drainage where the water level is maintained at a specified depth may also reduce blowing. In irrigated areas the surface is often wet down by overhead sprinkling at critical periods.

Terracing is a good moisture conservation practice where level terraces are suitable and where the slopes are gentle enough so that the water can be spread over a relatively large area. Such practices as contouring, strip cropping, and mulching are effective in increasing the total infiltration and thereby the total soil moisture available to crops. Field strip cropping generally does not conserve as much moisture as contour strip cropping, but it is somewhat more effective in reducing surface wind velocities.

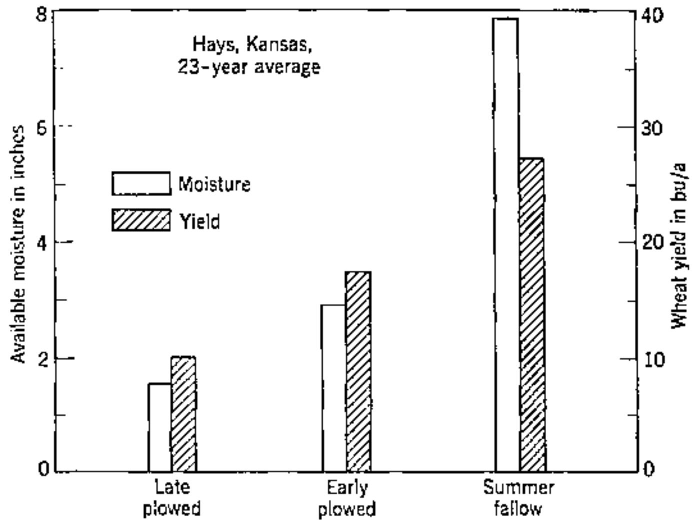

Fig. 7.9. Effect of time of seedbed preparation on available moisture at seeding time and on wheat yields. (From data by Hallsted and Mathews.[13])

7.7. Conditioning Topsoil. Since wind erosion is influenced to a large extent by the size and apparent density of aggregates, an effective method of conditioning the soil against wind erosion is by adopting practices that produce nonerosive aggregates or large clods. During the periods of the year when the soil is bare or has a limited amount of crop residue, control of erosion may depend on the degree and stability of soil aggregation.

Tillage may or may not be beneficial to soil structure, depending on the moisture content of the soil, type of tillage, and number of operations. For optimum resistance to wind erosion in semiarid regions it is desirable to perform primary tillage as soon as practical after a rain. The number of operations should be a minimum because tillage has a tendency to reduce soil aggregate size and to pulverize the soil. Secondary tillage for seedbed preparation should be delayed as long as practical.

Good crop and soil management practices are necessary to maintain good soil structure. In the Great Plains a typical recommended rotation consists of an intertilled crop (sorghum), spring-drilled crop (oats), fallow, and winter wheat. The organic matter in the soil should be held to a high level and lime and fertilizers applied where necessary.

Soil structure is affected by the climatic influences of the season, rainfall, and temperature. Freezing and thawing generally have a beneficial effect in improving soil structure where sufficient moisture is present. However, in dry regions the soil is more susceptible to erosion because of rapid breakdown of the clods into smaller aggregates.

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CHAPTER 8

Contouring, Strip Cropping, and Tillage

One of the basic engineering practices in conservation farming is the adjustment of tillage and crop management from uphill and downhill to contour operations. This adjustment in direction of implement travel provides a multiple attack on the forces of erosion through the effects of contouring, strip cropping, and specialized conservation tillage practices.

CONTOURING AND STRIP CROPPING

FIELD LAYOUT

8.1. Contouring. When plow furrows, planter furrows, and cultivation furrows run uphill and downhill, they form natural channels in which runoff accumulates. As the slope of these furrows increases, the velocity of the water movement increases with resulting destructive erosion.

In contouring, tillage operations are carried out as nearly as practical on the contour (see Art. 8.6). A guide line is laid out for each plow land, and the backfurrows or deadfurrows are plowed on these lines. On small fields of uniform slope, one guide line may be sufficient; on larger, more irregular fields, several lines may be required to assure that all tillage remains within the usual limits of 1 to 2 feet of fall per 100 feet.[35,36] Methods of plowing out point row areas are shown in Fig. 8.1.

On gently sloping land, contouring will reduce the velocity of overland flow. If ridge or lister cultivation is practiced, the storage capacity of the furrows is materially increased, permitting storage of large volumes of water. When contouring is used alone on steeper slopes or under conditions of high rainfall intensity and soil erodibility, there is an increased hazard of gullying because row breaks may release the stored water. Break-overs cause cumulative damage as the volume of water increases with each succeeding row.

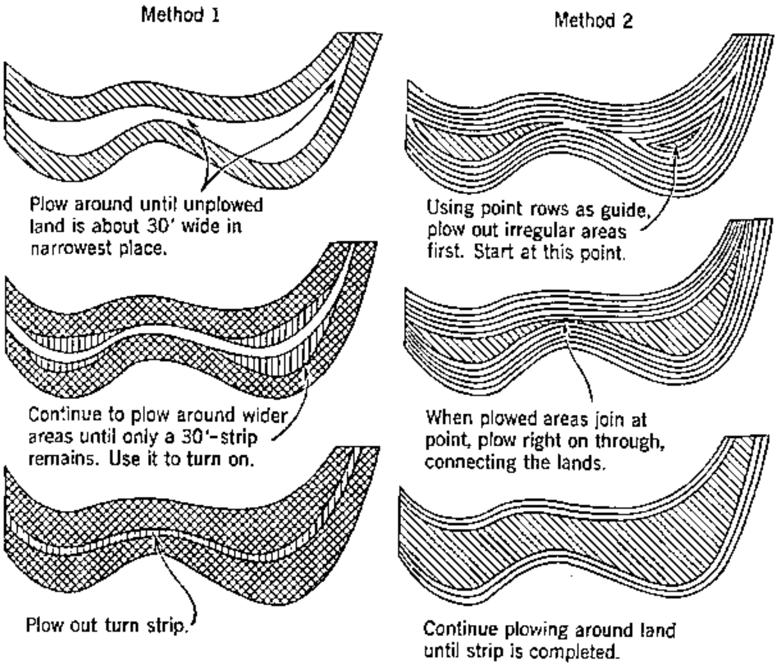


Fig. 8.1. Methods of plowing out point row areas. (Redrawn from Hay.[16])

The effectiveness of contouring is also impaired by changes in infiltration capacity of the soil owing to surface sealing. Depression storage is reduced after the tillage operations cease and settlement takes place. Studies in Ohio showed that contour cultivation together with good sod waterways reduced watershed runoff 75 to 80 per cent at the beginning of the season. This reduction dropped to as low as 20 per cent at the end of the year, leaving an annual average reduction in runoff, due to contouring, of 66 per cent.[13]

8.2. Strip Cropping. Strip cropping consists of a series of alternate strips of various types of crops laid out so that all tillage and crop management practices are performed across the slope or on the contour. Strip cropping is not a single practice; it is a combination of several good farming practices such as crop rotations, contour cultivation, proper tillage operations, stubble mulching, and cover cropping.[34]

The three general types of strip cropping are:[17,34] (1) Contour strip cropping consists of layout and tillage held closely to the exact contour and with the crops following a definite rotational sequence. (2) Field strip cropping consists of strips of a uniform width placed across the general slope; when used with adequate

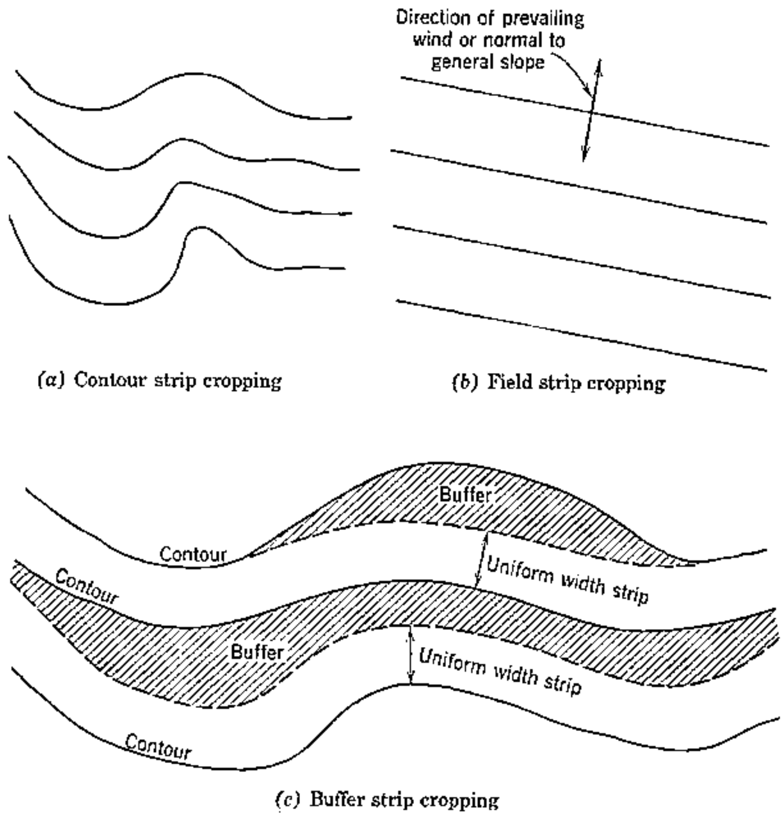
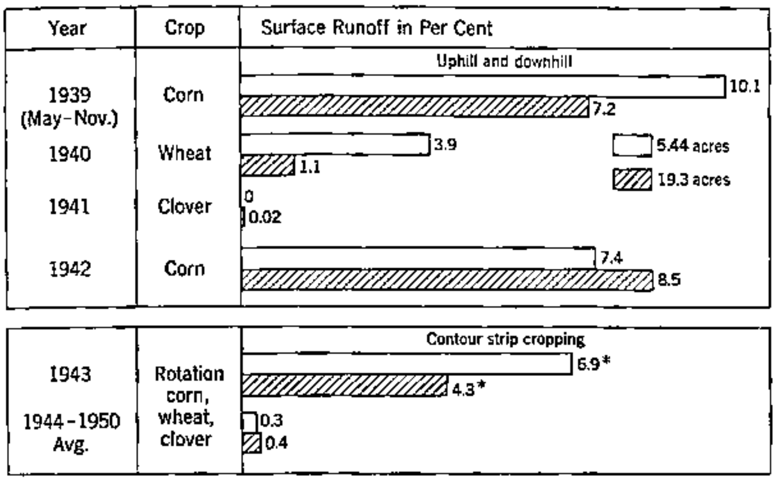


Fig. 8.2. Three types of strip cropping: (a) contour, (b) field, and (c) buffer.

grassed waterways the strips may be placed where the topography is too irregular to make contour strip cropping practical. Field strip cropping for wind erosion control consists of parallel strips placed crosswise to the direction of the prevailing wind (Chapter 7). (3) Buffer strip cropping consists of strips of some grass or legume crop laid out between contour strips of crops in the regular rotations; they may be even or irregular in width; they may be placed on critical slope areas of the field. Their ma purpose is to give more protection from erosion. The type usea depends on cropping systems, topography, and type of erosion hazards. These three types of layouts are shown diagrammatically in Fig. 8.2.



* Strips not entirely established.

Fig. 8.3. Per cent surface runoff from two agricultural watersheds at Blacksburg, Virginia. (1939–1942, uphill and downhill cultivation; 1943–1950, contour strip cropped.) (Redrawn from Edminster and Lillard.[11])

Rotations that provide strips of close-growing perennial grasses and legumes alternating with grain and intertilled crops are the most effective. Numerous types of rotations give maximum effective erosion control.[17,29,34] All natural waterways should be left as grassed waterways.

8.3. Effect of Contour Strip Cropping on Soil and Water Loss. When contour strip cropping is combined with contour tillage or terracing, it effectively divides the length of the slope, checks the velocity of runoff, filters out soil from the runoff water and facilitates absorption of rain.[17] Figure 8.3 shows the effect of contour strip cropping on water loss from two gaged watersheds.[11] Similar reductions in water loss and subsequent total soil loss from the field have been measured at many conservation experiment stations.[32,34]

8.4. Strip Cropping Layout. The three general methods of laying out strip cropping are: (1) both edges of the strips on the contour; (2) one or more strips of uniform width laid out from a key or base contour line; and (3) alternate uniform width and variable width correction or buffer strips.[34] Methods of layout vary with topography and with each individual's preference. The initial guide lines may be laid out from top, bottom, or middle of the slope. A maximum deviation of 1 to 2 feet above or below the true contour in any 100-foot interval is permitted.[35,36]

Strip widths vary with local conditions. When strip cropping is used with terraces, the width usually conforms to the terrace interval. In dry farming or other critical wind erosion areas, strip width may vary according to soil drifting conditions.[34] As strips should be of a width that is convenient to farm, they should approach some multiple of implement width. In the East and the South, a corn shock row 56 feet wide is usually considered the minimum. In the Middle West, limits of 50 to 130 feet are general.[35]

In the Southeast, the equation

$$W = 98 - 7(S - 10) \tag{8.1}$$

where W = width of strip in feet and S = slope of land in per cent is sometimes used to arrive at a safe design width for slopes ranging between 3 and 18 per cent.[36]

Maximum erosion protection is achieved when the point rows of a cultivated crop are thrown to the middle of the strip by planting parallel to the edges of the strips. Point row areas may also be left in permanent meadow.

In some areas insect damage, due to the extended exposed crop borders, has proved to be a serious disadvantage of strip cropping. Establishment of rotations that give a minimum of protective harbor to insects, spray programs, and other approved insect control measures reduce this problem. Crop damage from chinch bugs can often be avoided by growing corn or small grain with meadow in alternating strips in separate fields, thus eliminating the bordering corn and small grains.[34]

Grass strips and large meadow outlets can be grazed through use of portable electric fencing. Grazing may also be facilitated by establishing interfield rotations that provide that each yea one field will be entirely in meadow owing to the first and second years' meadows being adjoining.[34]

8.5. Contour Furrows on Pasture and Range Land. *Humid Areas.* Fertilization, clipping, chemical treatment for removing noxious weeds, reseeding and mulching thin areas, and, most important of all, controlled grazing constitute the basic principles of good conservation management on range and pasture land. In some areas contour furrows, ranging from single plow furrows to high bedded ridges, have been tried as a means of storing excess runoff. Research studies in the humid regions have failed to show any significant results from their use,[18,30] except when used as diversions in establishing cover on extremely eroded areas.

Arid and Semiarid Regions. Contour furrows on semiarid western pasture and range land have presented variable results. General observations showed that the furrows frequently had a high weed count for the first year or so. Desirable native grasses usually crowded out the weeds. In some studies, it was found that more desirable and higher-quality grasses developed adjacent to the furrow areas. Ridges placed at wide intervals because of poor moisture distribution gave less response than closely spaced ridges. High ridges tend to remain dry, thus reducing growth; similarly, deep, large-capacity channels frequently drown out desirable species.[33]

In some of the flatter, more arid range land the practice of pitting has been found effective. An eccentric one-way disk is used to make a series of pits about 3 inches deep, 18–20 inches long, and 16 inches apart. The basic principle of pitting is that the thinning effect on the plant cover provides more moisture and plant food for the remaining plants in addition to holding back runoff. At Archer, Wyoming, range land pitted in 1938 carried 44 per cent more sheep in 1944 than untreated range land. Over a 5-year period, the pitted range provided an average of 15 per cent more grazing than the untreated areas, and it also carried a greater per cent of grass cover, by weight, at the end of the grazing season.[33]

The selection and use of contour furrowing treatments must, because of soil, climate, and grass conditions, be determined on the basis of local conditions and experiences.

8.6. Establishing Contour Lines. Contouring operations must be laid out accurately if they are to be effective. Contour guide rows, terraces, and contour strip boundaries establish the pattern and accuracy of all subsequent contouring operations.

Hand levels, when carefully used and checked, are satisfactory for establishing contour guide lines and strip cropping. In areas with moderate to smooth topography, tractor-mounted contouring devices may also be used if all operating instructions are carefully followed. However, neither of these devices should be used for layout of terraces or diversions.

8.7. Effect of Contour Operations on Power Requirements. In developing the contour system of farming, the question of its effect upon power requirements, fuel consumption, time of operation per unit area, and over-all operating efficiency should be considered.

Working with various tillage and cultivating equipment, Barger[3] found that, based on 52 separate tests, with 3 tractors and 6 tillage implements on 10 plots, the time and fuel savings averaged 12.8 and 9.4 per cent, respectively, in favor of contour operation. In these studies, the average uphill and downhill draft was essentially equal to the contour draft for any given speed. However, because of the higher average speed of travel, the power requirement was greater on the contour plots than on the uphill and downhill plots. Where the uphill travel was one gear less than the downhill and contour travel, the average horsepower of the uphill and downhill travel is 19.5 per cent less than that on the contour. Where there was no gear change for uphill travel, thus providing a speed essentially equal to the downhill and contour travel, the horsepower requirements were practically identical. When the power requirements for all tests were summarized, they were about equal for both practices.[3]

Up-and-down grade operations contribute to other inefficiencies such as wheel slippage, stops for gear changes, and tractor and implement wear due to starts and stops. Operation of powered implements such as combines and balers is simplified when implement speed is uniform.

TILLAGE PRACTICES

Tillage in its simplest definition is the mechanical manipulation of the soil to provide soil conditions suited to the growth of crops, the control of weeds, and for the maintenance of infiltration capacity and aeration. Indiscriminate tillage, tillage without thought of topographic, soil, climatic, and crop conditions, will lead to soil deterioration through erosion and loss of structure. Well-planned tillage can serve as a means of preventing erosion, of improving structure, and of improving moisture relations.

EFFECT OF TILLAGE UPON THE SOIL

Tillage may consist of several types of soil manipulation, such as plowing, disking, harrowing, and cultivating. Each of these operations has definite mechanical functions to perform. Nichols[21] lists the functions of the plow as cutting loose, granulating, and inverting the furrow slice and turning under the residues. This granulation or crumbling of the furrow slice has been considered the first essential step in preparing a satisfactory seedbed.[27] On this basis, the primary tillage or plowing should break up into granules, of a size that will make a good seedbed, the soil structure that was formed by the preceding root systems and residues. The dynamics of this process have been shown by Nichols[21] in his studies on plow design. If, owing to soil conditions, moisture conditions, or factors involving the plow design, the initial plowing does not produce this granulation, then soil structure will deteriorate as the result of the increased surface preparation that must follow.[4] Careful selection of proper conditions for tillage is important, for the disintegration of soil structure can be brought about by poorly timed cultivation operations.[2,24]

8.8. Effect of Tillage Practices on Erosion. The effect of tillage upon erosion is a function of its effect upon such factors as aggregation, surface sealing, infiltration, and resistance to wind movement. Where intensive or excessive tillage has destroyed structure, increase in the erosion hazard results.

Tillage may also contribute heavily to erosion through its mechanical movement of the soil during the tillage process.

A slope of 9.8 per cent, when plowed on the contour throwing the furrow uphill, gave 9.7 per cent less soil movement uphill than soil movement downhill due to throwing the furrows downhill; however, the width of cut in the first instance was 4 per cent less.[20] Soil movement downhill during all harrowing and cultivating operations increased as the slope increased. It was concluded that turning furrows uphill during contour plowing was the most satisfactory means of compensating for the downhill movement of the soil during cultivation, harrowing, and the normal seasonal erosion processes.

TILLAGE FOR MULCH RESIDUES

Reductions in water, soil, and plant food losses as the result of contouring or strip cropping reflect only that which does not actually leave the field. Excess soil and water movement is still found within the cultivated strips. The soil from this year's cultivated strip is deposited at the top of the close-growing strip below it. Next year when this second strip is cultivated, this and other soil will move on down the hill another 60 to 100 feet. Similarly, there is a serious movement of soils from the terrace intervals to the terrace channels. On some topography, contouring, strip cropping, and terracing may be impractical, thus necessitating alternative conservation measures.

8.9. Erosion Losses under Clean-Cultivated Contoured Areas. Contoured control plots in Virginia having a slope length comparable to the average width of a cultivated strip produced the large soil and plant food losses that were shown in Fig. 6.10.[11] Similar studies in Georgia, based on terrace intervals rather than on strip crop intervals, are shown in Fig. 8.4.[32] On a slope of 5 per cent with summer cover crop residues available the average soil loss dropped below 5 tons per acre. With stubble mulch, the soil loss was nearly zero.

8.10. Principles of Mulch Tillage. Mulches of various materials, ranging from the pebble mulches of the Chinese through leaves, straw, hay, sawdust, and other organic materials to paper and even aluminum foil have been used to a limited extent in orchards and certain specialized truck or flower crops. Their use was based upon three major benefits: (1) the mulch breaks the fall of the rain drops, thus expending their energy and reducing or eliminating their dispersing action on the structure of the soil; (2) the many obstacles presented by leaves, stems, and roots within the mulch impede surface flow, thus effectively controlling sheet erosion; and (3) the mulch promotes infiltration through maintenance of an open soil structure as shown in Fig. 8.5.[7,10,11]

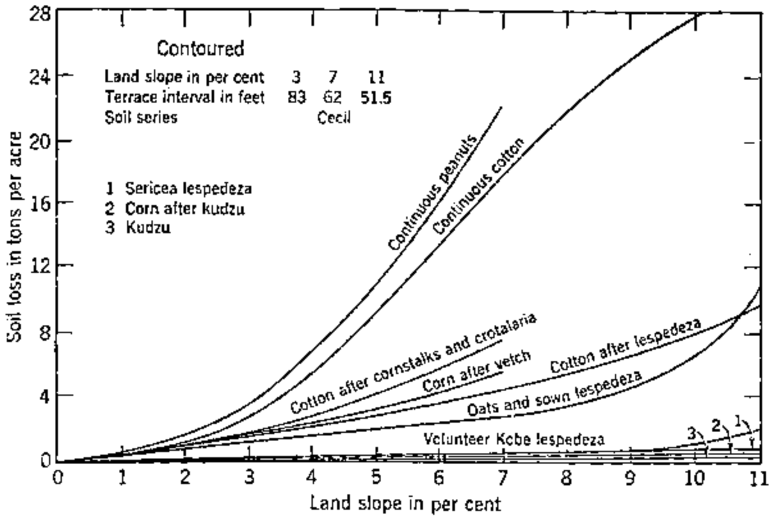


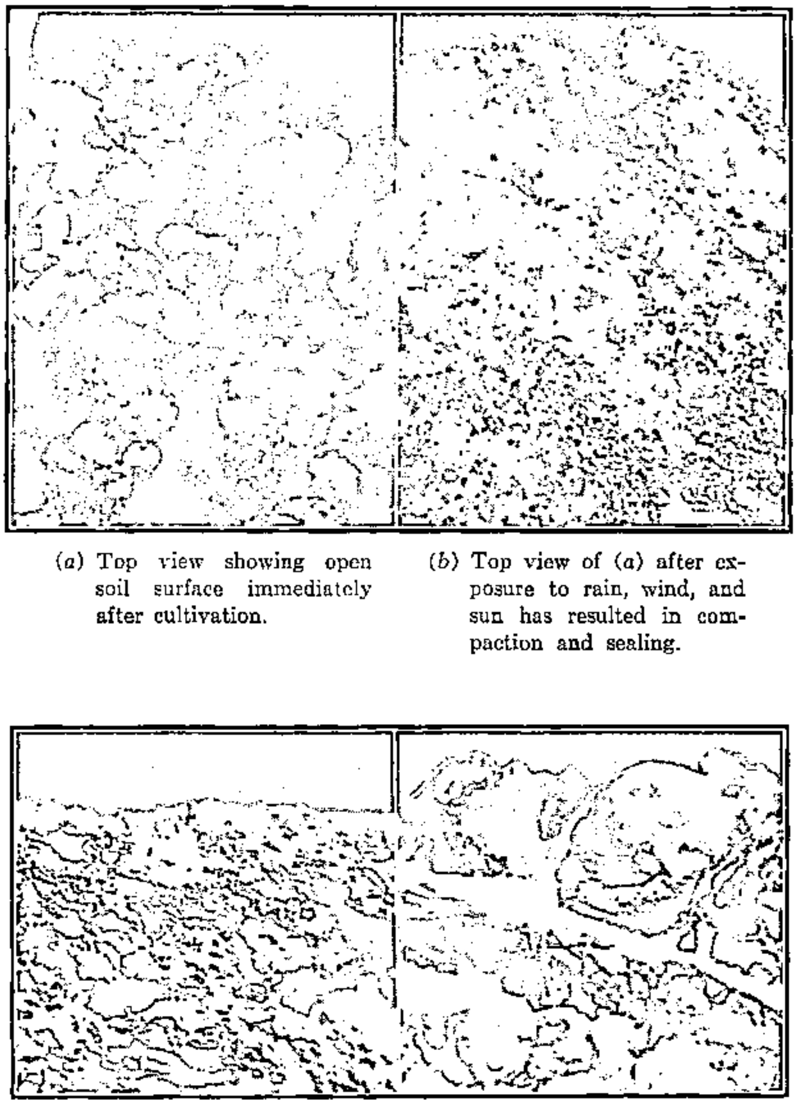
Fig. 8.4. Average annual soil losses from selected cropping practices on terrace interval slope lengths. (Southern Piedmont Conservation Experiment Station.[32])

It is self-evident that the hand application of artificial mulches such as listed above would be physically impossible on a large field scale. However, much progress has been made in the development of various tillage and crop residue management practices that will stimulate, on a large scale, the effectiveness of these artificial mulches.

In modern usage, mulch tillage or stubble mulching is a crop and soil management practice that utilizes the residual mulches of the preceding crop by leaving a large percentage of this vegetal residue on or near the surface of the ground as a protective mulch.[11] This surface layer of mulch provides for the same energy dissipation, surface runoff control, and infiltration protection that artificial mulches provide.

8.11. Subsurface Tillage. In arid and semiarid regions and in areas where the preceding or mulch crop consists of annuals such as grain straw and stubble, lespedeza, and sweet clover, seedbed preparation by means of subsurface tillage implements has generally been found to be satisfactory.[9,25]

These implements are of three principal types: (1) tillers with V-shaped sweeps, (2) straight-blade tillers, and (3) rod-weeder



(*a*) Top view showing open soil surface immediately after cultivation.

(*b*) Top view of (*a*) after exposure to rain, wind, and sun has resulted in compaction and sealing.

(*c*) Sectional view of (*b*), showing compacted stratified crusts.

(*d*) Sectional view of open porous soil protected from crusting by organic mulch.

Fig. 8.5. Photomicrographs showing the effect of mulch in maintaining an open soil structure.[10]

tillers.[2,9] Numerous studies have been made to determine the most satisfactory design for these types of implements to assure proper suction together with self-cleaning and adequate soil pulverization.[1,2,5,9,25] Each of these devices loosens and pulverizes the soil and cuts off the roots of weeds and other plants but does not invert the mulch crop. When tillage is performed at or below optimum moisture for plowing, maximum shattering and pulverization will be achieved. Better weed kill is also obtained under dry conditions. The depth of tillage depends on the type of crop to follow; i.e., corn, 5 to 6 inches, wheat, 4 to 5 inches, weeds, 2 to 3 inches.[9]

8.12. Surface Tillage. In extremely heavy mulches, particularly when they are perennials or when soil moisture is relatively high, it is sometimes necessary to partially invert or cut up the crop residue. This size reduction of residue is frequently done with disk plows, heavy-duty or bush and bog disks, one-way or vertical disk plows. Each of these implements tends to incorporate a portion of the mulch in the top 2 to 3 inches of the profile. While partial inversion or cutting the crop residue is moderately effective in controlling wind erosion and in maintaining the infiltration capacity of the soil, it is of less value in providing full protection to the immediate surface of the soil.

8.13. Mulch-Balk or Slit-Planting. In warm humid areas the rapid decomposition of organic residues makes it important to retain these sources of mulch in a living condition as late as possible into the growing season. The mulch-balk or slit method of mulch tillage was developed to meet this problem. It is particularly adapted to those areas utilizing winter and summer cover crops as the source of residue.[22,23]

Two to four weeks prior to seeding time an area 12 to 20 inches wide is thoroughly tilled by plowing, middle busting, or disk tilling to form a conventional seedbed for each row. At planting time or at first cultivation the remaining living mulch material in the row middles is cut loose and killed or retarded through the use of sweeps or conventional cultivation equipment. The principles of the balk or slit system are shown in Fig. 8.6.

8.14. Double-Cut Plow Method. In the early studies of mulch tillage under the humid conditions of the East and the South, mulch tillage equipment of the subsurface and surface type showed four major problems in their application: (1) subsurface tillage alone did not kill the heavy perennial sods, and crowns would re-root, causing excessive competition for moisture and plant food; (2) the organic residues did not decompose as rapidly on the surface as when plowed under, thus creating a plant food deficiency in the early growth period followed by

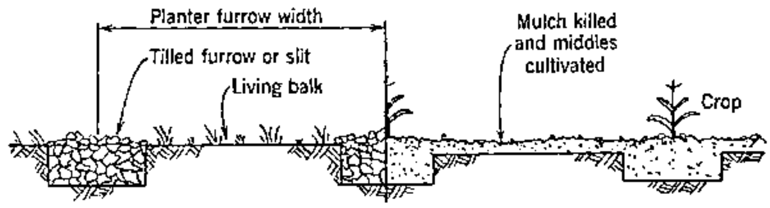


Fig. 8.6. The mulch-balk or slit system of mulch tillage before and after cultivation.

excessive release at the end of the season; (3) scarified, readily germinated weed seeds left on the surface by mulching caused excessive weed growth and competition; and (4) the heavy mulches from the perennial crops created a loose friable seedbed that contributed to low germination rates and poor stands.[11,19]

Studies in Virginia and other states developed the double-cut plow system of mulch tillage. The double-cut plow system utilizes a standard moldboard plow equipped with an additional beam or standard that carries a smaller moldboardless share behind and below the regular bottom. The plow is adjusted to completely invert the top 3-inch sod layer with the upper base and simultaneously subtill an additional 4 inches with the lower base, giving a total tillage depth of 7 inches. This operation is performed early in the spring. After 30 to 40 days of exposure to the sun and the drying action of the wind, the viable perennial root crowns are killed. The final seedbed is then prepared by using a spring tooth harrow or similar implement to break up these thin sod ribbons and to smooth and level the surface. Approximately 50 per cent of the mulch remains on the surface while the remainder is incorporated in the top 3 inches.[11,19]

This tillage procedure, together with deep placement of fertilizer, to put the plant food below the mulch and its bacteria, the use of disk planter furrow openers, and modified cultivation procedures, is bringing stubble mulching closer to a reality for the humid areas.

8.15. Supplementary Mulch Equipment. Mulch tillage has developed a need for several types of specialized supplementary equipment. Where subsurface tillers have been used, grain germination has been improved by smoothing and packing the seedbed with treaders.[2,9,25]

Deep furrow drills, sometimes equipped with seed press wheels, have provided a means of placing the seed in firm moist soil and away from contact with loose clumps of straw.[9,25] In planting row crops, superior results have been achieved with single or double-disk furrow openers that move the mulch to the side of the planter furrow.[11,19]

Although conventional cultivators of the sweep and chisel type will, with careful adjustment, work under mulch conditions, conventional plant shields clog and drag mulch over small plants. In most instances, disk hillers have been found to provide the necessary shielding action.

8.16. Effect of Mulch Tillage on Soil and Water Losses. Soil splash has been considered a measure of the effect of raindrop energy on soil erosion. Soil splash measurements on plots at Coshocton, Ohio, gave an average of 12.7 tons of soil splash per acre on turn plow plots and 7.5 tons per acre on mulch plots.[15]

Mulch tillage on contoured corn land reduced soil loss over 90 per cent. The largest amount of soil loss for any one storm on the mulch area was 0.25 ton per acre. The corresponding value for plowed contour corn land was 6.50 tons per acre.[14]

In dry land farming areas, one of the prime objectives of mulching is to reduce runoff and thus permit deeper storage of water for the succeeding crop. Through the use of portable wind tunnels, tests have been made of the comparative wind erosion under various quantities of cover (see Chapter 7). Stubble mulch fallow has consistently shown a lower soil loss than other management methods.

OTHER TILLAGE PRACTICES

8.17. Listing and Ridge Planting. In areas of low rainfall, areas in which a large per cent of the rainfall comes in short intense storms, in regions where gently sloping fields permits the use of contouring alone as an erosion control practice, and in some wind erosion problem areas, tillage is frequently carried out with listers. This implement, which is essentially a double wing plow, provides a furrow averaging 6 inches deep, 4 inches wide at the bottom, and 25 inches wide at the top. In some instances, a device is attached to create dams at intervals in the channel. Called basin listing, this controls row drainage and increases the water conservation value of the furrows.

The effectiveness of listing as a conservation measure has been shown in many studies. Iowa reports that on an erosive loess soil, over a 5-year period, contour listing of corn cut soil loss to about one-ninth that of uphill and downhill planting; water losses were reduced 2.4 inches.[37]

A similar practice referred to as ridge planting is used in drainage problem areas and with crops such as tobacco and sugar cane that require good drainage from the root zone. The crop is planted on the ridges rather than in the furrow as in conventional listing.

8.18. Subsoil Tillage. Deep tillage and subsoiling have been applied in various ways in the conservation program. The general objectives are to (1) deepen the effective plow zone depths and (2) break through and shatter plow soles and layers compacted by excessive implement traffic, impermeable soil horizons, or other barriers to the movement of moisture and roots through the profile.[5,6,12,28]

Summaries of subsoiling studies made by Chilcott,[6] and other more recent reviews, indicate that as a general practice subsoiling has not resulted in large yield increases or vastly improved soil conditions. Where subsoiling has been applied to problems of pans, soles, and other specialized profile conditions more significant results have been obtained. For example, in western irrigation lands, it has been found that on soils having a compacted plow pan, stratified soils, and soils having relatively thin compacted or cemented layers, subsoiling has improved irrigation water penetration. In that area, subsoiling is considered feasible and beneficial whenever there is a relatively thin breakable layer, having a lower permeability than the overlying materials and provided it lies within the top 15 inches of the profile.[26] All studies have indicated that the most effective results are obtained when the soil conditions are dry, thus contributing to the shattering action of the subsoiling.

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PROBLEMS

8.1. Determine strip widths for strip cropping in the southeastern states for a field having an average slope of 7 per cent. Using 42-inch rows, what strip widths should be laid out to accommodate two-row equipment? Four-row equipment?

8.2. Based on data in Fig. 8.3 (exclude 1939 and 1943 data), what percentage reduction in runoff could be expected by contour strip cropping a 5.44 acre field, using a corn-wheat-clover rotation? For 1 inch of rainfall, how many tons of water are held on the field as a result of strip cropping?

8.3. A field with slopes of 7.5 per cent requires 5 hours for one tillage operation and the consumption of 20 gallons of fuel. How much time and fuel could be saved by contouring as compared to farming uphill and downhill? What other factors could be considered in making such a comparison?

CHAPTER 9

Vegetated Outlets and Watercourses

The design of vegetated waterways is basically a problem of open-channel hydraulic engineering. It is more complex than the design of channels lined with concrete or other inert material because of variation in the roughness coefficient with depth of flow, stage of vegetal growth, hydraulic radius, and velocity.

9.1. Uses of Vegetated Outlets and Watercourses. Runoff from sloping land must flow to lower lands in a controlled manner which will not result in gully formation. Flow may be concentrated by the natural topography or by contour furrows, terraces, or other works of man. In any event considerable amounts of energy are dissipated as flow proceeds down a slope. Fifty cfs flowing 100 feet down a 5 per cent slope release energy at the rate of over 28 horsepower. If this energy acts upon bare soil, considerable quantities of soil particles will be detached and transported by the water. The resultant gullies may divide a field into several parts. Fields thus become smaller and more numerous, rows are shortened, movement from field to field is obstructed, and the farm value is decreased. Roads, bridges, buildings, and fences frequently are jeopardized by gully development. Soil carried from gullied areas contributes to costly downstream sedimentation damage.

Providing properly proportioned channels protected by vegetation, which absorbs the energy of runoff without damage, is frequently a complete solution to the problem of gully formation. For large runoff volumes or steep channels it may be necessary to supplement the vegetated watercourse by permanent gully control structures. Vegetated waterways should be used to handle natural concentrations of runoff or to carry the discharge from terrace systems, contour furrows, diversion channels, or emergency spillways for farm ponds or other structures. Vegetated waterways should not be used for continuing flows, such as may discharge from tile drains, as prolonged wetness in the waterway will result in poor vegetal protection.

DESIGN

9.2. Determination of Runoff. In the design of a vegetated watercourse, the functional requirements should be determined and then the channel proportioned to meet these requirements. The capacity of the waterway should be based on the estimated runoff from the contributing drainage area. The 10-year recurrence interval storm is a sound basis for vegetated waterway design. For exceptionally long watercourses it may be desirable to estimate the flow for each of several reaches of the channel to account for changing drainage area. For short channels the estimated flow at the waterway outlet is the practical design value.

9.3. Shape of Waterway. The cross-sectional shape of the channel as it is constructed may be parabolic, trapezoidal, or triangular. The parabolic cross section approximates that of natural channels. Under the normal action of channel flow, deposition, and bank erosion, the trapezoidal and triangular sections tend to become parabolic. In some channels no earthwork is necessary, the natural drainageway or meadow outlet is adequate, and only boundaries need to be defined.

A number of factors influence the choice of the shape of cross section. Channels built with a blade-type machine may be trapezoidal if the bottom width of the channel is greater than the minimum width of the cut. Triangular channels may also be readily constructed with such equipment. Trapezoidal channels having bottom widths less than a mower swath are difficult to mow. Flat triangular or parabolic channels with side slopes of 4:1 (4 feet horizontal to 1 foot vertical) or flatter may be easily maintained by mowing. Side slopes of 4:1 or flatter are also desirable to facilitate crossing with farm equipment.

Broad-bottom trapezoidal channels require less depth of excavation than do parabolic or triangular shapes. During low flow periods, sediment may be deposited in trapezoidal channels with wide, flat bottoms. Uneven sediment deposition may result in meandering of higher flows and development of turbulence and eddies which will cause local damage to vegetation. Triangular channels reduce sedimentation, but high velocities may damage the bottom of the waterway.

Parabolic cross sections should usually be selected for channels in natural waterways. A trapezoidal section with a slight V bottom is most easily constructed where the waterway is artificially located as in a terrace outlet along a fence line.

The geometric characteristics of the three shapes of cross sections are given in Fig. 9.1. This figure defines the three types of cross sections and gives formulas necessary for computing the hydraulic characteristics of each.

9.4. Selection of Suitable Vegetation. Soil and climatic conditions are primary factors in the selection of vegetation. Vegetation recommended for various regions of the United States is indicated in Table 9.1. Other factors to be considered are

Table 9.1 VEGETATION RECOMMENDED FOR GRASSED WATERWAYS*

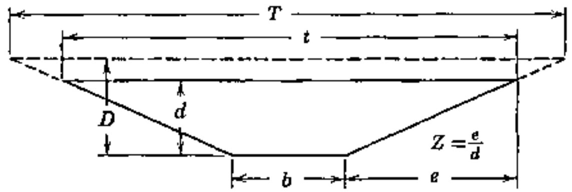
Geographical Area of U. S.	*Vegetation*
Northeastern	Kentucky bluegrass, red top, white clover
Southeastern	Kentucky bluegrass, Kentucky 31 fescue, Bermuda, brome, Reed canary
Upper Mississippi	Brome, Reed canary, alta fescue, Kentucky bluegrass
Western Gulf	Bermuda, King Ranch bluestem, native grass mixture, Kentucky 31 fescue
Southwestern	Intermediate wheat grass, western wheat grass, smooth brome, tall wheat grass
Northern Great Plains	Smooth brome, western wheat grass, red top-switch grass, native bluestem mixture

* From Soil Conservation Service.

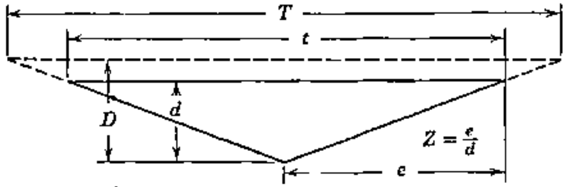
duration, quantity, and velocity of runoff, ease of establishment of vegetation, time required to develop a good protective cover, suitability to the farmer in regard to utilization of the vegetation as a seed or hay crop, spreading of vegetation to adjoining fields, cost and availability of seed, and retardance to shallow flows in relation to sedimentation.

9.5. Design Velocity. The ability of vegetation to resist erosion is limited. The permissible velocity in the channel is dependent upon the type, condition, and density of vegetation and the erosive characteristics of the soil. Uniformity of cover

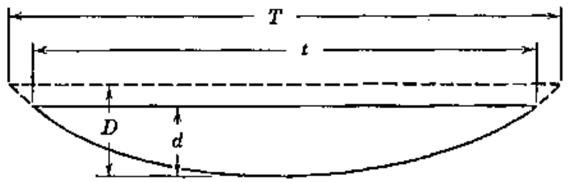
Note: Freeboard = $D-d$ for all sections



Trapezoidal cross section



Triangular cross section



Parabolic cross section

Cross-Sectional Area A	Wetted Perimeter	Hydraulic Radius R	Top Width t
$bd + Zd^2$	$b + 2d\sqrt{Z^2+1}$	$\frac{bd + Zd^2}{b + 2d\sqrt{Z^2+1}}$	$b + 2dZ$
Zd^2	$2d\sqrt{Z^2+1}$	$\frac{Zd}{2\sqrt{Z^2+1}}$	$2dZ$
$\frac{2}{3}td$	$t + \frac{8d^2}{3t}$	$\frac{t^2d}{1.5t^2 + 4d^2}$	$\frac{A}{0.67d}$

Fig. 9.1. Channel cross sections, notations, and formulas. (Redrawn from U. S. Soil Conservation Service.[5])

is very important, as the stability of the most sparsely vegetated area controls the stability of the channel. Permissible velocities for bunch grasses or other nonuniform covers are lower than those for sod-forming grasses. Bunch grasses produce nonuniform flows with high localized erosion. Their open roots do not bind the soil firmly against erosion.

Permissible velocities are also influenced by bed slope. Steeper channels produce increasing turbulence with intense localized erosion. Suggested design values for velocity are given in Table 9.2. It should be recognized that the design velocity is an

Table 9.2 PERMISSIBLE VELOCITIES FOR CHANNELS* LINED WITH VEGETATION

Cover	Permissible Velocity, fps					
	Erosion Resistant Soils			Easily Eroded Soils		
	% Slope			% Slope		
	0–5	5–10	Over 10	0–5	5–10	Over 10
Bermuda grass	8	7	6	6	5	4
Buffalo grass Kentucky bluegrass Smooth brome Blue grama	7	6	5	5	4	3
Lespedeza sericea Weeping lovegrass Kudzu Alfalfa Crabgrass	3.5			2.5		
Annuals for temporary protection	3.5			2.5		

* Modified from Ree.[2]

average velocity rather than the actual velocity in contact with the vegetation or with the channel bed. Figure 9.2 shows the velocity distribution in a grass-lined channel and illustrates this point. Though the average velocity in the cross section is about 2.5 fps, the velocity in contact with the vegetation and bed is less than 1 fps.

Design of vegetated waterways is based upon the Manning formula (see Appendix C).

9.6. Roughness Coefficient. Slope and hydraulic radius are evaluated readily from the geometry of the channel. However, the roughness coefficient is more difficult to evaluate. Extensive tests reported by Cox and Palmer,[1] Ree,[2] Ree and

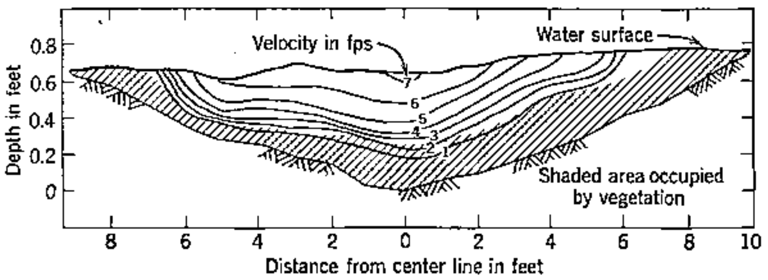


Fig. 9.2. Velocity distribution in a grass-lined channel. (Redrawn from Ree.[2])

Palmer,[3] and Smith[4] have been conducted at Stillwater, Oklahoma, Spartanburg, South Carolina, and McCredie, Missouri, to determine roughness coefficients for various types of vegetation. Figure 9.3 illustrates the complexity of the problem. The roughness coefficient varies tremendously with the depth of flow. Flows at very shallow depth encounter a maximum resistance because the vegetation is upright in the flow. The slight increase in resistance in the low flow range apparently is due to the greater bulk of vegetation encountered with increasing depth. Intermediate flows bend over and submerge some of the grass, and resistance drops off sharply, as more and more vegetation is submerged.

Resistance to flow is also influenced by the gradient of the channel. Decreasing resistance apparently results from higher velocities on steeper slopes with an accompanying greater flattening of the vegetation.[3] Type and condition of vegetation has a great influence on the retardance. Newly mowed grass offers less resistance than rank growth. Long plants, stems, and leaves tend to whip and vibrate in the flow, thus introducing and maintaining considerable turbulence. The cross-sectional shape of the channel has only minor influence upon the roughness coefficient in the range of cross sections commonly used.

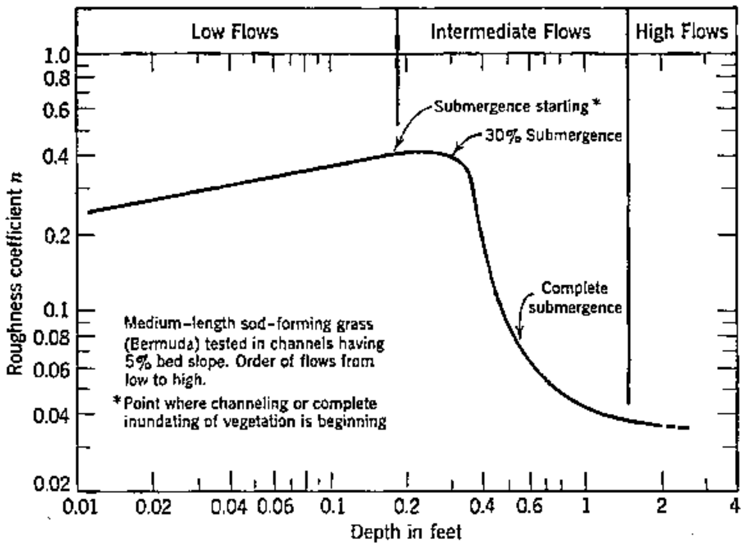


Fig. 9.3. Hydraulic behavior of a medium-length sod-forming grass. (Redrawn from Ree.[2])

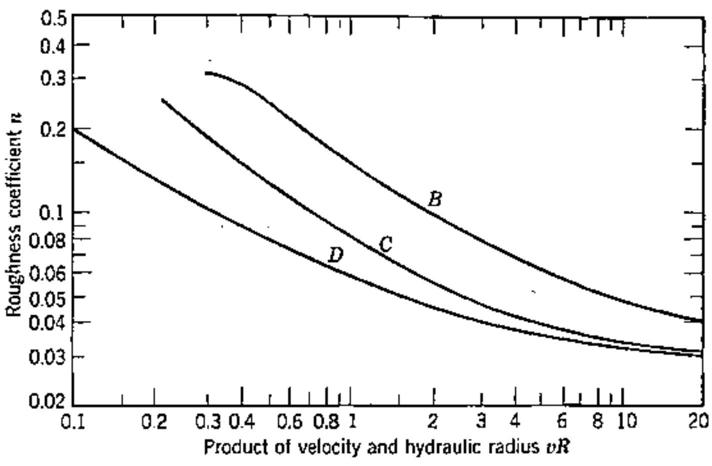


Fig. 9.4. Roughness coefficient as a function of vR for various retardance classes of vegetation. (Redrawn from Ree.[2])

The product vR, velocity multiplied by hydraulic radius, has been found to be a satisfactory index of channel retardance for

Table 9.3 Classification of Vegetal Cover According to Retardance*

Retardance Class	*Cover*	*Condition*
B	Alfalfa	Good stand, uncut (avg. 11 in.)
	Bermuda grass	Good stand, tall (avg. 12 in.)
	Blue grama	Good stand, uncut (avg. 13 in.)
	Brome	Long
	Kudzu	Dense or very dense, uncut
	Lespedeza sericea	Good stand, not woody, tall (avg. 19 in.)
	Reed canary	Long
	Weeping lovegrass	Good stand, tall or mowed (avg. 13–24 in.)
C	Bermuda grass	Good stand, unmowed (avg. 6 in.)
	Brome	Mowed
	Centipede grass	Very dense cover (avg. 6 in.)
	Common lespedeza	Good stand, uncut (avg. 11 in.)
	Crabgrass	Fair stand, uncut (10–48 in.)
	Grass mixture (orchard grass, red top, Italian ryegrass, common lespedeza)	Good stand, uncut (6–8 in.)
	Kentucky bluegrass	Good stand, headed (6–12 in.)
	Reed canary	Mowed
D	Bermuda grass	Good stand, cut to 2½ in.
	Common lespedeza	Excellent stand, uncut (avg. 4½ in.)
	Buffalo grass	Good stand, uncut (3–6 in.)
	Grass mixture (as above)	Good stand, uncut (4–5 in.)
	Lespedeza sericea	Good stand, cut to 2 in.

* Modified from Ree.[2]

design purposes. Vegetation has been grouped into five retardance categories designated A through E. Table 9.3 gives a portion of this classification of vegetation by degree of retardance, and Fig. 9.4 shows the n-vR curves for these retardance categories. In the past it has been common practice to use $n = 0.04$ for vegetated waterways. In many channels this may be satisfactory, but careful consideration should be given to the vegetation and flow conditions before doing so.

9.7. Channel Capacity. The channel must be proportioned to carry the design runoff at average velocities less than or equal to the permissible velocity. This is accomplished by application of the Manning formula. Design by the Manning formula is essentially a trial and error process though explicit solutions for channel dimensions can be made for certain cross sections. The channel should be designed to carry the runoff at a permissible velocity under conditions of minimum retardance which may be encountered during the runoff season. This condition establishes the basic proportions of the channel, for example, the bottom width of a trapezoidal channel. Additional depth should then be added to the channel to provide adequate capacity under conditions of maximum retardance. A freeboard of 0.3 to 0.5 foot should be added to the design depth. Example 9.1 illustrates design procedure.

Example 9.1. Design a trapezoidal grassed waterway to carry 200 cfs down a 3 per cent slope on erosion-resistant soil. The vegetation is to be Bermuda grass, and the channel will have a 4:1 side slope.

Solution. Reference to Table 9.2 shows the permissible velocity to be 8 fps. Table 9.3 lists Bermuda grass in retardance class D when mowed and in class B when long. To design the channel for stability, consider the mowed condition. The problem may be solved by trial and error from Fig. 9.4; however, Figs. C.2 through C.4 are included in the Appendix for convenience in solving problems of this type. Entering Fig. C.4 (retardance class D) with $v = 8$ fps and a slope of 3 per cent, $R = 1.08$. By trial and error, select a trapezoidal channel.

Try $b = 11$ feet, $d = 1.5$ feet, which results in $a = 25.5$ square feet, and $R = 1.09$; thus these channel dimensions are acceptable from the standpoint of stability. The designed channel must be deep enough to carry the flow at low velocities, which will result when the grass is long. Try a depth of 2 feet, which with the 11-foot bottom width gives $a = 38$ square feet and $R = 1.38$ feet. Entering Table C.2 with $R = 1.38$ and a slope of 3 per cent gives $v = 6.1$ fps. Checking the capacity, $Q = av = 232$ cfs, which is adequate.

Adding 0.3-foot freeboard, the design dimensions of the channel are 11 feet bottom width and 2.3 feet depth.

The example shows that the bottom width is determined by the need for not exceeding the permissible velocity under the mowed condition of minimum retardance and that the depth is determined by the need to provide capacity under conditions of maximum retardance.

Waterways are often designed on the basis of tables, charts, or rules of thumb developed for a particular area. An engineer or technician working in a given region gains confidence in such shortcuts, particularly adapted to waterway design under local conditions.

Immediately after construction the channel may be called upon to carry runoff under conditions of little or no vegetation. It is not practical to design for this extreme condition. In many channels it may be practical and desirable to divert flow from the channel until vegetation is established. In others the possibility that high runoff will occur before vegetation is established is accepted as a calculated risk.

9.8. Drainage. Waterways that are located in seepy draws or below seeps, springs, or tile outlets will be continually wet for long periods of time. The wet condition will inhibit the development and maintenance of a good vegetal cover and will maintain the soil in a soft, erosive condition. Subsurface drainage or diversion of such flow is essential to the success of the waterway.

Low continuous flow of surface water entering at some point may be intercepted by a catch basin and carried off by a tile drain. A concrete or asphalt trickle channel of 1 or 2 square feet cross section is sometimes placed in the bottom of a waterway to carry prolonged low flows. Seepage along the sides or upper end of the waterway may be intercepted by tile drains. Tile should be placed to one side of the center of the waterway to prevent washing out of tile in case of failure of the waterway.

WATERWAY CONSTRUCTION

9.9. Shaping Waterways. The procedure and amount of work involved in shaping a waterway depends upon the topographic situation and the equipment available. If the watercourse is to be located in a natural draw or meadow outlet where there is little gullying, only smoothing and normal seedbed preparation are required. Some improvement in alignment of the channel may be desired to remove sharp bends. This will improve the hydraulic characteristics, facilitate farming operations, and reduce channel maintenance. If the waterway is to reclaim an established gully, considerable earthwork is required. The gully must be filled and the waterway cross section established.

Small waterways may be easily shaped with regular farm equipment. Large gullies, however, can be most satisfactorily handled by a bulldozer or other heavy earth-moving equipment.

ESTABLISHMENT OF VEGETATION

9.10. Seedbed Preparation. Soil in the waterway should be brought to a high fertility level and limed in accordance with the soil and plant requirements. Manure worked into the seedbed provides needed plant nutrients and furnishes organic material which will help the soil to resist erosion. Specific lime and fertilizer needs depend upon local crop and soil conditions.

9.11. Seeding. Waterway seeding mixtures should include some quick-growing annual for temporary control as well as a mixture of hardy perennials for permanent protection. Seed should either be broadcast or drilled nonparallel to the direction of flow. Mulching after seeding helps to secure a good stand. Where high flows must be turned into a channel before seedings can become established, sodding may be justified.

WATERWAY MAINTENANCE

9.12. Causes of Failure. Failure of vegetated waterways may result from insufficient capacity, excessive velocity, or inadequate vegetal cover. The first two of these are largely a matter of design. The condition of the vegetation, however, is influenced not only by the initial preparation of the waterway but also by the subsequent management. Use of a waterway especially in wet weather, as a lane, stock trail, or pasture injures the vegetation and often results in failure. Terraces that empty into a waterway at too steep a grade may cut back into the terrace channel, injuring both terrace and waterway. Careless handling of machinery in crossing a waterway may injure the sod. When land adjacent to the waterway is being plowed, the ends of furrows abutting against the vegetated strip should be staggered to prevent flow concentration down the edges of the watercourse.

9.13. Controlling Vegetation. Waterways should be mowed and raked several times a season to stimulate new growth and control weeds. A rotary-type mower cuts the grass fine enough to make raking unnecessary. Annual application of manure and fertilizer maintains a dense sod. Any breaks in the sod should be repaired. Rodents that are damaging waterways should be controlled.

9.14. Sediment Accumulation. Good conservation practice on the watershed is the most effective means of controlling sedimentation. Accumulated sediments smother vegetation and restrict the capacity of the waterway. Extending vegetal cover well up the side slopes of the waterway and into the outlets of terrace channels helps to prevent sediment from being deposited in the watercourse. Control of vegetation to prevent a rank, matted growth reduces the accumulation of sediment. High allowable design velocities also decrease sedimentation.

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3. Ree, W. O., and V. J. Palmer, Flow of Water in Channels Protected by Vegetative Linings, *U. S. Dept. Agr. Tech. Bull. 967* (1949).
4. Smith, D. D., Bluegrass Terrace Outlet Channel Design, *Agr. Eng., 27:* 125–130 (1946).
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PROBLEMS

9.1. Determine the velocity of flow in a parabolic-, a triangular-, and a trapezoidal-shaped waterway, all having a cross-sectional area of 20 square feet, a depth of flow of 1.0 foot, a channel slope of 4 per cent, and a roughness coefficient of 0.04. Assume 4:1 side slopes for the trapezoidal cross section.

9.2. Design a parabolic-shaped grassed waterway to carry 50 cfs. The soil is easily eroded; the channel has a slope of 4 per cent; and a good stand of Bermuda grass, cut to 2½ inches, is to be maintained in the waterway.

9.3. Design a trapezoidal-shaped waterway with 4:1 side slopes to carry 20 cfs where the soil is resistant to erosion and the channel has a slope of 12 per cent. Brome grass in the channel may be either mowed or long when maximum flow is expected.

CHAPTER 10

Terracing

Terracing is a method of erosion control accomplished by constructing broad channels across the slope of rolling land. In 1945, the Soil Conservation Service[15] estimated that over 90,000,000 acres of cropland in the United States was in need of terracing.

10.1. Progress in Terracing. The first terraces consisted of large steps or level benches as compared to the broadbase terraces now common. For several thousand years bench terraces have been widely adopted over the world, particularly in Europe, Australia, and Asia. In the United States ditches which functioned as terraces were constructed across the slopes of cultivated fields by farmers in the southern states during the latter part of the eighteenth century.

10.2. Function of Terraces. Terracing of cultivated land is always combined with contouring. Since terracing requires an additional investment and causes some changes in farming procedures,[23] it should be considered only where other cropping or soil management practices, singly or in combination, will not provide adequate control.

The function of terraces is to decrease the length of the hillside slope, thereby reducing sheet and rill erosion, preventing the formation of gullies, and retaining runoff in areas of inadequate precipitation. In dry regions such conservation of moisture is important in the control of wind erosion. In most areas graded terraces are more effective in reducing erosion than runoff, whereas level terraces are effective in reducing runoff as well as controlling erosion.

10.3. Terrace Classification. The two major types of terraces are the bench terrace which reduces land slope and the broadbase terrace which removes or retains water on sloping land. Bench terraces are adapted to slopes of 25 to 30 per cent, are costly to construct, and are not suitable for farming with heavy machinery. The broadbase terrace is of major importance in the United States, whereas the bench terrace is of little more than academic interest.

The long process of development of the broadbase terrace has led to a variety of types, terms, and classifications. On the basis of construction they are classified as the Nichols or channel terrace, which is constructed from the upper side only, and the Mangum or ridge terrace, constructed from both sides.

BROADBASE TERRACE

A broadbase terrace is a broad surface channel or embankment constructed across the slope of rolling land. On the basis of primary function the broadbase terrace is classified as graded or level. The distinguishing characteristic of this terrace is farmability.

In addition to usual factors affecting runoff and erosion, soil and water losses from terraced areas are influenced by both the spacing and the length of the terrace, as well as the velocity of flow in the channel.

10.4. Graded Terrace. The graded terrace may be constructed with a variable or a uniform grade in the channel. In Wisconsin[16] a channel-type terrace has been developed with little or no ridge for use on poorly drained soils with slopes less than 4 per cent (see Chapter 14). Parallel terraces, applicable to relatively uniform slopes, facilitate farming operations by eliminating point rows.

The magnitude of soil losses from terraced areas has been determined from field investigations. At Bethany, Missouri, on slopes of 7.2 per cent the total soil loss was 7.6 tons per acre per year, of which 6.6 tons were deposited in the channel and 1.0 ton was removed in the runoff.[19]

Although newly constructed terraces may cause decreased yields for a few years, because of the reduced depth of topsoil in the channel and mixing of topsoil and subsoil in the terrace ridge, this effect can largely be overcome by adoption of an adequate fertility program. Results for graded terraces are not always consistent, but over-all yield increases of 10 to 25 per cent are not uncommon.

10.5. Level Terraces. Level terraces are constructed on the contour and are generally recommended in areas where the soil is sufficiently permeable to prevent overtopping of the ridge and serious damage to crops. In regions having very permeable soil, level terraces are used on slopes up to 20 per cent. In this event their function is largely that of preventing erosion. In semiarid regions the level terrace is often used for moisture conservation on slopes of 2 per cent or less. When these flatter slopes are uniform, level terraces may be laid out parallel. In the Great Plains on slopes of less than 2 per cent level terraces conserve moisture; therefore, wheat and cotton yields have been increased as much as 20 to 60 per cent. However, in these regions actual yields are low compared to those in more humid areas. Where level terraces are used on slopes over 2 per cent, water in the channel is spread over a relatively small area, reducing the area in which the moisture is conserved, thus less affecting crop yield.

TERRACE DESIGN

The design of a terrace system involves the proper spacing and location of terraces, the design of a channel with adequate capacity, and development of a farmable cross section. For the graded terrace, runoff must be removed at nonerosive velocities in both the channel and the outlet. Soil characteristics, cropping and soil management practices, and climatic conditions are the most important considerations in terrace design.

TERRACE SPECIFICATIONS

10.6. Terrace Spacing. Spacing is expressed as the vertical distance between the channels of successive terraces. For the top terrace the spacing is the vertical distance from the top of the hill to the bottom of the channel. This vertical distance is commonly known as the vertical interval or V.I. Although the horizontal spacing is useful in such matters as determining the row arrangement, the vertical interval is more convenient for terrace layout and construction.

Graded. Under given conditions graded terrace spacing is often expressed as a function of land slope by the empirical formula

$$\text{V.I.} = a + \frac{S}{b} \tag{10.1}$$

where a = constant.

b = another constant.

S = land slope above the terrace in per cent.

Spacings thus computed may be varied as much as 25 per cent to allow for soil, climatic, and tillage conditions. Terraces are seldom recommended on slopes over 20 per cent, and in many regions slopes from 10 to 12 per cent are considered the maximum. Numerical values of a and b in equation 10.1 are given in Table 10.1.

Table 10.1 VERTICAL INTERVAL CONSTANTS*

Geographical Areas of U. S.	a	b
Southwestern	Use V.I. = $2(S)^{1/2} + 0.7$	
Upper Miss. Valley	2	2
Southeastern		
0–6% slope	1	2
6–12% slope	2	3
Northeastern	2	3
Western Gulf		
West 96th meridian	1.5	2
East 96th meridian	1	2
Northern Great Plains	2	3

* Based on data from Soil Conservation Service.

Terrace spacing should not be so wide as to cause excessive rilling and the resultant movement of large amounts of soil into the terrace channel. On sandy soils excessive interterrace erosion makes the soils progressively sandier as the fines are carried away and the sand grains are left behind.

Reduction of the hillside slope length is the most important effect of terracing (see Fig. 6.8). At Bethany, Missouri, when a slope length of 600 feet was terraced at 100-foot intervals, soil loss was reduced by 65 per cent.[13]

Level. The spacing for level terraces is a function of channel infiltration and runoff; however, in more humid areas where erosion control is important, the slope length may limit the spacing. The runoff from the terraced area should not cause overtopping of the terrace, and the infiltration rate in the channel should be sufficiently high to prevent serious damage to crops. Spacings vary so widely in different parts of the country that general recommendations are not possible. In the Great

Plains regions level terraces are used on slopes up to 3 per cent with vertical intervals of 2 feet or less. Where level terraces are used in more humid regions, the spacings are comparable to those used for graded terraces.

10.7. Terrace Grades. Gradient in the channel must be sufficient to provide good drainage and to remove runoff at nonerosive velocities. The minimum slope is desirable from the standpoint of soil loss. With reference to slope in the channel, terraces are constructed with uniform or variable grades. Level terraces have zero grades.

In the uniform-graded terrace the slope remains constant throughout its entire length. Because of higher velocities in the upper portions of the channel of a uniform graded terrace, sediment is not as likely to be deposited as in the upper portions of the variable-graded terrace. A grade of 0.3 per cent is common in many regions; however, grades may range from 0.1 to 0.6 per cent, depending on soil and climatic factors. Generally, the steeper grades are recommended for impervious soils and long terraces. Uniform-graded terraces are best where drainage is a problem and where the terraces are short.

The variable-graded terrace is more effective because the capacity increases toward the outlet with a corresponding increase in runoff. The grade usually varies from a minimum at the upper portion to a maximum at the outlet end. The resulting reduced velocity in the upper reaches provides for greater absorption of runoff. Variable gradient makes possible greater flexibility in design; for instance, either constant velocity or constant capacity could be provided by varying the grade in the channel. Such designs are sometimes required, particularly in large diversion terraces.

10.8. Terrace Length. Size and shape of the field, outlet possibilities, rate of runoff as affected by rainfall and soil infiltration, and channel capacity are factors that influence terrace length. The number of outlets should be a minimum consistent with good layout and design. Extremely long graded terraces are to be avoided; however, long lengths may be reduced in some terraces by dividing the flow midway in the terrace length or at the ridge crest and draining the runoff to major natural waterways, and, when necessary, to constructed outlets at both ends of the terrace[4] (see Fig. 10.2). The length should be such that erosive velocities and large cross sections are not required. On permeable soils longer terraces may be permitted than on impermeable soils. The maximum length for graded terraces generally ranges from about 1000 to 1800 feet, depending on local conditions. The maximum applies only to that portion of the terrace that drains toward one of the outlets.

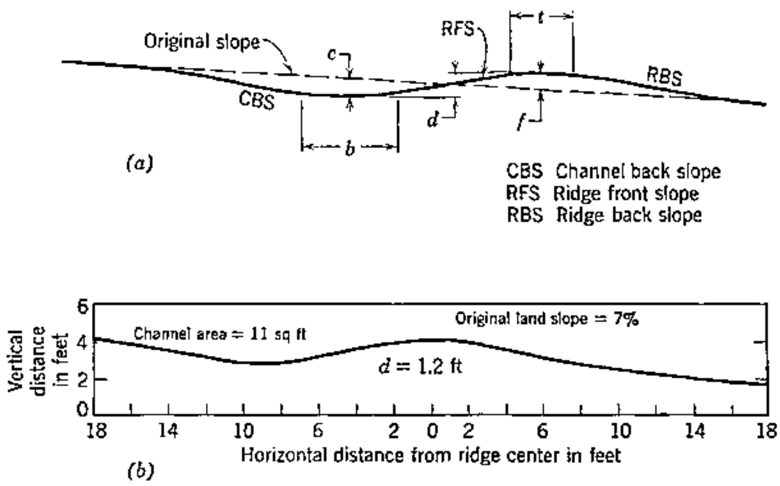

Fig. 10.1. Typical terrace cross sections. (*a*) Design cross section and (*b*) cross section after 10 years of farming. (Redrawn from Smith and others.[12])

There is no maximum length for level terraces, particularly where blocks or dams are placed in the channel every 400 or 500 feet. These dams prevent total loss of water from the entire terrace and reduce gully damage should a break occur. The ends of the level terrace may be left partially or completely open to prevent overtopping in case of excessive runoff.

10.9. Terrace Cross Section. The terrace cross section should provide adequate capacity, have broad farmable side slopes, and be economical to construct with available equipment. Cross sections can be described by side slopes of the channel and ridge, channel width, ridge width, and ridge height, as shown in Fig. 10.1. A typical design cross section is shown in (*a*) and a cross section after 10 years of farming in (*b*).

Recommended cross section dimensions may vary in different regions. As the slope increases, the width decreases and the depth of channel, d, and all side slopes increase, making the cross section more difficult to farm. The cross-sectional area of the channel should be at least 6 to 10 square feet, depending on channel capacity and runoff. Recommended dimensions for terrace cross sections for various slopes are shown in Table 10.2.

For practical reasons a terrace is usually constructed with a uniform cross section from the outlet to the upper end, although this construction results in the upper portion of the channel being overdesigned.

10.10. Runoff from Terraced Areas. With graded terraces the rate of runoff is more important than total runoff, whereas both rate and total runoff influence the design of level terraces. Graded terraces are designed as drainage channels or waterways, and level terraces function as storage reservoirs. The terrace channel acts as a temporary storage reservoir subjected to unequal rates of inflow and outflow. Inflow is affected by variables given in the runoff equation; outflow is influenced by the grade in the channel as well as by the inflow rate.

The design should be based on a recurrence interval of 10 years, and the runoff coefficient should be for the most severe condition, that is, bare saturated soil. Total runoff from a terrace interval can be determined from rainfall data after correcting for interception and infiltration losses.

CHANNEL CAPACITY

The channel capacity for graded terraces must be adequate to carry the design runoff for the most severe conditions. The Manning velocity formula given in Appendix C is satisfactory for design. The design velocity will vary with the erosiveness of the soil but should rarely exceed 2 feet per second for soil devoid of vegetation.

10.11. Terrace Channel Capacity. A roughness coefficient $n = 0.04$ should be used for design on tillable land, for soil without vegetation presents conditions under which maximum damage from overtopping is likely to occur. The channel depth should permit a freeboard of about 20 per cent of the design depth after allowing for settlement of the ridge (see Section 10.17).

Table 10.2 TERRACE DIMENSIONS

Graded Terrace*

Field Slope, %	Terrace Ridge Height, ft, d — Terrace Length, ft					Approximate Slope Ratio		
	200	400	600	800	1000	CBS	RFS	RBS
2	0.8	0.9	1.0	1.2	1.2	10:1	10:1	10:1
4	0.7	0.9	1.0	1.1	1.1	6:1	8:1	8:1
6	0.7	0.8	0.9	1.0	1.0	6:1	8:1	8:1
8	0.7	0.8	0.9	1.0	1.0	4:1	6:1	6:1
10	0.6	0.8	0.9	1.0	1.0	4:1	6:1	6:1
12	0.6	0.8	0.9	1.0	1.0	4:1	4:1	4:1
‡15	0.6	0.7	0.9	1.0	1.0	4:1	4:1	$2\frac{1}{2}$:1

Note: Above figures are settled ridge height and are based on 10-year runoff and a channel with 6-foot bottom. A top width of at least 2 feet should be provided.

Level Terrace*†

Field Slope %	Terrace Ridge Height, ft, d	Approximate Slope Ratio			Width, ft	
		CBS	RFS	RBS	Channel b	Ridge t
2	1.2	6:1	6:1	6:1	8	3
4	1.2	5:1	6:1	6:1	8	3
6	1.2	5:1	6:1	5:1	8	3
8	1.2	5:1	6:1	5:1	6	3
10	1.2	5:1	5:1	4:1	6	3
12	1.3	4:1	4:1	4:1	6	3
‡15	1.3	$3\frac{1}{2}$:1	$3\frac{1}{2}$:1	$2\frac{1}{2}$:1	6	3

* Based on data from U. S. Soil Conservation Service for Upper Mississippi Region.

† Channel capacity based on retaining 2-inches runoff.

‡ Terrace ridge and RBS to be kept in sod.

PLANNING THE TERRACE SYSTEM

The terrace system should be coordinated with the complete water-disposal system for the farm, giving adequate consideration for proper land use. Terrace systems should be planned by watershed areas and should include all terraces that may be constructed at a later date. Where practicable, adjacent farms having fields in the same drainage area may have joint terracing systems. Factors such as fence and road location must be considered.

10.12. Selection of Outlets. One of the first steps in planning is the selection of outlets or disposal areas. Since level terraces generally do not require outlets, their location and layout are greatly simplified. The design, construction, and maintenance of vegetated outlets and watercourses as discussed in Chapter 9 are applicable for terrace outlets.

The design runoff for the outlet is determined by summation of the runoff from individual terraces. However, if there are one or two long terraces and many short ones, variation in the time of concentration for the different terraces may cause inaccuracy in the application of the rational runoff formula.

Outlets are of many types, such as natural draws, constructed channels, sod flumes, permanent pasture or meadow, road ditches, waste land, concrete or stabilized channels, tile drains, and stabilized gullies. Natural draws, where properly vegetated, provide a desirable and economical outlet. Where these draws do not permit adequate field size, constructed waterways along field boundaries should be considered, provided other natural outlets are not available. The location of constructed outlets should permit the least interference with tillage operations, provide the most favorable length of terraces, permit satisfactory gradient in the outlet channel, and provide suitable conditions for establishing the required vegetative lining. Waste land or permanent pasture is suitable, provided erosion can be controlled. Terrace outlets onto pasture should be staggered by increasing the length of each terrace about 25 feet, starting with the lowest terrace. Sod flumes, concrete channels, and tile drains are to be avoided because of excessive cost. If mechanical structures or linings are necessary, a location to permit the shortest length of outlet may be desirable for economic reasons. Arrangements should be made with the Highway Department before outleting into road ditches. Road ditches and gullies must be used with caution and only after provisions have been made to prevent scouring and enlargement of the ditch or gully.

10.13. Terrace Location. After a suitable outlet is located, the next step is the location of the terraces. Factors that influence terrace location include: (1) land slope, (2) soil conditions, such as degree and extent of erosion, (3) proposed land use, (4) boulders, trees, gullies, and other impediments to cultivation, (5) farm roads, (6) fences, (7) row layout, (8) type of terrace, and (9) outlet. Minimum maintenance and adequate control of erosion are the criteria for good terrace location. Better alignment of terraces can usually be obtained by placing the terrace ridge just above eroded spots, gullies, and abrupt changes in slope. Satisfactory locations for roads and fences are on the ridge, on the contour, or on the spoil to the side of the outlet.

Unless there are obvious reasons for doing otherwise, the top terrace is laid out first, starting from the outlet end. It is important that the top terrace be located in the proper place, so that it will not overtop and cause failure of other terraces below. Some general rules for the location of the top terrace are: (1) The drainage area above the top terrace ordinarily should not exceed 3 acres. (2) If the top of the hill comes to a point, the interval may be increased to 1½ times the regular vertical interval. (3) On long ridges, where the terrace approximately parallels the ridge, the regular vertical interval should be used. (4) If short abrupt changes in slope occur, the terrace should be placed just above the break.

Obstructions or topographic features below the top terrace such as a boulder or tree may necessitate locating a terrace at that point first. This terrace is called the key terrace because other terraces are located from it. Terraces above the key terrace, which are located by determining the vertical interval as before, may require an adjustment in spacing in order to place the top terrace at the proper location. Terraces below the key terrace are located by using the normal vertical interval.

