



PDHonline Course C297 (6 PDH)

Drainage Engineering and the Rational Method

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Drainage Engineering and the Rational Method

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Course Content

COURSE OVERVIEW

This course will provide the basic skills and knowledge of drainage engineering, and help develop sound engineering judgement needed to solve drainage problems. The course will describe hydrology; calculate drainage areas, runoff, and runoff for successive areas; design open and closed channels; and control erosion; Describes the methods used to design drainage structures and will describe the proper procedures used to determine runoff using the Rational Method of runoff determination. Also this course covers the aspects of designing a culvert (closed channel) for a given situation. Design is based on environmental considerations such as slope, flow, and length of culvert needed, in addition to economic considerations. Often a drainage system will consist of a series of drainage areas, called successive areas, with upstream areas discharging runoff into lower areas. Due to this action, the runoff accumulates and increases as it passes through the system.

HYDROLOGY AND DRAINAGE AREAS

Drainage engineering is important to all areas. The entire serviceability of roads, buildings, and other structures depend on the adequacy of the drainage system. The objective of a drainage system is to minimize the effects of adverse weather conditions over the life of the project:

- Removing all surface water from project areas.
- Intercepting and disposing of runoff from adjoining areas.
- Intercepting and removing detrimental ground water.

Hydrology is the natural science that deals with the process governing the depletion and the replenishment of water resources on and beneath the surface of the earth. knowledge of the principles of hydrology is necessary to be able to estimate the rate of runoff in a given area.

The hydrologic cycle has seven main phases. The phases include precipitation (gain), interception, infiltration, evapotranspiration, detention, transpiration, and runoff (losses). Figure 1 shows the phases of the hydrologic cycle.

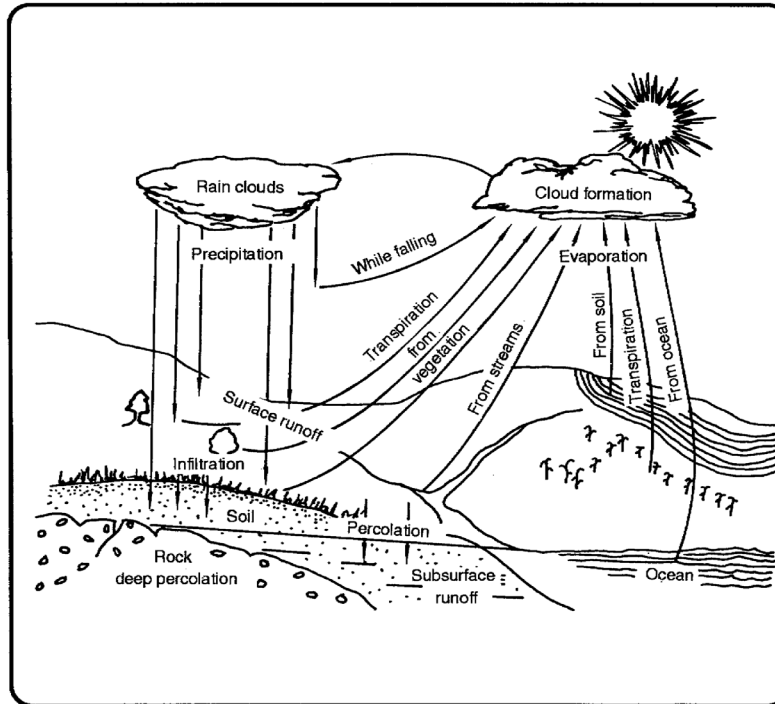


Figure 1. Phases of the hydrologic cycle

Precipitation or *rainfall* is considered the main moisture delivery mechanism. This is a primary concern to most drainage engineers. The Rational Method will be used to estimate the proportion of rainfall that actually becomes runoff. The total amount of rainfall is not considered as a whole, because one of the six natural losses, or a combination of losses, would subtract a portion of the total amount of rainfall. This amount would be considerably less than the total amount of precipitation. The losses of the hydrologic cycle are explained below:

Interception is the proportion of water that is caught by the leaves or the canopy of the trees. This amount of water would have no effect on the runoff estimate because it would not contribute to your project. The water that is caught by vegetation would stay in place until it is evaporated back into the air.

Infiltration is the proportion of water that sinks into the ground. This water may become part of the underground water found in aquifers, or a small part may be evaporated back into the hydrologic cycle. In many cases the losses from infiltration will be the largest.

Infiltration depends upon the following factors:

- The type and condition of the soil.
- The type of cover (grass, bushes, and trees) or bare soil.
- The general slope in the area.

Evapotranspiration is the combined loss of water from a given area for a time by evaporation from the soil surface and transpiration from plants. Evaporation is the portion of rainfall that is lost back into the atmosphere as a result of water changing into vapor. This vapor rises because it is lighter than air and becomes another phase in the hydrologic cycle.

Detention is the accumulation of water in low spots. An example of this would be a lake or a pond. From

the detention area, the water either infiltrates into the ground, or it evaporates back into the atmosphere.

Transpiration is the process whereby plants extract moisture from the soil by their roots. Eventually the plants will transpire part of the moisture back into the atmosphere as water vapor.

The amount of water that is left after these losses extract a portion of the total amount of rainfall is called *runoff*. Water left over after these losses will flow across the surface. Runoff is the amount of water that you would have to estimate in order to design drainage structures.

INTERPRETATION OF THE HYDROGRAPHY

Precipitation includes all forms of water which fall to the earth's surface; however, the rainstorm is the primary source of precipitation which concerns the engineer on drainage problems. Rainfall data for the locality being considered is important in designing a drainage system that will ensure removal of surface water and control of ground water. Most drainage design is based on peak runoff from a relatively small drainage area (less than a square mile). Such peak flows are ordinarily caused by short duration, high-intensity storms such as thunderstorms.

Figure 2 shows three bar charts illustrating the typical intensity/duration relationships in a short rainstorm, a medium rainstorm, and a long rainstorm. These relationships are generally consistent. They are based on data from a large number of weather stations. Notice that the shorter the storm, the greater the intensity. Stated another way--intensity and duration are inversely proportional. Now look at the hydrography shown in Figure 3.

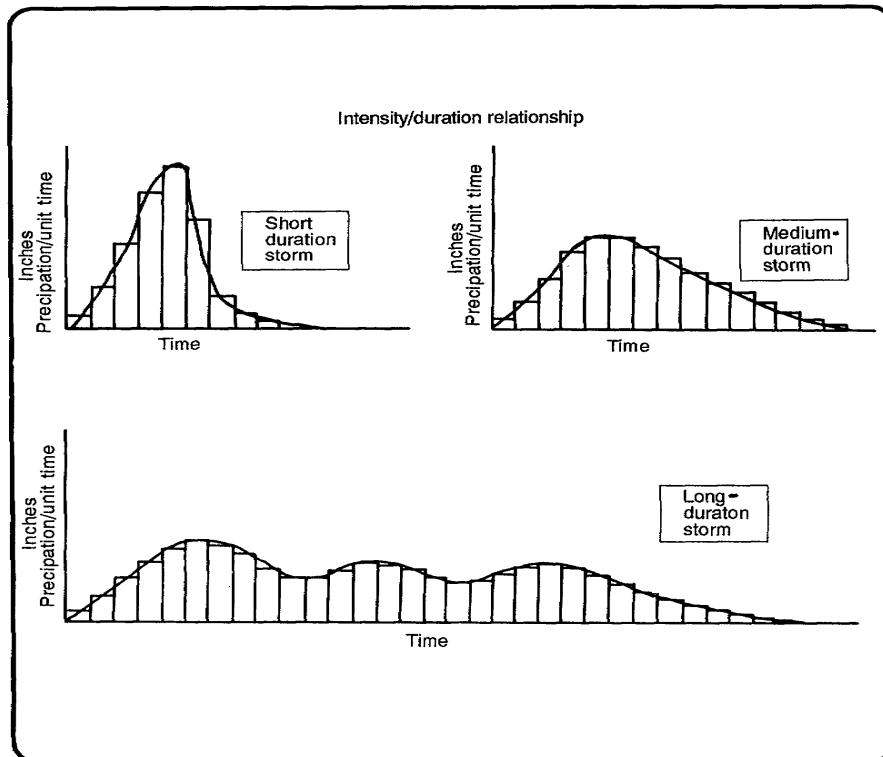


Figure 2. Bar charts of rainstorms

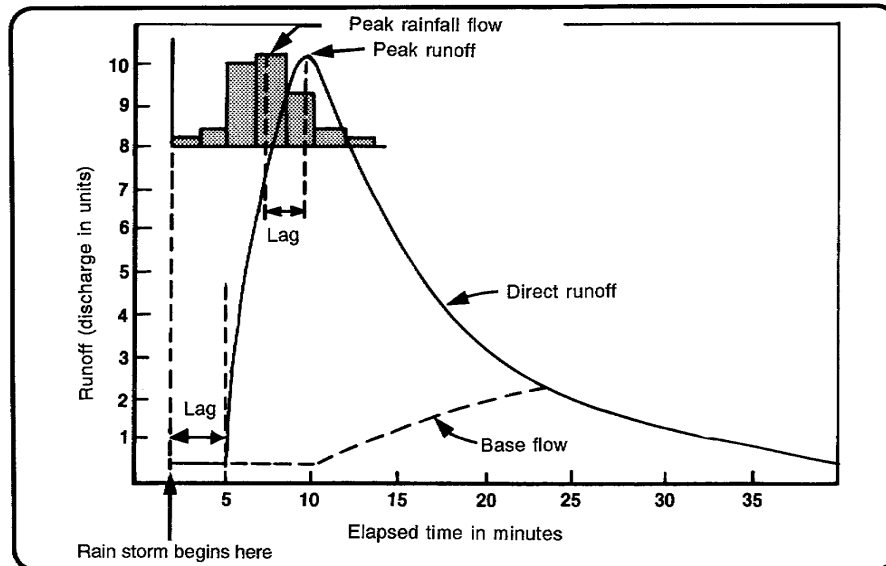


Figure 3. Hydrograph of a brief rainfall

The hydrograph illustrates the relationship between the duration and intensity of the runoff in a 15-minute rainstorm. Intensity is the rate at which the water is running in the drainage structure.

The bar graph of the intensity and duration of the rainstorm has been superimposed on the hydrograph. It illustrates that the peak runoff lags behind the peak flow. This lag is caused by factors affecting loss of water during rainfall and the size of the delineated drainage area. The time from the beginning of the storm until the point of peak runoff is called the *time of concentration (TOC)*.

Rainfall Intensity

The rainfall intensity used in the Rational Method is a measurement of the rate of precipitation expected to fall on your watershed area. This rainfall rate is based on historical records of rainfall rates compiled for a specific location. The rainfall intensity is the depth of water falling per unit of time, and it is expressed in inches per hour (in/hr). The following factors affect the rainfall intensity:

Drainage systems are built for a period of time. The longer the period of time, the larger and more expensive the design becomes. However, with the longer design comes greater protection. The time period that you design your culvert to last is called the *design storm* or the *return period of the storm*.

The rainfall intensity is based on storm frequency. This basically means that you will design a culvert based on the probable occurrence of a storm that will occur on an average of once during the design life of the structure. Engineers design structures to last a given amount of time depending on the consequences of a culvert failure and the costs of designing it for greater protection. For culverts on roads, highway departments usually use return periods of 10 to 50 years depending on the importance of the road. For example, a culvert designed for a ten year period is expected to fail on an average of once every ten years, since it has been designed to withstand a storm that will occur on an average of once every ten years.

Drainage structures are designed on the probable occurrence of a storm. Because larger storms occur less frequently in nature, the longer the period of design is used, the longer the protection afforded.

In general a ten year design life is usually assumed, however the local government or state may require a

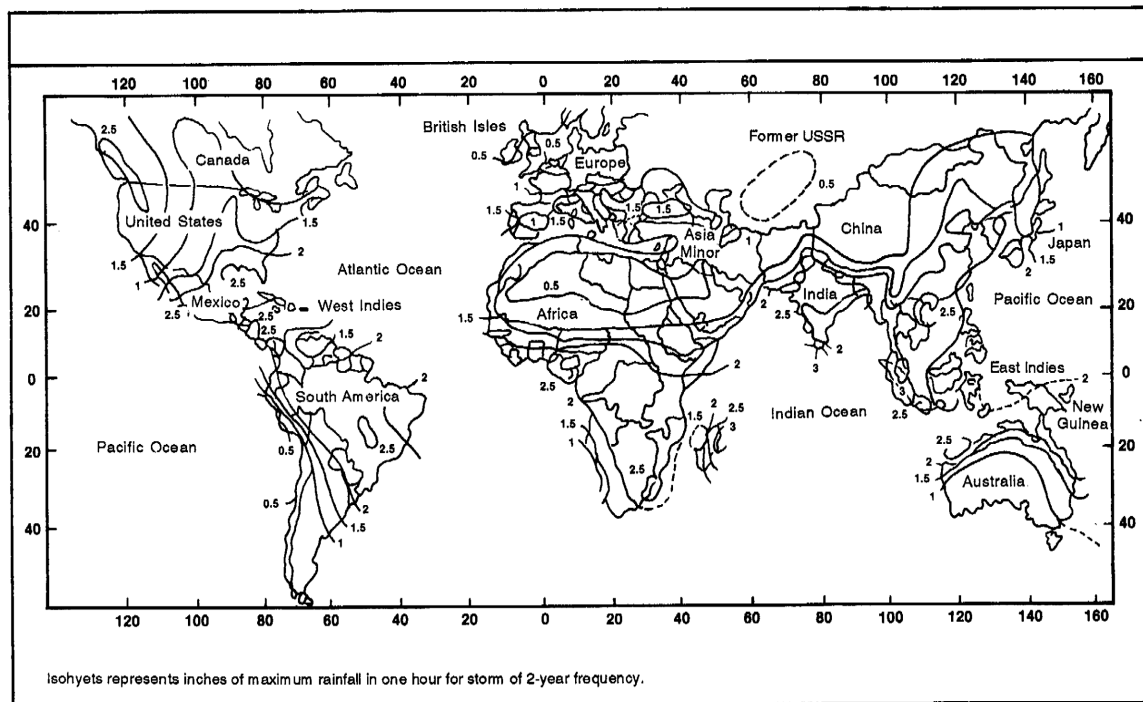
longer design life. In other words, the ten year storm select as the design storm. This basically means that the culvert should fail on average of once in its design life of ten years. It would be unreasonable to base a design on a 100-year storm, unless stated otherwise, since it could require more materials and a higher cost.

The characteristics of rainfall vary considerably from one part of the country to another. Nevada, Arkansas, and Oklahoma, do not receive as much rain as Washington State. Nor would one expect heavy rains in North Dakota in December.

The best source of data for determining the intensity to be used in design is local records (pinpoint data). If rainfall rates have been recorded for the project location, then information and the data should be obtainable. This information would come directly from the weather service or state where the project is located.

THE ISOHYETAL MAP

When pinpoint data is not available for any location in the vicinity of the project site, use the isohyetal map (see Figure 4). Isohyets are lines of equal intensities drawn through geographic locations on a topographic map. Just as a contour line represents equal elevations on a topographic map, isohyets represent equal intensities of rainfall. The isohyetal maps are based on a two-year, ten year, fifty year, and one hundred year design storm with a 60-minute duration.



To read a value on the isohyetal map, locate the project location and then read the value of the appropriate isohyet(s). Do not interpolate. Use the following rules when determining the rainfall intensity on the map:

- When your project location touches an isohyet, read the value of that isohyet.
- If your project location falls between two isohyets, use the larger value.

- Within an encircling isohyet, read the value of the encircling isohyet.

A few examples

Southern Australia	1.0 in/hr
North Dakota	1.5 in/hr
Florida	2.5 in/hr
Washington, D.C.	1.5 in/hr

DEPTH OF INTENSITY CONVERSIONS

The intensity of rainfall is always expressed in the units of in/hr. When listening to a weather report, it may be reported how much precipitation has fallen in a specified period of time, quite often the last 24 hours. This is the total amount of rain that fell. If it is stated that the total amount of rain that fell in the last 24 hours is 2 inches, there is no information on what the intensity, or rate of rainfall was. Did the rain fall evenly for 24 hours; in which case the rate would be approximately 0.083 in/hr (2 inches divided by 24 hours), or did the entire amount of rain fall in 30 minutes; in which case the intensity is 4 in/hr (2 inches divided by 0.5 hours). As can be seen, the rate can make a big difference. In the first case, there was a nice, gentle rain, while in the second case, there was a real “gully washer.” The gentle rain would cause little, if any, flooding; the heavy rain would probably cause severe ponding and flooding in some locations.