A typical terrace layout is shown in Fig. 10.2. The top terrace (1*a* and 1*b*) is a diversion which intercepts runoff from the pasture and prevents overflow on the cultivated land below. Since the slope below terrace 1*a* is uniform, terraces 2*a* and 3*a* are laid out parallel to it. Because terraces 2*bc* and 3*bc* are each longer than 1600 feet, outlets are provided at each end.

Level terraces are located in much the same manner as graded terraces. On flat slopes in the Great Plains, level terraces are sometimes constructed so that runoff is allowed to flow from one terrace to the next by opening alternate ends of the terraces.

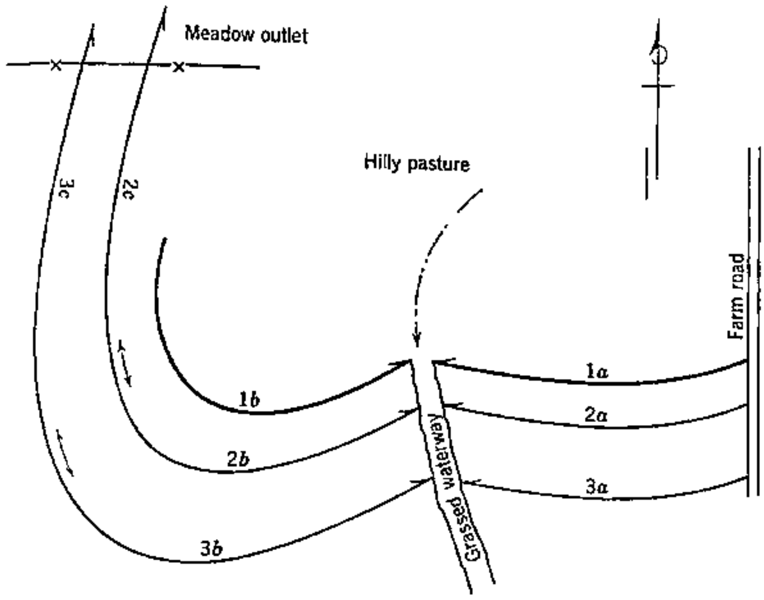

Fig. 10.2. Typical layout for graded terraces.

10.14. Layout Procedure. A good tripod level and the application of surveying techniques along with field experience are sufficient for terrace layout. When available, topographic maps are especially helpful in planning parallel terraces.

The first step is to determine the predominant slope above the terrace and then to obtain a suitable vertical interval. Stakes are normally set along the proposed terrace every 50 feet, although intervals are shorter if turns in the line are sharp. The grade in the channel is provided by placing the stakes on the desired grade, allowance being made at the outlet to compensate for the difference in the elevation of the constructed terrace channel and the stake line. Additional terraces are staked in the same manner. Several trial terraces may be necessary before their exact location is selected.

After the terrace lines are staked some realignment is necessary to eliminate sharp curves so as to provide greater convenience in farming. The general procedure is shown in Fig. 10.3. The small crosses represent the stakes as originally located with an instrument. Realignment of these stakes should be limited to provide a cut of not more than 0.5 foot below the bottom of the channel as normally constructed or a ridge height not in excess of 3 feet.

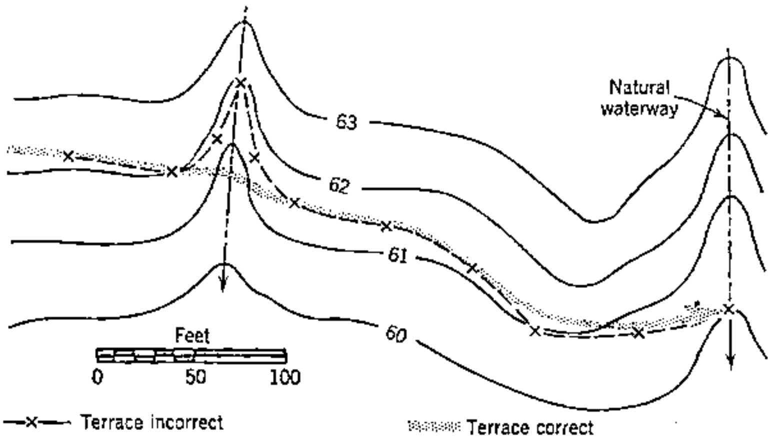

Fig. 10.3. Realignment of terraces after staking. (Redrawn from Hamilton.[8])

TERRACE CONSTRUCTION

10.15. Construction Equipment. A variety of equipment is available for terrace construction. Terracing machines may be classified on the basis of size as light equipment adapted to power available on the farm or heavy equipment designed for a number of earth-moving jobs. Terracing machines are classified as four types according to methods of moving soil: as shown in Fig. 10.4, lift and roll, throw, scrape and push, and carry.[9]

Requirements of an ideal terracing machine are (1) to displace soil laterally to the desired place in the ridge, (2) to construct terraces at a high rate of speed, (3) to build terraces effectively on all slopes up to 15 or 20 per cent, (4) to place topsoil on or near surface of the ridge, and (5) to have a low initial and operating cost.

10.16. Factors Affecting Rate of Construction. The rate of construction of terraces is affected chiefly by the following factors: (1) equipment, (2) soil moisture, (3) crops and crop residues, (4) degree and regularity of land slope, (5) soil tilth, (6) gullies and other obstructions, (7) terrace length, (8) terrace cross section, and (9) experience and skill of the operator. Soil and crop conditions are likely to be most suitable in the spring and fall. The equipment should be adapted for efficient operation, considering slope and soil conditions. Above slopes of about 12 per cent, heavier equipment, such as bulldozers, motor patrols, and elevating graders, is desirable.

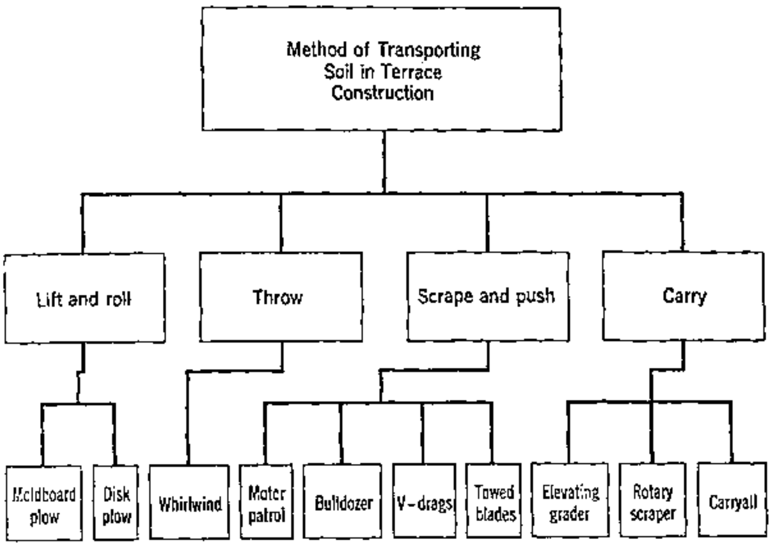

Fig. 10.4. Classification of terracing equipment based on methods of moving soil. (Redrawn from Hermsmeier.[9])

The relative rates of terrace construction based on channel capacity for terraces built in loess soils in Western Iowa[9] on slopes of 5 per cent are as follows: moldboard plow, 1.0; disk plow, 1.2; whirlwind, 2.8; motor patrol, 3.2; elevating grader, 4.0; and bulldozer, 6.4. For a given channel capacity the rate of construction decreases as the land slope increases. On 10 per cent slopes the rate of construction is reduced about 24 per cent for the whirlwind terracer, 33 per cent for the moldboard plow and motor patrol, 32 per cent for the elevating grader, 50 per cent for the disk plow, and 59 per cent for the bulldozer as compared to the rates at 5 per cent slope.[9] The actual rate of

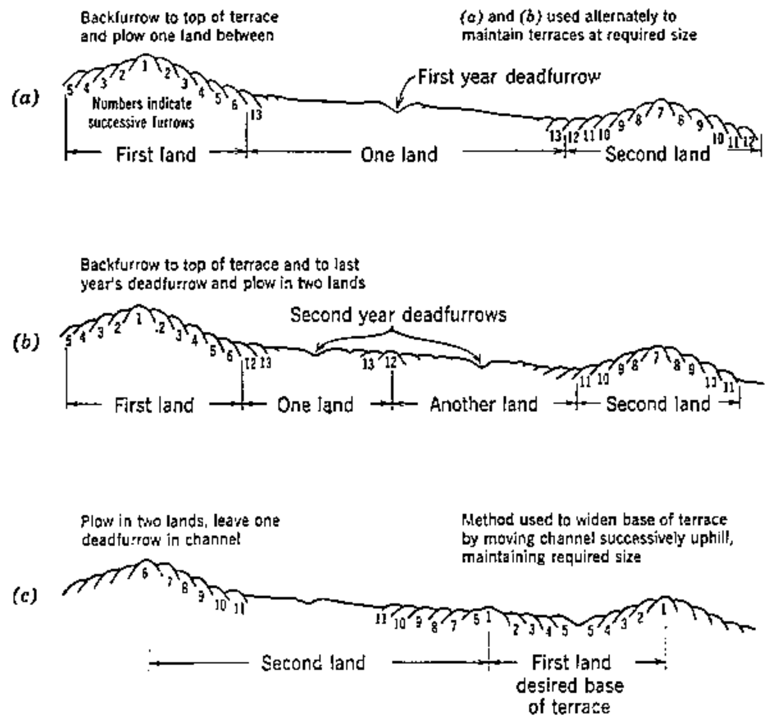

Fig. 10.5. Methods of maintaining terraces by plowing. (Redrawn from Gowder and Martin.[7])

construction for a moldboard plow may vary from about 100 to 250 feet per hour.

10.17. Settlement of Terrace Ridges. The amount of settlement in a newly constructed terrace ridge depends largely on soil and moisture conditions, type of equipment, construction procedure, and amount of vegetation or crop residue. The percentage of settlement based on unsettled height will vary as follows: (1) moldboard plow or bulldozer, 10 to 20 per cent; (2) elevating grader or whirlwind, 15 to 25 per cent; and (3) blade grader (motor patrol) 0 to 5 per cent.[9,14] These data are applicable for soils in good tillable condition with little or no vegetation or residue and for normal construction procedure. In general, those machines that move over the loose fill during construction provide greater compaction than those that throw or carry the soil onto the ridge, such as the whirlwind and the elevating grader.

TERRACE MAINTENANCE

Proper maintenance is as important as the original construction of the terrace. However, it need not be expensive since normal farming operations will usually suffice. The terrace should be watched more carefully during the first year after construction.

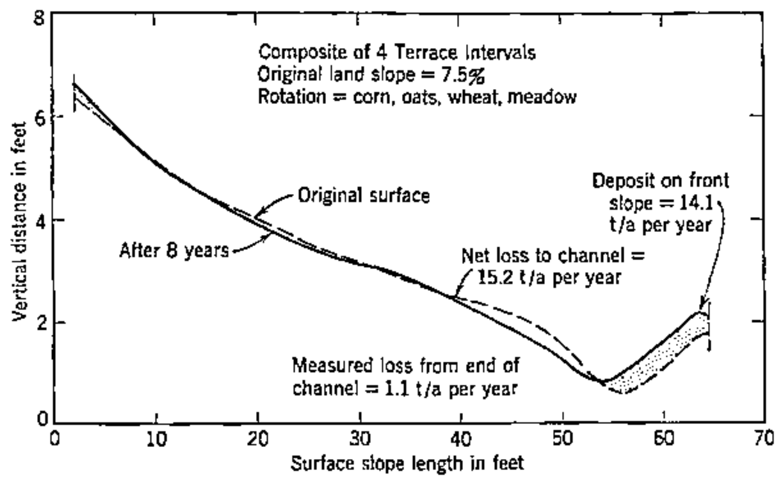

Fig. 10.6. Effect of one-way plowing on soil movement within the profile of terraced lands at Bethany, Missouri. (Redrawn from Zingg.[19])

10.18. Tillage Practices. In a terraced field all farming operations should be carried out as nearly parallel to the terrace as possible. The most evident effect of tillage operations, after several years, is the increase in the base width of the terrace. Three methods of plowing terraced fields are shown in Fig. 10.5. Procedure for plowing out point rows is similar to that for contoured areas given in Chapter 8. Other tillage practices such as stubble-mulch operations, disking, and harrowing as well as listing and planting can be performed parallel to the plow furrows.

The effect of one-way plowing, in which the furrow slice is moved up the slope and the deadfurrow placed in the channel,

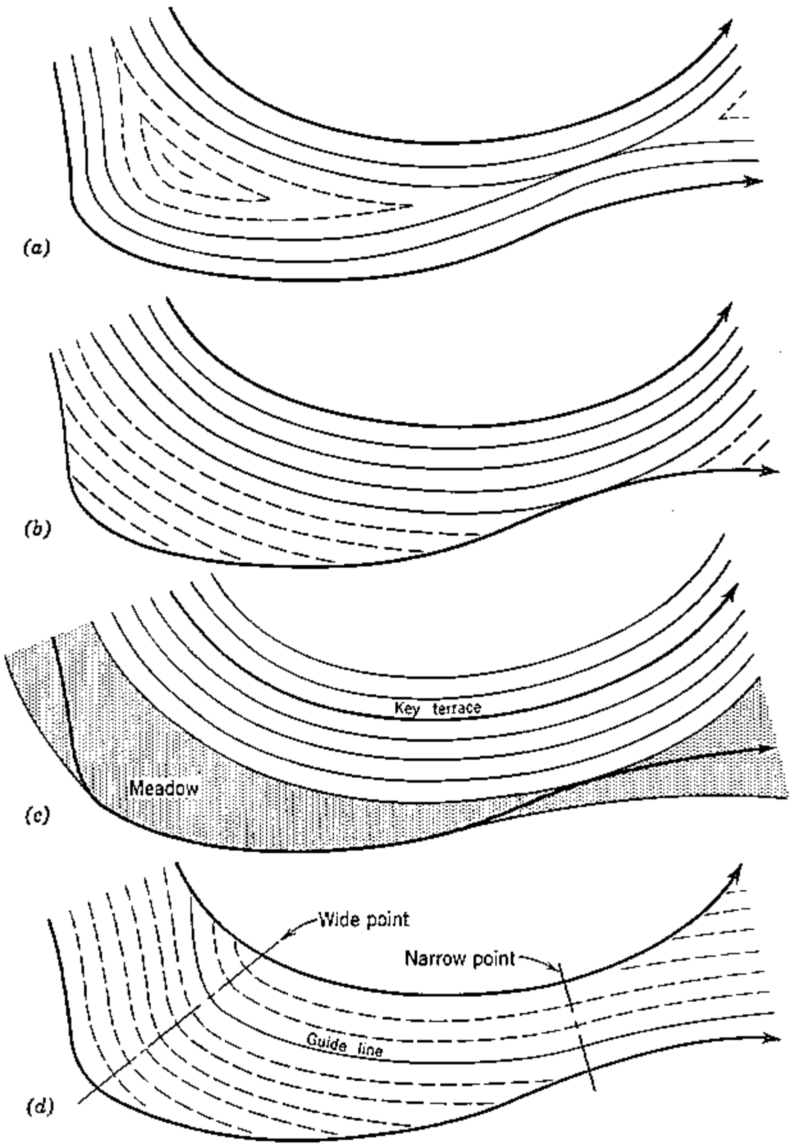

Fig. 10.7. Methods of row layout for terraced areas. (Redrawn from Doggett and Copley,[9] and Lehmann and Hay.[10])

is shown in Fig. 10.6. Except for a soil loss of about 7.4 per cent from the channel, the soil has been transferred from the interterraced area to the upper slope of the ridge below.

10.19. Row Layout. Row layout for intertilled crops may be parallel to each terrace, parallel to one or more terraces, or parallel to either of two adjacent terraces as shown in Fig. 10.7. In (*a*) the rows are laid out along the terrace above and below the ridge with the point rows midway between terraces; in (*b*) point rows are just above the terrace channel; in (*c*) point rows are left in small grain or meadow, and intertilled rows are parallel to alternate terraces; and in (*d*) rows are established by the so-called string method.[5,6] This name refers to the use of a string in laying out the guide row. In field layout the direction of travel is in the direction of flow. The principle of the string method is that rows run parallel to the lower terrace where the interval is widening and parallel to the upper terrace where the interval is narrowing. This method, developed in North Carolina, assures adequate drainage for the rows. On irregular topography slope changes cause a corresponding change in row grade which increases as the distance from the base terrace, regardless of the slope in the channel. Methods (*a*) and (*c*) are suitable for maintenance of the terraces and provide least interference in tillage and harvesting. In methods (*b*) and (*d*) turns at the ends of the point rows are made in the channel. Buffer strips shown in method (*c*) eliminate point rows, but their use will depend on the size of the area lost to row crops, number of point rows, and the advantage of the buffer strip crop.

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PROBLEMS

10.1. On one graph plot two curves with the slope in per cent (0 to 10 per cent) as the abscissa, and the vertical interval and the horizontal spacing for graded terraces as ordinates. Follow recommendations specified for your area.

10.2. Determine the time required for a flow of 8 cfs to travel a distance of 100 feet in a terrace channel having a slope of 0.3 per cent and a cross section with side slopes recommended for graded terraces on a 7 per cent slope.

10.3. Determine rod readings for the first four 50-foot stations for a uniform-graded terrace having a slope of 0.3 per cent if the rod reading at the outlet is 5.6 feet. If the stake line is 0.5 foot higher than the finished terrace channel, what should the rod readings be at these four stations?

10.4. In staking a 400-foot uniform-graded terrace having a grade of 0.4 per cent, what should be the rod readings at each 50-foot station if the rod reading at the outlet is 4.8 feet and the bottom of the finished channel will be at the same elevation as the stake line? If the V.I. for the next lower terrace is 4.5 feet, what would normally be the rod reading at the outlet of this second terrace?

10.5. Design a 900-foot terrace for a constant velocity of 2 fps, assuming that 3 cfs of runoff enters the channel at the upper end of each 300-foot section and n is 0.04. Use a triangular-shaped cross section with side slopes of 5:1.

CHAPTER 11

Gully Control

The underlying causes of gully formation have been discussed in Chapter 6. Gully formation, of course, may be prevented by removal of these causative factors. However, the engineer frequently is confronted with the problems of controlling further development of established gullies and of reclaiming seriously gullied areas. He should remember that a gully is often a symptom of improper land use and that adequate and economical control and reclamation of gullies can be accomplished only when the basic cause of gullying has been removed. The basic approach to gully control involves (1) reduction of peak flow rates through the gully and (2) provision of a stable channel for the flow that must be handled.

11.1. Methods of Gully Stabilization. Stabilization of gullies is accomplished best by providing a vegetal protection for the gully channel and modifying the cross section and grade of the channel to limit flow velocities to a level that the vegetation can withstand. The problem of limiting velocities will be simplified greatly if the flow through the gully is reduced by application of conservation measures to the contributary watershed. Application of terraces or diversions may remove completely the flow from some gullies. Small- and medium-sized gullies often may be completely controlled by transforming them into vegetated watercourses. Modification of the gradient in a gully channel usually requires application of permanent structures requiring careful engineering design.

11.2. Planning Gully Control. Control of gullies may be an expensive operation. The costs of controlling of gully must be balanced or exceeded by the benefits accrued. Benefits of gully control may include protection of land values in areas threatened by further development of the gully and protection of buildings, roads, and fences. More difficult to evaluate, but of real economic importance, is the prevention of dissection of fields into small areas with accompanying loss of efficiency in the operation of field machinery. Gully development also may present a safety hazard to machine operation.

Off-site benefits also deserve consideration. Gullying may be the primary source of sediment which clogs downstream channels, thus contributing to flood hazards and drainage problems on both rural and urban lands.

The justifiable expenditure for controlling a gully and the type of control to be used will depend upon the use that can best be made of the gullied land after reclamation, the protection afforded to adjacent areas both up and downstream by the reclamation measures, and the social and economic impact of the reclamation upon the community.

Gullies that have advanced to the stage of healing or natural stabilization warrant little expenditure for reclamation, as the damages from further development will be negligible. On the other hand, gullies that are in one of the first two stages of development (see Chapter 6) may be controlled profitably because of the large potential for further damage that they possess. Badly gullied fields that cannot be restored to usefulness as cropland except at prohibitive expense may be best returned to woodland or permanent pasture. Where gully damage has been moderate and the land is potentially productive, restoration of gullied areas to cropland may be profitable in spite of relatively high costs.

VEGETATION

Stabilization of small gullies by vegetation has been discussed in Chapter 9. Larger gullies may be controlled by plantings of trees or vines or by creating an environment conducive to reestablishment of natural vegetation.

11.3. Natural Revegetation. If the runoff that has caused the gully is diverted, and livestock is fenced from the gullied area, plants will begin to come in naturally. A gradual succession of plant species eventually will protect the gullied area with grasses, vines, shrubs, or trees native to the area in question. In some gullied areas, the development of vegetation may be stimulated by fertilizing and by spreading a mulch to conserve moisture and protect young volunteer plants. Vertical gully banks may be roughly sloped to prevent caving and provide improved conditions for natural seeding. The opportunity to provide protective cover by natural revegetation frequently is overlooked, and unnecessary expenditures are made for structures and plantings.

11.4. Artificial Revegetation. Selection of vegetation to be established artificially in a reclaimed gully should be governed by the use intended for the planted area. Grasses and legumes may be planted if the vegetation is to be used for a hay or pasture crop. Where gullies are reclaimed as drainageways in cultivated fields, sod-forming vegetation should be selected to permit crossing of the drainageway with farm machines.

In some areas trees and shrubs are easier to establish in gullies than are grasses, particularly if the gully is not to be shaped to permit operation of farm implements. Locally adapted trees planted on gullied areas may be utilized for fence posts or rough timber. Shrubs, such as dogwoods and lespedezas, are desirable for establishing the gullied area as a wildlife refuge. Trees and shrubs should be planted in accordance with local recommendations for control of erosion and establishment of wildlife refuges.

11.5. Sloping Gully Banks. Bank sloping should be done only to the extent required for establishment of vegetation or for facilitating tillage operations. Where trees and shrubs are to be established, rough sloping of the banks to about 1:1 should be sufficient. Where gullies are to be reclaimed as grassed waterways, sloping of banks to 4:1 or flatter usually is desired.

Sloping banks of small gullies may be accomplished with plows, disks, and other farm tools. Large gullies must be shaped with heavy earth-moving equipment.

DIVERSIONS

Most effective control of gullies is by complete elimination of runoff into the gully or the gullied area. This may often be accomplished by diverting runoff from above the gully and causing it to flow in a controlled manner to some suitably protected outlet.

11.6. Design of Diversions. A diversion is a channel constructed around the slope and given a slight gradient to cause water to flow to the desired outlet. The capacity of diversion channels should be based upon estimates of peak runoff for the 10-year recurrence interval. Bottom widths and side slopes may vary with soil, land slope, and individual preference, but side slopes of 4:1 and the bottom widths sufficient to permit mowing are desired. Diversions should be designated in accordance with the principles set forth for vegetated waterways. The diversion should be located far enough above the gully overfall so that sloughing of the gully head will not threaten the diversion. Water may be discharged into pastures, woodlands, or vegetated outlets. Construction and maintenance of diversions is similar to that described for grassed waterways.

STRUCTURES

Provision of a stable channel for runoff, one of the fundamental steps in gully control, frequently involves reducing the gradient of the channel to maintain velocities below an erosive level. Gully control structures perform this function. Much of the fall in the gully being treated is taken up at the structures which are designed to dissipate the energy of the falling water. The gradient of the channel reaches between structures should maintain nonsilting and nonscouring velocities.

11.7. Temporary and Permanent Structures. *Temporary Structures.* Temporary structures can only be recommended in situations where cheap labor and materials can be utilized. Increasing mechanization and high labor costs have resulted in a great decline in the popularity of temporary gully control structures. In general, shaping the gully and establishing vegetation in accordance with the principles discussed in Chapter 9 and in the present chapter provide more efficient and effective control.

In Smith's report[10] on the performance of 50 temporary structures which had been used on the Soil Conservation Service experimental farm at Bethany, Missouri, only 5 per cent of the structures were found to have functioned as intended. It was concluded that vegetal protection was established as easily without temporary structures as with them.

Temporary structures are constructed of creosoted planks, rock, logs, brush, woven wire, sod, or earth.[5] These structures should be designed in accordance with the broadcrested weir formula. Design for a 10-year recurrence interval is recommended.

Permanent Structures. Structures constructed of permanent

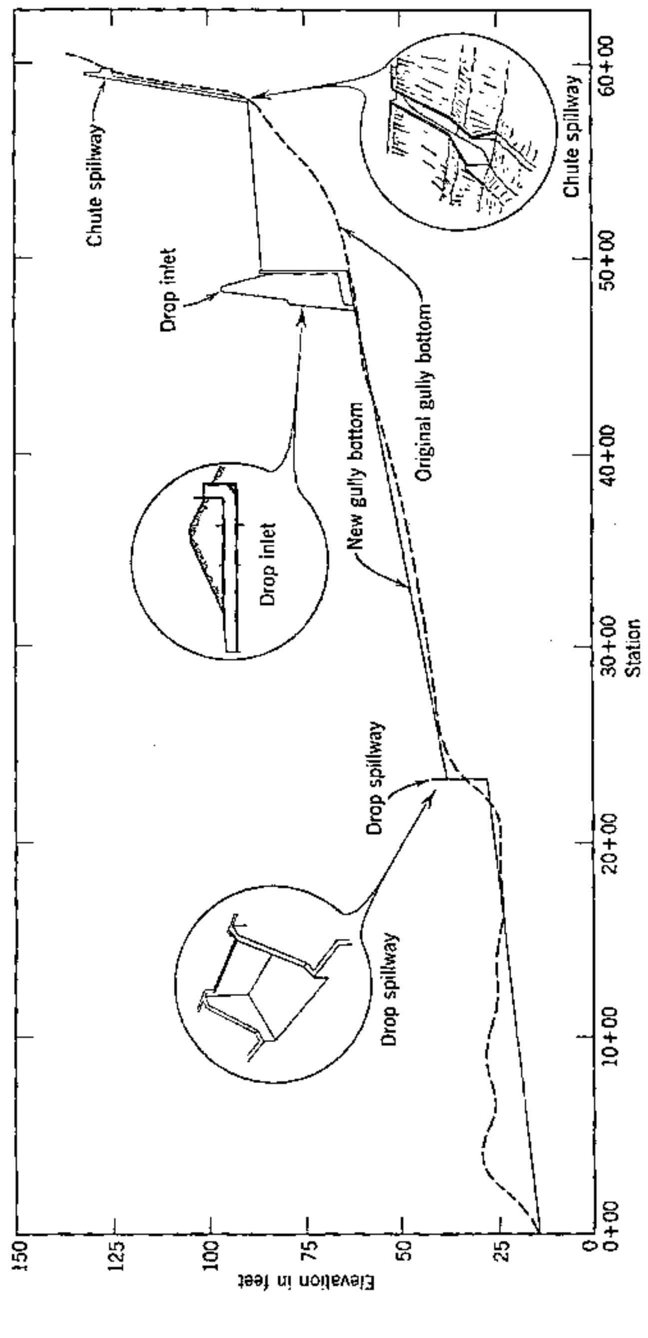


Fig. 11.1. Profile of a gully showing the application of several types of permanent structures.

materials may be required to control the overfall at the head of a large gully, to drop the discharge from a vegetated waterway into a drainage ditch, to take up the fall at various points in a gully channel, or to provide for discharge through earth fills. Figure 11.1 shows the profile of a gully which has been reclaimed by methods involving the use of several types of permanent structures. Standard designs are available from references of the Soil Conservation Service and the Bureau of Reclamation.

11.8. Functional Requirements of Control Structures. Not only must a gully control structure have sufficient capacity to pass the design discharge, but the kinetic energy of the discharge must be dissipated within the confines of the structure in a manner and to a degree that will protect both the structure and the downstream channel from damage. The two primary causes of failure of permanent gully control structures are (1) insufficient hydraulic capacity and (2) insufficient provision for energy dissipation.

11.9. Design Features. The basic components of a hydraulic structure are the inlet, the conduit, and the outlet. Structures are classified and named in accordance with the form that these three features take. Figure 11.2 identifies the various types of inlets, conduits, and outlets that are commonly used. In addition to these hydraulic features, the structure must include suitable wing walls, side walls, head wall extensions, and toe walls to prevent seepage under or around the structure and to prevent damage from such local erosion as may occur. These structural components are identified in Fig. 11.4 for one common type of structure. It is important that a firm foundation be secured for permanent structures. Wet foundations should be avoided or provided with adequate artificial drainage. Surface soil and organic material should be removed from the site to allow a good bond between the structure and the foundation material.

Models. The design criteria for gully control structures have been developed from intensive observation of the behavior of small-scale laboratory models. The results of such laboratory studies have been summarized in empirical formulas, graphs, or tables which relate certain critical dimensions of the structure to characteristics of the flow.

Model studies of open channel structures are based on the

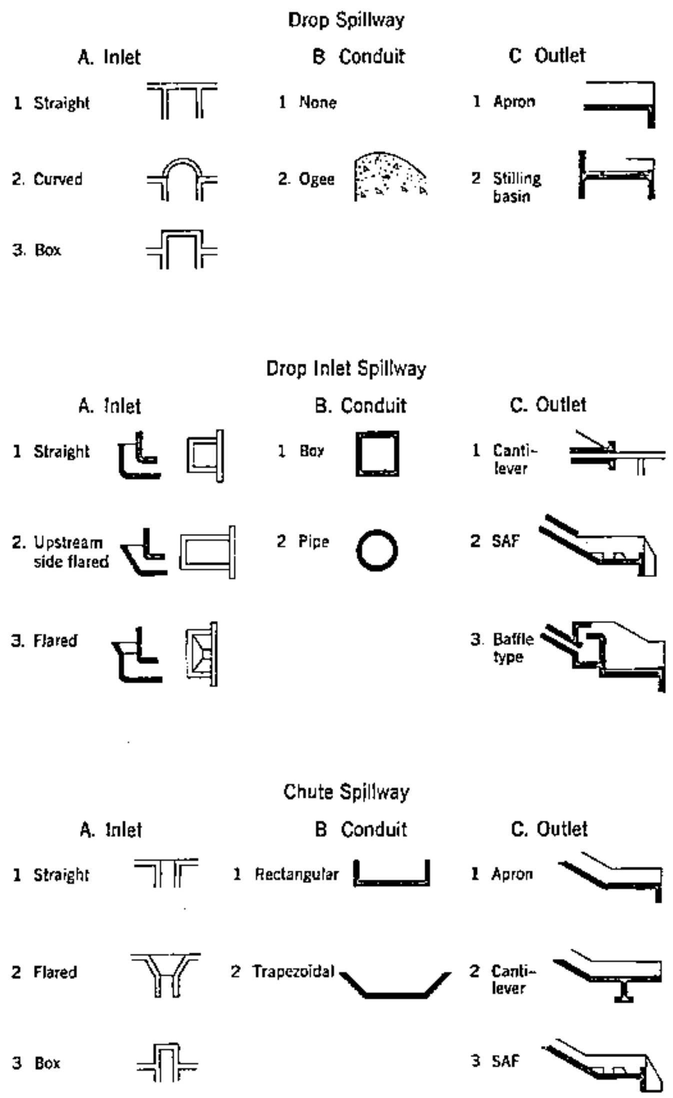


Fig. 11.2. Classification of components of hydraulic structures. (Modified from Soil Conservation Service.[12])

Froude law which requires that the Froude number of flow through the model must be equal to the Froude number of flow through the prototype. Application of this law assumes that the force of gravity is the only force producing motion. Other forces, such as fluid friction and surface tension are neglected. The Froude number is defined by the American Society of Civil Engineers, Committee of the Hydraulics Division on Hydraulic Research,[1] as

$$F = \frac{v^2}{gd}$$

Most control structures include a section at which flow at critical depth (explained below) occurs. The Froude law will

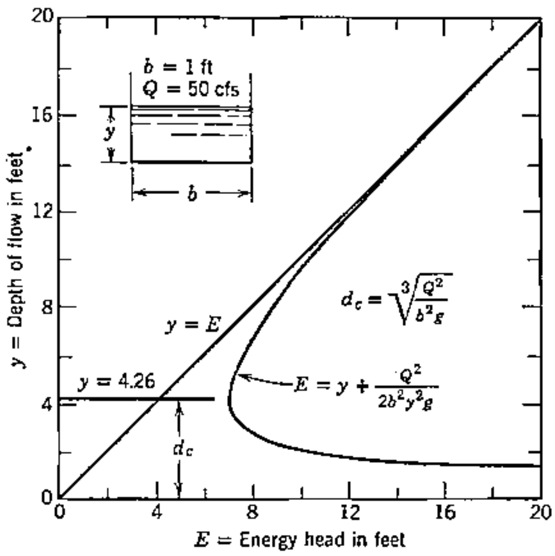


Fig. 11.3. Energy head for constant discharge at varying depths.

be satisfied if the critical depth of flow at such a section in the prototype (the full-scale structure) is equal to the scale factor times the critical depth of flow at that section in the model. Thus design equations for certain components of structures are often expressed as functions of critical depth.

A given quantity of water in an open conduit may flow at two depths having the same energy head. When these depths coincide, the energy head is a minimum and the corresponding depth is termed the critical depth. This is illustrated by Fig. 11.3. The expression for critical depth at a rectangular section may be developed as follows. The specific energy head at a section with reference to the channel bed is

$$E = y + \frac{v^2}{2g} = y + \frac{Q^2}{2a^2g} = y + \frac{Q^2}{2b^2y^2g} \tag{11.1a}$$

Differentiating with respect to y,

$$\frac{dE}{dy} = 1 - \frac{Q^2}{b^2y^3g} \tag{11.1b}$$

Setting $dE/dy = 0$ to determine y when E is a minimum, and letting this value of y be d_c,

$$1 - \frac{Q^2}{b^2y^3g} = 0$$

$$d_c = \sqrt[3]{\frac{Q^2}{b^2g}} \tag{11.2}$$

In the operation of models the flow rate and the various dimensions of the model are varied, and the operation of the model is observed. The influence of the variables on erosion of the downstream channel is checked by observation of the scour pattern produced in a sand bed at the outlet section of the model.

Additional information on models may be found in *Similitude in Engineering* by Murphy[9] and in the ASCE Manual of Engineering Practice No. 25, *Hydraulic Models.*[1]

DROP SPILLWAYS

11.10. Types. Two types of drop spillways are shown in Figs. 11.4 and 11.5. Drop spillways may have a straight, arched, or box-type inlet. The energy dissipator may be a straight apron or some type of stilling basin.

11.11. Function and Limitations. Drop spillways are installed in gullies to establish permanent control elevations below which an eroding stream cannot lower the channel floor. These structures control the stream grade not only at the spillway crest itself but also through the ponded reach upstream. Drop structures placed at intervals along a channel can stabilize it by changing its profile from a continuous steep gradient to a series

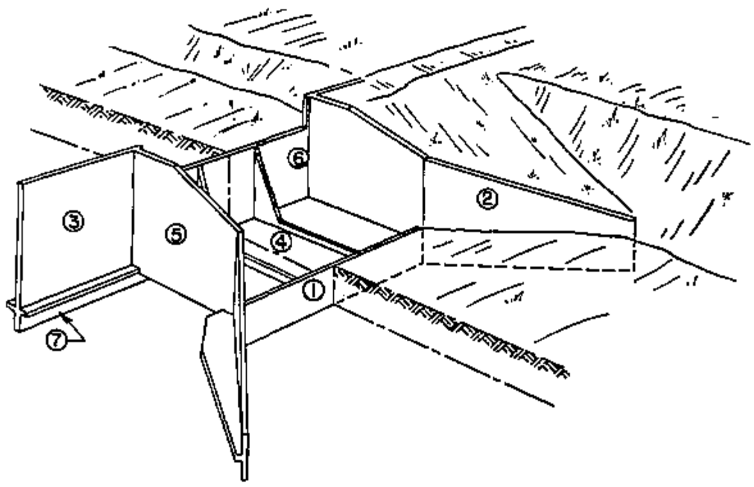


Fig. 11.4. Drop spillway showing structural components.

1. Toe wall
2. Wing wall
3. Head wall extension
4. Apron
5. Side wall
6. Head wall
7. Cutoff wall

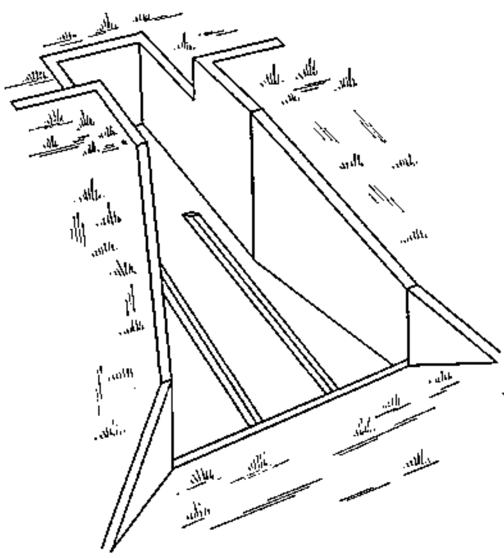


Fig. 11.5. Box-inlet drop spillway.

of more gently sloping reaches. Where relatively large volumes of water must flow through a narrow structure at low head, the box-type inlet is preferred. The curved inlet serves a similar purpose and also gives the advantage of arch strength where masonry construction is used. Drop spillways are usually limited to drops of 10 feet, flumes or drop-inlet pipe spillways being used for greater drops.

11.12. Design Features. *Capacity.* The free flow capacity for drop spillways is given by the weir formula:

$$Q = CLh^{3/2} \tag{11.3}$$

The length L is the sum of the lengths of the three sides of a box inlet, the circumference of an arch inlet, or the crest length of a straight inlet. The value of C varies considerably with entrance conditions. Blaisdell and Donnelly[4] have prepared correction charts to modify C for a wide range of conditions of entrance and crest geometry for box inlets. Where the ratio of head to box width is 0.2 or greater, the ratio of the width of the approach channel to the total length L is greater than 1.5, and no dikes or other obstacles are within $3h$ of the crest, a value of $C = 3.2$ may be used with an accuracy of ± 20 per cent. A value of $C = 3.2$ will also give satisfactory results for the straight inlet. The inlet should have a freeboard of 0.5 foot above h.

Whenever the tailwater is nearly up to or above the crest of the inlet section, submergence decreases the capacity of the structure. When such conditions occur in field design, special reference should be made to Blaisdell and Donnelly,[4] King,[6] or other information, on the performance of submerged weirs.

Apron Protection. The kinetic energy gained by the water as it falls from the crest must be dissipated and/or converted to potential energy before the flow is discharged from the structure. For straight-inlet drop structures the dissipation and conversion of energy are accomplished in either a straight apron or a Morris and Johnson stilling basin. Dimensions of the straight apron are given in Fig. 11.6. Dimensions for the Morris and Johnson stilling basin are given in Fig. 11.7. For larger structures the Morris and Johnson outlet is preferred, as it results in a shorter apron and the transverse sill induces a hydraulic jump at the toe of the structure. The longitudinal sills serve to straighten the flow and prevent transverse components of velocity from eroding the side slopes of the downstream channel. The flow

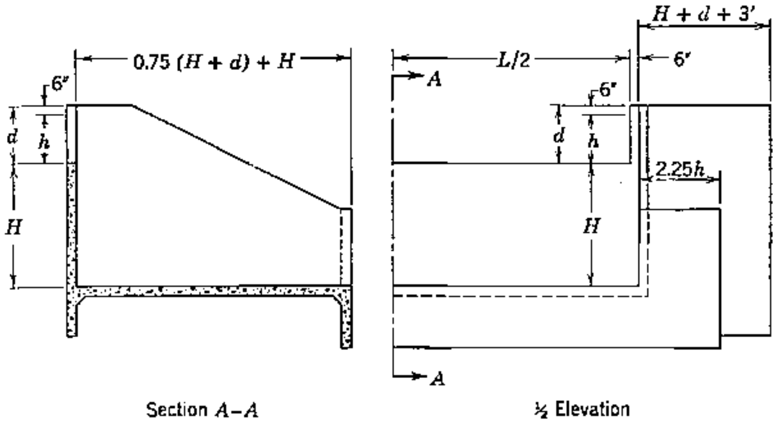


Fig. 11.6. Design dimensions for drop spillway with straight inlet and straight-apron outlet. (Redrawn from reference 11.)

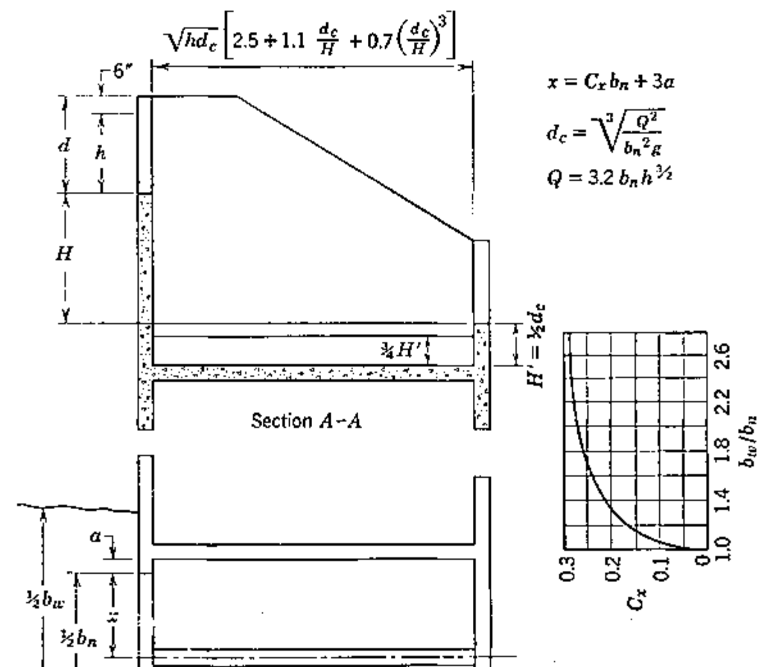


Fig. 11.7. Design dimensions for drop spillway with straight inlet and Morris and Johnson outlet. (Redrawn from Morris and Johnson,[8] p. 21, Fig. 3.)

pattern through a Morris and Johnson stilling basin is shown in dimensionless form in Fig. 11.8.

The stilling basin design for the box-inlet drop spillway is given in Fig. 11.9.

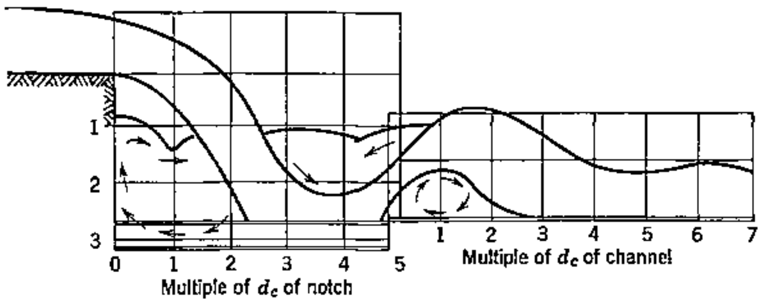


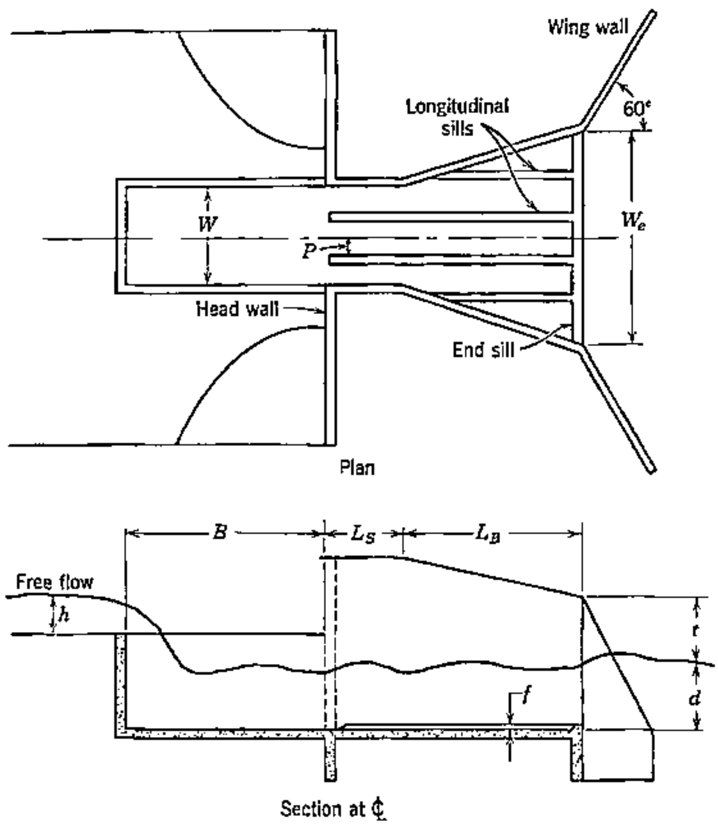
Fig. 11.8. Flow pattern through a drop spillway with a Morris and Johnson stilling basin. (Redrawn from Morris and Johnson,[8] p. 35, Fig. 11*b*.)

CHUTES

Flumes or chutes carry flow down steep slopes through a concrete-lined channel rather than by dropping the water in a free overfall.

11.13. Function and Limitations. Chutes may be used for the control of heads up to 16 or 20 feet. They usually require less concrete than do drop-inlet structures of the same capacity and drop. However, there is considerable danger of undermining of the structure by rodents, and, in poorly drained locations, seepage may threaten foundations. Where there is no opportunity to provide temporary storage above the structure, the flume with its inherent high capacity is preferred over the drop-inlet pipe spillway. The capacity of a chute is not decreased by sedimentation at the outlet.

11.14. Design Features. *Capacity.* Flume capacity normally is controlled by the inlet section. Inlets may be similar to those for straight-inlet or box-inlet drop spillways, and in such inlets capacity formulas already discussed will apply. Blaisdell and Huff[3] have investigated the performance of other types of flume entrances. Two of them are shown in Fig. 11.10.



(Design formulas)

Critical depth in straight section, $d_c = \sqrt[3]{\dfrac{Q^2}{W^2 g}}$

Critical depth at exit of basin, $d_{ce} = \sqrt[3]{\dfrac{Q^2}{W_e^2 g}}$

Minimum $L_S = d_c\left(\dfrac{0.2}{B/W} + 1\right)$

Minimum $L_B = \dfrac{W + 2B}{2B/W}$

Minimum $d = 1.6 d_{ce}$ with $W_e < 11.5 d_{ce}$

$f = d/6$ $\quad P = W/6$ to $W/4$

When $W_e > 2.5W$, locate 2 additional sills midway between center sills and side walls at end sill

Minimum $t = d/3$

Fig. 11.9. Design dimensions for the box-inlet drop spillway. (Redrawn from Blaisdell and Donnelly.[4])

The capacity of the Wisconsin-type flume entrance is given by

$$Q = 3.50L^{0.94}h^{1.56} \tag{11.4}$$

and capacity of the 2:1 flume entrance is given by

$$Q = 3.75L^{0.9}h^{1.6} \tag{11.5}$$

Outlet Protection. The cantilever-type outlet should be used where the channel grade below the structure is unstable. In other situations, either the straight-apron or SAF outlet is used.

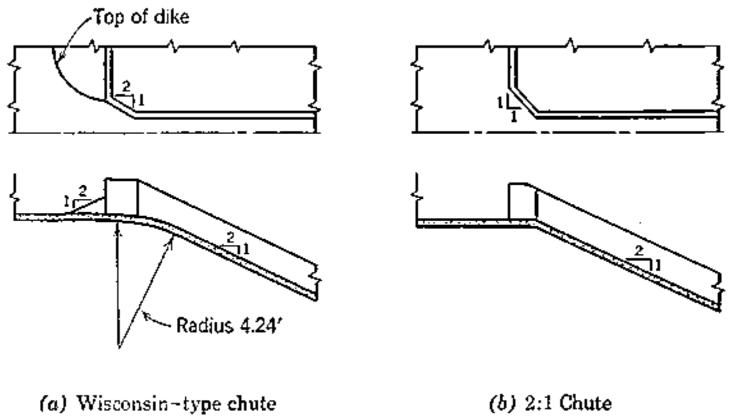
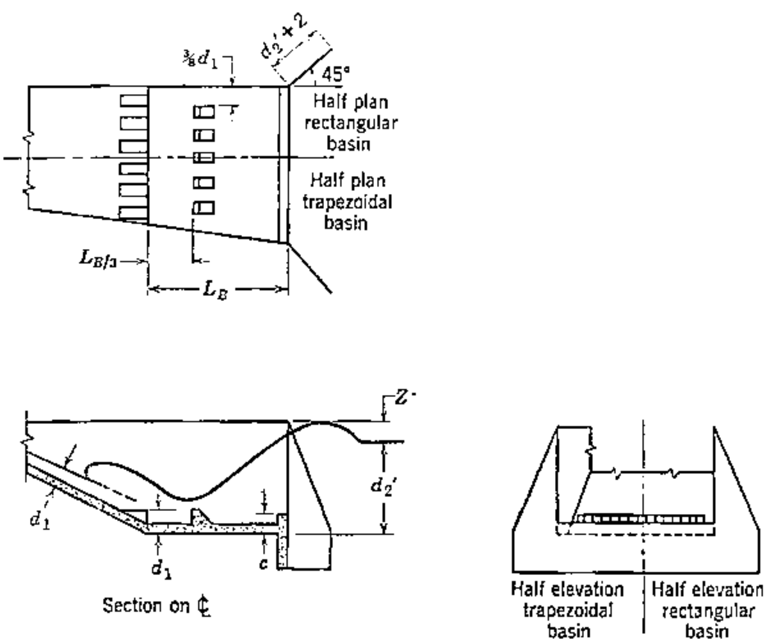


Fig. 11.10. Two types of flume entrances. (Redrawn from Blaisdell and Huff.[3])

The straight apron is applicable to small structures. Design of the SAF stilling basin has been developed by Blaisdell.[2] Figure 11.11 shows dimensions of this type of outlet protection.

MISSOURI-TYPE FLUME

11.15. Function and Limitations. Sometimes referred to as the formless flume, the Missouri-type flume has the advantage of low cost construction. It may be used to replace drop spillways where the fall does not exceed 7 feet and the width of notch required does not exceed 8 feet. The flume is constructed by shaping the soil to conform to the shape of the flume and applying a 5-inch layer of concrete reinforced with woven wire mesh. No forms are needed; thus the construction is simple and inexpensive. The Missouri-type flume should not be used where water is impounded upstream because of the danger of undermining the structure by seepage, or where frost is a problem.



(Design formulas)

$$L_B = \frac{4.5d_2}{F^{0.38}}, \text{ for } 3 < F < 300 \qquad F = \frac{v_1^2}{gd_1}$$

Floor blocks to occupy 40 to 55 percent of stilling basin width.

$$c = 0.07d_2, \qquad z = d_2/3$$

$$d_2 = (-1 + \sqrt{8F + 1})\, d_1/2$$

$$d'_2 = 1.4F^{0.45}d_1$$

d_2 = theoretical tailwater depth

d'_2 = actual tailwater depth

Fig. 11.11. The Saint Anthony Falls (SAF) stilling basin. (Redrawn from Blaisdell,[2] p. 485, Fig. 1.)

11.16. Design Features. Figure 11.12 shows the design features and dimensions of the Missouri-type flume. The capacity is given by

$$Q = 3.85Lh^{3/2} \tag{11.6}$$

The depth of the notch is h plus a freeboard of 3 to 6 inches.

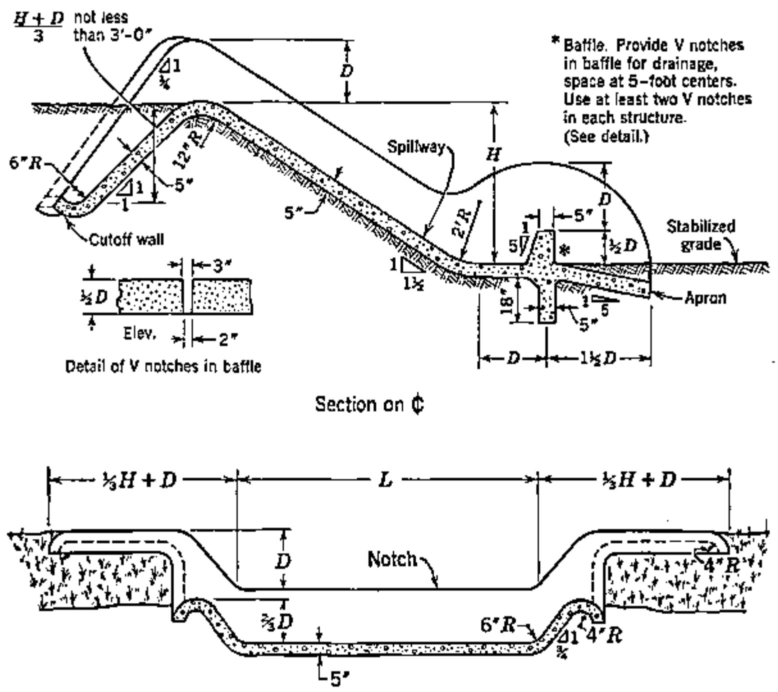
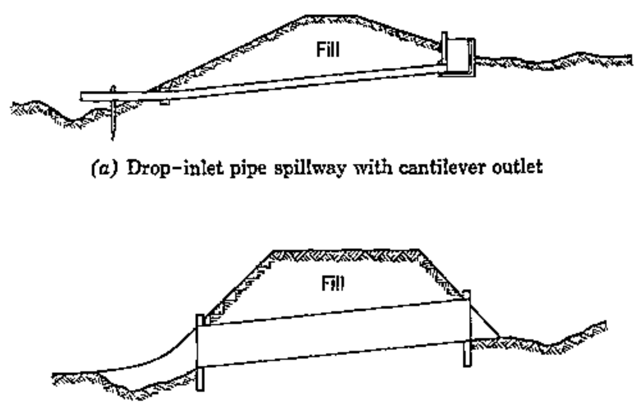


Fig. 11.12. The Missouri-type flume. (Based on design by Wooley and others.[13])

PIPE SPILLWAYS

11.17. Types. Pipe spillways may take the form of a simple conduit under a fill, or they may have a riser on the inlet end and some type of special outlet protection. These conditions are illustrated by Fig. 11.13. Pipes may be round, square, rectangular, or arch in cross section.

11.18. Function and Limitations. The pipe spillway used as a culvert has the simple function of providing for passage of water under an embankment. When combined with a riser or drop inlet, the pipe spillway serves to lower water through considerable drop in elevation and to dissipate the energy of the falling water. Drop-inlet pipe spillways are thus frequently used as gully control structures. This application is usually made where water may pond behind the inlet to provide temporary storage. The hydraulic capacity of pipe spillways is related to the square root of the head, and hence they are relatively low-capacity structures. This characteristic is used to advantage where discharge from the structure is to be restricted.



(a) Drop-inlet pipe spillway with cantilever outlet

(b) Simple culvert

Fig. 11.13. Pipe spillways.

11.19. Design Features. *Culverts.* Culvert capacity may be controlled either by the inlet section or by the conduit. The headwater elevation may be above or below the top of the inlet section. The several possible flow conditions are represented in Fig. 11.14. Solution of a culvert problem is primarily the determination of the type of flow that will occur under given headwater and tailwater conditions. Consider a culvert as shown in Fig. 11.14 *a* and *b*. Pipe flow (conduit controlling capacity) will occur when the slope of the culvert is less than the neutral slope, s_n:

$$s_n = \tan\theta = \frac{K_c \dfrac{v^2}{2g}}{\sqrt{1 - \left(K_c \dfrac{v^2}{2g}\right)^2}} \tag{11.7a}$$

and, when the conduit is on neutral slope,

$$\sin\theta = \frac{H_f}{L} = K_c \frac{v^2}{2g} \tag{11.7b}$$

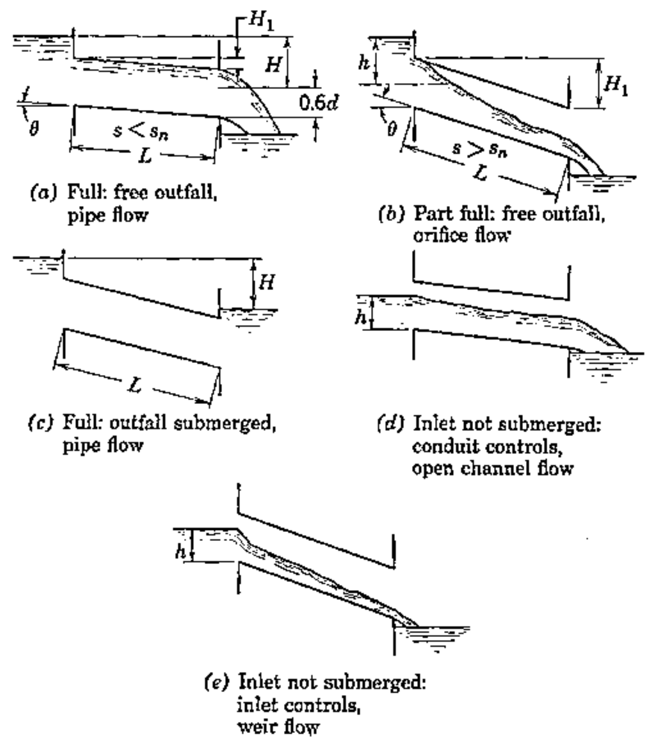


Fig. 11.14. Possible conditions of flow through culverts. (Modified from Mavis.[7])

The capacity of a culvert under conditions of pipe flow is given by

$$Q = \frac{a\sqrt{2gH}}{\sqrt{1 + K_e + K_cL}} \tag{11.8}$$

Values of K_c for circular and square culverts are given in Appendix D. Values of the roughness coefficient, n, for conduits may be found in Appendix C.

If the conduit is at greater than neutral slope and the outlet is not submerged, the flow will be controlled by the inlet section, and orifice flow prevails. Capacity is then given by

$$Q = aC\sqrt{2gh} \tag{11.9}$$

The coefficient, C, for a sharp edge orifice is 0.6. For more detailed values and for other orifices, consult King[6] or other hydraulic books. Examples 11.1 and 11.2 serve to clarify the above discussions.

Example 11.1. Determine the capacity of a 30-inch-diameter corrugated culvert 60 feet long. The culvert entrance is square-edged. Elevation of the inlet invert is 419.7 feet, and the elevation of the outlet invert is 419.0 feet. Headwater elevation is 425.0 feet, and tailwater elevation is 416.0 feet.

Solution. Assume pipe flow prevails, and apply equation 11.8. From Appendix D, find $K_e = 0.5$, $K_c = 0.0341$. Head loss, H, through the structure is from the headwater elevation to an elevation of $0.6d$ above the elevation of the outlet invert.

$$Q = \frac{4.91\sqrt{2(32.2)(4.5)}}{\sqrt{1 + 0.5 + 0.0341(60)}} = 44.4 \text{ cfs}$$

$$v = \frac{Q}{a} = \frac{44.4}{4.91} = 9.05 \text{ fps}$$

Determine the normal slope of the culvert for a discharge of 44.4 cfs. Apply equation 11.7a.

$$s_n = \frac{0.0341\dfrac{(9.05)^2}{2(32.2)}}{\sqrt{1 - \left(0.0341\dfrac{(9.05)^2}{2(32.2)}\right)^2}} = 0.0433$$

Actual slope of the culvert is 0.0117. Since the culvert is at less than normal slope, pipe flow prevails and 44.4 cfs is the discharge.

Example 11.2. Determine the capacity of the 30-inch culvert of example 11.1 if the elevation of the outlet invert is 410.6 feet, and the tailwater elevation is 408.0 feet.

Solution. Assume pipe flow, and calculate the discharge by equation 11.8.

$$Q = \frac{4.91\sqrt{2(32.2)(12.9)}}{\sqrt{1 + 0.5 + 0.0341(60)}} = 75.4 \text{ cfs}$$

$$v = \frac{75.4}{4.91} = 15.3 \text{ fps}$$

The normal slope is found to be 0.125. Actual slope of the culvert is 0.153. Since the culvert is at greater than normal slope, pipe flow will not exist. Entrance conditions will prevail, and the problem is solved by application of the orifice flow formula, equation 11.9.

$$Q = 4.91(0.6)\sqrt{64.4(4.05)} = 47.6 \text{ cfs}$$

An alternate solution may be made by reference to Fig. 11.15.

$$\frac{H}{D} = \frac{5.3}{2.5} = 2.12$$

From the figure, $\frac{Q}{D^{5/2}} = 4.8$

$$Q = 4.8(D^{5/2}) = 4.8(2.5)^{5/2} = 47 \text{ cfs}$$

In situations where the headwater elevation does not reach the elevation of the top of the inlet section, there is again the possibility of control of flow by either the conduit or the inlet section. In Fig. 11.14*d* and *e*, the conduit controls if the slope

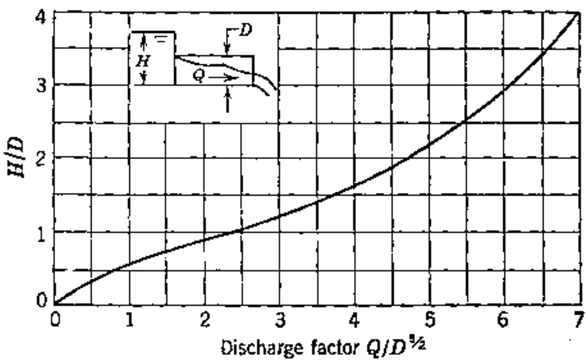


Fig. 11.15. Stage-discharge relationship for control at inlet section. Square-edged entrance to circular pipe. (Redrawn from Mavis.[7])

of the conduit is less than that required to move the possible maximum inlet flow at a depth equal to the headwater depth above the inlet invert minus the static head loss due to entrance losses and acceleration. Conditions of control at the entrance section occur when the slope of the conduit is greater than that required to move the possible flow through the inlet. For conditions of control by the entrance section, solution for circular culverts may be made from Fig. 11.15. This figure will also apply when the inlet is submerged. Examples 11.3 and 11.4 clarify the solution for conditions of an unsubmerged inlet.

Example 11.3. Determine the capacity of a 5-foot-diameter concrete culvert 100 feet long. The culvert entrance is square-edged. Elevation of the inlet invert is 517.6 feet, and the elevation of the outlet invert is 510.5 feet. Headwater elevation is 521.0 feet, and tailwater elevation is 500.0 feet.

Solution. Assume that the conduit controls and entrance conditions are not limiting. Neglect for the moment the loss of static head at the culvert entrance due to acceleration of the flow entering the culvert.

Under these assumptions, the depth of flow in the culvert would be 3.4 feet. Calculating the flow by the Manning formula, $a = 14.3$ square feet, $n = 0.015$, $R = 1.46$ feet, and $s = 0.071$. Q then equals 486 cfs. Checking in Fig. 11.15, $H/D = 0.68$ and $Q = 56$ cfs. Since only 56 cfs can enter the culvert, it is inconceivable that flow approaching 486 cfs could occur; thus entrance conditions prevail and the capacity is 56 cfs.

Example 11.4. Determine the capacity of a culvert as in example 11.3 but having the outlet invert at an elevation of 517.55 feet.

Solution. As before, we note that the maximum possible flow through the inlet is 56 cfs. However, the culvert is on a very flat slope, and we may expect conduit flow conditions to limit the flow to less than 56 cfs. Assume a flow depth in the conduit of 2.5 feet. Then $a = 9.82$ square feet, $n = 0.015$, $R = 1.25$, and $s = 0.0005$. Then by the Manning formula $v = 2.56$ fps and $Q = 24.9$ cfs. Now assume the approach velocity is negligible; then the loss of static head at the culvert entrance due to acceleration is

$$\frac{v^2}{2g} = \frac{2.56^2}{64.4} = 0.102 \text{ feet}$$

Depth of water at the entrance is 3.4 feet, and a loss of 0.102 feet would give 3.3 feet, which does not correspond with our assumption of 2.5 feet. Thus the first assumption of flow depth was in error. Now assume a flow depth in the culvert of 3.25 feet. Then $a = 13.5$ square feet, $n = 0.015$, $R = 1.44$, and $s = 0.0005$. Then by the Manning formula $v = 2.83$ fps and $v^2/2g = 0.12$ feet. Subtracting 0.12 from the entrance depth of 3.4 feet leaves a flow depth of 3.28, which is sufficiently close to 3.25. Thus flow is limited by the conduit, and the discharge is 38 cfs. For rectangular culverts, the discharge when the unsubmerged inlet controls may be calculated from the broadcrested weir formula.

$$Q = CLh^{3/2}$$

A value of $C = 2.7$ may be used, but King[6], *Handbook of Hydraulics*, should be referred to for detailed information. When the conduit controls, solution may be made by the Manning formula.

Drop Inlets. The discharge characteristics of a drop-inlet pipe spillway are given in Fig. 11.16. At low heads, the crest of the riser controls the flow, and discharge is proportional to $h^{3/2}$. Under this condition, the discharge should be calculated as outlined in Art. 11.12. When this type of flow equals the capacity of the conduit or conduit inlet section, the flow becomes proportional to the square root of the total head loss through the structure or the head on the conduit inlet.

Outlet Protection. For small culverts or drop-inlet pipe spillways, a cantilever-type outlet is usually satisfactory. The

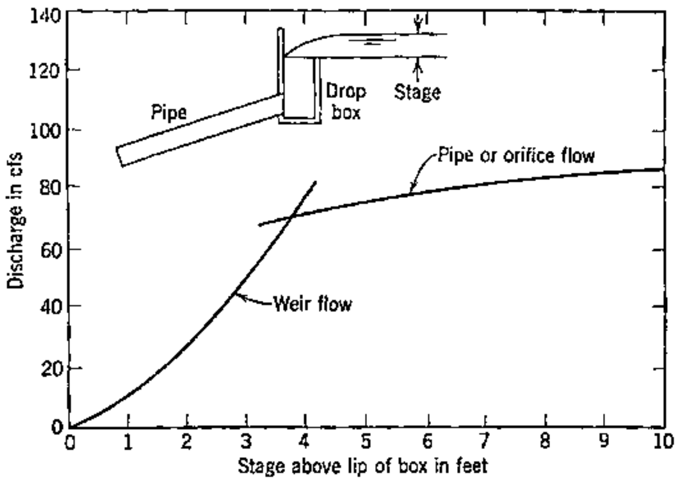


Fig. 11.16. Discharge characteristics of a drop-inlet pipe spillway.

straight-apron outlet may be used in some instances. Large drop-inlet pipe spillways may be provided with the SAF stilling basin designed as discussed in Art. 11.14.

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PROBLEMS

11.1. What is the maximum capacity of a straight-drop inlet having a crest length of 10 feet and a depth of flow of 3 feet?

11.2. Determine the design dimensions for the drop spillway in Problem 11.1, using the straight-apron outlet and the Morris and Johnson outlet if the drop in elevation is 5 feet. The waterway is 15 feet wide and dimension a (Fig. 11.7) is 6 inches.

11.3. Determine the crest length for a straight-inlet drop spillway to carry 265 cfs if the depth of flow is not to exceed 3 feet. What should be the dimensions of a square-box inlet for the same conditions?

11.4. Determine the design dimensions for a 5×5-foot box-inlet drop spillway to carry 250 cfs if the end sill is 10 feet in length.

11.5. What is the capacity of a Wisconsin-type flume 6 feet wide when the flow depth is 2 feet? Of a Missouri-type flume?

11.6. Determine the discharge of a 100-foot 3×3-foot concrete box culvert having a square entrance, $n = 0.013$, $s = 0.003$ foot/foot, and elevations of 36.0 feet at the center of the conduit at the outlet, 42.3 feet for the headwater, and 40.5 feet for the tailwater.

11.7. Tabulate and plot the head-discharge curve (up to 5-foot depth above the crest) for a 3×3-foot drop inlet attached to an 18-inch-diameter concrete pipe ($n = 0.015$) 150 feet in length. Assume pipe flow controls in the conduit; tailwater height is not higher than the center of the pipe at the outlet; radius of curvature of the pipe entrance is 0.3 foot; and difference in elevation between crest and center of pipe at outlet is 18 feet.

CHAPTER 12

Embankments and Reservoirs

In all land-use programs the availability of water for crops, livestock, and many miscellaneous purposes is of primary importance. Farm ponds and reservoirs provide a logical source of such water, for they may be designed and adjusted to fit the individual land-use plans and conditions that exist on the farm. Conservation and protection of land also depend upon the control of excess waters. Earth embankments in the form of dikes, levees, and detention dams are important protective structures.

The design of earth dams and embankments that are effective and safe requires thorough integration of the principles of soil physics and soil mechanics with sound engineering design and construction principles.

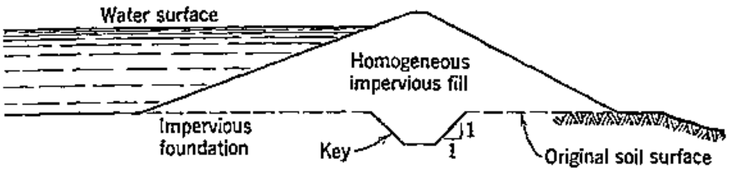
12.1. Uses of Reservoirs and Embankments. *Farm Ponds.* The farm pond is a multiple-use pond that, depending on its size and location, may furnish a supply of water for irrigation, livestock, spray water, fire protection, fish production, recreation, or any combination of these uses.[20,31] To assure that the water supply will be adequate, *all* the potential uses of the pond must be considered in the original design.

Flood Control Reservoirs. Earth dams are common for head-water flood control reservoirs because of their adaptability to a wide range of foundation conditions, their use of "on-site" construction materials, and their relatively low construction cost. Equipped with a controlled mechanical outlet, these reservoirs permit emptying of flood waters at a rate commensurate with the capacity of the stream channel below the structure.

Dikes and Levees. Earth embankments are used largely for dikes and levees for protection of land areas adjacent to streams subject to frequent flooding. They may also prevent reflooding of areas that have been pump-drained or that are subject to tidal overflow.[22]

12.2. Types of Earth Embankments. Design of embankments for water control is predicated upon (1) the nature of the foundation materials, i.e., stability, depth to impervious strata, relative permeability, and drainage conditions, and (2) the nature and availability of the construction materials.

The three major types of earth fills are: (1) the *simple embankment* type which is constructed of relatively homogeneous



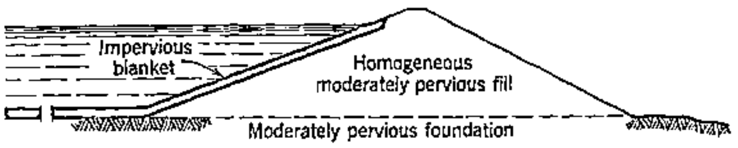


Fig. 12.1. (*a*) Simple embankment utilizing "key" construction and (*b*) simple embankment utilizing an impervious "blanket" seal.

soil material and is either keyed into an impervious foundation stratum, as shown in Fig. 12.1*a*, or is constructed with an upstream blanket of impervious material, as shown in Fig. 12.1*b*. This type is limited to low fills and to sites having sufficient volumes of satisfactory fill materials available. (2) The *core type* of design utilizing, within the dam, a central section of highly impermeable or puddled soil materials extending from above the water line to an impermeable stratum in the foundation. In some instances an upstream blanket is used in conjunction with this design. These designs, shown in Fig. 12.2, reduce the percentage of high-grade fill materials needed for construction. (3) The *diaphragm* type uses a thin wall of concrete, steel, or wood to form a barrier against seepage through the fill. A "full-diaphragm" cutoff extends from above the water line down to and sealed into an impervious foundation stratum.

The "partial-diaphragm" does not extend through this full range and is sometimes referred to as a cutoff wall.

Earth dams may be constructed by one of two methods: (1) rolled fills, in which the soil material is spread in thin, uniform layers and then compacted at optimum moisture until maximum

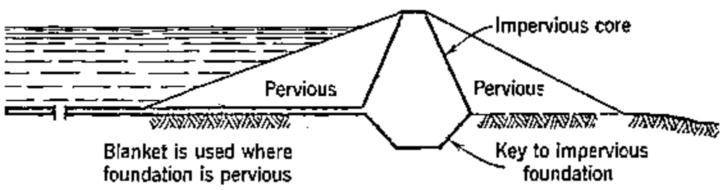


Fig. 12.2. Embankment utilizing a central core and key of impermeable materials extending from above the water line to the foundation. (Blanket is used where foundation is permeable.)

density is achieved, or (2) hydraulic or semihydraulic fills in which the fill materials are transported to the site and placed wholly or partially by hydraulic means. This latter type is used mainly on extremely large dams.[5,9]

12.3. Essential Requirements for Reservoirs. To assure an effective pond or reservoir, certain basic requirements must be met: (1) The topographic conditions at the pond site must allow economical construction; cost is a direct function of fill length or height, for these dimensions determine cubic content. (2) An adequate and reliable supply of water, free from mine, organic, or chemical pollution must be available. (3) Soil materials must be available to provide a stable, impervious fill. (4) All ponds must be equipped with adequate mechanical and emergency spillway facilities to maintain a uniform water depth during normal conditions and to safely manage flood runoff. (5) All ponds must be equipped so that they may be drained to facilitate maintenance and fish management. (6) Adequate safety equipment must be provided around drop-inlet structures and other hazardous portions of the dam. (7) All design specifications must be adhered to in construction, and a sound program of maintenance must be followed to protect against damage by wave action, erosion, burrowing animals, livestock, farm equipment, and careless recreational use. All of these must be carried out to assure safety of the structure and to prevent damage to property below.

12.4. Types of Storage Ponds and Reservoirs. The four major types of ponds in common use are: (1) dugout ponds fed by ground water, (2) ponds fed by surface runoff, (3) spring- or creek-fed ponds, and (4) off-stream storage ponds.[3]

Dugout Ponds. Dugout ponds are limited to areas having slopes of less than 4 per cent and a prevailing reliable water table within 3 to 4 feet of the ground surface. Design is based on the storage capacity required, depth to the water table, and the stability of the side slope materials.

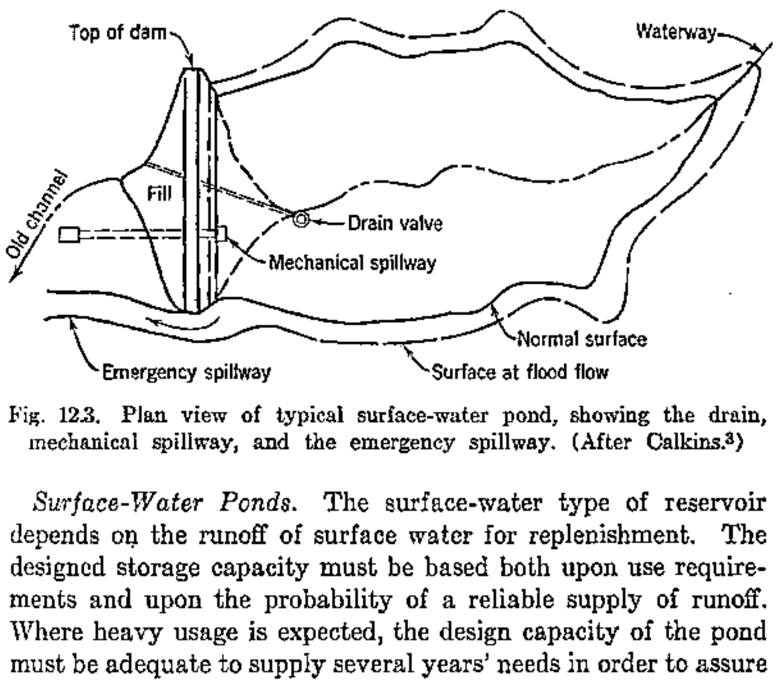


Fig. 12.3. Plan view of typical surface-water pond, showing the drain, mechanical spillway, and the emergency spillway. (After Calkins.[3])

Surface-Water Ponds. The surface-water type of reservoir depends on the runoff of surface water for replenishment. The designed storage capacity must be based both upon use requirements and upon the probability of a reliable supply of runoff. Where heavy usage is expected, the design capacity of the pond must be adequate to supply several years' needs in order to assure time for recharge in the event of a sequence of one or more years of low runoff.[16] Emergency spillways protect the structure from overtopping by flood flows, and a mechanical spillway handles sustained flows that would injure vegetation in the flood spillway. This type pond is shown in Fig. 12.3.

Spring- or Creek-Fed Ponds. Spring- or creek-fed ponds consist of either a scooped-out basin below a spring or a reservoir formed by a dam across a valley or depression below the spring. Such ponds (Fig. 12.3) must be designed to maintain the pond surface below the spring outlet. This eliminates the hazard of diverting the spring flow due to the increased head from the pond. When the spring flow is adequate to meet use requirements, surface waters should be diverted out of the pond to reduce sedimentation and to reduce spillway requirements.

Offstream Storage Pond. The offstream or by-pass pond is constructed adjacent to a continuously flowing stream, and an intake, through either a pipe or open channel, diverts water from the stream into the pond. Controls on the intake permit reduction in sedimentation, particularly if all flood water can be diverted from the pond. Proper location and diking are essential to protect against stream overflow damage.

PRINCIPLES OF DESIGN

Regardless of the structure size, the basic design principles apply equally to all structures. In conservation programs dams and embankments fall into two general classes: farm ponds and reservoirs having a total height above ground level not to exceed 15 feet; and structures between 15 and 50 feet in height. Structures in excess of 50 feet in height have specialized design requirements not discussed in this book. All structures must be designed to conform with state and local laws and ordinances.

12.5. Site Selection. Dams for livestock water, irrigation, or fire protection must be located where the impounded water may be used most effectively with a minimum of pumping and piping. Topographic features must be carefully studied to eliminate need for excessively large structures. For example, the slope of a channel floor of a reservoir should be less than 8 per cent.[30,31,32] Flood control reservoirs must be located where a minimum of damage will result from inundation of the storage area and where maximum flood peak storage capacity can be obtained at minimum cost. Levees and dikes must be so located that they protect a maximum area of land from flooding but do not seriously constrict the floodway, thereby increasing the flood stages.[22]

The site for water storage structures is also dependent upon a contributing watershed capable of supplying the necessary runoff. The probable rates and volumes of runoff should be determined by the methods outlined in Chapter 4. Approximate volumes of runoff may be determined from Fig. 12.4.

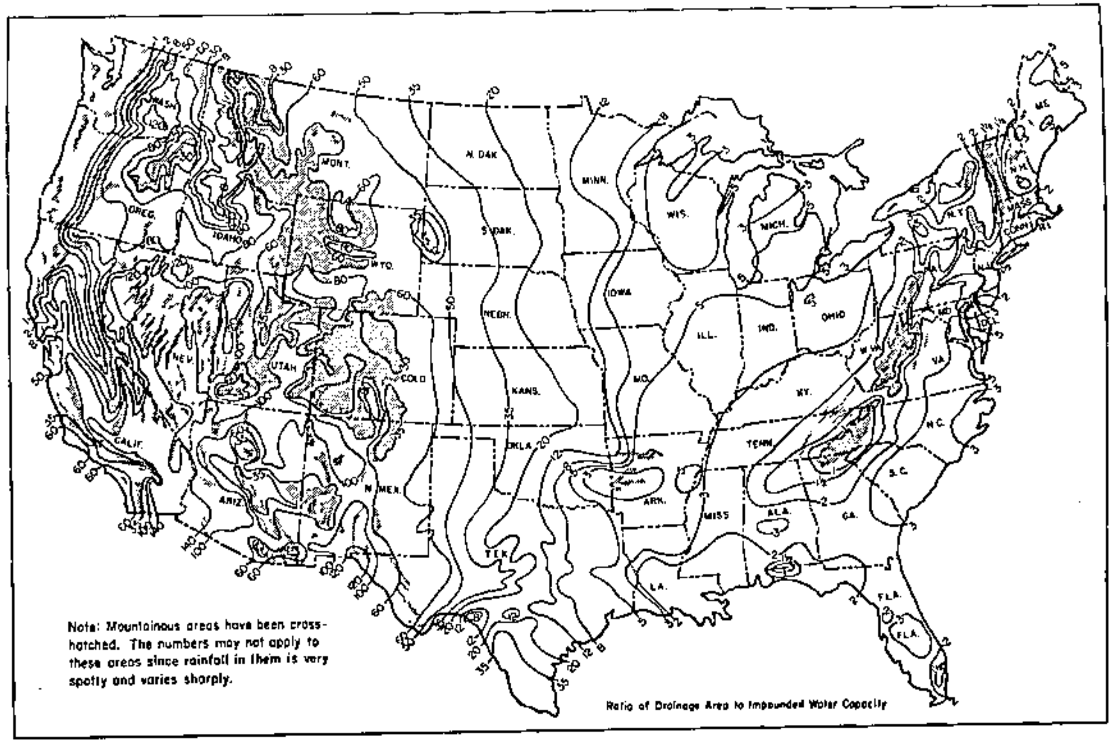


Fig. 12.4. A general guide for estimating the approximate size of drainage area required for a desired storage capacity of 1 acre-foot in farm ponds and reservoirs. (From Hamilton and Jepson.[8])

These are general values that should be checked with local hydrologic data.

Design of the storage capacity and watershed requirements should take into account both evaporation and seepage losses. The evaporation values can be obtained from U. S. Bureau Records. Seepage losses cannot be accurately predicted in advance; however, experienced engineers learn to recognize and evaluate potential seepage losses based on foundation and site investigations.

The watershed should also be examined for evidence of excessive sediment-producing areas. Cost of necessary control measures, such as reforestation, pasture development, terracing, or construction of sediment basins should be considered.

A careful examination should be made of both the geological and the soil conditions at the dam site and in the reservoir area in order to locate and design for such features as sinks, outcrops of fissured rocks, gravel seams, and gravel beds.

12.6. Storage Capacity of Farm Reservoirs. The value of a farm reservoir lies in its reliability in supplying water for all intended uses. This can only be achieved through careful analysis and planning.[17]

Pond capacity for supplying water for livestock, spray, or irrigation can be computed on a direct gallonage basis. Table 12.1 presents standard consumption values for computing water requirements.[16] In areas of low runoff expectancy, it may be necessary to provide storage capacity for several years requirements in order to assure availability of water in the event of two or more successive dry periods.[16]

Capacity for fire protection depends on the number of structures to be protected. Consultation with local fire departments provides estimates based on the types of equipment available. Departments equipped with ultra-high pressure pumps require less available capacity than those equipped with standard pressure equipment.

Ponds of less than ¼ surface acre make it difficult to keep the supply of fish in proper balance and also produce a low volume of fishing. The average family-sized pond is about 1 surface acre; larger ponds require close supervision and management and extensive fishing to maintain control.[6]

The average livestock or recreation pond does not exceed 15

Table 12.1 APPROXIMATE WATER CONSUMPTION VALUES FOR GENERAL FARM USE*

Average Quantity of Water Required for:	*Gallons per Day*	*Acre-Feet per Year*
Each member of family, all purposes	35 to 100	0.039 to 0.110
Each horse	10	.011
Each steer or dry cow	12	.013
Each cow producing milk	25–30	.034
Each hog	2	.002
Each sheep	1.5	.002
Each 100 chickens	4	.004

Orchard spraying:	
Apples	1 gallon per year of age per application
Peaches	1 gallon per year of age per application
Irrigation (humid regions):	1 to 1.5 acre-foot per acre per year

* Modified from Kirkpatrick.[16]

feet in depth; irrigation ponds are frequently designed for 30 or more feet of depth. Added depth gives greater capacity and also provides a better ratio of pond area to depth, thus reducing evaporation losses.

12.7. Foundation Requirements. Earth dams and embankments may be built upon a wide range of foundation conditions provided prior investigation has been made to determine the needs.

On small dams these investigations may be limited to auger borings. On larger structures the subsurface exploration should be more thorough. Wash borings, test pits, and other standard procedures should be employed to determine the underlying soil and geologic conditions. Discussion of these exploration methods will be found in many references.[5,14,18,23,24,25,26,27,33]

Foundation materials can be classified as follows:[5] (1) *ledge rocks.* Under earth-filled dams ledge rocks present a potential permeability hazard and frequently need grouting. (2) *Fine uniform sands.* If below "critical density" (void ratio at which a soil can undergo deformation without change of volume) fine uniform sands must be consolidated to prevent flow when saturated under load. (3) *Coarse sands and gravel.* From the stability standpoint they will consolidate under load. An upstream blanket may be required to prevent seepage losses. (4) *Plastic clays.* They require careful analysis to assure that shear stress imposed by the weight of the dam is less than the shear strength of the foundation material; flattened side slopes may be required to reduce shear stress.

Knowledge of porous strata, preglacial gorges, geologic faults, and other hazardous conditions will be of value in design of the structure.

12.8. Design to Suit Available Materials. The design of a dam or embankment should be based upon the most economical use of the available materials immediately adjacent to the site. For example, if satisfactory core materials are unavailable and must be hauled some distance, the hauling cost should be compared to the cost of a thin-section diaphragm of concrete or steel.

Cross section design depends on both the foundation conditions and the fill material available.[5,21] Where depth to an impervious foundation is not too great and where supplies of quality core materials can be found, designs shown in Fig. 12.5*a* and 12.5*b* can be used. This may be simplified as in Fig. 12.5*c* where there is an impervious foundation, if care is taken to bond the fill and core to the foundation. The combination core-and-blanket design shown in Fig. 12.5*d* is adapted to sites having extremely deep pervious foundations. Where foundations of low shear resistance are encountered, a design that gives a larger loading area together with good foundation drainage, is shown in Fig. 12.5*e*. Other designs may be developed to utilize diaphragms alone and in combination with cores and other construction features to meet specific conditions.

For optimum compaction and water-holding capacity, experiments and experience have shown that soils having 70 to 90 per cent sharp, well-graded sand; 25 per cent to as low as 5 per cent plastic clay; and enough silt to give good gradation are most satisfactory.[11,12] Soils having high shrinkage and swelling characteristics and ungraded soils, when their use cannot be avoided, should be placed in the downstream interior of the embankment. Here they are subject to less moisture change and because of overburden weight have less volume change than if placed elsewhere in the embankment. Soils having higher percentages of graded sandy materials resist changes in moisture,

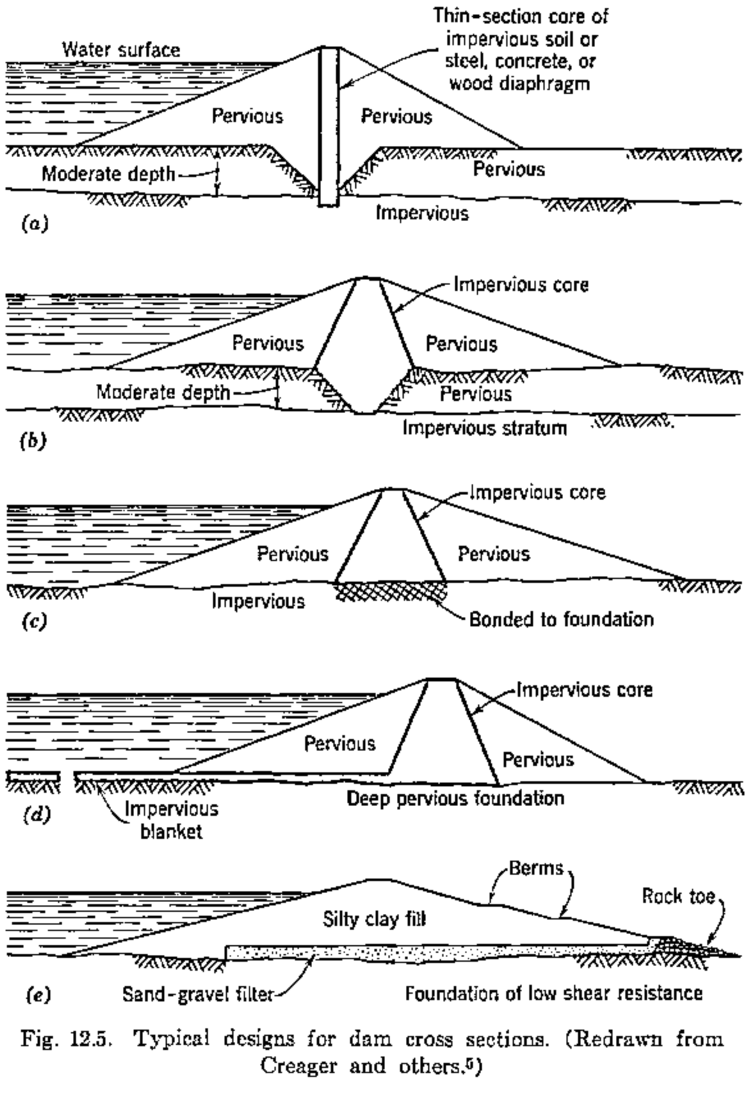


Fig. 12.5. Typical designs for dam cross sections. (Redrawn from Creager and others.[5])

temperature, and internal stresses. Organic soils should be entirely eliminated from the fill.

In levee construction, the selection of material is usually limited to that found adjacent to the structure. As these materials change in their characteristics along the route of the levee, it will be necessary to adjust the cross-sectional design of the embankment in compensation.

12.9. Side Slopes and Berms. The side slope of an earth dam or levee is dependent upon the height of the structure, the shearing resistance of the foundation soil, and upon the duration of inundation.

On structures less than 50 feet high with average materials the side slopes should be no steeper than 3:1 on the upstream face and 2:1 on the downstream face. Well-graded soils when carefully placed and compacted may be left on a 2:1 slope on both faces. Coarse, uncompactable soils may need side slopes of 3:1 or 4:1 to assure stability.[19,26,29,30,31,32] On dams exceeding 50 feet in height a complete analysis should be made of the horizontal shear strength of both the upstream and downstream slopes.[5]

In levee construction on major waterways having flood peaks of long duration that cause maximum saturation of the levee, side slope ratios may range as high as 7:1.[22] These flat slopes are necessary because levee materials are frequently of poor quality, are unstable, and frequently receive no compaction.

Most conservation levee construction is limited to those protecting farm lands on the upper tributaries. Since they are subjected to short-duration flood crests, minimum side slopes of 2:1 may be provided if they are properly protected at critical points. Levees that protect land from back flow after pump drainage, as in certain muck areas, tidal marshes, and swamp areas, should be designed and constructed by the same criteria established for dams.

On dams exceeding 30 feet in height the problem of controlling erosion on the downstream face is often severe. The use of berms (Fig. 12.5*e*) or shoulders 6 to 10 feet wide across the slope provides control. These berms should slope toward the dam to form a gutter from which a system of drains may conduct runoff water safely to the toe of the dam. Berms of this type may adjust dam cross sections so as to lower the position of seepage outflow on the downstream face. Where rip-rap is placed on the face of a dam, a berm should be provided as a shoulder to serve as a foundation for the rip-rap.[5]

12.10. Top Width. Top widths of dams vary with the height and purpose to which the dam is being put. The minimum top width for dams up to 15 feet in height should be 8 feet.[30] If the top is to serve as a roadway, this minimum should be increased to 12 feet to provide a 2-foot shoulder to prevent raveling. Road water should be controlled to prevent erosion of the side slopes.

The top width of dams exceeding 15 feet in height may be designed by the empirical formula:[33]

$$W = 2(H)^{\frac{1}{2}} + 3 \tag{12.1}$$

in which W = top width in feet (minimum to be 8 feet).
H = maximum height of embankment in feet.

In levee construction top or crown width may vary from a minimum of 3 feet to 20 or more feet. On levees subject to short-duration flood peaks, 3-foot crowns are adequate.[22] On levees subject to sustained peaks the width usually equals twice the square root of the height of the structure.[2]

12.11. Freeboard. The distance between the maximum designed high water or flood peak level in the reservoir and the top of the settled dam or embankment constitutes the net freeboard. Reference is sometimes made to gross freeboard, or "surcharge," which is the distance between the crest of the mechanical spillway and the top of the dam. Net freeboard should be used in all design work.

The net freeboard should be sufficient to prevent waves or spray from overtopping the embankment or from reaching that portion of the fill that may have been weakened by frost action[33] (that depth of soil loosened by frequent alternate freezing and thawing—this depth is seldom over 6 inches).

Wave height for moderate-size reservoir areas can be determined by Hawksley's formula:[14]

$$h = 0.025(D_f)^{\frac{1}{2}} \tag{12.2}$$

where h = height of wave in feet from trough to crest under maximum wind velocity, and D_f = fetch or exposure in feet.

All freeboards should be based on the water level at the maximum flood heights for which the dam and spillways are designed. For example, if critical frost action is 0.5 foot, maximum wave height 2 feet, and the designed flood storage depth is 2 feet, then the total designed gross freeboard above the mechanical spillway crest would be 0.5 + 2 + 2 or 4.5 feet with a net freeboard of 2.5 feet. This relationship is illustrated in Fig. 12.6. Additional freeboard should be added as a factor of safety where lives and high property values would be endangered by a dam failure.

Levee freeboard is designed on the same basis as dams. Freeboard for dikes on tidal waters should be designed on the basis of Stephenson's wave formula.[5,14] This value should be added to the maximum flood tide elevation when total freeboard is being determined.

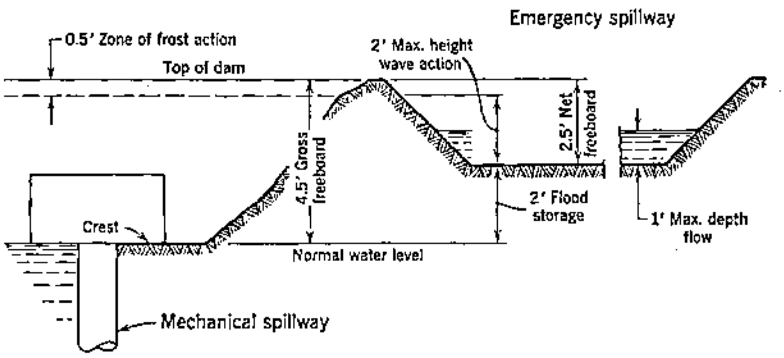


Fig. 12.6. Sectional diagram showing the relative position of the factors considered in the design of the freeboard.

12.12. Settlement. Rolled-filled embankments which have been placed in thin layers and compacted at optimum moisture and which are on an unyielding foundation will not settle more than about 1 per cent of the total fill height.[5] On deep plastic foundations settlement may reach 6 per cent.[5] Fills for small farm ponds and reservoirs usually do not receive as thorough compaction as the larger structures do; therefore the fill is usually constructed to a height 10 per cent higher than the designed settled fill height for all points along the embankment.[29,30]

To determine the necessary surcharge or additional fill material needed to bring the structure to the permanent finish grade, the following settlement formula is used:[5]

$$S = \frac{(e_1 - e_2)}{(1 + e_1)} y \qquad (12.3)$$

where S = settlement in feet.
y = depth of soil being consolidated in feet.
e_1 = void ratio (Chapter 5) before consolidation.
e_2 = void ratio after consolidation.

In levee construction a minimum allowance for settlement of 20 to 25 per cent is usually made because dragline or conveyor placement without intensive compaction does not give a high degree of consolidation.[22]

12.13. Seepage. The characteristics of the soil materials in both the foundation and the fill largely affect the degree of seepage that passes through and under the dam. All earthen structures that impound water develop a "seepage line" that is essentially the line of saturation.

The position of this line in the structure is determined by (1) the permeability of the fill materials and of the foundation, (2) the position and volume of ground water flow at the site, (3) the type and design of any core wall or cutoff within the embankment, and (4) the location of drainage devices to collect seepage in the downstream portion of the structure.

In large structures it is important to determine exactly at what point the seepage line intersects the downstream slope under various proposed construction designs, so that adequate core walls, berms, and toe drains can be specified to prevent destructive seepage outflow on the downstream face of the dam.

Several methods for mathematically determining both the potential position of this seepage line and the rate of seepage under various proposed designs, utilizing the soil materials at hand and under specified degrees of compaction, have been developed. It is, however, beyond the scope of this book to discuss these methods in detail.[4,5,15,23,25,26]

The most effective method of controlling seepage through or under a structure is by constructing a core of dense, compacted, well-graded soil material that extends from bed rock or an impervious stratum to a point above the water line.

The minimum height of a cutoff seal should be one-half the height of the dam; its depth below original ground surface will depend upon the position of the impermeable stratum. Side slopes of the trench should not be less than 1:1.[31] Where the cutoff seal is extended as a full core, it should be designed with a minimum top width of 4 feet and the side slopes should not be steeper than ½:1.[31]

Where satisfactory soil materials for cores are unavailable or where there is an excessive distance through the pervious foundation material to impeding strata, cutoff or core walls of steel sheet piles or reinforced concrete may be used. The effectiveness of this device is based on the care taken in installation and in keying the cutoff into the soil materials.[5]

Where cores and cutoffs are not feasible due to construction difficulties, an upstream blanket may be substituted as shown in Fig. 12.1*b*. The principle of this blanket is to lengthen the path of percolation under the dam. The efficiency of such blankets is lowered owing to the horizontal permeability being several times greater than the vertical permeability, thus requiring a considerable length of blanket to equal vertical cutoffs or cores.

Blankets should not be applied to the upstream face of a dam where frequent, total, or rapid drawdown is anticipated because of the danger of slumping, due to the internal hydrostatic pressure in the saturated portion of the dam, and of cracking upon drying. Uncontrolled stock watering or wading also causes disturbance of the blanket.

Where blankets are used on small structures, they usually extend out from the toe of the dam 8 to 10 times the depth of the water at the dam.[5] Minimum thickness at the top of the dam is 2 feet; at the toe it should be 3.5 feet.[30] For larger structures the blanket thickness can be determined by the formula:[33]

$$t = 2 + 0.02d \tag{12.4}$$

where t = thickness in feet for a blanket of very impermeable material, and d = distance in feet from upstream end of blanket to any point on the blanket for which the thickness is to be determined.

Details on the construction of concrete core walls and other specialized seepage control devices may be found in many references.[5,9,15,33]

12.14. Sealing Ponds and Reservoirs. The preceding section deals mainly with problems of reducing seepage through and under earth structures. Frequently farm ponds are needed at sites where the storage area will not hold water. In such instances it is necessary to resort to a "bag-type" construction

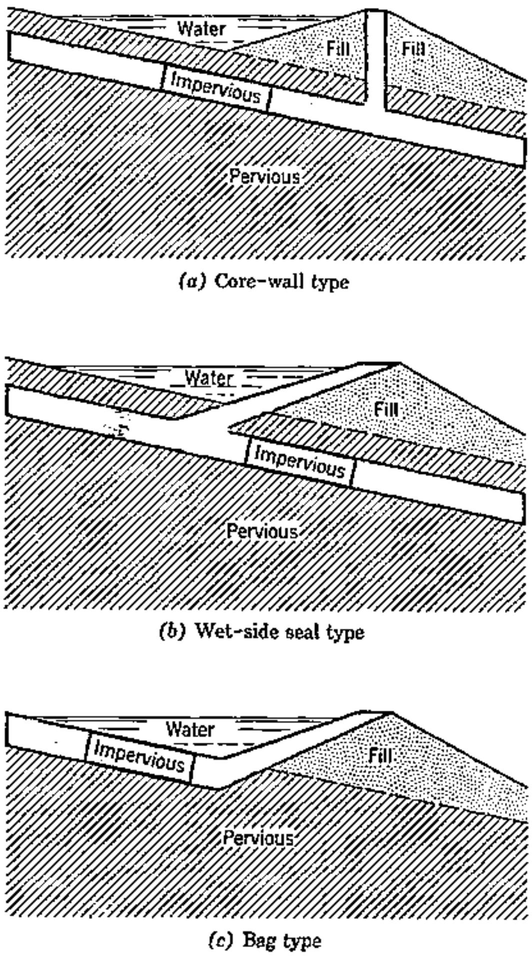


Fig. 12.7. Three comparative methods of pond sealing. (From Holtan.[11,12])

in which an impervious blanket is constructed over the entire impounding area, as illustrated in Fig. 12.7.[11,12]

Where the soil materials on the pond bottom approach the proportions of 70 per cent sharp well-graded sand, 20 to 25 per cent clay, and sufficient silt to provide good gradation of particle size, the seal can be accomplished by ripping the soil to a depth of 10 to 12 inches and then recompacting at optimum moisture with a sheepsfoot roller until maximum density is attained.[11] When proper soils are not available at the site, materials must be selected from adjacent borrow pits, mixed, spread, and compacted.

Support of the head of stored water is essential; therefore, consideration must be given to the necessary mantle thickness needed to prevent the water pressure from forcing the soil out through underlying rock crevices. Under most conditions, a minimum of 2 feet of soil should exist between the compacted blanket and the rock formations. Soils with the particle size gradation outlined above have sufficient structural resistance to support the weight of the water, thus preventing slippage and flow of the soil mantle into underlying rock crevices. Soils having high clay contents do not have this resistance to superimposed loads. Therefore, to provide the needed supporting power, high clay content soil mantles must have greater thickness.[11,12]

Where satisfactory construction materals are unavailable, seals may sometimes be accomplished by incorporating swelling clays, such as bentonite, with the soil. This material swells to fill the interstices between the soil particles, thereby decreasing the permeability. Bentonites alone, will not support the weight of the stored water; therefore, they should be used only with soils containing a minimum of 10 to 15 per cent sand, which will provide the supporting capacity. Rates of application must be adjusted to individual soil conditions. Specialists should be consulted to make this determination.[11,12]

12.15. Drainage. In relatively impervious and homogeneous dams the seepage line frequently appears high on the downstream slope, resulting in a saturated unstable soil condition. The construction of a system of toe filters or drains will lower the seepage line until it intersects with the drain.

The capacity of toe filters or drains should be at least twice the computed maximum seepage discharge through the dam.[5] Figure 12.8 illustrates several types of effective drainage systems. Full details of design and construction may be found in textbooks on dam design.[5,9,14,33]

Adequate drainage is equally important in levee construction, for a saturated impounding structure is always potentially dangerous. A tile line or drainage ditch located near the land side toe at a depth that will affect the lower end of the satura-

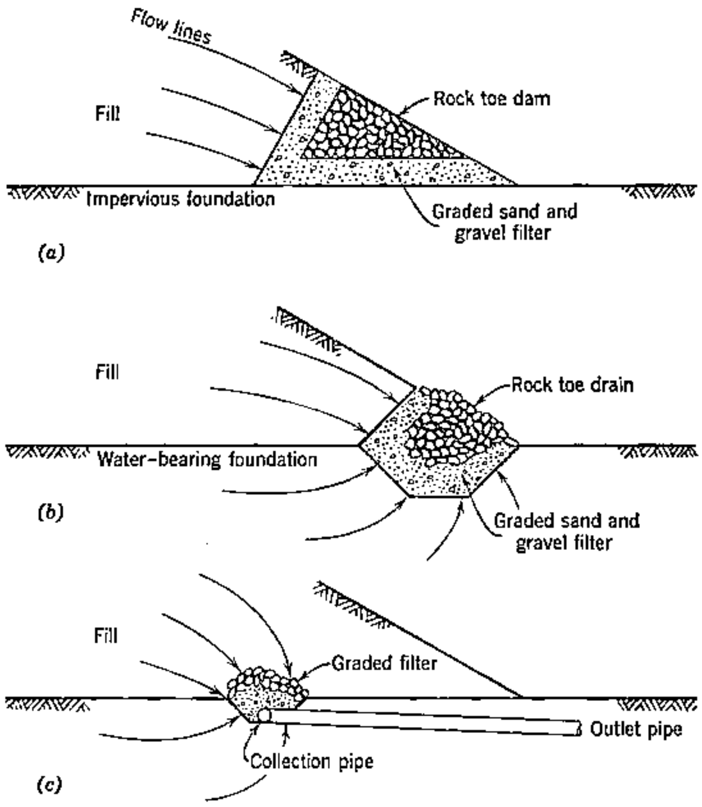


Fig. 12.8. Three types of toe drains: (*a*) simple rock-fill toe, (*b*) deep rock toe designed to drain both the fill and the water-bearing foundation, and (*c*) pipe-type drain with collection pipe.

tion line is a standard design.[22] These drains are carried away from the levee and outletted through controlled relief wells.

12.16. Wave Protection. If the water side of dams or levees is exposed to considerable wave action or current, it should be protected either by covering with nonerodible material called rip-rap or protected by energy dissipaters such as anchored floating logs.[5,9,14,31,33]

Rip-rap may consist of hand-placed or dumped stone or various types of concrete block or slab construction. It should be carefully installed on a bedding or cushion layer of graded gravel or crushed stone from 9 to 18 inches deep, depending on quality. This layer prevents the clay and silt from being washed out by wave action, thus causing the rip-rap stone to settle into the embankment. Full details for various types of protection are given in many references.[5,9,14,33]

A dense sod of Bermuda grass or bluegrass will usually give slope protection on levees that are not subjected to excessive wave action. In exposed areas rip-rapping or willow or cottonwood revetments must be used.[7,22]

12.17. Mechanical Spillways. Most farm ponds and reservoirs dealt with in conservation programs are protected from overtopping by vegetated emergency spillways. This vegetation will not survive if base flow entering the pond and frequent minor flood-flows keep the spillways unduly wet. Therefore, in order to protect these spillways and to maintain a more uniform water level, various types of mechanical spillways are provided.

These consist of various types of structures (Chapter 11) having their entrance openings set at the elevation at which the pond is to be maintained and extending through the dam to a safe outletting point below the dam. The entrance elevation depends on the level of permanent storage desired. On most farm ponds it is 6 to 12 inches below the entrance lip of the emergency spillway. On flood retention reservoirs it will range from 2 feet to the entire depth of the pond, depending on the need for a permanent pool for livestock or other use.

Any durable material may be used for the mechanical spillway, provided it has a long life expectancy and is not subject to easy damage by settling, loads, or blows. The portion passing through the dam should be placed on a firm foundation of slowly permeable material, preferably on undisturbed soil, and should be located to the side of the original channel. When rigid conduits, such as concrete, cast-iron, or asbestos cement are placed on rock foundations, they must be cradled in concrete to the point of maximum diameter. Flexible conduits, such as steel or corrugated metal pipe, may be cradled in compacted impermeable soil material.

All such structures passing through the dam should be covered by thin, well-tamped layers of high-grade material. Where the fill surrounding the pipe is of low quality, concrete or metal

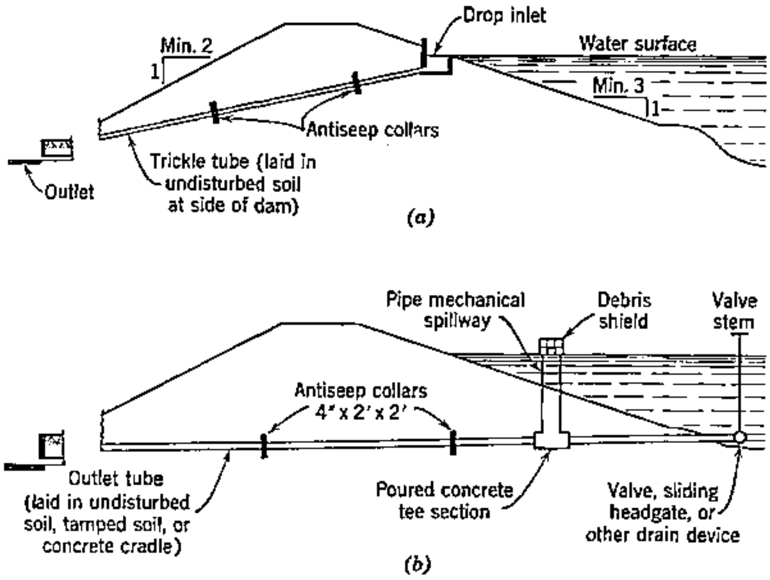


Fig. 12.9. Sectional sketch of two types of mechanical spillway designs: (*a*) drop inlet and pipe, and (*b*) vertical riser and drain pipe combined. (Where the drop-inlet spillway (*a*) is used, the drain is constructed as in (*b*) less the tee and riser pipe.)

seepage collars, extending out a minimum of 2 feet in all directions, should be used. The number of collars should be sufficient to increase the length of the seepage path by 20 per cent. All construction forms should be stripped from concrete structures before covering. All pipe joints should be carefully packed and sealed. Where concrete or mortar joints are used, it is advisable to complete the backfilling and tamping before it has set, thus avoiding danger of cracking the green concrete. A minimum of 4 feet of protective fill should be hand-placed over the structure before heavy earth-moving equipment is allowed to pass over it.[29,31,33]

The mechanical spillway may, in some instances, serve as a drain for cleaning, repairing, and restocking the pond. One of several types of installations is shown in Fig. 12.9.[31] All valve assemblies should be equipped to operate from above the surface of the water. All mechanical inlets should be protected with a screening device that prevents floating objects, turtles, and other foreign objects from entering and clogging the pipe. The device should be so designed that, after becoming loaded with debris, it will not retard the entrance of water. Outlets should be carried well below the toe of the dam and protected against scouring (see Chapter 11).

Mechanical spillway sizes may be determined from Table 12.2. After estimating the average base flow, the approximate seepage flow after a storm (in some areas this is estimated at 5 to 8 per cent of the peak flow), or the minimal flood flow from small storms that may be expected, this estimate is compared with the discharge values in Table 12.2. The diameter of the drain pipe and the riser can then be determined.[30]

Table 12.2 RECOMMENDED SIZES OF DRAIN AND RISER PIPE FOR TUBE-TYPE MECHANICAL SPILLWAYS*

Discharge Capacity Q, cfs	*Diameter of Drain Pipe,* in.	*Diameter of Corresponding Riser Pipe,* inches for head, h		
		1.0 ft	1.2 ft	1.5 ft
Up to 2.7	8	8		
2.7– 4.7	10	10		
4.8– 7.6	12	12		
7.7–13.2	15	18	15	
13.3–21.0	18	30	21	18
21.1–31.1	21	42	36	24
31.2–42.8	24		42	30

* Recommendations for corrugated metal pipe. For smooth pipe, use next smaller size (8-inch minimum).

From U. S. Soil Conservation Service.[30]

12.18. Emergency Spillways. All ponds must be equipped with one or more emergency spillways that will safely by-pass flood runoff that exceeds the temporary storage capacity of the reservoir. Where a natural spillway, such as a depression in the rim of the impounding area does not exist, it is necessary to construct one or more trapezoidal channels in undisturbed earth.

After careful estimation of the peak rates and amounts of runoff to be handled, application of the broadcrested weir formula and vegetated waterway design procedures will determine the cross-sectional area required (Chapters 4, 9, 11). The design capacity for emergency spillways may be estimated by determining the ratio of the reservoir capacity above spillway level to the total emergency storm runoff volume. Ratios of 0.3 and 0.7 may permit reduction of spillway design capacities to 86 and 40 per cent of the peak flow, respectively; where the ratio is less than 0.3, design the spillway for full flow.

Where flood-flow storage is planned, the principles of flood routing outlined in Chapter 13 also apply. Side slopes of spillway channels should not exceed 2:1, and the channel grade should be approximately 2 per cent.[30] The width of the spillway at each end should be gradually increased by a minimum of 25 per cent greater width than the main channel. The designed depth of flow should not exceed 1 foot.

The channel bottom must be finished level to prevent concentration of flood-flow. Dense vegetation should be established by seeding or sodding. Outlets should extend well below the dam or into an adjacent waterway and should be well-protected against scouring.

12.19. Stock-Watering Facilities. Livestock should not be allowed free access to a pond because of the damage that they do to the fill and banks. Water should be piped in a 1¼- to 1½-inch pipe to a tank on a well-drained area not less than 50 feet below the dam. The intake should be a perforated or screened riser that will take water at least a foot above the pond bottom, thus reducing sediment in the tank. In extreme conditions a graded sand-and-gravel filter should be placed around the intake.[13] Details of valves, overflows, and other items are shown in Fig. 12.10.[29]

STABILITY OF EARTH DAMS

The stability of earth dams and levees is a measure of the stability of that structure to remain unmoved by the forces acting upon it. Experience has shown that an earth structure

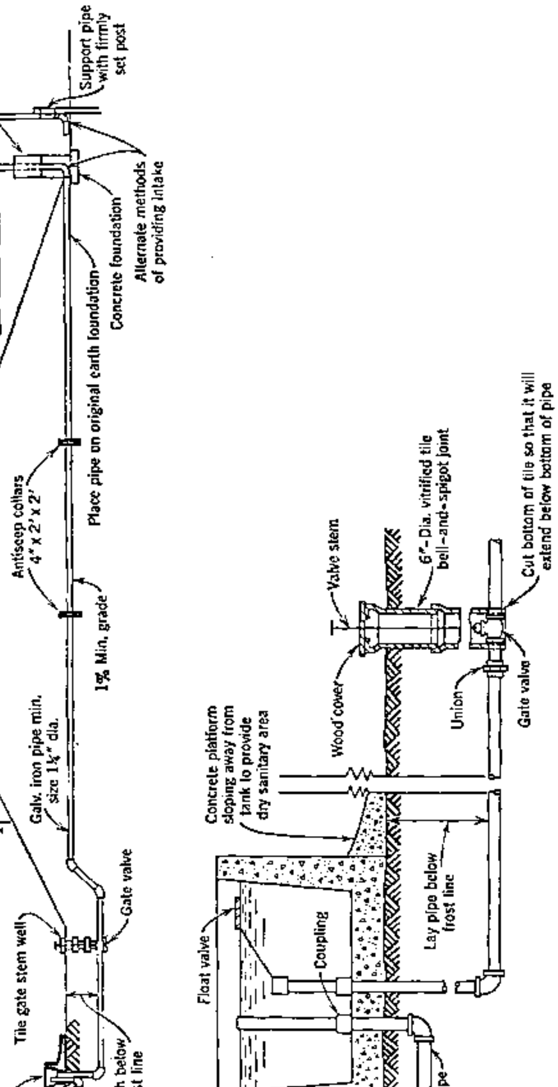


Fig. 12.10. Typical stock water tank installation. (Redrawn from reference 29.)

is stable and has a good factor of safety when the ratio of the forces capable of resisting movement to those forces that cause movement is approximately 1.5.[5]

In earth structures up to 50 feet in height, seepage is a major factor affecting stability. Thorough attention to seepage control in design can eliminate this factor.

In larger structures engineers should investigate more fully such factors as stability against headwater pressure, resistance to horizontal shear, and the shearing stress in foundations. Full consideration of these factors will be found in numerous references.[5,23,24,26,27]

CONSTRUCTION

Construction details are best learned through actual practice and field experience. However, a few basic factors must be considered in the construction of rolled-fill dams and embankments of nominal height.

12.20. Clearing and Stripping. All trees, stumps, and major roots should be removed from the site. Sod and topsoil should be removed and stock-piled. Before the placement of any fill material, the original ground surface should be thoroughly plowed and disked parallel to the length of the dam. If necessary, it should be sprinkled to assure that it is at optimum moisture for compaction. Proper moisture conditions make it possible to force some of the fill material into the original surface, thus eliminating any dividing plane between the fill and the foundation.

12.21. Core Construction. After the site has been cleared, the cutoff trench is excavated. The compacted core should be constructed only of carefully selected materials laid down in thin blankets at optimum moisture as determined by the Proctor density test (Chapter 5).

12.22. Compaction. The proper compaction of the fill is a major factor in rolled-fill earth dam construction. The fill material should be placed in thin layers, evenly, over the entire section of the dam. The thickness of layers for pervious soils should be limited to 8 to 10 inches in thickness; the more plastic and cohesive soils should not exceed 4 to 6 inches in thickness.[5] In placing the fill material on the dam, the fill should be nearly horizontal with a slope of 20:1 to 40:1 away from the center of the dam. If, upon the approach of rain, the surface is left in a fairly smooth condition, the resultant surface drainage will keep the surface from becoming saturated.[5]

Laboratory tests can be made to determine closely the optimum moisture content for each type of soil to achieve the necessary maximum consolidation (see Chapter 5). Generally, for smaller dams, a rule of thumb that may be used is: *the soil is at optimum moisture when it is too wet for good tilth but not wet enough to exude moisture under compaction.*[12]

The degree of compaction to be achieved is specified as a ratio, in per cent, of the embankment density to a specified standard density for the soil.[24] In earth dam construction, the degree of compaction should run 85 to 100 per cent of the maximum Proctor density[24] (Chapter 5).

Moisture control, where the fill materials are too dry, may be obtained by sprinkling. Overwet soils can be dried more rapidly by disking and light working to give maximum exposure.

Soils with high clay contents should be carefully compacted with the soil slightly dryer than the lower plastic limit to prevent the formation of shear planes called *slickensides*. Pervious materials, such as sands and gravels, consolidate under the natural loading of the embankment. However, additional compaction does aid in passing the *critical density*, in increasing shear strength, and in limiting embankment settlement. Compaction of these noncohesive materials is best achieved when they are nearing saturation.

Selection of proper compaction equipment is important. The sheepsfoot roller is best suited for compacting fills. Its weight may be varied by adding water to the drum, and the compaction per unit area may be adjusted to various numbers and sizes of tamping feet.

Table 12.3 indicates approximate tamping areas and unit pressures best suited to three general classes of soils. When the proper equipment is used for a particular soil, the feet will penetrate deeply into the fill on the first pass, compacting it near the bottom. On subsequent passes, it will continue to consolidate the soil from the bottom up. When final compaction has been achieved, the feet will penetrate only 1 or 2 inches.

Special precautions should be taken in compacting materials

Table 12.3 AREAS OF TAMPING FEET AND GROUND PRESSURES AS ADAPTED TO THREE BROAD TYPES OF SOILS*

Type of Soil	*Area per Tamping Foot,* sq. in.	*Contact Pressure,* psi
Sandy	10–12	50–100
Silty clay	7–9	100–200
Clay	5–6	200–400

* Reprinted by permission from *Soil Engineering* by Spangler,[24] International Textbook Co., 1951.

close to core walls, collars, pipes, conduits, etc. All such mechanical structures should be constructed so that they are wider at the bottom than at the top; thus settlement of the soil will create a tighter contact between the two materials. Thin layers of soil, at moisture contents equal to the remainder of the fill, should be tamped into place next to all structures. This can best be done with hand-operated pneumatic or motor tampers.[28] Heavy hauling equipment should be given varied routes over a fill to prevent overcompaction along the travel ways.

12.23. Trimming of Slopes. All slopes should be trimmed carefully to the design values.[14] Leaving excess soil on slopes steeper than that called for places an additional load on the face of the dam. When the embankment becomes saturated, slides are apt to occur at the point of loading.

12.24. Protection of the Top and Downstream Slopes. Definite steps must be taken to vegetate the slopes.[1] Wherever possible, topsoil should be spread evenly over the area. On the more pervious fills, the depth of topsoil should be 12 inches or more. An agronomist should be consulted for instructions on fertilizing and seeding practices that will give the maximum protective growth under the existing conditions.

PROTECTION AND MAINTENANCE

Because of the relatively high investment that is made in the construction of any dam, levee, or dike, it is important that a complete program of protection and maintenance be adopted to assure protection to this investment.

12.25. Protection. For best results the entire pond area should be fenced to prevent damage to embankments, spillways, and banks. Where this is not practicable, the spillways and dam at least should be protected.

To minimize damage from sedimentation the entire watershed should be protected by adequate erosion control practices. Buffer or filter strips of dense close-growing vegetation should be maintained around the pond edge.

12.26. Inspection and Maintenance. A properly designed and constructed earth dam, well-sodded and protected, should require a minimum amount of maintenance. However, as insurance on the investment, a regular inspection and maintenance program should be established. Particular attention should be given to surface erosion, the development of seepage areas on the downstream face or below the toe of the dam, the development of sand boils, and other evidence of piping, evidence of wave action, and damage by animals or human beings. Early recognition and repair of such conditions will prevent development of dangerous conditions that will continue to increase the cost of repair as long as they proceed undetected and unchecked.

Excessive weed growth, development of insect breeding areas at the pond edges, and, in fish ponds, depletion of the fertility level should all be noted and quickly corrected. Under no conditions should trees be permitted to grow on or near the embankment.

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PROBLEMS

12.1. Determine the top width for a dam having a maximum height of 20 feet. How far from the center line should the slope stakes be set at the highest point?

12.2. If the critical frost depth is 0.5 foot, maximum exposure of the water surface 1300 feet, and maximum design flood depth above the mechanical spillway 4 feet, determine the net and the gross freeboard.

12.3. Assuming a storage loss of 50 per cent by seepage and evaporation, determine the storage capacity of a farm pond to supply water for 100 steers, 50 milk cows, 50 hogs, and 1000 chickens, and to irrigate a 2-acre garden.

12.4. Determine the minimum size drainage area for 12 acre-feet of storage if the pond is located in southern Ohio.

CHAPTER 13

Headwater Flood Control

To appreciate the place of headwater flood control in the over-all flood control problem, the broader aspects of flood control must be understood. Headwater areas include the watersheds of small rivers and their tributaries with a total drainage area of 1000 square miles or less. From the viewpoint of the agricultural engineer all flood control activity downstream from this area constitutes downstream engineering. Floods may be classified as small- or large-area floods. The small-area flood is here arbitrarily limited to 1000 square miles or less, the same size as a headwater area.

Since downstream floods are more spectacular and damages more evident, the upstream flood too often has been neglected. Major downstream floods in the United States from 1902–1945 have caused damages estimated at over $2,000,000,000 with an average loss of $630 per square mile of drainage area.[2] Flood damages per unit area may be many times greater for small- than for large-area floods.

Although the Federal Government develops and supervises most flood control programs through such agencies as the War Department's Corps of Engineers and the U. S. Department of Agriculture's Soil Conservation Service and Forest Service, there are exceptions where local groups have organized and financed flood control programs. Outstanding among these is the Miami Conservancy District.[8] In general, state governments have not financed or individually administered flood control programs.

13.1. Types of Floods. A flood may be defined as an overflow or inundation from a river or other body of water. The distinction between normal discharge and flood flow is generally determined by the stage of the stream when bankful. Most floods occur on the flood plains adjacent to rivers and streams and result from such natural causes as excessive rainfall and melting snow. Occasionally, tidal waves or hurricanes cause flooding. With respect to loss of life, reservoir failures have produced some of the worst floods, but fortunately these seldom occur.

Large-Area Floods. Large-area floods occur over a large area from storms of low intensity having a duration of a few days to several weeks. Since melting snow may also contribute to or even cause large-area floods, seasons of maximum total precipitation do not necessarily coincide with the time of occurrence of large-area floods. In many parts of the country these floods come in the late winter or early spring. Generally, large-area floods cause greatest damage to metropolitan areas as well as considerable agricultural losses, whereas small-area floods cause major damage to agricultural land.

Small-Area Floods. Small-area floods occur over a small area from storms of high intensity having a duration of 1 day or less. Harrold[5] found that in Ohio small-area floods occurred in the summer between the months of May and September. Ninety per cent of the annual amount of rain, falling at rates greater than 1 iph fell during this 5-month growing season. Since 85 per cent of the annual soil loss occurred during this period, such floods cause great damage to agricultural land through soil erosion, which in turn results in sediment accumulation in rivers and reservoirs. These floods usually do not produce high runoff on large streams but often cause serious local damage.

13.2. Economic Aspects of Flood Control. Flood control is an economic problem in which the protection of life and property as well as the public welfare must be evaluated in balancing the annual savings from flood control against construction and maintenance costs.

Damages. Flood damages may be classified as (1) *direct* losses to property, crops, and land, which can be determined in monetary values; (2) *indirect* losses, such as depreciated property, traffic delays, and loss of income; and (3) *intangible* losses not subject to monetary evaluation, including community insecurity, health hazards, and loss of life. Since the distinction between direct and indirect losses is one of degree, there has been a lack of uniformity in evaluating damages. Indirect losses are difficult to determine and are often estimated as a percentage of the direct losses. Although the damage to land frequently goes unnoticed, soil erosion from the uplands, sedimentation in reservoirs, stream channels, and flood plains, and pollution of water supplies greatly affect the economy of the entire watershed. An investigation[14] of reservoirs in the United States revealed that

38 per cent have a useful life of only 1 to 50 years and 24 per cent, a life of 50 to 100 years.

Flood water causes damage by inundation and by high velocities. Though in some instances sediment deposits may be beneficial to farm lands, more frequently deposits of fine soil particles and sand have a damaging effect. Bridges, buildings, roads, farm lands, and stream channels are often destroyed by flood waters having high velocities. Some of these damages may be classed as nonrecurrent, depending on the nature of replacements and repairs. For example, a bridge replaced well above the high water level will probably not be in danger of subsequent damage. However, most damages, such as those from inundation and damage to land, are recurrent in nature.

Flood damages cannot always be prevented, but they may be reduced by flood forecasting and by using proper flood control measures. On large rivers flood warnings can be issued several days in advance of the flood. In this connection the U. S. Army Engineers are employing large-scale models of the Mississippi River for predicting downstream river stages. Longer range forecasts can be made by using hydrologic data. Approximately 15 per cent of the annual flood loss could be prevented by accurate forecasting.[16]

Benefits. In general, benefits from flood control are based on reduction of losses. For example, when a reservoir reduces all floods by a given depth, the benefits are determined from the difference in damages at the two stages and by the number of floods during a specified period of time. Aside from benefits to individuals and industries, there are public benefits, such as economic stability and better social conditions.

13.3. National Flood Control Policy and Legislation. Prior to 1936 there was no definite federal policy for the control of floods. With the exception of reclamation projects, federal undertakings were principally concerned with navigation. Flood control activities in the United States are summarized below.

Swamp Acts of 1849 and 1850. These acts granted the states the rights to sell all unsold swamp and overflowed land and to use the receipts for drainage, reclamation, and flood control activities. As a result, a large number of levees were constructed along the lower Mississippi River.

"Dam Act" of 1899 As Amended in 1906 and 1910. These

acts provided that specific approval of Congress must be received before dams, bridges, or other structures are placed in or across navigable waterways. The Corps of Engineers is authorized under these acts to impose regulations for the construction of dams that affect navigation.

Flood Control Act of 1928. Under this legislation the Corps of Engineers was directed to prepare plans for flood control on all tributaries of the Mississippi River and to determine the effect of a system of reservoirs. It is interesting to note that this is the first time reservoirs were considered in national flood control planning.

Flood Control Act of 1936 As Amended. A federal flood control policy was here established whereby the federal government authorized flood control activities on navigable streams and their tributaries. The act definitely specified that in planning for flood control the lands on which the flood waters originate must be considered. Federal investigations of watersheds and measures for runoff and waterflow retardation as well as erosion control were delegated to the U. S. Department of Agriculture. Investigations and improvements on rivers and waterways for flood control and allied purposes were delegated to the War Department. The program under the Department of Agriculture is carried out through the Soil Conservation Service and the Forest Service in cooperation with local, state, and other federal agencies. In making preliminary examinations and surveys, and in the prosecution of watershed operations, forest lands and range areas used in connection with forests are handled by the Forest Service, and farm and ranch lands by the Soil Conservation Service.

Flood Control Act of 1944. This legislation supplemented the original act of 1936. The provisions of this act require that plans and investigations be referred to the states concerned for investigation and consultation before submission to Congress. This act also provided the enabling legislation for the Pick-Sloan Plan in the Missouri River Basin.

13.4. Downstream Flood Control. Flood control activities on all navigable streams and on many tributaries that are not navigable would be classed as downstream. The Corps of Engineers from the beginning has been largely responsible for downstream measures.

Control on Lower Reaches. Levees, channel improvement, and diversion floodways are the principal methods employed to reduce damage from floods. On the Mississippi River levees were the only means of control for many years. As levees were built further upstream, valley storage was reduced and flood heights were increased, resulting in a continuous process of raising and straightening of levees. In more recent years it has been necessary to include such procedures as stabilization and protection of channel banks, straightening of the channel by cutoffs, construction of floodways, and dredging operations in order to maintain a navigable stream and to provide flood protection.

Control on Upper Reaches. On watersheds of about 5000 square miles or less reservoirs may provide adequate flood protection. As in the lower reaches, levees for local protection, channel improvement, channel straightening, and stream bank protection are applicable flood control measures. Such methods may be more economical than a system of reservoirs, but it should be remembered that, in general, levees speed the flow and accentuate the problem in unimproved areas below.

Reservoirs for flood control may be classified as natural or artificial. Lakes, swamps, and other low areas on the land surface have a tendency to reduce flood heights. Even a lake that is full at the beginning of a storm will have a regulating effect on stream flow. The extent of this effect depends upon the ratio of the surface area of the lake to the size of the watershed. A good example of a lake-regulated stream is the St. Lawrence River. The Great Lakes which drain into the river control the flow to such an extent that the maximum is not more than 20 per cent greater than the minimum flow.[9] This may be compared to the Missouri River which has a maximum stage 2900 per cent greater than the minimum flow. If single-purpose flood control reservoirs are to be economically feasible, the protected area must be of high value. In Ohio the Miami River and Muskingum Valley Conservancy Districts and in New England[2] the Connecticut River and Merrimack River projects are examples of flood reservoir projects.

Where water is stored for two or more uses, as illustrated in Fig. 13.1, reservoirs are classified as multiple-purpose structures. Requirements for flood control conflict to some extent with other objectives. For maximum flood prevention the reservoir must be empty at the beginning of the storm period and the stored water must be discharged as rapidly as the capacity of the stream below will allow. Water for power, irrigation, navigation, and water supply is withdrawn gradually at times when it can be used to best advantage. For the most part, irrigation water is needed only during the growing season. Water requirements

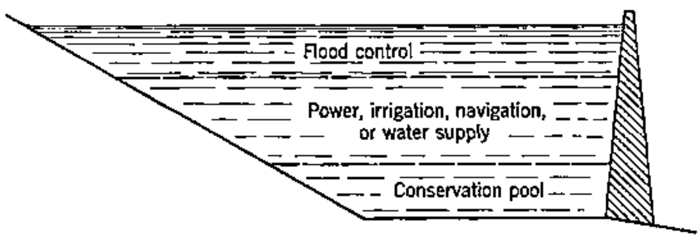

Fig. 13.1. Multiple-purpose reservoir.

for generating electrical energy vary with daily and seasonal loads. Water released during the summer for navigation also aids in alleviating stream pollution and may provide a more uniform water supply for cities downstream. Where a reservoir can be constructed with a capacity greater than required for other purposes, the additional capacity may be available for flood control. The Hoover dam on the Colorado River, the Grand Coulee on the Columbia, Fort Peck dam on the Missouri River, and Norris dam in Tennessee are examples of multiple-purpose reservoirs although not all of them are for flood control.

13.5. Headwater Flood Control. The primary difference between flood control measures for large areas and those for headwater control is that in the latter such factors as the effect of crops, soils, tillage practices, and conservation measures are considered. Headwater flood control is most effective in the control of flash floods of short duration which occur rather frequently, that is, two or three times a year. The effectiveness of headwater measures decreases the greater the distance downstream.

13.6. Need for Integrated Flood Control Program. Because of the many private and governmental interests in all aspects of watershed management, there is a real need for an integrated flood control program. Private interests include those of property owners, civic groups, and private enterprise organizations, such as power companies. Governmental interests include local and state governments, the U. S. Departments of Agriculture and Interior, and the U. S. Army Engineers. The objectives of these federal agencies vary widely and the legislative authorizations under which their programs are carried out are not always coordinated. In 1950 the President's Water Resources Policy Commission[11] was of the opinion that the division of responsibility and authority inevitably results in inefficiency and in an inadequate program.

METHODS OF HEADWATER FLOOD CONTROL

Two general classes of headwater flood control measures are those that retard flow or reduce runoff by land treatment or reservoirs and those that increase the flow by channel improvement, channel straightening, and levees.

REDUCING FLOOD-FLOWS

In general, flood-flows are predicted from stream flow records, developed hydrographs, empirical formulas, meteorological data, and previous high water marks. Methods of reducing these floods include (1) watershed treatment in which the storage of water is increased on the surface and in the soil profile, (2) flood control reservoirs, and (3) underground storage. Underground storage is accomplished by spreading the flow over a considerable area. This method is applicable only in special situations, particularly in arid regions where the water is later used for irrigation.

Measures that retard the flow or reduce runoff are economically and physically more desirable because (1) all visible evidence or danger of the flood is removed, (2) the flow in the stream is more uniform, thus providing greater recharge of the ground water and a more adequate water supply, (3) an important step toward the conservation of natural resources is achieved, (4) higher crop production results, especially in areas where conservation of moisture is important, and (5) reduction of sedimentation in the lower tributaries is accomplished.

13.7. Watershed Treatment. Watershed treatment includes all practices applied to the land that are effective in reducing flood runoff and controlling erosion. Proper land use

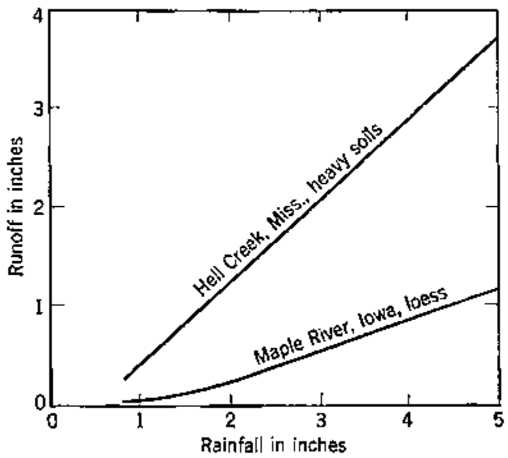

Fig. 13.2. Effect of soil on flood runoff. (Redrawn from Cook,[3] p. 130, Fig. 3.)

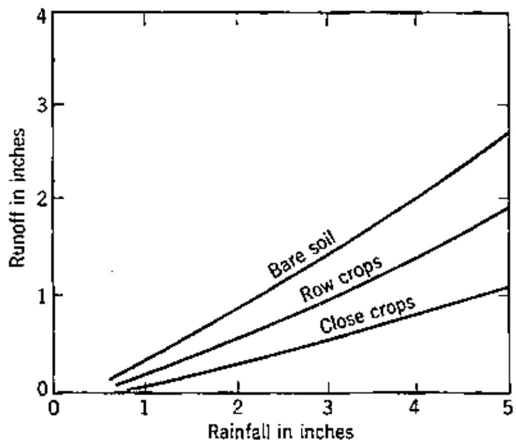

Fig. 13.3. Effect of vegetation on flood runoff. (Redrawn from Cook,[3] p. 130, Fig. 4.)

is necessary for adequate watershed control. The choice of practices depends largely on hydrologic factors and soil conditions.

Land treatment may increase the (1) amount of surface storage, (2) rate of infiltration, and (3) capacity of the soil to store water. Runoff retardation by land treatment is largely dependent on vegetative cover and favorable soil surface conditions. For example, in Figs. 13.2 and 13.3 the effects of different soils and crops on flood runoff are indicated. Results of soil differences are shown in Fig. 13.2 with about 3 times as much runoff occurring from heavy soils in Mississippi as from permeable loess soils in Iowa. Based on data from small runoff plots, the effect of vegetation on runoff for varying amounts of rainfall is shown in Fig. 13.3.

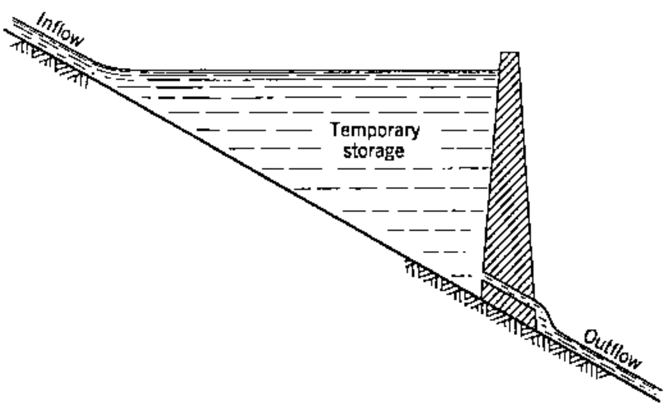

Fig. 13.4. Diagrammatic representation of detention reservoir operation.

13.8. Reservoirs. The two types of reservoirs are the flood storage reservoir and the detention reservoir. Both types reduce flood peaks by backing up water during periods of heavy runoff and discharging the water slowly. The principal difference is that the detention reservoir operates automatically by discharging through one or more fixed openings in the dam; whereas the storage reservoir discharges through adjustable gates. The chief advantage of the flood storage reservoir is its flexibility of operation. With either type of reservoir the greatest reduction in flood flow occurs just below the reservoir, the effect decreasing with the distance downstream.

Detention reservoirs such as shown in Fig. 13.4 may have one or more discharge openings of constant dimensions. These reservoirs have emergency spillways to handle runoff in excess of the design flood. Practically all headwater flood control reservoirs are of the detention type. Reservoirs on the watersheds of the Trinity River in Texas, Sandstone Creek in Oklahoma, the Little Sioux River in Iowa, as well as in the Miami Conservancy District[8] are examples of such structures. The principal advantage of the detention reservoir is its ease of operation, simplicity, and the fact that the discharge is based on design rather than on reliance upon humans.

INCREASING CHANNEL CAPACITY

The purpose of increasing channel capacity is to enable the flood water to move faster, thus decreasing height and duration of floods and reducing flood damages. Increasing the capacity of a stream may be accomplished (1) by channel improvement, (2) by channel straightening, and (3) by levees.

13.9. Channel Improvement. Channel improvement here includes those measures that increase the channel capacity, namely, enlarging the cross-sectional area and increasing the velocity. In narrow flood plains, channel improvement as well as channel straightening is considered the major method of flood control because under such conditions levees are usually not economical.

Increasing Cross Section. Increasing the cross section may be accomplished by deepening or widening the channel and by removing trees and sandbars from the watercourse. On small streams by removing trees and sandbars the effectiveness of the channel may be increased as much as one-third to one-half,[9] but in large streams the effect is considered negligible.

Increasing Velocity. Removing debris and vegetation have a greater effect on the roughness coefficient in small streams than in large streams. The hydraulic radius and resulting velocity can be increased by widening or deepening the channel. For the same increase in cross-sectional area, deepening the channel is more effective than widening. The depth may be increased by using levees as well as by dredging or cleaning out the channel. The two ways of increasing the channel slope are: (1) deepening the channel or lowering the water level at the outlet, and (2) straightening. Deepening the channel or lowering the water level at the outlet can be accomplished by increasing the cross-sectional area at the outlet, and removing debris or sand bars.

13.10. Channel Straightening. The principal method of straightening streams is to provide cutoffs. A cutoff is a natural or artificial channel which shortens a meandering stream, as

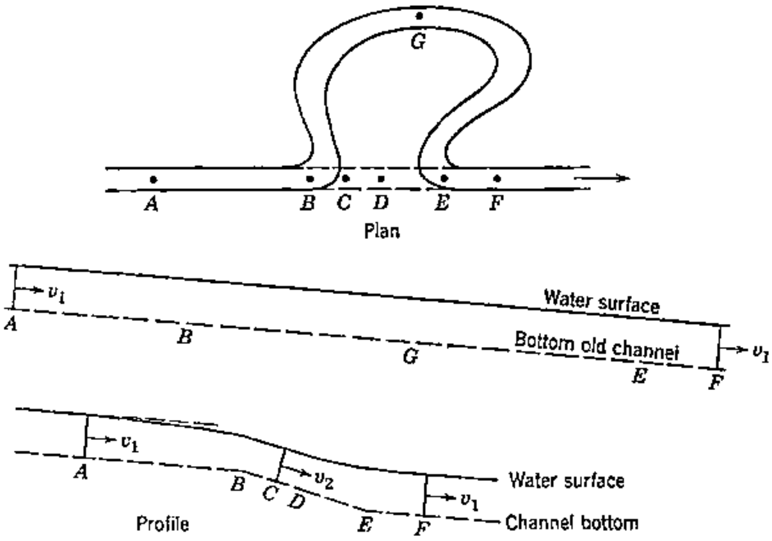

Fig. 13.5. Effect of cutoffs on stream flow.

shown in Fig. 13.5. The purpose of a cutoff is to increase the velocity, to shorten the channel length, and to decrease the length of levees. The length of the stream channel may be shortened as much as one-half the original length.[9]

Cutoffs are desirable where (1) the stream capacity in the bend is less than the capacity in other parts of the channel, (2) the capacity of the entire channel is to be increased with levees, (3) construction of the cutoff is more economical than increasing the capacity around the bend, and (4) the cutoff does not detrimentally affect the flow characteristics of the stream.

The effect of cutoffs is not always desirable. On alluvial streams cutoffs alone do not necessarily solve flood problems, as they may cause serious stream bank erosion above and below the cutoff and sediment deposits below. In some streams cutoffs cause extreme lowering of the channel and its tributaries. In others the increased gradient results in flow rates in excess of downstream channel capacity.

Cutoffs increase the velocity in the affected portion of the stream by increasing the hydraulic gradient. Because some water was formerly stored in the channel and in the flood plain along the bend *BGE* in Fig. 13.5, the cutoff also increases the stage downstream.

In a meandering stream cutoffs may occur naturally. Since erosive forces are a function of the velocity and depth of the stream as well as of the angle between the banks and the direction of flow, these forces are continually changing. Where there are bends in the stream, the soil is eroded from the concave bank, is carried downstream on the same side, and is deposited at the end of the bend, forming a bar which deflects the current to the other side of the channel. As the cutting continues, the stream becomes more crooked and a natural cutoff results.

The effect of a single cutoff, either natural or man-made, for steady flow conditions in the channel is shown in Fig. 13.5. Before the cutoff the stream flowed on a uniform slope around the bend *BGE*. After the stream is straightened, it flows from *B* directly to *E*, which is about one-fourth of its previous length. Because of the increased slope from *B* to *E*, the velocity is increased from v_1 to v_2 with a corresponding decrease in the depth of flow. The decreased depth causes the water surface to be lowered to point *A* upstream. In section *CD* flow takes place at a uniform depth with velocity v_2. At *D* the velocity begins to decrease but is still greater at *E* than at *A*, and likewise the depth is less at *E* than at *A*. Below *E* there is a concave upward surface which is known as the backwater curve.[9] At some point *F*, the end of the backwater curve, the velocity and depth is the same as at *A*. The cutoff is effective in lowering the stage through the cutoff section as well as above and below, to points *A* and *F*, respectively. It is interesting to note that the effect extends further upstream than downstream. Since cutoffs are usually short, the points *C* and *D* may overlap so that the velocity in the cutoff section does not correspond to the channel slope.

The effect of cutoffs becomes more complicated for unsteady flow and for a series of cutoffs. During a storm period the flow is constantly increasing and points A, C, D, and F change with the stage of the stream. Unless the slope or cross section of a stream is changed between cutoffs, a series of cutoffs acts similarly to a single cutoff.

13.11. Levees. Levees are embankments along streams or on flood plains designed to confine the river flow to a definite width for the protection of surrounding land from overflow. Levees may be designed either to confine the river flow for a considerable distance or to provide local protection.

The effect of confining water between levees is (1) to increase the velocity through the leveed section, (2) to increase the water surface elevation during floods, (3) to increase the maximum discharge at all points downstream, (4) to increase the rate of travel of the floodwave, and (5) to decrease the surface slope of the stream above.[9] Levees for protection of local areas have less effect on flood-flow; however, the end result of any levee system is a reduction in valley storage.

The location, spacing, and height of levees must be adjusted to provide adequate capacity between the levees, to provide protection to the flood plain area, and to be economical in cost. The design and construction of levees is discussed in Chapter 12.

PREVENTATIVE MAINTENANCE

Preventative measures for maintaining the capacity of the stream channel include those which affect erosion in the channel itself and those which reduce sediment from upper tributaries. Maintenance in the channel is required to prevent the collection of debris and to reduce sediment from caving banks.

13.12. Bank Protection. The two classes of bank protection are: (1) those which retard the flow along banks and cause deposition and (2) those which cover the banks and prevent erosion.

Retarding the flow along stream banks is desirable to control meandering, to protect the bank, thereby reducing deposition below, and to protect highways, railroads, and other structures near the channel. A common method of control is to build retards extending into the stream from the banks. Materials to construct these retards include piles, trees, rocks, and steel framing. Such retards, sometimes referred to as jetties, serve to decrease the velocity along the concave bank and, hence, increase deposition of sediment.

A method of locating retards is shown in Fig. 13.6. The first major retard at A is located by the intersection of the projected center line of flow with the concave bank. In locating the second major retard C a line HB is drawn parallel to the above projected center line and through the end of the retard A. The intersection of this line with the concave bank locates point B. AC is then made equal to twice AB. Additional retards are located by the intersection of a line connecting the end points of the two

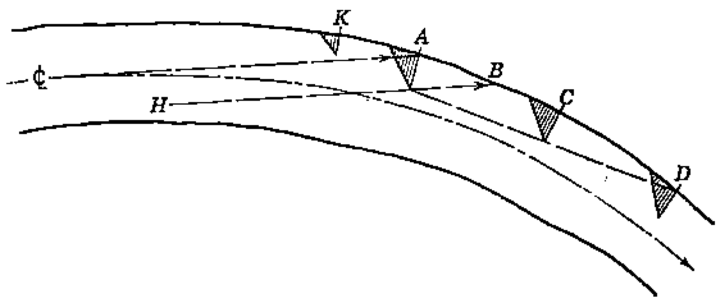

Fig. 13.6. Design and location of retards. (Redrawn from Saveson and Overholt.[12])

previous retards with the concave bank (see D). An auxiliary retard at K is located a distance AB upstream from A and is extended into the stream about one-half the length of the other retards. The retards should extend into the stream at an angle of 45 degrees for a distance of about 30 per cent of the channel width.[12] On small streams the spacing of the retards may be made equal to the stream width, and the length, 0.25 the spacing.[1] On 30-degree curves or over, continuous bank protection should be provided rather than retards.

Vegetative or mechanical control measures are methods of preventing stream-bank erosion. Plants suitable for vegetative control are grass, shrubs, and trees. Mechanical measures to cover the stream bank include such devices as wood and concrete mattresses, rock or stone, asphalt, and sacked or monolithic concrete.

13.13. Reduction of Sediment and Debris. Sediment and debris in stream channels can be reduced by deposition in suitable settling basins or by land treatment.

Sediment from high-velocity streams in cultivated watersheds is deposited on flood plain areas and in the stream channels. Such sediment reduces the effectiveness of drainage ditches and the productivity of agricultural land. Although settling basins are often satisfactory, good land treatment accompanied by channel cleanout may be more practical.

Sedimentation and debris basins have three essential features: an inlet, a settling basin, and an outlet. Sediment-laden water from a stream may be diverted into a large settling basin where a portion of the sediment is deposited as a result of greatly reduced velocities. At the lower end of the basin the flow is then returned to the stream channel. Such settling basins are eventually filled with sediment, thus necessitating the use of a new area. In western Iowa settling basins have been used to reduce sedimentation in channels across a wide flood plain.

The barrier system of removing debris and sediment from mountain streams was developed in Utah.[17] Large debris is deposited as the flood spreads out at the mouth of the canyon and the finer material settles out in a settling basin. Additional features of the system consist of (1) a barrier or cross dike, (2) lateral dikes, and (3) temporary drift dams.

FLOOD ROUTING

Flood routing is the process of determining the reservoir stage, storage volume, and outflow rate corresponding to a particular hydrograph of inflow. Flood routing procedure may be applied to detention and storage reservoirs as well as for large streams.

13.14. Principles of Flood Routing. Flood routing involves inflow into the reservoir, outflow through the structures in the dam, and storage in the reservoir. In designing reservoirs it is necessary to know the height of the dam and the capacity of outlet structures. The problem in flood routing is to determine the relationship among inflow, outflow, and storage as a function of time. This problem can be solved by the following continuity equation for unsteady flow:

$$i\,dt = o\,dt + s\,dt \qquad (13.1)$$

where i = inflow rate for a small increment of time.
o = outflow rate for a small increment of time.
s = rate of storage for a small increment of time.
dt = a small increment of time.

This equation must be satisfied at any and all times during the period from the beginning of inflow until outflow has stopped. Hence, it must also be satisfied for any given time interval between the above limits. Equation 13.1 may also be written:

$$t\frac{i_1 + i_2}{2} = t\frac{o_1 + o_2}{2} + S \tag{13.2}$$

where t = any time interval.
S = change in volume of storage during time t.
$_1$ and $_2$ = subscripts denoting beginning and end of the time interval.

The assumption here is that the water surface in the reservoir is level, and evaporation and seepage losses are negligible. In large reservoirs there may be considerable backwater effect, increasing reservoir storage, but this effect is negligible in small headwater reservoirs.

13.15. Methods of Flood Routing. Of the two general methods of flood routing,[10] the first method involves the division of the inflow hydrograph into time intervals of short duration so that during each period inflow and outflow rates may be assumed to be constant. Available spillway storage and spillway discharge curves must be known. Either observed or developed inflow hydrographs are necessary in the solution. The second method involves analytical integration procedures in which a given flood hydrograph is replaced by an equivalent flood of uniform intensity. The available storage curve is represented by an empirical formula, and it is necessary to know only the exponent of the spillway discharge curve. The first flood routing method gives more accurate results, although analytical integration methods provide a more direct solution to the continuity equation.