Using pinpoint data obtained for the Washington, D.C., area over a 50-year period, data from the worst storm (the 50-year storm) can be converted from depth to intensity as follows:

<u>Duration</u>	<u>Amount of Rainfall</u>		<u>Rate (in/hr)</u>
5 minutes	0.77” = (0.77”) x $\frac{(60 \text{ minutes})}{(5 \text{ minutes})}$	=	9.24 in/hr
10 minutes	1.21” = (1.21”) x $\frac{(60 \text{ minutes})}{(10 \text{ minutes})}$	=	7.26 in/hr
15 minutes	1.51” = (1.51”) x $\frac{(60 \text{ minutes})}{(15 \text{ minutes})}$	=	6.04 in/hr
30 minutes	2.43” = (2.43”) x $\frac{(60 \text{ minutes})}{(30 \text{ minutes})}$	=	4.86 in/hr
60 minutes	3.43” = (3.43”) x $\frac{(60 \text{ minutes})}{(60 \text{ minutes})}$	=	3.43 in/hr
120 minutes	3.91” = (3.91”) x $\frac{(60 \text{ minutes})}{(120 \text{ minutes})}$	=	1.96 in/hr

It’s to be noticed that as the duration increases, the intensity decreases. Intensity and duration are inversely proportional.

USING RAINFALL INTENSITY/DURATION CURVES

As shown in Figure 2 there is a fairly consistent relationship between the average intensity of rainfall for a period of one hour and the average rates of a comparable frequency of rainfall. Meteorologists have proven this statistically. The relationship between intensities at one duration and intensities of other durations within the same storm are expressed in Figure 5. With this chart you can adjust the rainfall duration of the 60-minute storm to a different duration. This adjustment will be needed later in the Rational Method of runoff determination.

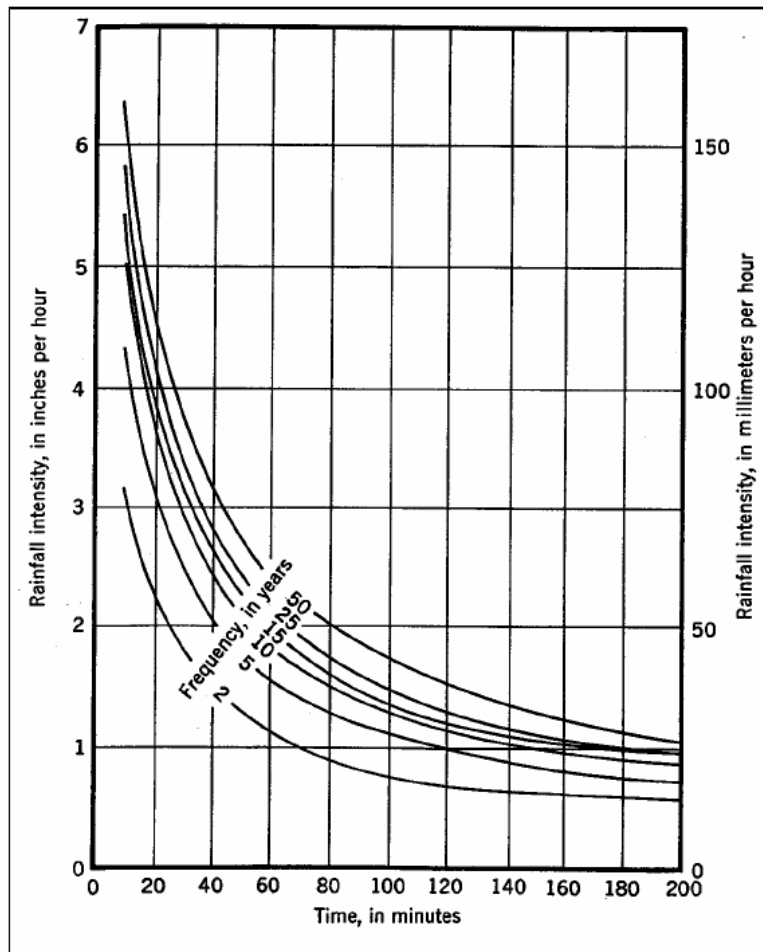


Figure 5. Standard rainfall intensity-duration curves

The examples of the intensities are for a 60-minute storm. This now has to be adjusted to a particular duration that is critical for the project under construction, similar to the adjustment made in the use of pinpoint data. Once the critical duration has been determined, the adjustment is made on the “Standard Intensity-Duration” curve which is given in Figure 5.

I_{60, 10 years}

Critical Duration

I_{adj}

1.0 in/hr
1.5 in/hr
2.0 in/hr

40 minutes
20 minutes
10 minutes

2.5 in/hr
3.8 in/hr
4.8 in/hr

DELINEATE A DRAINAGE AREA AND ESTIMATE THE ACREAGE

The terms *drainage basin*, *drainage area*, and *watershed*, are synonymous terms that are used interchangeably. A drainage basin is that portion of the earth surface that contributes water in a rainfall event to the drainage basin outlet--your drainage structure.

Delineation is the process of drawing a boundary that completely encompasses and defines the area of rainfall contribution to your drainage structure.

Once a drainage basin outlet has been selected, the process of delineation can be accomplished using the following steps:

- Locate all proposed or existing drainage structures.
- Locate all existing terrain high points.
- Draw flow lines away from high points. *Flow lines* are graphic representations drawn on a topographic map that indicates the direction of water flow as it crosses terrain. Several rules must be followed when drawing flow lines.
 - Flow lines must always indicate downhill flow of water.
 - Flow lines are always drawn perpendicular to contour lines. This is because water always flows downhill at right angles to elevation changes.
 - Flow lines never cross delineation lines. This is because the delineated boundary defines the divide or the outer boundaries of rainfall contribution.

Draw the delineated boundary or the dividing line starting at the top of a terrain high point, down a spur and across a saddle connecting another spur and saddle. The flow arrows will actually show a pattern of water moving in two directions (see figure 6). Figure 7 shows the delineation of a spur.

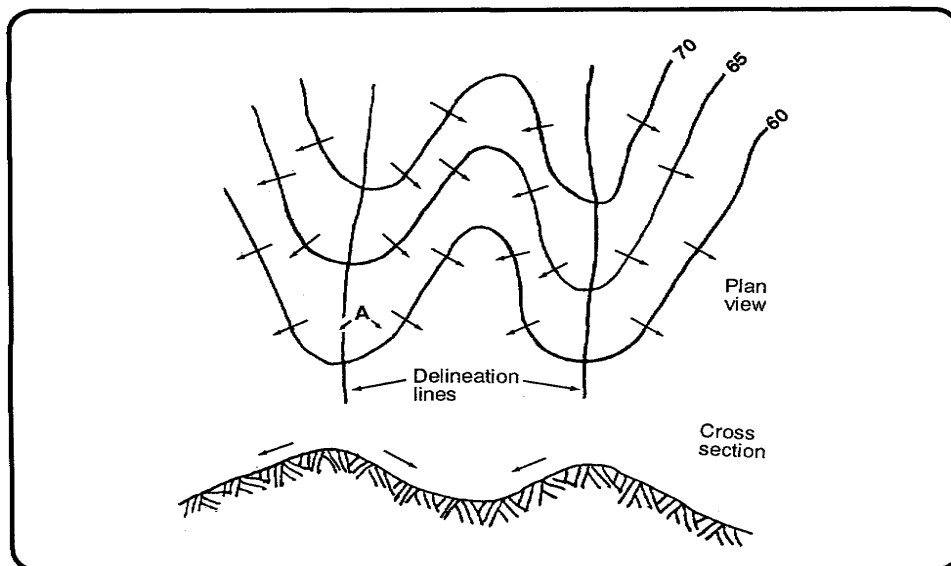


Figure 6. Delineation of multiple spurs or hills

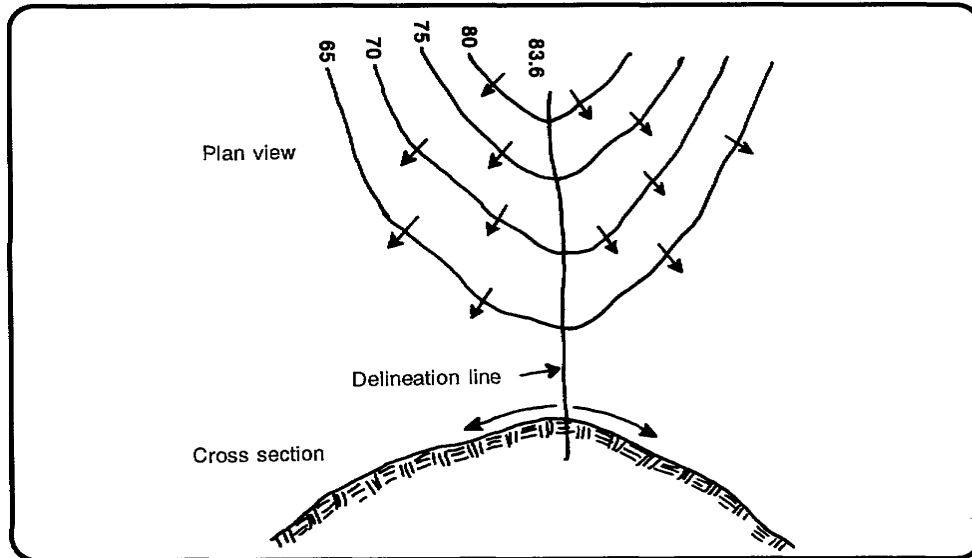


Figure 7. Delineation of a spur or hill

Figure 8 shows the delineation of a saddle. Notice that the flow arrows show a divergence of water on spurs. The delineation line is drawn between the opposing arrows (see A and B). The cross section of the swale represents the actual converging point for water that will be contributed to the southern half of the delineated saddle.

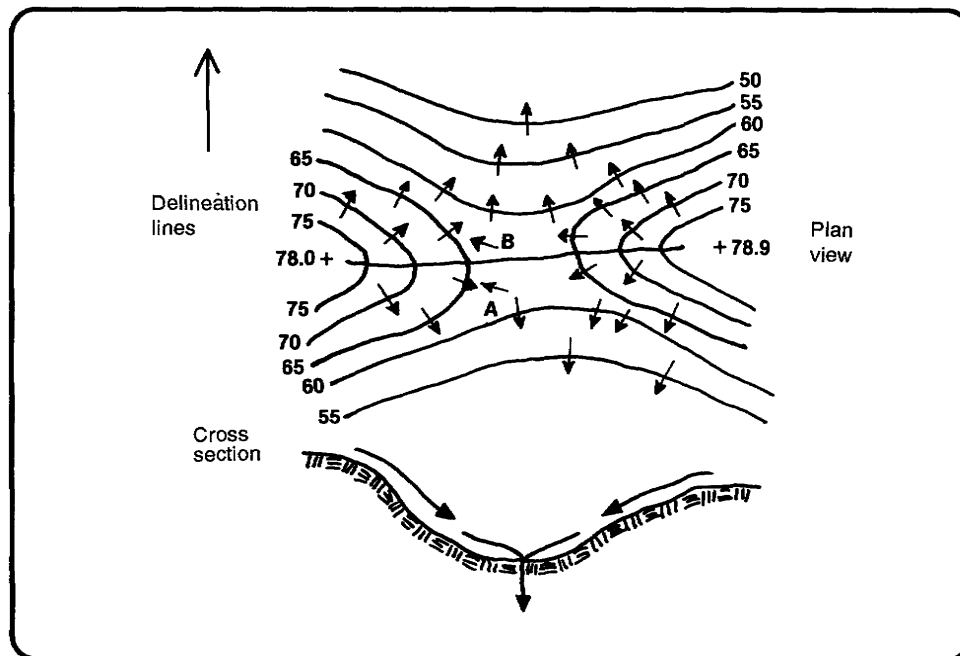


Figure 8. Delineation of a saddle

Most delineated boundaries will be adjacent to roadways. Because most roads on level ground use crowned profiles for road surface drainage, the delineated boundary would be marked down the center of the road. This is illustrated by the flow arrows in Figure 9, which shows a delineated drainage basin for an airfield project

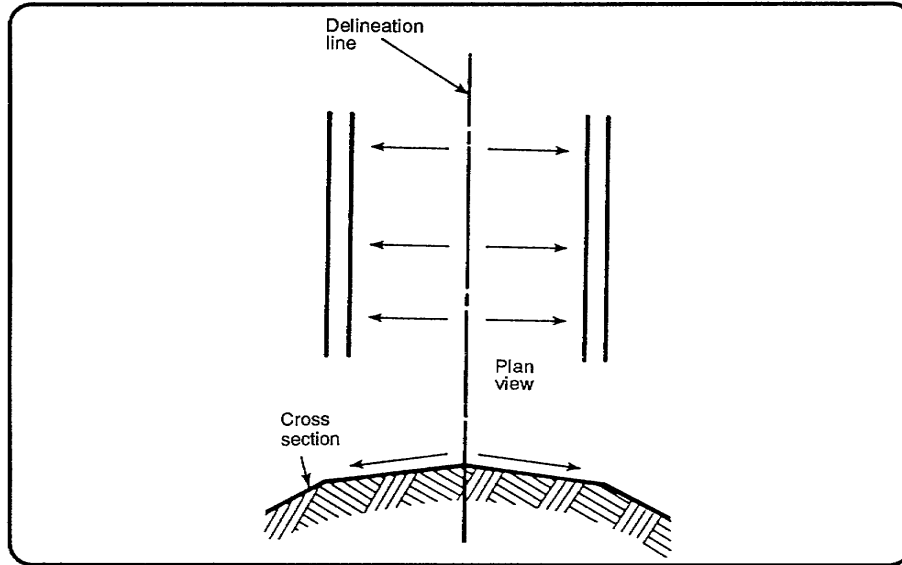


Figure 9. Delineation of roads or runways

STRIPPER METHOD

The *Stripper Method*, illustrated in Figure 10, is a variation of the Geometric Shapes Method. Approximate the watershed area by drawing a series of lines an equal distance apart. Then approximate the area by a number of trapezoids with the first and last segments approximating triangles. The heights of all these trapezoids are identical. The total area of all the trapezoids may then be expressed by the following formula:

NOTE: A map of the drainage area and a ruler will be needed.

A = Area

b = base

h = height

$$A = 1/2h(b_1) + 1/2h (b_1 + b_2) + 1/2h (b_2 + b_3) + 1/2h (b_3 + b_4)...+ 1/2h (b_8)$$

This may be factored and reduced to the following:

$$A = 1/2h (b_1 + b_1 + b_2 + b_2 + b_3 + b_3 + b_4 + ...b_8)$$

Which simplifies to:

$$A = h (b_1 + b_2 + b_3 + b_4 + ...b_8) + h(b_i) \text{ (or the sum of all the bases)}$$

Using the Stripper Method, measure (graphically) each base, take the sum, and multiply by the height (the perpendicular distance between any pair of bases, which is a constant). The distance between the parallel lines may be any value but must be constant throughout the area being measured. For irregularly shaped watersheds, draw the parallel lines closer together to ensure greater accuracy.

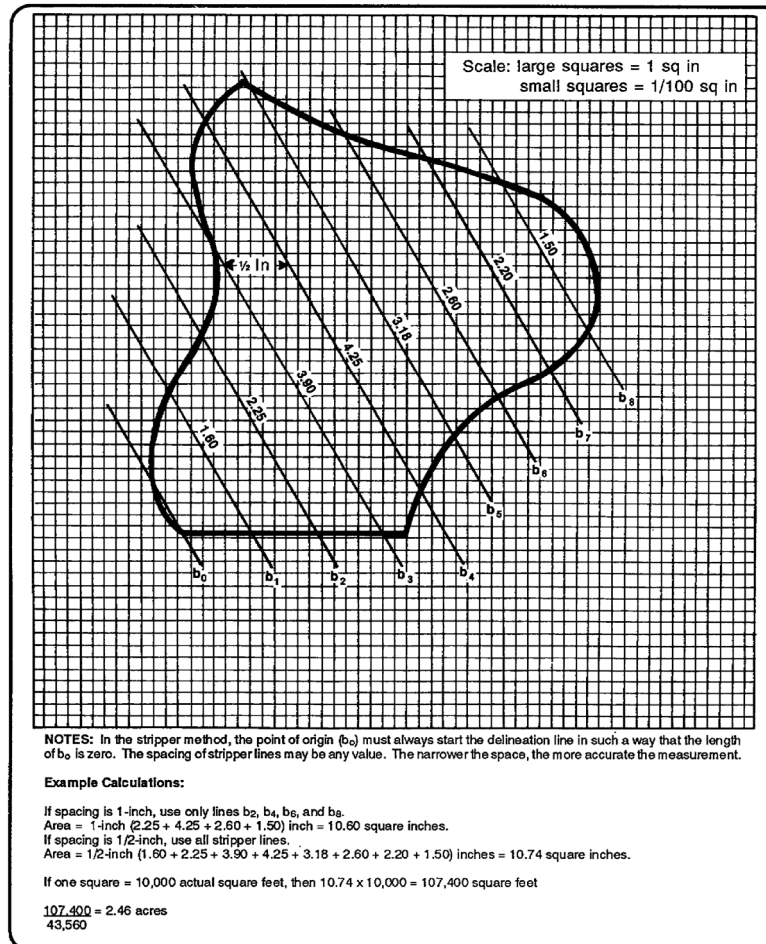


Figure 10. Area measurement - Stripper Method

RATIONAL METHOD

The Rational Method is used for estimating peak runoff to design a structure for a desired estimated lifetime, based on watershed characteristics and statistically probable rainfall.