The many procedures for solving equation 13.1 include the analytical, the graphical, the mechanical, and the electrical. Mechanical procedures, including slide rules and flood routing machines, and electrical procedures, such as electronic flood routing machines, have been developed.[6] Analytical and graphical procedures are similar, and many of them are basically trial-and-error solutions.

13.16. Elements of Flood Routing. In any flood routing procedure the factors that must be considered are (1) inflow hydrograph, (2) spillway discharge, (3) available spillway storage, and (4) outflow hydrograph.

Inflow Hydrograph. Inflow hydrographs are the same as runoff hydrographs, discussed in Chapter 4.

Spillway Discharge. The spillway discharge curve represents the depth-discharge relation of the mechanical spillway. The rate of discharge is influenced by the hydraulic head and by the type and size of spillway. In soil and water conservation engineering the most common types of structures for detention reservoirs are box inlets, orifices, and pipe conduits (see Chapter 11).

In detention reservoirs box-type drop inlets, orifices, or both are satisfactory for intake structures; however, a pipe or rectangular conduit is required to carry the water through the dam. At low heads orifice or weir flow controls the discharge, but, when the capacity of the discharge pipe is reached, the vertical riser fills with water and pipe flow governs the discharge.

Since the selection of the mechanical spillway is a trial-and-error solution, it is convenient to estimate the approximate size of the outlet tube. With peak runoff, total runoff, drainage area, and available storage known, the rate of outflow and consequently the approximate size of the pipe may be determined by the equation:[4]

$$\frac{Q_o}{Q} = 1.25 - \left[\frac{18V}{RA} + 0.06\right]^{1/2} \tag{13.3}$$

where Q_o = rate of outflow when the pipe first flows full in cubic feet per second.
Q = peak inflow in cubic feet per second.
V = available storage in acre-feet.
R = runoff in inches.
A = drainage area in acres.

Available Spillway Storage. The available spillway storage curve represents the depth-capacity relation of a reservoir above the elevation of the mechanical spillway. The volume of storage for various stages of the reservoir is determined from the topography of the storage basin. Where considerable accuracy is desired, the volume can be computed from a contour map by either the average end area method or by the prismoidal formula given in Appendix I.

Outflow Hydrograph. The outflow hydrograph shows the rate of outflow (spillway discharge) as a function of time. This hydrograph must be determined by some method of flood routing.

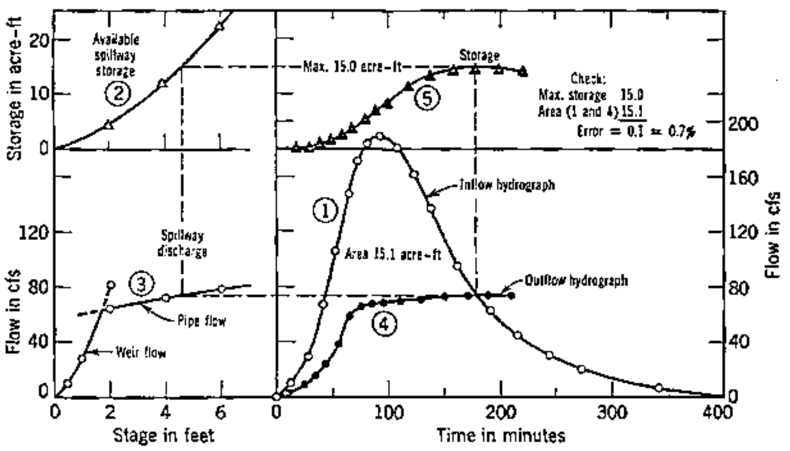

Fig. 13.7. Graphical flood routing procedure.

13.17. Graphical Flood Routing Procedure. The general procedure for graphical flood routing[19] is illustrated in Example 13.1. This method is a graphical integration of the equation:

$$S = \int_0^t (i - 0)\, dt \qquad (13.4)$$

Figure 13.7 shows the inflow hydrograph 1, the available spillway storage curve 2, and the spillway discharge curve 3. These curves are necessary in the development of the outflow hydrograph 4 and the storage curve 5. In Fig. 13.7 it should be noted that curve 3 must have the same scale of ordinate as curve 1 and the same scale of abscissa as curve 2. The scale of abscissae of curves 2 and 3 should be chosen so as to make these curves fairly steep; however, this cannot be accomplished for

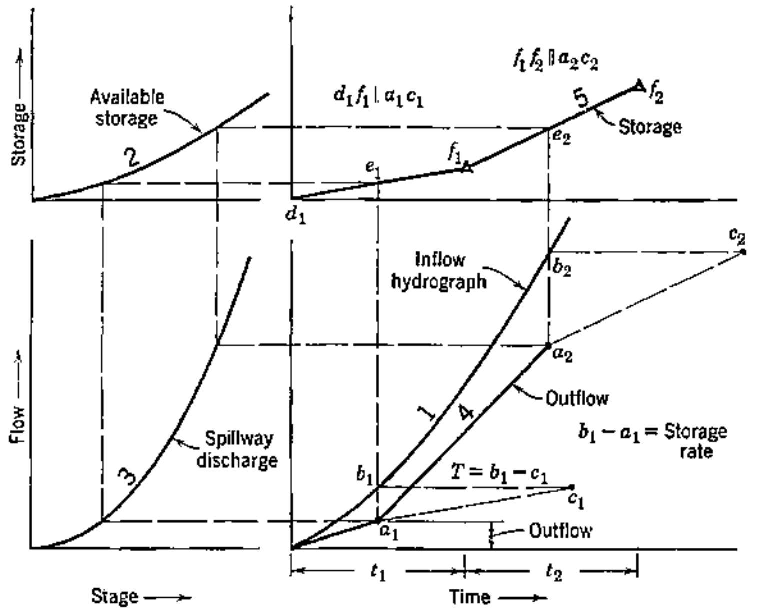

Fig. 13.8. Development of the outflow hydrograph and storage curve.

curve 3, where the spillway is of the pressure conduit or orifice type. The origin for curves 1, 2, and 3 should be selected so that they do not overlap.

The graphical flood routing procedure necessitates the computation of a conversion-time interval T in order to transfer from the flow curve to the storage curve. This conversion-time interval can be obtained from the equation:

$$T = \frac{(\text{1 unit on storage scale in acre-ft}) \times 43{,}560}{(\text{1 unit on flow scale in cfs}) \times 60} \tag{13.5}$$

where T = conversion-time interval in minutes. The conversion-time interval is the time required for a flow measured by 1 unit of ordinate on the flow scale to accumulate the same unit of storage on the storage scale.

The steps in determining curves 4 and 5 are:

1. On the hydrograph time scale, select a short interval t_1 as shown in Fig. 13.8. Assume an average rate of outflow for t_1, and plot it as point a_1 at the midpoint of the time interval.

It is not ordinarily necessary to select time intervals of less than 2.5 per cent of the total runoff period. A shorter time interval may be required where there are sharp breaks in the curve.

2. From point b_1 on curve 1 directly above a_1, measure the distance T horizontally to the right, thereby locating point c_1. Point b_1 is the average rate of inflow into the reservoir during the time interval t_1. The distance a_1b_1 represents storage during the time interval t_1.

3. The slope of the line a_1c_1 represents the average rate of storage for the time interval t_1. A flow equal to b_1 minus a_1 measured on the flow scale will in time T accumulate an amount of spillway storage equal to f_1 measured on the storage curve.

4. In Fig. 13.7, locate point d_1 at the origin of storage curve 5.

5. From point d_1 draw a line parallel to line a_1c_1. Locate point e_1 at the midpoint of the time interval on this line as shown in Fig. 13.8.

6. The accuracy of the assumption of point a_1 can be checked as follows: From e_1 project horizontally to the left to curve 2, then vertically downward to curve 3, and then horizontally to the right to a vertical line through point a_1. If this last projection intersects the vertical line through a_1 at a_1, the assumption of the value of a_1 was correct. If it does not intersect at point a_1, then a new trial value of a_1 must be selected and the above procedure repeated.

7. Continue the graphical solution by selecting a new time interval t_2, and repeat the steps described above until the outflow hydrograph intersects the inflow hydrograph. It should be noted that true points on the outflow hydrograph fall at the midpoints of the selected time intervals t_1, etc., and the true points on the storage curve occur at the end of the time intervals. The point at which the outflow and inflow hydrographs intersect will be the maximum value of the outflow hydrograph, and directly above this point the storage curve will have zero slope and reach a maximum.

The accuracy of the graphical construction (Fig. 13.7) can be checked by comparing the volume of storage from the area between curves 1 and 4 with the maximum spillway storage as obtained from curve 5. The results should check within 1 per cent.

After curves 4 and 5 are completed, the maximum water stage in the reservoir, the maximum outflow discharge, and the maximum storage can be obtained. The reduction in peak flow can be determined from the difference in peaks between the outflow and inflow hydrographs. The maximum water level in the reservoir as obtained from curve 2 is the elevation for the bottom of the emergency spillway. Such routing procedures are adapted to detention reservoirs, farm ponds, and other structures that have considerable storage capacity above the mechanical spillway.

Example 13.1. Design a combination flood control reservoir and farm pond for a site that has a drainage area of 120 acres. The total runoff for a 50-year recurrence interval is 3.5 inches, assuming an infiltration rate of 0.2 iph and the peak runoff of 190 cfs. A depth of 8 feet in the pond is available below an elevation of 96 feet. The storage capacity of the reservoir above 96 feet is shown by curve 2 in Fig. 13.7. A box-inlet spillway and circular concrete outlet pipe are to be used in the outlet structure. The outflow of the pipe when first flowing full should be about 65 cfs. By graphical flood routing procedure, determine the size of the outlet structure, the volume of storage in the reservoir available for flood control, the maximum water stage, elevation of the emergency spillway, and maximum height of the dam, allowing a net freeboard of 3 feet and a flow depth of 1 foot in the emergency spillway.

Solution. Assuming a stage of 5.0 feet above a crest elevation of 96 feet, the maximum storage from curve 2, Fig. 13.7, is 17.0 acre-feet. From equation 13.3 the rate of outflow when the pipe first flows full is

$$\frac{Q_o}{190} = 1.25 - \left[\frac{18 \times 17}{3.5 \times 120} + 0.06\right]^{1/2}$$

$$Q_o = 190 \times 0.362 = 68.8 \text{ cfs}$$

This estimate is near enough to the design requirement of 65 cfs. Assume a 3 × 3.5-foot box inlet (crest 9.5 feet) and a 30-inch outlet pipe. (Box-inlet area should be twice the area of the pipe.) Using the weir formula with a C of 3.0 and the pipe flow formula with K_e of 1.0, n, of 0.014, and L of 110 feet, compute the spillway discharge curve 3 in Fig. 13.7. Q_o of curve 3 is 65 cfs and is satisfactory.

Develop the inflow hydrograph. From Chapter 4,

$$V = \frac{3.5}{12} \times 120 = 35 \text{ acre-feet}$$

$$u = 35 \times 0.000303 = 0.0106 \text{ acre-feet}$$

$$w = 0.01 \times 190 = 1.9 \text{ cfs}$$

$$k = 726 \times \frac{0.0106}{1.9} = 4.05 \text{ minutes}$$

Multiply the coordinates of each point on the basic hydrograph (see Chapter 4) by k and w, respectively, for each point, and plot the inflow hydrograph shown in Fig. 13.7. Determine the conversion-time interval,

$$T = \frac{10}{40} \times \frac{43,560}{60} = 181.5 \text{ minutes}$$

By graphical flood routing described in Art. 13.17, develop the outflow hydrograph and storage curve.

From curve 5, available spillway storage is 15.0 acre-feet. From curve 2, maximum stage is 4.6 feet, and elevation of emergency spillway is 100.6 feet. Maximum settled height of dam is $8 + 4.6 + 3 + 1 = 16.6$ feet. Assumption of 5.0 feet of storage is close enough to design depth of 4.6 feet to be satisfactory.

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PROBLEMS

13.1. Before a cutoff was made on a meandering stream the length of the channel around the bend (*BGE* in Fig. 13.5) was 4200 feet, and the stream gradient was 0.08 per cent. After the cutoff was made, the distance *BE* was 2100 feet. If the velocity in the old channel was 2 fps, how much has the cutoff reduced the time of flow from *B* to *E*, assuming the same hydraulic radius and roughness coefficient for the old and the new channel?

13.2. Design a system of retards for a stream 50 feet wide where the channel makes a 45-degree turn on an 8-degree curve. Determine the length and spacing of retards by making a scale drawing of the stream.

13.3. If the storage scale is 20 acre-feet per inch and the flow scale is 100 cfs per inch in graphical flood routing, what is the conversion-time interval? How long will it take for a flow of 100 cfs to store 100 acre-feet?

13.4. By graphical flood routing, determine the maximum water level for the reservoir in Example 13.1, using all available storage for flood control. Elevation of the crest of the box inlet is 88 feet, and the elevation of the center of the pipe at the outlet is 84 feet. The accumulated storage available at each 2-foot stage above the crest is 0, 0.5, 1.4, 3.3, 6.2, 10.7, 17.0, and 26.1 acre-feet. Use a 3 × 3-foot box inlet and 24-inch-diameter pipe having the same coefficients and length of pipe as in example 13.1.

13.5. Design the outlet structure for Problem 13.4, using the hydrograph in Fig. 4.6 and the runoff volume and peak 3.18 inches and 216 cfs, respectively. Maximum elevation of the emergency spillway (maximum water level in the reservoir) should not exceed 99 feet.

CHAPTER 14

Field Surface Drainage

Surface drainage is the oldest, most widely accepted, and usually the most economical method of removing excessive water from the land. It is especially suited to tight soils which are impracticable to drain by subsurface methods and to areas where large quantities of water must be handled. Surface drainage may be accomplished with large outlet channels called open ditches and small field ditches for draining individual fields of excess surface water and for supplementing tile drainage. The design, construction, and maintenance of small field ditches will be considered in this chapter, and open ditches are discussed in Chapter 15.

The selection of surface drainage facilities for individual field areas depends largely on the topography, soil characteristics, crops, and availability of suitable outlets. Since topography plays such an important role in the design and layout, this chapter has been subdivided into surface drainage of ponded areas, of flat fields, and of sloping land. Ponded areas are frequently found in glaciated regions where the topography is relatively flat and geologic erosion has not had time to develop natural outlets. Flat or level land having impermeable subsoils with shallow topsoil frequently requires surface drainage because tiling is not practicable or economical. Claypan or tight alluvial soils are examples. On these flat fields water may accumulate because of excess rainfall, flooding from uplands, or overflow from streams. Sloping land may be wet because of poor internal drainage or hillside seeps. The importance of these problems is indicated by Beauchamp[3] who stated that the 8 states in the upper Mississippi region contained approximately 5,000,000 acres of tight soil on which surface drainage is needed.

Excessive surface water can be removed by one or more of the following processes: drainage by natural or constructed channels, by infiltration, or by evaporation and transpiration. Evaporation is usually inadequate, and, if the soil is impervious, surface drainage is the only remaining method. It must be remembered that shallow surface ditches cannot remove subsurface water and give the benefits incident to good tile drainage discussed in Chapter 16. Surface drainage may be required even though tile drainage is possible and good soil management practices are carried out.

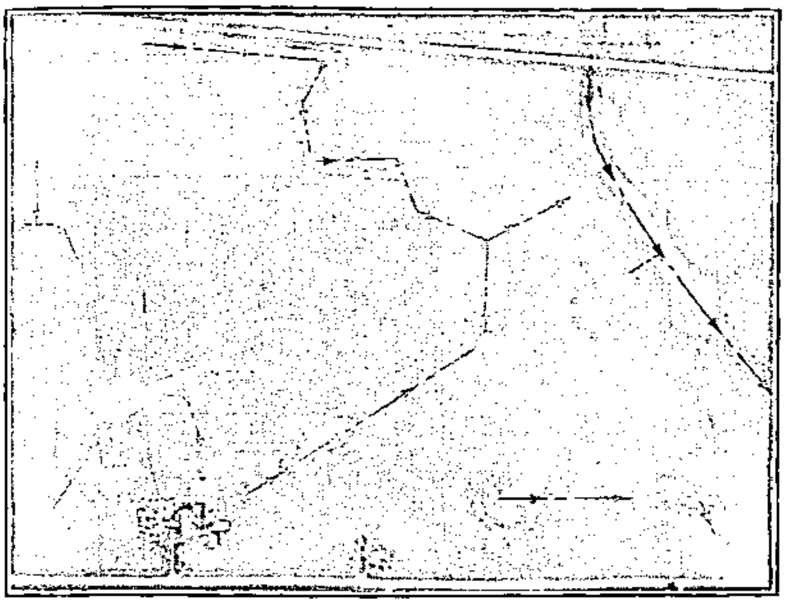

Fig. 14.1. Plan of a random field ditch system for surface drainage. (From Barnes.[2])

SURFACE DRAINAGE OF PONDED AREAS

Surface water from ponded areas may be removed by surface field ditches or by subsurface drains.

14.1. Random Field Ditches. Field ditches are here defined as shallow ditches with flat side slopes that can be crossed with farm machinery. These ditches are best suited to the drainage of scattered depressions or potholes where the depth of cut is not over 3 feet and the length of cut is not excessive. Such field ditches are adapted to the drainage of one or more potholes as shown in Fig. 14.1.

The design of field ditches is very similar to the design of grass waterways, as discussed in Chapter 9. Where farming operations cross the ditch, the side slopes should be flat; that is, 8:1 or greater for depths of 1 foot or less and 10:1 or greater for depths over 2 feet. Minimum side slopes of 4:1 are possible if the field is farmed parallel to the ditch.[3] The depth is determined primarily by the topography of the area, outlet conditions, and the capacity of the channel. A minimum cross-sectional area of 5 square feet is recommended.[3] The grade in the channel should be such that the velocity does not cause erosion or sedimentation. Maximum allowable velocities for various soil conditions are given in Chapter 15. Minimum velocities vary with the depth of flow; however, these range from about 1 to 2 feet per second for depths of flow less than 3 feet. Under Iowa conditions the maximum grade for sandy soil is 0.15 per cent and for clay soils 0.20 per cent. The roughness coefficient for field ditches may be taken as 0.04 if more reliable coefficients are not available. The capacity of the ditch is usually not considered for areas less than 5 acres provided the minimum design specifications are met. However, where the area is larger than 5 acres, the capacity should be based on a 10-year recurrence interval storm, allowance for minimum infiltration and interception losses being made. Since most field crops are able to withstand inundation for only a short period of time without damage, it is desirable to remove surface water within 12 hours.

The layout of a typical random field ditch system is shown in Fig. 14.1. The natural depression areas are indicated as well as the center line of the drains. Normally, the channel should follow a route that provides minimum cut and least interference with farming operations. Where possible, it is desirable to drain several potholes with one ditch. The outlet for such a system may be a natural stream, constructed drainage ditch, or protected slope if no suitable ditch is available. Where the outlet is a broad, flat slope, the water is permitted to spread out on the land below. This type of outlet is practical if the drainage area is small.

14.2. Subsurface Methods. The three principal methods of draining ponded areas by subsurface means are: a surface inlet to tile, a tube outlet, and pumping from a sump. Vertical outlet drains are sometimes feasible in special situations where the true water table is low and where an impervious layer is underlain by pervious sand, gravel, or rock formations. In New

Jersey, vertical tile outlets and holes filled with porous material (extending from the surface through the impermeable soil) are recommended where suitable porous strata are present.[5] Such outlets, however, are not generally practical. The surface drainage of potholes is frequently accomplished by tile surface inlets as described in Chapter 17. However, in many situations the tile outlet is not of sufficient depth to permit a surface inlet, or an existing tile is too small to provide adequate drainage. Where a surface inlet is not practical and field ditches are not feasible because of excessive cut, the ponded area may be drained

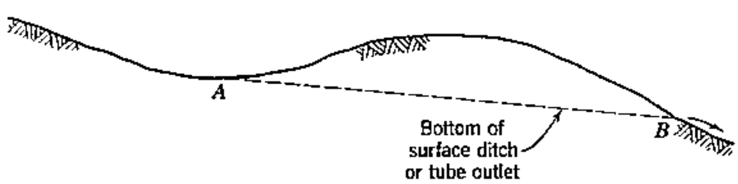

Fig. 14.2. Draining a ponded area with a surface ditch or tube outlet.

with a tube outlet. The grade line for the tile is indicated in Fig. 14.2 as AB. To be practical suitable inlet and outlet structures should be installed at A and B, respectively. Where the depth of cut is excessive for either of these methods, pumping may be practicable. The water may be pumped into a nearby tile drain or over the adjoining ridge.

14.3. Construction. The selection of equipment and procedure for construction varies with the depth of cut and quantity and distribution of excavated soil. For shallow cuts up to 1 foot, moldboard and disk plows, small scrapers, blade graders, and other light equipment can be used to advantage. For depths of cut up to 2.5 feet, such equipment as motor patrols, scrapers, and heavier terracing machines are suitable. Construction of ditches by the above methods may require spreading the spoil from both sides of the ditch to prevent ponding back of the spoil. For deep cuts over 2.5 feet, bulldozers equipped with push or pull back blades and carryall scrapers are most suitable. They may be used to fill the pothole area or other depressions near the point of excavation. Filling of small potholes reduces the area of ponding. Such heavier equipment may be suitable for shallower cuts if the job is large enough. In soil too wet to be handled with machinery, blasting may be practicable.

The field layout of a surface drain is shown in Fig. 14.3. Stakes are usually set every 50 feet along the center line. A

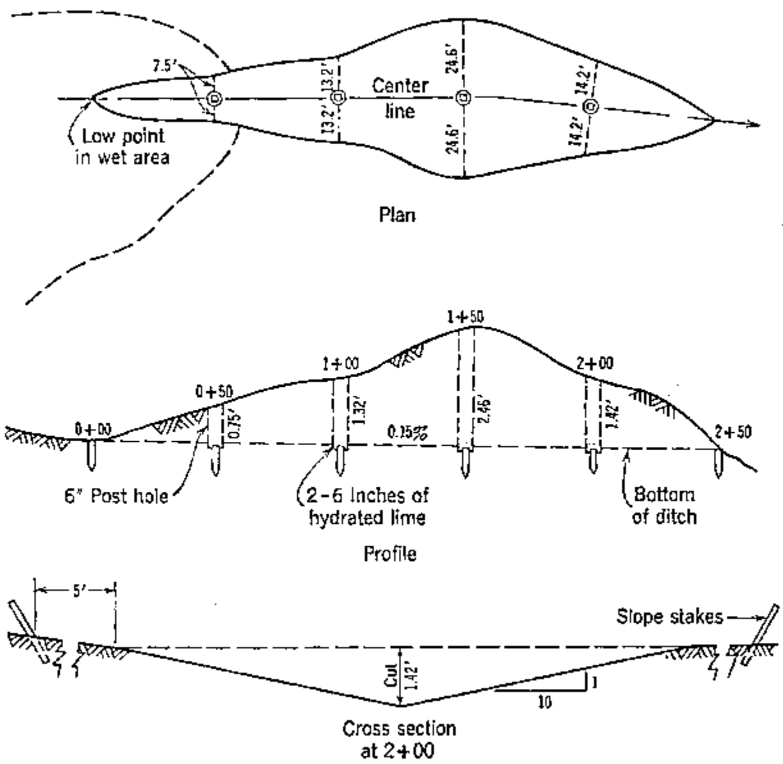

Fig. 14.3. Construction layout for a surface drain.

post hole may be dug at each station, a grade stake set in the bottom of the hole, and 3 or 4 inches of lime or other light-colored material placed in the bottom. As construction progresses, the appearance of lime indicates that the drain is approaching grade. Slope stakes are placed 5 feet farther out from the center line than the computed distance shown in the plan view. Another method of layout is to offset the stake line from the center line and to establish hub and guard stakes as described in Art. 17.26 for tile drains. As construction proceeds, the depth of cut measured from the hub stake can be checked with a hand level or by sight bars.

14.4. Maintenance. Such field ditches can usually be maintained by normal tillage practices. The plow should be raised when crossing the field ditch, and, after the entire field is completed, the ditch area should be plowed out, the deadfurrow being left in the channel. Other tillage operations may be performed in any direction but should be parallel to as many ditches as practical. Minor depressions not needing ditch drainage may require land smoothing or leveling. These operations will be discussed later.

SURFACE DRAINAGE OF FLAT LAND

Flat land is here considered as having slopes of less than 2 per cent, the major portion of which is less than 1 per cent. The two primary methods of surface drainage for flat land are: land smoothing and field ditching. The field ditch systems are further divided into (1) bedding, where the plow deadfurrows serve as drainage ditches; (2) parallel field ditches having larger cross-sectional areas and wider spacings than the deadfurrows in bedding; and (3) parallel lateral open ditches several feet in depth and with spacings similar to (2).

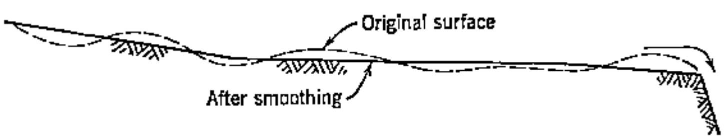

Fig. 14.4. Principle of land smoothing.

14.5. Land Smoothing.[4,9] Land smoothing, sometimes called land forming or grading, is the operation of producing a plane land surface with a continuous slope. Land smoothing has often been misnamed land leveling. Land smoothing is generally recommended as a supplement to field surface drainage or irrigation.

The principle of land smoothing is shown in Fig. 14.4 with the vertical scale exaggerated. The objective is to remove the high areas and fill the low spots so as to produce a surface with a continuous slope. All tillage operations should be performed parallel to the slope, and care should be taken to prevent the formation of ridges, particularly at the turn area near field ditches.

Surface drainage by land smoothing alone except for necessary outlet ditches is now being considered in many areas. Land smoothing for surface drainage is applicable only on areas having small depressions, although on very flat land a grade may be built up by smoothing operations. If large potholes are present, they may be easily drained with random ditches. In land smoothing, leveling equipment should go over the area several times. For example, levelers should be operated in both directions across the field and then diagonally both ways. Smoothing operations may be required for several years since fill material has a tendency to settle.

14.6. Bedding. Bedding is a method of surface drainage consisting of narrow-width plow lands in which the deadfurrows run parallel to the prevailing land slope. The area between two adjacent deadfurrows is known as a bed. Bedding is most practicable on flat slopes of less than 1.5 per cent where the soils are slowly permeable and tile drainage is not practicable.

The design and layout of a bedding system involves the proper spacing of deadfurrows, depth of bed, and grade in the channel. The width of bed depends on the land slope, drainage characteristics of the soil, and cropping system. Bed widths recommended by Beauchamp[3] for the upper Mississippi region vary from 23 to 37 feet for very slow internal drainage, from 44 to 51 feet for slow internal drainage, and from 58 to 93 feet for fair internal drainage. Milner[7] states that a 9-inch depth of bed is ideal for Ohio conditions. However, this depth depends on the soil characteristics and tillage practices. As shown in Fig. 14.5, the depth may vary from 0.5 foot to 1.5 feet, allowing one-half of this depth for the deadfurrow. The grade in the channel must be continuous without low spots or back fall. Since the land is flat, very little deviation from true grade is permissible. The length of the beds may vary from 300 to 1000 feet.[3] If the field is longer, an additional collection ditch should be installed. Where the collection ditch is parallel to a fence line, a turn strip approximately one-half the bed width should be provided. Tillage and row direction in the turn strip should be parallel to the fence line or the collection ditch. In the bedded area the direction of farming may be parallel or normal to the deadfurrows. The purpose of the narrow bed width is to permit movement of water laterally to the deadfurrows. Tillage practices parallel to the beds have a tendency to retard such

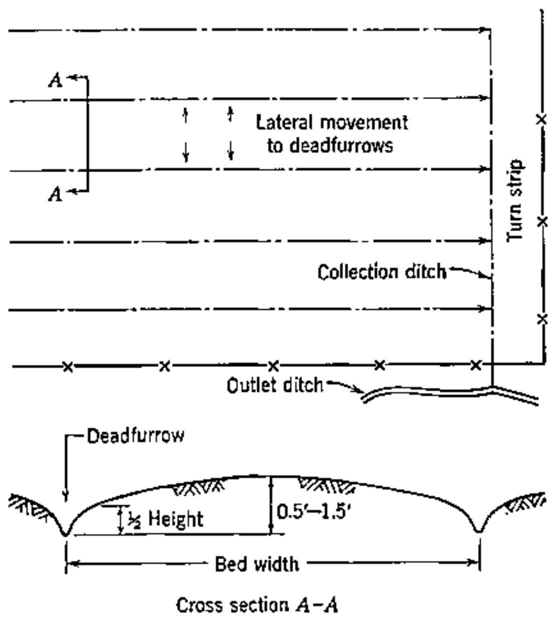

Fig. 14.5. Bedding system of surface drainage.

movement. Plowing is always parallel to the deadfurrows. Sometimes the channels are grassed, but this practice is usually undesirable because water movement is retarded and sediment tends to accumulate in the channel.

14.7. Parallel Field Ditch System. Parallel field ditches are similar to bedding except that the drains are spaced farther apart and have a greater capacity than the deadfurrows. This system is well adapted to flat, poorly drained soils with numerous small depressions which must be removed by land smoothing.

The design and layout is similar to that for bedding except that the drains need not be equally spaced and the water may move to only one of the ditches The layout of such a field system is shown in Fig. 14.6. As in bedding, the turn strip is provided where the ditches border a fence line. The size of the ditch may be varied, depending on grade, soil, and drainage area. The depth of the ditch should be a minimum of 0.75 foot and have a minimum cross-sectional area of 5 square feet.[3] The side slopes should be 8:1 or flatter to facilitate crossing with farm machinery. As in bedding, plowing operations must be parallel

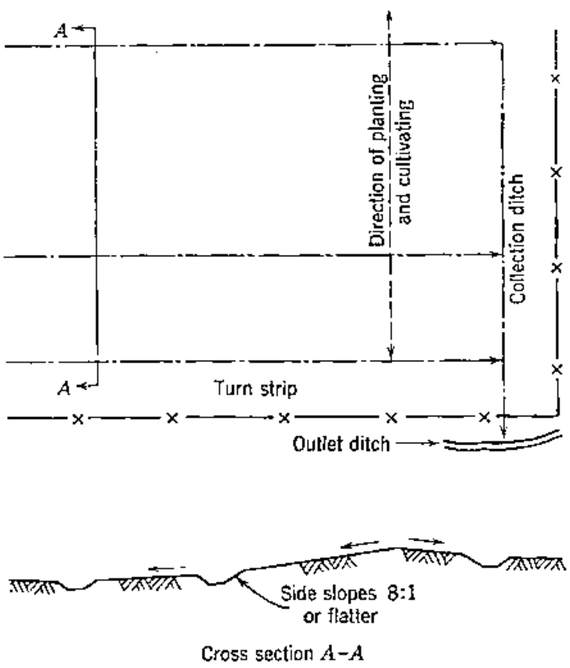

Fig. 14.6. Parallel field ditch system of surface drainage. (Adapted from Beauchamp.[3])

to the ditches, but planting, cultivating, and harvesting are normally perpendicular to them. The rows should have a continuous slope to the ditches. The maximum length for rows having a continuous slope in one direction is 600 feet,[3] allowing a maximum spacing of 1200 feet where the rows drain in both directions. In very flat land with little or no slope, some of the excavated soil may be used to provide the necessary grade. However, the length and grade of the rows should be limited so as to prevent damage by erosion. On highly erosive soils which are slowly permeable, the slope length (ditch spacing) should be reduced to 300 feet or less.

The cross section for field ditches may be V-shaped, trapezoidal, or parabolic. The W-ditch shown in Fig. 14.7 is essentially two parallel single ditches with a narrow spacing. All of the soil is placed between the ditches, making the cross section similar to that of a road. The advantages of the W-ditch are: (1) it allows better row drainage because spoil does not have to be spread; (2) it may be used as a turn row; (3) it may serve as a field road; (4) it can be constructed and maintained with ordi-

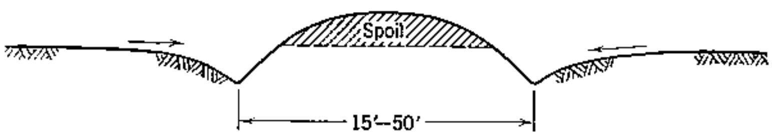

Fig. 14.7. W-ditch for surface drainage.

nary farm equipment; and (5) it may be seeded to grass or row crops. The disadvantages of the W-ditch are: (1) the spoil is not available for filling depressions; (2) a greater quantity of soil must be moved; and (3) a larger area is occupied by ditches. The minimum spacing for the W-ditches varies from about 15 to 50 feet, depending on the size of the drains. The cross-sectional area and shape of the W-ditch is nearly the same as for a single ditch. The W-ditch is best adapted to relatively flat land where the rows drain toward the ditch from both directions.

14.8. Parallel Lateral Open Ditch System. The parallel lateral open ditch system is similar to the field ditch system except that the ditches are deeper and cannot be crossed with most machinery. These drains, illustrated in Fig. 14.8, are arbitrarily called open ditches and are distinguished from field ditches. For clarity the minimum size for open ditches is here given as 2 feet deep and side slopes of 4:1 or less. The purpose of lateral open ditches is to control the ground water table and to provide surface drainage. These ditches are applicable for draining peat and muck soils to provide initial subsidence before tile is installed, and sometimes they are used as a substitute for tile drains.

The design specifications for lateral open ditches applicable to three soil conditions in the upper Mississippi Region are given in Table 14.1. Since lateral open ditches are considerably deeper than collection ditches, overfall protection must be provided at outlets 1 and 2 indicated in Fig. 14.8. This protection may be

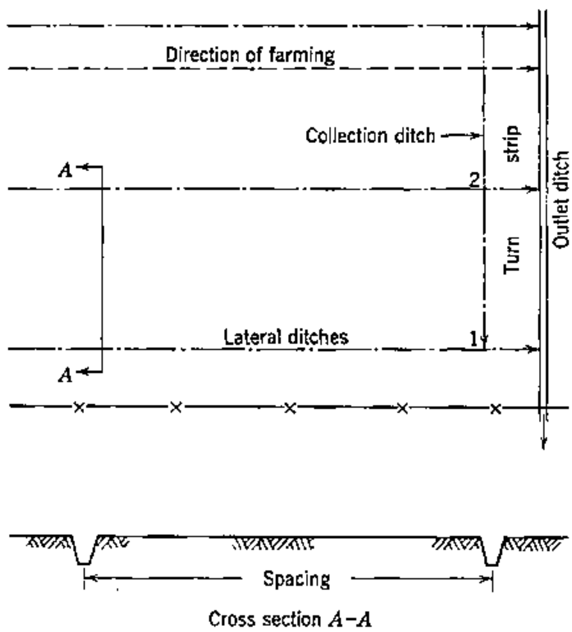

Fig. 14.8. Parallel lateral open ditch system for water table control and surface drainage. (Adapted from Beauchamp.[3])

obtained with a suitable permanent structure, by providing a gradual slope near the outlet, or by establishing a grassed waterway.

Since these drains are too deep to cross with farm machinery, farming operations are parallel to the ditches. A collection ditch or quarter drain should be provided for row drainage. As in other methods of drainage on flat land, the surface must be smoothed and large depressions filled or drained by random field ditches.

For water table control during dry seasons dams with removable sections are placed at various points in the open ditches to maintain the water at the required level. During wet seasons the gates are opened, and the system provides surface drainage. In highly permeable soils, such as sand, peat, and muck, crop yields may be greatly increased by controlled drainage. Deep permeable soils underlain with an impervious material provide the best conditions for successful water table control. The water level may be regulated by gravity, by pumping, or by

Table 14.1 DITCH SPECIFICATIONS FOR WATER TABLE CONTROL*

	Sandy Soil	*Other Mineral Soils*	*Organic Soils, Peat and Muck*
Maximum spacing in feet	660	330	200
Minimum slide slopes	1:1	1½:1	Vertical to 1:1†
Minimum bottom width in feet	4	1	1
Minimum depth in feet	4	2.5	3

* From Beauchamp.[3]

† Vertical for raw peat to 1:1 for decomposed peat and muck.

a combination of gravity drainage and pumping. The depth at which the water table is to be maintained depends largely on the crop to be grown, soil, seasonal conditions including the quantity of water available, topography, and climatic conditions. In organic soils a high water table is desirable to provide water for plant growth, to control subsidence, and to reduce fire and wind erosion hazards.[6] In these soils the water level should be maintained from 1.5 to 4 feet below the surface, depending on the crop.

14.9. Construction. The first step in construction is to smooth the land surface and fill or drain depressions too large to be removed by smoothing. Land levelers[1,9] used in irrigated areas are satisfactory for land smoothing. Bulldozers, motor patrols, carryalls, and scrapers are suitable for filling depressions that cannot be filled by smoothing. For bedding construction, ordinary farm equipment, such as moldboard plows, disks, and drags, are satisfactory. Heavier equipment, such as bulldozers, motor patrols, and carryalls, may be more practical for constructing the parallel field ditches. Equipment and methods of constructing open ditches is discussed in Chapter 15. Outlet and collection ditches should be constructed about 0.5 to 1.0 foot deeper than ditches that drain into them if sufficient grade is available. It may be necessary to dig for grade; that is, the channel grade is greater than the land slope, making the ditch deeper at the lower than at the upper end.

14.10. Maintenance. Surface ditches must be adequately maintained if they are to function properly. Such maintenance is particularly important on flat land since a very small obstruction in the channel may cause flooding of a sizable area. Tillage implements should be lifted when crossing ditches to avoid blocking the channel. If this procedure is not followed, the channel should be opened when needed. When the soil is soggy and wet, equipment should not cross deadfurrows, field ditches, or grass waterways. Livestock may also damage such channels during rainy seasons. Pasturing at other times, however, is desirable. Plowing parallel to shallow surface ditches usually is adequate for maintenance.[7]

SURFACE DRAINAGE OF SLOPING LAND

The drainage of sloping land may be feasible with cross-slope ditches. Such channels usually function both for surface drainage and for erosion control. When designed specifically for the control of erosion, these ditches are called terraces. Diversion ditches (see Chapter 11) are sometimes utilized to divert runoff from low-lying areas, thus reducing the drainage problem.

14.11. Cross-Slope Ditch Systems. The cross-slope ditch system of surface drainage was developed in Wisconsin.[10] It is primarily adapted to soils with poor internal drainage where tiling it not practicable and with slopes of 4 per cent or less having numerous shallow depressions. This land is generally too steep for bedding or field ditches since farming up and down the slope results in excessive erosion.

The design and layout of cross-slope ditches are similar to terraces discussed in Chapter 10. As shown in Fig. 14.9*a*, excess soil not required to fill depressions is spread in a thin layer downslope from the channel. The cross-sectional area ranges from 5 to 8 square feet. In Wisconsin the recommended channel grade varies from 0.1 to 1.0 per cent, but 0.5 per cent is most desirable.[10] In general, the spacing, layout procedure, and maintenance are the same as for terraces. However, on very flat slopes the horizontal spacing probably should not exceed 150 feet.

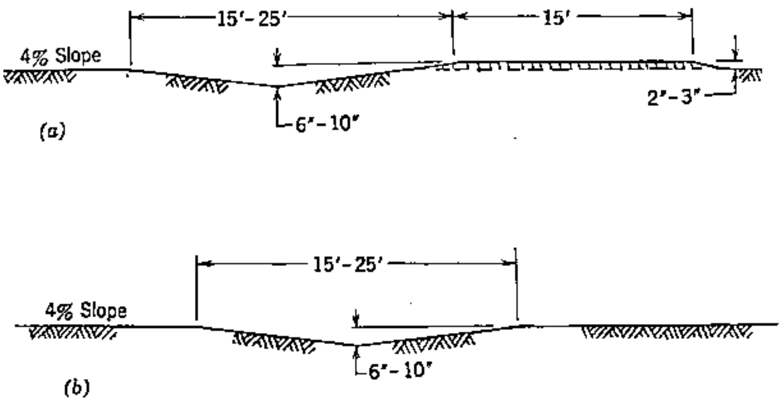

Fig. 14.9. Cross sections of cross-slope ditches. (Redrawn from Wojta.[10])

14.12. Construction and Maintenance. The construction and maintenance of cross-slope ditches are similar to that for terraces, described in Chapter 10. Usually both terracing machines and land-smoothing equipment are required. The flat cross sections are easily crossed with farm machinery. Tillage operations should be parallel to the channel to prevent overtopping since the height of the ridge is usually low.

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PROBLEMS

14.1. How many cubic yards of soil must be moved to construct the surface drain shown in Fig. 14.3?

14.2. If the soil from the surface drain in Fig. 14.3 is to be used to fill a pothole, how long will it take a 45-hp bulldozer to construct the drain, assuming an average haul distance of 200 ft?

CHAPTER 15

Open Ditches

The United States Census of 1950 shows that more than 155,000 miles of open ditches, mostly in organized drainage districts, have been constructed in 38 states.[18] This figure would be much greater if all privately owned drainage ditches were included. Several million acres of fertile land have been drained and total crop production greatly increased. Open ditches provide outlets for tile drains and tributary ditches and remove surface water directly. Open ditch systems generally drain large areas and often involve several property owners.

15.1. Preliminary Survey. The preliminary survey is an initial investigation of the proposed project. Carried out in sufficient detail to provide a tentative design and to make possible a valid estimate of the economic feasibility of the plan, the preliminary survey should include an estimate of the potential productivity of the soil, and physical features of the watershed, as well as rainfall and runoff data. The fertility of the soil, proposed land use of the area, and degree of drainage anticipated largely determine the maximum allowable expenditure. The physical features of the watershed should include (1) outlet possibilities, (2) topography including natural channels, depressions, and lake levels, (3) size of the area, (4) texture and physical condition of the soil, (5) sediment-producing characteristic of the drainage area, (6) bridges, highways, railroads, and canals, (7) property boundaries, and (8) present land use. The distance to natural outlets, the elevation of the outlet, and elevation of the lowest point to be drained are important. All available maps including aerial photographs should be obtained before starting the survey. The field survey work should be checked and sufficient reference points established to eliminate duplication in subsequent surveys. By making soil borings to depths greater than the proposed ditch, texture and other soil characteristics can be determined. The soil properties may affect the shape of the cross section and the design velocities. The size, elevation, and condition of existing structures, such as culverts and tile outlets should be determined.

DRAINAGE REQUIREMENTS

For a given watershed area the required capacity of an open ditch is considerably different than the design capacity of a grassed waterway. Open ditches generally have flatter slopes, lower velocities, steeper side slopes, and a greater depth of channel flow than do grassed waterways. While grassed waterways are designed to carry peak runoff, open ditches are designed to remove water much more slowly, but still rapidly enough to prevent serious damage to the crop.

15.2. Factors Determining the Rate of Water Removal. The rate of removal of water by open ditches is influenced by (1) rainfall, (2) size of the drainage area, (3) runoff characteristics including slope, soil, and vegetation, (4) fertility of the soil, (5) crops, (6) degree of protection warranted, and (7) frequency and height of flood waters from rivers and creeks. Although the degree of protection is very important in design, it is one of the most difficult factors to evaluate because costs must be compared to anticipated flood damages. More frequent flooding is permissible for agricultural land than for homes and building sites. Likewise, timberland requires less intensive drainage than does cultivated land.

15.3. Design Runoff. The runoff for open ditch design may be expressed in three ways: drainage coefficient, cubic feet per second per acre, and cubic feet per second per square mile. The drainage coefficient is defined as the depth of water measured in inches which is to be removed in a 24-hour period from the entire drainage area. Conversion factors for these three units of measurement are given in Appendix H.

A method of determining the required rate of water removal, generally used for larger areas, is by an empirical formula:

$$Q = kM^x \tag{15.1}$$

where Q = runoff in cubic feet per second.
k = a constant.
M = watershed area in square miles.
x = an exponent.

Table 15.1 RUNOFF CONSTANTS FOR OPEN DITCH DESIGN*

	Degree of Drainage	Drainage Area, sq mi	Constants k	Constants x	Remarks
		Southeastern Area			
		0–6	46	0.8	Approx. avg. for delta and coastal areas
		0–6	79	0.93	Hilly to mountainous areas
		Upper Mississippi Area			
A	Excellent	0–100†	150	0.6	Not max. runoff, but good overflow protection
B	Excellent	0–100†	72	0.7	Ohio, Ind., Ill., Ia., Mo., Ky.
C	Very good	0–200†	37	0.83	Ohio, Ind., Ill., Ia., N. Mo.
C	Good	0–200†	37	0.83	Ky., S. Mo.
D	Fair	0–200†	24	0.85	Ohio, Ind., Ill., Ia., N. Mo.
		Northern Great Plains Area			
		2–200	24	0.85	Normal conditions
		Northeastern Area			
		Use Upper Mississippi Area data			
		Western Gulf Area			
		0.5–500	130	0.7	Max. for hill areas
		5–500	45	0.83	Cropland, general
		0.03–125	15	0.83	Range land

* Based on data from Soil Conservation Service, U.S.D.A.

† Data from curves for flat watersheds having average slopes less than 25 feet per mile (approximately 0.5 per cent). (Constants k and x based on design curves.)

The constants k and x vary with the degree of drainage desired and the location (see Table 15.1). Relying on experience and judgment the engineer should modify these data to meet local conditions.

Quite frequently watersheds with upper tributaries having rather steep slopes and high runoff-producing characteristics drain into an alluvial valley. Open ditches are then often necessary to carry the water across the flood plain from the hill area to the natural outlet.

CHANNEL DESIGN

An open ditch properly designed to provide adequate drainage at reasonable cost should provide the following conditions: (1) velocity of flow such that neither serious scouring nor sedimentation will result; (2) sufficient capacity to carry the design flow; (3) hydraulic grade line low enough to drain the land; and (4) side slopes that will not cave or slide into the ditch. The engineer should have a knowledge of the capabilities and limitations of the various types of construction equipment and should consider these factors in the design of the system.

15.4. Grades. The engineer frequently has little choice in the selection of grades for open ditches since the maximum grade is determined largely by the outlet elevation, elevation and distance to the lowest point to be drained, and depth of ditches. Where open ditches drain flat land, the grade should be as steep as possible without exceeding maximum permissible velocities. The grade should also be as uniform as possible without causing excessive cuts and should conform, in general, to the natural land slope along the ditch route.

15.5. Cross Section. The design dimensions of a ditch cross section are shown in Fig. 15.1. Open ditches are designed with trapezoidal cross sections. The size of the ditch will vary with the velocity and quantity of water to be removed. Since the discharge increases toward the outlet, the cross-sectional area of the main for a constant velocity is increased with the drainage area. Where the cross section is enlarged, the change should be gradual so that unnecessary turbulence is not introduced.

Depth. The depth at all points along the channel should be sufficient to adequately drain the area. Where tile drains outlet into the ditch, a minimum depth of from 4 to 6 feet is required. In peat and muck soils the ditch should be made deeper to allow for subsidence. Because of reduced velocities, sediment accumulates more readily and vegetation grows more abundantly in shallow than in deep ditches. In some instances an allowance is made for accumulation of sediment, depending on channel velocities and soil conditions.

Side Slopes. Channel side slopes are determined principally by soil texture and stability. The most critical condition for caving occurs after a rapid drop in the flow level leaves the banks saturated. For the same side slopes, the deeper the ditch

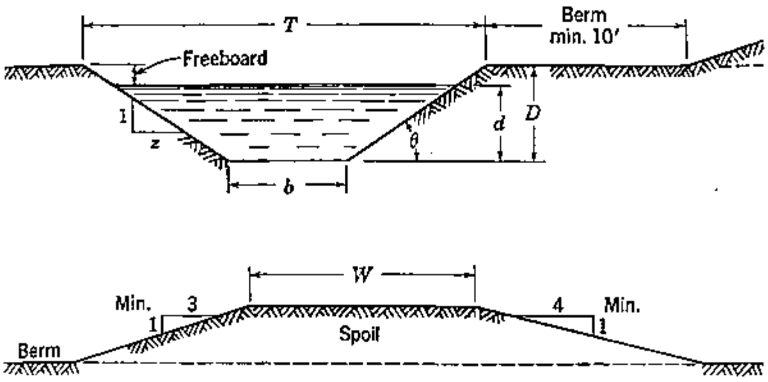

Fig. 15.1. Elements of an open-ditch cross section.

the more likely it is to cave. Side slopes should be designed to suit soil conditions, regardless of the limitations of some construction equipment. Suggested side slopes for different soils and for shallow and deep ditches are shown in Table 15.2. Whenever

Table 15.2 Side Slopes for Open Ditches*

	Side Slopes	
Soil	*Shallow Channels* (up to 4 feet)	*Deep Channels* (4 feet and over)
Peat and muck	Vertical	¼:1
Stiff (heavy) clay	½:1	1:1
Clay or silt loam	1:1	1½:1
Sandy loam	1½:1	2:1
Loose sandy	2:1	3:1

* By permission, from *Land Drainage and Flood Protection,* by Etcheverry,[2] copyright, 1931, McGraw-Hill Book Co.

possible, these slopes should be verified by experience and local practices. Very narrow ditches should have slightly flatter side slopes than wide ditches because of greater reduction in capacity in the narrow ditches should caving occur.

Bottom Width. After the channel grade, depth, and side slopes are selected, the bottom width can be computed for a given discharge. The bottom width for the most efficient cross section and minimum volume of excavation is determined by the formula:

$$b = 2d \tan \frac{\theta}{2} \tag{15.2}$$

where b = bottom width, d = design depth, and θ = side slope angle (see Fig. 15.1). For any side slope it can be shown mathematically that, for a bottom width computed from Equation 15.2, the hydraulic radius is equal to one-half the depth. The minimum bottom width should be 4 feet except in small lateral ditches. It is not always possible to design for the most efficient cross section because of construction equipment limitations, allowable velocities, etc.

Spoil Banks. The excavated soil may be placed on one or both sides of the ditch, depending on the type of equipment and the size of the ditch. If the spoil bank is to serve as a levee, the spoil is normally placed on only one side. Wherever possible, the spoil bank should be spread until it blends into the adjoining field, thus permitting cultivation near the edge of the ditch. Where levees or access roads are desired the spoil should be spread to conform to the cross section in Fig. 15.1. The berm width or the distance from the edge of the ditch to the edge of the spoil should be a minimum of 10 feet for shallow ditches 4 to 5 feet deep. The berm provides a degree of protection against caving and may be used as a place for the ditching machine to operate. The steeper the side slopes, the greater should be the berm width. For deep ditches with side slopes of 1:1 the berm width should be twice the depth, and for side slopes of 2:1 the berm width should be equal to the depth but never less than the minimum of 10 feet. The spoil bank should have a minimum slope of 3:1 on the channel side and 4:1 on the land side. The depth and top width of the spoil bank vary with the size of the ditch. Unless the spoil bank is to be used as a levee, breaks or gaps which allow surface water to drain to the channel should be provided or tube inlets may be installed through the spoil bank (Chapter 11).

15.6. Velocities. The Manning equation has been generally accepted in drainage work (see Appendix C). Experience and knowledge of local conditions are helpful in the selection of the roughness coefficient. In general, a value of 0.035 is satisfactory for medium-sized ditches with bottom widths of 5 to 10 feet, and 0.04 is suitable for smaller ditches with bottom widths of 4 feet.[17]

Typical velocity distribution in an open ditch is shown in Fig. 15.2. The maximum velocity occurs near the center of the stream and slightly below the surface. The average velocity

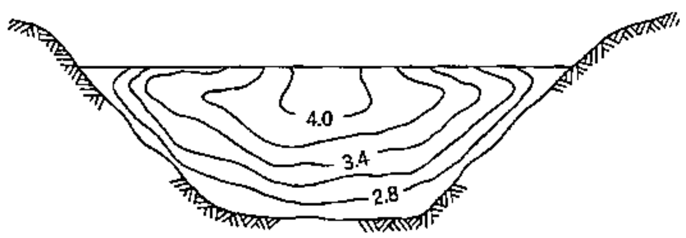

Fig. 15.2. Typical velocity distribution in an open ditch. (Redrawn by permission from *Applied Fluid Mechanics*, by O'Brien and Hickox,[11] copyright, 1937, McGraw-Hill Book Co.)

in this stream, having a rather straight and relatively smooth channel, was 3.32 fps with a maximum of 4.2. Around curves the maximum velocity and the velocity contours move toward the outer bank.

There is no definite optimum velocity for open ditch design, but maximum and minimum limits may be approximated. Velocities should be low enough to prevent scour and, wherever possible, high enough to prevent sedimentation.

A summary of allowable average velocities for open ditch design is given in Table 15.3. The data, obtained from a survey of irrigation canals in western United States,[5] may be used in absence of more suitable information. Greater velocities are allowed for some soils when the stream flow contains sediment because deposition produces a well-graded channel bed resistant to erosion. Where the channel is winding or curved, the maximum velocity should be reduced about 25 per cent.

In open ditch design it is desirable to maintain sufficient velocity to prevent sedimentation in the channel. Usually an average velocity of 2 to 3 fps for shallow channels is sufficient if the maximum allowable velocities are not exceeded. In ditches that flow intermittently, vegetation may retard the flow to such

Table 15.3 ALLOWABLE VELOCITIES FOR OPEN DITCHES*

	Maximum Velocity,† fps, after Aging		
Original Soil	*Clear Water, No Detritus*	*Water Transporting Colloidal Sediment*	*Water Transporting Silts, Sands, Gravels, or Rock Fragments*
Fine sands, and sandy loams	1.5	2.5	1.5
Silt loams, loams, volcanic ash, and noncolloidal sediment	2.0	3.0	2.0
Colloidal clay, colloidal sediment, and fine gravel	2.5	5.0	3.0
Soils graded from silt to cobbles	3.5	5.0	5.0
Coarse gravel, cobbles, shingles, shales, and hardpans	4.0	5.5	5.0

* Based on data from Fortier and Scobey.[5]

† Add 0.5 fps for depths over 3 feet, and subtract 0.5 fps where a powerful abrasive is contained in the water.

an extent that it is difficult if not impossible to maintain adequate velocities at low discharges. For this reason it may be desirable to design for maximum or even higher velocities, since scouring for short periods of time at high flows may aid in maintaining the required cross section.

15.7. Channel Capacity. Open ditches should carry the design runoff from the drainage area. After a trial slope and cross section for the ditch have been selected, the velocity is computed for the design depth. It may be necessary to modify the ditch cross section to provide suitable velocity and capacity (see Example C.1, Appendix C).

The depth of the constructed ditch should provide a freeboard allowance ($D - d$ in Fig. 15.1) of 20 per cent of the total depth, D. The freeboard provides a discharge safety factor as well as allows a reasonable depth for sediment accumulation.

15.8. Alignment. Proper alignment includes the design of straight channels and, where changes in direction are necessary, gradual curves to prevent excessive bank erosion.

The radius of curvature depends on the velocity and stability of the side slopes. If gradual curves will not eliminate erosion in the ditch, it may occasionally be necessary to decrease the velocity by increasing the width or side slopes or to provide bank protection. It is convenient to express the radius of a curve by

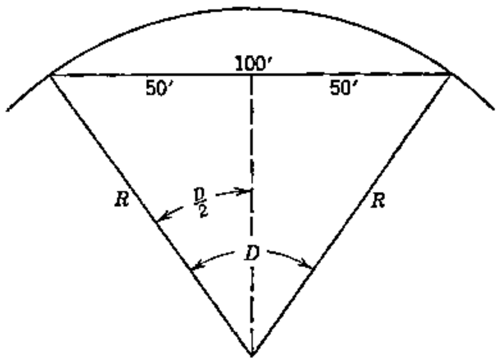

Fig. 15.3. Degree of curve.

the degree of curve, which is defined as the angle subtended at the center of a circle by a 100-foot chord. The relationship between the degree of curvature and radius of curvature shown in Fig. 15.3 is expressed by the equation:

$$R = \frac{50}{\sin D/2} \tag{15.3}$$

where R = the radius of curvature and D = the degree of curvature. Ditch curves usually range from 4 degrees for large-capacity ditches or for channels with steep side slopes to 20 degrees for ditches with small capacity or with relatively flat side slopes.[12]

Circular curves are generally satisfactory in open ditch design. The procedure for laying out such curves is given in Appendix I.

15.9. Junctions. The junction of one ditch with another should be such that serious bank erosion, scour holes, or sedimentation will not occur. Where the general direction of the main is perpendicular to that of the lateral, the junction may be curved near the outlet of the lateral. The angle at which the two channels join should be about 30 degrees.[12] For small channels having low velocities the angle is less important than for larger ditches having higher velocities.

The bottom of the main and the lateral should join at the same elevation. If the lateral is shallower than the main, the overfall at the junction may be eliminated by increasing the grade of the lateral in the last 200 or 300 feet or by increasing the slope on the entire lateral. A steep grade at the junction is not desirable but may be better than having an overfall. Since maximum velocities occur at the higher stages, the increased slope near the outlet may not be serious because the water in the main will back up into the lateral. The most serious condition exists when the main is at low flow and the lateral has a large discharge.

15.10. Controlled Drainage Structures. As discussed in Chapter 14, controlled drainage with open ditches requires structures in the channel to maintain the water at the required level. To provide an economical and satisfactory design, foundation conditions and drainage flow as well as pumping requirements should be considered. Seepage around and below structures is a problem, particularly in peat and muck soils. Control structures should be designed to handle the same rate of water removal as the open ditch. Where gravity outlets cannot be obtained, sumps and pumping facilities must be installed.

Several types of control structures are the burlap bag dam, timber or sheet piling cutoff, reinforced concrete structure, and a combination culvert and control gate. The elevation views of some of these structures are illustrated in Fig. 15.4. Since these structures are designed for organic soils, the cutoff wall is larger than required for more impermeable soils. All control dams should be provided with an erosion-controlling apron constructed below the dam as for a drop spillway. Burlap bag dams with a facing of timber (Fig. 15.4*a*) are suitable for low heads and small drainage areas. These bags are filled with a weak cement mixture and tamped together before wetting. Timber or sheet piling (Fig. 15.4*b*) may be placed to the side and under the ditch. This type of dam provides a greater barrier against seepage and is most suitable in deep organic soils or other highly permeable materials. Reinforced concrete dams (Fig. 15.4*c*) are better

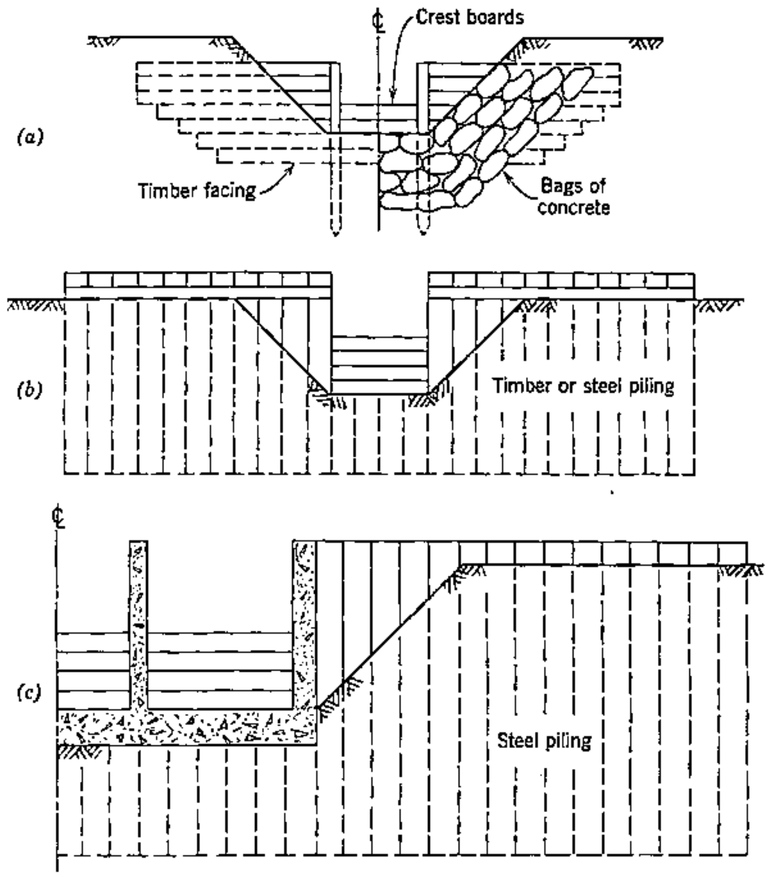

Fig. 15.4. Controlled drainage structures for organic soils. (Adapted from Shafer.[15])

suited for handling large quantities of drainage flow. Where seepage is a problem, piling may be required both under and to the sides of the structure. Combination culvert and control gate structures (not illustrated) with crestboards at the upper end of the conduit may provide an economical and practical dam. All structures are equipped with crestboards which may be placed in the openings when the water level is to be increased or removed when drainage is required.

LAYOUT

Open ditches are arbitrarily designated as mains, submains, laterals, and field ditches. The drainage plan should incorporate as needed levees, pump installations, field and open ditch systems, and tile drains into a coordinated system.

15.11. Location Survey. After the ditch system has been designed and approved, and construction has been authorized, the engineer proceeds with the field layout. The general location and alignment of the channel has normally been determined by the preliminary survey. If the system is small, the preliminary and location surveys may be made at the same time. In staking the field layout, minor changes in location or design are sometimes desirable.

Since the location of open ditches requires experience and good judgment combined with a careful study of local conditions, only a few general rules can be given: (1) follow the general direction of natural drainageways, particularly with mains and submains; (2) provide straight channels with gradual curves, especially for large ditches; (3) locate drains along property lines if practicable; (4) make use of natural or existing ditches as much as possible; (5) use the available grade to best advantage, particularly on flat land; and (6) avoid unstable soils and other natural conditions that increase construction and maintenance costs.

15.12. Staking. In making the location survey prior to construction, center line stakes and slope stakes are set at each station. On level or nearly level topography the offset of the slope stakes from the center line can be easily computed by adding one-half the bottom width plus the side slope ratio (z) times the depth. On irregular land the slope stakes may be set by trial and error as illustrated in Appendix I.

CONSTRUCTION

Open ditch construction includes the excavation of original ditches and the rehabilitation of existing drainage systems. Rehabilitation work may be necessary because of poor design, improper construction, or inadequate maintenance.

15.13. Types of Equipment. As shown in Table 15.4, ditch construction equipment may be classified according to function and the type of earth-moving action involved.[10] In regard

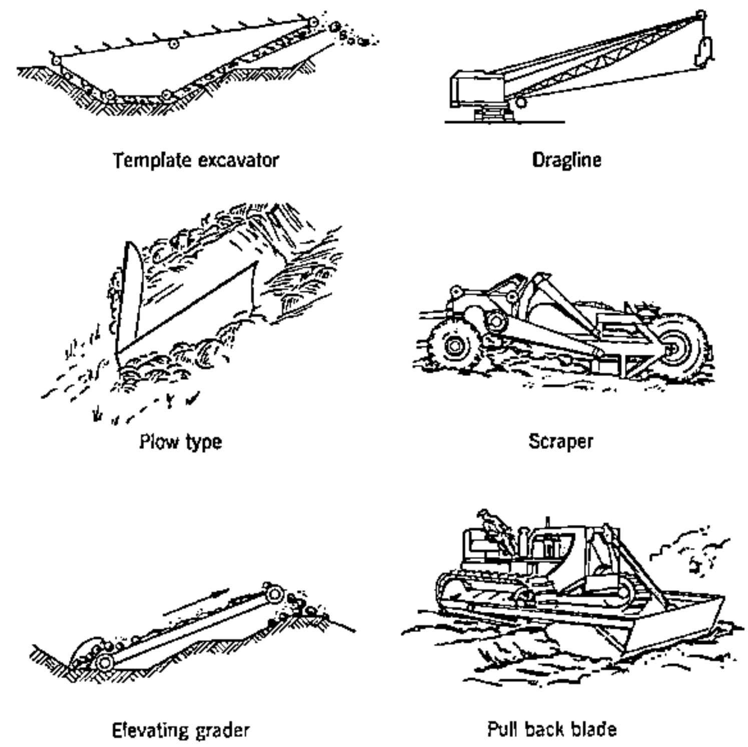

Fig. 15.5. Open-ditch excavation equipment.

to function, construction equipment may either cut and carry the soil or cut, spread, and push the spoil. In regard to type of earth-moving action, it may be either continuous or intermittent. Several types of machines are illustrated in Fig. 15.5.

Continuous-action machines generally have a higher output than intermittent-motion equipment. Examples of continuous-action machines are the wheel and template excavators. The wheel excavator is similar to the wheel-type trenching machine (Fig. 15.5) except that the resulting side slopes are less than vertical. The template excavator digs a somewhat larger ditch

Table 15.4 CLASSIFICATION OF EARTH-MOVING EQUIPMENT FOR OPEN DITCH CONSTRUCTION

	Action	
Function	*Continuous*	*Intermittent*
Excavation	Wheel excavator (trench type)	Dragline (scraper-bucket excavator)
	Plow-type ditcher*	Clamshell
	Template excavator	Hoe
	Blade grader*	Shovel
	Elevating grader*	Scraper
	Hydraulic dredge	Bulldozer
		Pull back blade
Spoil spreading	Blade grader†	Bulldozer†
	Tillage machines*	Scraper
	Terracing machines*	Pull back blade†

* Continuous except for turning at the ends.

† Either continuous or intermittent, depending on the method of operation.

than the wheel-type machine, but both types excavate ditches whose cross sections cannot be varied except by modifying the equipment. Both machines construct the ditch as they move along the ditch line. Plow-type ditchers are similar in construction and operation to an ordinary lister, but the ditcher is a much larger machine. Blade and elevating graders, though not precisely continuous in action, are suitable only in soils where sufficient traction can be obtained in the bottom of the ditch. Continuous-action machines for spreading the spoil include the bulldozer and the blade grader.

Intermittent-action machines are quite generally used for open ditch construction (see Table 15.4). For cleanout work, draglines are sometimes equipped with trapezoidal-shaped buckets, with wide tracks for straddling the ditch, and with a special gear to provide continuous movement of the machine. The hoe (Fig. 18.1) is known by a variety of names, such as the back hoe, trench hoe, and drag hoe. In wet areas these machines may be mounted on floats. As land excavators, they may be equipped with rubber tires or with crawler-type tread. Since scrapers and bulldozers require ditch bottoms that are firm, they are not suitable for most open ditch work. However, tractors with pull back blades are satisfactory. The bulldozer is perhaps the most widely used machine for spreading spoil banks, but scrapers may be satisfactory for small ditches. Such machines as the dragline and clamshell, however, may deposit the spoil so that very little, if any, spreading is required.

15.14. Factors Influencing the Selection of Equipment. The shape and design dimensions of the channel are influenced by the type of equipment available. For maximum efficiency machinery should be selected to fit the requirements of the individual job. In many instances the engineer is called upon to evaluate bids of contractors. Such bids may be rejected if the contractor does not have suitable equipment to do the work.

The three types of operations in earth-work construction are digging, hauling, and placing. A machine that can perform all three operations is desirable since such a machine usually provides the most economical construction. For example, a dragline can dig the soil, move it from the ditch to the spoil bank, and place the spoil in the desired position.

The selection of equipment depends on such factors as moisture conditions, type of soil, degree of accuracy required, shape and dimensions of the channel and the spoil bank, moving requirements, volume of work, and financial considerations. For drainage ditch construction the selection depends largely on moisture conditions and to a degree on the nature of the soil.

15.15. Explosives. Blasting with dynamite may be satisfactory for constructing new ditches or cleaning out old ones in such areas as swamps and wet natural channels that are not readily accessible with earth-moving machinery. It is generally more economical and practical to use machinery than to use explosives. At best, blasting is hazardous work and should be attempted only by experienced and qualified persons.[7]

15.16. Estimating Costs. In preparing open ditch designs, the engineer should determine the economic feasibility of the proposed project. Such an analysis includes a comparison of the costs with the expected benefits. Since costs vary widely from section to section and from year to year, specific data is of little value. Costs for a drainage district project include (1) construction costs, frequently determined by contractor bids; (2) right-of-way costs; (3) damage to land, roads, bridges, fences, and railroads; (4) engineering expenses; (5) attorneys' fees and other legal expenses; and (6) fees of commissioners, assessors, and other district officials. Construction expenses include charges for excavation, bridges, clearing the land of timber and brush, and such auxiliary structures as tile outlets and protective channel work. On an individual farm the expenses are primarily those of construction and engineering. The rate of earthwork construction for bulldozers, scrapers, and draglines is given in Appendix F. Since the data are average values for a large number of conditions, the information should not be used directly unless verified by local conditions. Although it is difficult to estimate actual production time on a percentage basis, many authorities on heavy earth-moving equipment estimate that the actual production time is as low as 30 to 50 per cent of the time actually spent in the field.

MAINTENANCE

In most instances maintenance is a continuing process and should begin soon after construction is completed. Maintenance may be divided into two phases: preventative maintenance before failure, and corrective maintenance after partial or complete failure. For example, good land-use practices on the watershed to reduce erosion are preventative measures, and cleanout of sediment and tree removal are corrective measures.

15.17. Causes for Deterioration of Open Ditches. The failure of open ditch systems results from one or more of three conditions: poor design, improper construction, and lack of adequate maintenance.[8]

The major causes for deterioration of open ditches are (1) sedimentation in the channel; (2) excessive growth of vegetation; (3) channel and bank erosion; (4) improper land use on the watershed; (5) poor location and alignment; (6) improper depth; (7) inadequate culvert and bridge capacity; (8) failure to provide the necessary legal arrangements to fix responsibility and to collect maintenance expenses for drains involving more than one landowner; and (9) general lack of interest in maintenance by the public, landowners, and drainage district officials.

15.18. Preventative Maintenance. Although the best possible design may have been developed and the ditches may have been constructed according to plan, maintenance measures are always necessary for proper functioning of the drainage system.

Such structures as sedimentation basins and diversion ditches reduce the amount of cleanout work in the channels. The control of excessive tree, brush, grass, and weed growth along ditch channels is an important preventative maintenance measure. The effect of weed growth on channel capacity is shown from a survey by Ramser[13] in Missouri, Arkansas, Mississippi, and Illinois. The study showed that ditch capacities were reduced as much as 75 per cent of the original capacity. The primary methods of control consist of spraying, mowing, grubbing and clearing, grazing, and burning. A number of chemical sprays, such as 2,4-D, have largely replaced more costly hand-grubbing and clearing methods. The rate of application, method of application, and effectiveness of chemical sprays are adequately covered in other publications.[3,4] Burning vegetation along ditches may be accomplished when the material is dry; flame burners may be suitable when the vegetation is green. Burning and spraying of ditches may be required several times during the growing season. Since grazing may be detrimental, particularly in ditches with steep side slopes, this practice should be used only with extreme caution.

Bank erosion resulting in channel sedimentation may be reduced by leveling and seeding the spoil bank and by repairing tile outlet failures, surface water inlet tubes, and other evidences of erosion. If the spoil banks are sufficiently flat, they may be farmed with the remainder of the field. In organic soils where wind erosion causes serious filling of the ditches, permanent vegetation (grass) on the spoil banks is satisfactory. Proper land use combined with good conservation practices in the upper tributaries will greatly reduce maintenance of open ditches on the flatter land. There is also danger of sheet erosion on land near the ditches even though the slope may be relatively flat. In most states property owners in the lower tributaries can not legally force landowners on the higher land to use proper conservation practices.

15.19. Corrective Maintenance. After the original installation, changes in cross section, grade, or alignment of the ditch may be necessary for proper functioning of the system. An improper outlet or inadequate grade may cause sedimentation in the channel. Side slopes that cave and fill the ditch need to be reshaped to a more stable slope. Sediment bars or sharp curves causing meandering may necessitate straightening the ditch or flattening the curves. Widening of the ditch and enlarging of culverts and other obstructions may also be necessary. Where serious scouring occurs, drop spillways may be constructed if more economical measures are not adequate.

Even though all possible steps have been taken to reduce sedimentation, cleanout work is often eventually necessary. In general, equipment for constructing the original ditch is suitable. As for ditch construction, moisture conditions largely determine the best type of equipment.

15.20. Maintenance Costs. Because the amount of maintenance varies so widely with different conditions, it is difficult to estimate maintenance costs. Preventative maintenance measures are generally cheaper than corrective maintenance measures, and likewise timely maintenance is more economical than delayed maintenance activity. Because of the small volume of soil to be moved, cleanout work per unit volume may be three to five times the original excavation cost.[1] Where maintenance has been badly neglected, it may cost more to reclaim an old ditch than to construct a new one.

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PROBLEMS

15.1. Determine the design runoff for an open ditch to provide good drainage in your area for a watershed of 10 acres, 100 acres, and 10 square miles.

15.2. Compute the most efficient bottom width for an open ditch to carry a flow 8 feet deep in silt loam soil. What are the velocity and the channel capacity if the hydraulic gradient is 0.09 per cent?

15.3. A farmer in eastern Ohio desires to drain 640 acres of alluvial sandy loam land. A topographic survey indicates that the maximum slope is 10 feet per mile. Assuming very good drainage, design the ditch cross section including the spoil bank.

15.4. Design an open ditch to carry the runoff from a 14-square mile watershed in your area. The slope of the land along the route of the ditch is 0.27 per cent, the average slope of the watershed is 0.5 per cent, and the soil is heavy clay. Assume that an excellent degree of drainage is desired. Ditch should provide sufficient depth for tile drains.

15.5. Determine deflection angles for the layout of an 8-degree curve if the ditch makes a 45-degree change in direction (see Appendix I). The P.C. is at station 3 + 50.

15.6. How long would it take a 45-hp bulldozer to construct 100 feet of ditch in Problem 15.3, assuming a haul distance of 100 feet?

15.7. How long would it take a 1½-cubic yard dragline to construct 100 feet of ditch in Problem 15.4, assuming wet and sticky soil conditions?

CHAPTER 16

Subsurface Drainage Principles

It is well recognized that plants need air as well as moisture in their root zones. Water in the soil in excess of the field capacity (see Chapter 5) restricts the aeration of the soil and inhibits plant growth. Where conditions of topography, ground water table, or impermeable subsoil inhibit the natural movement of gravitational water from the root zone, artificial subsurface drainage is an essential practice if a high level of productivity is to be developed and maintained. Surface drainage can eliminate ponded water and reduce the amount of water entering the soil profile, but only natural or artificial subsurface drainage can lower a water table to provide an aerated root zone.

16.1. Benefits of Subsurface Drainage. Subsurface drainage is an important conservation practice. Wet lands are usually topographically situated so that when drained they may be farmed with little or no erosion hazard. Sloping land may thus be used less intensively to achieve the same over-all production of intertilled crops. Subsurface drainage properly designed and installed on soil susceptible to this treatment is a sound investment. Many soils having poor natural drainage are, when properly drained, rated among the most productive soils in the world.

Specific benefits of subsurface drainage are: (1) aeration of the soil for maximum development of plant roots and desirable microorganisms; (2) increased length of growing season because of earlier possible planting dates; (3) decreased possibility of adversely affecting soil tilth through tillage at excessive moisture levels; (4) improvement of soil moisture conditions in relation to the operation of tillage, planting, and harvesting machines; (5) removal of toxic substances, such as alkali which in some soils retards plant growth; and (6) greater storage capacity for water, resulting in less runoff and a lower initial water table following rains. Through these benefits drainage enhances farm productivity by (1) adding productive acres without extending farm boundaries, (2) increasing yield and quality of crops, (3) permitting the use of good soil management on the farm as a whole, (4) assuring that crops may be planted and harvested at optimum dates, and (5) eliminating inefficient machine operation caused by wet areas in fields.

The relationship of subsurface drainage to root development is illustrated in Fig. 16.1. A high water table in the spring inhibits root development, leaving a plant with an inadequate root system during the summer months, but a low water table in the spring allows maximum root development.

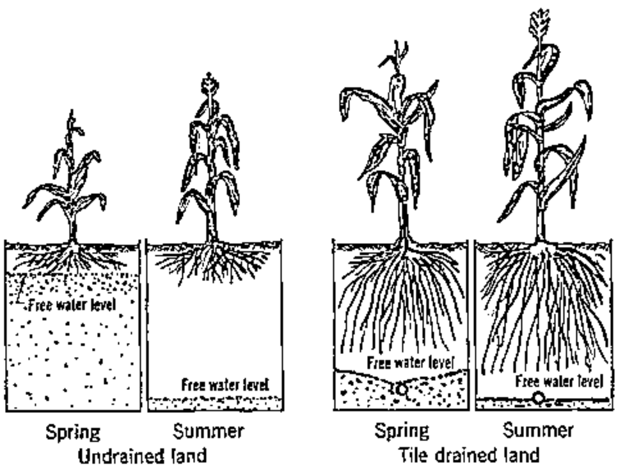

Fig. 16.1. Root development of crops grown on drained and undrained land. (Redrawn from Manson and Rost.[12])

The effect of drainage on soil temperature is shown in Fig. 16.2. Temperature measurements were taken at 1-inch and 4-inch depths in drained and undrained soil. The drained soil reached a higher temperature than the undrained soil with maximum differences of 13.7 and 6.2° F at 1- and 4-inch depths, respectively. The greater effect at the 1-inch depth is highly desirable for good seed germination. Drainage influences soil temperature because of the differences between wet and dry soil in specific heat, thermal conductivity, and evaporation. The specific heat of soil particles is about 0.2 for most soils as compared to 1.0 for water. The thermal conductivity of dry soil is one-third to one-half that of water.[14] Evaporation uses solar energy which would otherwise be available to warm the soil.

The benefits of drainage can be realized only when the soil is potentially productive if drained. In many areas it may be desirable to leave soil undrained and utilize the land as a recreation and wildlife area. In other soils the cost of adequate artificial subsurface drainage may exceed the potential benefits making subsurface drainage economically unsound.

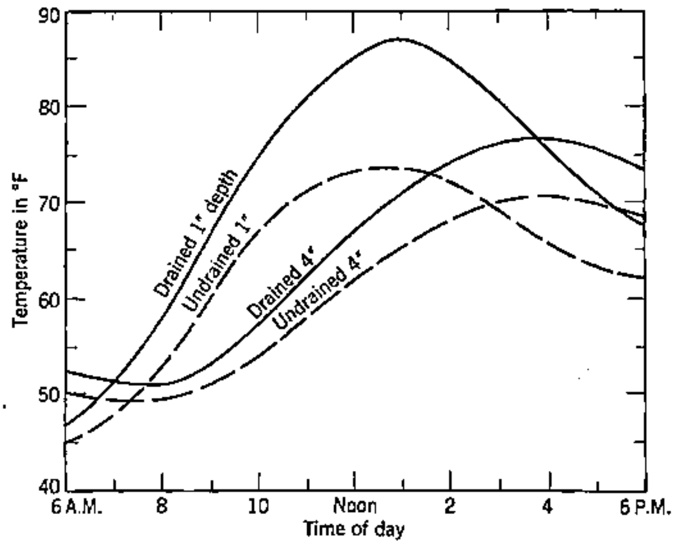

Fig. 16.2. Effect of drainage on soil temperature. (Data from Mosier and Gustafson.[13])

METHODS OF SUBSURFACE DRAINAGE

Artificial subsurface drainage is accomplished by providing horizontal passages in the subsoil into which gravitational water may seep to flow to suitable outlets. Early techniques of subsurface drainage used narrow trenches in fields. Since open trenches hampered field operations, stones, brush, or poles were often placed in the bottom of the trenches which were then backfilled. Modern practice uses clay or concrete tile, unlined *mole* channels, or perforated pipe to provide drainage passages.

16.2. Tile Drains. Tile used in lateral drains (drains intended to receive water directly from the soil as contrasted with main lines which received most of their flow from laterals) are nearly always hollow cylinders one foot in length, 4, 5, or 6 inches in inside diameter, and having a wall thickness of about $\frac{1}{12}$ their inside diameter. These tile may be made of burned clay or of concrete. The tile are laid end to end in the bottom of a trench which is then backfilled. Water enters the tile line through the cracks between the ends of adjoining tile. The tile line is given a slight slope to cause free flow of water toward the desired outlet. A network of tile laterals is connected by a system of main lines which drain to a surface outlet (see Chapter 17). The function of the tile is to stabilize the drainage channel.

16.3. Mole Drains. Mole drains are cylindrical channels artificially produced in the subsoil without digging a trench from the surface (see Chapter 17). They are similar to tile drains except that they are not lined with tile or other stabilizing material. The inherent stability of the soil itself is depended upon for maintenance of the drainage channel.

16.4. Perforated Pipes. Subsurface channels may be stabilized with perforated pipe or tubing. These conduits may be placed in a trench which is then backfilled, or they may be drawn into the soil by a mole plow (see Chapter 17 and 18). Perforated corrugated steel pipe is often used where a high structural strength is required to withstand surface loads or where unstable soil conditions require the rigidity of a long pipe.

MOVEMENT OF WATER INTO SUBSURFACE DRAINS

Water moves into subsurface drains in response to the hydraulic gradient which the earth's gravitational field develops in the mass of soil water surrounding the drain. Many contributions have been made to knowledge of the fundamental nature of the movement of water into subsurface drains. Although it is not yet possible to incorporate this knowledge into specific design procedures, the fundamental knowledge offers a basis for intelligent application of engineering judgment in the design of subsurface drainage systems. The flow equations and relationships discussed here have resulted from solutions of LaPlace's equation, presented in Chapter 5. In many problems the solution has been completely analytical. In others the solution has been accomplished through application of electric analogues.

16.5. Factors Affecting Rate of Flow into Drains. The movement of water into drains is influenced by many variables:

1. *Soil permeability.* The horizontal and vertical permeability of the several soil horizons.
2. *Depth of drain.* The depth of the drain below the surface and the drain location with respect to the various soil horizons.
3. *Drain openings.* The size and distribution of openings into the drain; for example, cracks between tile, perforations, or unlined channels.
4. *Distribution of potential at the flow boundaries.* The configuration and location of the free water surface, and the presence and magnitude of artesian pressure or of back pressure in the drain.
5. *Drain spacing.* The horizontal distance between individual drains.
6. *Drain diameter.* The diameter of the drain tubes.

The influence of many of these variables will be discussed for specific drainage situations.

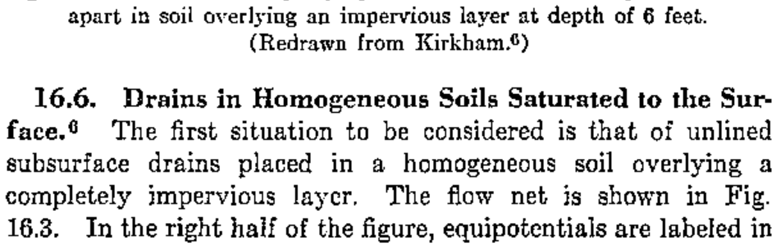

Fig. 16.3. Flow net for equally spaced drains 4.5 feet deep and 40 feet apart in soil overlying an impervious layer at depth of 6 feet. (Redrawn from Kirkham.[6])

16.6. Drains in Homogeneous Soils Saturated to the Surface.[6]

The first situation to be considered is that of unlined subsurface drains placed in a homogeneous soil overlying a completely impervious layer. The flow net is shown in Fig. 16.3. In the right half of the figure, equipotentials are labeled in feet of water. In the left half of the figure, streamlines are labeled with the fraction of the total flow which occurs between the given streamline and the zero streamline (midpoint between drains).

Examination of this flow net shows that 60 per cent of the inflow at the soil surface enters the soil within 2 feet on either side of the drain. The streamlines are much closer together immediately over the drain than at some distance from it. This means that water enters the soil more rapidly over the tile than midway between tile. The closeness of equipotential lines near the drain shows that nearly 50 per cent of the total potential is dissipated within 2 diameters of the drain.

For this situation, flow into an individual drain is given approximately by:[6]

$$Q = \frac{2\pi K(t + y - r)}{\log_e \dfrac{2y - r}{r}} \tag{16.1}$$

where Q = flow into a unit length of drain per unit time.
K = permeability.
t = depth of water ponded on the soil surface.
y = depth from soil surface to center of drain.
r = radius to outside of drain.

This formula is a close approximation when the depth to an impermeable layer is at least twice the depth of the drains, and when the drain spacing is at least five times the depth of the drains. Figure 16.4 shows the influence of drain depth and diameter as calculated from equation 16.1. Inflow is nearly proportional to drain depth in a homogeneous soil and is only slightly affected by drain diameter.

Conclusions of practical significance that can be drawn from this analysis are:

1. Drain inflow is directly proportional to permeability in homogeneous soils.

2. If soil strata of low permeability are not encountered, the effectiveness of tile will increase nearly proportionately to increased depth.

3. Tile should be laid directly under areas where surface water accumulates.

4. Since a large portion of the total hydraulic head is dissipated near the tile, enveloping tile in permeable media (straw, corncobs, gravel) will increase their effectiveness.

5. Tile diameter is not an important factor in determining tile inflow in the usual range of tile lateral sizes.

16.7. Flow to Drains in Heterogeneous Soils. Homogeneous soils are seldom encountered in field practice. It is therefore

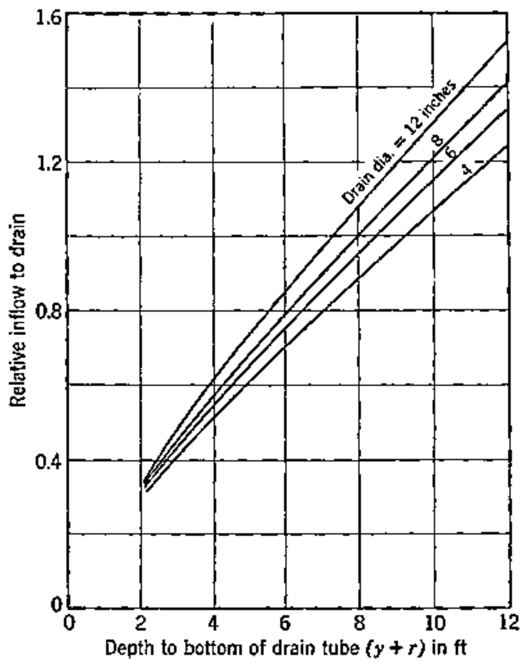

Fig. 16.4. Effect of drain depth and diameter on inflow. (Redrawn from Kirkham.[6])

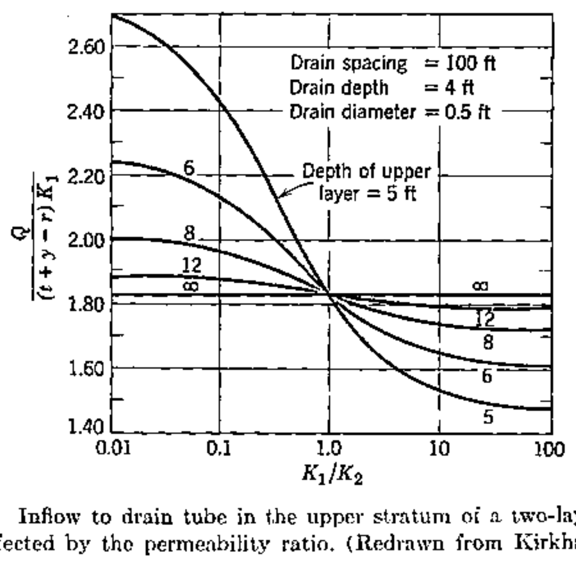

Fig. 16.5. Inflow to drain tube in the upper stratum of a two-layered soil as affected by the permeability ratio. (Redrawn from Kirkham.[8])

of interest to examine the influence of soil strata of different permeabilities. Analytical solution has been made for flow into drain tubes in soil having two distinct permeability layers.[8] The inflow to drains in the upper layer of a two-layered soil saturated to the surface is shown in Fig. 16.5 as a function of

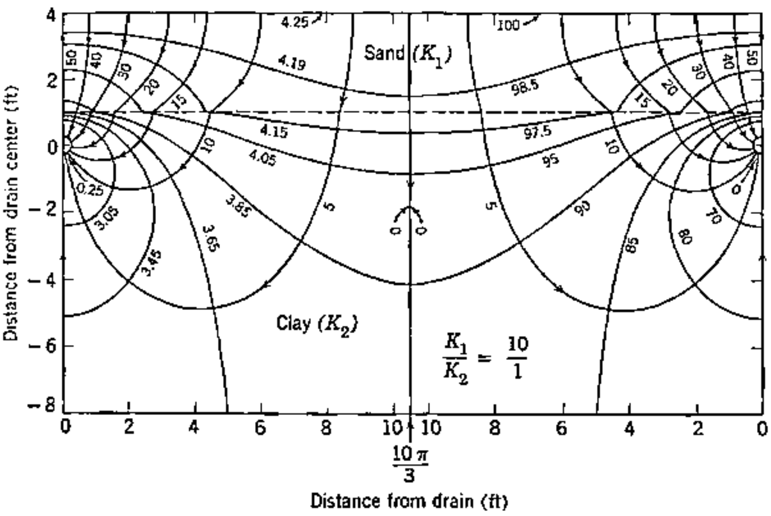

Fig. 16.6. Flow net for drain tubes in the lower stratum of a two-layered soil having a relatively permeable upper stratum. (Redrawn from Kirkham.[8])

the depth of the upper soil layer and the ratio of the permeability, K_1, of the upper layer to the permeability, K_2, of the lower layer.

In Fig. 16.6 the tile is embedded in the lower of the two permeability layers, and Fig. 16.7 shows the changing inflow to the drains as K_1/K_2 varies from 1 to 100. When K_1/K_2 is less than 1, streamlines in the upper layer are vertical.

A particularly interesting problem in flow in heterogeneous soils occurs if it is assumed that the tile is placed in the lower layer of soil but that the trench is backfilled with material from the more permeable upper layer. This problem has been solved by the iteration method mentioned in Chapter 5.[10] Under such conditions the relative inflow for variation in K_1/K_2 is: $K_1/K_2 = 1$, $Q = 100.00$; $K_1/K_2 = 5$, $Q = 46.1$; $K_1/K_2 = 10$,

$Q = 42.2$; $K_1/K_2 = 100$, $Q = 41.1$. When the ratio of K_1/K_2 is 5 or more, very little water flows through the impermeable stratum, and most of the flow moves through the upper stratum to the backfilled trench and thence to the drain.

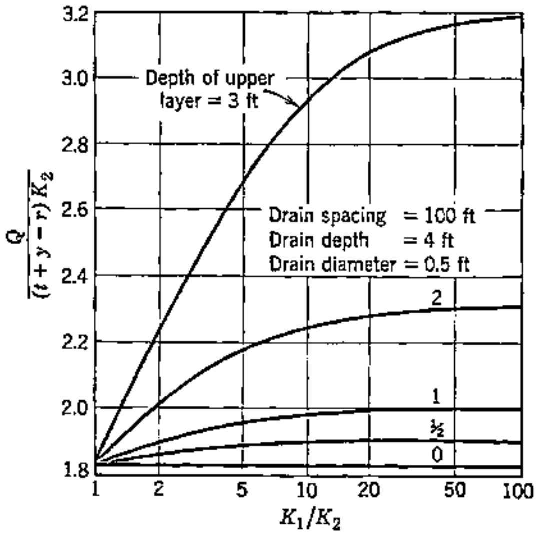

Fig. 16.7. Inflow to drain in the lower stratum of a two-layered soil as affected by permeability ratio. (Redrawn from Kirkham.[8])

From these analyses of flow in heterogeneous soils it is seen that, if depth of drains must be gained by embedding the drains in a relatively impermeable stratum, much of the effect of depth in increasing drain inflow may be lost. However, since drain depth determines the limit of the water table lowering, there is an advantage in depth beyond the criterion of increased rate of water removal. In many soils a stratum of very low permeability exists at relatively shallow depth, making tiling an ineffective practice.

16.8. Flow to Drains in Anisotropic Soils.[11] For the horizontal permeability, K_h, of a soil to differ from the vertical permeability, K_v, of the same soil is not unusual. When K_h exceeds K_v the flow distribution is greatly improved. With reference to Fig. 16.3, increasing K_h while holding K_v constant would result in a streamline pattern approaching a uniform inflow at the soil surface. Decreasing K_h with respect to K_v would have the opposite effect.

16.9. Flow to Interceptor Drains. Often subsurface drains are installed across a slope to intercept seepage from higher ground. The drains may be effective in drying wet hillsides, protecting valuable lowlands, or maintaining waterways in a dry and stable condition. Flow nets for interceptor drains have been determined by electric analogues.[3] Figure 16.8 shows the flow net to an interceptor drain laid upon an impermeable

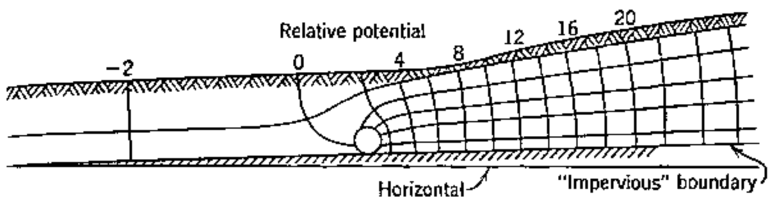

Fig. 16.8. Flow net to interceptor drain laid upon impermeable boundary. (Redrawn from Childs.[3])

layer. Study of the streamlines shows that most of the flow from above has entered the drain. This study has shown that interceptor drains placed on the impermeable boundary are much more effective than those placed at some distance above the impermeable boundary. Where interceptors must be placed in the impermeable layer a permeable backfill should be used.

16.10. Influence of Drain Openings. *Cracks between Tile.* Tile are generally laid in 1-foot lengths with cracks ranging up to about ¼ inch between adjoining tile. Analytical solution of the effective of crack width on inflow to tile has given the results shown in Table 16.1.[7] The data show that in the range of field practice doubling the crack width will increase the inflow to tile by about 10 per cent. It is thus important in laying tile to leave as large a crack width as is permissible as limited by the stability of the soil material. If a tile line is enclosed in a graded gravel envelope, the effect is the same as providing an unlined drain, for the resistance of the gravel to seepage flow is negligible.

Perforations. The influence of perforations on inflow to drain tubes has been studied analytically and by application of electric analogues.[15] In a 6-inch-diameter drain tube, increasing the diameter of perforations from ¼ inch to ½ inch increased the flow 68 per cent for 4 holes per foot and 46 per cent for 10 holes per foot. The inflow was nearly proportional to the number of holes up to 4 holes per foot; beyond 20 holes per foot additional perforations gave little additional inflow.

Table 16.1 Relative Inflow into Cracks and Perforations on Drains of 6-Inch Outside Diameter

Crack Width, in. per ft of length	*Number of ¼-Inch Dia. Perforations,** per ft of length	*Relative Inflow,†* *Depth y,* 4 ft
1/64	14	0.327
1/32	16	0.356
1/16	21	0.392
⅛	28	0.433
¼	42	0.485
Unlined drain		1.000

* From Schwab and Kirkham.[15] For use where perforated tubes are used in lieu of standard tile.

† From Kirkham.[7]

16.11. Water Table Movement. Under field conditions when water is not ponded on the soil surface, the plane of saturation over a drain recedes as water enters the drain. The rate of this recession is of particular interest in relation to crop growth. The problem of the falling water table in homogeneous soils has been studied by electric analogues[4] and by iteration methods.[9] Figure 16.9 gives the result of these latter studies. The parameter TK/f characterizes the soil and time relationships involved. T is the time in days since the onset of drawdown, K is the permeability in feet per day, and f is the drainable pore space expressed as a decimal fraction. For example, in a soil having a permeability of 0.8 foot per day and a pore space of 0.4, TK/f would equal 4 after 2 days, and at this time the water table would have assumed the position designated by $TK/f = 4$ in Fig. 16.9.

In Art. 16.6 it was pointed out that ponded water entered the soil more rapidly over the tile than midway between tile. Under ponded conditions surface water moves horizontally to equalize the effect of differential drawdown rates. However, after sur-

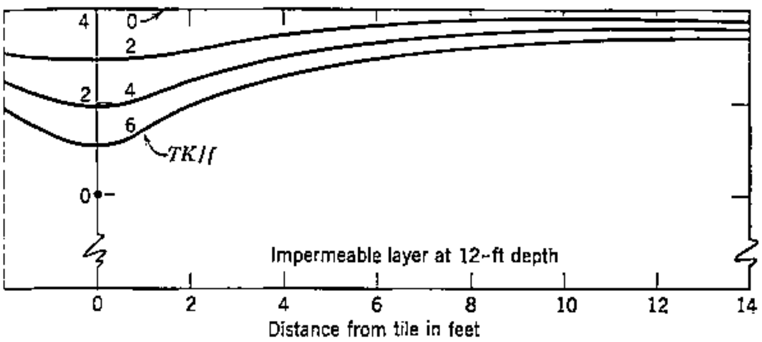

Fig. 16.9. Successive water table positions over drain tubes 6 inches in diameter, 4 feet in depth, and 28 feet apart. (Redrawn from Kirkham and Gaskell.[9])

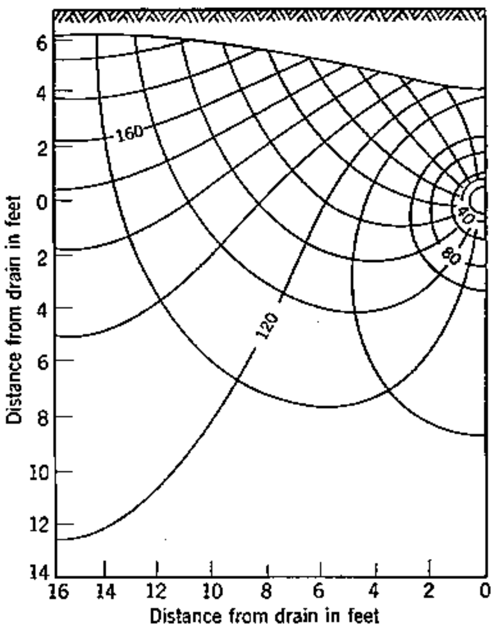

Fig. 16.10. Flow net for a drain tube in a homogeneous soil under a typical drawdown surface. (Redrawn from Childs.[2])

face water is removed, horizontal movement is not rapid enough to equalize the water level, and the characteristic drawdown surface develops. Application of electric analogue techniques to drawdown studies shows the distribution of potential under the condition of the falling water table. Figure 16.10 gives the flow net for such a condition. The falling water table gives rise to increasing horizontal components of potential gradient, causing

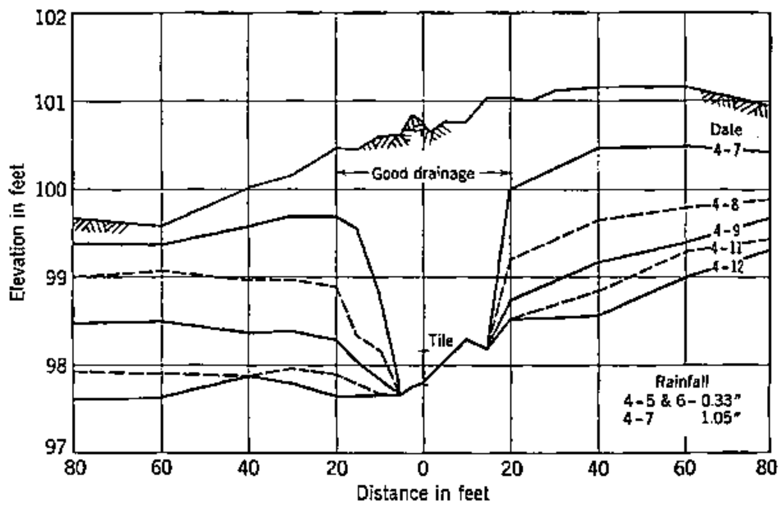

Fig. 16.11. Drawdown curves measured in the field. (Redrawn from Kidder and Lytle.[5])

increased movement of water from between the drains which acts to partially compensate for the drawdown immediately over the drains.

Drawdown may be measured in the field by observing the water level in small-diameter wells placed at intervals between the drains. Results of drawdown studies by such techniques are given in Fig. 16.11. The curves do not have the smooth form of the theoretical curves because of the lack of homogeneity in the soil. In particular the slope of the actual curves near the tile is much steeper than the same portion of the theoretical curves. This may probably be accounted for by the relatively higher permeability for the backfilled trench than for the undisturbed soil.

The drawdown requirements for good drainage are not definitely established. One criterion that has been used in the lowering of the water table from the surface 12 inches in 24 hours and from the surface 21 inches in 48 hours.[5] By this criterion "good drainage" is obtained within 20 feet of the tile in Fig. 16.11.

16.12. Drainage Coefficient. In field design of drainage systems the drainage coefficient is the index of the flow capacity to be provided in mains and submains. The drainage coefficient is defined as the depth of water to be removed from the drainage area in a unit of time. Units are commonly inches of water in 24 hours, and the drainage coefficient is usually referred to simply as "inches." The coefficient on a particular job is usually arrived at on the basis of past experience with similar soil, crop, and climatic conditions. Selection of a given drainage coefficient does not necessarily indicate that this amount of water will be able to move through the soil into the tile laterals.

The development of fundamentally sound methods relating tile drainage design to quantitative measurements of soil properties is needed. Some contributions toward this end have been made,[1,16] but they have been essentially empirical in nature and have not been demonstrated as applicable beyond the soils for which they were developed.

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PROBLEMS

16.1. Determine the outflow from 1000 feet of 5-inch tile at a depth of 4 feet in completely saturated soil, assuming a permeability of 2 inches per hour. The effective crack spacing between tile is ⅛ inch.

16.2. If the inflow into a 4-inch diameter drain at a depth of 4 feet is 0.3 cubic foot/hr/ft, what is the inflow into the drain at a depth of 8 feet? The inflow into a 12-inch diameter drain at the same depth?

16.3. If the flow into a 5-inch tile (0.5 foot outside diameter) 4 feet deep and spaced 100 feet apart in homogeneous soil ($K = 10$ iph) is 0.5 cubic foot/hr/ft, what would be the inflow into these tile if placed in a 6-foot-depth upper stratum of a two-layered heterogeneous soil where the top stratum and the lower stratum have permeabilities of 10 and 1 iph, respectively? What would be the inflow if the drains were placed in a lower stratum with a 2-foot-depth upper stratum above? Assume $K_1 = 100$ iph and $K_2 = 10$ iph.

16.4. What percentage increase in flow could be expected for 5-inch drain tile (0.5-foot outside diameter) at a depth of 4 feet by placing tile ¼ inch apart on the ends as compared to close fitting tile? Assume an effective crack spacing of $\frac{1}{64}$ inch for tile placed as close together as possible.

16.5. Determine the drainage coefficient for 1000 feet of tile if the system is installed with an interval of 100 feet between tile lines and the outflow is 200 cubic feet/hr.

CHAPTER 17

Subsurface Drainage Design

TILE DRAINAGE

The design of a tile drainage system includes the layout and arrangement of the tile lines, selection of a suitable outlet, proper depth and spacing of laterals, determination of the length and size of tile, selection of good quality tile of adequate strength, and the design of such accessories as surface inlets and outlet structures.

TYPES OF SYSTEMS

Tile drainage systems in general use are the natural, herringbone, gridiron, and interceptor types shown in Fig. 17.1. Combinations of two or more of these types are frequently required for the complete drainage of an area. Basic knowledge of the principles of drainage is essential, and experience is desirable for the proper selection of the type of system and layout of the tile lines.

17.1. Natural or Random. The natural or random system is widely adopted in fields that do not require complete drainage with equally spaced laterals. This system is quite flexible as well as economical since the tile lines follow natural draws or other low depressions. Such a system is particularly adapted to the drainage of small or isolated wet areas.

17.2. Herringbone. The herringbone system is adapted to areas that have a concave surface or a narrow draw with the land sloping to it from either direction. The main line is laid out nearly normal to the slope and follows the low area. Despite the large amount of double drainage (land drained both by the laterals and the main or submain), the herringbone system is particularly suitable where the laterals are long and the waterway requires thorough drainage.

17.3. Gridiron. The gridiron system is similar to the herringbone system except that the laterals enter the main from only one side. The gridiron pattern is more economical than the herringbone system because the number of junctions and the double-drained area are reduced. Where the waterway is of considerable width, a main is placed on both sides of the water-

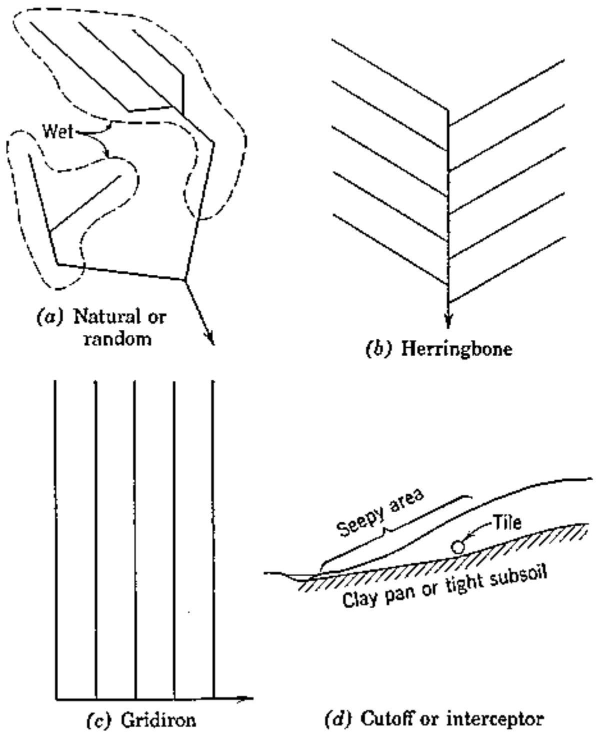

Fig. 17.1. Common types of tile drainage systems.

way. This system, known as the double main system, is essentially two separate gridiron patterns. Since the mains need not cross the waterway at critical points, serious erosion problems may be prevented. Although right-angle junctions are shown in Fig. 17.1*c*, main and laterals may intersect at angles less than 90 degrees.

17.4. Cutoff or Interceptor. The cutoff or interceptor drain is normally placed near the upper edge of a wet area as shown in Fig. 17.1*d*. The usual cause for such wet conditions is the outcropping of impermeable strata on or near the surface. Frequently, this condition exists along waterways. To drain such areas the interceptor drain is frequently installed on both sides of the waterway.

OUTLETS

17.5. Types of Outlets. The two principal types of outlets for tile drains are gravity and pump. Pump outlets (see Chap-

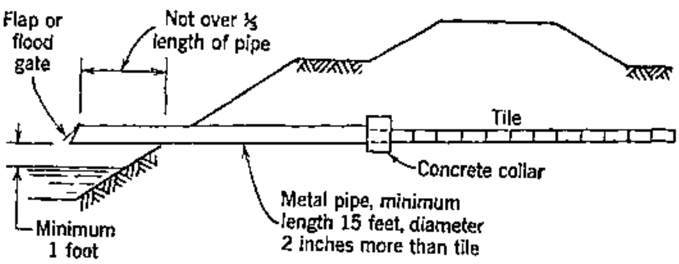

Fig. 17.2. A suitable gravity outlet for tile drains. (From Schwab and others.[18])

ter 19) may be considered where the water level at the outlet is higher than the bottom of the tile outlet for any extended period of time.

Gravity outlets, by far the most common, include other tile drains, constructed waterways, natural channels, or wells. Outlet ditches should have sufficient capacity to carry surface runoff and tile flow. Where the drainage system is connected to other tile drains, the outlet should have sufficient capacity to carry the additional discharge. As shown in Fig. 17.2, corrugated metal or other rigid pipe is recommended. Some type of grill or flap gate over the end is desirable to prevent entry of rodents. If there is danger of flood water backing up into the drain, an automatic flood or tide gate may be installed in place of the flap gate. The end of the outlet pipe should be a minimum of 1 foot above the normal water level in the ditch. To prevent damage due to high velocities in the ditch or failure from snow loads, the exposed end of the pipe should not extend beyond the bank more than one-third its total length. The minimum total length should be 15 feet, and the diameter should be approximately 2 inches larger than the tile size. In connecting the pipe to the tile a concrete collar may be installed. When available, drop inlets and other permanent structures described in Chapter 11 provide suitable outlet facilities.

Vertical drainage outlets (wells) are essentially wells extending into a porous soil layer or open rock formation in the lower horizons. The existence of a substratum that can continually take in large quantities of water and that can be reached without prohibitive cost is the exception rather than the rule. Since there is no positive method for locating or for predicting the permanent capacity of such outlets, their use involves considerable risk. Also, if tile outflow is contaminated with sediment, sewage, or dissolved salts, there is danger of polluting the ground water. For these reasons, vertical outlets are not generally recommended.

17.6. Requirements of a Good Outlet. The importance of a good outlet is indicated by the fact that a high percentage of failures of drainage systems is due to faulty outlets. The requirements of a good outlet are to: (1) provide a free outlet with minimum maintenance, (2) discharge the outflow without serious erosion or damage to the tile, (3) keep out rodents and other small animals, (4) protect the end of the tile line against damage from the tramping of livestock as well as excessive freezing and thawing, and (5) prevent the entrance of flood water where the outlet is submerged for several hours.

DEPTH AND SPACING

17.7. Depth. The factors that affect the depth of tile drains are soil permeability, outlet depth, spacing of laterals, depth to the impermeable layer in the subsoil, and limitations of trenching equipment. Tile depth as here considered applies only to laterals and not to the depth of mains, since the depth of the main is governed primarily by outlet conditions and topography.

Tile depth, defined as the distance from the surface to the bottom of the tile, varies considerably in different soils. Under no conditions should the amount of cover over the top of the tile be less than 2 feet. This minimum is necessary to protect the tile from heavy surface loads and to prevent shifting of the tile. In uniformly permeable mineral soils the depth of laterals usually varies from 2.5 to 5 feet. In deep organic soils where initial settlement has taken place, the minimum depth should not be less than 4 feet. In arid regions under irrigation, particularly in alkali soils, drains are placed as deep as 6 to 8 feet.[23]

Where the subsoil is relatively impermeable, the tile should be placed on or above the impermeable layer. If tile must be placed in the impermeable layer, the trench should be backfilled with permeable soil.

17.8. Spacing. In humid regions most tile are spaced 60 to 150 feet apart and up to 300 feet in very permeable soils. Where high-value crops are grown or under special conditions, spacings of 30 to 50 feet are sometimes necessary; however, such spacings are normally not economically feasible. In irrigated areas of the West spacings ranging from 150 to 600 feet are practicable.

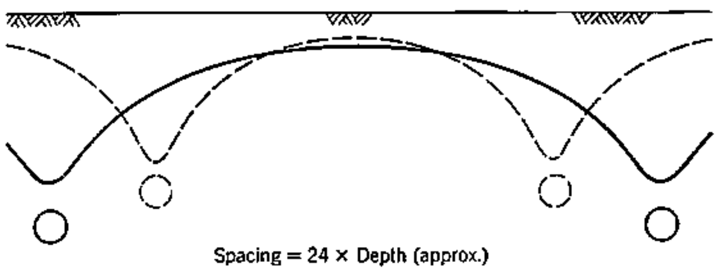

Fig. 17.3. Relationship between depth and spacing (approximate). (Based on data from Kirkham and Gaskell.[6])

17.9. Relationship between Depth and Spacing. A definite relationship between depth and spacing of drains exists. For soils of uniform permeability, the deeper the drains, the wider is the spacing necessary to provide the same degree of drainage. A rule, which has been verified approximately by theoretical analysis for moderately permeable soils at depths of 3 and 4 feet, states that each foot of depth permits a spacing of 24 feet.[3] As shown in Fig. 17.3, tile 3 feet deep and 66 feet apart drain about the same volume of soil in a given time as tile 4 feet deep and 100 feet apart. The principal advantage of the deeper drains at wider spacing is a saving of approximately one-third in the cost. However, the saving decreases with depths greater than 4 feet because of increased installation costs.

In general, the depth and spacing of tile drains varies largely with soil permeability, crop and soil management practices, kind of crop, and the extent of surface drainage. With good crop and soil management practices, the depth and spacing for normal conditions vary within the limits given in Table 17.1.

Table 17.1 AVERAGE DEPTH AND SPACING OF TILE DRAINS

Soil	*Relative Permeability**	*Spacing,* feet	*Depth,* feet
Clay	Slow	30–50	3.0–3.5
Clay loam	Slow to moderately slow	40–70	3.0–3.5
Average loam	Slow to moderately slow	60–100	3.5–4.0
Fine sandy loam	Moderate	100–120	4.0–4.5
Sandy loam	Moderate to moderately rapid	100–200	4.0–5.0
Peat and muck	Moderate to moderately rapid	100–300	4.0–5.0
Irrigated soils	Variable	150–600	5.0–8.0

* Permeability as classified by O'Neal.[14]

17.10. Methods of Determining Depth and Spacing. The determination of the depth and spacing of tile drains has been studied by many investigators. Because of the many variables involved and soil differences in various parts of the country, no method has yet been developed that is satisfactory for all areas. Depth and spacing formulas have been developed by Neal[13] in Minnesota, Aronovici and Donnan[2] in California, and Walker[22] in Virginia. With all these methods, the procedure in brief is as follows: assume a drainage rate, determine suitable characteristics of the soil which are a measure of its drainability, select a suitable depth for the tile, and then compute the spacing. Neal[13] based his equations on the moisture equivalent, lower plastic limit, upper plastic limit, and the percentage of clay in the subsoil; the other two methods are analytical solutions based on soil permeability. Each of these methods appears to give satisfactory results in soils for which the formulas were developed.

SIZE OF TILE DRAINS

17.11. Selection of Drainage Coefficients. The drainage coefficient (see Art. 16.12) for tile drains depends largely on rainfall. It is difficult to correlate rainfall with the drainage coefficient since the distribution of rainfall during the growing season and its intensity must be considered along with evaporation and other losses. For example, where the ground is dry, a very intense storm of short duration produces rapid surface runoff and little infiltration. However, rains of low intensity over a long period of time may produce high rates of outflow from the tile.

The selection of a drainage coefficient is based primarily on experience and judgment. Recommended drainage coefficients for various geographical areas are shown in Table 17.2. The drainage coefficient should be such as to remove excess water rapidly enough to prevent serious damage to the crop. Under normal conditions where adequate surface drainage is provided by natural or constructed channels, the drainage coefficient is less than that required when surface water must be removed through surface inlets to the tile. In many instances removal of surface water necessitates doubling or increasing the drainage coefficient. Where the underlying stratum is sand or other porous material, normal coefficients may be reduced. If blind inlets (Art. 17.16) are provided, normal coefficients should be increased but not as much as for surface inlets (Art. 17.15). For truck crops and for peat or muck soils coefficients may be greatly increased as indicated in Table 17.2.

Table 17.2 Drainage Coefficients for Tile Drains*

	Field Crops on Mineral Soils		
Geographical Area of U. S.	*Normal Adequate Surface Drainage*	*With Surface Inlets*	*Truck Crops and Special Soil Conditions*
Northeastern	3/8	3/4–1	3/4
Southeastern	1/2	1	..
Upper Mississippi	3/8	1/2–1	3/4
Northern Great Plains	1/4–3/8	1/2	..

Note: These coefficients may vary depending on special local conditions.

* Based on data from the Soil Conservation Service.

In irrigated areas the discharge from tile lines may be expected to vary from about 10 to 50 per cent of the water applied.[23] In these regions the drainage coefficient may be influenced somewhat by the size of the area contributing to the flow since not all of the area is irrigated at the same time.

Seepage water should also be considered in selecting the drainage coefficient.

17.12. Grades. Maximum grades are limiting only where tile are designed for near maximum capacity or where tile are embedded in unstable soil. Tile embedded in fine sand or other unstable material may become undermined and settle out of alignment unless special care is taken to provide joints that fit snugly against one another. Under extreme conditions it may be necessary to install bell and spigot tile, tongue and groove concrete tile, metal pipe, or tar-impregnated tubing. On mains steep grades up to 2 or 3 per cent are not objectionable, provided the capacity at all points nearer the outlet is equal to or greater than the tile above.

A desirable minimum working grade is 0.2 per cent. Where sufficient slope is not available, the grade may be reduced to that indicated in Table 17.3. In some soils these minimum grades should be higher so as to provide sufficient velocity to remove sediment from the tile. Minimum grades are sometimes based on a minimum velocity of about 1.5 fps at full flow. The grade should not be less than the minimum, unless special precautions are taken during construction.

Table 17.3 Minimum Grades for Tile Drains

Size Tile, in. (inside diameter)	*Minimum Grade*, %	*Velocity at Full Flow*, fps
4	0.15	1.02
5, 6	0.1	0.96, 1.09
8, 10	0.08	1.19, 1.37
12 or larger	0.05	1.22 or more

17.13. Determination of Drainage Area. Normally, the drainage area is computed from the length and spacing of the drains. Where surface inlets are installed, the contributing watershed is the drainage area rather than the area drained by the tile. The area drained by the tile and the contributing watershed represent minimum and maximum areas, respectively. Where excess seepage occurs or where excess surface water must be handled, the drainage area should be somewhere between the minimum and maximum values.

17.14. Size of Laterals and Mains. The size of tile drains depends on the drainage area, the drainage coefficient, and the grade. Four-inch tile is the minimum recommended size; however, 5- and 6-inch tile are considered minimum sizes in many areas. It is easy to justify 5-inch tile, since the cost of the tile is only slightly more, whereas the capacity is 80 per cent greater than that of 4-inch. Another advantage of 5-inch tile is that small deviations from true grade, resulting in sedimentation, will affect the capacity of 5-inch tile less than that of 4-inch. The capacity of the tile is determined from the Manning formula, using n equal to 0.0108. A graphical solution of this formula is given in Appendix C.

The size of the main is determined from the drainage area of all connected tile drains. It should not be computed from the maximum capacity of all laterals. The capacity at any point in the system should be adequate to carry the discharge of the drainage system above, allowance being made for surface inlets and possible additional tile lines.

ACCESSORIES

Accessories for tile drainage systems include special facilities, such as surface inlets, sedimentation basins, and blind inlets.

17.15. Surface Inlets. As shown in Fig. 17.4, a surface inlet, sometimes called an open inlet, is an intake structure for the removal of surface water from potholes, road ditches, other depressions, and farmsteads. Whenever practicable, surface water should be removed with surface drains (Chapter 14) rather than surface inlets.

Surface inlets should be properly located and constructed. They should be placed at the lowest point along fence rows or in land that is in permanent vegetation. Where the inlet is in a cultivated field, the area immediately around the intake should be kept in grass. The surface inlet should be constructed of bell and spigot tile (minimum size 6 inches) with sealed mortar joints on the vertical riser and extending at least 6 feet on either side along the tile line. Galvanized metal pipe or a manhole constructed of brick or monolithic concrete is also satisfactory. At the surface of the ground a concrete collar should extend around the intake to prevent the growth of vegetation and to hold it in place. On top of the riser a beehive cover or other suitable grate is necessary to prevent trash from entering the tile.

Where a surface inlet is necessary on a main tile line, it is good practice to locate the surface inlet at the end of a short lateral 6 to 12 feet in length. Such construction may eliminate failure of the system if the surface inlet should become damaged.

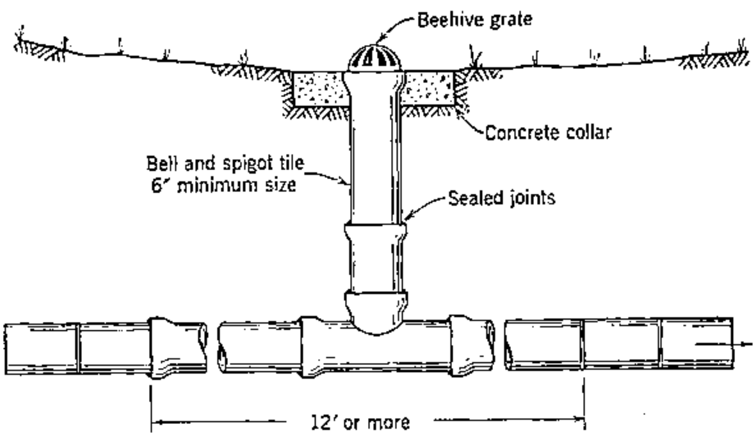

Fig. 17.4. Surface inlet for tile drains.

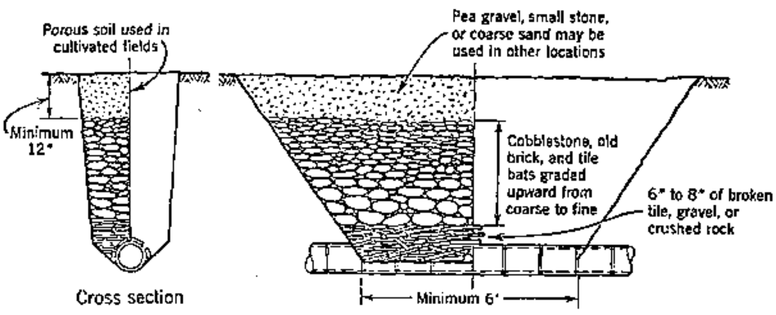

Fig. 17.5. Blind inlet or French drain. (Redrawn from reference 20.)

17.16. Blind Inlet or French Drain. Where the quantity of surface water to be removed is small or the amount of sediment is too great to permit surface inlets to be installed, blind inlets may, at least temporarily, improve drainage. Though these inlets often do not function satisfactorily for more than a few years, they are economical to install and do not interfere with farming operations.

As shown in Fig. 17.5, a blind inlet is constructed by backfilling the tile trench with various gradations of material. The coarsest material is placed immediately over the tile, and the size is gradually decreased toward the surface. Since the soil surface has a tendency to seal, the area should be kept in grass or permanent vegetation if possible. Other materials, such as corncobs and straw, are considered less dependable than more durable materials.

17.17. Sedimentation Basin. Soils containing large quantities of fine sand frequently cause sedimentation since the particles enter the tile through the cracks. A sedimentation basin is any type of structure that provides for sediment accumulation, thus reducing deposition in the tile.

A sediment basin may be desirable where the grade in the tile is greatly reduced, where several laterals join the main at one station, and where surface water enters the tile. The structure shown in Fig. 17.6 has a turned-down elbow on the outlet for retarding the outflow when the basin becomes filled with sediment. Where structures do not have this feature, the tendency is to neglect the cleanout of the basin. Sedimentation basins are seldom installed in small farm drainage systems since they are more applicable to large drains.

Junction boxes may be installed where several tile lines join at different elevations. Except for the catch basin, they are similar to the structure shown in Fig. 17.6. To facilitate farming operations, the top of the manhole should be placed 12 inches or more below the surface.

17.18. Controlled Drainage Structures. Controlled drainage structures in tile lines for maintaining the ground water at a specified level in the tile are similar to sedimentation basins except that crestboards are placed in the structure to keep the water at the desired level. The functioning of such structures is similar to that for control dams in open ditches described in Chapter 15. The design and use of control facilities are described by Morris.[12] Controlled drainage with tile is essential in the management of organic soils because of its effectiveness in controlling subsidence.

17.19. Relief Wells and Breathers. Relief wells and breathers are small-size vertical risers extending from the tile line to 1 foot above the surface. The riser should be made of steel pipe or cemented bell and spigot tile and should be lo-

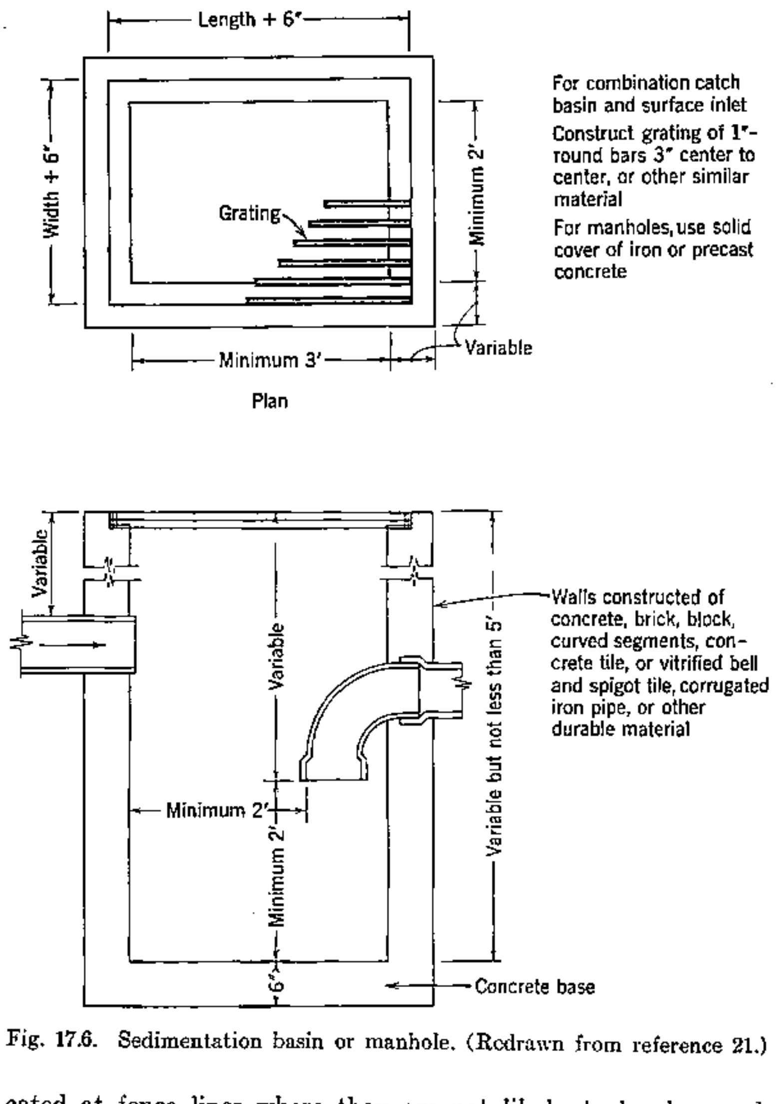

Fig. 17.6. Sedimentation basin or manhole. (Redrawn from reference 21.)

cated at fence lines where they are not likely to be damaged. Breathers are installed on long lines to prevent the development of a vacuum. However, some engineers do not feel that breathers are justified. Relief wells serve to relieve the excess water pressure in the tile during periods of high outflow, thus preventing blowouts. A relief well should be installed where a steep section of a main changes to a flat section, unless the capacity of the flat section exceeds the capacity of the steep section by 25 per cent.[21]

SELECTION OF TILE

Only high-quality tile should be installed in a drainage system. It is false economy to install second-grade or poor-quality tile. Drain tile are primarily of two kinds: clay and concrete. Clay tile are made of shale, fire clay, or surface clay; concrete tile are made of Portland cement and a suitable aggregate. Except for certain soil conditions, either clay or concrete tile of good quality is satisfactory.

17.20. Characteristics of Good Drain Tile. Good drain tile should have the following characteristics: (1) resistance to weathering and deterioration in the soil, (2) sufficient strength to support static and impact loads under conditions for which they are designed (see Chapter 18), (3) low water absorption, that is, a high density, (4) resistance to alternate freezing and thawing, (5) relative freedom from defects, such as cracks and ragged ends, and (6) uniformity in wall thickness and being true to shape. Drain tile that meet specifications of the American Society for Testing Materials, ASTM (C4-50T)[1] have the essential qualities listed above (see Appendix E). Specifications have been prescribed for two classes of drain tile, namely, standard and extra-quality. Standard quality tile are satisfactory for drains of moderate size and depths found in most farm drainage work.

17.21. Concrete Tile. Concrete tile should be made with high-quality materials and be properly cured. The basic requirements for making concrete tile are discussed in detail by Manson and Miller.[7,8,10] Where concrete tile are to be installed in acid or alkali soils, the tile must be made with cements having specific chemical characteristics. Curing methods will also depend upon the degree of acidity or alkalinity of the soil. For nearly neutral soils where strength is the principal criterion, a steam-cure of 30 hours at 155° F or a water-cure of 28 days at 72° F is adequate.[8]

Good-quality concrete tile are very resistant to freezing and thawing but may be subject to deterioration in acid and alkaline soils. In these soils concrete tile should be used only if approved by local recommendations. The amount of corrosive action on concrete is proportional to the degree of acidity as measured by the pH scale.[10] By visual inspection it is more difficult to determine the quality of concrete tile than that of clay tile.

17.22. Clay Tile. Clay tile should be well burned, with no checks or cracks, and should have a distinct ring when tapped with a metal object. Ordinary drain tile are not burned as hard as vitrified sewer tile. Clay tile made from shale are more durable and usually have less absorption than those made from surface clays.[11] It is difficult to establish satisfactory relationships between the frost-resistance of clay tile in the field and standard laboratory tests. Evidence that freezing and thawing has some effect is indicated from a survey of drainage installations in the upper Mississippi Valley. These tile had been in service from 30 to 40 years. Where the tile were less than 1.8 feet in depth, there was some disintegration, but where the depth was greater than 1.8 feet there was no evidence of frost action.[11]

Clay tile are not generally affected by acid or alkaline soils. When subjected to frequent alternate freezing and thawing conditions, it is safer to use concrete tile, although some clay tile is resistant to frost damage. Where clay tile are laid with less than 2.5 feet of cover, they should be extra-quality. In most mineral soils the kind of tile (clay or concrete) is not as important as the quality.

17.23. Other Types of Drains. Several other types of drain pipe are available commercially, including tar-impregnated, corrugated metal, and plastic tubing. These materials are manufactured in lengths greater than that normally used for clay or concrete tile; therefore, it is necessary to perforate these tubes for drainage purposes. The effect of perforations on flow into such tubes is discussed in Chapter 16. Tar-impregnated pipe as used in sewer systems is suitable where excessive loads are not encountered. Corrugated metal pipe is especially suitable for crossing high embankments and for extremely shallow depths where the load would be prohibitive for ordinary tile. Extensive use of metal pipe is limited largely by the cost of the material. Plastic tubing may be installed with a mole plow, thus reducing the cost of installation since an open trench is not required.

DESIGN PROCEDURE

The tile drainage system should be coordinated with existing and proposed surface ditches and other subsurface drains.

17.24. Preliminary and Location Surveys. The first step in design is to make a preliminary survey of the area, much as described for open ditches in Chapter 15. Soils should be investigated to determine whether tile drainage is practicable and economical. If so, sufficient information should be obtained to permit the selection of an adequate depth and spacing for the drains. The extent and depth of impermeable strata and the source of seepage should be determined where applicable. On flat land, topographic maps may be prepared, especially where an extensive drainage system is planned. If the drainage system is not extensive, the preliminary survey and the location survey may be made at the same time.

Experience is desirable in the proper location of tile drains, but a few general rules are: (1) place the outlet at the most suitable location; (2) provide as few outlets as possible; (3) lay out the system with short mains and long laterals; (4) use the available slope to best advantage, especially on flat land; (5) follow the general direction of natural waterways, particularly with mains and submains on land with considerable slope; (6) avoid routes that result in excessive cuts; (7) avoid crossing waterways except at an angle of 45 degrees or more; and (8) avoid soil conditions that increase installation and maintenance costs.

17.25. Alignment. Where a change in direction of tile lines is necessary, a junction box, fitted tile, or a minimum radius of curvature of 5 feet should be used. The alignment of tile at junctions is shown in Fig. 17.7. Where sufficient slope is available, the grade line of the lateral should intersect the main near the top of the tile. However, if sufficient slope is not available, the grade line of the lateral may intersect the main at a lower elevation but should always be high enough to permit the center line of the lateral to intersect the center line of the main. A difference in elevation (z in Fig. 17.7) between the extended grade line of the lateral and the main is

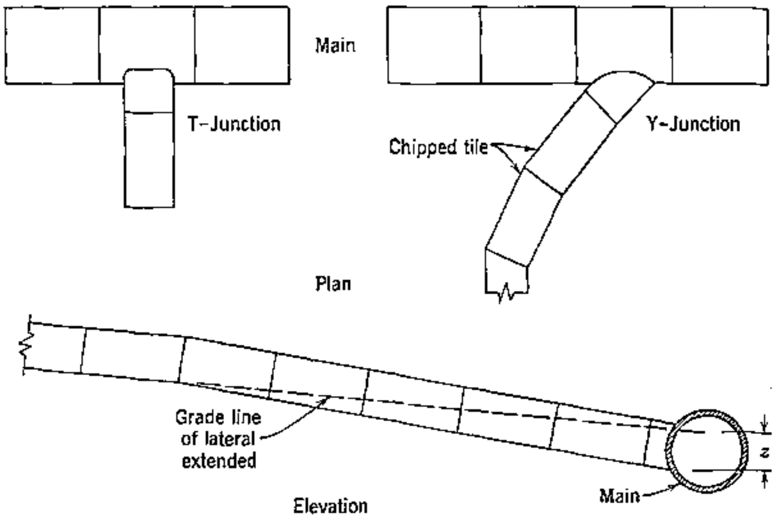

Fig. 17.7. Vertical and horizontal alignment at junctions.

adjusted by increasing the slope in the last few tile on the lateral.

17.26. Staking. The tile line is staked by placing hub and guard stakes at 50- or 100-foot stations, starting at the outlet (sta. 0+00). To prevent these stakes from being removed during construction, the stake line must be offset about 5 feet from the center line of the trench, depending on the machine. After the tile system has been laid out, the elevations of the hub stakes are determined to the nearest hundredth of a foot.

Tile drains like open ditches are arbitrarily designated in decreasing order of importance as mains, submains, and laterals. As shown in Fig. 17.8, the first lateral above the outlet to be connected to main A may be designated $A1$, the next $A2$, etc. The lines entering $A1$ may be indicated as $A1.1$, $A1.2$, etc.

Example 17.1. Determine the quantity of each size of tile required for the drainage system shown in Fig. 17.8. The spacing of the lines is 100 feet and the slope in Main A is 0.2 per cent, $A1$ and $A2$ are 0.12 per cent, and all remaining laterals are 0.3 per cent. Design for field crops on mineral soil in Ohio. Minimum size tile is 5-inch.

Solution. From Table 17.2, select a drainage coefficient of ⅜ inch. Assume that Main A above 2 + 80 drains only a 50-foot width on one side of the tile, and all other tile lines drain a 100-foot width. By calcu-

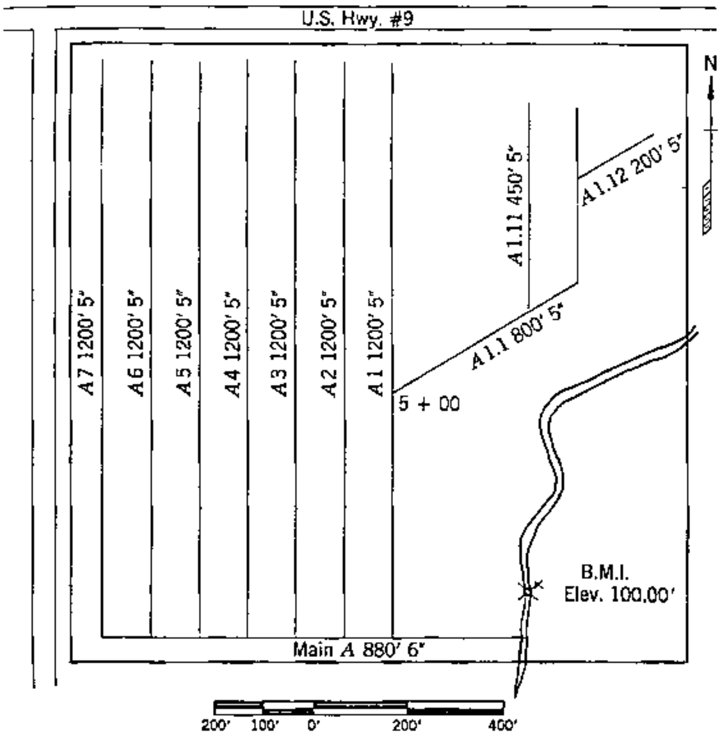

Fig. 17.8. A suitable tile drainage map.

lation from the Manning formula or from Fig. C.5, Appendix C (see Example C.2), the following lengths of tile are required:

Line	*Slope, %*	*D.A.,* acres	*Tile Dia.*	*Length,* ft
Laterals				
*A*3 to *A*7	0.3	2.76 ea.	5	6,000
*A*2	0.12	2.76	5	1,200
*A*1.1, *A*1.11, A1.12	0.3	3.33	5	1,450
*A*1	0.12	6.09	5	1,200
Main *A* at				
4 + 80 above *A*3	0.2	11.50	5	400
2 + 80 above *A*1	0.2	17.02	6	200
0 + 00	0.2	23.95	8	280

Assuming 3 per cent breakage, 10,558 5-inch, 206 6-inch, and 288 8-inch tile are required as well as 1 8×5-, 2 6×5-, and 7 5×5-inch manufactured junctions.

MOLE DRAINAGE

Mole drainage is accomplished by pulling a plug through the soil, which compresses the subsoil and forms a cylindrical mole channel. Cross-section and profile views of a mole drain are shown in Fig. 17.9. As indicated, the soil is fractured above and to the side of the channel.

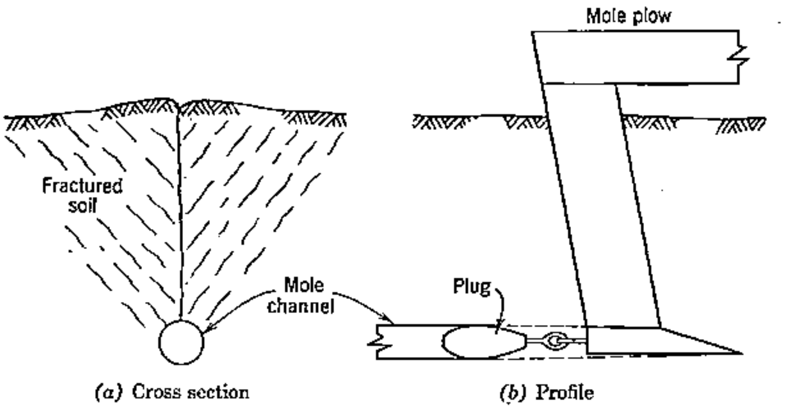

Fig. 17.9. Mole drainage: (a) cross section of a mole channel, and (b) method of forming a mole drain.

Moling is a temporary method of drainage. Where soil conditions are suitable, moles function efficiently for the first few years and then gradually deteriorate. Mole drains have been quite successful in England and New Zealand. In these countries their maximum life is as much as 10 to 15 years. Except in parts of Louisiana and Florida, mole drainage in the United States has not been generally effective.

17.27. Factors Affecting the Durability of Mole Drains. The following factors influence the life of mole drains: (1) structural stability of the subsoil, (2) soil moisture content at time of moling, (3) amount and intensity of rainfall, (4) seasonal temperature variations, (5) depth, (6) diameter of mole channel, and (7) installation practices. Since mole drains fail principally because of soil falling into the channel, the stability of the soil is very important. Although high clay soils are generally the most suitable, the clay content is not necessarily a good index for moling. In Iowa a study of three soils showed that the greater the clay content the more rapid the failure of the channel. No laboratory technique, applicable to all conditions, has yet been developed for determining the suitability of a soil for moling. The moisture content at the time of moling should be as high as possible, provided the soil can support the tractor. High-intensity rains may cause soil to wash into the channel, particularly if the rain occurs soon after installation. Freezing and thawing apparently have a tendency to cause deterioration and failure. The depth of moling usually varies from a minimum of 1.5 to 4 feet, depending on the availability of equipment and the depth to the most stable subsoil. Mole channels usually range in diameter from 3 to 6 inches, although the small sizes are probably more stable than larger ones.

17.28. Design and Layout. Where mole drains are suitable, they are laid out similarly to tile drains. Mole drains may empty into open ditches, tile lines, or other mole channels. If the moles empty into open ditches, a short length of metal tubing may be useful in protecting the outlet. Mole drains may be pulled across tile lines backfilled with permeable material.

The spacing and depth of mole drains is generally less than that of tile. Since mole drains are often installed in heavy soils and are low in cost, spacings ranging from 5 to 30 feet are suitable, depending on the depth and degree of drainage desired. In Florida, moles have been installed at a depth of 30 inches and a spacing of 12 feet.[4]

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PROBLEMS

17.1. Calculate the discharge of a 6-inch tile flowing full if the slope is 0.4 per cent.

17.2. How many acres will a 10-inch tile drain with a slope of 0.3 per cent if the tile are placed in mineral soil in North Carolina and field crops are to be grown?

17.3. Determine the quantity of each size of tile required for a gridiron system as shown in Fig. 17.1c, if the length of each of the five laterals is 1310 feet and the main is 1000 feet. Slope in the laterals is 0.12 per cent and in the main 0.3 per cent. Spacing of the laterals is 100 feet. Design the system for field crops in your area.

17.4. What size tile is required to remove the surface water through a surface inlet if the runoff accumulates from 20 acres and the slope in the tile is 0.4 per cent? Design for field crops in your area.

17.5. If a surface inlet draining 20 acres is added at the upper end of the main in Problem 17.3, what size tile are required from the surface inlet to the outlet?

17.6. What slope is required to provide a velocity of 1.5 fps at full flow in a 4-inch tile? In a 5-inch? In a 12-inch?

17.7. If a tile drainage system draining 30 acres flows full for 3 days following a period of heavy rainfall, determine the effluent volume during this period if the system was designed for a D.C. of ½ inch.

17.8. The grade elevation of a 16-inch main at the junction of a 5-inch lateral is 91.62 feet. Assuming a wall thickness of ½ and 1 inch for the 5- and 16-inch tile, respectively, what should be the grade elevation for the lateral?

17.9. Determine the depth of flow as a percentage of tile diameter to obtain maximum discharge through a tile.

CHAPTER 18

Installation and Maintenance of Subsurface Drains

TILE DRAINAGE

Although a tile drainage system is adequately designed and is staked out according to plan, it will not function satisfactorily unless properly installed and maintained. The tile trench should be dug to the specified grade, the bedding conditions and width of the trench should be such as to prevent overloading of the tile, good workmanship should be secured in laying the tile and in making junctions, the cost of installation should be reasonable, and the drainage system should be mapped and properly recorded. Proper maintenance is essential to the continued satisfactory operation of the system.

INSTALLATION PRACTICES

Installation should always begin at the outlet and progress upstream so that seepage water is free to move to the outlet. The installation procedure includes digging to an established grade, laying the tile, and backfilling.

18.1. Trenching Methods. Tile are usually installed either by hand digging or with a trenching machine. Occasionally tile are pulled into the soil with a mole plow.[13]

Hand Digging. Hand methods are most applicable for small jobs, for repair and maintenance of existing drainage systems, and for installations where soil and climatic conditions will not permit the use of machinery. Generally, hand tilers can work under more adverse soil conditions than is possible with machines.

Trenching Machines. Trenching machines may be divided into four general classes: (1) plows and scoops, (2) wheel excavators, (3) endless-chain excavators, and (4) hoe excavators, as shown in Fig. 18.1. Plows and scoop-type machines are sometimes used to loosen the soil so that it can be removed more easily by hand. However, the type shown in Fig. 18.1 carries the soil out of the trench on a conveyor. Wheel exca-

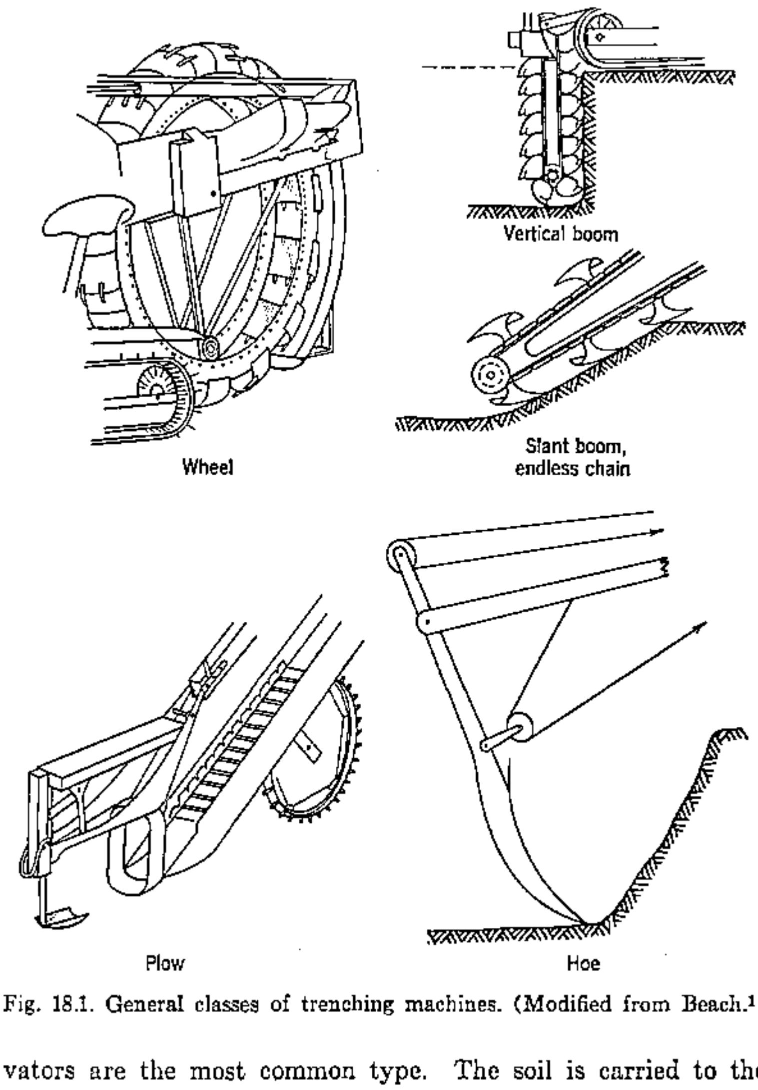

Fig. 18.1. General classes of trenching machines. (Modified from Beach.[1])

vators are the most common type. The soil is carried to the top of the wheel and then dropped onto a conveyor belt. Endless-chain excavators may be subdivided into two groups: the vertical-boom and the slant-boom type. The vertical-boom type reduces the amount of handwork where obstructions, such as pipelines or cables, are encountered. Hoe excavators were previously described in Chapter 15. On many hoe-type machines the digging bucket is simply a concave blade of the desired width. This class of excavator is suitable for making deep cuts, removing large stones, and making junctions. They also work satisfactorily in wet subsoil.

A trenching machine should dig a trench to a uniform grade under a variety of soil conditions. A crumber (shoe) following the digging mechanism is helpful, as it obviates the necessity of hand finishing the ditch bottom. To ensure good bedding conditions and to align the tile, the crumber should make a curved or V-shaped bottom in the trench. If the shoe is curved, a keel for making a groove should be attached to the bottom. Machines not equipped with facilities for maintaining grade should not dig closer than about 6 inches to the grade line, the bottom of the ditch then being finished by hand. Trenching machines having a long wheelbase generally maintain a more satisfactory grade than shorter machines, and heavy machines are more suitable for deep cuts and wide trenches.

The factors that influence the rate of installation for a trenching machine are: (1) soil moisture; (2) soil characteristics, such as hardness, stickiness, stones, and submerged stumps; (3) depth of trench; (4) condition of the trenching machine; (5) skill of the operator; (6) width of the trench; and (7) delays due to interruptions during operation. An extremely wet soil may stop machine operation; soil with a low moisture content may not affect the digging rate to any extent. Soils that stick to the buckets of the machine, sandy soils that fall apart easily, stones in the subsoil, and deep cuts reduce digging speeds. Increasing the depth from 3 to 5 feet decreased the digging speed by 56 per cent under Iowa and Minnesota conditions.[3] Although the width of the trench may not be important for machines with sufficient power, the rate of installation may be reduced especially for trenches wider than 16 inches.

The daily output for a trenching machine at depths from 3 to 5 feet is given in Appendix F. A ditching machine operated by a contractor in northern states can be expected to install tile only about 150 working days during the year.[3] Studies showed that a total of 66.2 per cent of the available working hours were lost. This time loss is accounted for as follows: weather 18.6 per cent, repairs 14.0 per cent, making junctions 10.1 per cent, miscellaneous 9.3 per cent, moving from job to job 8.2 per cent, and servicing of the machine 6.0 per cent.[3]

18.2. Methods of Establishing Grade. Regardless of whether the trench is dug by hand or by machine, the two principal ways of establishing grade are the line and the sight

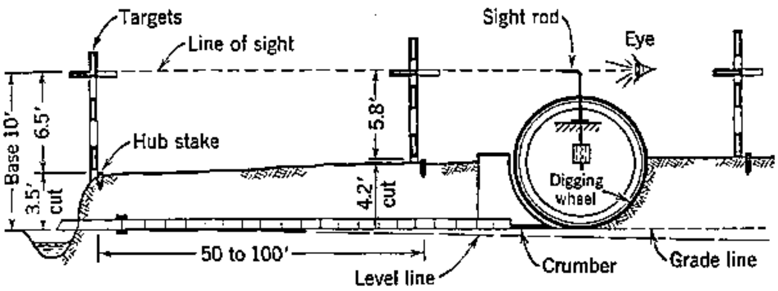

Fig. 18.2. Setting targets for trenching machines.

methods. These methods are essentially the same except that string is stretched between stations in the line method and sighting targets are set at each station in the sight method. As shown in Fig. 18.2, the depth of the trench is established by sighting over two targets. The line of sight or string line must be parallel to the grade line of the tile. The distance from the line of sight to the grade line is known as the *base distance*, which in Fig. 18.2 is 10 feet. The target is set near the hub stake, and the cut is subtracted from the *base distance* to obtain the difference in elevation between the top of the hub and the line of sight. For example, at the first station the target is 6.5 feet (10 — 3.5) above the hub stake. At least three consecutive targets should be set so that errors in design or calculation may be detected.