Runoff is the surface water reaching the outlet of the watershed as a result of rainfall. The amount of runoff is the amount of precipitation less the amount of water that does not become runoff for a variety of reasons. These reasons or losses include:

- Infiltration.
- Interception.
- Evaporation.
- Transpiration.
- Detention (storage).

These natural losses were discussed earlier in this course.

The amount of water remaining after these losses taking a share of the total amount of precipitation is called *runoff*. This runoff will flow over the ground and become discharge at the basin outlet.

The Rational Method of estimating this runoff stems from very simple reasoning. To size a structure, you must know the maximum rate of flow through resulting from runoff from the watershed. It is important to remember that we are talking about volume per unit of time expressed in cubic feet per second (cfs), thus talking about discharge. The Rational Method does not apply to total volume of a parameter that might be used in designing ponding structures.

The Rational Method says that the rate of flow depends on several factors:

- The size of the watershed.
- The rate of rainfall.
- The proportion of rainfall that actually becomes runoff.

Runoff then equals a coefficient representing the proportion of rainfall that actually becomes runoff times the volume of water per unit of time that falls on the watershed. Volume of water per unit of time is the intensity of rainfall in inches per hour times the area of the watershed.

The Rational Method formula is expressed this way:

$$Q = CIA$$

Where:

- Q = Quantity of discharge in cfs.
- C = The coefficient of runoff.
- I = The rainfall intensity (in/hr).
- A = The area of the watershed in acres.

When looking at the equation, it appears that the units on the right do not match those on the left.

How could cubic feet per minute equal inches per hour?

$$\text{Cubic feet per second} = \frac{\text{inches of rain}}{\text{Per hour}}$$

Actually the units do match because, very conveniently, 1-inch of rain falling on 1-acre of ground equals 1.008 cfs.

$$\frac{1/12 \text{ foot} \times 43,560 \text{ square feet per acre}}{3,600 \text{ seconds}} = 1.008 \text{ cfs}$$

This is so close to unity that no correction factor is needed in the formula.

For simplicity the Rational Method is based on an idealized rainstorm; one that really never occurs in nature. In this rainstorm, the rainfall is steady and uniform over the entire watershed. In its attempts to represent real rainstorms, these idealized rainstorms have higher rates of rainfall for storms that are short in duration. For such idealized storms, the maximum discharge can take place only when the entire basin contributes simultaneously to that runoff. This is because it takes time for water to flow from where it falls, over the surface, to the outlet of the watershed (see Figure 11).

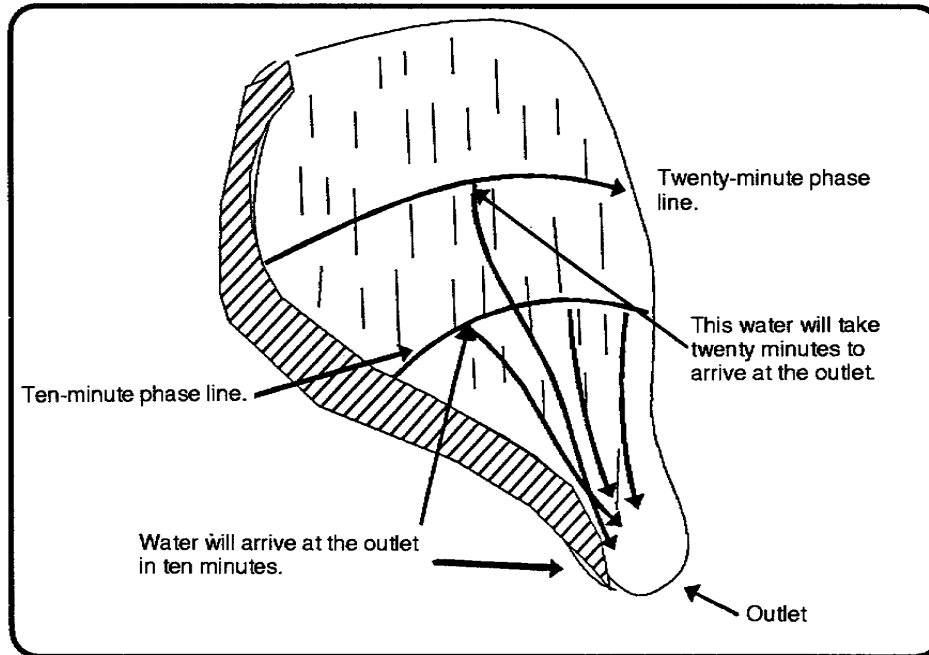


Figure 11. Flow across a regular drainage basin

Water that falls close to the culvert will arrive at the outlet sooner; rain that falls at the farthest reaches of the basin will arrive later.

For discharge at the outlet to be maximum, the runoff closest to the culvert must continue to arrive at the same time runoff from the farthest (with respect to time) point of the watershed reaches the outlet.

Consequently, to get maximum runoff rate, the steady, uniform rain must last at least as long as the time it takes water to flow from the farthest point of the watershed to the outlet.

The time it takes for water to move from the most remote part of the basin to the outlet is a very important parameter in the Rational Method called the *TOC*.

The Rational Method is used only for small drainage basins (areas less than or equal to 1,000 acres). This is because the idealized storm in the Rational Method approximates thunderstorms. These types of storms are small in area, but move rapidly through an area.

The Rational Method should be used for areas without active streams. Also, the Rational Method should not be used if there are ponds or lakes within the watershed. There are several reasons for this. Streams flowing through the watershed greatly complicate the runoff estimation process, and are, therefore, beyond the scope of the Rational Method. The Rational Method does not consider storage from lakes or ponds.

The Rational Method should be used only if the basin has a regular shape. If the basin is not of a regular shape, then *TOC* assumptions may not be accurate. Sound judgement must be used to determine if the basin is regular in shape.

Before the Rational Method is used, it is important to remember that it is a standard design method for relating probable precipitation over a small watershed to the probable maximum runoff rate at the outlet of the watershed. It is a simple approximation of a very complex process and it cannot be regarded as a

precise forecast. In reality, uniform rainfall does not occur, and the coefficient of runoff isn't really constant in time, even within the same storm. The method's accuracy cannot be verified because the idealized storm does not exist and records of measured precipitation and runoff to verify the method statistically are not available. In short, the Rational Method is not cut and dried. It will require engineering judgment.

However, the Rational Method is simple to apply; it gives consistent results, and most importantly, it is the standard method adopted by most state highway departments.

RATIONAL METHOD DESIGN PROCEDURES

Use the following steps when doing design procedures:

STEP 1. Delineate the drainage basin. The area of rainfall contribution to the structure must be defined by delineation.

STEP 2. Delineate areas by soil and cover type. Since the type of cover has an effect on the coefficient of runoff the watershed must be segmented according to its respective cover type. This information can usually be obtained from the topographic map of the area. If available, a soils map can also be used.

STEP 3. Determine the acreage for each basin and subarea. If not known this can be done using the Stripper Method.

STEP 4. Classify the basin as simple or complex.

- Simple Area: One soil or cover type makes up 80 percent or more of the total basin.
- Complex Area: No one cover makes up 80 percent of the total area in the basin.

The simple formula can be used to calculate simple or complex areas:

$$\frac{\text{Largest subarea}}{\text{Subarea1} + \text{subarea2} + \text{subarea3} \dots} \times 100 = \text{percent of the largest subarea}$$

STEP 5. Determine representative flow paths. Picking the representative flow paths is the most complex part of the Rational Method procedure. In this process you will have to use sound engineering judgment. Representative flow paths will be used in two separate processes and will affect two different variables in the Rational Method equation.

a. Representative flow paths will be used to account for the time it takes water to flow across the ground and reach the outlet. This process will be used to obtain the TOC. The TOC will then be used to adjust the rainfall intensity by using the standard intensity curve in Figure 5. Procedures for obtaining the TOC will be discussed later in the course material .

b. Representative flow paths will also be used to represent the average slope in the area. This procedure will be outlined later in the course material.

STEP 6. Divide the flow paths into two sections--sheet and channel flow. This step will be needed in order to account for the time it will take water to move across the watershed. Sheet flow is slower than water flowing in a ditch, so you will have to calculate their respective times differently. These procedures will be discussed in the course material.

STEP 7. Determine the slope of each section of the flow path. Since slope affects the speed at which water moves across the basin and thus affects the runoff coefficient, you must calculate the slope of each path. This information will be needed when using the sheet flow graphs and the ditch flow tables later in the course. The slope of each path can be calculated from information provided by the topographic map representing the delineated basin. Use the following calculation:

$$\frac{\text{Beginning elevation} - \text{Ending elevation}}{\text{Path length}} \times 100 = \text{Percent of slope}$$

STEP 8. Determine the average slope of the area. Every cover type within the basin will have a coefficient of runoff. This coefficient will have to be adjusted according to the average slope of the terrain in that respective area. This adjustment is accomplished using a coefficient table that has been figured with three different slope ranges and their corresponding "C" values. (This will be covered later in the course material.)

Define the flow paths as being within a range of:

- Less than or equal to 2 percent.
- Greater than 2 percent, but less than 7 percent.
- Greater than 7 percent.

STEP 9. Find the "C" value of each basin or subarea based on soil/cover type and slope from Table 1. Calculate "C" if the watershed is complex. Use the following formula:

$$C_{\text{wtd}} = \frac{C_1A_1 + C_2A_2 + \dots + C_NA_N}{A_1 + A_2 + \dots + A_N}$$

Where:

C = Coefficient of runoff for each subarea.

A = The size of each subarea.

Table 1. Runoff coefficients

Soil and Cover Classification	C-Values					
	Slope ≤2 Percent (%)		Slope >2 and <7 Percent		Slope ≥7 Percent	
	With Turf	Without Turf	With Turf	Without Turf	With Turf	Without Turf
¹ GW, GP, SW, SP	0.10	0.20	0.15	0.25	0.20	0.30
² GMd, SMd, ML, MH, PT	0.30	0.40	0.35	0.45	0.40	0.50
³ GMu, GC, SMu, SC, CL, OL, CH, OH	0.55	0.65	0.60	0.70	0.65	0.75
Wooded area	0.20	0.20	0.20	0.20	0.20	0.20
Asphalt pavement		0.95		0.95		0.95
Concrete pavement		0.90		0.90		0.90
Gravel/macadam		0.70		0.70		0.70

¹GW - Well graded gravels or gravel-sand mixture with 5 percent or less amount of fines; GP - Poorly graded gravels or gravel-sand mixture with little or no fines; SW - Well graded sands or gravelly sand mixture with 5 percent or less amount of fines; SP - Poorly graded sands or gravelly sand mixture with 5 percent or less amount of fines.

²GMd - Silty gravel; desirable base and subbase material; SMd - Silty sands and poorly graded sand-silt mixture; undesirable base and subbase material; ML - Inorganic silt; silty fine sands; MH - Inorganic silt micaceous or diatomaceous silty soil; PT - Peat soil.

³GMu - Silty gravel; undesirable base and subbase material; GC - Clayey gravel, gravel-sand-clay mixture; SMu - Silty sands and poorly graded sand-silt mixture; undesirable base and subbase material; SC - Clayey sands, sand-clay mixture; CL - Inorganic clay of low to medium plasticity, lean clays; OL - Organic silt and organic silt-clay of low plasticity; CH - Inorganic clay of high plasticity, fatty clays; OH - Organic clay of medium to high plasticity.

STEP 10. Determine the travel time for each flow path and select the longest time as a basin TOC. Use Table 2 to calculate flow time in sheet conditions. Use Figure 12 to calculate the travel time in a ditch condition. Use the following formula to obtain the flow time in a ditch:

$$\text{Flow time ditch} = \frac{\text{Path length feet (ft)}}{\text{Velocity feet per minute (fpm)}}$$

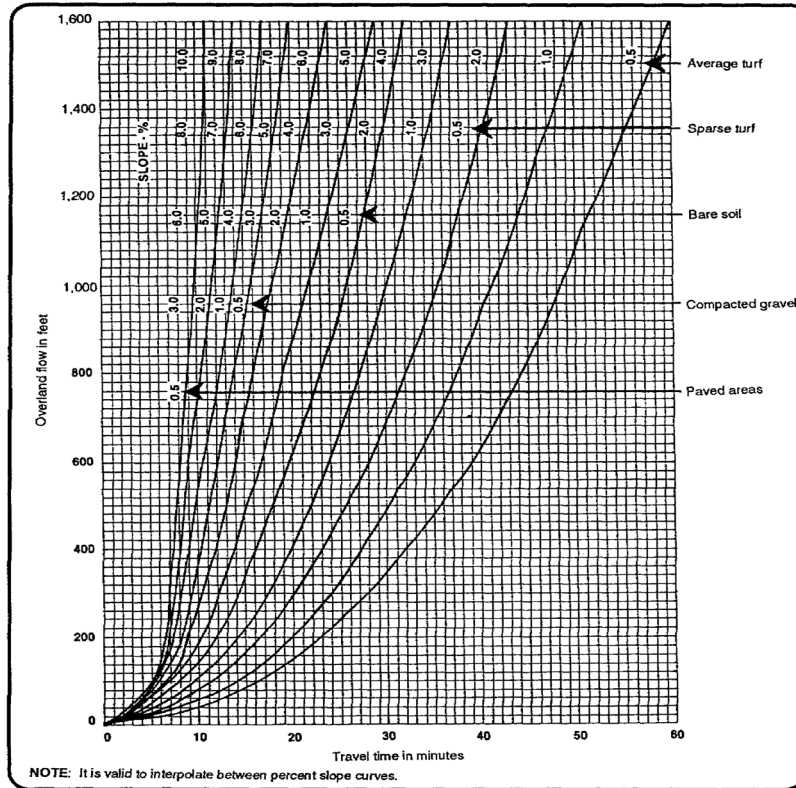


Figure 12. Sheet flow travel time graph

Table 2. Ditch flow travel times

Slope	Velocity	Slope	Velocity
0.5	115	1.8	188
0.6	120	1.9	194
0.7	125	2.0	200
0.8	130	2.1	205
0.9	135	2.2	210
1.0	140	2.3	215
1.1	146	2.4	220
1.2	152	2.5	225
1.3	158	2.6	230
1.4	164	2.7	235
1.5	170	2.8	240
1.6	176	2.9	245
1.7	182	3.0	250

Ditch length = Ditch travel time
Velocity

STEP 11. Find the rainfall intensity (I) for a 2-year, 60-minute storm from the world isohyetal map Figure 4

STEP 12. Adjust the rainfall intensity to match the TOC using the Standard Intensity-Duration Curves (Figure 5).

STEP 13. Calculate Q (discharge) using the “C” value from the step 9, times the “I_{adj}” from Step 12, and the “A” from Step 3 as follows:

$$Q = CIA$$

You now have enough information to put the Rational Method to use in an example. First, let's assume that our example area meets the criteria of the Rational Method.

- Its total area is less than 1,000 acres.
- The watershed has no lakes or flowing streams in it.
- The basin has a regular shape.

Example:

Estimate runoff using the Rational Method in the following example with the given information provided below:

Location: Northern California

Surface composition and cover:

GMd soil with average turf	47.9 acres
Compacted gravel surface	0.9 acres

NOTE: Information on delineation areas and subareas will be provided for this example. Usually the information would be obtained from a topographic map of the area.

Solution:

STEPS 1, 2, and 3 are given.