18.3. Laying the Tile. Tile should be laid on true grade with the bottom of the trench properly shaped to provide good alignment. The crack spacing between individual tiles should be about ⅛ inch, unless the soil is sandy. In unstable soils the crack spacing should be as small as possible to prevent inflow of fine sand. Where crack spacings are more than ⅛ inch in sandy soils and ¼ inch in clay soils, the openings should be covered with pieces of broken tile or other durable material. For this purpose continuous strips of coal-tar-impregnated paper are suitable as well as easy to install. If the tile are not square on the ends, they may be rotated so that the widest opening will be on the bottom and the narrowest opening on top.

If for some reason the trench is excavated below grade, it should be refilled to the desired grade with well-graded gravel. If the trench does not have water in it, moist soil may be suitable if thoroughly compacted under the tile.

The upper ends of all tile lines should be closed tightly with flat slabs of concrete, clay blocks, or stones. Tile bats are not suitable unless sealed with concrete.

18.4. Junctions. A properly made junction is an essential part of the tile system. The lateral may connect with the main at an angle of 90 degrees; however, where full flow is expected, an angle of 45 degrees directs the water downstream, thus resulting in less turbulence. If the lateral crosses the main at an angle greater than 45 degrees, the last 5 or 10 tile can be placed on a curve with a radius of not less than 5 feet. Cracks more than ¼ inch wide should be closed by chipping the tile on the ends and by using tile bats (broken tile) to cover the cracks. Where a difference in elevation between the main and the lateral is allowed at the junction, the tile are connected as shown in Fig. 17.7. If the laterals are short, T-junctions are satisfactory, otherwise Y-junctions should be used.

Manufactured junctions are more satisfactory than those fabricated in the field. A homemade junction can be fitted together by chipping and breaking ordinary tile and by using concrete mortar completely around the connection. Tile bats alone are not suitable.

18.5. Checking Grade. Immediately after installation and before backfilling, the tile should be checked for grade, alignment, and other design specifications. Allowable variation from true grade should be within reasonable limits. Some engineers allow a variation of 0.01 foot per inch of tile diameter for sizes up to 10 inches. Under no circumstances should there be backfall in the tile.

18.6. Blinding. As shown in Fig. 18.3, blinding is accomplished by placing 6 to 8 inches of loose, mellow topsoil in the trench immediately after laying the tile. The purpose of blinding is to prevent the tile from being moved out of alignment, to prevent breakage during the backfilling operation, and to provide permeable soil in contact with the tile. If stones are present in the soil, the minimum depth of blinding should be

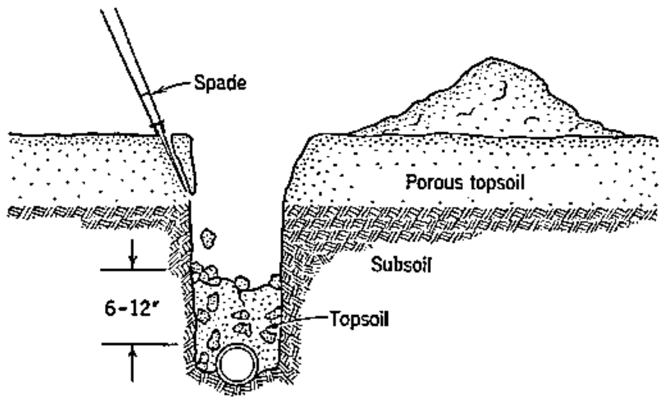

Fig. 18.3. Blinding the tile.

about 12 inches. In tight soils it may be desirable to backfill the trench with corncobs, cinders, well-graded gravel, and other porous materials. However, such materials are normally not effective for more than a few years.

18.7. Backfilling. Backfilling of the tile trench can be accomplished with many machines, such as bulldozers, hoes, graders, manure loaders, and blades on small tractors. All soil should be replaced and heaped up so that after settlement the trench can be crossed with tillage equipment. If the soil contains stones, extreme care should be taken to prevent breakage of the tile.

SPECIAL INSTALLATION PROBLEMS

18.8. Peat and Muck. Before tile are installed in peat and muck soils the land should be surface-drained to secure initial subsidence. The tile should be laid on the underlying mineral soil, provided the mineral soil is not too deep and not impermeable. In peat or muck soils 6-inch tile is often recommended as the minimum size because of differential subsidence.

18.9. Quicksand. Quicksand is not a type of sand but rather a soil condition in which fine sand in a saturated, fluid condition is buoyed up by hydrostatic pressure from below. Some of the following recommendations may be helpful in planning and installing tile drains under quicksand conditions: (1) install tile only during the driest season; (2) use only tile of high quality which are particularly straight, have square ends, and are uniform in diameter and wall thickness; (3) install tile which are at least 5 or 6 inches in diameter; (4) use grades of not less than 0.4 per cent; (5) protect all cracks between individual tile with durable materials, covering the upper two-thirds of the tile circumference; (6) install each tile line as rapidly as possible and without interruptions to prevent settlement of the crumber on the machine; (7) blind the tile with coarse material, such as gravel, sawdust, or sod; (8) prevent the caving-in of the ditch bank prior to and during backfilling operations; and (9) provide careful maintenance of the tile line, particularly if sinks or blowholes develop.[4] Under some situations, reducing the hydrostatic pressure by pump drainage may be practicable.

18.10. Gravel Envelope. In irrigated areas of the West it is common practice to place gravel in the bottom of the trench before the tile is laid and to surround the tile with a layer of gravel about 2 inches in thickness.[14] Although the gravel envelope reduces friction losses owing to crack restrictions, the purpose of the gravel envelope is largely to filter out the sand. In humid regions gravel envelopes are not normally recommended.

LOADS ON CONDUITS

Loads on underground conduits include those caused by the weight of the soil and by concentrated loads due to the passage of equipment or vehicles. At shallow depths concentrated loads largely determine the strength requirements of conduits; at greater depths the load due to the soil is the most significant. Loads for both conditions should be determined, particularly where the depth limitations of the controlling load are not known. For drain tile the weight of the soil usually determines the load. Concentrated loads may be calculated by methods described in Appendix G.

18.11. Types of Conduit Loading. For purposes of analyzing loads, the two types of underground conduits are ditch conduits and projecting conduits, as shown in Fig. 18.4. Ditch conduit conditions apply to narrow trenches, and projecting con-

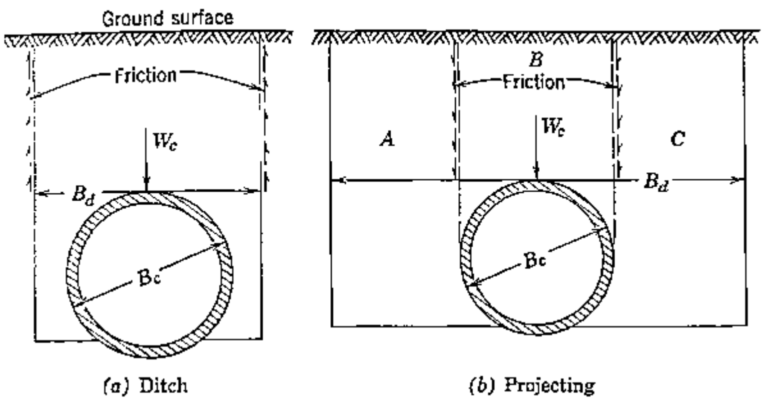

Fig. 18.4. Ditch- and projecting-type conduits. (Redrawn from Van Schilfgaarde and others.[12])

duit conditions to trenches wider than 2 or 3 times the outside diameter of the tile. Projecting conditions generally exist when the settlement of soil prisms A and C shown in Fig. 18.4b is greater than prism B. Since this condition applies to conduits placed under an embankment on undisturbed soil as in road fills or pond dams, the conduit projects above a relatively solid surface; hence its name.

18.12. Load Factors Based on Bedding Conditions. The load factor is the ratio of the strength of a conduit under given bedding conditions to its strength as determined by the three-edge bearing test. As shown in Fig. 18.5, four classes of bedding conditions are generally recognized.

For nonpermissible bedding conditions, shown in Fig. 18.5a, no attempt is made to shape the foundation or to compact the soil under and around the conduit. Such a bedding has a low load factor (1.1) and is not suitable for drainage work.

For ordinary bedding conditions (L.F. 1.5) the bottom of the trench must be shaped to the conduit for at least one-half its width. The conduit should be surrounded with fine, granular soil extending at least 0.5 foot above the top of the tile.

For first-class bedding the conduit must be placed in fine, granular material for 0.6 the conduit width and should be entirely surrounded with this material extending 1 foot or more above the top. In addition, the blinding material should be placed by hand and thoroughly tamped in thin layers on the sides and above the conduit. Although the load factor for first-

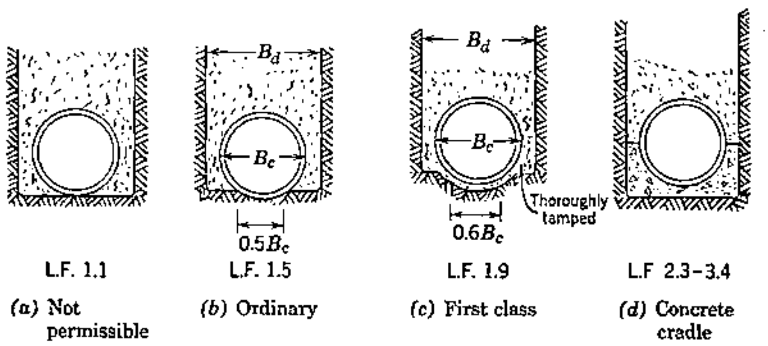

Fig. 18.5. Typical bedding specifications for ditch conduits. (Redrawn from Spangler,[11] p. 873, Fig. 13.)

class bedding is 1.9, it is seldom necessary in tile drainage work.

The concrete cradle bearing is constructed by placing the lower part of the conduit in plain or reinforced concrete. Such construction is not practical in conservation work except for earth dams or high embankments where excessive loads are encountered. This type of bedding provides the best conditions, with load factors varying from 2.2 to 3.4.

Based on a study of trench bottoms made by several ditching machines, load factors were found to vary from 0.9 to 2.5.[12] With a keel on the bottom of a curved trencher shoe or a V-shaped bottom, a load factor of 1.5 or greater may be expected.

18.13. Load Analyses. Loads on both ditch conduits and projecting conduits were investigated by Marston,[6] Spangler,[11] and Schlick.[8] For loads on ditch conduits it is assumed that the density of the fill material is less than that of the original soil. As settlement takes places in the backfill, the sides of the ditch resist such movement (Fig. 18.4*a*). Because of the upward frictional forces acting on the fill material, the load on the conduit is less than the weight of the soil directly above it. In its simplest form the ditch conduit load formula is:[6]

$$W_c = C_d w B_d^2 \tag{18.1}$$

where W_c = total load on the conduit.
C_d = load coefficient for ditch conduits.
w = unit weight of fill material in pounds per cubic foot.
B_d = width of ditch at top of conduit.

The backfill directly over a projecting conduit (prism B in Fig. 18.4*b*) will settle less than the soil to the sides of the conduit (prisms A and C). For projecting conditions the load on the conduit is greater than the weight of the soil directly above it, because shearing forces due to greater settlement of soil on both sides are downward rather than upward. The projecting conduit formula for wide ditches is:[6]

$$W_c = C_c w B_c^2 \tag{18.2}$$

where C_c = load coefficient for projecting conduits and B_c = outside diameter of the conduit. The load coefficients C_d and C_c are functions of the frictional coefficient of the soil, the height of fill, and, respectively, the width of the ditch or the width of the conduit. Since these coefficients are rather complex, C_d and C_c curves, shown in Fig. 18.6, have been developed. The load on the conduit is the smaller value as computed from equation 18.1 or equation 18.2 (see Example 18.1). The width of trench for a given conduit that results in the same load when computed by both equations is known as the transition width.

Drain tile should be installed so that the load does not exceed the required average minimum crushing strength of the tile, as given in Appendix E. Whenever practicable, ditch conduit conditions should be provided rather than projecting conduit conditions. To prevent overloading in wide trenches, a narrow subditch can be excavated in the bottom of the main trench. The width of the subditch measured at the top of the tile determines the load, regardless of the shape of the trench above this point. The subditch need not extend above the top of the tile.

A common method of construction in deep cuts is to remove the excess soil with a bulldozer and to dig the subditch with a trenching machine.

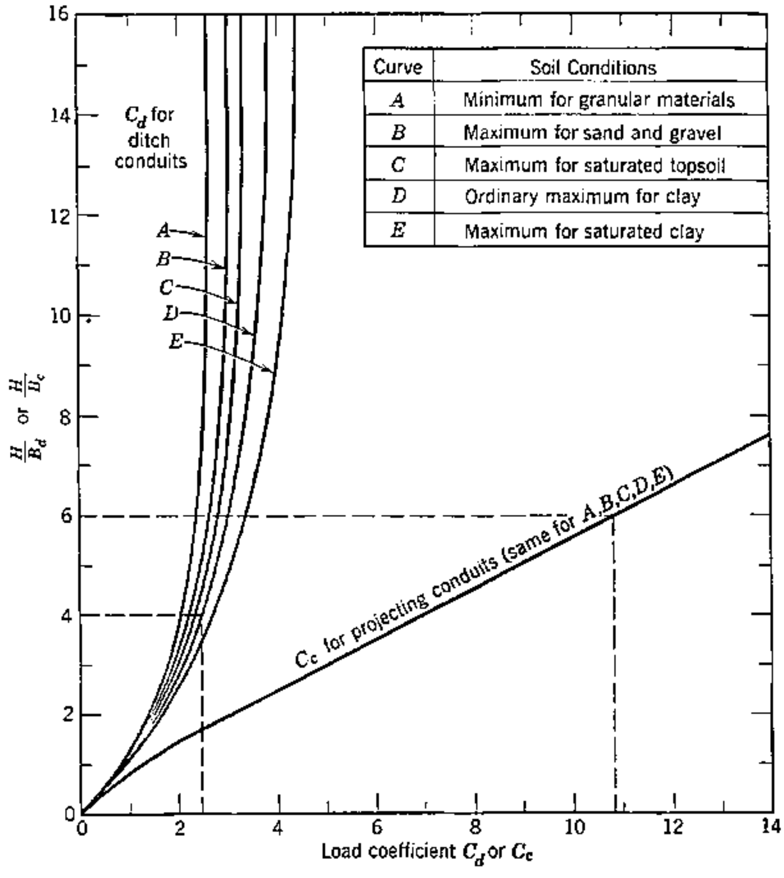

Fig. 18.6. Load coefficients for rigid drain tile. (From Spangler.[11])

A nomograph for computing loads for ordinary clay backfill is given in Appendix G. For trench widths of 16 and 18 inches or less, standard-quality and extra-quality tile, respectively, may be installed at any depth.

Example 18.1. Determine the static load on 10-inch tile (B_c = 1.0 ft) installed 7.0 feet deep in a ditch 18 inches wide if the backfill is ordinary clay weighing 120 pcf. What quality tile is required, assuming ordinary bedding conditions?

Solution. $H = d - B_c$, where H = depth to top of tile and d = depth to bottom of tile. $H = 7.0 - 1.0 = 6.0$. Read from Fig. 18.6 for H/B_d of 4.0, $C_d = 2.4$. Substitute in equation 18.1 to obtain load for ditch conditions.

$$W_c = 2.4 \times 120 \times 1.5^2 = 648 \text{ lb/lin ft}$$

Read from Fig. 18.6 for H/B_c of 6.0, $C_c = 10.8$. Substitute in equation 18.2 to obtain load for projecting conditions.

$$W_c = 10.8 \times 120 \times 1.0^2 = 1296 \text{ lb/lin ft}$$

The lower value, 648, is the actual load; hence ditch conditions apply. To allow for variation in tile strength and bedding conditions include a safety factor of 1.5, which results in a design load of (1.5 × 648) 972 lb/lin ft.

From Fig. 18.5 the load factor for ordinary bedding is 1.5, and from Appendix E the supporting strength of standard-quality tile is 800 lb/lin ft, based on the three-edge bearing method. Strength for ordinary bedding conditions is 1200 lb/lin ft (1.5 × 800), which is sufficient to support a design load of 972 lb/lin ft.

ESTIMATING COST

The three principal items of cost for a tile drainage system are installation, engineering, and materials cost, such as tile, outlet structures, and surface inlets. In the Midwest, the total cost of tiling for 4-foot depth and 5-inch tile is approximately 45 per cent for materials, 45 per cent for installation, and 10 per cent for engineering. In California, the total cost for 6-inch tile at depths of 6 to 6½ feet is about 3 times the cost of the tile; that is, labor and installation costs constitute about two-thirds of the total.[15]

18.14. Installation Cost. The cost of installation depends primarily on the method of installation; depth of cut; size of tile; presence of unusual soil conditions, such as stones and quicksand; and the number of junctions. Most ditching machine contractors install tile at a fixed price per unit length, including trenching, laying the tile, blinding, and backfilling. An additional charge is usually made for each 0.1 foot overcut below a specified depth and for 8-inch tile or larger. Frequently, contractors charge extra for removing stones and for making junctions.

18.15. Material Costs. The principal materials to be considered are: tile, tile junctions, gravel, and material for constructing accessory facilities, such as outlets and surface inlets. The relative cost of drain tile is given in Table 18.1. Sewer tile or metal pipe are suitable for outlet structures, surface inlets, and relief wells. Manufactured junctions either T- or Y-shape usually cost about 10 times as much as one length of ordinary tile.

Table 18.1 RELATIVE COST OF DRAIN TILE BY SIZE

Nominal Size, in.	*Relative Cost*
4	80
5	100
6	130
8	210
10	350
12	480

18.16. Engineering and Supervision Costs. Engineering and supervision costs normally vary from 5 to 10 per cent of the total cost. Engineering services should include the preliminary survey, location of the tile, staking the line, designing the system, and checking the grade and other specifications after installation.

18.17. Other Costs. Where the system is large and more than one property owner is involved, other costs may include the charge for the right-of-way, clearing of trees, removal and replacement of fences and bridges, enlargement of outlet ditches, etc. These costs vary widely for different installations.

MAPPING THE DRAINAGE SYSTEM

A suitable map should be made of the drainage system and filed with the deed to the property. By having a good map, additional tile lines can be added, resulting in an effective system without unnecessary expense. Much time, effort, and money have been wasted because the location of old lines was not known. A record of the tile system is also of considerable value to present and future owners for maintenance and repair. The essential features of a tile drainage map are shown in Fig. 17.8. The drainage system may also be placed on a topographic map or on an aerial photograph. An aerial photograph is more desirable because it permits easier location of the tile lines.

MAINTENANCE

18.18. Causes for Tile Drain Failures. As a result of 30 years' experience in 36 states, Shafer[9] classified all tile drain failures into five categories: (1) lack of inspection or maintenance, (2) improper design, (3) improper construction, (4) manufacturing processes and materials used, and (5) physical structure of the soil. As an indication of the relative importance of these classes of failures, a survey in Ohio[9] showed that the per cent of failure due to each of these causes was (1) 28.5, (2) 27.9, (3) 23.0, (4) 20.6, and (5) none, respectively. Although failures caused by poor physical structure of the soil are not very extensive, the installation of tile in such soils might be properly classified as poor design. In other instances improper crop and soil management practices may cause destruction of the soil structure, resulting in partial or complete failure of the drainage system. The principal causes of concrete and clay tile failures are the lack of resistance to freezing and thawing and inadequate strength. In design the major causes of failure are insufficient capacity, tile placed at too shallow a depth, and lack of auxiliary structures, such as surface inlets. Improper construction, which in the survey accounted for 23.0 per cent of the failures, results from too wide crack spacings between the tile, improper bedding conditions, poor junctions, nonuniform grade, careless backfilling, and poor alignment. The lack of inspection and maintenance in the above survey was the major cause for failure, representing 28.5 per cent of the total. Such failures are due mainly to the washout of surface inlets and outlet structures, piping over the tile line, clogging of tile from root growth, and failure to keep the outlet in good condition.

18.19. Preventative Maintenance. In comparison to open ditches tile drains require relatively little maintenance. Where open ditch outlets are provided, the ditch should be kept free of weed and tree growth, and the end of the outlet pipe should be covered with a flood gate, a screen, or a flap gate. The outlet ditch must not fill with sediment so as to obstruct the flow of the water from the tile. Surface water should not be diverted into the ditch at or near the tile outlet. Where shallow depths are required, tile should be protected from tramping of livestock and should not be crossed with machinery, particularly during wet periods.

Tile should be kept free of sediment and other obstructions. Roots of trees and certain other plant roots may grow into the tile and obstruct the flow, especially if the tile is fed by springs supplying water far into the dry season. Brush and trees, particularly willow, elm, cottonwood, soft maple, and eucalyptus must be removed if within 100 feet of the tile line. Where it is impractical to remove this vegetation near the tile, sealed bell and spigot tile, tar-impregnated pipe, or metal pipe should be installed. Sugarbeet and alfalfa roots have been known to enter tile lines, but these plants usually do not cause trouble because the roots die and are washed down the drain.

During the first year after installation tile lines should be carefully watched to detect evidences of failure. Sinkholes over the line indicate a broken tile or too wide a crack spacing. Such holes should be inspected and the necessary repairs made. Surface water should be diverted across the tile trench since this water may eventually wash out the tile. Sediment basins should be cleaned at regular intervals. Surface inlets must be kept free of weed growth and sediment.

18.20. Corrective Maintenance. Tile which become filled with sediment or plant roots may be cleaned by digging holes along the tile line every 50 feet or at shorter intervals. Between these holes the material in the tile may be removed with a suitable plug, swab, or sewer rod. Where sufficient water and tile grade are available, sediment can be washed out. If the above methods are not practical, it may be more economical to reinstall the entire line. Broken tile failures may be located by digging down to the tile at various points along the line. If water rises in any of the resulting holes, the failure is nearer the outlet.

MOLE DRAINAGE

18.21. Installation and Maintenance. Where mole drainage is feasible, each mole drain is normally pulled in from an open ditch. A variety of mole plows are available for installing mole drains. Types having hydraulic controls on the blade for maintaining grade are the most suitable.

Track-type tractors are required for pulling in mole drains. The power requirements vary from about 30 to 70 horsepower for depths of 2 and 3 feet, respectively. Smaller power units equipped with winches are also satisfactory.

The best time to install mole drains is when the soil surface is dry and firm enough to support the power unit, and at the same time the subsoil is sufficiently wet and plastic to produce a smooth channel behind the mole plug. Where the soil is too dry power requirements are high, excessive fracture of the soil takes place, and a smooth, stable channel cannot be formed.

Because mole drains are temporary, little maintenance can be justified. It is generally desirable to pull in new drains where old ones have failed. Mole channels with considerable slope should be watched for evidence of erosion since such a channel may become the source of a gully.

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PROBLEMS

18.1. Select the grade for a 600-foot tile line outleting into an open ditch so that cuts do not exceed 4.2 feet. Design for an average depth of 4 feet. Elevation of the water surface in the ditch is 82.50, and hub elevations of successive 100-foot stations are 86.81, 87.29, 87.92, 88.15, 88.40, 88.61, and 88.79.

18.2. Determine the static load on 12-inch drain tile installed at a depth of 10 feet in a trench 24 inches wide, assuming that the maximum weight of saturated topsoil in the backfill is 110 pcf. Calculate the design load, using a safety factor of 1.5.

18.3. What quality of drain tile are required to withstand the design load as determined in Problem 18.2 if ordinary bedding conditions are provided?

18.4. For the tile in Problem 18.3, what width of trench should be dug to permit the installation of standard-quality tile?

18.5. Based on the design load, what bedding conditions are necessary if only standard-quality tile were available for the installation described in Problem 18.2?

18.6. Estimate the cost of the drainage system in Fig. 17.8. The average cut in the main is 4.5 feet and for the laterals is 4.0 feet. *Installation cost:* 4.0 feet or less, $12 per 100 ft.; overcut, $0.30 per 0.1 foot per 100 ft. *Materials:* Tile price based on Table 18.1, using $105 per *M* for 5-inch tile and adding 5 per cent for breakage; corrugated metal outlet pipe at $1 per foot for 8-inch pipe; tile junctions at $1 each. *Engineering:* 5 per cent of total materials and installation costs.

CHAPTER 19

Pumps and Pumping

Although the agricultural engineer is not generally required to design pumping equipment, he should be able to determine the proper type of pump, the number and size of pumps required, and the size of power units. In addition, he may be called upon to design the plant installation, estimate the cost of operation, and supervise construction and operation of the plant.

Pumping plant installations are encountered principally in drainage and irrigation enterprises. Pumping plants in drainage provide outlets for open ditches and tile drains. In irrigation, pumping from wells and storage reservoirs into irrigation canals or other reservoirs is common practice.

TYPES OF PUMPS

Of the many types of pumps available, the centrifugal, propeller, and reciprocating pumps are by far the most common. From these three types, pumps may be selected for a wide range of discharge and head characteristics. Reciprocating pumps, sometimes called piston or displacement pumps, are capable of developing high heads, but their capacity is relatively small. These pumps are common for home water systems. They are not ordinarily suitable for drainage and irrigation, especially if sediment is present.

CENTRIFUGAL

Centrifugal pumps are economical in cost; simple in construction, yet they produce a smooth, steady discharge; small compared to their capacity; easy to operate; and suitable for handling sediment and other foreign material.

19.1. Principle of Operation. The centrifugal pump consists of two main parts: (1) the impeller or rotor which adds energy to the water in the form of increased velocity and pressure, and (2) the casing which guides the water to and from the impeller. As shown in Fig. 19.1, the water enters the pump at the center of the impeller and passes outward through the rotor to the discharge opening. By changing the speed of the pump, the discharge can be varied. Theoretically, the so-called pump laws for a centrifugal pump state that (1) the discharge

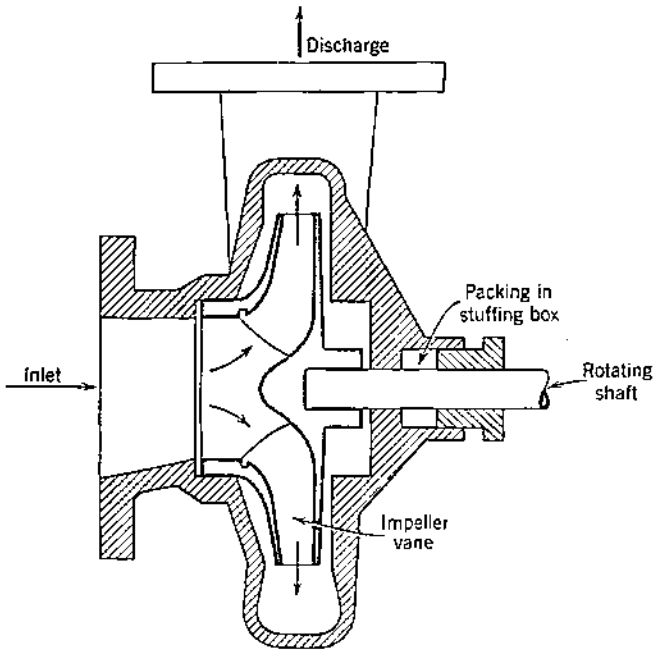
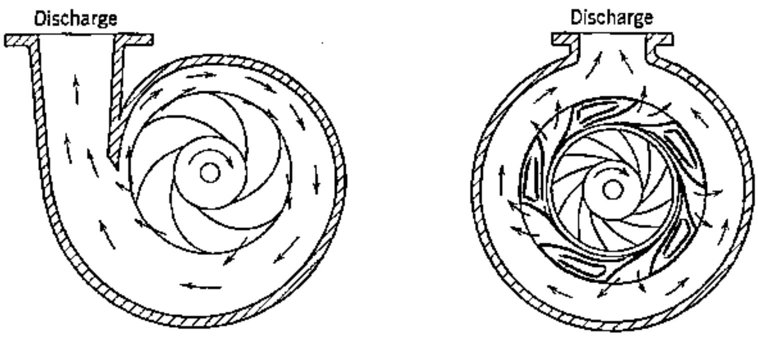


Fig. 19.1. Cross section of a horizontal centrifugal pump (single suction enclosed impeller). (Redrawn, by permission of the publisher, from Binder,[2] *Fluid Mechanics*, 2nd edition, p. 289, copyright 1949, by Prentice-Hall, N. Y.)

is directly proportional to the speed of the impeller, (2) the head varies as the square of the speed, and (3) the power varies as the cube of the speed. For a pump of a given size, changing the diameter of the impeller, thereby the peripheral velocity, has the same effect as changing the speed.

19.2. Classification. With regard to the construction of the casing around the impeller, centrifugal pumps are classified as volute or turbine, as shown in Fig. 19.2. The volute-type pump is so named because the casing is in the form of a spiral with a cross-sectional area increasing toward the discharge opening.

Turbine-type pumps, sometimes called diffuser-type, have stationary guide vanes surrounding the impeller. As the water leaves the rotor, the vanes gradually enlarge and guide the water to the casing, resulting in a reduction in velocity with the kinetic energy converted to pressure. The vanes provide a more uniform distribution of the pressure. Turbine pump casings may be either circular as shown or spiral like volute casings.

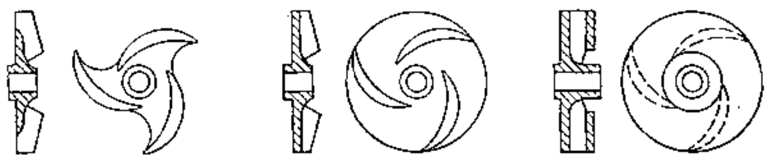


(a) Volute (b) Turbine

Fig. 19.2. (a) Volute- and (b) turbine-type centrifugal pumps. (Redrawn from Bennison.[1])

Centrifugal pumps are built with horizontal or vertical drive shafts and with different numbers of impellers and suction inlets. The suction inlet may be either single or double acting, depending on whether the water enters from both sides or one side of the impeller. Single-suction, horizontal centrifugal pumps are frequently used where the suction lift does not exceed 15 to 20 feet. Practically all turbine pumps are of the vertical type. Pumps with more than one impeller are known as multiple-stage pumps, sometimes referred to as deep-well turbine pumps. Both volute and turbine centrifugal pumps have multiple impellers, but they are more common in the turbine type. Deep-well turbine pumps are here considered as centrifugal pumps with one of the following types of impellers to be described: three centrifugal types, mixed flow, and propeller (see Fig. 19.5). Some authorities prefer to classify these pumps separately from ordinary centrifugals because all impellers do not function entirely on the centrifugal principle.

19.3. Centrifugal-Type Impellers. The design of the impellers greatly influences the efficiency and operating characteristics of the pump. Centrifugal-type impellers shown in Fig. 19.3 are classified as open, semienclosed, and enclosed. The open-type impeller has exposed blades which are open on all sides except where attached to the rotor. The semienclosed impeller has a shroud (plate) on one side; the enclosed impeller



(a) Open (b) Semienclosed (c) Enclosed

Fig. 19.3. (a) Open, (b) semienclosed, and (c) enclosed centrifugal-type impellers. (Redrawn from Stepanoff.[11])

has shrouds on both sides, thus enclosing the blades completely. The open and semienclosed impellers are most suitable for pumping suspended material or trashy water. Enclosed impellers are generally not suitable where suspended sediment is carried in the water as this material greatly increases the wear on the impeller.

In deep-well turbine pumps mixed-flow impellers, which are a combination of axial-flow and centrifugal-type impellers, are often used. Such impellers have a higher capacity than the centrifugal type. Water movement in the pump is the result of both centrifugal force and direct thrust.

19.4. Performance Characteristics. In selecting a pump for a particular job the relationship between head and capacity at different speeds should be known as well as pump efficiency. Curves that provide this data are called *characteristic* curves, as shown in Fig. 19.4. The head-capacity curve shows the total head developed by the pump for different rates of discharge. At zero flow the head developed is known as the shut-off head. Head losses in pumps are caused by friction and turbulence in the moving water, shock losses due to sudden changes in momentum, leakage past the impeller, and mechanical friction. For a given speed the efficiency can be determined for any discharge from these characteristic curves. A pump should be selected that will have a high efficiency for a wide range of discharges. For example, from the 2000 rpm curve in Fig. 19.4, 70 per cent efficiency or more can be obtained for capacities varying from 750 to 1320 gpm at heads of 230 and 170 feet, respectively.

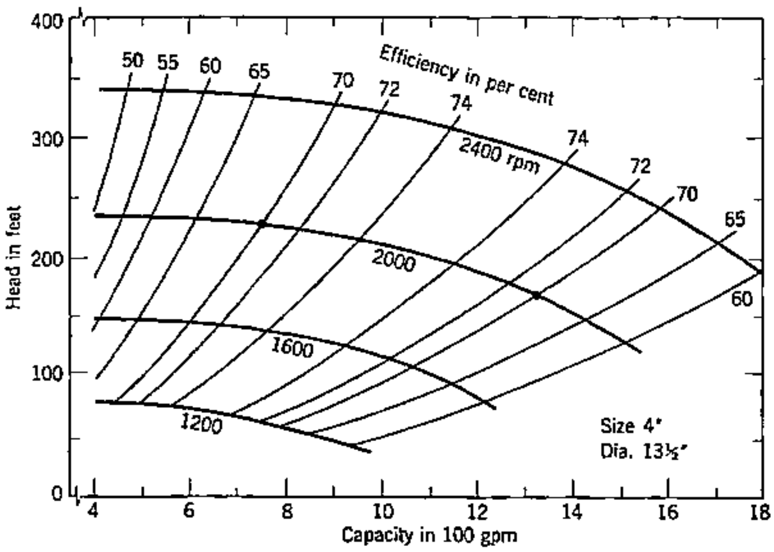
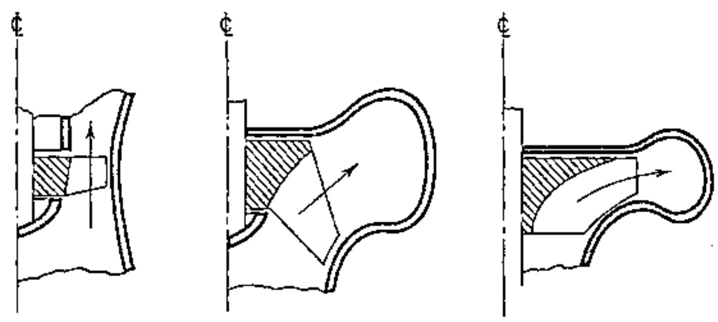


Fig. 19.4. Performance characteristics of a centrifugal pump. (Courtesy the Deming Company.)

Characteristic curves can be obtained from the pump manufacturer. Such curves will vary in shape and magnitude, depending on the size of the pump, type of impeller, and over-all design. Performance characteristics are normally obtained by tests of a representative production line pump rather than by tests of each pump manufactured.

PROPELLER

19.5. Principle of Operation. As distinguished from centrifugal pumps, the flow through the impeller of a propeller-type pump is parallel to the axis of the driveshaft rather than radial. These pumps are also referred to as axial-flow or screwtype pumps. The principle of operation is similar to that of a boat propeller except that the rotor is enclosed in a housing. The mixed-flow impeller is a combination propeller and centrif-



(a) Propeller (b) Mixed flow (c) Centrifugal

Fig. 19.5. Flow through propeller, mixed-flow, and centrifugal pumps.

ugal-type rotor. As shown in Fig. 19.5, there is considerable radial flow in mixed-flow pumps. Many propeller pumps have

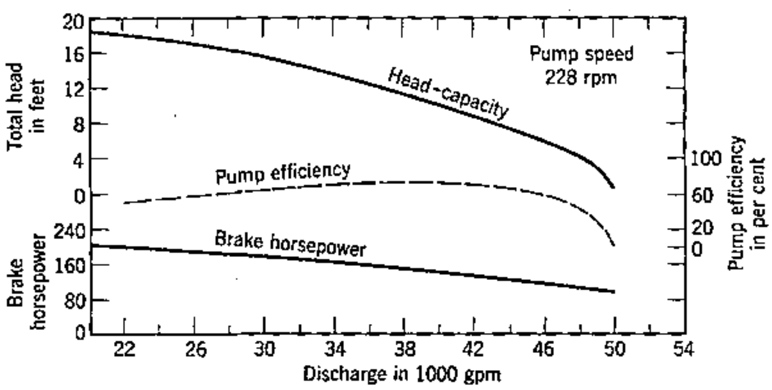


Fig. 19.6. Typical performance curves for a propeller-type pump. (Redrawn from Sutton.[13])

diffuser vanes mounted in the casing, similar to the turbine-type centrifugal pump.

19.6. Performance Characteristics. Propeller pumps are designed principally for low heads and large capacities. The discharge of these pumps varies from 600 to 70,000 gpm with speeds ranging from 450 to 1760 rpm and heads usually not more than 30 feet.[15] Although most propeller pumps are of the vertical type, the rotor may be mounted on a horizontal shaft.

Performance curves for a large propeller pump are shown in Fig. 19.6. This illustration represents a typical method of presenting characteristic curves for pumps operating at constant speed. In comparison to centrifugal pumps, the efficiency curve is much flatter, heads are considerably less, and the horsepower curve is continually decreasing with greater discharge. With propeller pumps the power unit may be overloaded by increasing the pumping head. These pumps are not suitable where the discharge must be throttled to reduce the rate of flow.

PUMPING

Although in many respects pumping water for irrigation is similar to that for drainage, design requirements differ. For example, in pump drainage heads are generally 20 feet or less, whereas for irrigation heads up to 150 feet are common.

19.7. Power Requirements and Efficiency. For a given discharge the power requirements for pumping are approximately proportional to the static head. This head or vertical lift is usually the difference in elevation between the water surface at the source and at the outlet. Where the discharge pipe is higher than the water at the outlet, the head is measured to the highest point. Power requirements for pumping are computed by the formula:

$$\mathrm{hp} = \frac{Qwh}{550\,E_p} \tag{19.1}$$

where hp = (input) horsepower delivered to pump.
Q = discharge rate, cubic feet per second or gallons per second.
w = specific weight of water, 62.4 lb/cu ft or 8.34 lb/gal.
h = total head in feet (suction plus discharge).
E_p = pump efficiency as a decimal fraction.

Considerable energy is utilized in overcoming friction losses in the pump, valves, and pipes; and for velocity and pressure head losses. Because of such losses pumping plant efficiencies range from about 75 per cent under very favorable conditions to as low as 20 per cent or less for unfavorable conditions.[5] A well-designed pump should have an efficiency of 70 per cent or more over a wide range of operating heads.[13] Such efficiencies in the field are difficult to maintain because of wear in the pump and other factors. It is good practice to check the field installation to be sure that satisfactory efficiencies are obtained. If efficiencies are low, the source of difficulty should be located and corrected, as discussed by references 1, 3, and 9. The over-all efficiency of the installation which includes the efficiency of the power plant and the pump should also be considered.

19.8. Power Plants and Drives. Power plants for pumping should deliver sufficient power at the specified speed with maximum operating efficiency. Internal combustion engines and electric motors are by far the most common types of power units. The selection of the type of unit depends on (1) the amount of power required, (2) initial cost, (3) availability and cost of fuel or electricity, (4) annual use, and (5) duration and frequency of pumping.

Internal combustion engines operate on a wide variety of fuels, such as gasoline, diesel oil, natural gas, and butane. Where the annual use is more than 800 to 1000 hours, the diesel engine may be justified.[15] Otherwise, it may be more practical to use a gasoline engine. For continuous operation water-cooled gasoline engines may be expected to deliver 70 per cent of their rated horsepower, diesel engines 80 per cent, and air-cooled gasoline engines 60 per cent. Where a vertical centrifugal or a deep-well turbine pump is to be driven with an internal combustion engine, the right angle gear drive is the most efficient method of transmitting power.[9] Belt drives may utilize either flat or V-belts suitable for driving vertical shafts or gear drives. V-belts are more efficient than flat belts.

Electric motors operate best at full-load capacity. They have many advantages over internal combustion engines, such as ease of starting, low initial cost, low upkeep, and suitability for mounting on horizontal or on vertical shafts. Direct drive motors are preferred because gears and belts are eliminated. Deep-well pumps are now available with watertight vertical motors. The motor is submerged in the well near the impellers, thus eliminating the long shaft otherwise required.

19.9. Selection of Pump. Performance curves serve as a basis for selecting a pump to provide the required head and capacity for the range of expected operating conditions at or near maximum efficiency. The factors that should be considered include the head-capacity relationship of the well or sump from which the water is removed, space requirements of the pump, initial cost, type of power plant, and pump characteristics, as well as other possible uses for the pump. Storage capacity, rate of replenishment, and well diameter sometimes limit the pump size and type. For example, a large drainage ditch provides a nearly continuous source of water, whereas a well usually has a small storage capacity.

The initial cost that can be justified depends largely on the annual use and other economic considerations. The size of the power plant and type of drive should be adapted to the pump. In off-seasons pumps used intermittently may be made available for other purposes.

Since the characteristics of pumps of different types vary widely, only centrifugal, mixed-flow turbine, and propeller pumps are compared in Table 19.1. The centrifugal pump is suitable for low capacities at heads up to several hundred feet, the propeller pump for high capacities at low heads, and the mixed-flow pump for intermediate heads and capacities. Since impellers of these pumps may be placed near to or below the water level, the suction head developed may not be critical. Horizontal centrifugal pumps are best suited for pumping from surface water supplies, such as ponds and streams, provided the water surface does not fluctuate excessively. Where the water level varies considerably, a vertical centrifugal or a deep-well turbine pump is more satisfactory.

Table 19.1 Comparison of Pumps of Different Types

Type of Pump	*Suction Head*	*Discharge Head*	*Discharge Capacity*
Centrifugal	Medium	High, 12 or over*	Medium
Mixed-flow turbine	Low-medium	Medium, 6 to 26*	Medium-high
Propeller	Low	Low, 10 or less*	High

* For efficient pumping. From Sutton.[13]

19.10. Pumping Costs. The cost of pumping consists of fixed and operating costs. Fixed costs include interest on investment, depreciation, taxes, and insurance. Included in the investment is the cost of constructing and developing the well, the pump, the power plant, pump house, and water storage facilities. Construction costs for wells depend largely on the size and depth of the well, construction methods, nature of the material through

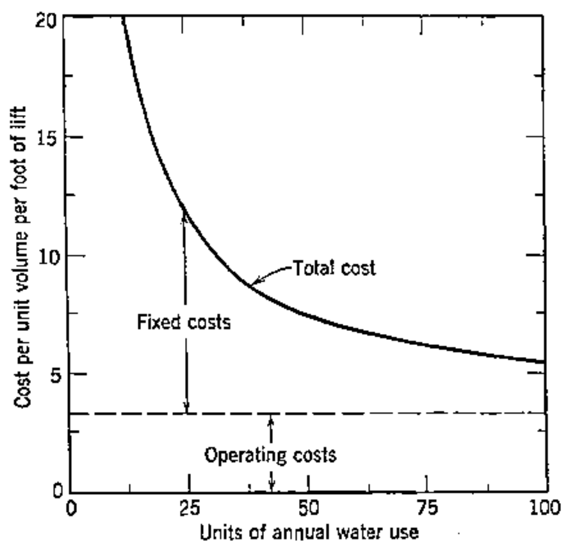


Fig. 19.7. Effect of annual water use on total pumping cost per unit of volume.

which the well is drilled or dug, type of casing, length and type of well screen, and the time required to develop and test the well. Recommended annual depreciation rate for electric motors is 5 per cent, internal combustion engines 10 to 15 per cent, centrifugal pumps 7 to 10 per cent, well and casing 3 per cent, and pump house 3 per cent. Taxes and insurance are approximately 1.5 per cent of the total investment. Operating costs include fuel or electricity, lubricating oil, repairs, and labor for operating the installation. Fuel costs for internal combustion engines are generally proportional to the consumption, whereas the cost of electrical energy decreases with the amount consumed. A demand charge or minimum is made each month regardless of the amount of energy consumed. This demand charge may or may not include a given amount of energy.

In a study of nine drainage pumping plants (electric power) in the upper Mississippi Valley, 32.6 per cent of the total cost was found to be for fixed costs, 52.9 per cent for power, 7.4 per cent for labor, and 7.1 per cent for other expenses. The average over-all efficiency for these plants was 41.8 per cent, the average lift 9.45 feet, and the average cost for electricity 1.76 cents per kilowatt-hour.[12]

The pumping cost per unit volume depends largely on the total quantity pumped per season. The effect of annual water use on the unit cost per foot of lift is shown in Fig. 19.7. The unit cost for 12 units of annual water use is about three times that for 100 units. Because of lower lifts and smaller investment, pumping for drainage is usually not as expensive as for irrigation.

PUMPING FOR DRAINAGE

Pumps offer a means of providing drainage where gravity outlets are not otherwise available. Typical applications of pump drainage include land behind levees, outlets for tile and open ditches, and flat land where the gravity outlet is at considerable distance. Drainage pumping plants are normally required to handle large capacities at low lifts.

19.11. Pumping Plant. The essential elements of a pump drainage installation are a pump with its power unit and a sump or other storage basin. Most pumps operate only during the wet spring months, but some installations draining extremely low land may operate more or less continuously throughout the year. Usually, these plants operate intermittently and are used only 10 to 20 per cent of the time.[8]

For economical performance it is better to operate a small pump for several days than to run a large pump intermittently for short periods. Frequently, drainage installations lack storage capacity and permit a small variation in head. Under these conditions automatically controlled electric motors are especially suitable. Internal combustion engines are usually started manually, but an automatic shut-off can be provided. The number of stopping and starting cycles should not exceed more than two per day for nonautomatic operation; several per hour are allowable for automatic controls. For this reason engine-operated plants require a larger storage capacity than those electrically operated. For large installations two pumps with different capacities may be necessary, one for handling surface runoff

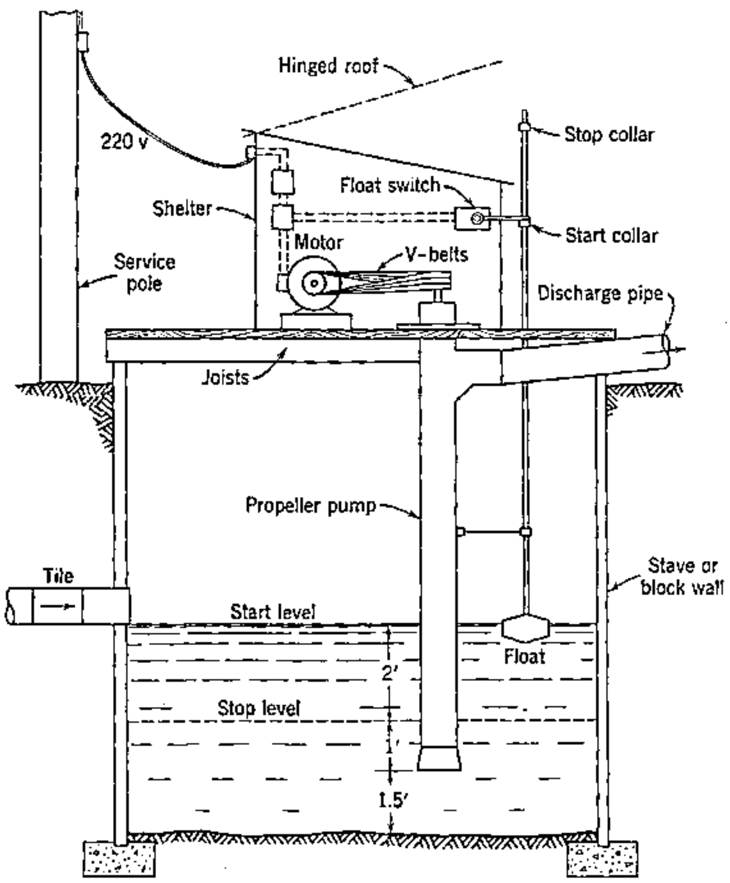


Fig. 19.8. Typical small drainage pumping plant. (Redrawn from Larson.[6])

during wet seasons and the other for seepage or flow from tile drains. However, for most small farm pumping plants only one pump is justified.

The pumping plant should be installed to provide minimum lift and located so that the pump house will not be flooded. A typical farm pumping plant for tile or open ditch drainage of small areas is shown in Fig. 19.8. A circular sump is recommended for such an installation since less reinforcing is required and it is easier to construct. The electric motor is usually operated automatically by means of start and stop collars on the float rod. Where open ditches provide storage, the sump can be reduced in size. The discharge pipe should outlet below the minimum water level in the outlet ditch, where practicable, to reduce the pumping head to a minimum.

19.12. Design Capacity. A pumping plant should be large enough to provide adequate drainage. The capacity at maximum lift is the most satisfactory basis for design. Drainage coefficients for pumping plant design vary according to soil, topographic, rainfall, and cropping conditions. Recommended coefficients for the upper Mississippi Valley range from 0.3 to 0.6 inches. Requirements in Southern Texas and Louisiana are as high as 3 inches, including pumping discharge and reservoir storage.[13] In Florida a drainage coefficient of 1 inch is suitable for field crops on organic soils. Truck crops may require and economically justify a drainage coefficient of as much as 3 inches.[13]

The design of small farm pumping plants may be simplified by using the normal drainage coefficients recommended for open ditches in Chapter 15. Where electricity is available, electric motors with automatic controls are recommended for small installations. For drainage areas larger than 100 acres storage capacity should be obtained by enlarging the open ditches or by excavating a storage basin. Constructed sumps as shown in Fig. 19.8 are generally too expensive where the diameter is greater than 16 feet. Where storage is available in open ditches or other reservoirs, sumps about 6 feet in diameter or 6 feet square are satisfactory.

PUMPING FOR IRRIGATION

Pumping requirements for irrigation vary considerably, depending upon such factors as the source of water, method of irrigation, and the size of area irrigated. Water may be obtained from wells, rivers, canals, pits, ponds, and lakes. Pumping from surface supplies is very similar to pump drainage except where pressure irrigation is practiced. Much higher heads are normally required where water is pumped from wells or pumped to pressure irrigation systems.

19.13. Hydraulics of Wells. The three general types of wells are: dug, driven, and drilled. The dug well is constructed by physically removing the soil in its original condition, a driven well by driving a pipe and well point in the soil or by jetting, and the drilled well by using percussion or rotary drills and removing the soil from the well as sediment. Dug and driven wells are normally limited to depths of about 50 feet; drilled

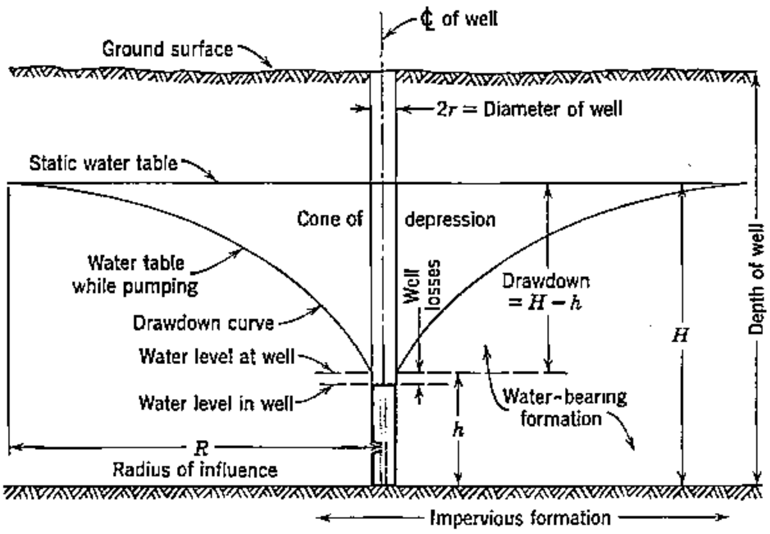


Fig. 19.9. Cross section of a typical gravity well in homogeneous soil. (Redrawn from Houk.[4])

wells may be several thousand feet deep. For irrigation purposes, drilled wells are the most common. Since the cost of deep-drilled wells is generally high, small-diameter test wells for exploration purposes eliminate guesswork and waste of funds.

Wells may be classed as gravity, artesian, or a combination of artesian and gravity, depending on the type of acquifer supplying the water. The following explanations apply to gravity-type wells.

A cross section of a well installed in homogeneous soil overlying an impervious formation is shown in Fig. 19.9. Under static conditions the water level will rise to the water table.

When pumping begins, the water level in the well is lowered, thus removing free water from the surrounding soil. The water table around a well assumes the general form of an inverted cone although it is not a true cone. The distance from the well to where the static water table is not lowered by drawdown is known as the radius of influence. The water level at the edge of the well will be slightly higher than in the well because of friction losses through the perforated casing. The effect of the number and size of perforations on inflow has been investigated by Muskat.[7] For a given rate of pumping the water table surrounding a well in time reaches a stable condition. The shape of the drawdown curve depends on the soil permeability and the aeration porosity (see Chapter 5). In ground water hydrology the specific yield is the quantity of water that a formation will supply when drained by gravity.

The rate of flow into a gravity well, illustrated in Fig. 19.9, can be computed as follows:

$$Q = \frac{\pi K(H^2 - h^2)}{\log_e R/r} \tag{19.2}$$

where Q = rate of flow in cubic feet per day.
K = permeability in feet per day.
H = height of the static water level above the bottom of the water-bearing formation.
h = height of the water level at the well, measured from the bottom of the water-bearing formation in feet.
R = radius of influence in feet.
r = radius of the well in feet.

The permeability of the acquifer is directly proportional to the discharge and may be estimated by one of the methods described in Chapter 5. The radius of influence given in Table 19.2 may be estimated from the texture and other characteristics of the acquifer. Since the discharge varies inversely as the logarithm of the radius of influence, an error in estimating this radius results in a much smaller error in the discharge. Equation 19.2 also indicates that the discharge is proportional to the logarithm of the well diameter. Assuming an R of 1000 feet and other conditions constant, a well 2 feet in diameter will yield about 10 per cent more than a well 1 foot in diameter.

Table 19.2 RADIUS OF INFLUENCE OF WELLS*

Soil Formation and Texture	*Radius of Influence,* ft
Fine sand formations with some clay and silt	100–300
Fine to medium sand formations fairly clean and free from clay and silt	300–600
Coarse sand and fine gravel formations free from clay and silt	600–1000
Coarse sand and gravel, no clay or silt	1000–2000

* From Bennison.[1]

For a given well there is a definite relationship between drawdown and discharge. For thick water-bearing acquifers or artesian formations, the discharge-drawdown relationship is

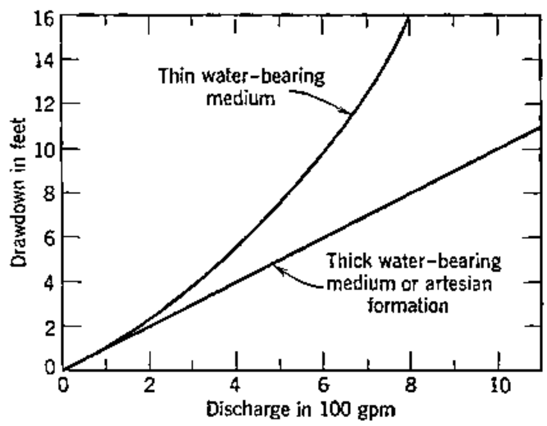


Fig. 19.10. Relationship between drawdown and discharge of a well. (Redrawn from Rohwer.[9])

nearly a straight line. The drawdown curve for artesian conditions is a piezometric surface; under nonartesian conditions it coincides with the phreatic surface. As shown in Fig. 19.10, the discharge-drawdown relationship can be obtained by pumping the well at various rates and plotting the drawdown against the discharge. Test pumping a well should be continued for a considerable length of time. Short pumping tests are often misleading. It has been found that even 24-hour tests are not long enough, and 30-day tests are more likely to indicate the true capacity of the well.[1]

After a well is drilled, it is generally necessary to wash out the fine particles in the soil surrounding the casing. This process is commonly referred to as *developing the well.* It greatly increases the permeability of the surrounding medium, thus increasing the flow. By placing a gravel envelope around the well, the effective diameter is increased to the diameter of the gravel envelope without a larger casing. Developing a well has a similar effect. Frequently, the discharge may be increased two to three times by proper development.[5] Various methods have been devised for developing wells, including washing and jetting with water or compressed air, plunging a sand bucket up and down in the casing, and pumping at various rates so as to alternately raise and lower the water level over a wide range.

In the above discussion only the hydraulics of a single well were considered. Where more water is needed several wells may be required. These are often connected in batteries so that all wells may be pumped by the same pump.

19.14. Pumping from Wells. Water pumped from wells may be discharged into open ditches or into pipe lines for distribution to irrigated areas. Wells located in the vicinity of the area to be irrigated have many advantages as compared to canals supplying water from distant sources.

Because of excessive pressure (water hammer), which may develop from starting or stopping the pump when pumping into pipe lines, surge tanks should be installed in the discharge line, as illustrated in Fig. 19.11. However, they are usually not required for portable sprinkler irrigation systems. Surge tanks that are open to atmospheric pressure are suitable for pumping into pipe lines made of thin metal, concrete, or vitrified clay pipe.[15] The surge tank must be high enough to allow for any increase in elevation of the pipe line, for friction head, and for a reasonable freeboard. When the total head exceeds 20 feet, an enclosed metal tank having an air chamber is usually more practical than a surge tank.[15]

In computing the total head for the pump, the velocity, pressure, and static head must be considered as well as losses in the pipe, valves, and couplings. The maximum head at which pumping is practical varies with the locality, value of the crop, and other economic factors. In some areas irrigation pumping is carried out for heads up to 300 feet or more.

The pump should be fitted to the well. By plotting the head-capacity curve for the pump and the drawdown-discharge curve for the well, as in Fig. 19.10, the intersection of the two

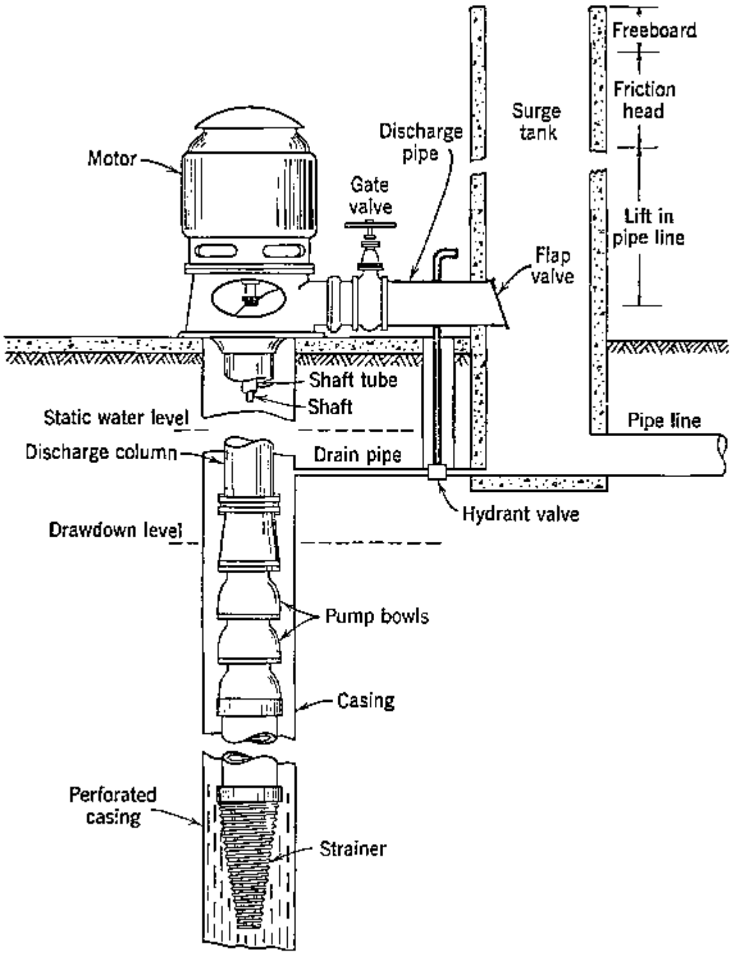


Fig. 19.11. Two-stage deep-well turbine pump installation with a surge tank for low heads. (Redrawn from Rohwer[9] and Wood.[15])

curves gives the operating capacity of the plant. Such a set of curves for a typical field installation is shown in Fig. 19.12. The most desirable conditions for operating the pump occur when the capacity is 700 gpm and the head 44 feet. At this discharge the pump also operates at its maximum efficiency (65 per cent) and the power unit at nearly its maximum horsepower. If the discharge is changed, there is no danger of overloading the power

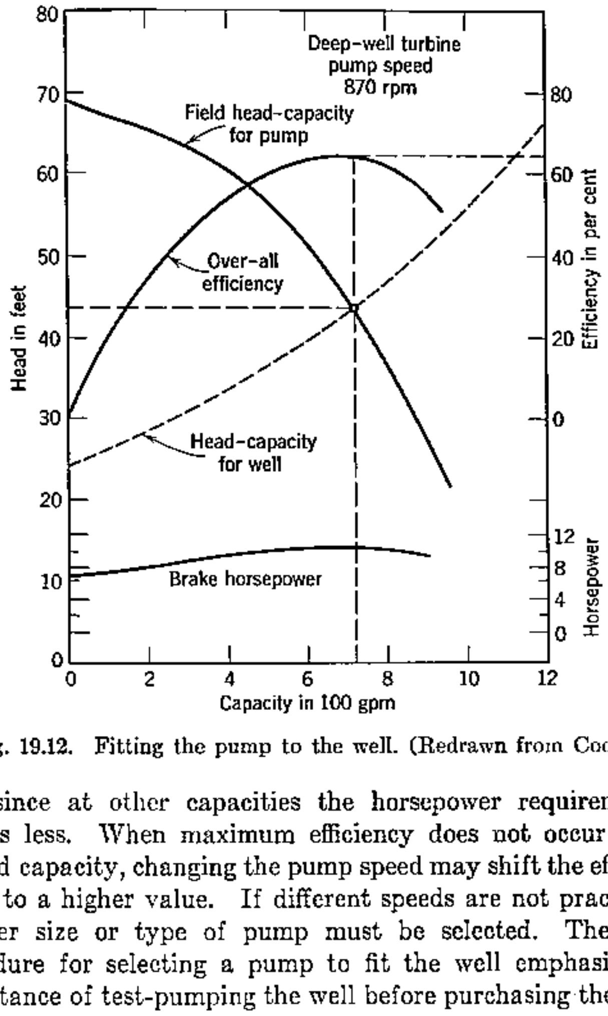


Fig. 19.12. Fitting the pump to the well. (Redrawn from Code.[3])

unit since at other capacities the horsepower requirement is always less. When maximum efficiency does not occur at the desired capacity, changing the pump speed may shift the efficiency curve to a higher value. If different speeds are not practicable, another size or type of pump must be selected. The above procedure for selecting a pump to fit the well emphasizes the importance of test-pumping the well before purchasing the pump.

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PROBLEMS

19.1. A centrifugal pump discharging 400 gpm against 80 feet of head and operating with an efficiency of 60 per cent requires 11.5 hp at 1000 rpm. What is the theoretical discharge if the speed is increased to 1500 rpm, assuming the efficiency remains constant? What is the theoretical head and horsepower at 1500 rpm?

19.2. From the performance curves for a centrifugal pump shown in Fig. 19.4, what is the pump efficiency at 2000 rpm for a head of 200 feet? What is the discharge? What is the efficiency and discharge if the head is reduced to 150 feet?

19.3. What are the power requirements for pumping 1000 gpm against a head of 150 feet, assuming a pump efficiency of 65 per cent? An electric motor of what size is required?

19.4. Compute the flow rate into a gravity well 24 inches in diameter if the depth of the water-bearing stratum is 80 feet, the drawdown is 30 feet, soil permeability is 3 inches per hour, and the radius of influence is 600 feet.

CHAPTER 20

Sprinkler Irrigation

Irrigation is defined as the artificial application of water for the purpose of supplying sufficient moisture for plant growth. Agriculture in the seventeen western states has always been largely dependent upon irrigation for efficient production of most crops. There has been increasing emphasis on irrigation in eastern states to supplement rainfall during periods of moisture deficiency. Sprinkler irrigation is included in this book as a guide to those persons who may encounter the problem in connection with other conservation activities but who do not become specialists in irrigation.

For successful irrigation there must be an adequate supply of water of suitable quality, application of proper amounts of water at proper times, suitable methods for applying water, facilities for removing excess water without erosion, and a favorable ratio of returns to costs of irrigation. The reader may refer to more complete books on irrigation, such as Israelsen,[9] Roe,[14] and Thorne and Peterson.[18]

WATER SUPPLY

The ultimate source of all irrigation water is precipitation. However, it is precipitation that does not occur at a time or place to be directly available to crops. Therefore irrigation water must be stored in surface reservoirs or as ground water, and later conveyed to the area to be irrigated.

20.1. Quality of Water. The quality of irrigation water depends on the amount of suspended sediment and chemical constituents in the water. Where fine sediment is applied to sandy soil, the gradation and fertility of the soil may be improved. However, sediment from eroded areas may reduce fertility and soil permeability.

Chemical suitability of water is influenced by soil, crops, irrigation practices, and climate, as well as by total quantity, relative concentrations, and nature of dissolved salts. To define the chemical quality of irrigation water the total concentration of salts, the amount of sodium, and the amount of boron should be determined.[25] The electrical conductivity of water is a good indication of its quality. Standards have been developed by the U. S. Regional Salinity Laboratory for different classes of irrigation water.[10] Not all chemicals are injurious to plants. It is only when the concentration is too high that injury occurs. Salts may accumulate over a period of time in the soil and be injurious to plants even though the water has a low concentration of salts. Such accumulation of salts may be removed by proper drainage practices.

20.2. Surface Water Supplies. Water for irrigation is stored on the surface either in natural lakes or constructed reservoirs. Such reservoirs range in size from many million acre-feet for large multiple-purpose reservoirs to ponds with a few acre-feet of storage. In general, the cost per unit of storage decreases as the capacity of the reservoir is increased.

Small lakes and farm ponds are normally suitable only for small irrigation projects. Such reservoirs may be used to advantage for irrigation in the East. Careful consideration should be given to the capacity of the pond and the number of acres to be irrigated, with allowance for losses, including evaporation and deep seepage. Natural streams may provide a source of irrigation water for at least a portion of the irrigation season. In general, stream flow seldom coincides with irrigation demands. When water is needed in late summer months, the flow in streams is often extremely low.

When the use of a stream for water supply is contemplated, the dependability of the flow should be considered. Stream flow data may be obtained from such reports as the U. S. Geological Survey Water Supply Papers. Special attention should be given to the flow of the stream in very dry years, as it is during these periods that irrigation is most needed. In many instances maximum use is made of stream flow, but, when the flow is inadequate, reservoir or ground water supplies are used. The prospective user of surface waters should consider the legal problems incident to water rights.

20.3. Ground Water Supplies. Ground water may be obtained from wells, springs, and dugout ponds. Wells, by far the most common, are either shallow or deep, depending on the ground water depth. Where ground water reaches the surface because the underlying strata are impervious, springs or seeps may develop. In areas where the ground water table is near the surface, dugout ponds or open pits are useful for the storage of ground water. Excellent references on ground water supplies and their development are numerous.[13,17,19]

USE OF WATER

With increasing demands for irrigation water and with limited supplies available, more effective use of water is becoming essential. Application losses include evaporation, deep percolation, and surface runoff.

Table 20.1 EVAPO-TRANSPIRATION COEFFICIENTS, K, FOR IRRIGATED CROPS*

Crop	Length of Growing Season or Period	Evapo-Transpiration Coefficient K	
		Western States	*Southeastern States*
Alfalfa	Between frosts	0.80–0.85†	0.70–0.80‡
Corn	4 months	0.75–0.85	0.60–0.70
Cotton	7 months	0.60–0.65	0.50–0.55
Grains, small	3 months	0.75–0.85	0.60–0.65
Grain sorghums	4 to 5 months	0.70	—
Orchard, citrus	7 months	0.50–0.65	—
Pasture, grass	Between frosts	0.75	0.65–0.75
Potatoes	3½ months	0.65–0.75	0.60–0.65
Rice	3 to 5 months	1.00–1.20	0.85–1.00
Sugar beets	6 months	0.65–0.75	—
Truck crops, small	3 months	0.60	0.50–0.55

* From Blaney and Criddle.[3]

† Lower values for coastal areas; the higher values for areas with an arid climate.

‡ Tentative values recommended by H. F. Blaney. Lower values for coastal areas and entire state of Florida; higher values for remainder of region.

20.4. Estimating Evapo-Transpiration. Evapo-transpiration, sometimes referred to as consumptive use, can be estimated by the Blaney-Criddle method outlined in Chapter 3 and illustrated in Example 20.1. Evapo-transpiration coefficients K for irrigated crops are given in Table 20.1.

20.5. Water Losses. Under conditions of sprinkler irrigation water losses due to deep percolation and surface runoff may be held to a minimum. Water need not be applied faster than it will move into the soil, or in quantities in excess of what may be held in the root zone. Chief water losses are thus evaporation from the spray, plants, and soil surface, and excess application on small areas incident to the overlapping of sprinkler patterns. These losses may be accounted for by a water application efficiency of 70 per cent. This efficiency should be increased to 80 per cent for humid areas and coastal fog belts, and decreased to 60 per cent for hot dry climates. The efficiency is decreased 5 per cent for each 5 mph of wind above a base of 5 mph, and decreased 5 per cent for each 5 per cent slope above a base of 12 per cent.[23]

The water application efficiency is defined[9] by

$$E_a = \frac{W_s}{W_f} \times 100 \tag{20.1}$$

where E_a = water application efficiency (farm application efficiency) in per cent.

W_s = irrigation water stored in the root zone (available for plants).

W_f = water pumped into the system.

20.6. Irrigation Requirements. The irrigation requirement is the quantity of water, exclusive of precipitation, to be supplied by artificial means. Irrigation requirements are dependent not only on evapo-transpiration but also on water application efficiency, precipitation, and water supplied by percolation or capillary movement from ground water. An estimation of irrigation requirements is illustrated in Example 20.1.

Example 20.1. Determine the evapo-transpiration and irrigation requirements for wheat (small grain) at Plainview, Texas, if the water application efficiency is 65 per cent.

Solution. Data taken from Blaney.[2]

Month	*t**	*p*†	*f*‡	*Rainfall**
April	59.2	8.80	5.21	1.92
May	67.5	9.72	6.56	2.58
June	75.6	9.70	7.33	3.04
Total			19.10	7.54

* Mean monthly temperature in degrees F and rainfall can be obtained from Weather Bureau records for the locality.

† Monthly per cent of daytime hours of the year is available from Israelsen[9] (p. 310), Blaney and Criddle[3] (pp. 16 and 48), or Roe[14] (p. 177).

‡ Monthly evapo-transpiration factor ($tp/100$).

From equation 3.3 and Table 20.1, $K = 0.80$, $U = 0.80 \times 19.10 = 15.28$ inches.

$$\text{Irrigation water required} = \frac{(15.28 - 7.54)}{0.65} = 11.9 \text{ inches.}$$

Monthly evapo-transpiration factors and average monthly precipitation for all the western states have been tabulated by Blaney and Criddle.[3]

20.7. Crop Needs. Water requirements and time of maximum demand vary with different crops. Although growing crops are continuously using water, the rate of transpiration depends on the kind of crop, degree of maturity, and atmospheric conditions, such as humidity and temperature. Where sufficient water is available, the moisture content should be maintained within the limits for optimum growth. The rate of growth at different soil moistures varies with different soils and crops. Some crops are able to withstand drought or high moisture content much better than others. During the later stages of maturity, the water needs are generally less than during the maximum growing period. When crops are ripening, it is desirable to discontinue irrigation.

From soil moisture measurements or from the appearance of the soil or the crop, the irrigator is able to determine when water should be applied. Indicator plants which are more sensitive to moisture deficiency than the field crop may be planted in the same field to indicate the time of irrigation. Measuring soil moisture (see Chapter 5) is generally the most satisfactory procedure since it is then possible to predict, a few days in advance, the time of irrigation.

The effect of the available water supply on the yield of alfalfa is shown in Fig. 20.1. The curves show typical relationships between the available water in the soil and the crop yield. On the top curve the yield increases for 4 feet of water or less, but where more than 4 feet is applied the yield begins to decrease. Similar curves could be developed for other crops and other areas.

20.8. Seasonal Use of Water. To make maximum use of available water, the irrigator should have a knowledge of the water requirements of crops at all times during the growing season. It may be possible to select the crop to fit the water supply. The seasonal use of water for a few field crops is shown in Fig. 20.2. Although the data were obtained in

Nebraska, the curves are fairly typical of other irrigated regions. Sugar beets, potatoes, and corn generally require very little water early in the season, but during the late summer months a large quantity is needed. Oats, which are fairly typical of other small grains, require a considerable quantity of water in late May and June. Alfalfa takes a relatively large quantity of water from the early part of the season until the late summer months because it is a continuously growing crop. The water requirement for corn is similar to that for sugar beets.