STEP 4. Classify the basin as simple or complex.

$$\text{Subarea}_1 = \frac{\text{Subarea}_1 \text{ GMd with turf } 47.9 \text{ acres}}{\text{Subarea}_1 - 47.9 \text{ acres} + \text{subarea}_2 - .9 \text{ acres}} \times 100 = 98 \text{ percent GMd soil (Percentage)}$$

Since GMd soil makes up greater than 80 percent of the total area, the basin is classified as a *simple* area. This basically means that the entire area will be represented with one “C” value.

STEP 5. Determine representative flow paths. For this example we will use Figure 13.

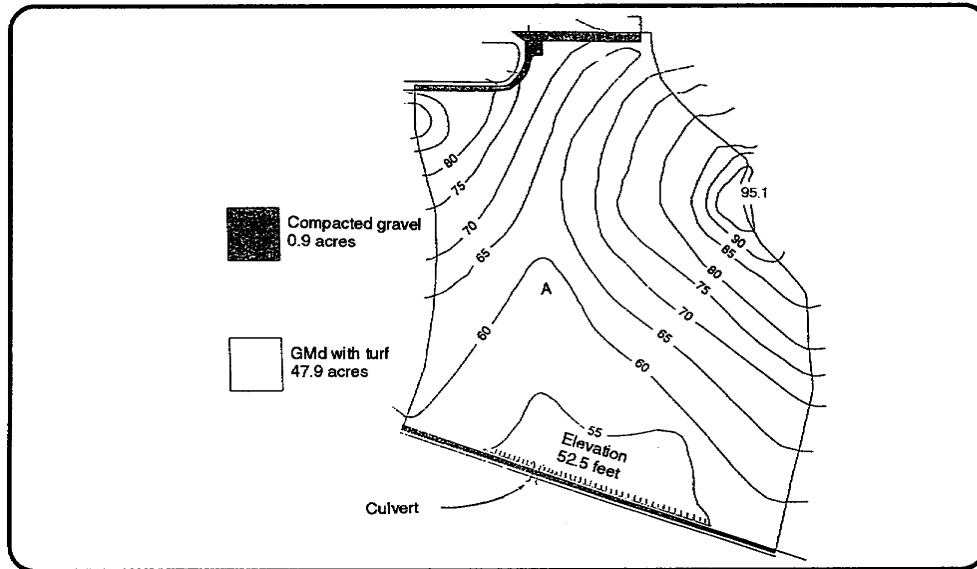


Figure 13. Delineated basin

Refer to Figure 14. The delineated area can be broken down into two sections. On the west side there is a hill with an elevation of 96.0 feet. On the east side there is a much larger hill with an elevation of 95.1 feet. The basin is divided by a north/south running swale. Notice also that the side on the eastern hill has a swale (see “A” on Figure 14) dividing the hill into two segments. This suggests that we can represent the water basin with three representative flow paths. One path on the western hill and two paths on the eastern hill. If we draw a flow path starting at the western hill top midway between the delineated lines, we could represent most of the flank of the western hill (see arrow). Water moving down the hill would eventually gather in the swale turn south (downhill) to the culvert.

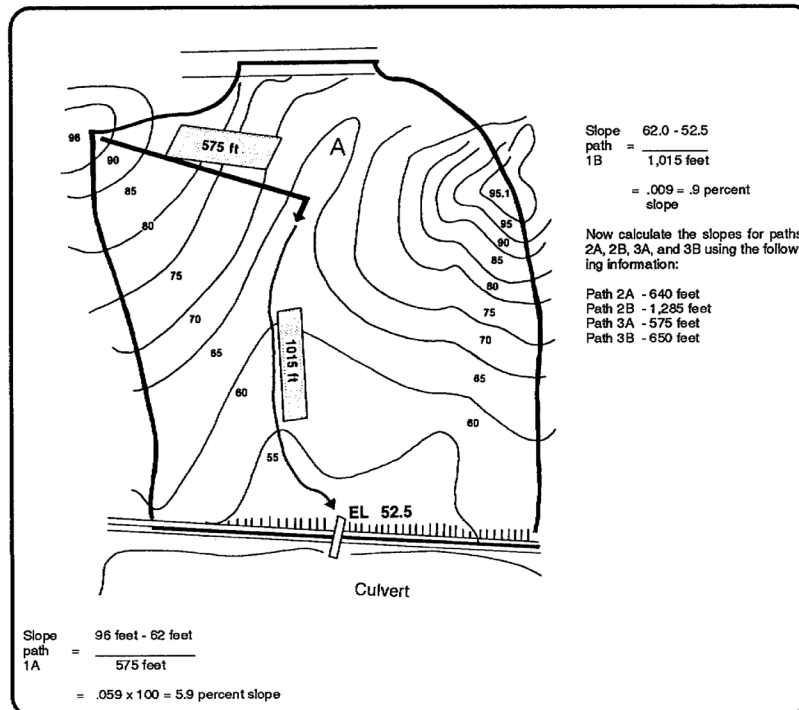


Figure 14. Representative flow path 1

The flow path for path 1 (see Figure 14) will be drawn as far to the north as possible. Notice that the flow arrows indicate a change in direction, meaning that the water would turn on the north side of the ridge, outside of the delineated area, thereby contributing runoff to another watershed.

Refer to Figure 15. Now let's represent the watershed on the western side of the eastern hill. We can represent the north side of the basin with a flow path from the top of the hill, to the bottom of the swale, and just like path 1, the water will turn at the bottom of the swale. This we'll name path 2. Path 3 will start at the top of the eastern hill and move in the hillside swale directly to the culvert. This will represent the southern half of the hillside, from the south by southwest running ridge to the eastern edge of the watershed.

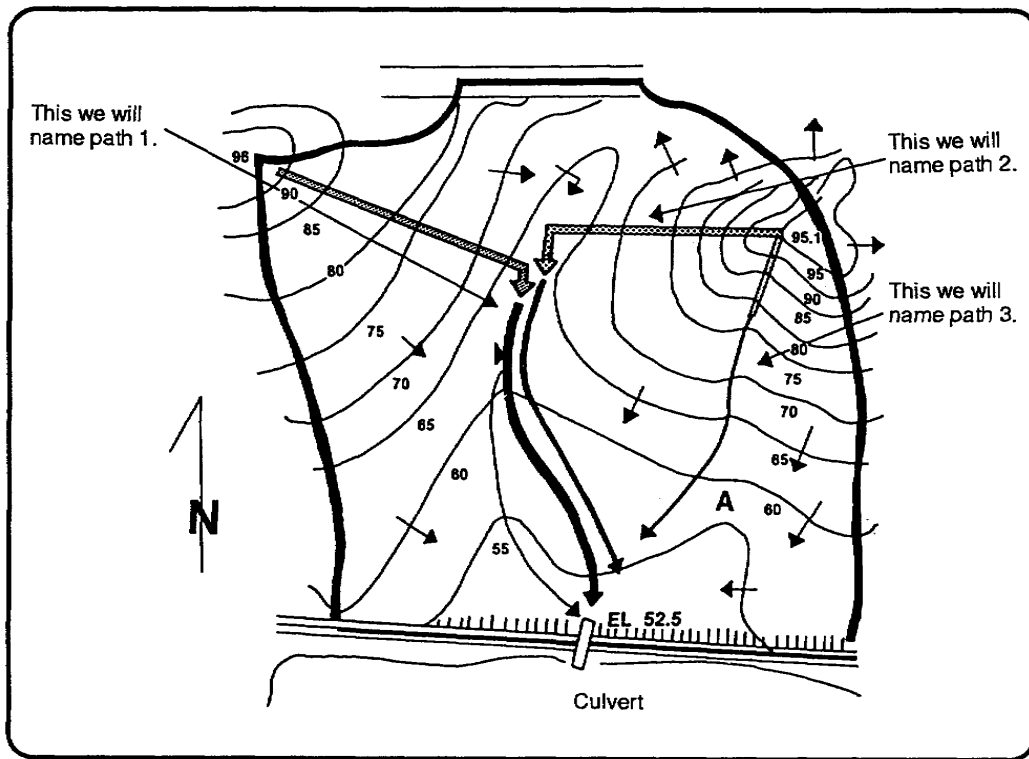


Figure 15. Example basin

STEP 6. Divide flow paths into sheet and ditch (channel) flow. The water that begins to flow down path 1 (Figure 15) will move slowly, gaining speed as it progresses down the slope. Since the layer depth of the water is at a minimum, the cover and the vegetation will retard the water's velocity. This is known as a *sheet flow* condition, often referred to as *overland flow*. After the water flows a certain distance, it will converge in the center and its velocity will increase. Eventually it will gather into a ditch flow condition. Water will move faster in a ditch flow condition and will take less time to reach the culvert.

Refer to Figure 15. When the water from path 1 reaches the bottom, it will converge into a ditch at the bottom of the swale. From the swale to the culvert it will flow in a ditch condition. Since water is flowing at two different rates, we will separate path 1 into two lengths, path 1A (sheet flow), path 1B (ditch flow).

A rule of thumb is that water turning in a swale or uphill kinks in the contour lines are good indications of a transition from sheet to ditch flow.

Sheet flow will transition to ditch flow at about 500 feet (+ -200 feet). Although this would vary with slope and cover type.

STEP 7. Determine the slope of each section of each flow path. Since slope affects the velocity of water in the paths, the slope of each path must be calculated. This can be done by using the contour lines. Path 1a starts on the hill at the elevation of 96.0 feet. The path ends in the middle of the swale at 62.0 feet. The slope can be calculated using the slope equation. See Figure 14, and Step 7.

Now that all of the slopes have been calculated, it's a good idea to tabulate the data. The following format shown in Table 3 is suggested:

Table 3. Basin flow paths

Path	Cover/Type	Length (Feet)	Velocity	Slope (Percent)	Travel Time
1A	Average turf	575	_____	5.9	_____
1B	Ditch section	1,015	_____	0.9	_____
2A	Average turf	640	_____	5.2	_____
2B	Ditch section	1,285	_____	0.8	_____
3A	Average turf	575	_____	5.0	_____
3B	Ditch section	650	_____	2.3	_____

This format will be used throughout the course section.

Refer to Figure 15.

STEP 8. Determine the average slope of the basin or subareas. The slope of the paths will have an effect on the basin and likewise on the “C” value. Because of this, if there is more than one flow path in the basin or each subarea, we will have to get an average slope of the entire basin or each respective subarea. This is the second process in which representative flow paths will be used.

Refer to Figure 16. We'll start with path 1. Notice that path 1B runs downhill to the culvert in the same swale as path 2B. Path 1B is redundant. It provides the same information as path 2B. For this reason, we will eliminate it from the average for path 1. Now path 1 will have a path average of 5.9 percent (the former path 1A).

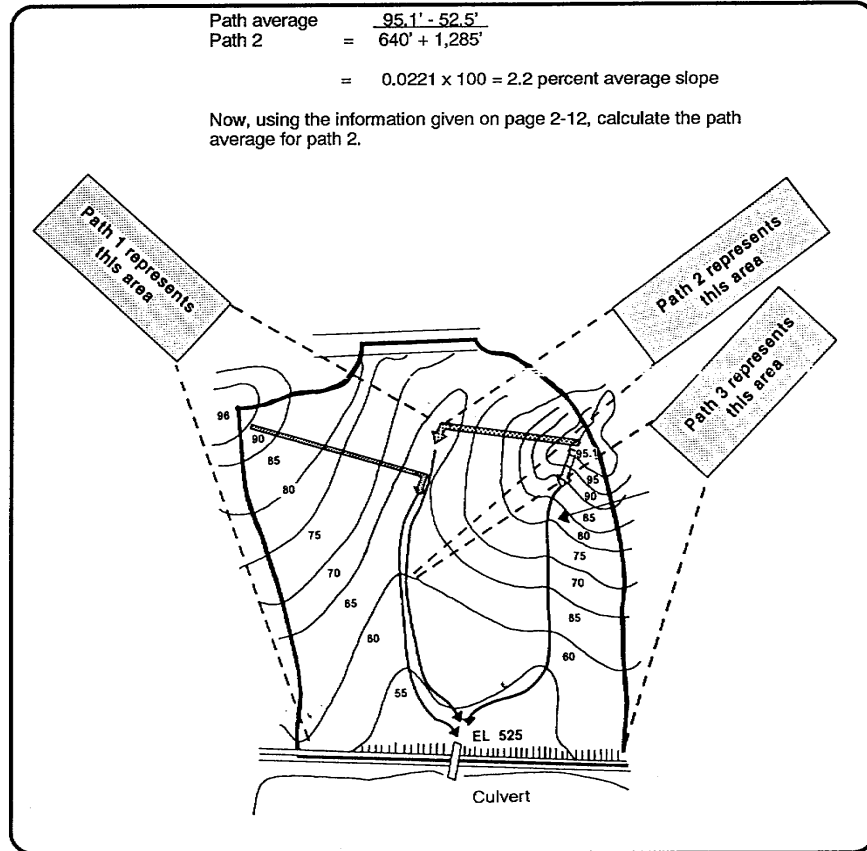


Figure 16. Slope representation

Now the average for path 2 must be calculated. It is important to remember that the average for path 2 is not the arithmetical average of path 2A and 2B. If the arithmetical average of both of these paths were taken and used as an average, the watershed area for path 2 would be misrepresented (see Figure 17). By taking the difference in elevation from the start of path 2a on the eastern hilltop and the end of path 2B at the culvert inlet and dividing the difference by the cumulative total of path lengths 2A and 2B, we can accurately calculate the average slope for the entire watershed area that path 2 represents.

The path slope average can be calculated using the following formula:

Weight average =

$$\frac{(L_{\text{Subpath1}} \times S_{\text{Subpath1}}) + (L_{\text{Subpath2}} \times S_{\text{Subpath2}}) + (L_{\text{SubpathN}} \times S_{\text{SubpathN}})}{L_{\text{Subpath1}} + L_{\text{Subpath2}} + L_{\text{SubpathN}}}$$

Where:

- L = Length of each path (feet)
- S = Slope of each path in ft/ft

Using path 2 as an example:

$$\frac{(640' \times .052) + (1,285' \times .008)}{640' + 1,285'} = \frac{43.56'}{1,925'} = 2.26 \text{ percent}$$

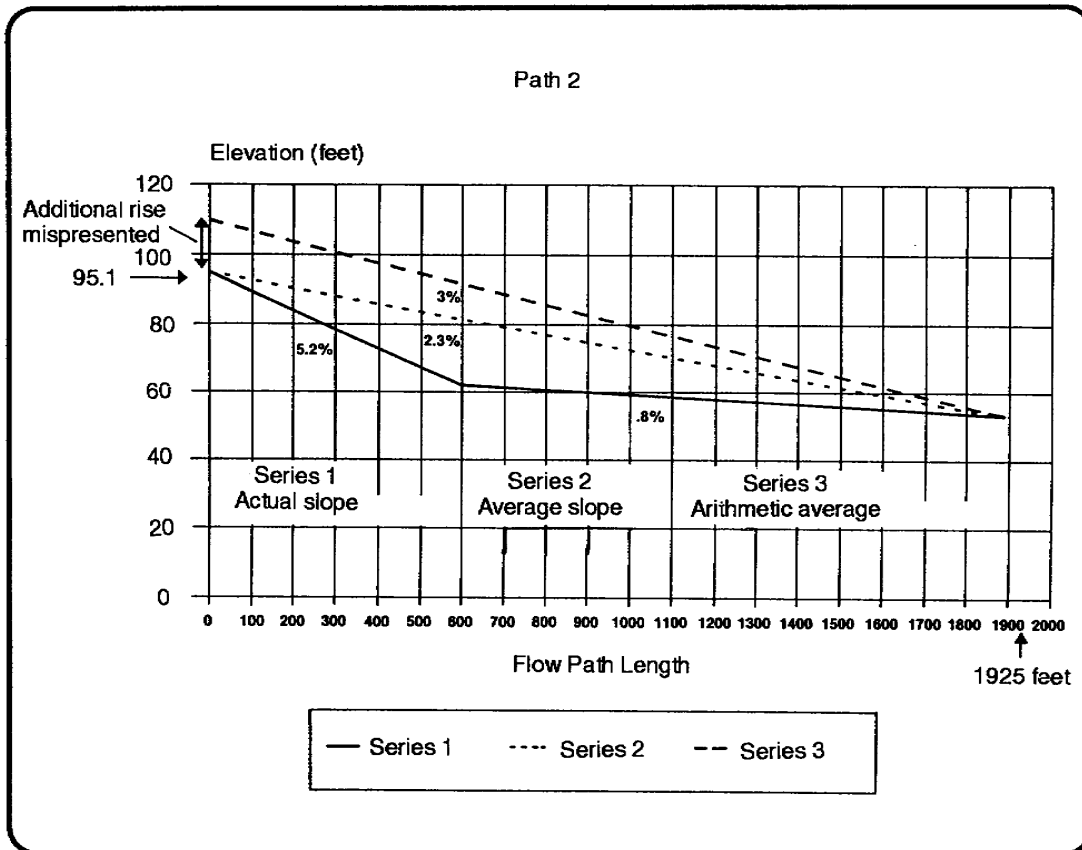


Figure 17. Slope comparison

With the path averages calculated, the overall slope of the area or subareas now must be calculated in order to obtain the right coefficient from the “C” value chart. This can be accomplished by taking the arithmetical average of all of the paths in the entire basin, if it is classified as simple, or the average of all of the paths in each subarea if the basin is complex.