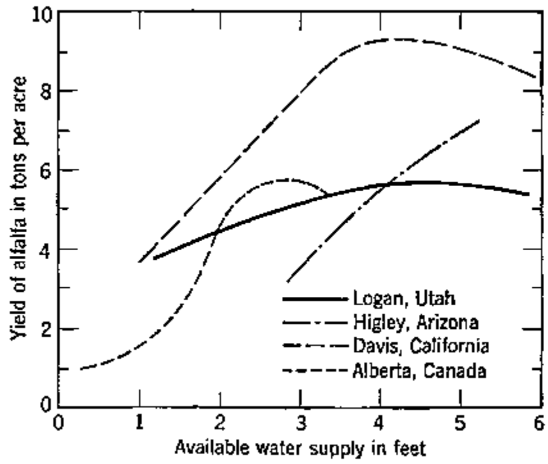

Fig. 20.1. Effect of total water supply on alfalfa yield. (Redrawn from Thorne and Peterson.[18])

As shown in Fig. 20.2, the greatest moisture change occurs in the upper foot of soil. Except for alfalfa, 80 to 90 per cent of the total water consumed is taken from the upper 3 feet. Since alfalfa is a deep-rooted plant, it is able to use water to depths of 6 feet. The depth of the root zone largely determines the quantity of water to be applied at each irrigation. For example, wetting the top 3 feet of soil should be adequate for sugar beets.

20.9. Moisture Deficiency Recurrence. The duration and length of dry periods during the growing season in humid and semihumid areas largely determine the economic feasibility of supplemental irrigation. Moisture deficiency during the months

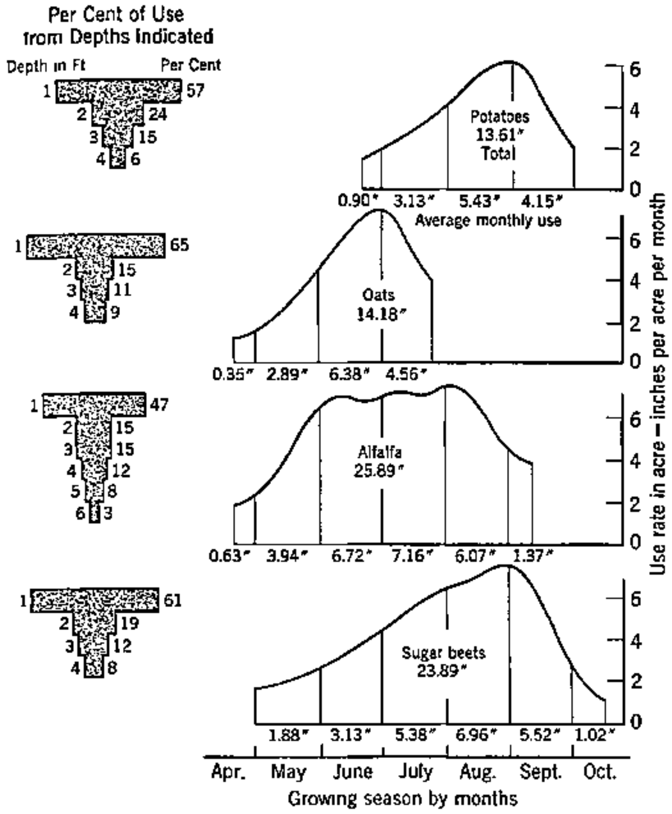

Fig. 20.2. Seasonal use of water by field crops. (Redrawn from Bowen.[4])

of June, July, and August is more serious than in earlier or later months. Drought recurrence frequency is discussed in Chapter 2.

SPRINKLER IRRIGATION SYSTEMS

Any method of irrigation that is suitable in arid regions is likewise adaptable to more humid regions. Sprinkler irrigation has been particularly popular in humid regions because it is suitable to a wide range of topographic conditions, soils, and crops and because surface ditches and land smoothing are not necessary. The portable rotating-head sprinkler system is common not only in humid regions but in the West as well. Handbooks and design pamphlets on sprinkler irrigation have been prepared by many commercial companies, land-grant colleges, and federal agencies.[16,20,21,22,23]

20.10. Components of Systems. In systems using portable pipe, the water is applied by means of a pump, a main line, and portable lateral lines equipped with sprinkler heads at suitable intervals.

A pump usually lifts the water from the source, pushes it through the distribution system, and through the sprinklers. In all cases it is important that the pump should have adequate capacity for present and future needs. This capacity generally falls between 100 and 800 gallons per minute for most small farm irrigation installations.

The second component of the system, the main line, may be either movable or permanent. Movable mains generally have a lower first cost and can be more easily adapted to a variety of conditions; permanent mains offer a saving in labor and reduced obstruction to field operations. Water is taken from the main either through a valve placed at each point of junction with a lateral or in some cases through either an ell or a T-section that has been supplied in place of one of the couplings on the main.

The laterals are usually 20- or 30-foot lengths of aluminum or other lightweight metal pipe connected with couplers. In some cases the couplers are permanently attached to the pipe (see Fig. 20.3*a*). Some systems are constructed so that they can be pulled by a tractor or are mounted on wheels so that they can be rolled across the field.

For rotating sprinklers, the sprinkler heads most often used have two nozzles, one to apply water at a considerable distance from the sprinkler and the other to cover the area near the sprinkler center (Fig. 20.3*b*). Of the devices to rotate the sprinkler, the most usual, also shown in Fig. 20.3*b*, taps the sprinkler head with a small hammer activated by the force of the water striking against a small vane connected to it. Sprinklers designed to cover a considerable area have a slow rate of rotation, about one revolution per minute.

A number of sprinkler heads are available for special purposes. Some provide a low-angle jet for use in orchards. Some work at especially low heads, of say 5 psi, and others operate only in a part circle. Giant sprinkler units discharge 300 to 500 gpm at pressures of 80 to 100 psi and throw the water several hundred feet (Fig. 20.3*b*). In general, these work at higher pressures than the smaller units and result in greater pumping costs. On

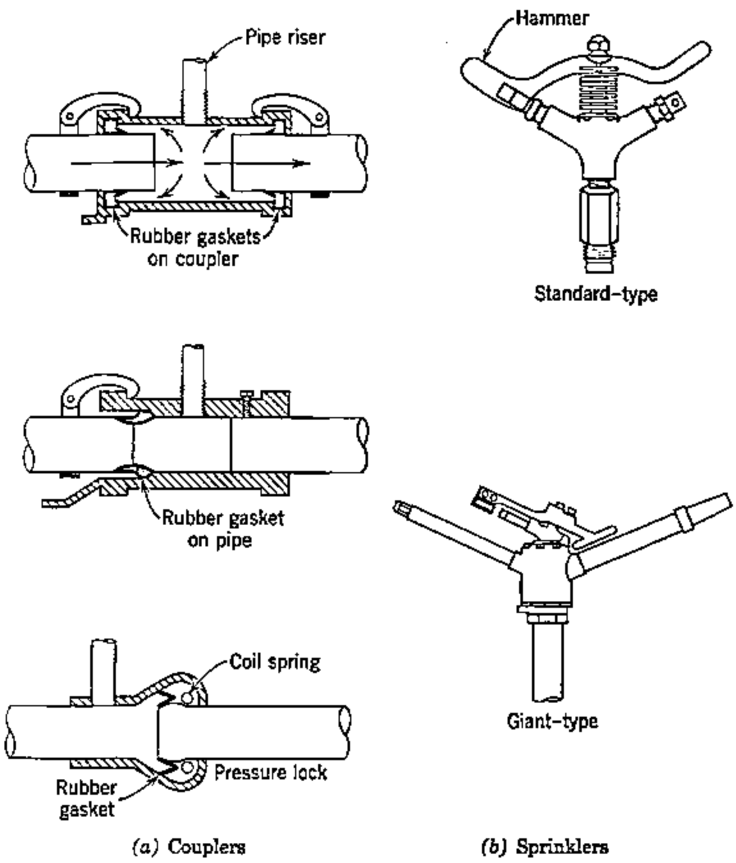

Fig. 20.3. Quick-connecting couplers and sprinkler heads.

the other hand, because these large units cover a much greater area, a smaller number of moves is required.

Some installations use perforated pipe instead of rotary sprinklers. This type of distribution equipment consits of thin-walled, slip-joint pipe with lines of small holes near the top.

Water is sprayed over a strip 50 feet wide when the system is operated at about 15 psi.

20.11. Distribution Pattern of Sprinklers. A typical distribution pattern showing the effect of wind for a single

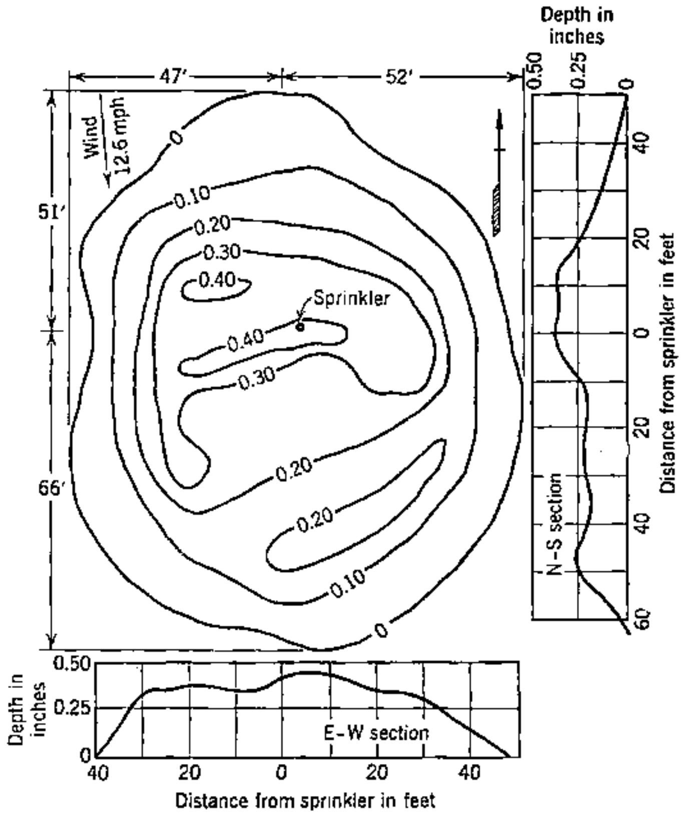

Fig. 20.4. Distribution pattern for a single sprinkler, showing the effect of wind. (Pressure 30 psi, and discharge 19.6 gpm.) (Redrawn from Christiansen.[5])

sprinkler is illustrated in Fig. 20.4. Since one sprinkler does not apply water uniformly over the area, the overlapping of sprinkler patterns is relied upon to provide more uniform coverage. The distribution pattern shown in Fig. 20.5 illustrates how the overlapping patterns combine to give a relatively uniform distribution between sprinklers. Although Fig. 20.5 shows relatively uniform distribution over the area, wind will skew the pattern so as to give less uniform distribution.

20.12. Other Uses for Sprinkler Irrigation Equipment. Although alternate uses of sprinkler equipment are not usually considered in determining the economic feasibility of the project, they may be particularly desirable in certain areas. Principal alternate uses include fertilizer application, protection against frost,[14] and stand-by drainage pumping plants. If liquid

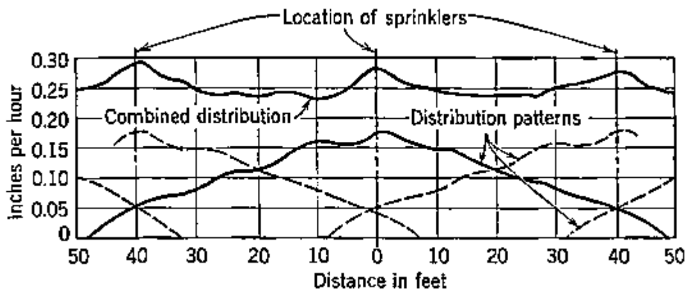

Fig. 20.5. Typical distribution patterns, overlapping to give relatively uniform combined distribution. (Redrawn from Gray.[8])

fertilizers are applied with the water, they may be introduced at the most desirable time. For frost protection in the early spring and late fall heavy applications of water are normally required. Water must be applied during the time that frost is likely to occur and must be continued until all the ice is removed from the plant.

20.13. Cost Comparison of Sprinkler Systems. The relative annual cost for sprinkler irrigation systems operating at various pressures is shown in Table 20.2. The relative cost for 5 irrigations per season applies to humid regions; 17 irrigations is typical for more arid conditions. At each irrigation 1½ inches of water were applied. In this analysis the most economical system for both 5 and 17 irrigations is system *C*, and the most costly is *E*. Where power costs are lower than 2 cents per kilowatt-hour, the high-pressure system *E* will compare more favorably than indicated. These data represent an analysis for given conditions, and each layout should be evaluated separately, on the basis of current prices and local conditions.

Table 20.2 RELATIVE ANNUAL COSTS PER ACRE FOR SPRINKLER IRRIGATION*

System	Relative Pressure (Type)	Spacing $S_1 \times S_2$, ft	Application Rate, iph	Pumping Rate, gpm	Relative Cost per Acre per Year: Equipment (20-yr)	Power, 5 Irr.†	Labor, 5 Irr.	Total Cost: 5 Irrigations	Total Cost: 17 Irrigations
A	Low	20 × 40	0.25	263	60	16	51	127	278
B	Moderate	40 × 60	0.30	246	51	27	42	120	275
C	Moderate high	60 × 80	0.50	275	43	26	31	100	232
D	Moderate high	80 × 100	0.75	504	52	34	33	119	272
E	High	200 × 200	1.00	415	70	70	34	174	413

* Based on data from McCulloch.[12] Assuming a 40-acre square tract, nearly level, with a well at the center.

† Power cost at 2 cents per kilowatt-hour.

DESIGN

Not only should the sprinkler system be properly designed hydraulically and economical in cost, but also the design should be adapted to the availability of labor for moving the sprinklers and the pipe. The frequency of moving laterals, system layout, and capacity of the system should be carefully considered for each farm.

20.14. Basic Design Data. The three basic facts to be established before the design of a sprinkler irrigation system is initiated are the limiting rate of application, the irrigation period, and the depth of application. The rate of application is limited by the infiltration capacity of the soil. Application at rates in excess of the soil infiltration capacity result in runoff with accompanying poor distribution of water, loss of water, and erosion. An example of maximum water application rates for various soil conditions is given in Table 20.3. These values may be used as a guide where reliable local information is not available.

Table 20.3 SUGGESTED MAXIMUM WATER APPLICATION RATES FOR SPRINKLERS FOR AVERAGE SOIL, SLOPE, AND CULTURAL CONDITIONS*

Soil Texture and Profile Conditions	Maximum Water Application Rate for Slope and Cultural Conditions, iph			
	0% Slope		10% Slope	
	w/cover	bare	w/cover	bare
Light sandy loams uniform in texture to 6′	1.7	1.0	1.0	0.6
Light sandy loams over more compact subsoils	1.2	0.7	0.7	0.4
Silt loams uniform in texture to 6′	1.0	0.5	0.6	0.3
Silt loams over more compact subsoil	0.6	0.3	0.4	0.1
Heavy-textured clays or clay loams	0.2	0.1	0.1	0.08

* Data from Soil Conservation Service.[21]

The depth of application and the irrigation period are closely related. Irrigation period is the time required to cover an area with one application of water. The depth of application will depend on the available moisture-holding capacity of the soil.

Under humid conditions rains may bring the entire field up to a given moisture level. As the plants use this moisture, the moisture level for the entire field decreases. Irrigation must be started soon enough to enable the field to be covered before plants in the last portion to be irrigated suffer from moisture deficiency.

One recommended system[6] is to commence irrigation when the moisture level of the field reaches 55 per cent of the available moisture capacity. The net depth of application under this plan is equal to 45 per cent of the available moisture capacity. The irrigation period is set so that the entire irrigated area will be covered before the finishing end of the field reaches a moisture level below 10 per cent of the available moisture. Typical moisture-holding capacities are 1.5 inches per foot for sandy loam, 1.8 inches per foot for silt loam, and 2.0 inches per foot for clay loam.[23] Table 20.4 indicates the general magnitude of the rate of moisture use by selected crops.

Table 20.4 Root Depth and Peak Rate of Moisture Use for Certain Crops*

		Peak Rate of Moisture Use, in./day		
		Climate		
Crop	*Root Depth, ft*	*Cool*	*Moderate*	*Hot*
Alfalfa	3.0	0.15	0.20	0.30
Beans	3.0	0.12	0.16	0.25
Corn	3.0	0.15	0.20	0.25
Pasture	1.5	0.12	0.16	0.25
Potatoes	1.5	0.10	0.12	0.14
Strawberries	1.0	0.12	0.16	0.25

* Modified from Soil Conservation Service.[23,24]

Example 20.2. A sprinkler irrigation system is to be designed to irrigate 40 acres of corn on a deep silt loam soil. The field is flat. Determine the limiting rate of application, the irrigation period, the net depth of water per application, the depth of water pumped per application, and the required system capacity in acres per day.

Solution. From Table 20.3 the limiting application rate is 0.5 iph when the soil is bare. The available moisture-holding capacity of the soil is 1.8 inches per foot and the depth of the root zone from Table 20.4 is 3 feet. The total available moisture capacity is thus 5.4 inches. Of this, 45 per cent is 2.4 inches, which is the net depth of application. Assuming a water application efficiency of 70 per cent, the depth of water pumped per application is 3.43 inches. From Table 20.4 the peak rate of use by the crop is 0.15 inches per day. The irrigation period is thus 2.4/0.15 = 16 days. To cover this field in 16 days the system must be able to pump and discharge 3.43 inches on 2.5 acres per day. These figures are then used as guides in the selection of the irrigation equipment.

20.15. Arrangements of Mains and Laterals. The number of possible arrangements for the mains, laterals, and sprinklers is practically unlimited. The arrangement selected should allow a minimal investment in irrigation pipe, have a low labor requirement, and provide for an application of water over the total area in the required period of time. The most suitable layout can be determined only after a careful study of the conditions to be encountered. The choice will depend to a large extent upon the types and capacities of the sprinklers and the pressure to be used. In many systems, the laterals are moved 60 feet at each setting and the sprinklers are spaced every 40 feet on each lateral.

Typical layouts for sprinkler irrigation systems are shown in Fig. 20.6. The layout in Fig. 20.6 is suitable where the water supply can be obtained from a stream or canal alongside the field to be irrigated. This arrangement either eliminates the main line or requires a relatively short main, depending on the number of moves for the pump. Less pipe is required for this method than for any of the others. The layout illustrated in Fig. 20.6*b* and *c* are suitable where the water supply is from a well or pit. In Fig. 20.6*b* the two laterals are started at opposite ends of the field and are moved in opposite directions. Since the farther half of the main supplies a maximum of one lateral at a time, the diameter of this section can be reduced. This arrangement is well suited to day and night operation when the required amount of water can be added in about 6 or 8 hours. The system shown in Fig. 20.6c is designed for higher rates of application. While line *A* is in operation, the operator moves line *B*. When the required amount of water has been applied, line *B* is turned on and then line *A* is moved. With this pro-

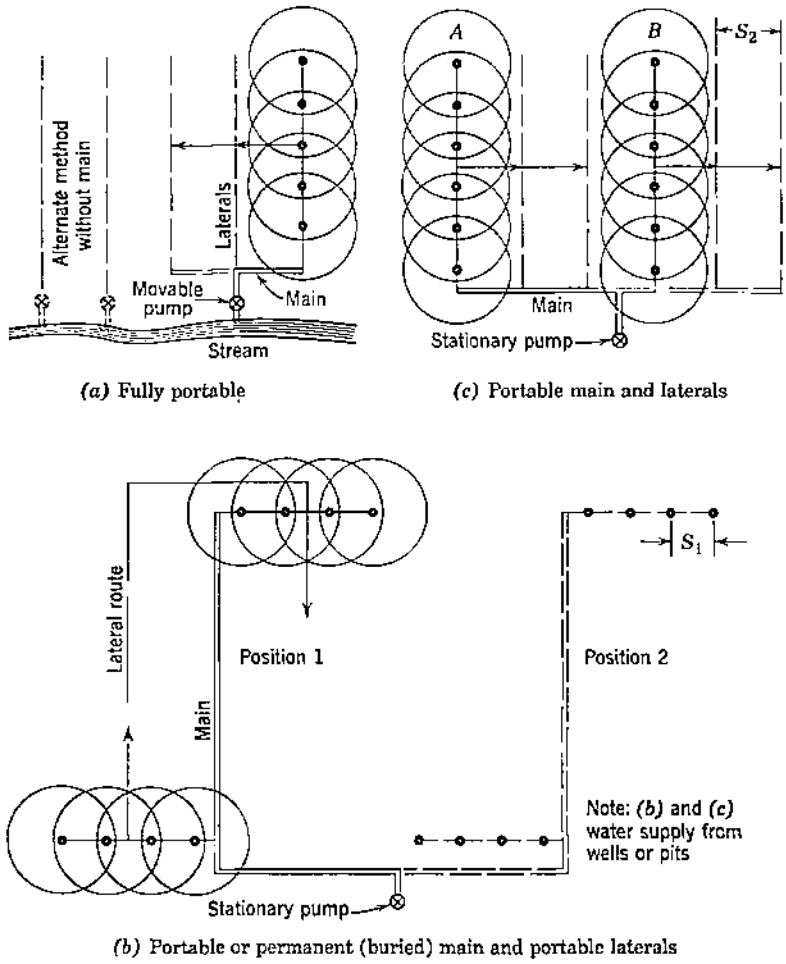

Fig. 20.6. Typical field layout of main and laterals for rotating-head sprinkler systems. (Redrawn in part from McCulloch.[11])

cedure the capacity of the pump needs to be adequate to supply only one lateral.

20.16. Capacity of the Sprinkler System. The capacity of a sprinkler system depends on the area to be irrigated, depth of water application at each irrigation, frequency of application, and actual operating time for each irrigation. It is convenient to express the capacity by the following formula:

$$Q = da/t \tag{20.2}$$

where Q = capacity of the system.
d = depth of application.
a = area covered.
t = total time of operation.

20.17. Sprinkler Capacity. When the rate of application and the spacing of the sprinklers has been determined, the required sprinkler capacity can be computed by the formula

$$q = S_1 S_2 r \tag{20.3}$$

where q = discharge of each sprinkler.
S_1 = sprinkler spacing along the line.
S_2 = sprinkler spacing between lines.
r = rate of application.

For example, a spacing of 40 by 60 feet and an application rate of 0.40 iph require a sprinkler with a capacity of 10 gpm.

The actual selection of the sprinkler is based largely upon design information furnished by manufacturers of the equipment. The choice depends primarily upon the diameter of coverage required, pressure available, and capacity of the sprinkler. The theoretical discharge of a nozzle may be computed from the orifice flow formula (see equation 11.9):

$$Q = aC\sqrt{2gh} \tag{20.4a}$$

For simplification of calculations, this may be reduced to

$$q = 29.85 C d_n^2 P^{1/2} \tag{20.4b}$$

where q = nozzle discharge in gpm.
C = coefficient of discharge.
d_n = diameter of the nozzle orifice in inches.
P = pressure at the nozzle in psi.

The coefficient of discharge for well-designed, small nozzles varies from about 0.95 to 0.98. Some nozzles have coefficients as low as 0.80. Normally, the larger the nozzle, the lower is the coefficient. Where the sprinkler has two nozzles, the total discharge is the combined capacity of both.

20.18. Size of Laterals and Mains. Laterals and mains should provide the required rate of flow with a reasonable head loss. For laterals the sections at the distant end of the line have less water to carry and may therefore be smaller. However, many authorities advise against "tapering" of pipe diameters in laterals, as it then becomes necessary to keep the various pipe sizes in the same relative position. The system may also be less adaptable to other fields and situations.

The total pressure variation in the laterals, when practicable, should not be more than 20 per cent of the higher pressure.[1] If the lateral runs up or downhill, allowance for this difference in elevation should be made in determining the variation in head. If the water runs uphill, less pressure will be available at the nozzle; if it runs downhill, there will be a tendency to balance the loss of head due to friction.

Scobey's equation for friction or head loss in pipes may be expressed as [5,15]

$$H_f = \frac{K_s L Q^{1.9}}{D^{4.9}} (1.45 \times 10^{-8}) \tag{20.5}$$

where H_f = total friction loss in line in feet.
K_s = Scobey's coefficient of retardation.
L = length of pipe in feet.
Q = total discharge in gpm.
D = inside diameter of pipe in feet.

Although this formula was developed for uniform flow, it may be adapted to lateral pipe with sprinkler outlets by multiplying the friction loss H_f by a factor F to obtain the actual loss. Computed values of F are given in Table 20.5. For example, if the head loss for ordinary pipe is 10 psi, the corresponding head loss for 8 sprinklers on the same length of line is 4.1 psi. When the computed diameter is a fractional size, the next largest nominal diameter should be selected.

Recommended values of K_s for design purposes are 0.32 for new Transite pipe, 0.40 for steel pipe or portable aluminum pipe and couplers, and 0.42 for portable galvanized steel pipe and couplers.[23]

The diameter of the main should be adequate to supply the laterals in each of their positions. The rate of flow required for each lateral may be determined by the total capacity of the sprinklers on the lateral. The position of the laterals that gives the highest friction loss in the main should be used for design purposes. The friction loss in the main may be computed by equation 20.5. Allowable friction loss in the main varies with the cost of power and the price differential between different

Table 20.5 CORRECTION FACTOR F FOR FRICTION LOSSES IN PIPES WITH MULTIPLE OUTLETS*

No. of Sprinklers	*Correction Factor, F*
1	1.0
2	0.634
4	0.480
6	0.433
8	0.410
10	0.396
12	0.388
14	0.381
16	0.377
18	0.373
20	0.370
22	0.368

* From Christiansen.[5]

diameters of pipe. The most economical size can best be determined by balancing the increase in pumping costs against the amortized cost difference of the pipe. Pumping against friction presents a cost continuing for as long as the system is operated.

The design capacity for sprinklers on a lateral is based on average operating pressure. Where the friction loss, H_f, in laterals is within 20 per cent of the average pressure, the average head, H_a, for design in a sprinkler line can be expressed approximately by[5]

$$H_a = H_o + \tfrac{1}{4}H_f \tag{20.6}$$

where H_o is the pressure at the sprinkler on the farthest end. Thus, the average pressure is equal to the pressure at the farthest end plus one-fourth the friction loss. Where the lateral is on nearly level land or on the contour, the head (pressure) at the main,

$$H_n = H_o + H_f \tag{20.7}$$

or, by solving for H_o in equation 20.6 and substituting in equation 20.7,[23]

$$H_n = H_a + \tfrac{3}{4}H_f \tag{20.8}$$

On sloping land it may be necessary to make allowance for differences in elevation.

20.19. Pump and Power Units. In selecting a suitable pump (see Chapter 19), it is necessary to determine the maximum total head against which the pump is working. This head may be determined by

$$H_t = H_n + H_m + H_j + H_e + H_s \qquad (20.9)$$

where H_t = total design head against which the pump is working.
H_n = maximum head required at the main to operate the sprinklers on the lateral at the required average pressure.
H_m = maximum friction loss in the main.
H_j = elevation difference between the pump and the junction of the lateral and the main.
H_e = head as a result of a difference in elevation between the first and last sprinklers on the lateral.
H_s = elevation difference between the pump and the water supply after drawdown.

The amount of water that will be required is determined by multiplying the number of sprinklers by the capacity of each. When the total head and rate of pumping are known, the pump may be selected from rating curves or tables furnished by the manufacturer.

The size of the power unit required depends on the discharge, pressure, and efficiency of the pump. Power requirements may be computed by equation 19.1.

Although this method involves several approximations, it is adequate for practical design.

Example 20.3. Determine the size of sprinklers, lateral, pump, and power unit for the layout in Fig. 20.6*a* with the following conditions given: $H_a = 92.4$ feet (40 psi), $H_j = 2.2$ feet, $H_e = 7.0$ feet (3.0 psi), $H_s = 9.0$ feet, $r = 0.5$ iph, maximum length of main = 160 feet, $S_1 = 40$ feet, $S_2 = 60$ feet, and allowable variation of pressure in the lateral = 20 per cent.

Solution. Determine sprinkler and lateral capacity: From equation 20.3, the required discharge of each sprinkler,

$$q = \frac{40 \times 60 \times 0.5}{96.3} = 12.5 \text{ gpm}$$

From equation 20.4*b*, the theoretical discharge of each sprinkler (using $^{13}\!/_{64}$- and $^{5}\!/_{32}$-inch nozzles with $C = 0.98$),

$$q = 29.85 \times 0.98 \times 40^{1/2} \; [(^{13}\!/_{64})^2 + (^{5}\!/_{32})^2] = 12.1 \text{ gpm}$$

Lateral capacity for 10 sprinklers = 121 gpm.

Determine diameter of lateral and main: Total allowable variation of pressure in lateral = 0.20 × 40 = 8.0 psi.

Allowable variation of pressure due to friction = 8.0 − H_e = 8.0 − 3.0 = 5.0 psi (11.5 feet).

Compute H_f for 3-inch-diameter lateral (16 gage; wall thickness, 0.051 inch) from equation 20.5, using K_s = 0.40.

$$H_f = \frac{0.40 \times 400 \times 121^{1.9}}{0.241^{4.9}} \times 1.45 \times 10^{-8} = 22.4 \text{ feet}$$

From Table 20.5, select F, and compute friction loss for multiple outlet pipe, 22.4 × 0.396 = 8.9 feet (within allowable of 11.5).

Pipe Outside Dia., in.	*Lateral 400 Feet* $H_f \times F$	*Main 160 Feet* H_m
2.5	22.0	—
3	8.9*	8.5
4	2.0	2.1*
5	—	0.7

* Select 3-inch lateral and 4-inch main.

Determine head required at the main:
From equation 20.8,

$$H_n = 92.4 + (3/4 \times 8.9) = 99.1 \text{ feet}$$

Determine pump size: Total capacity of pump, 121 gpm. From equation 20.9,

$$H_t = 99.1 + 2.1 + 2.2 + 7.0 + 9.0 = 119.4 \text{ feet}$$

Select pump from manufacturer's data to deliver 121 gpm at a head of 120 feet.

Determine size of power unit: Obtain pump efficiency from manufacturer's rating curves (65 per cent) and from equation 19.1,

$$\text{hp} = \frac{121 \times 8.34 \times 119.4}{60 \times 550 \times 0.65} = 5.6$$

Select power unit capable of continuously furnishing 5.6 hp (see Chapter 19), such as a 7.5-hp electric motor or 8.0-hp water-cooled internal combustion engine.

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PROBLEMS

20.1. Determine the comsumptive use and irrigation requirement for small grain to be grown where K is 0.65. The average monthly temperatures for the 3 months of the growing season are 70.8, 74.1, and 76.2, respectively. The percentages of daytime hours for the same period are 8.7, 9.3, and 9.5, and the average rainfall is 1.30, 1.91, and 2.13 inches, respectively. Assume the water application efficiency is 60 per cent.

20.2. A 3-inch application of water measured at the pump increased the average moisture content of the top 2 feet of soil from 18 to 24 per cent. If the average dry density of the soil is 75 pcf, what is the water application efficiency?

20.3. Determine the required capacity of a sprinkler system to apply water at a rate of 0.5 iph. Two 620-foot sprinkler lines are required. Fifteen sprinklers are spaced at 40-foot intervals on each line, and the spacing between lines is 60 feet.

20.4. Allowing 1 hour for moving each 620-foot sprinkler line, how many hours would be required to apply a 2-inch application of water to a square 40-acre field? How many days are required assuming 10-hour days?

20.5. Determine the discharge in gallons per minute for a sprinkler operating at 40 psi and having $\frac{5}{32}$- and $\frac{7}{64}$-inch-diameter nozzles with a discharge coefficient of 0.96.

20.6. At what rate in inches per hour would the sprinkler in Problem 20.5 apply water if the sprinkler spacing is 40×60 feet?

20.7. Compute the total friction loss for a sprinkler irrigation system having a 4-inch-diameter main 800 feet long and one 3-inch lateral 380 feet long. The pump delivers 135 gpm and there are 10 sprinklers on the lateral. Portable aluminum pipe (16 gage) and couplers are used throughout.

20.8. Design a sprinkler irrigation system for a square 40-acre field to irrigate the entire field within a 10-day period. Not more than 16 hours per day are available for moving pipe and sprinkling. Two inches of water are required at each application to be applied at a rate not to exceed 0.35 iph. A 75-foot well located in the center of the field will provide the following discharge-drawdown relationship: 200 gpm-40 feet; 250 gpm-50 feet; 300 gpm-65 feet. Design for an average pressure of 40 psi at the sprinkler nozzle. Highest point in the field is 4 feet above the well site, and 3-foot risers are needed on the sprinklers. Assuming a pump efficiency of 60 per cent and assuming that the engine will furnish 70 per cent of its rated output for continuous operation, determine the rated output for a water-cooled internal combustion engine.

CHAPTER 21

Land Clearing

Modern power machinery has provided an entirely new means of providing fast and effective land clearing under even the most difficult conditions. Through careful selection of methods and equipment the agricultural engineer can accomplish any degree of clearing justified by the intended land use.

In the application of a modern land-use program in which each acre of land is used in accordance with its capabilities, it is frequently wise to develop idle, brush-covered, and occasionally forested lands, so that they may replace for cultivation or pasture, lands that are less suited to such uses. Similarly, major improvements such as enlarging fields, preparing pond and reservoir sites, and building access roads frequently require clearing. Such clearing operations should be planned and executed so as to minimize soil profile disturbance.

TREES AND BRUSH

Each clearing project has its own individual characteristics. No single method or type of equipment is economically or structurally suited to all land clearing jobs. Therefore, each project must be carefully studied by an engineer to determine the most feasible approach.

21.1. Factors in Selection of Methods and Equipment. The four major factors that must be considered in analyzing a land clearing project are: (1) proposed use of the area, (2) the physical characteristics of the area, (3) the characteristics of the material to be removed, and (4) the economic factors involved.

The use to which the land is to be put determines the degree of clearing necessary. Reservoir sites and rough range land may need only cutting and brush removal, but improved pastures and cultivated areas will require complete removal of stumps, roots, and rocks, and then leveling and disking operations.

The physical features of the land closely control equipment and method selection. On rough gullied areas, equipment must be suited to carrying out the necessary land grading and smoothing. Areas having extremely large stumps or boulders require larger tractors and heavier implements.

The characteristics of the plant growth are extremely important factors to consider. Areas of light brush, palmetto, mesquite, or similar plants may be mowed or cut close to the ground, or, if complete clearing is desired, an undercutting or root-cutting implement followed by a root rake for lifting and piling may be needed. Larger stumps, particularly those having tap roots, will require heavy crawler tractors equipped with dozer blades or stumpers. Table 21.1 gives a partial summary of the average rooting characteristics of some common trees and shrubs.[18]

Plant growth characteristics influence clearing methods; for example, in clearing palmetto, it is necessary to restrict the depth of undercutting to 2½ to 4 inches, whereas in scrub oak and other brush it is desirable to undercut well below plow depth. Plant materials that are subject to sprouting or repropagation may require, in addition to thorough root raking and disking, a program of clearing maintenance with frequent brush cutting or chemical spray treatments during the first year or two after the initial work is completed. Fibrous materials that will easily rot may be left as a surface mulch or incorporated in the soil as organic matter by disking. Heavy brush, stumps, and trees should be piled or windrowed for burning. This requires equipment that will develop high, dense piles with a minimum of soil that would impede burning.

In determining the economics of a clearing job, particular consideration must be given to the size of the area, the number and proximity of other jobs, and the size and type of materials to be removed. Time requirements for different types of clearing vary with soil, growth density, equipment type, and operator experience[6] (see Appendix F). Time factors due to seeding dates or seasonal weather changes also determine the size and number of units. Differentials in property values due to clearing, temperature, rainfall, labor supplies, and other working conditions must also be considered.

Table 21.1 ROOTING CHARACTERISTICS OF COMMON TREES AND SHRUBS*

Root Type	Description	Deciduous	Coniferous	Brush Type Growth
	Deep tap root with short laterals	N. Red Oak, Mockernut, Hickory, Shellbark Hickory, Pignut Hickory, Shagbark Hickory	Longleaf Pine, Shortleaf Pine	
	Short to intermediate tap roots with moderate to long laterals	Overcup Oak, Swamp Chestnut Oak, Pin Oak, Walnut	Pond Pine, Slash Pine, Pitch Pine†, Eastern White Pine†, Norway Pine, Loblolly Pine‡	Cactus
	Flat, spreading, medium shallow	Water Oak, Swamp Willow, Swamp Cottonwood, Cottonwood, Quaking Aspen, Bigtooth Aspen, Red Gum, Tupelo Gum, Black Gum, Black Locust, River Birch, Yellow Birch, Bald Cypress, Hackberry, Green Ash, White Ash, Red Ash, Black Ash, American Elm, Red Maple, Silver Maple, Sugar Maple, Box Elder, Beech, Sweet Buckeye, Sycamore, Red Spruce	Loblolly Pine‡, Scrub Pine, So. White Cedar, Hemlock, Balsam, Fir	Mesquite, Blackbrush, Creosote Bush, Palmetto, Chaparral
	Deep, wide, and spreading	Black Cherry, Chestnut Oak, Black Jack Oak, S. Red Oak, Post Oak, Yellow Poplar, Cucumber Tree, Black Birch, Hardy Catalpa, Honey Locust, Basswood, Butternut, Black Walnut, White Oak	E. Red Cedar†, White Pine, Fir	Scrub Oak, Osage Orange

* After Gill.[18] In general these are the rooting characteristics; however, variations in soil type and in moisture conditions may vary these forms. † On dry soil sometimes has long tap. ‡ Deep early, later spreading.

LIGHT CLEARING

Many of the areas being cleared for agricultural purposes consist mainly of abandoned lands having a regrowth of brush and scrub trees. The deep Southeastern areas are frequently infested with palmetto, scrub oak, and pine. In the range lands of the Southwest clearing of black-brush, cactus, mesquite, cedar, and other brushy types of growth is needed for extensive range improvement.[30] In California the chaparral lands present similar problems.[27]

In complete clearing of such lands, four criteria must be met: (1) adequate removal of the roots, bulbs, and crowns to prevent undue sprouting and regrowth; (2) systematic piling or windrowing to facilitate burning and to provide regular, easily managed cleared areas; (3) a minimum displacement of the topsoil; and (4) a smooth seedbed sufficiently graded and cleared of roots and stones to allow full use of planting, cultivating, and harvesting equipment.

21.2. Surface Cutting. Where the plant species are nonsprouting they can be controlled by surface cutting if the proposed land use does not require full removal of stumps and roots.

Light brush, with a stem diameter of less than $1\frac{1}{2}$ inches, can be clipped with heavy-duty sickle mowers. These sickle bars are shorter and heavier than standard bars and are equipped with stub guards and extra hold-down clips. Where sage brush or heavy grass is also encountered, underserrated sections are recommended. The tractor is run in low gear to give a high sickle speed in comparison to the forward speed.[12,13,14] After sickle cutting, it is usually necessary to remove the cuttings from the fields with buck rakes.[13]

A number of rotary brush cutters will operate, depending upon their size and construction, in brush having stem diameters up to 3 to 4 inches. These cutters consist of two or more sets of high-strength steel blades revolving in a horizontal plane at high speed. After the initial cut, the brush is pulled up into the blades by the fanning action of the blades, where it is chopped and shredded into a mulch as it passes through them. This shredding eliminates further raking and piling. Such equipment should be carefully selected to provide adequate safety shields, slip clutch, and shear pin devices to prevent undue breakage, and adequate adjusting devices to allow a range in cutting heights. Figure 21.1*a* illustrates the basic design of this type of cutter.

Various types and sizes of brush beaters, consisting of a series of weighted chains revolving about a horizontal shaft, create an effect similar to a hammermill.[2,14] Thorough shredding of the material by these beaters usually makes it unnecessary to rake and pile the debris.

Various types of circular saws may be attached to garden or farm tractors in cutting larger brush and small trees. Though power chain saws are very maneuverable and well suited to intermediate-size trees, they are not efficient in clearing brush and small trees.

Several companies have developed brush and tree cutters that consist of a modified dozer blade equipped with either straight or serrated cutting blades attached to the lower edge.[17,28] Mounted on the C-frame of a crawler tractor, these cutters slice through brush and trees from 8 to 14 inches in diameter, depending on the number of cuts made.

Extremely heavy stalk cutters, consisting of a horizontal weighted drum (3000 to 21,000 pounds with power requirements of 20 to over 100 hp) to which are attached various types of blades, may be used to knock down and crush brushy growth.[9,17] These machines (an example of which is shown in Fig. 21.1*b*) have a potential wide application for crushing dead brush and small trees that have been killed with chemicals.

21.3. Undercutting. If cultivated crops or improved pastures are to be grown on a brushy area, it is necessary to remove the stumps and roots to below plowing and cultivation depths. With a plant such as palmetto, it is essential to cut the feeder roots below the horizontal rhizomes and then remove the latter from the field to prevent undue regrowth.

One type of undercutter consists of a special blade mounted on a motor grader. This is particularly effective in light and medium infestations of palmetto; in land free of stumps and rocks, it can be operated up to 5 mph, thus cutting 3 to 4 acres per hour. In heavier infestations of palmetto and in fields having stumps, an implement having a V-blade mounted on a frame with two sections of a heavy adjustable disk trailing, as shown in Fig. 21.1*c*, has obtained kills up to 95 per cent. Requiring

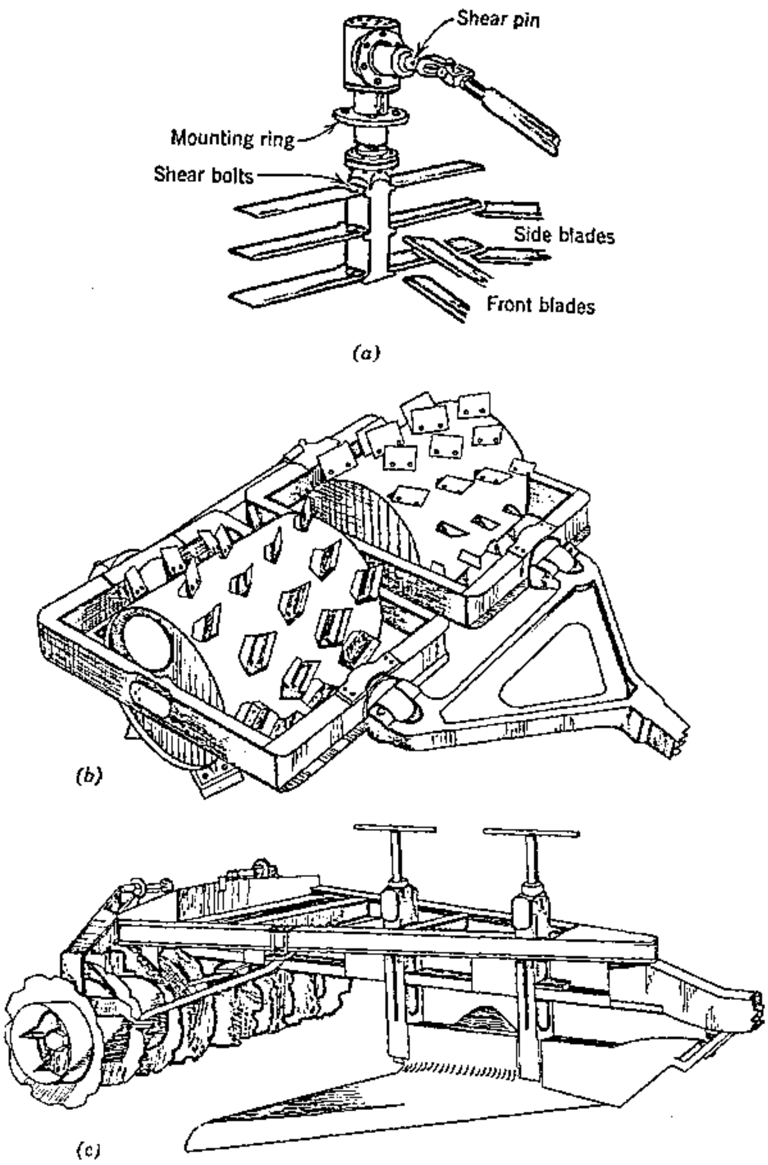

Fig. 21.1. Surface cutters, choppers, and undercutters. (*a*) Cutaway of a rotary cutter, (*b*) rolling chopper, and (*c*) undercutter with mounted disk. (Redrawn from references 9 and 19.)

60 drawbar horsepower (dhp) or over, this implement cuts from 2 to 3 acres per hour.

A heavier type of undercutter or root plane consists of a cable-operated tool bar mounted on a crawler tractor, to which is attached two extra large rolling coulters followed by two standards, each holding a V-type sweep with face-hardened cutting edges.[17] These sweeps may be led into and planed out of the ground without undue disturbance of the soil; they may be operated at any depth from 2 to 18 inches.

21.4. Heavy Disking. Where lightly brushed areas are to be seeded, the brush can be cut and shredded into the seedbed by heavy disks or *bush and bog* harrows with 26- or 28-inch cutting disks and ranging in weight up to 5000 pounds. Heavy disks are effective in rocky areas or where there are hidden stumps, for their rolling action prevents breakage when they hit such objects.[26]

21.5. Chemical Methods. The two general types of chemical agents are:

1. *Selective herbicides,* which generally kill broadleaf plants at given dosages yet may not affect the growth of grasses and certain other plants. The hormone-type chemicals, 2-4-D and 2,4,5-T are examples of selective herbicides that are absorbed and transported within the plant.[30]

2. *Nonselective (contact) herbicides,* which kill the above ground parts of most plants that are treated. Ammate, arsenicals, dinitro compounds, pentachlorophenol, petroleum products, and trichloroacetates are representatives of the nonselective type.[30]

The selective herbicides, available in salt, amine, and ester forms, are used with carrying agents such as water, oil-water emulsions, and oils. Certain nonselective materials may be added to selective materials to make a more effective solution. These solutions may be applied as sprays to the foliage, to freshly cut stumps, to frills or girdled areas on trees, or directly to the trunk. Tractor or truck sprayers may be used effectively on moderate-sized areas of low brush which is not too dense to allow uniform coverage. On large areas or on dense high brush, application by airplane or helicopter is most feasible.

The selection and safe management of such herbicides is most complex. Their effectiveness is a function of rate, carrying agent, method of application, and time of application and, in some instances, is based on the interaction between two or more types of materials; therefore, their use should be under the supervision of specialists in that field.[3,5,11,23,24,30]

21.6. Miscellaneous Methods. In exceptional cases, small areas of light brush may be cleared with off-peak-season hand labor. Where limited acreages are concerned, goats, sheep, and cattle may be used to clear by grazing.[4,8,25] Controlled burning may be relatively effective on certain types of light brush that will not resprout from a bud ring. However, attendant with burning are the dangers of its effect on erosion, distribution of wildlife, and its danger to adjacent areas.

INTERMEDIATE AND HEAVY CLEARING

21.7. Methods and Equipment. When areas to be cleared include lands covered with large brush and timber, clearing operations must be carefully and systematically planned to provide equipment and methods that will provide maximum efficiency. To assure quality lumber the usual practice is to cut and remove all merchantable timber before clearing operations. Large stumps are sometimes left 2 to 3 feet high to facilitate removal. Pulpwood may be either precut or cut out after felling in the stumping operation.

Selection of equipment depends largely upon the intended land use. Where topsoil is essential, bulldozers should be replaced by root rakes and clearing blades that will rake and scarify but will not pile up the soil. Where a number of large stumps are encountered stumpers are efficient because they concentrate all the effort on a small area. As shown in Fig. 21.2*b*, this device attaches to the C-frame by a center pin. Weighing from ½ to ¾ of a ton, it will penetrate the ground up to 24 inches in an area 30 to 36 inches wide.

The general maxim of clearing contractors is *the larger the tractor, the cheaper the job*. Economy lies in having ample horsepower, thus eliminating as far as possible costly digging out and special machinery to supplement power deficiencies. The most popular tractor sizes for contractors range between 65 and 90 rated dhp; when existing agricultural power units are

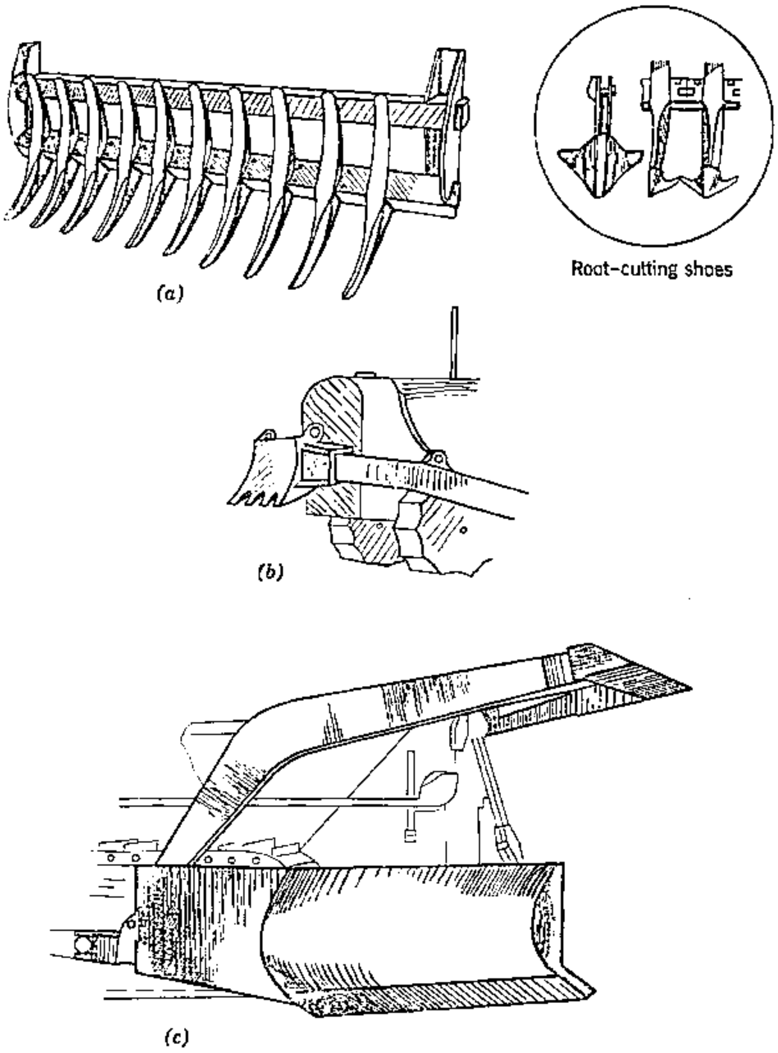

Fig. 21.2. Specialized equipment for clearing and preparing land. (*a*) Root rake, (*b*) stumper, and (*c*) tree dozer with knockdown beam. (Redrawn from reference 17.)

used, this may drop to 40 hp. Tractors of greater than 100 hp are used for heavy or extensive clearing.

On the small and intermediate-size tractors, hydraulic controls are preferred. These permit additional downward pressure on the cutting blade and fast, exact positioning of the blade.

On the larger equipment, the basic weight of the implements makes hydraulic control less important, and here cable controls are preferred. With both dozer blade and root rakes, the preference is for mountings that permit both angling and tilting to facilitate rooting out rocks, cutting tree roots, constructing ditches, and completing rough grading.

Quickly interchangeable blades, rakes, and stumpers make it possible to convert equipment to fit specific needs.[1,10,19,21] Some root rakes and rock rakes may be equipped with wearing caps to decrease tooth wear and damage. Shoes providing wings with cutting edges may be attached to rakes when an extensive root-cutting surface is needed. The capacity of the rake shown in Fig. 21.2*a* could be increased by top guards and gathering wings.

In standing trees a "tree-dozer" with a knockdown beam will permit application of thrust 7 to 8 feet above the ground, thus providing maximum leverage. The curved V-blade then grubs under, lifting and moving the loosened tree to the side. This type of equipment is shown in Fig. 21.2*c*.

Removal of large trees and stumps is best accomplished by cutting the roots on two or more sides with the blade and then pushing at the highest possible point. Tap-rooted trees and stumps can best be removed by digging out with the dozer to a point at which the diameter is such that a thrust applied at that point will break the root below tillage depth.[28]

Where large acreages must be cleared for dam or reservoir sites, the chain and ball method has been found effective. A large (8 feet in diameter) steel ball, attached to cables or anchor chains, is pulled through the area by two large tractors. Trees and brush are knocked down for later piling.

21.8. Follow-Up Equipment. Root rakes and similar specialized equipment permit thorough combing of cleared areas to remove roots, stump fragments, small brush, and undergrowth. Careful raking and finishing are essential to ensure against broken cultivation implements.

For pasture and range land, final finishing can be accomplished with heavy disk plows and harrows. These implements cut and incorporate into the soil for quick decomposition the small brush, twigs, and root fragments that remain after the major clearing operation.

21.9. Disposal of Debris. A definite pattern of clearing should be established to facilitate removing debris. Unless there are gullies, swamps, or other areas in which the debris can be buried, a system of windrows should be set up. This system is economical for it permits working in two directions. It also leaves large rectangular areas that are more easily farmed than when debris is piled at random.

If the windrows are tall, compact, and relatively free of soil, they will, upon drying, burn well. Root and brush rakes permit this type of piling. These rakes may be used to compact partly burned windrows to assure a more complete burn.

21.10. Explosives and Other Methods. Where adequate power is not available for clearing heavy stumps or where only a few scattered stumps exist, other methods of clearing may be substituted. These include explosives, various stump pulling devices that provide added leverage for hand or animal power, and hand grubbing. The safe use of explosives requires special training and experience.[16,20]

BOULDERS AND STONES

In many areas of the country, particularly in the glaciated regions, surface and near-surface boulders and stones contribute serious hazards to modern farming. Scattered boulders prevent the use of modern equipment, stone fences restrict field sizes, and shallow-buried stones interfere with high-speed tillage. Modern engineering makes possible the removal of many of these hazards.

21.11. Machine Methods. Small scattered stones are usually removed by hand picking. However, machines have been devised that will scoop up the soil to plow depth and sieve out small stones by passing it over a chain shaker similar to that in a potato digger.[22]

Large boulders ranging up to 5 and 6 feet in diameter may be removed with heavy tractors equipped with various types of blades. Heavy-duty stone rakes are usually more effective than dozer blades because they disturb and move less topsoil while they are grubbing under the boulder. Where boulders are to be moved only short distances to field edges or gullies, they can be picked up or rolled with power equipment. On longer moves, it is more economical to push a number of them onto a stone sled that may be pulled to the dumping point.

21.12. Burying. In order to reduce the cost of stone removal due to long hauls, a technique of burying the stone, well below tillage depths, has been developed.[15] Where scattered boulders exist, a series of pits are first dug with a bulldozer, pan, shovel, hoe, or dragline. The boulders are then pushed into these pits and the soil replaced over them. Excess soil can be spread or used for grading and filling gullies and holes left by the removal of the stones.

In the removal of stone fences, a ditch is dug close to and parallel with the fence.[15] It is then a simple matter to bulldoze the stones into this ditch and cover them. In both instances, care must be taken to assure adequate depth of covering soil to prevent damage to both normal and deep tillage equipment.

21.13. Use of Explosives. Where heavy power equipment is not available or where only a few large boulders exist, explosives provide a means of breaking up the larger stones into sizes that can be handled by farm power and equipment. Full details on the use of explosives for removing and breaking rock may be found in a number of references.[7,16,20]

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CHAPTER 22

Legal Aspects of Soil and Water Conservation

Although an agricultural engineer is not expected to give legal advice, he should have a working knowledge of the main legal principles involved in soil and water conservation since in everyday practice questions arise the answers to many of which require consultation with legal authorities. In many instances, the decision of an engineer in the design of an installation may be greatly influenced by legal considerations. The engineer comes in contact with the law most frequently when dealing with contract and tort (civil) law; with legal interpretation of work specifications; and with laws regarding group water organizations, such as district enterprises organized under state laws, the acquisition of rights-of-way, and water rights of land and water users. The engineer may also be called upon to serve as an expert witness in court and should have some rather definite ideas as to how meaningful facts may best be presented to juries.

In the development of this country, it was found necessary to enact certain legislation which would provide for the public welfare. For example, laws were needed to facilitate the draining of otherwise good agricultural land where the project was for the benefit of several landowners and the public as well. If no such enactments were in effect, one property owner could prevent the development of the project unless he could be induced to cooperate.

The legal aspects of drainage, erosion control, and irrigation enterprises are considered here. In addition, certain phases of common law, statutory law, and a few legal documents are discussed in limited detail. The legal phases of engineering are presented in other references,[15,22] and the aspects of the law that apply to the farm are discussed in Hannah.[7] The President's Water Resources Policy Commission[18] has prepared a summary of federal laws relating to water resources, including navigation, flood control, irrigation, drainage, and land use, as well as a very brief statement regarding state water laws in the 17 western states.

22.1. Kinds of Law. Our modern judicial system has been derived principally from three distinct sources: (1) English common law, (2) Roman and French civil law, and (3) statutory enactments.[3,15] English, French, and Roman law and local customs constitute the basis for our present common law. Common law is that which comes into being by custom, usage, or precedent, and consists of the principles that have been established and upheld by state courts. An example of a common law which is adopted by usage is the share of the crop belonging to the tenant. Since both English and Roman law were adopted more or less indiscriminately in various localities, present state laws may follow the common law in one state, the civil law in another, or a combination of the two in others. Although most states recognize English common law, civil law was introduced by the colonists in Louisiana and is still in effect.[3,15] Statutory laws are those enacted by regularly constituted legislative bodies, such as the U. S. Congress, State Legislatures, and, by delegated authority, municipal subdivisions of the state.[3,15] A statutory law may repeal any common law that is in conflict with it, so long as this does not violate constitutional principles, and may also repeal any previous statutory law in conflict with the new law.

When a conflict between parties is brought before a court of justice, it must first be determined whether there is any statutory or common law that covers the case. If there is no applicable law, it may be necessary to extend by analogy a common law principle. Finally, if no such principle exists, it may be the duty of the court to decide the case according to its best judgment or hold that the matter is for the legislature to consider.

22.2. Contracts. Frequently, a landowner or an organized group does not own suitable equipment for doing construction or installation work. Under such conditions, the job is done by contract which is an agreement, enforceable by law, between the parties concerned. The essential features of a valid contract include (1) an agreement between *competent parties*, which includes all adults except the insane and others specified by law; (2) a *lawful subject matter*, i.e., it must not violate statutory laws, contradict common law, or be forbidden by public policy; (3) *a consideration* or exchange of things having value; and (4) *an agreement* or mutual understanding (meeting of the minds) and consent to the terms considered.[22] In construction contracts the agreement should describe the general nature and location of the work, specify time limits during which the contract is in effect, and state the contract price. Although a valid contract may be either oral or written, a written contract is always preferable because it helps to prevent misunderstandings and encourages greater consideration of the details by both parties involved; it is much easier to prove in court.

All contracts should be written in clear, concise language and as simply as may be consistent with the scope and detail of the undertaking so that a minimum of misinterpretation will result. For many construction jobs approved blank forms can be obtained on which the necessary information can be entered. If the contract contains detailed specifications or if the blank forms are not suitable, it should be prepared very carefully, preferably by an attorney. The fees of an attorney are low in comparison to the cost and waste of time that may result from an improperly prepared contract. It should be remembered that any contract that specifies something forbidden by law is void.

22.3. Easements. An easement is an interest or right that a person or organization may hold in the land of another without actually having continuous possession of the land. The most common form is a right-of-way easement across the land for tile drain outlets, highways, power lines, and access roads where a landowner cannot travel from his property to a public highway without crossing the land of another property owner. This type of easement is usually limited to a narrow strip of land and is called an *easement of way*.

Easements of way may be acquired by stipulation in a deed, by long-continued use for the statutory period (usually 20 years), by contract or lease, or by eminent domain where the right to public use is involved. Some types of easements are binding on future owners, and others are nontransferrable. An easement of way may become void by nonuse of the property. Legal counsel should be consulted when contracts are made or easements are secured.[7,15]

DRAINAGE

A body of common law and statutory law is applicable to farm drainage and organized drainage enterprises. The civil law rule adopted in 13 states specifies that the owner of higher land by virtue of its position is entitled to the natural advantage of drainage, and that the lower owner must receive the natural flow in drainageways or swales.[3,17]

The English common law rule applicable in 18 states considers diffused surface waters not in well-defined channels to be a common enemy, thus a land owner has the unrestricted right to protect his land against diffused surface water from adjoining land.[3,17] By either rule, drainage waters may not be unduly collected, concentrated, and discharged upon the land of another so as to cause damage. Thus, runoff should not be materially greater in quantity or velocity than would occur naturally.

Since the common law may not permit the construction of a drainage outlet across the land of another, the various states have enacted statutes to correct limitations in the common law and to make possible the drainage of potentially productive wet land where several landowners are involved. The most common types of drainage enterprises are mutual enterprises and drainage districts.

MUTUAL DRAINAGE ENTERPRISES

Many states have laws that provide for the organization of mutual drainage enterprises. To establish such an enterprise the landowners involved must be fully in accord with the plan of operation and with the apportionment of the cost. After the agreement has been drawn up and signed, it must be properly recorded in the drainage record of the county or other political subdivision. The local court may be asked to name the district officials, sometimes called commissioners, who are responsible for the functioning of the district, or they may be named in the agreement. The principal advantage of the mutual district is that less time is required to establish an organization and the costs are held to a minimum. Because it may be difficult for several landowners to come to an agreement, particularly on the division of the costs, such districts are difficult to organize where the number of landowners is large or where considerable area is involved. However, there are a large number of these small enterprises, and much drainage has been accomplished in this manner.

DRAINAGE DISTRICTS

A drainage district is a local unit of government established under state laws for the purpose of constructing and maintaining satisfactory outlets for the removal of excess surface and subsurface water. It is different from a mutual enterprise in that minority landowners can be compelled to go along with the project. Levee districts formed for the purpose of keeping out excess flood water are similar to drainage districts in their organization. For further details on drainage districts consult other references.[2,17,20,27] A uniform law for drainage, irrigation, or flood control districts has been proposed by Harman.[9]

22.4. Organization. Although the laws of the various states differ in detail, the general procedures for organizing drainage districts are similar in all states. The U. S. Sixteenth Census of 1940 has a concise summary of all state drainage laws, and other references have more details regarding all phases of district organization and function.[4,6,14]

Petition. The first step in the formation of a district is the preparation of a petition signed by the required number of landowners. The number of signers varies from one to a majority of the landowners or in some states the number of owners representing a majority of the land area. The petition should state (1) the purpose for organizing the district; (2) the general boundary of the land to be included; (3) names and addresses of landowners involved; (4) description and acreage of land owned by each; (5) the approximate starting point, routes, and outlet of the proposed improvement; and (6) other statements required by law. The petition should also include a request for the appointment of a competent engineer to make a preliminary survey. A bond must accompany the petition in most cases. This bond should be in sufficient amount to pay organization expenses and the cost for the preliminary survey in the event that the district is not approved. The petition should then be filed with the official designated by law, usually the clerk of the county court or auditor.

Preliminary Report. In compliance with the petition, the court appoints an engineer or in some states a board of viewers. The engineer then makes his preliminary survey and submits a report to the court or other officials, giving the general location, character, benefits, and cost of the proposed improvement. The court considers this report advisory in nature and an aid in deciding whether to establish the district.

Hearing. After the preliminary report has been filed, the landowners involved are notified either personally or by publication of the time and place for presenting their views on the establishment of the district. In some states landowners may file claims for damages prior to the date set for the hearing. This hearing is for the purpose of deciding whether there is sufficient objection to or evidence in favor of the formation of the district. If there is too much objection and other unfavorable evidence, the court may dismiss the petition. In any event the court should dismiss the petition if the benefits do not exceed the costs by a suitable margin. If the case is dismissed, the costs may be proportioned among the landowners signing the petition or otherwise distributed in accordance with the laws of the state. If the court rules in favor of the petition, the district is then organized. A board of officers, usually called commissioners or supervisors, is elected from the landowners involved, or in some states the county board of supervisors serves as the official board. The commissioners elect their own officers and proceed to administer the district in accordance with the laws of the state. The commissioners then employ an engineer to make the final survey and to supervise the construction of improvements.

22.5. Awarding the Contract. After the final plans of the engineer have been submitted and approved, the next step is to secure a reliable contractor to do the work. On large projects, it may be desirable to employ a disinterested consulting engineer to approve plans and specifications. The contract is usually awarded by advertising for bids, in accordance with the laws of the state. It is desirable to reserve the right to reject any and all bids since the lowest bid may not always be reliable. The engineer should be present at the time that the bids are let to answer questions regarding the plan. The successful bidder is placed under bond, and a written contract is completed as previously described.

22.6. Powers and Characteristics. Because drainage districts must have authority to carry out required drainage work, they are given certain definite powers in accordance with state laws. Some of the powers and characteristics of a properly organized drainage district are as follows: it may (1) exist as a form of corporation; (2) borrow money and issue bonds; (3) be financially responsible and sue and be sued in court; (4) levy and collect taxes on each tract of land to the extent of the benefits derived; (5) possess the power of eminent domain, permitting the district to condemn property; (6) construct necessary improvements to drain the land; and (7) hold and transfer property.[27]

22.7. Estimation of Benefits. After the contract for the work has been let and the project is under construction, a suitable board of assessors is appointed by the court to estimate the benefits. In some states the benefits are evaluated in monetary terms, and in other states numerical factors or percentages are estimated from which the costs are distributed. Since more dissatisfaction and hard feeling apparently result from the distribution of benefits than from any other cause, great care should be exercised in making these estimates.

Several factors must be considered in estimating benefits, namely: (1) wetness of the land or need for drainage, (2) thoroughness of drainage, (3) distance to natural outlet or main drain, (4) potential fertility of the soil, (5) condition of the land, (6) increased accessibility, and (7) other local criteria that may be applicable. Very wet land obviously will be benefited more than land that needs drainage only occasionally. The thoroughness of drainage will vary with different areas in the district because it is often not possible to drain adequately all land at a reasonable cost. When benefits are evaluated on this basis, the engineer should be consulted since he is familiar with the drainage plan. The land near constructed drains should normally be assessed more heavily than that further away since the cost for adequate private drains will be less than that for land at a greater distance. Land near the natural outlet ordinarily is not assessed as heavily as that at the upper end of the district since the land farthest from the outlet is making use of the entire length of the drainage system. However, local conditions may dictate otherwise. The relative fertility of the soil or its ability to produce when adequately drained should serve as an important basis in arriving at the benefits.[20] The condition of the land refers to such impediments to cultivation as rocks, brush, and trees. Since land with such impediments may be suitable only for pasture or timber, even though completely drained, it cannot be assessed as heavily as tillable areas. In some instances relatively high land isolated by wet land may be made more accessible by district improvement.

22.8. Methods of Assessing Agricultural Land. Some state laws specify the particular method by which agricultural land shall be assessed and the basis for making these estimates. Where the method is not specified, the assessment should be accomplished by some logical and systematic procedure that can be easily explained and understood by landowners. Thoroughness and fairness in distributing the costs is much more important than the method. In all cases the assessments must be proportional to the benefits, and the ratio of benefits to assessments must be the same throughout the district.[2]

In addition to the benefits that accrue to agricultural land there may be benefits to highways, railroads, towns, and other property in the drainage district. These benefits are usually assessed as a percentage of the total cost of the improvements based on the actual benefits received. Further details on the assessment of property may be found in other references.[2,17,20]

A brief description of some of the common methods of assessment are described below.

Flat-Rate. The flat-rate method is easiest to evaluate since the cost is distributed uniformly on each acre in the district. Although not suitable for most drainage assessments, it is employed mainly in levee districts.

Increased Value. The increased value of the land is the difference between the present value and the estimated value after drainage improvements. Either the market value of the land or the revenue that it would produce may serve as a suitable basis for evaluation. This method is sometimes preferred because it is simple and readily understood by both assessors and landowners and also gives definite information regarding the value of the land for prospective bondholders.[2,17]

Classification. In the classification method the land is divided into five classes, namely, A, B, C, D, and E, which have assessment ratios of 5, 4, 3, 2, and 1, respectively. Although this method may result in less work for the assessors, it is very difficult to divide the land into five categories without causing injustice; injustice raises legal questions.

Percentage. Under the percentage method the land is divided into tracts of equal size, normally 40 acres, and each tract is evaluated on the basis of 100, which is considered the maximum benefit. The total number of benefit units for each landowner are thus determined, and the cost is distributed proportionally. A modification of this method permits the evaluation of any size area as one tract so long as the percentage benefit is the same. Under this modified system the number of benefit units can be determined by multiplying the percentage by the number of acres involved. Although this system appears to be quite detailed and systematic, it may be rather difficult to explain to landowners.

The following example illustrates the modified percentage method of computing assessments and the procedure for handling damages and nonagricultural benefits.

Example 22.1. Determine the total assessment for each landowner from the following data:

Owner	*Area Affected by Drainage, acres*	*Percentage Benefit*	*Benefit Units*	*Assessment*	
A	30	60	18	360	$1360
	50	100	50	1000	
B	80	55	44	880	1960
	40	60	24	480	
	30	100	30	600	
C	50	90	45		900
D	20	65	13	260	740
	60	40	24	480	
E	40	30	12		240
Total	400 acres		260		$5200

Costs

Construction		
Construction contract		$4500
Fees and expenses:		
Legal expenses	$250	
Expenses of district officials	100	
Engineering fees	400	
		750
Damages:		
Owners A, C, D	300	
Highway	250	
		550
Total Costs		$5800
Nonagricultural Benefits		
Highway	$ 50	
Railroad	300	
Town lots	250	
		600
Total Net Cost to Landowners		$5200

$$\text{Landowner assessment} = \frac{5200}{260} = \$20 \text{ per benefit unit.}$$

22.9. Damages. Damages that result from the construction of a drainage ditch must be considered separately from the benefits (see Example 22.1) because such damages are estimated prior to establishment of the district and are frequently paid before construction starts. Most damages result from the purchase of land for the right-of-way, the cutoff of irregular portions from a tract of land by an open ditch, the loss of growing crops during construction, damages to fences and other structures, and damages to railroads, town property, and highways. Where the right-of-way must be purchased, the value of the land should be determined according to the best judgment of the assessors rather than by strict rules. Damages to rights-of-way not purchased are the difference between the market value of the property before and immediately after construction. Many courts have ruled that no damages can be collected for the right-of-way of tile drains or for open ditches located in a natural watercourse.[2] Although the assessors determine damages, the final decision, if not acceptable to the landowner, rests with the court.

22.10. Financing. After assessments have been completed, damages determined, and expenses computed, an assessment roll is prepared showing the names of each landowner and the amount of his assessment. This roll is filed with the court and a hearing is held regarding assessments and damages. After making the necessary adjustments, the court orders the assessment roll approved. The cost of the improvement is then assessed against the property, and the tax is collected. Usually, 10 per cent is added to the cost of the improvement to take care of unforeseen expenses.[27] The usual method of assessment is by special taxes on the land benefited rather than by general taxation.

The two principal methods of financing drainage districts are: (1) by cash payment in advance of construction, and (2) by issuing bonds. After the tax levy has been made, landowners are usually given a certain time within which the payment may be made by cash. However, the issuance of bonds is the more common procedure.

IRRIGATION

The legal aspects of irrigation here will include only a discussion of water rights. Where water supply limits the potential area to be irrigated, there must be some legal control of water rights and water development. These aspects are important in the 17 western states as well as in the more humid eastern states. Although only 5 per cent of the total land irrigated in the United States is in the 31 eastern states, the problem of water rights needs considerable attention because of inadequate state water laws to cope with increased demand on water resources.

In the West irrigation enterprises consist largely of partnership enterprises, mutual irrigation companies, commercial companies, and irrigation districts. Irrigation districts are organized similarly to drainage districts, and almost without exception drainage is authorized as a related activity. Information regarding these various types of irrigation enterprises can be found in other references.[12,19] On the basis of the area irrigated, individual and partnership enterprises are extensive in all three regions of the West, whereas cooperative or mutual enterprises are prevalent in the Mountain states and organized districts in the Pacific and Plains states.

22.11. Water Rights. Two basic divergent doctrines regarding the right to use water exist, namely, riparian and appropriation. They are recognized either separately or as a combination of both doctrines in different states. Both doctrines apply only to surface water in natural watercourses and to water in well-defined underground streams.

Riparian Doctrine. The riparian doctrine which is a principle of English common law recognizes the right of a riparian owner to make reasonable use of the stream's flow, provided the water is used on riparian land. Riparian land is that which is contiguous to a stream or other body of surface water. The right of land ownership also includes the right of access to and use of the water, and this right is not lost by nonuse. Reasonable use of water generally implies that the landowner may use all that he needs for drinking, for household purposes, and for watering livestock. Where large herds of stock are watered or where irrigation is practiced, the riparian owner is not permitted to exhaust the remainder of the stream, but he may use only his equitable share of the flow in relation to the needs of others similarly situated.[3,11] This doctrine exists in all the eastern states and is retained in part in a few of the western states. Since few eastern states have statutory laws governing water rights, this doctrine is based mostly on court decisions.

Doctrine of Prior Appropriation. The doctrine of prior appropriation is based on the priority of development and use; i.e., the first to develop and put water to beneficial use has the prior right to continue his use. The right of appropriation is acquired mainly by filing a claim in accordance with the laws of the state. The water must be put to some beneficial use, but the appropriator has the right to all water required to satisfy his needs at the given time and place. This principle assumes that it is better to let individuals, prior in time, take all the water rather than to distribute inadequate amounts to several owners. Water rights are not limited to riparian land and may be lost by nonuse or abandonment.

This doctrine is recognized in all the 17 western states although in some it is in combination with the riparian doctrine. The right of appropriation applies specifically to the states of Arizona, Colorado, Idaho, Montana, Nevada, New Mexico, Utah, and Wyoming.[10] A combination system is generally recognized in the states of North Dakota, South Dakota, Nebraska, Kansas, Texas, California, and Washington.[10] To a more limited extent the combination system is also applicable to Oklahoma and Oregon.

EROSION CONTROL AND COMBINATION ENTERPRISES

The basic feature of the drainage district is that the benefits must exceed the cost, but this stipulation is not as easily applied to the control of erosion because soil losses are rather intangible and difficult to evaluate. Enterprises organized for the purpose of carrying out two or more engineering phases of soil and water conservation are here considered as combination enterprises.

SOIL CONSERVATION DISTRICTS

As a condition to receiving benefits under the Soil Conservation and Domestic Allotment Act, passed by Congress in 1935, the states were required to enact suitable laws providing for the establishment of soil conservation districts. The first district was organized in 1937, and by 1950 all states had enacted district laws.[18]

This legislation has been patterned largely after the *standard state soil conservation districts law.*[26] After such districts are established in local communities, they may request technical assistance from such agencies as the Soil Conservation Service, Cooperative Extension Service, and county officials for carrying out erosion control and other land-use management activities.