The overall slope of the area can be calculated using this formula:

$$\begin{aligned} \text{Overall slope for the basin or each subarea} &= \frac{\text{Path}_{\text{avg}1} + \text{Path}_{\text{avg}2} + \dots + \text{Path}_{\text{avg}N}}{N} \\ \text{Overall slope for the example basin} &= \frac{5.9\% (\text{Path 1}) + 2.3\% (\text{Path 2}) + 3.6\% (\text{Path 3})}{3} \\ &= 3.9\% \text{ overall slope for the basin (simple area)} \end{aligned}$$

Since this example is for a simple area, you will represent this entire basin with one “C” value for an overall slope of 3.9 percent. If this example was a complex area, you would calculate the overall slope for each subarea and use a “C” weighted formula.

STEP 9. Find the “C” value for the basin or subarea. Now you can go to Table 1 and find the correct “C” value. GMD with turf slope 2 percent > < 7 percent 0.35. Refer to Figure 18.

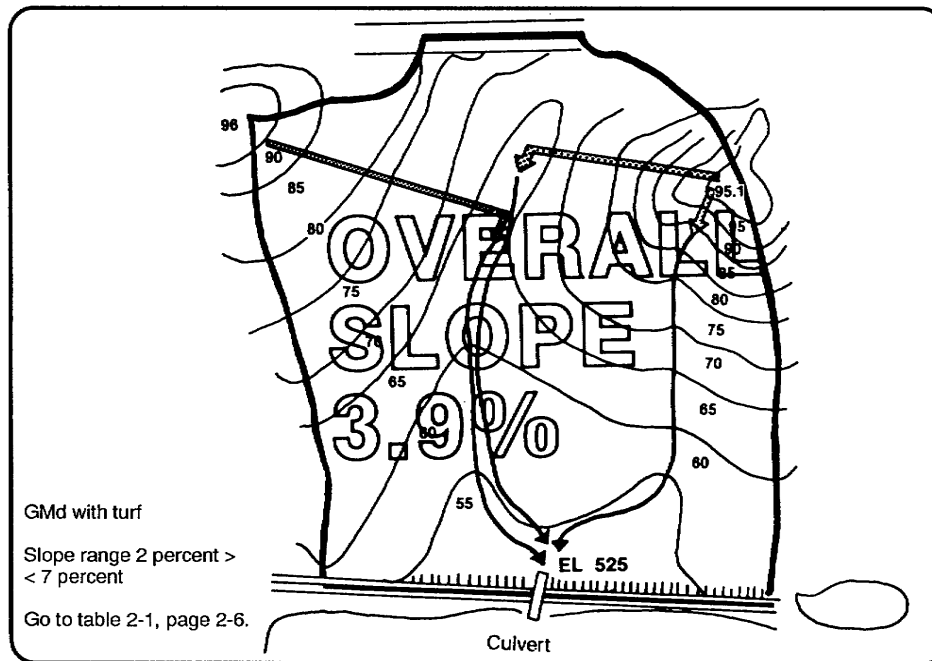


Figure 18. Basin

STEP 10. Determine the travel time of each flow path and select the longest time at the basin TOC. Again, we will use representative flow paths to estimate the travel time. Remember, we are trying to account for the time it will take water to flow from the start of each path to the culvert inlet, so that we can determine the TOC.

To calculate the travel time the two flow conditions that exist need to be considered, sheet and ditch flow.

Sheet flow will be estimated using Figure 12. Notice that on the graph there are a series of curves, each with line and curvilinear portions. The slope of each curve indicates the velocity at a given point along the flow path. In the curvilinear portion, the slope is initially zero and steepens until it becomes linear. This represents the fact that initially water moves slowly and picks up speed as the depth of the water increases. It is initially slow-flowing (sheet flow) and gradually it becomes faster (ditch flow) as it progresses. At some point along the path the turbulent flowing water reaches a steady state velocity. Figure 15 shows that the slope and “slickness” of the flow path dictates how quickly it transitions from sheet to ditch flow.

NOTE: If the slope exceeds the value of the 10 percent curve, the water will be at a steady state of velocity. Use the 10 percent curve.

To use the graph, enter at the right hand vertical edge at the corner the type of the flow path, horizontally to the left, until you reach the curved label labeled with the slope of the path. Follow it up or down until it intersects the horizontal line that equals, the flow-path length. Finally, read travel time which is found by drawing a vertical line down to the time in minutes axis on the bottom of the chart.

Using the chart below, calculate the ditch travel time using the tabulated format given in Step 7.

Table 4. Basin flow paths

Path	Cover/Type	Length (Feet)	Velocity	Slope (Percent)	Travel Time
1A	Average turf	575	<u>N/A</u>	5.9	14.5
1B	Ditch section	1,015	<u> </u>	0.9	<u> </u>
					Total <u> </u>
2A	Average turf	640	<u>N/A</u>	5.2	16.5
2B	Ditch section	1,285	<u> </u>	0.8	<u> </u>
					Total <u> </u>
3A	Average turf	575	<u>N/A</u>	5.0	16.5
3B	Ditch section	650	<u> </u>	2.3	<u> </u>
					Total <u> </u>

Flow travel time in a ditch condition can be estimated using the ditch flow table. The table lists velocities in a ditch for various slopes. Travel time can then be calculated using this simple formula:

$$\text{Travel time} = \frac{\text{Path length (ft)}}{\text{Velocity (fpm)}}$$

To get the total travel time, the time the water is flowing in the ditch must also be calculated. Path 1B will be calculated using the ditch flow table. Go back to the tabulated format. Path 1B has a slope of 0.9 percent. Using the following equation and the above listed information on path 1B, the travel time is calculated as:

$$\text{Travel time} = \frac{1,015 \text{ ft}}{135 \text{ fpm}} = 7.5 \text{ minutes}$$

Now calculate ditch time for paths 2B and 3B. Use the information provided in the tabulated format on

Table 6 and the ditch flow table (Table 5). The total travel time can be obtained by adding the sheet flow time and the ditch flow time for the respective path.

Path 1A sheet flow time = 14.5 minutes

Path 1B ditch flow time = $\frac{7.5 \text{ minutes}}{22 \text{ minutes}}$

Table 5. Ditch flow

Slope (Percent)	Velocity (fpm)	Slope (Percent)	Velocity (fpm)
0.5	115	1.8	188
0.6	120	1.9	194
0.7	125	2.0	200
0.8	130	2.1	205
0.9	135	2.2	210
1.0	140	2.3	215
1.1	146	2.4	220
1.2	152	2.5	225
1.3	158	2.6	230
1.4	164	2.7	235
1.5	170	2.8	240
1.6	176	2.9	245
1.7	182	3.0	250

At 3 percent, slope velocity will be at a steady state.

Steady state is a maximum velocity. Water moving over land will not exceed 250 fpm.

Now that the travel times for all the paths have been calculated, the TOC can be found. The TOC is the path with the longest travel time. In our example, path 2 has the longest travel time, 26.4 minutes. The table below give the times listed for the example problem.

Table 6. TOC

Path	Cover/Type	Length (Feet)	Velocity	Slope (Percent)	Travel Time	
1A	Average turf	575		5.9	14.5	
1B	Ditch section	1,015	<u>135</u>	0.9	7.5	
					Total 22	
2A	Average turf	640		5.2	16.5	
2B	Ditch section	1,285	<u>130</u>	0.8	<u>9.9</u>	
					Total 26.4	← TOC 26.4 minutes
3A	Average turf	575		5.0	16.5	
3B	Ditch section	650	<u>215</u>	2.3	<u>3.04</u>	
					Total 22	

STEP 11. The "I" value for this area is 2.5 in/hr.

STEP 12. Adjust the rainfall intensity "I" value to match the basin TOC.

To read the chart, enter the graph with the basin TOC of the bottom axis. Draw a vertical line up until you intersect with the standard curve from the isohyetal chart (refer to figure 5). Then draw a horizontal line to the left axis and obtain the rainfall intensity adjusted I_{adj} in in/hr. In this example, the standard intensity is adjusted to 3.5 in/hr based on the basin TOC of 26.4 minutes.

STEP 13. Calculate the runoff in cfs using the Rational Method equation $Q = CIA$.

In this example, the "C" (0.35) x "A" (48.8 acre) x "I" (3.5) in/hr = 59.8 cfs.

DETERMINE THE INTENSITY (I) WITH AND WITHOUT PINPOINT DATA

In the formula $Q = CIA$, "I" equals the intensity of the rainfall. Intensity is the rate at which rain is falling. It is expressed in units of in/hr.

To determine "I" for a given drainage area:

- Estimate the drainage area design TOC.
- Calculate I adjusted to the area design TOC.

ADJUSTED "I"

Having already learned how to estimate the TOC, the next step is to calculate "I" adjusted to the TOC.

The first step is to obtain a source of rainfall data. Pinpoint data is preferable, if available. Table 7 shows rainfall pinpoint data for Washington, D.C. It covers the ten worst storms in each time category for 50 years. The storms are listed in descending order of magnitude. The worst five-minute rainfall may or may not have occurred during the same rainstorm as the worst 15-minute rainfall; hence, the use of the term storm is actually a composite of as many as six actual rainfall events over the 50 years of record.

Table 7. Rainfall data - Washington, D.C., 1900-1950

Worst	Duration (Minutes)						
	5	10	15	30	60	120	360
	Precipitation (Inches)	Precipitation (Inches)	Precipitation (Inches)	Precipitation (Inches)	Precipitation (Inches)	Precipitation (Inches)	Precipitation (Inches)
1	0.77	1.21	1.51	2.43	3.42	3.91	5.38
2	0.67	1.20	1.44	2.11	2.79	3.33	5.15
3	0.56	0.96	1.25	1.77	2.66	3.25	3.69
4	0.55	0.94	1.18	1.42	2.48	3.22	3.48
5	0.53	0.68	0.83	1.39	2.29	2.71	3.10
6	0.40	0.62	0.68	1.02	1.57	2.01	2.94
7	0.38	0.59	0.62	0.65	1.42	1.88	2.69
8	0.34	0.50	0.52	0.64	1.02	1.19	1.90
9	0.31	0.45	0.48	0.53	0.78	1.15	1.76
10	0.21	0.38	0.38	0.51	0.62	1.06	1.63

If the records contain data over a long period of time, the formula is $T = n/m$ and the recurrence interval calculated will be sufficiently accurate for design purposes.

The recurrence interval of the first-ranked storm is:

$$T_r = \frac{n}{m} = \frac{50 \text{ years}}{1} = 50$$

Where:

- T_r = Recurrence interval, or time of return in years
- n = Length of record (number of years data)
- m = Order number, in descending magnitude (rank of storm)

To find out which storm occurs at the interval of the design storm, say five years--

$$T_r = 5$$

Therefore, $5 = \frac{50 \text{ years}}{m}$

Therefore, $m = 10$ (the tenth worst storm).

When pinpoint data is not available, the engineer should refer to an isohyetal map. To review the basic rules:

- When the drainage area falls on an isohyetal line, read the value of that isohyet.
- When the drainage area falls inside an encircling isohyetal line, read the value of that isohyet.
- When the drainage area falls between the isohyetal lines, DO NOT INTERPOLATE. Read the value of the larger isohyet.

To obtain “I” expressed in in/hr, you may have to perform the depth to intensity conversion as explained in Lesson 1, Part B. Next adjust the duration so that it equals the design TOC. To do this, utilize Figure 5, the Standard Intensity/Duration Curves.

CALCULATION OF RUNOFF FOR SUCCESSIVE AREAS

Unfortunately, the increase in runoff is not the simple summation of the peak runoff of each individual area. Various factors, such as storage and peak flow attenuation, act on the individual peak flows. Peak flow(s) from the upstream area(s) will probably not arrive at the lower outlet simultaneously with the peak from the downstream dependent area. Hence, the total peak flow is less than the sum of the individual peak flows. To estimate with precision the quantity of accumulated runoff a procedure has been developed where TOC is recalculated for each of the successive drainage areas. Naturally, as TOC increases, the rainfall intensity, I, decreases.

The following terms are used in connection with computing $Q = CIA$ for successive areas:

- Independent area : A single area which does *not* receive runoff from some upstream area(s).
- Dependent area: An area which receives runoff from upstream area(s).
- Parallel areas: Two or more areas which are discharging runoff into the same dependent downstream area.
- TOC_x : TOC redefined to be the maximum travel time from the most hydraulically remote representative point in the series of areas to a particular outlet, X.
- Inlet time $_x$: Formerly called TOC. Inlet time $_x$ is the maximum representative flow time for runoff originating in area X arriving at outlet X, whether dependent or independent
- Inlet time $_{W-X}$: The sum of TOC_x (the TOC at the outlet upstream in area W) plus DT_{W-X} (the “ditch time (DT),” transit time through area X, from outlet W to outlet X).
- Subarea: An individual drainage area in the series of drainage areas.

Runoff Estimation Review

Preparatory work should be done before doing an estimation summary by using the following steps.

STEP 1: Delineating every subarea in the series of areas.

STEP 2: Determining the intensity of the two-year, 60-minute rainfall.

STEP 3: Determining for each subarea:

- Acreage, A.
- C, and C weighted, if necessary.
- Inlet time.

STEP 4: Determine for each dependent subarea the ditch time (or transit time) ditch time from the upper outlet to the lower outlet.

- Culvert flow: Assume 300 fpm, unless you can be more precise.
- Ditch flow: Use Figure 12.

Specific Q Determination

Working systematically, start at the uppermost subarea and proceed downstream (subarea X is upstream of subarea Y).

Determine the subarea TOC (TOC, using the new definition) using these simple rules.

- Rule for an independent subarea: $TOC_X = \text{Inlet time}_X$
- Rule for a dependent subarea: First compare Inlet time_X and $\text{Inlet time}_{X-Y} = TOC_X + DT_{X-Y}$. Then select TOC_Y equal to the larger value based on the comparison.

Adjust I60 minute, 10 year to $I_{TOC, 10 \text{ year}}$.

Calculate Q for subarea X: $Q_X = C_X I_{TOC_X} A_X$

Proceed downstream to subarea Y and repeat Steps 1, 2, and 3.

Sum the accumulated runoff to get the total runoff at the outlet of subarea Y: $Q_{\text{outlet Y}} = Q_X + Q_Y$

Continue working downstream until the runoff at the lowest outlet in the series is calculated.

Remember that there is a limitation of 1,000 acres when using the Rational Method. Always check the accumulated acreage to ensure it does not exceed 1,000 acres.

As you can see from the summary above, the method of calculating runoff for successive areas differs from the Rational Method in that each subarea must be delineated separately and the TOC must be obtained for each individual subarea.

The schematic example (see Figure 19) used every step outlined in the successive areas summary. Numerical values must of course, be found for each variable C_X , I_X , and A_X , use the procedures outline in the sections above.

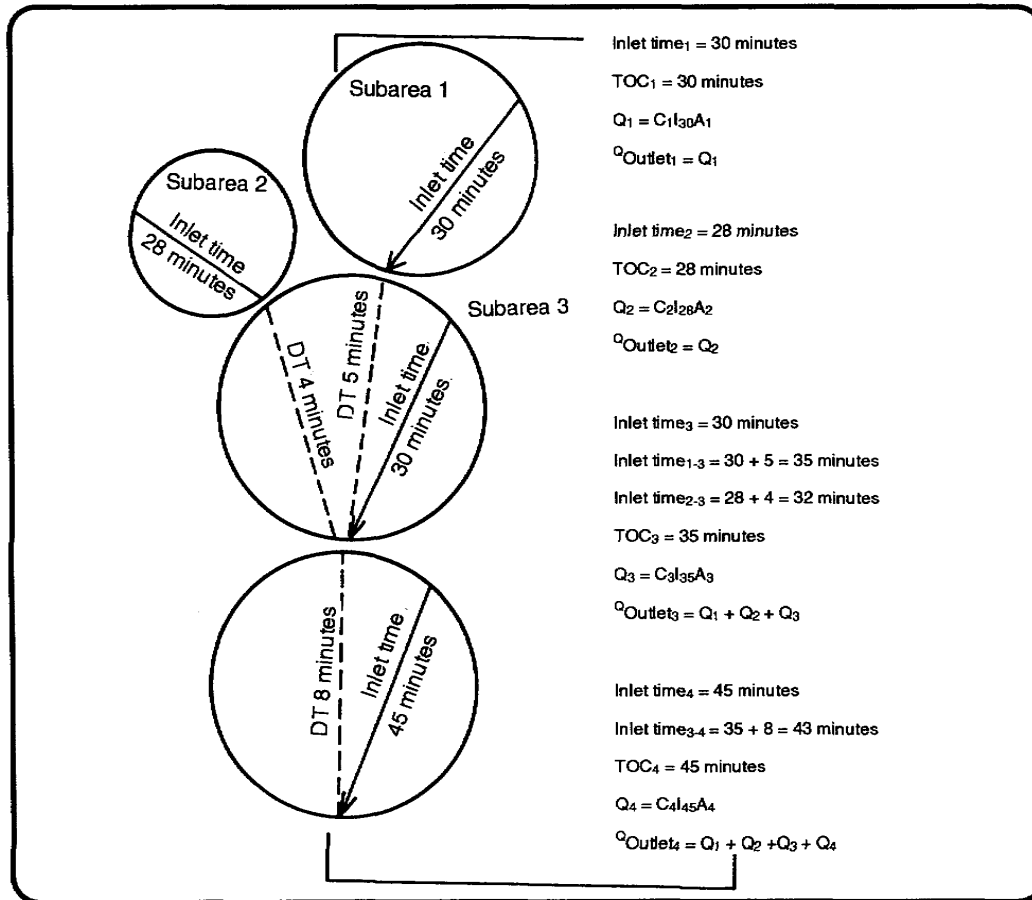


Figure 19. Schematic example of successive areas

HYDRAULIC THEORY AND THE EQUATION OF CONTINUITY AS IT APPLIES TO OPEN CHANNEL DESIGN

FLUID FLOW TYPES

Any discussion of hydraulics must begin with a study of the types of fluid flow.

It is reasonable to assume that to design an open channel capable of carrying the quantity of water (Q) without erosion or deposition problems, it is necessary to understand the flow of water in the channel. Fluid flow occurs in two states: *laminar flow* or *turbulent flow*.

Laminar Flow

In laminar flow, the water moves in parallel layers with no cross currents. Osborne Reynolds showed laminar flow in his classical dye experiment in overland or sheet flow as an example of this type of flow in nature. The chief advantage of describing the flow as laminar is that the total mathematics theory has been developed to describe it.

Turbulent Flow

In turbulent flow, the water is mixed thoroughly with velocity fluctuations in all directions. Almost all the flow in nature is turbulent. A good example of turbulent flow is the flow in a mountain stream.

Turbulent flow does not have a precise mathematical theory describing the flow characteristics. Most of the theory is based upon empirical equations such as Manning's Equation.

FLOW ASSUMPTIONS

The state of flow is dependent on the strength of the viscosity forces or the thickness of the fluid. (Honey is very viscous while water is not.) *Laminar flow* is flow where the viscosity forces predominate and the fluid flows in smooth paths; for example, honey being poured out of a jar. *Turbulent flow* occurs when the viscous forces are relatively weak. It is characterized by individual water particles moving in random patterns while the aggregate motion represents the forward movement of the fluid. Natural channels contain turbulent flow. (Normally, the only time laminar flow in water occurs in nature is in thin films called *sheet flow* where the depth is extremely small.)

First Assumption

A basic assumption in open channel design is that the flow in the channel is steady. This means the depth is not varying rapidly with time. Although this assumption is not absolutely true in practice, generally changes in flow in open channels are slow and the errors introduced by this assumption are very small.

Second Assumption

Another basic assumption is that flow in the channel is uniform. The depth throughout a section of channel of constant dimension is constant. This assumption is found to be essentially correct for channels of reasonable slope that are not extremely short. This means the slope of the water surface is the same as the channel bottom. Flows over spillways or waterfalls are not uniform, while flows in rivers, ditches, and canals are the uniform type.

Third Assumption

The third assumption is *continuity of flow*. The equation of continuity states that within the section under consideration, energy is neither created nor destroyed, thus the principle of conservation of mass holds.

The equation is expressed as $Q = AV$ where:

Q = Rate of flow in cfs.

A = Cross-sectional area in square feet (sf).

V = Velocity in feet per second (fps).

Remember how to determine Q ($Q=CIA$). However, when designing an open channel, use the equation of continuity: $Q = AV$ to check that the design is the right size to carry the water.

MANNING'S EQUATION

The size and gradient of the channel used to carry the design storm runoff may be determined by Manning's Equation for velocity which states:

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

Where: V = Velocity of flow in fps.

R = Hydraulic radius = $\frac{\text{Cross-section area of the channel}}{\text{Wetted perimeter (feet)}}$

S = Slope of the channel in feet per foot (ft/ft), also called hydraulic slope.

“n” = Roughness coefficient or friction factor which depends on the material lining the channel.

Runoff water is responsible for the majority of construction failures. The engineer must be able to design and construct ditches that will carry the proper amount of runoff without undesirable side effects.

Since design of the open channel depends so much on the use of Manning's Equation, this lesson will discuss variables in greater detail.

VELOCITY OF FLOW, V

Channels may have different side slopes and cross-sectional areas of flow and still have the same rate of runoff on the same longitude slope, but the velocity of flow in each ditch will be different. Since excess velocity will cause erosion in a ditch and damage to adjacent structures, it must be held within certain limits. The maximum permissible velocity depends entirely on the soil in which the ditch is constructed. Look at the maximum permissible velocities on Table 8.

Table 8. Values for Manning's "n" and maximum permissible velocities for channel Design

Ditch Lining			Manning's "n"	V _{max} fps	
1. Natural earth without vegetation					
Rock, smooth and uniform					
Rock, jagged and irregular			0.035—0.040	20	
Soils with vegetation			0.040—0.045	15—18	
Coarse grained	Gravel and gravelly soils	GW	0.022—0.024	6—7	
		GP	0.023—0.026	7—8	
		GM	d	0.023—0.025	3—5
			u	0.022—0.024	2—4
		GC	0.024—0.026	5—7	
	Sand and sandy soils	SW	0.020—0.024	1—2	
		SP	0.022—0.024	1—2	
		SM	d	0.020—0.023	2—3
			u	0.021—0.023	2—3
		SC	0.023—0.025	3—4	
Fine grained	Silts and clays	LL < 50	CL	0.022—0.024	2—3
			ML	0.023—0.024	3—4
			OL	0.022—0.024	2—3
		LL ≥ 50	CH	0.022—0.023	2—3
			MH	0.023—0.024	3—5
			OH	0.022—0.024	2—3
			PT	0.022—0.025	2—3
Highly organic					
Average turf, erosion resistant soil			0.050—0.070	4—5	
Average turf, easily eroded soil			0.030—0.050	3—4	
Dense turf, erosion resistant soil			0.070—0.090	6—8	
Dense turf, easily eroded soil			0.040—0.050	5—6	
Clean bottom with bushes on side			0.050—0.080	4—5	
Channel with tree stumps, no sprouts			0.040—0.050	5—7	
Channel with tree stumps, with sprouts			0.060—0.080	6—8	
Dense weeds			0.080—0.120	5—6	
Dense brush			0.100—0.140	4—5	

Table 8. Values for Manning’s “n” and maximum permissible velocities for channel design (continued)

	Ditch Lining	Manning's "n"	V_{max} fps
	Dense willows		0.150—0.200
	Construction		V_{max} fps
	Good	Poor	
2. Paved			
Concrete with all surfaces, trowel finish	0.012	0.014	20
Concrete with all surfaces, float finish	0.013	0.015	20
Concrete with all surfaces, formed, no finish	0.014	0.016	20
Concrete bottom, float finished with sides of dressed stone in mortar	0.015	0.017	18—20
Concrete bottom, float finished with sides of random stone in mortar	0.017	0.020	17—19
Concrete bottom, float finished with sides of dressed stone or smooth concrete rubble (riprap)	0.020	0.025	15
Concrete bottom, float finished with sides of rubble or random stone (riprap)	0.025	0.030	15
Gravel bottom, sides of formed concrete	0.017	0.020	10
Gravel bottom, sides of random stone in mortar	0.020	0.023	8—10
Gravel bottom, sides of random stone or rubble (riprap)	0.023	0.033	8—10
Brick	0.014	0.017	10
Asphalt	0.013	0.016	18—20

For a given rate of runoff and a given longitudinal slope, the velocity of flow may be controlled by the shape of the ditch, specifically the ditch side slope, and the ditch lining. Generally, a broad, shallow cross-section is desirable.

HYDRAULIC SLOPE, S

The hydraulic slope will be considered as the slope of the channel bottom, or longitudinal slope. S is normally determined by the natural slope of the site being drained. The minimum grade satisfactory for clear channels is 0.5 percent of S/0.005 ft/ft. On grades not exceeding 2 percent, checkdams may be used. On grades exceeding 8 percent or where erosion is a major problem, as in sand cuts, ditches should be paved.

DITCHES AND FUNCTIONS

The following are the three basic ditches and the functions they perform on a construction project.

An interceptor ditch is located on the hillside above a structure. Its function is to intercept runoff before it reaches the structure and direct its flow to a desired location. Generally it is used above the hill cuts to decrease erosion on the cut slope.

A diversion ditch carries water away from structures to low areas. It is often used in conjunction with interceptor

ditches.

A side ditch is located along the side of a road. It collects water that runs off the road or surrounding ground and transports it to a drainage structure or diversion ditch.

These ditches are illustrated in Figure 20.

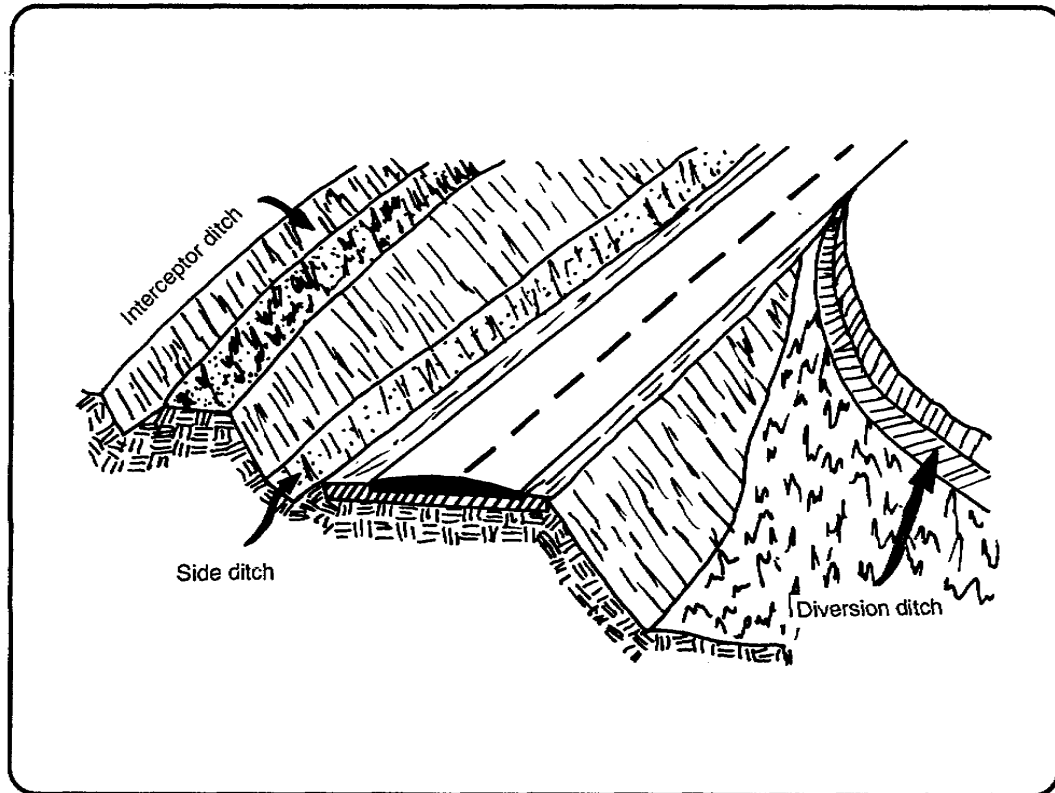


Figure 20. Ditch determined by function

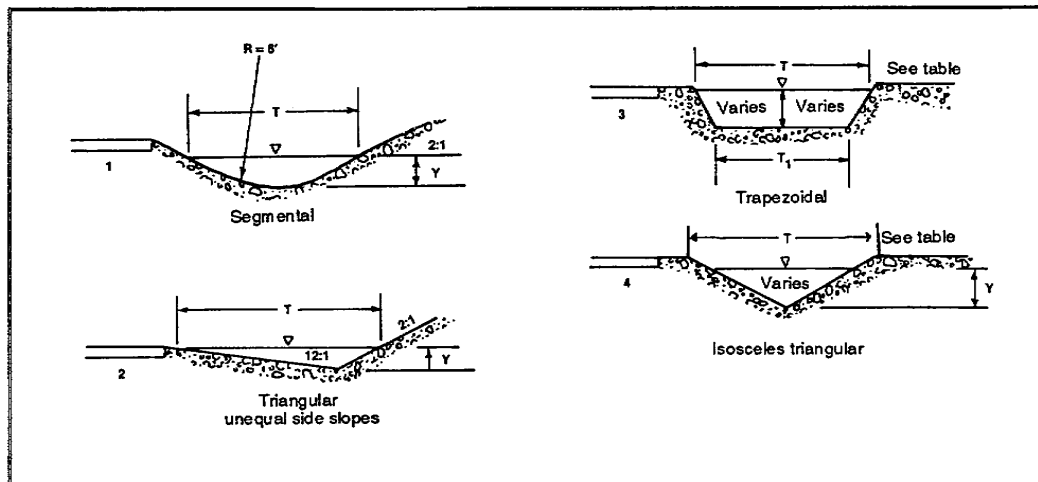
DITCH SHAPES OR SECTIONS

Channels or ditches are classified by their shapes. The five basic types are-

- Symmetrical triangular or V
- Nonsymmetrical triangular or V
- Symmetrical trapezoidal
- Nonsymmetrical trapezoidal
- Segmental

Table 9 shows the commonly used shapes with corresponding data to use with Manning's Equation. Any of these shapes may be used as side interceptor and diversion ditches. The primary consideration in the selection of a cross-section is its capacity to meet hydraulic requirements and the availability of equipment and use.

Table 9. Ditch sections



Drainage Ditches - Common Sections

Properties of Ditches

Number	Dimensions				Hydraulics			
	Side Slopes	Bw	d	Tw	A (Ft ²)	Wp (Ft)	R (Ft)	R ^{2/3}
1	—	—	6 1/2"	5' 0"	1.84	5.16	0.356	0.502
2	12:1 & 2:1	—	6"	7' 0"	1.75	7.14	0.245	0.392
3-A	1 1/2:1	2' 0"	1' 0"	5' 0"	3.50	5.61	0.624	0.730
3-B	2:1	2' 0"	1' 0"	6' 0"	4.00	6.47	0.618	0.726
3-C	3:1	2' 0"	1' 0"	8' 0"	5.00	8.32	0.601	0.712
3-D	1 1/2:1	3' 0"	1' 6"	7' 6"	7.88	5.41	0.937	0.958
3-E	2:1	3' 0"	1' 6"	9' 0"	9.00	9.71	0.927	0.951
3-F	3:1	3' 0"	1' 5"	12' 0"	11.25	12.49	0.901	0.933
3-G	1 1/2:1	3' 0"	2' 0"	9' 0"	12.00	10.21	1.175	1.114
3-H	2:1	3' 0"	2' 0"	11' 0"	14.00	11.94	1.173	1.112
3-I	3:1	3' 0"	2' 0"	15' 0"	18.00	15.65	1.150	1.097
3-J	1 1/2:1	4' 0"	3' 0"	13' 0"	25.50	14.82	1.721	1.436
3-K	2:1	4' 0"	3' 0"	16' 0"	30.00	17.42	1.722	1.437
3-L	3:1	4' 0"	3' 0"	22' 0"	39.00	22.97	1.698	1.423
4-A	2:1	—	1' 0"	4' 0"	2.00	4.47	0.447	0.584
4-B	3:1	—	1' 0"	6' 0"	3.00	6.32	0.475	0.609
4-C	2:1	—	2' 0"	8' 0"	8.00	8.94	0.895	0.929
4-D	3:1	—	2' 0"	12' 0"	12.00	12.65	0.949	0.965
4-E	2:1	—	3' 0"	12' 0"	18.00	13.42	1.341	1.216
4-F	3:1	—	3' 0"	18' 0"	27.00	18.97	1.423	1.265
4-G	7:1	—	1' 0"	14' 0"	7.00	14.14	0.495	0.626
4-H	7:1	—	2' 0"	28' 0"	28.00	28.28	0.990	0.993
4-I	7:1	—	3' 0"	42' 0"	63.00	42.43	1.485	1.302

Where: Bw = Bottom width
d = Depth of flow

Tw = Top width of ditch

A = Area of ditch flow

Wp = Wetted perimeter
R = Hydraulic radius

Some of the factors in determining ditch shape are discussed below.