22.12. Purpose. The purpose of the soil conservation district is to provide a local group organization for the conservation of soil, moisture, and related resources and to promote better land use. Such an organization provides a means by which assistance may be requested from state, federal, and private agencies as well as the development of cooperative agreements with these agencies. In addition, demonstrational projects may be provided, equipment secured, and the services of technicians obtained so that individual farmers may more easily establish conservation practices.

22.13. Organization. Although the following organizational procedure is based on the standard districts law, individual state statutes vary somewhat from this procedure.

The standard law provides for a state soil conservation committee consisting of a chairman and from three to five members. In addition, representatives of the state college and the Soil Conservation Service may serve as members, as well as others indicated by state laws. The duties and powers of the committee are, briefly, to assist, supervise, and coordinate the programs of the districts; to secure the cooperation and assistance of federal and state agencies; and to disseminate information to the districts by advice and consultation.

The formation of a soil conservation district is quite similar to that of a drainage district. According to the standard law any 25 occupiers of land within the proposed district may file a petition with the state soil conservation committee. Within 30 days after the petition has been filed, the state committee shall give notice of a hearing at which time objections to or arguments for creating the district are discussed. If the hearing is favorable, the question is then submitted to the people for a referendum vote. If the referendum passes by at least a majority of the votes cast, the state committee can recommend the formation of the district. Within 30 days after the district is organized, 3 supervisors, also called commissioners or directors, are elected. In some states this election is held at the time of the referendum. Their term of office shall be 3 years, but the term of 1 supervisor shall expire each year. In addition to the 3 elected supervisors the state committee may appoint 2 members. In some states the governing body of the district consists only of the 3 elected members. Although the supervisors receive no salary except for necessary expenses, they may employ a secretary and other technical experts as required. The standard law also states that, at any time after 5 years of operation, the district may be discontinued according to the same procedure by which it was organized, that is, petition, hearing, and referendum.

Funds for operating the district are obtained by state appropriations; from funds, services, and properties made available through the Soil Conservation Service; and from other sources. The district may obtain funds by rental of district-owned or leased equipment and facilities.

22.14. Powers. A soil conservation district may have the following powers, provided they are not in contradiction to other state laws: (1) to conduct surveys, investigations, and research relating to soil erosion control programs; (2) to conduct demonstrational projects; (3) to carry out preventative and control measures on the land; (4) to cooperate and make agreements with farmers and to furnish technical and financial aid; (5) to make available to land occupiers, through sale or rent, machinery, equipment, fertilizers, etc.; (6) to develop conservation plans for farms; (7) to take over erosion control projects either state or federal; (8) to lease, purchase, or acquire property in order to carry out objectives of the program; and (9) to sue or be sued in the name of the district.[26] In practice the development of conservation farm plans and technical assistance in adopting conservation practices have been the principal results of the district setup.

The supervisors also have authority in some states to formulate land-use regulations, provided they are approved by a majority of the land occupiers by a referendum vote. In 3 states the vote must carry by 90 per cent.[5] These regulations may include provisions for carrying out terracing, building ponds, installing conservation structures, and adopting various types of tillage practices and cropping programs. They may also specify that certain lands should be retired from cultivation.

If land-use regulations are approved, they have the same authority as other local laws. The three general methods of enforcing these regulations are: (1) A violation of these regulations subjects the land occupier to trial for a misdemeanor and is punishable by a fine or otherwise as determined by state laws. (2) If any land occupier causes damage to other land by violation of any regulation, the damaged land occupier may recover such damages through court action. (3) Where land occupiers fail to conform to the land-use regulations, the supervisors may, upon authorization of the court, go on the land, do the necessary work, and collect the costs for the work. The land occupier may request release from certain land-use regulations by appealing to a board of adjustment.

Up to the present time, few land-use regulations have been adopted by soil conservation districts. There are 15 states that have no provision whatsoever for land-use regulations in their soil conservation district law.[5] In many of the remaining 33 states, the referendum regarding land-use regulations must pass by a high percentage of the votes, thus presenting a real obstacle to the adoption of such regulations. Only three such regulations have come to the attention of the Soil Conservation Service.[5]

WIND AND SOIL EROSION DISTRICTS

Wind and soil erosion districts have been organized in a few of the southwestern and western states.[7] Although these districts are similar in organization to soil conservation districts, they are generally given the power to raise funds through taxation.[7]

CONSERVANCY DISTRICTS

Conservancy districts are those enterprises generally organized for the purpose of soil conservation and flood control. They are often organized in the same general manner as drainage districts. Conservancy districts are authorized in a number of states, including Ohio, Indiana, Florida, Iowa, Minnesota, Kansas, and Texas.

RURAL ZONING

Rural zoning enabling acts have been adopted in California, Colorado, Georgia, Michigan, Minnesota, Missouri, Pennsylvania, Tennessee, Virginia, Washington, and Wisconsin.[24] This type of zoning is similar to that which is already in existence in towns and cities.[21] It provides a framework for land management both public and private. Examples of rural zoning ordinances include land-use regulations and restrictions to settlement in sparsely populated areas.

COMPARISON OF DISTRICT ORGANIZATIONS

Characteristics and statistics of drainage and soil conservation districts are compared in Table 22.1. The characteristics shown do not apply to all state laws, but they represent the range of data applicable to most states. Both types of districts are set up as governmental subdivisions of the state and are subject to state laws. The statistics for drainage districts include county drains. Although the area included in soil conservation districts

Table 22.1 Comparison of Drainage and Soil Conservation Districts

Item	Drainage District 1950	Soil Conservation District 1950
Statistics:		
Number in U. S.	3362* (only 30 states)	2400 (1953)
Total acreage included	95,000,000*,† (approx.)	1,248,000,000‡
Capital investment	$649,000,000*,† (1940)	
Avg. investment per acre	$7.95 (1940)	
Purpose:	Construction, operation, and maintenance of drainage facilities	Promote conservation of soil and water resources and wise land use
Organization:		
Nature of enterprise	Governmental subdivision of state (corporation)	Governmental subdivision of state (voluntary cooperative participation)
Governing body	3–5 elected members	3–5 elected members (2 appointed)
How organized	Petition-hearing-court order	Petition-hearing-referendum (50–75% favorable vote)
Number required to sign petition	1 person to 51% of landowners or by owners of a majority of the acreage	25 persons to 20% of land occupiers or landowners
Size of district:	No limit (usually several hundred acres to several square miles)	No limit (usually many square miles, such as a county in the Midwest)
Powers:		
Levy taxes	Yes	No
Issue bonds	Yes	No
Own property	Yes	Yes
Enforce regulatory powers	Yes	No, except by voluntary agreement or ordinance approved by landowners

* Includes totals for drainage districts and county drains larger than 500 acres.

† From reference 23.

‡ From reference 25.

is impressive, only one-sixth of the farms in districts (1950) is covered by farmer-cooperative agreements.[25] Information on the capital investment in soil conservation districts is not available. Except for district assistance and federal conservation payments, each individual farmer pays for his conservation improvements.

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PROBLEMS

22.1. Determine the total cost to landowners for the construction of an open ditch by a drainage district from the following data: excavation for ditch, $16,000; erosion control structures, $1200; damage to town lots and owners B and D, $900; attorney's fees, $500; engineer's fees, $1200; benefits to county road, $500; and benefits to town lots, $300.

22.2. If the total benefit units for the drainage district in Problem 22.1 are 320, determine the assessment of owner A if he has 80 acres receiving 50 per cent benefit and 40 acres receiving 20 per cent benefit. Use the modified percentage method.

APPENDIX A

Rainfall Characteristics

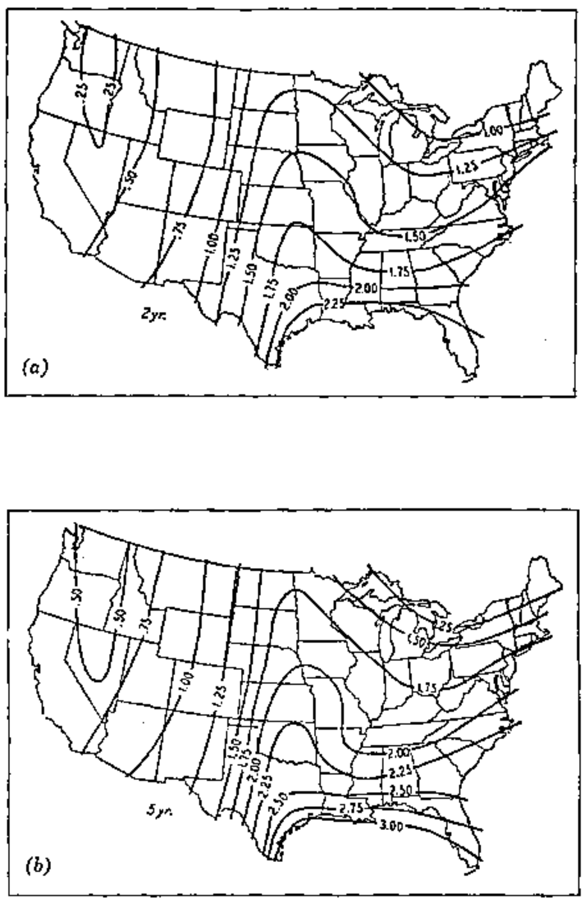

Fig. A.1. One-hour rainfall in inches to be expected at recurrence intervals of 2, 5, 10, 25, 50, and 100 years. [Redrawn from D. L. Yarnell, Rainfall Intensity-Frequency Data, *U. S. Dept. Agr. Misc. Publ. 204* (1935).]

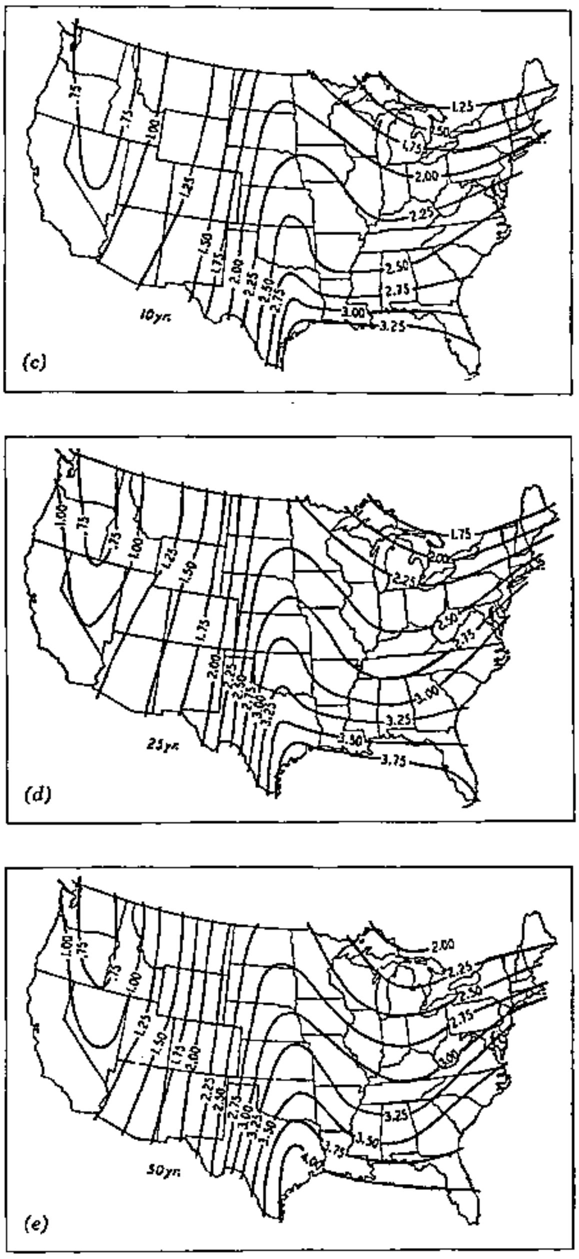

Fig. A.1 (*continued*).

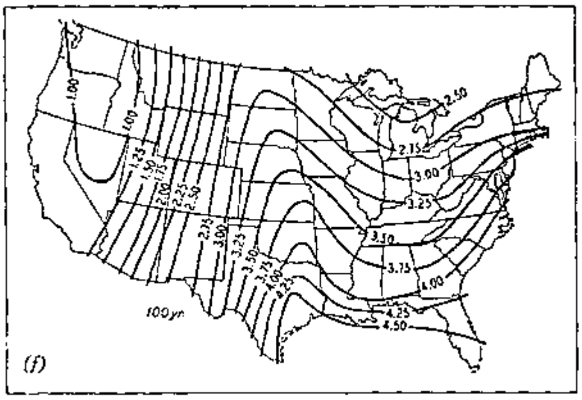

Fig. A.1 (*continued*).

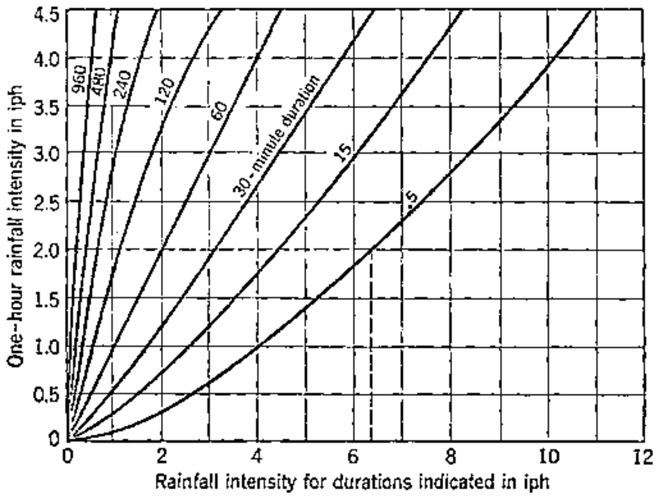

Fig. A.2. Relation of one-hour rainfall intensities to intensities at other durations. [Redrawn from G. A. Hathaway, Military Airfields—Design of Drainage Facilities, *Trans. Am. Soc. Civil Engrs.*, *110:* 700, Fig. 14 (1945).]

Example A.1. Determine the rainfall intensity and the total 5-minute rainfall to be expected once in 10 years for a storm of 5-minutes duration at Chicago, Illinois.

Solution. From Fig. A.1 the 1-hour intensity is 2.0 iph. Converting the 1-hour intensity to a 5-minute intensity from Fig. A.2 gives 6.4 iph. Total 5-minute rainfall equals 6.4/12 or 0.53 inch.

APPENDIX B

Runoff Determination

COOK'S METHOD

Table B.1 Runoff-Producing Characteristics for the Determination of Summation W *

Designation of Watershed Characteristics	Runoff-Producing Characteristics			
	100 Extreme	75 High	50 Normal	25 Low
Relief	(40) Steep, rugged terrain, with average slopes generally above 30%	(30) Hilly, with average slopes of 10 to 30%	(20) Rolling, with average slopes of 5 to 10%	(10) Relatively flat land, with average slopes of 0 to 5%
Soil infiltration	(20) No effective soil cover, either rock or thin soil mantle of negligible infiltration capacity	(15) Slow to take up water; clay or other soil of low infiltration capacity, such as gumbo	(10) Normal; deep loam with infiltration about equal to that of typical prairie soil	(5) High; deep sand or other soil that takes up water readily and rapidly
Vegetal cover	(20) No effective plant cover; bare or very sparse cover	(15) Poor to fair; clean-cultivated crops or poor natural cover; less than 10% of drainage area under good cover	(10) Fair to good; about 50% of drainage area in good grassland, woodland, or equivalent cover; not more than 50% of area in clean-cultivated crops	(5) Good to excellent; about 90% of drainage area in good grassland, woodland, or equivalent cover
Surface storage	(20) Negligible; surface depressions few and shallow; drainageways steep and small; no ponds or marshes	(15) Low; well-defined system of small drainageways; no ponds or marshes	(10) Normal; considerable surface-depression storage; lakes, ponds and marshes less than 2% of drainage area	(5) High; surface-depression storage high; drainage system not sharply defined

* After U. S. Soil Conservation Service, *Engineering Handbook for Farm Planners. Upper Mississippi Valley Region III, Agr. Handbook No. 57*, U. S. Government Printing Office, 1953.

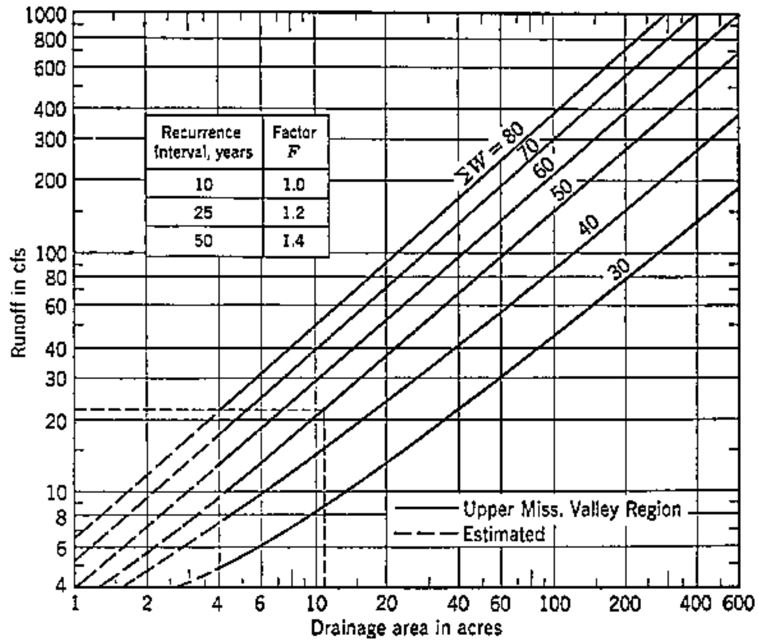

Fig. B.1. Runoff for 10-year recurrence interval. (See Table B.1 to obtain summation W.) (Data from U. S. Soil Conservation Service, *Engineering Handbook for Farm Planners. Upper Mississippi Valley Region III, Agr. Handbook No. 57,* U. S. Government Printing Office, 1953.)

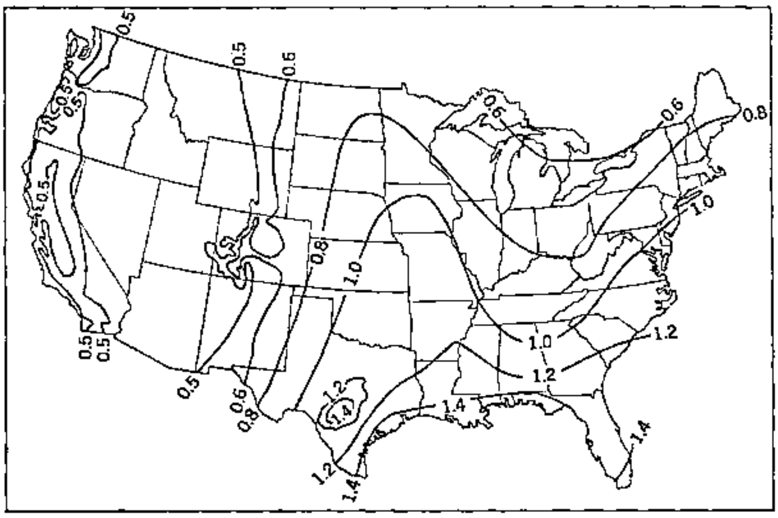

Fig. B.2. Rainfall factors for use with Fig. B.1. [Redrawn from C. L. Hamilton and H. G. Jepson, Stock-Water Developments: Wells, Springs, and Ponds, *U. S. Dept. Agr. Farmers' Bull. 1859* (1940).]

RATIONAL METHOD

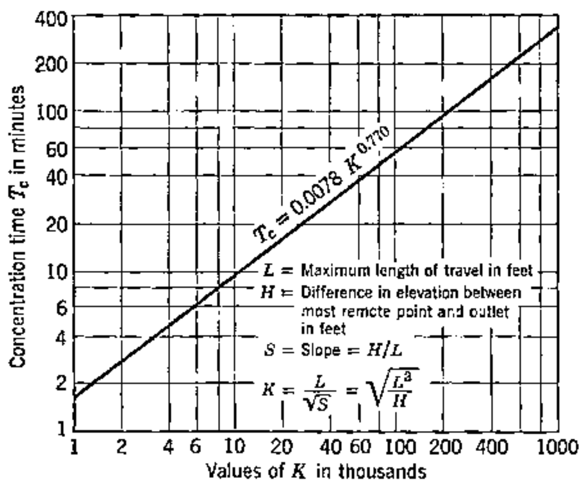

Fig. B.3. Time of concentration for small watersheds. [Redrawn from G. R. Williams, *Hydrology*, in H. Rouse, *Engineering Hydraulics*, John Wiley & Sons, New York, 1950. Based on data from U. S. Soil Conservation Service in California and C. E. Ramser, Runoff from Small Agricultural Areas, *J. Agr. Research, 34:* 797–823 (1927).]

Table B.2 RUNOFF COEFFICIENTS

Topography and Vegetation	*Values of C in Q = CiA* — *Soil Texture* — *Open Sandy Loam*	*Clay and Silt Loam*	*Tight Clay*
Woodland			
Flat 0–5% Slope	0.10	0.30	0.40
Rolling 5–10% Slope	0.25	0.35	0.50
Hilly 10–30% Slope	0.30	0.50	0.60
Pasture			
Flat	0.10	0.30	0.40
Rolling	0.16	0.36	0.55
Hilly	0.22	0.42	0.60
Cultivated			
Flat	0.30	0.50	0.60
Rolling	0.40	0.60	0.70
Hilly	0.52	0.72	0.82
Urban areas	*30% of Area Impervious*	*50% of Area Impervious*	*70% of Area Impervious*
Flat	0.40	0.55	0.65
Rolling	0.50	0.65	0.80

APPENDIX C

Manning Velocity Formula

Table C.1 ROUGHNESS COEFFICIENT n FOR MANNING FORMULA

Line No.	*Type and Description of Conduits*	*n Values* *Min.*	*Design*	*Max.*	*References**
	Channels, Lined				
1	Concrete	0.012	0.014 –0.016	0.018	1
2	Concrete, rubble	0.017		0.030	1
3	Metal, smooth (flumes)	0.011		0.015	1
4	Metal, corrugated	0.0213	0.0235–0.024	0.0258	6
5	Wood, planed (flumes)	0.010	0.012 –0.015	0.015	1, 5
6	Wood, unplaned (flumes)	0.011	0.013	0.015	1, 5
	Channels, Earth				
7	Earth bottom, rubble sides	0.028	0.030 –0.033	0.035	1
8	Large drainage ditches, no vegetation	0.025	0.035	0.035	3
9	Small drainage ditches	0.035	0.040	0.040	3
10	Stony bed, weeds on bank	0.025	0.035	0.040	1
11	Straight and uniform	0.017	0.0225	0.025	1
12	Winding, sluggish	0.0225	0.025	0.030	1
	Channels, Vegetated (grassed waterways) (See Chapter 9)				
	Dense, uniform stands of green vegetation more than 10 inches long				
13	(*a*) Bermuda grass	0.04		0.20	4
14	(*b*) Kudzu	0.07		0.23	4
15	(*c*) Lespedeza, common	0.047		0.095	4
	Dense, uniform stands of green vegetation cut to a length less than 2.5 in.				
16	(*a*) Bermuda grass, short	0.034		0.11	4
17	(*b*) Kudzu	0.045		0.16	4
18	(*c*) Lespedeza	0.023		0.05	4
	Natural Streams				
19	(*a*) Clean, straight bank, full stage, no rifts or deep pools	0.025		0.033	1

Table C.1 (*continued*).

Line No.	*Type and Description of Conduits*	*n Values* *Min.*	*Design*	*Max.*	*References**
20	(*b*) Same as (*a*) but some weeds and stones	0.030		0.040	1
21	(*c*) Winding, some pools and shoals, clean	0.035		0.050	1
22	(*d*) Same as (*c*), lower stages, more ineffective slopes and sections	0.040		0.055	1
23	(*e*) Same as (*c*), some weeds and stones	0.033		0.045	1
24	(*f*) Same as (*d*), stony sections	0.045		0.060	1
25	(*g*) Sluggish river reaches, rather weedy or with very deep pools	0.050		0.080	1
26	(*h*) Very weedy reaches	0.075		0.150	1
	Pipe				
27	Cast iron, coated	0.011	0.012 –0.013	0.014	1, 2
28	Cast iron, uncoated	0.012		0.015	1
29	Clay or concrete drain tile (4–12 inch)	0.010	0.0108	0.020	7
30	Concrete	0.010		0.017	1, 2
31	Metal, corrugated	0.021	0.025	0.0255	6
32	Steel, riveted and spiral	0.013	0.015 –0.017	0.017	1, 2
33	Vitrified sewer pipe	0.010	0.013 –0.015	0.017	1, 2
34	Wood stave	0.010	0.012 –0.013		1, 2
35	Wrought iron, black	0.012		0.015	1
36	Wrought iron, galvanized	0.013	0.015 –0.017	0.017	1, 2

* References:
1. Horton, R. E., Some Better Kutter's Formula Coefficients, *Eng. News, 75:* 373–374 (1916).
2. King, H. W., *Handbook of Hydraulics*, 3rd edition, McGraw-Hill Book Co., New York, 1939.
3. Ramser, C. E., Flow of Water in Drainage Channels, *U. S. Dept. Agr. Tech. Bull. 149* (1929).
4. Ree, W. O., and V. J. Palmer, Flow of Water in Channels Protected by Vegetative Linings, *U. S. Dept. Agr. Tech. Bull. 967* (1949).
5. Scobey, F. C., The Flow of Water in Flumes, *U. S. Dept. Agr. Tech. Bull. 393* (1933).
6. Straub, L. G., and H. M. Morris, Hydraulic Tests on Corrugated Metal Culvert Pipes, *Univ. Minn. St. Anthony Falls Hydraulic Lab. Tech. Paper 5*, Series B (1950).
7. Yarnell, D. L., and S. M. Woodward, The Flow of Water in Drain Tile, *U. S. Dept. Agr. Bull. 854* (1920).

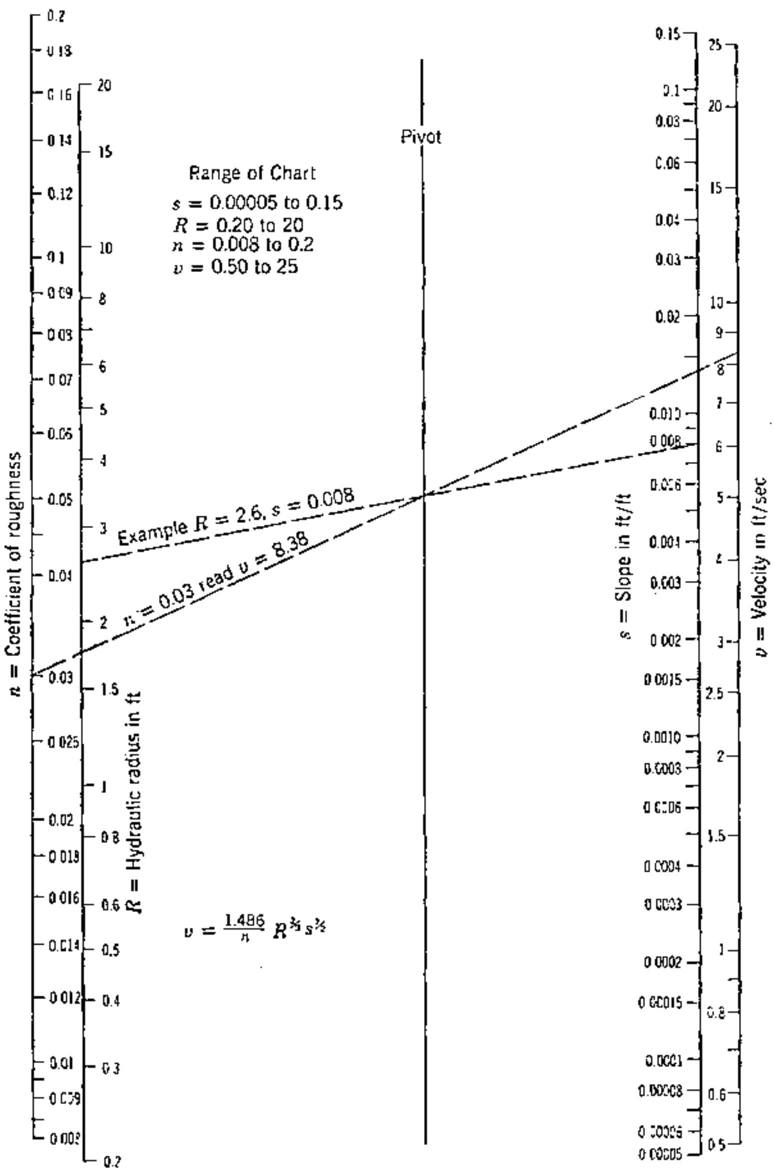

Fig. C.1. Nomograph for solving the Manning formula. (Redrawn from U. S. Soil Conservation Service, *Engineering Handbook, Hydraulics Section 5*, 1951.)

Example C.1. Design an open ditch to carry 156 cfs on a slope of 0.09 per cent, assuming ditch side slopes of 2:1, bottom width of 4 feet, and a clean, straight channel with no rifts or deep pools.

Solution. From Table C.1, select a roughness coefficient of 0.03. Make a trial solution, using a depth of 4 feet. From Fig. 9.1

$$R = \frac{bd + zd^2}{b + 2d(z^2 + 1)^{1/2}} = \frac{16 + 32}{4 + 8(5)^{1/2}} = 2.19$$

From Fig. C.1, read $v = 2.5$ fps or substitute in the Manning formula, $v = (1.486/n)R^{2/3}s^{1/2}$, and calculate.

$$Q = av = 48 \times 2.5 = 120 \text{ cfs (36 cfs too low)}$$

Next try $d = 4.5$ feet, from which $R = 2.42$, and determine $v = 2.68$ fps.

$$Q = 58.5 \times 2.68 = 156 \text{ cfs}$$

which agrees with the desired capacity.

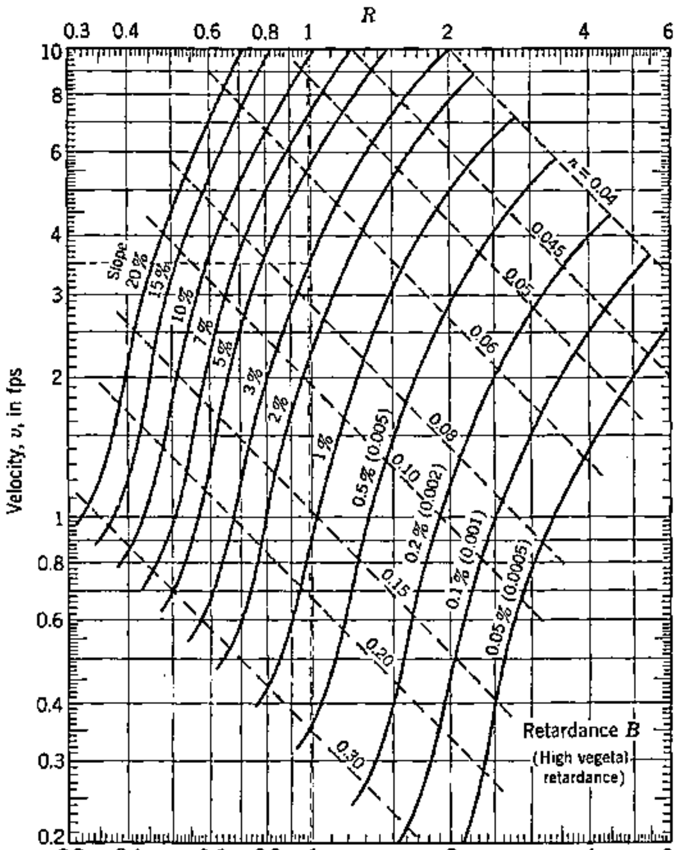

Fig. C.2. Solution of the Manning formula for vegetal-lined channels of retardance Class B (high vegetal retardance). From U. S. Soil Conservation Service, *Handbook of Channel Design for Soil and Water Conservation, SCS-TP-61* (1947).

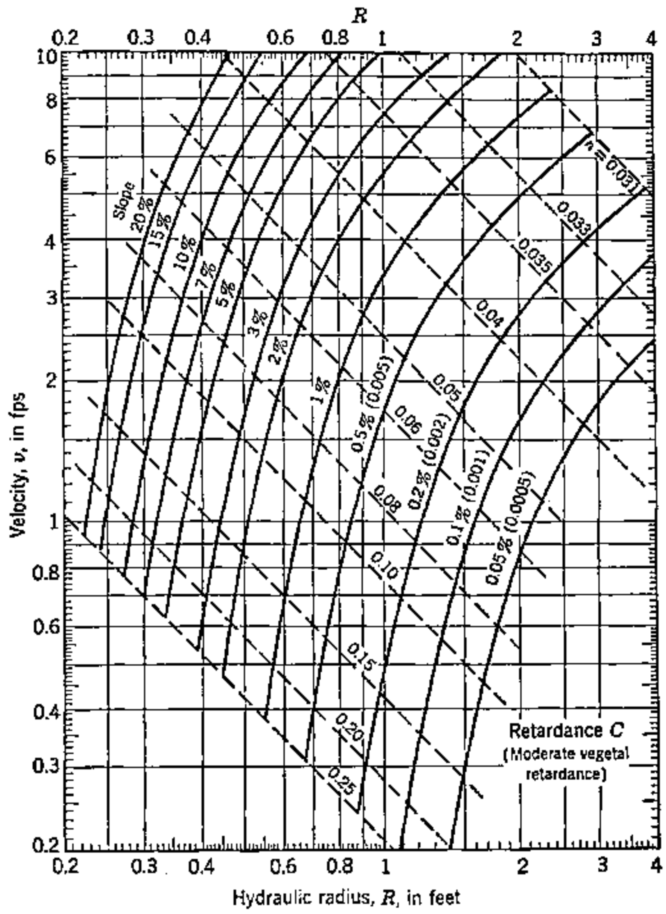

Fig. C.3. Solution of the Manning formula for vegetal-lined channels of retardance Class C (moderate vegetal retardance). (From reference in Fig. C.2.)

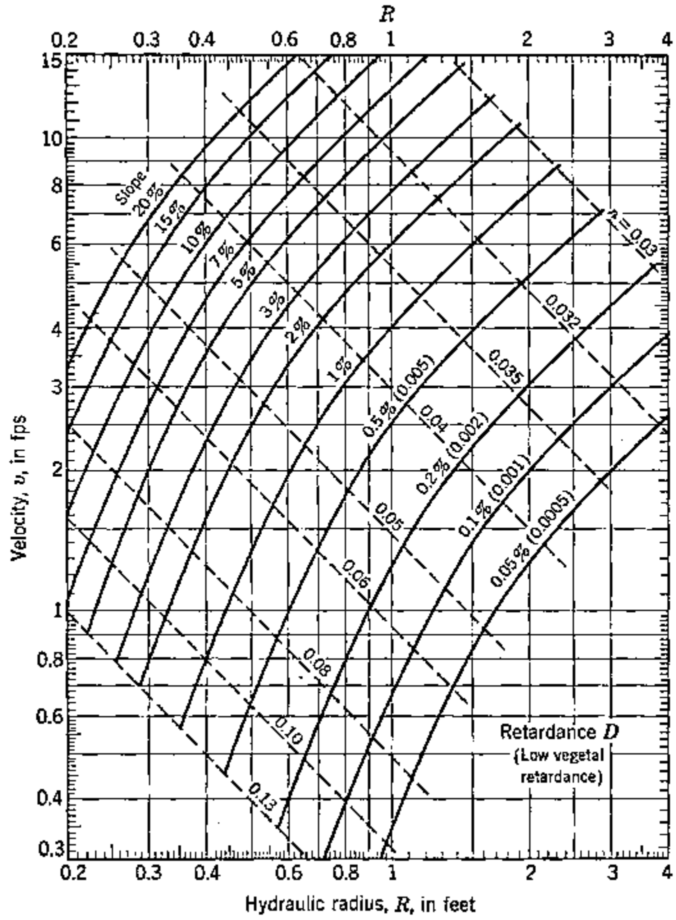

Fig. C.4. Solution of the Manning formula for vegetal-lined channels of retardance Class D (low vegetal retardance). (From reference in Fig. C.2.)

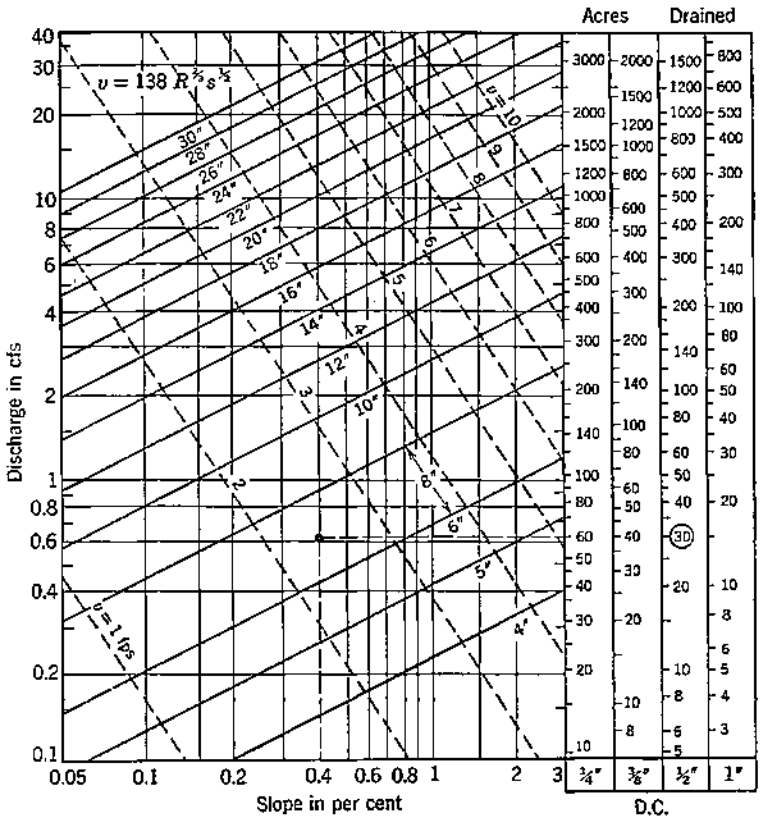

Fig. C.5. Chart for determining the size of tile drains. [Redrawn from D. L. Yarnell and S. M. Woodward, The Flow of Water in Drain Tile, *U. S. Dept. Agr. Bull. 854* (1920).]

Example C.2. Determine the diameter of a tile main on a grade of 0.4 per cent, draining 30 acres with a drainage coefficient of ½ inch. *Solution.* From Fig. C.5, the intersection of a horizontal line through 30 acres having a D.C. of ½ inch and a vertical line through a grade of 0.4 per cent indicates that an 8-inch tile is required.

APPENDIX D

Pipe and Conduit Flow

Table D.1 HEAD LOSS COEFFICIENTS FOR CIRCULAR CONDUITS FLOWING FULL*

$$K_c = \frac{5087n^2}{d_i^{4/3}}$$

Pipe Diam, in	Flow Area, sq ft	Manning Coefficient of Roughness n															
		0.010	0.011	0.012	0.013	0.014	0.015	0.016	0.017	0.018	0.019	0.020	0.021	0.022	0.023	0.024	0.025
6	0.196	0.0467	0.0565	0.0672	0.0789	0.0914	0.1050	0.1194	0.1348	0.151	0.168	0.187	0.206	0.226	0.247	0.269	0.292
8	0.349	.0318	.0385	.0458	.0537	.0623	.0715	.0814	.0919	.1030	.1148	.1272	.140	.154	.168	.183	.199
10	0.545	.0236	.0286	.0340	.0399	.0463	.0531	.0604	.0682	.0765	.0852	.0944	.1041	.1143	.1249	.136	.148
12	0.785	.0185	.0224	.0267	.0313	.0363	.0417	.0474	.0535	.0600	.0668	.0741	.0817	.0896	.0980	.1067	.1157
14	1.069	.0151	.0182	.0217	.0255	.0295	.0339	.0386	.0436	.0488	.0544	.0603	.0665	.0730	.0798	.0868	.0942
15	1.23	.0138	.0166	.0198	.0232	.0270	.0309	.0352	.0397	.0446	.0496	.0550	.0606	.0666	.0727	.0792	.0859
16	1.40	.0126	.0153	.0182	.0213	.0247	.0284	.0323	.0365	.0409	.0455	.0505	.0556	.0611	.0667	.0727	.0789
18	1.77	.01078	.0130	.0155	.0182	.0211	.0243	.0276	.0312	.0349	.0389	.0431	.0476	.0522	.0570	.0621	.0674
21	2.41	.00878	.01062	.0126	.0148	.0172	.0198	.0225	.0254	.0284	.0317	.0351	.0387	.0425	.0464	.0506	.0549
24	3.14	.00735	.00889	.01058	.0124	.0144	.0165	.0188	.0212	.0238	.0265	.0294	.0324	.0356	.0389	.0423	.0459
27	3.98	.00628	.00760	.00904	.01061	.0123	.0141	.0161	.0181	.0203	.0227	.0251	.0277	.0304	.0332	.0362	.0393
30	4.91	.00546	.00660	.00786	.00922	.01070	.01228	.0140	.0158	.0177	.0197	.0218	.0241	.0264	.0289	.0314	.0341
36	7.07	.00428	.00518	.00616	.00723	.00839	.00963	.01096	.0124	.0139	.0154	.0171	.0189	.0207	.0226	.0246	.0267
42	9.62	.00348	.00422	.00502	.00589	.00683	.00784	.00892	.01007	.01129	.0126	.0139	.0154	.0169	.0184	.0201	.0218
48	12.57	.00292	.00353	.00420	.00493	.00572	.00656	.00747	.00843	.00945	.01053	.01166	.0129	.0141	.0154	.0168	.0182
54	15.90	.00249	.00302	.00359	.00421	.00488	.00561	.00638	.00720	.00808	.00900	.00997	.01099	.0121	.0132	.0144	.0156
60	19.63	.00217	.00262	.00312	.00366	.00424	.00487	.00554	.00626	.00702	.00782	.00866	.00955	.01048	.0115	.0125	.0135

* From U. S. Soil Conservation Service, *Engineering Handbook, Hydraulics Section 5*, 1951.

Table D.2 HEAD LOSS COEFFICIENTS FOR SQUARE CONDUITS FLOWING FULL *

$$K_c = \frac{29.16n^2}{R^{4/3}}$$

Conduit Size, ft	Flow Area, sq ft	Manning Coefficient of Roughness n				
		0.012	0.013	0.014	0.015	0.016
2 × 2	4.00	0.01058	0.01242	0.01440	0.01653	0.01880
2½ × 2½	6.25	.00786	.00922	.01070	.01228	.01397
3 × 3	9.00	.00616	.00723	.00839	.00963	.01096
3½ × 3½	12.25	.00502	.00589	.00683	.00784	.00892
4 × 4	16.00	.00420	.00493	.00572	.00656	.00746
4½ × 4½	20.25	.00359	.00421	.00488	.00561	.00638
5 × 5	25.00	.00312	.00366	.00425	.00487	.00554
5½ × 5½	30.25	.00275	.00322	.00374	.00429	.00488
6 × 6	36.00	.00245	.00287	.00333	.00382	.00435
6½ × 6½	42.25	.00220	.00258	.00299	.00343	.00391
7 × 7	49.00	.00199	.00234	.00271	.00311	.00354
7½ × 7½	56.25	.00182	.00213	.00247	.00284	.00323
8 × 8	64.00	.00167	.00196	.00227	.00260	.00296
8½ × 8½	72.25	.00154	.00180	.00209	.00240	.00273
9 × 9	81.00	.00142	.00167	.00194	.00223	.00253
9½ × 9½	90.25	.00133	.00156	.00180	.00207	.00236
10 × 10	100.00	.00124	.00145	.00168	.00193	.00220

* From U. S. Soil Conservation Service, *Engineering Handbook, Hydraulics Section 5*, 1951.

Example D.1. Compute the head loss in 300 feet of 24-inch-diameter concrete pipe flowing full and discharging 30 cfs. Assume $n = 0.015$.

Solution. $v = Q/a = 30/3.14 = 9.55$ fps; $v^2/2g = (9.55)^2/64.4 = 1.42$ feet

$$H_f = K_c L\,(v^2/2g) = 0.0165 \times 300 \times 1.42 = 7.03 \text{ feet}$$

Example D.2. Compute the discharge of a 250-foot, 3x3 square conduit flowing full if the loss of head is determined to be 2.25 feet. Assume $n = 0.014$.

Solution. $v^2/2g = H_f/K_c L = 2.25/(0.00839 \times 250) = 1.073$ feet

$$v = (64.4 \times 1.073)^{1/2} = 8.31;\; Q = 9 \times 8.31 = 74.8 \text{ cfs}$$

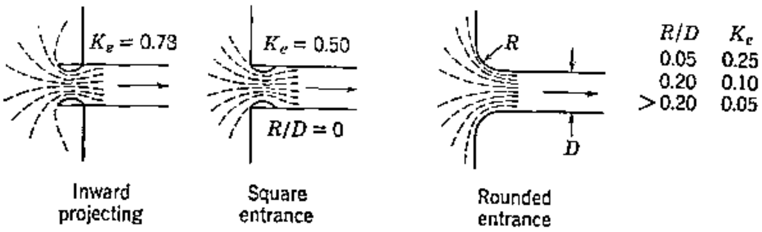

Fig. D.1. Entrance loss coefficients for conduits. [From U. S. Soil Conservation Service, *Engineering Handbook, Hydraulics Section 5*, 1951, and F. T. Mavis, The Hydraulics of Culverts, *Penn. Eng. Expt. Sta. Bull. 56* (1943).]

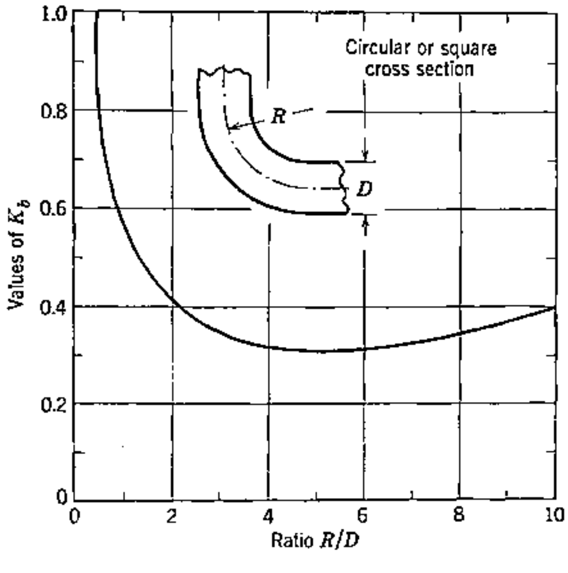

Fig. D.2. Friction loss coefficients at bends, K_b. (From U. S. Soil Conservation Service, *Engineering Handbook, Hydraulics Section 5*, 1951.)

APPENDIX E

Drain Tile Specifications*

E.1. Classes of Drain Tile. These specifications cover two classes of drain tile: *standard* for tile laid in trenches of moderate depths and widths and *extra-quality* for tile laid in trenches of considerable depths or widths, or both. Drain tile subject to these specifications may be made of shale; fire clays; surface clays, suitably burned; or Portland cement.

E.2. Chemical Requirements and Tests. The purchaser may specify special requirements for resistance of drain tile to damage where soils or drainage waters are markedly acid (*p*H of 6.0 or lower) or where they contain unusual quantities of soil sulfates, chiefly sodium or magnesium, singly or in combination (assumed to be 3000 ppm or more).

E.3. Physical Requirements and Tests. *Strength and Absorption.* Strength and absorption requirements for tile up to 24 inches in diameter are given in Table E.1.

Table E.1 PHYSICAL REQUIREMENTS FOR DRAIN TILE

Tile Diameter, in	Standard			Extra-Quality		
	Supporting* Strength, lb per ft	Max. Absorption, %		Supporting* Strength, lb per ft	Max. Absorption, %	
		Clay Tile	Concrete Tile		Clay Tile	Concrete Tile
4–12	800	13	10	1100	11	8
15	870	13	10	1100	11	8
18	930	13	10	1200	11	8
21	1000	13	10	1400	11	8
24	1130	13	10	1600	11	8

* Three-edge bearing method.

Size and Minimum Lengths. The nominal sizes of drain tile shall be designated by their inside diameter. Tile less than 12 inches in diameter shall be not less than 1 foot in length; 12- to 30-inch tile, not less than their diameter; and 30-inch tile or larger, not less than 30 inches in length.

Other Physical Properties. Some of the general physical requirements for the two classes of drain tile are given in Table E.2. Drain tile while dry

* Condensed from ASTM Standard Specifications for Drain Tile, *ASTM Designation C4-55* (1955).

shall give a clear ring when stood on end and tapped with a light hammer. They shall also be reasonably smooth on the inside. Drain tile shall be free from cracks and checks extending into the tile in such a manner as to decrease its strength appreciably. They shall be neither chipped nor broken so as to decrease their strength materially or to admit soil into the drain.

Table E.2 Distinctive General Physical Properties of Drain Tile

Physical Properties Specified	*Standard*	*Extra-Quality*
Number of freezings and thawings (reversals)	36	48
Permissible variation of average diameter below specified diameter, per cent	3	3
Permissible variation between maximum and minimum diameters of same tile, percentage of thickness of wall	75	65
Permissible variation of average length below specified length, per cent	3	3
Permissible variation from straightness, percentage of length	3	3
Permissible thickness of exterior blisters, lumps, and flakes which do not weaken tile and are few in number, percentage of thickness of wall	20	15
Permissible diameters of above blisters, lumps, and flakes, percentage of inside diameter	15	10
General inspection	Rigid	Very rigid

APPENDIX F

Earth-Moving Rates

Table F.1 DRAGLINE CAPACITY AND OPTIMUM DEPTH OF CUT *

Class of Material	Bucket Size, cu. yd								
	3/8	1/2	3/4	1	1 1/4	1 1/2	1 3/4	2	2 1/2
Light moist clay or loam	5.0†	5.5	6.0	6.6	7.0	7.4	7.7	8.0	8.5
	70‡	95	130	160	195	220	245	265	305
Sand or gravel	5.0†	5.5	6.0	6.6	7.0	7.4	7.7	8.0	8.5
	65‡	90	125	155	185	210	235	255	295
Good common earth	6.0†	6.7	7.4	8.0	8.5	9.0	9.5	9.9	10.5
	55‡	75	105	135	165	190	210	230	265
Clay, hard, tough	7.3†	8.0	8.7	9.3	10.0	10.7	11.3	11.8	12.3
	35‡	55	90	110	135	160	180	195	230
Clay, wet, sticky	7.3†	8.0	8.7	9.3	10.0	10.7	11.3	11.8	12.3
	20‡	30	55	75	95	110	130	145	175

* From Power Crane and Shovel Association, Proper Sizing of Excavators and Hauling Equipment, *Tech. Bull. 3* (1949).

† Upper line is optimum depth of cut in feet.

‡ Lower line is the capacity in cubic yards per hour for grade level loading and 90 degree swing.

Table F.2 RATE OF TRENCH EXCAVATION FOR WHEEL-TYPE MACHINES *

Depth, ft	Rate of excavation†	
	Continuous Operation fph	Average Footage per 10-Hour Working Day‡
3.0	430	1290
3.5	370	1110
4.0	310	930
4.5	260	780
5.0	200	600

* Based on data from L. L. DeVries. Performance and Operating Costs of Tile Trenching Machines, Unpublished M. S. Thesis, Iowa State College Library, Ames, Iowa, 1951.

† Average soil conditions and trench width sufficient for 4-, 5-, or 6-inch tile.

‡ Includes 70 per cent time lost due to weather, repairs and servicing, junctions, moving machines, and miscellaneous.

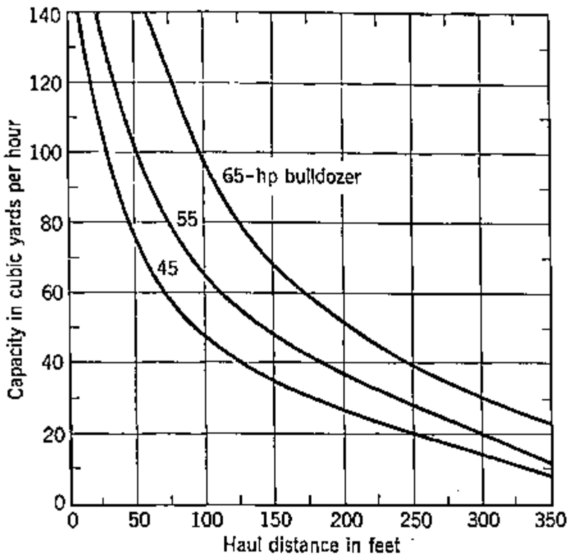

Fig. F.1. Capacity of bulldozers for various lengths of haul. (Redrawn from U. S. Soil Conservation Service, *Engineering Handbook*, Chapter VI, Conservation Irrigation, Pacific Region VII, Portland, Oregon.)

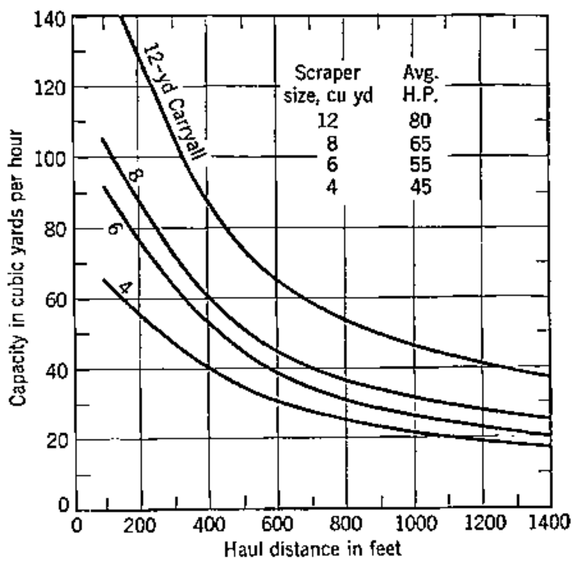

Fig. F.2. Capacity of wheeled scrapers for various lengths of haul. (Redrawn from reference in Fig. F.1.)

Table F.3 Rate of Backfilling Narrow Trenches

Equipment	*Lineal Feet per Hour**
Tractor (2-bottom plow) with manure loader	300–500
Hoe with 3-foot blade	500–750
Bulldozer (45 hp)	750–1000

* Based on assumption of 25 per cent time loss.

Table F.4 Rate of Construction for Shallow Surface Ditches *

Machine	*Cubic Yards per Hour*
Average Depth of Cut 1 Foot or Less	
Disk plow (2-blade)	50–125
Disk terracer (1-blade)	25–75
Whirlwind terracer†	30–100
Moldboard plow (2–14″) and utility blade	25–50
Average Depth of Cut 1–2.5 Feet	
Whirlwind terracer†	30–75
Bulldozer 45 hp (from Fig. F.1 for 100-ft haul)	45
Scraper 4 cu yd (from Fig. F.2 for 100-ft haul)	65

* Based on data from K. K. Barnes, Improvement of Surface Drainage in Iowa, Unpublished M. S. Thesis, Iowa State College Library, Ames, Iowa, 1948.

† Equipped with a special ditching rotor.

Table F.5 Average Rates for Clearing Brush *

Operation	*Acres per Hour*
Knocking down	0.8 –1.0
Piling brush	1.25–1.5
Plowing or root raking	1.25–1.5
Piling roots	2.0

* From R. A. Hall, Brush Control with Heavy Machinery, *Agr. Eng.*, *27:* 458 (1946).

APPENDIX G

Loads on Underground Conduits

Underground conduits should be installed such that the load does not exceed the required minimum average crushing strength. For clay and concrete tile, the required strength is set forth in *ASTM C4-50T,* Tentative Specifications for Drain Tile, 1950. The nomograph shown in Fig. G.1 is based on Marston's formulas and is presented to simplify the computation of loads on conduits embedded in thoroughly wet clay. From the figure, loads can be computed for either ditch or projecting conditions. Loads so calculated include a factor of safety of 1.5. Maximum allowable depth for a conduit of specified strength may also be determined. The following notation applies:

D = depth of trench to bottom of tile.
H = depth of trench to top of tile.
B_c = outside diameter of tile.
B_d = width of trench measured at the top of the tile.
W_c = load on tile in pounds per linear foot.
w = weight of soil in pounds per cubic foot.

Example G.1. Determine the load on a 10-inch tile installed 8.5 feet deep in a ditch 18 inches wide in ordinary clay (Fig. G.1).

Solution. Projecting conduit

$$\frac{H}{B_c} = \frac{D - B_c}{B_c} = \frac{8.5 - 1.0}{1.0} = 7.5$$

Place straight edge on nomograph on points $H/B_c = 7.5$ and $B_c = 1.0$, and read on right line in Fig. G.1, $W_c = 2470$ lb/lin ft

Ditch conduit

$$\frac{H}{B_d} = \frac{D - B_c}{B_d} = \frac{8.5 - 1.0}{1.5} = 5.0$$

Read on right line $W_c = 1130$ lb/lin ft. The lower value, 1130 lb/lin ft, is the design load to be used.

Example G.2. Determine the allowable depth to install 10-inch standard quality tile (strength 1200 lb/lin ft) in a trench 18 inches wide in ordinary clay, assuming ordinary bedding conditions (L.F. 1.5).

Solution. Projecting conduit

Using $B_c = 1.0$ and $W_c = 1200$, read on left line $H/B_c = 3.8$.

$$H = 3.8 \times 1.0 = 3.8$$

$$D = H + B_c = 3.8 + 1.0 = 4.8 \text{ feet}$$

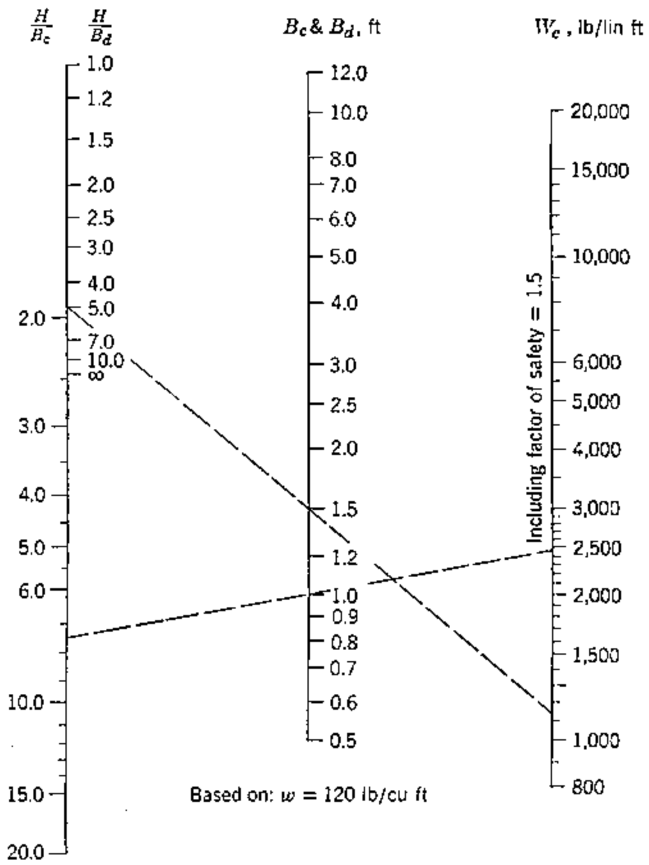

Fig. G.1. Nomograph for loads on conduits installed in ordinary clay. [From J. van Schilfgaarde and others, Effect of Present Installation Practices on Draintile Loading, *Agr. Eng.* *32:* 371–374, 378 (1951).]

Ditch conduit

Using $B_d = 1.5$ and $W_c = 1200$, read on left line $H/B_d = 5.5$.

$$H = 5.5 \times 1.5 = 8.2$$

$$D = H + B_c = 8.2 + 1.0 = 9.2 \text{ feet}$$

Since this example is the reverse of the previous case, the greater depth, 9.2 feet is the maximum permitted.

Example G.3. Determine the average load per lineal foot and the total load transmitted to 24-inch drain tile installed at a depth of 5.4 feet from a static concentrated load of 1000 pounds directly over the center of the tile.

Solution. Depth to top of conduit = depth to bottom of tile $- B_c =$

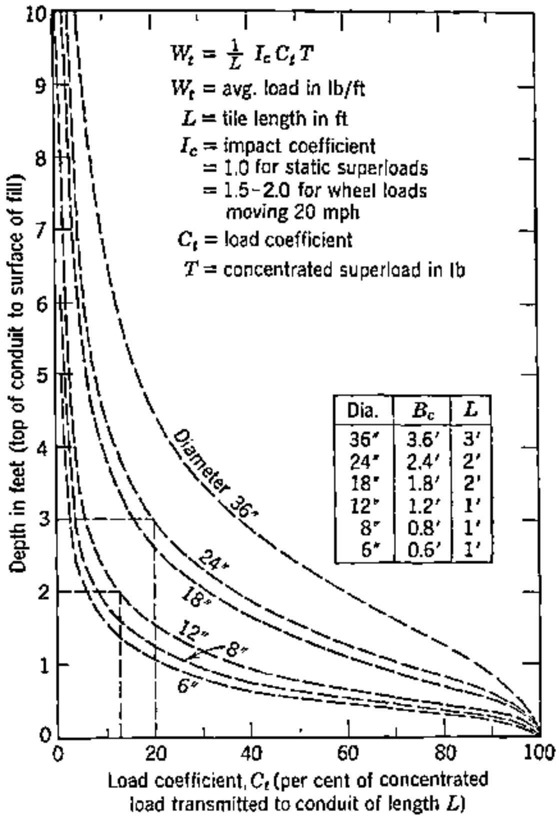

Fig. G.2. Concentrated surface load coefficients. [Based on investigations by M. G. Spangler and others, Experimental Determinations of Static and Impact Loads Transmitted to Culverts, *Iowa Eng. Expt. Sta. Bull. 79* (1926), and A. Marston, The Theory of External Loads on Closed Conduits in the Light of the Latest Experiments, *Iowa Eng. Expt. Sta. Bull. 96* (1930).]

$5.4 - 2.4 = 3.0$ feet. From Fig. G.2, read $(100 \times C_t) = 20$ per cent and

$$W_t = (1/2) \times 1.0 \times 0.20 \times 1000 = 100 \text{ lb/lin ft}$$

$$\text{Total load} = LW_t = 2 \times 100 = 200 \text{ pounds}$$

Example G.4. Determine the average load per lineal foot on a 12-inch tile installed at a depth of 3.2 feet if a 1000-pound concentrated load is moving at 20 mph.

Solution. From Fig. G.2 for a depth of 2.0 feet $(3.2 - 1.2)$, read $(100 \times C_t) = 13$ per cent, and select $I_c = 2.0$.

$$W_t = (1/1) \times 2.0 \times 0.13 \times 1000 = 260 \text{ lb/lin ft}$$

APPENDIX H

Conversion Constants

Table H.1 CONVERSION OF RUNOFF UNITS

Drainage Coefficient, in.	Cfs per Acre	Gpm per Acre	Cfs per Sq Mile	Gpm per Sq Mile
1/16	0.0026	1.18	1.68	754
1/8	0.0052	2.36	3.36	1,508
1/4	0.0105	4.71	6.72	3,017
3/8	0.0157	7.07	10.08	4,525
1/2	0.0210	9.43	13.44	6,034
5/8	0.0262	11.79	16.80	7,542
3/4	0.0315	14.14	20.16	9,051
7/8	0.0367	16.50	23.52	10,559
1	0.0420	18.86	26.88	12,068

Table H.2 CONVERSION OF VOLUME UNITS

Unit	Cubic Inches	Gallons	Cubic Feet	Cubic Yards	Acre-Feet
Cubic inch	1	0.00433	5.79×10^{-4}	2.14×10^{-5}	1.33×10^{-8}
Gallon	231	1	0.134	0.00495	3.07×10^{-6}
Cubic foot	1,728	7.48	1	0.0370	2.30×10^{-5}
Cubic yard	46,656	202	27	1	6.20×10^{-4}
Acre-foot	7.53×10^{7}	3.26×10^{5}	43,560	1,610	1

Table H.3 CONVERSION OF MAP SCALE UNITS *

Scale	*Feet per Inch*	*Inches per 1000 Feet*	*Inches per Mile*	*Miles per Inch*	*Acres per Sq In.*	*Square Miles per Sq In.*
1:600	50.00	20.00	105.60	0.009	0.057	0.0009
1:1,200	100.00	10.00	52.80	0.019	0.230	0.00036
1:2,400	200.00	5.00	26.40	0.038	0.918	0.0014
1:5,000	416.67	2.40	12.67	0.079	3.986	0.0062
1:7,920	660.00	1.515	8.00	0.125	10.000	0.0156
1:12,000	1000.00	1.000	5.280	0.189	22.957	0.0359
1:24,000	2000.00	0.500	2.640	0.379	91.827	0.1435
1:62,500	5208.33	0.192	1.014	0.986	622.744	0.9730
1:63,360	5280.00	0.189	1.000	1.000	640.000	1.00
Formulas	$\frac{\text{Scale}}{12}$	$\frac{12{,}000}{\text{Scale}}$	$\frac{63{,}360}{\text{Scale}}$	$\frac{\text{Scale}}{63{,}360}$	$\frac{(\text{Scale})^2}{43{,}560\times 144}$	$\frac{(\text{Ft per in.})^2}{(5280)^2}$

* From U. S., B.P.I.S.A.E., Soils Survey Manual, *U. S. Dept. Agr. Handbook*, *18* (1951).

Table H.4 MISCELLANEOUS CONVERSION CONSTANTS

Volume—Weight:

1 gal of water = 8.34 lb
1 cu ft of water = 62.4 lb

Pressure:

1 atmosphere = 14.7 psi
= 76 cm mercury
= 29.92 in. mercury
= 33.93 ft water
1 psi = 2.31 ft water
1 ft of water = 0.434 psi

Power:

1 horsepower = 550 ft-lb per sec
= 746 watts
1 kilowatt (kw) = 1.341 horsepower (hp)

APPENDIX I

Useful Formulas and Procedures

VOLUME FORMULAS

The average end area formula for computing the volume of storage in a reservoir is:

$$V = \frac{d}{2}(A_1 + A_2) \tag{I.1}$$

where V = volume of storage in acre-feet.
d = vertical distance between end areas in feet.
A_1 and A_2 = end area in acres.

The prismoidal formula is:

$$V = \frac{d}{6}(A_1 + 4A_m + A_2) \tag{I.2}$$

where A_m = middle area in acres halfway between the end areas.

Where preliminary surveys are made by taking slopes in the reservoir area, the storage may be estimated from the approximate formula for a frustrum of a cone,*

$$V = A_0 d + \frac{0.85 d^2 A_0^{1/2}}{S} \tag{I.3}$$

where A_0 = area in acres at spillway crest.
d = depth of water above spillway crest.
S = average slope of reservoir banks, through range of d, in per cent.

LAYOUT OF CIRCULAR CURVES

The procedure to be followed in laying out a circular curve is as follows: The transit is first set up at the point of intersection (P.I.) as indicated in Fig. I.1 and the angle I is measured. Next calculate the tangent distance by the formula:

$$T = R \tan \frac{I}{2} \tag{I.4}$$

where T = the tangent distance in feet.
I = the intersection angle in degrees.

The point of curvature P.C. and the point of tangency P.T. are located by measuring the computed distance T from the P.I. Set up the transit at P.C. (sta. 3 + 10), and locate stations on the curve by chaining and measuring

* M. M. Culp, The Effect of Spillway Storage on the Design of Upstream Reservoirs, *Agr. Eng.*, *29*: 344–346 (1948).

off deflection angles as computed by the equation:

$$e = \frac{c}{100} \cdot \frac{D}{2} = \frac{cD}{200} \tag{I.5}$$

where e = deflection angle in degrees.
c = chord length in feet.
D = degree of curve.

From this equation 100-foot stations require deflection angles of $\frac{1}{2}D$, 50-foot stations $\frac{1}{4}D$, etc. After the first station beyond the P.C. is located, the deflection angle for each succeeding station is the summation of the

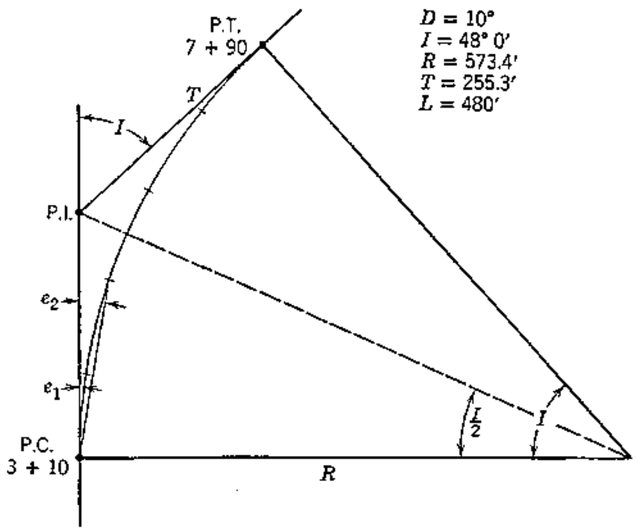

Fig. I.1. Layout procedure for a circular curve.

deflection angles for all previous chord distances. Since most curves are rather flat, the arc distance is nearly equal to the chord length for 50- or 100-foot stations. The total length of the curve is:

$$L = 100\frac{I}{D} \tag{I.6}$$

The design of a circular curve is illustrated by the following problem.

Example I.1. Design a 10-degree curve for Fig. I.1 if the angle I is 48° 0'.

Solution. From equation 15.3, $R = 50/\sin 5° = 573.6$ feet and, from equation I.4, $T = 573.6 \tan 24° = 255.4$ feet.

From equation I.5, the deflection angle to sta. 4 + 00 is: $e_1 = \frac{90 \times 10}{200} =$ 4.5°, and similarly e_2 for sta. 5 + 00 is 9.5° (4.5 + 5), etc.

From equation I.6, $L = \frac{100 \times 48}{10} = 480$ feet, and $310 + 480 = 790$ or P.T. sta. $7 + 90$.

SETTING SLOPE STAKES

In making the location survey prior to construction of a dam or a ditch, center line stakes and slope stakes are set at each station or at more frequent intervals to guide the operator. On level or nearly level topography the offset of the slope stakes from the center line can be easily computed by adding one-half the top or bottom width plus the side slope ratio (z) times the depth. On irregular land the slope stakes are set by trial

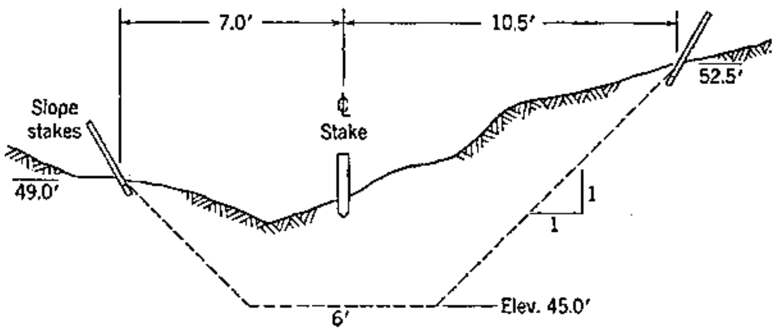

Fig. I.2. Setting slope stakes on uneven ground.

and error. Although the following procedure, illustrated in Fig. I.2, applies to ditch location, the same method is applicable to earth dam construction. First, the offset distance from the center line is estimated and an elevation for the point determined. If the depth from this point to the bottom of the ditch corresponds to the computed distance to the center line, the slope stake has been set correctly. For example, in Fig. I.2 the slope stake is at an elevation of 52.5. The depth to the bottom of the ditch at this point is 7.5 feet, and the computed distance is 10.5 feet from the center line. If the measured distance is 10.5, the stake is set correctly. However, if this distance is not 10.5, a new trial point must be selected, the elevation determined, and the distance from the center line again compared to the computed distance.

APPENDIX J

Gravel Filters*

Experiments have shown that practically no impregnation of coarse material by fine material will take place if, when the flow is, for example, down through a filter and each successive layer of material is composed of particles such that for the 15 per cent size (15 per cent smaller than and 85 per cent larger than) the diameter is 9 times that of the 15 per cent

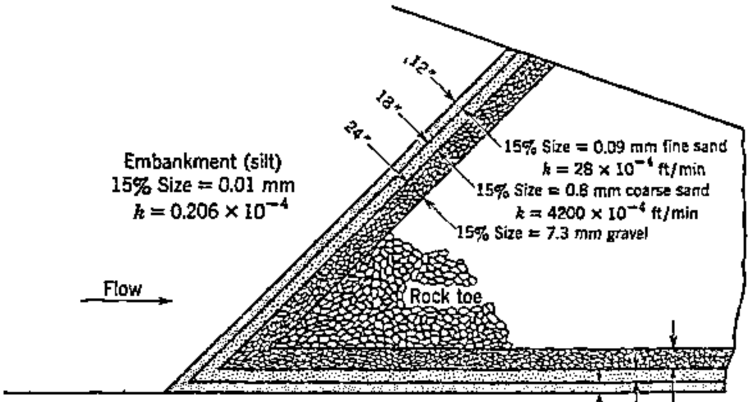

Fig. J.1. Example of a filter to protect a silt embankment. (Based on Bertram's ratio for particle sizes. From Justin and others, *ibid.*)

size of the layer above, assuming that the material is at least 50 per cent compacted.

Figure J.1 illustrates an example of this ratio in use. Beginning with an embankment of fine silt and increasing the diameter of the 15 per cent size with each successive layer by a ratio of 9, the accompanying table is developed:

Layer Number	*Thickness of Layer*	*Diameter of 15% Size*	*Approximate Permeability*
1. Embankment	Indefinite	0.01 mm	0.206×10^{-4} ft per min
2. Fine sand	12 in.	0.09 mm	28.0×10^{-4} ft per min
3. Coarse sand	18 in.	0.81 mm	4200×10^{-4} ft per min
4. Gravel	24 in.	7.3 mm	Not limiting

* J. D. Justin and others, *Engineering for Dams*, Vol. III, John Wiley & Sons, New York, 1945.

The above permeabilities are not exact but are close enough to what would be found by test to be indicative. Theoretically each layer of the filter could be very thin, but practically a reasonable thickness is necessary in order to make sure that some slight readjustment during or after construction does not rupture the layer. As a practical matter, the thickness of each filter should be at least 50 times the diameter of the 15 per cent size and no sand layer should be less than 12 inches thick.

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