The triangular or V-shaped ditch is good for flows of less than 60 cfs. A 3:1 side slope allow vehicles to get out of

the ditch easily and prevent accidents related to overturned vehicles.

The trapezoidal ditch is good for flows larger than 60 cfs. A 1:1 side slope are sufficient where there is no possibility of the ditch being a vehicle hazard.

A 60 cfs limit between triangular and trapezoidal ditches should be used only as a guide. The type of ditch that is used will also depend on the equipment available to construct the ditch. For example, if scrapers are not available, then the construction of a trapezoidal ditch may be much more difficult than a large triangular ditch cut with a grader.

The *symmetrical ditch* is either a V or trapezoidal configuration, and both are suitable for interceptor, diversion, or main channels between culverts.

The *nonsymmetrical ditch* is used on road side ditches to prevent vehicle hazard and to decrease the possibility of erosion on the side of the ditch closest to the road. It can be used in either the V or the trapezoidal configuration. The 3:1; 1:1 nonsymmetrical V is widely used since the 1:1 backslope does not have to be worked by the grader. When the 3:1 side is cut, the 1:1 slope will be cut automatically by the front edge of the blade, thus the grader only works the ditch from one side--the side closest to the road structure.

A segmental ditch is a ditch cut by explosives. It is suitable for swampy areas where equipment is unable to work.

HOW TO PREVENT EROSION

Erosion presents many difficult problems in designing open channels. Erosion tends to change the shape of channels, which can cause many problems near structures such as roads, bridges, and culverts. Erosion also presents problems when the eroded material is deposited where it is not wanted.

The primary cause for erosion is the velocity of the water in the channel exceeding the velocity at which soil of the channel will erode, "n". The primary method for preventing erosion is to lower the velocity in the channel below the soil erosion velocity.

Checkdams are the most common structure used to reduce velocity in ditches that have longitudinal grades not exceeding 8 percent.

Note that in Manning's Equation the velocity is dependent on the slope of the channel. If the slope is increased, the velocity increases. One method of decreasing the velocity of the water in a channel to decrease erosion is to decrease the slope. Since constructing a new ditch at a different slope is impractical, the most common approach is to construct a checkdam. It works by decreasing the slope of the water surface. Eventually soil is deposited on the ditch bottom and the ditch slope itself is decreased.

Checkdams should be considered when the slope ranges between 2 and 8 percent. Channels with slopes of 2 percent or less generally do not require extensive erosion controls. Usually with slopes in excess of 8 percent, it is more economical to pave the ditch with asphalt or concrete material.

Checkdams should extend at least 24 inches into the bottom and sides of the ditch. The top of the ditch should be at least 12 inches above the top of the checkdam. The effective height of the checkdam should be at least 12 inches, but not more than 36 inches. An apron of rubble or riprap should extend at least 4 feet from the face of the checkdam on the discharge side. A weir notch must be cut in the top of the checkdam with a capacity large enough to discharge the anticipated runoff.

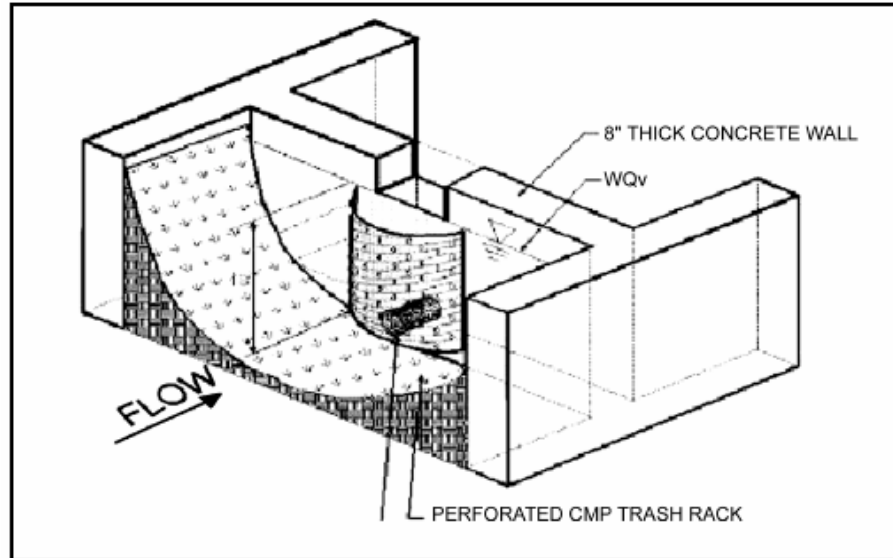


Figure 21. Concrete Checkdam

The spacing between checkdams can be found by using the following formula:

$$S = \frac{100(H)}{A-B}$$

Where:

- S = Spacing in feet between checkdams. This value should not be less than 50 feet for a practical solution
- H = The height of the bottom of the weir notch from the channel bottom. This value should not be greater than 3 feet unless the dam is to be structurally designed. The practical lower limit for H is 1 foot to prevent unnecessary work.
- A = Slope of the original ditch in percent
- B = Desired slope in percent. This value should be set at 2 percent as this is the maximum slope that will not require additional erosion control.

For H = 3 feet, S = 50 feet, and B = 2 percent, we can solve for the maximum grade for which simple checkdams can be used.

$$50 = \frac{100(3)}{A-2} \qquad A = 8 \text{ percent}$$

It is considered uneconomical to space checkdams closer than 50-foot intervals. An alternate solution should be considered for shorter intervals, such as the use of concrete, timber, or boulder linings. For slopes over 8 percent, consideration should be given to paving the channel.

Example: Original slope = 5 percent; chosen desired slope = 2 percent; and chosen H = 3 feet. Find S.

$$S = \frac{100(H)}{A-B}$$

$$S = \frac{100(3)}{5-2} = \frac{300}{3} = 100 \text{ feet}$$

The spacing between checkdams is 100 feet.

The weir notch must be designed to carry the flow in the ditch.

Problems which occur involving checkdams include erosion around the dam. Causes include the weir notch being too small or being clogged with debris so that flow occurs across the top of the dam. This causes erosion where the dam is anchored into the ground, and if continued long enough, will erode around the dam. If this occurs, there will be more damage to adjacent structures than if the dam was never installed.

RIPRAP AND RUBBLE

Since the values of “n” are in the denominator in Manning's Equation, increasing “n” will decrease the velocity. To increase “n” the effective roughness of the ditch must be increased. This can be accomplished by protecting the channel with riprap and rubble.

The method involves placing rocks or rubble in the ditch bottom so that the rocks prevent the soil from being eroded. Usually the rocks should be larger than 6 inches. They should be hand-placed and compacted individually in at least two layers. The riprap not only prevents erosion, but will also tend to decrease the velocity in the channel due to its high “n” value, which makes it useful for lowering velocities in sections of channel when making transitions to soil ditches from paved to other high-velocity ditches.

DECREASING THE HYDRAULIC RADIUS

Decreasing the hydraulic radius (R) will also decrease the velocity in a channel. This can be accomplished by increasing the wetted perimeter (P) in relation to the area (A); in other words, making the ditch wider and shallower. For example, consider the following ditch cross-sections (see Figure 22).

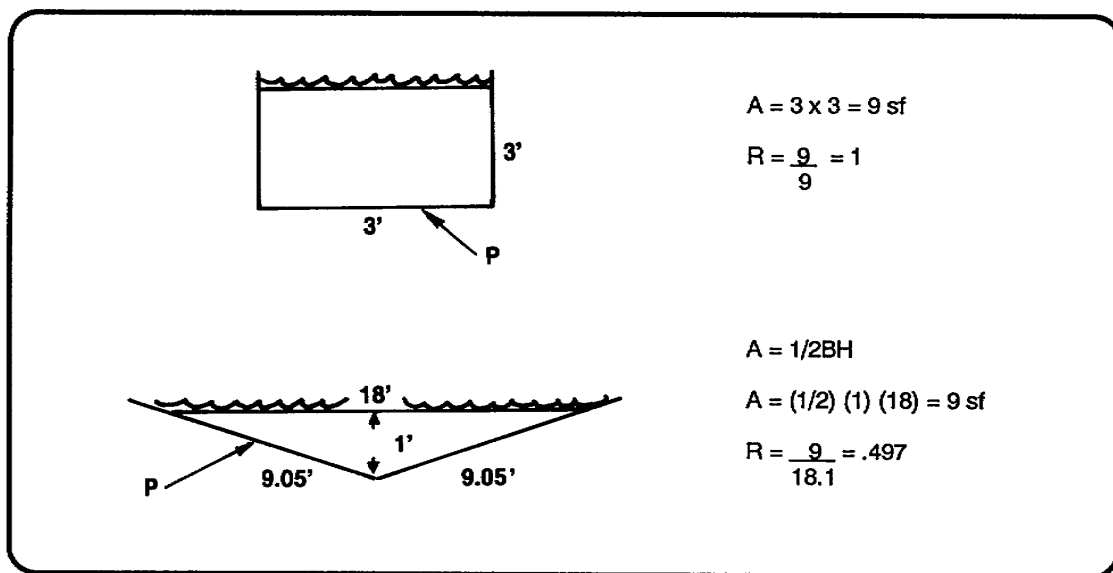


Figure 22. Comparing hydraulic radii

In this example we can see that the triangular ditch has a hydraulic radius approximately one-half of the rectangular ditch and thus would be expected to have a lower velocity since the larger surface area causes greater friction.

PAVEMENT

Another form of channel protection is pavement. Paving the ditch with asphalt or concrete will prevent erosion of the soil. Due to the low “n” values of pavements, the velocity may increase enough to prevent a problem where the pavement in the ditch terminates. Special protection may be required to slow down the velocity before allowing the flow to continue in an earthen channel. Two possible solutions are a stilling basin or riprap. Additionally, due to pavements' low “n” values, it may be used to increase the velocities if they are too low, thus preventing deposition.

Some sort of the channel protection is required if the slope of the channel is above 2 percent, and it may be a good solution for slopes lower than 2 percent with excessive velocities. In any event, channel protection and/or ditch shape may be used together as they generally are the most effective methods of erosion control. Due to the large effort of installation, do not use checkdams unless absolutely necessary.

DESIGNING AN OPEN CHANNEL

OPEN CHANNEL DESIGN

Earlier in this course learned how to use the Rational Method to obtain Q, the quantity of flow in cfs. Q allows you to use the first basic equation necessary to open channel design. It is the equation of continuity, $Q = VA_w$. The second basic equation in Manning's Equation,

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

The variables in these equations are:

- Q = Quantity of flow (cfs)
- V = Velocity of flow (fps)
- A_w = Cross-sectional area of water (square feet (sq ft))
- “n” = Roughness coefficient
- R = Hydraulic radius = $\frac{A_w}{P}$
- S = Slope (gradient) of the channel ft/ft = $\frac{\text{Beginning elevation (ft)} - \text{Ending elevation (ft)}}{\text{Distance}}$

To design an open channel for a known area, a engineer works within the constraints of known variables to decide on the best values for the unknown variables.

The known values are:

- Q-The value of Q is the first step in determining the shape of the ditch. If Q is greater than 60 cfs, then the ditch should be trapezoidal. If Q is less than 60 cfs, then the ditch should be triangular.
- S-Slope generally follows the natural terrain. Small modifications may be made for short sections.
- V-Velocity is determined initially by the type of soil. Check Table 10 for the point at which erosion occurs.

Figure 23 shows the velocity (V) relationships.

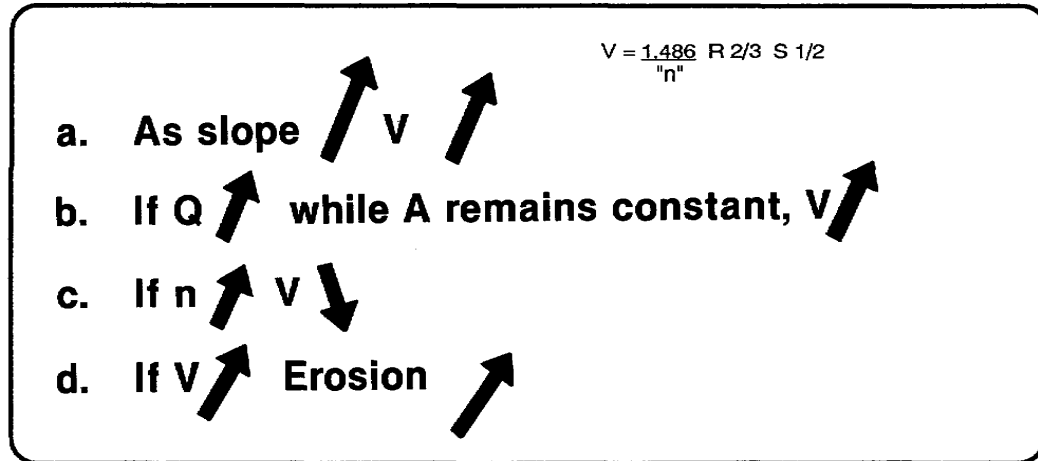


Figure 23. Velocity relationships

Table 10. Hydraulic radius (R) and area (A) of asymmetrical triangular channel

Depth (d) (Feet)	Slope Ratio											
	1:1		1 1/2:1		2:1		2 1/2:1		3:1		4:1	
	A	R	A	R	A	R	A	R	A	R	A	R
0.5	0.25	0.18	0.38	0.21	0.50	0.22	0.63	0.23	0.75	0.24	1.00	0.24
0.6	0.36	0.21	0.54	0.25	0.72	0.27	0.90	0.28	1.08	0.28	1.44	0.29
0.7	0.49	0.25	0.74	0.29	0.98	0.31	1.23	0.32	1.47	0.33	1.96	0.34
0.8	0.64	0.28	0.96	0.33	1.28	0.36	1.80	0.37	1.92	0.38	2.56	0.39
0.9	0.81	0.32	1.21	0.37	1.62	0.40	2.03	0.42	2.43	0.43	3.24	0.44
1.0	1.00	0.35	1.50	0.42	2.00	0.45	2.50	0.45	3.00	0.47	4.00	0.49
1.1	1.21	0.39	1.82	0.46	2.42	0.49	3.03	0.51	3.63	0.52	4.84	0.53
1.2	1.44	0.42	2.16	0.50	2.88	0.54	3.60	0.56	4.32	0.57	5.76	0.58
1.3	1.69	0.46	2.54	0.54	3.38	0.58	4.23	0.60	5.07	0.62	6.76	0.63
1.4	1.96	0.50	2.94	0.58	3.92	0.63	4.90	0.65	5.88	0.66	7.84	0.68
1.5	2.25	0.53	3.38	0.62	4.50	0.67	5.63	0.70	6.75	0.71	9.00	0.73
1.6	2.56	0.57	3.84	0.67	5.12	0.72	6.40	0.74	7.68	0.76	10.24	0.78
1.7	2.89	0.60	4.34	0.71	5.78	0.76	7.23	0.79	8.67	0.80	11.56	0.83
1.8	3.24	0.64	4.86	0.75	6.48	0.80	8.10	0.84	9.72	0.85	12.96	0.87
1.9	3.61	0.67	5.42	0.79	7.22	0.85	9.03	0.88	10.83	0.90	14.44	0.92
2.0	4.00	0.71	6.00	0.83	8.00	0.90	10.00	0.93	12.00	0.95	16.00	0.97
2.5	6.25	0.88	9.38	1.04	12.50	1.12	15.63	1.16	18.75	1.19	25.00	1.21
3.0	9.00	1.06	13.50	1.25	18.00	1.34	22.50	1.39	27.00	1.42	36.00	1.46
3.5	12.25	1.24	18.38	1.45	24.50	1.56	30.62	1.62	36.75	1.66	49.00	1.70
4.0	16.00	1.41	24.00	1.66	32.00	1.78	40.00	1.85	48.00	1.90	64.00	1.74
	5:1		6:1		7:1		8:1		9:1		10:1	
	A	R	A	R	A	R	A	R	A	R	A	R
0.5	1.25	0.25	1.50	0.25	1.75	0.25	2.00	0.25	2.25	0.25	2.50	0.25
0.6	1.80	0.29	2.16	0.30	2.52	0.30	2.88	0.30	3.24	0.30	3.60	0.30
0.7	2.45	0.34	2.94	0.35	3.43	0.35	3.92	0.35	4.41	0.35	4.90	0.35
0.8	3.20	0.39	3.84	0.39	4.48	0.40	5.12	0.40	5.76	0.40	6.40	0.40
0.9	4.05	0.44	4.86	0.44	5.67	0.45	6.48	0.45	7.29	0.45	8.10	0.45
1.0	5.00	0.49	6.00	0.49	7.00	0.49	8.00	0.50	9.00	0.50	10.00	0.50
1.1	6.05	0.54	7.26	0.54	8.47	0.55	9.68	0.55	10.89	0.55	12.10	0.55
1.2	7.20	0.59	8.64	0.59	10.08	0.59	11.52	0.60	12.96	0.60	14.40	0.60
1.3	8.45	0.64	10.14	0.64	11.83	0.64	13.52	0.64	15.21	0.65	16.90	0.65
1.4	9.80	0.69	11.76	0.69	13.72	0.69	15.68	0.69	17.64	0.70	19.60	0.70
1.5	11.25	0.74	13.50	0.74	15.75	0.74	18.00	0.74	20.25	0.75	22.50	0.75
1.6	12.80	0.78	15.36	0.79	17.92	0.79	20.48	0.79	23.04	0.80	25.60	0.80
1.7	14.45	0.83	17.34	0.84	20.23	0.84	23.12	0.84	26.01	0.84	28.90	0.85
1.8	16.20	0.88	19.44	0.89	22.68	0.89	25.92	0.89	29.16	0.89	32.40	0.90
1.9	18.05	0.93	21.66	0.94	25.27	0.94	28.88	0.94	32.49	0.94	36.10	0.95
2.0	20.00	0.98	24.00	0.99	28.00	0.99	32.00	0.99	36.00	0.99	40.00	1.00
2.5	31.25	1.25	37.50	1.23	43.75	1.24	50.00	1.24	56.25	1.24	62.50	1.24
3.0	45.00	1.47	54.00	1.48	63.00	1.48	72.00	1.49	81.00	1.49	90.00	1.49
3.5	61.25	1.72	73.50	1.72	85.75	1.73	98.00	1.74	110.25	1.74	122.50	1.74
4.0	80.00	1.96	96.00	1.97	112.00	1.98	128.00	1.98	144.00	1.98	160.00	1.99

STEP BY STEP DESIGN OF A CHANNEL FOR A ROADSIDE DITCH

Given the following information.

Design Example. Problem: Design a diversion ditch using the given information.

Known: Location-Northern California

Q = 27.0 cfs (from Rational Method)

Soil = GMd

STEP 1. Determine Q for the basin, given 27 cfs.

STEP 2. Determine the slope of the channel.

Slope = 0.5 percent = 0.005 ft/ft (invert elevation of No 4 = 52.0 ft; mean elevation of swamp = 49.5 ft; change in elevation = 52.0 - 49.5 = 2.5 ft; length = 500 ft; therefore, slope = 2.5/500 = 0.005 ft/ft)

STEP 3. Select the trial value for “n” and V from Table 8.

NOTE: For “n” and V, use mean values.

n = 0.024 (from Table 8; use average of “n” values range, unless additional information indicates otherwise)

V = 3-5 fps (from Table 8 on page 4-6) (Use the mean value for V_{max}.)

STEP 4. Determine the hydraulic radius using “n” and V from STEP 3 and slope (S) from STEP 2. Using Figure 24, page 4-7 or the equation, determine the hydraulic radius.

Table 11. Ditch selection

Ditch Type	Selection Side Slope	Given V	Nomograph R	Table 4-3 (page 4-14)			Calculate Q cfs
				R	A-sq ft	d-ft	
V Sym	3:1						

STEP 5. Determine the type of ditch based on the Q design.

Select Ditch. Since the Q design (QD) is less than 60 cfs, the ditch should be triangular. Side slopes for the triangular ditch should be 3:1 as a first trial. They can be changed if the ditch is found to be not suitable. Enter this information in the columns under the ditch selection in the ditch design work sheet. Also, the ditch should be symmetrical (Sym), since it is not adjacent to a road.

Select Velocity. The erosion velocity for the GMd soil is 3-5 fps. This means that at 3 fps the soil in the ditch will begin to erode and at 5 fps the whole ditch will be eroding. Since we do not want to exceed the V_{max} of 5 fps, the best initial choice usually is 1 fps lower than V_{max} or 4 fps.

Table 12. First trial velocity

Ditch	Selection	Given	Nomograph	Table 4 (page 4-14)			Calculate
Type	Side Slope	V	R	R	A-sq ft	d-ft	Q cfs
V Sym	3:1	4.0					

Using the nomograph in Figure 24, we must first locate a turning point on the turning line. This line is the center line on the nomograph. The turning point is found by locating the slope(s) in ft/ft in the column at the far left of the chart and the roughness coefficient (“n”) on the far right column and connecting them with a straight line. Where the line crosses the turning line is the turning point. This turning point will remain the same as long as S and “n” do not change.

NOTE: Another method using the Manning Equation with a calculator is shown below.

The hydraulic radius is found by connecting the velocity in the second column from the right, and the turning point by a straight line through to the R scale and reading the hydraulic radius (R) off the second column from the left. This gives the required R for any given V in Manning's Equation. In this example, R equals 0.87 which shows the calculator method for solving the radius.

Table 13. Calculated radius

Ditch	Selection	Given	Nomograph	Table 4-3 (page 4-14)			Calculate
Type	Side Slope	V	R	R	A-sq ft	d-ft	Q cfs
V Sym	3:1	4.0	0.87				

Determining the Hydraulic Radius Using Manning’s Equation

If your calculator has the function \sqrt{x} and y^x , you can determine R more quickly and accurately than with the nomograph provided in the workbook Transposing Manning's Equation,

$$V = \frac{1,486 R^{2/3} S^{1/2}}{“n”} \rightarrow R = \left[\frac{V “n”}{1.486 \sqrt{S}} \right]^{3/2}$$

Since n and S are constant for various trial iterations, you need only enter different values for V to arrive at an acceptable value for R. Therefore, if S and n are constant, the above equation for R becomes:

$$R = [V \times K]^{1.5} \text{ where } K = \frac{“n”}{1.486 \sqrt{S}}$$

To calculate R, use the above equation for numerous iterations until you have “bracketed” flowrate Q within acceptable limits. Below are keystrokes which will work on most calculators.

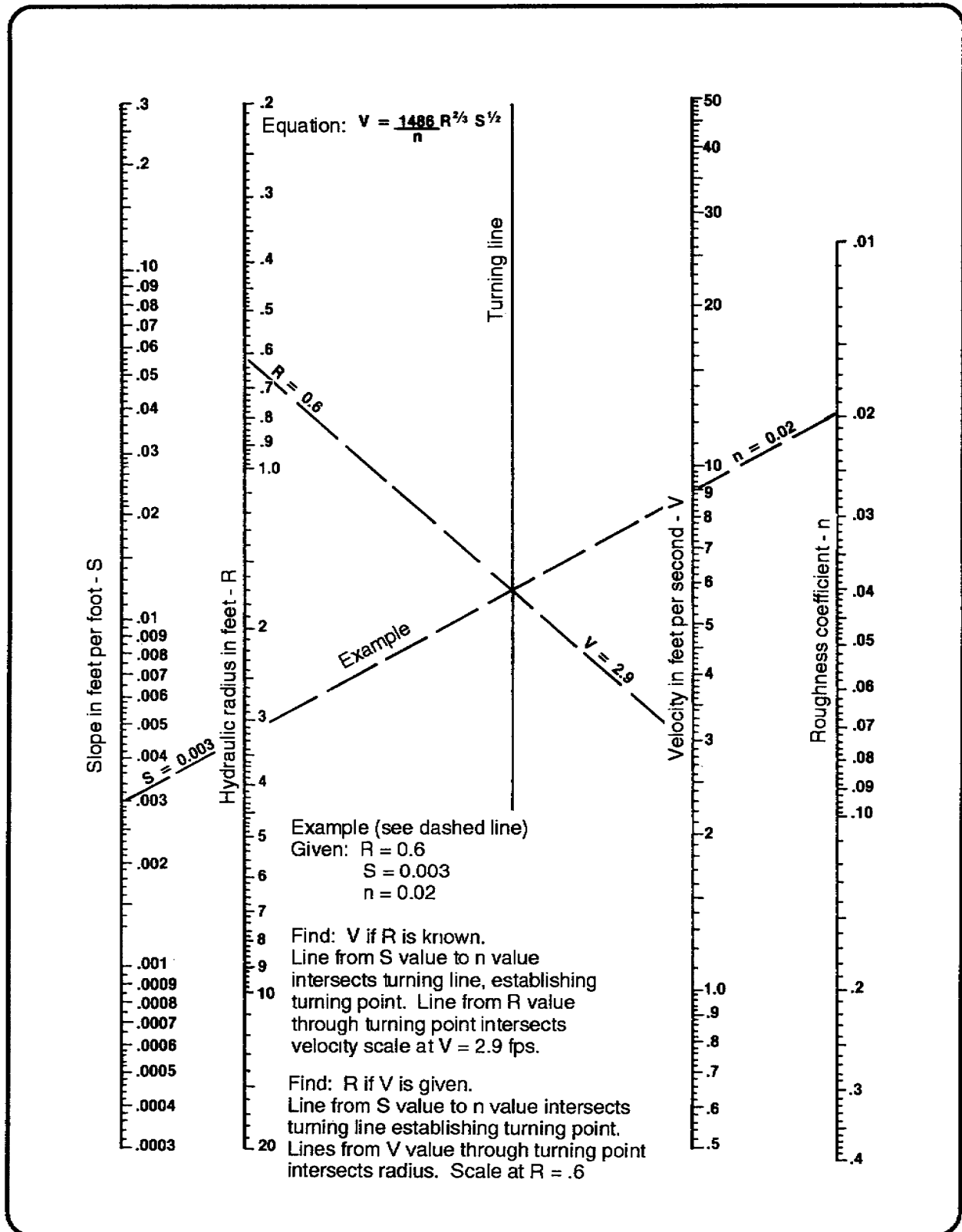


Figure 24. Nomograph for Manning's Equation

STEP A: First, input and store values for K:

.xxx	Enter value for S
\sqrt{x}	Square root of S
x 1.486	Times constant of 1.486
= 1/x	Calculate and invert denominator
x .xxx	Times value for n
= STO or M+	Store constant K

STEP B: To calculate R for various values of V:

RCL or MRC	Recall K
x V	Times value for velocity
=	Equals
Y ^x 1.5	Raise product to 3/2 power
=	Displays value for R

NOTE: RCL stands for recall memory and MRC stands for clear memory.

STEP C: Repeat Step B for different V values until you reach an acceptable R and Q.

Find Hydraulic Radius in Ditch Table

STEP 6. Select the appropriate hydraulic radius from the appendix.

Use the ditch table, Table 10. There are four types of tables; V symmetrical, V nonsymmetrical, trapezoidal symmetrical, and trapezoidal nonsymmetrical. For this problem, use Table 9.

Locate the columns representing the side slopes of the ditch being designed, for example, 3:1.

Locate the R values that fall above and below, or the one that is exactly equal to the R value found from the nomograph of Manning's Equation. In the example, the values are 0.85 and 0.90. **USE THE LOWER VALUE!** For example, use 0.85.

Table 14. Selected appendix radius

Ditch Type	Selection Side Slope	Given V	Nomograph R	Table 4-3 (page 4-14)			Calculate Q cfs
				R	A-sq ft	d-ft	
V Sym	3:1	4.0	0.87	0.85			

Record Area and Depth

Associated with R = 0.85 the area is 9.72 square feet and the depth (d) found in the column at the far left is 1.8 feet. Record these values under the appropriate headings in the ditch design work sheet.

Table 15. Selecting the area and depth

Ditch Type	Selection Side Slope	Given V	Nomograph R	Table 4-3 (page 4-14)			Calculate Q cfs
				R	A-sq ft	d-ft	
V Sym	3:1	4.0	0.87	0.85	9.72	1.8	

Check for Q

STEP 7. Calculate the ditch Q using the Continuity Equation and the cross-sectional area (A) and the depth (d) from the appendix.

Make a check to see if this particular ditch will meet the requirements of our design by performing the calculation using the Continuity Equation.

Table 16. Q calculation

Ditch	Selection	Given	Nomograph	Table 4-3 (page 4-14)			Calculate
Type	Side Slope	V	R	R	A-sq ft	d-ft	Q cfs
V Sym	3:1	4.0	0.87	0.85	9.72	1.8	38.9

$$Q_c = AV$$

$$Q_c = (9.72 \text{ sq ft})(4 \text{ fps})$$

$$q_c = 38.9 \text{ cfs}$$

- Trial Number 1. This calculated quantity of flow (Q_c) must fall within +5 percent of the design Q_d .

If this requirement is not met, then a new velocity should be tried. If the Q_c is higher than Q_d , then a lower velocity than the previous trial should be used. For this example, since 38.8 cfs is greater than 28.35 cfs, the upper limit of the acceptable range, a velocity of 3.5 fps would be appropriate for the next trial.

- Trial Number 2

STEP 2 $V = 3.5 \text{ fps}$

STEP 3 $R = 0.72$

STEP 4 $R = 0.71$

STEP 5 $A = 6.75; d = 1.5 \text{ ft}$

STEP 6 $Q_c = (3.5 \text{ fps})(6.75 \text{ sq ft}) = 23.6 \text{ cfs}$

STEP 7 $Q_c = 25.65$, so the next trial velocity must be larger than 3.5 fps but smaller than 4.0 fps. Try 3.6 fps.

Table 17. Second trial velocity

Ditch	Selection	Given	Nomograph	Table 4-3 (page 4-14)			Calculate
Type	Side Slope	V	R	R	A-sq ft	d-ft	Q cfs
V Sym	3:1	4.0	0.87	0.85	9.72	1.8	38.9
	3:1	3.5	0.71	0.71	6.75	1.5	23.6

- Trial Number 3

STEP 2 $V = 3.6 \text{ fps}$

STEP 3 $R = 0.76$

STEP 4 $R = 0.76$

STEP 5 $A = 7.68; d = 1.6 \text{ ft}$

STEP 6 $Q_c = (3.6 \text{ fps}) (7.68 \text{ sq ft}) = 27.6$

STEP 7 $27.6 > 25.65 < 28.35$, so this depth is within the range and thus defines the ditch water depth, area, and velocity.

Table 18. Third trial velocity

Ditch Type	Selection Side Slope	Given V	Nomograph R	Table 4-3 (page 4-14)			Calculate Q cfs
				R	A-sq ft	d-ft	
V Sym	3:1	4.0	0.87	0.85	9.72	1.8	38.9
	3:1	3.5	0.7 - 0.71	0.71	6.75	1.5	23.6
	3:1	3.6	0.74	0.76	7.68	1.6	27.6

Determine if Ditch is Appropriate

The process tells us that a symmetrical triangular ditch, in GMd soil with a slope of .005 ft/ft and 3:1 side slopes, that carries 27.0 cfs runoff will have a water depth of 1.6 feet and a velocity of 3.6 fps.

Since the velocity of the ditch is greater than 3 fps, the ditch can be considered self-cleaning. The peak runoff will remove the silt buildup from the channel bottom; thus, it requires little maintenance. With V at 3.6 fps, there will be some erosion of the ditch but it should not be a large maintenance problem. Therefore, the ditch is deemed appropriate and should be used.

Determine Cutting Depth

STEP 8. Calculate the cutting depth (d) and draw a sketch of the ditch.

It is desirable to have the water level in the ditch at least 6 inches below the edge of the ditch as a factor of safety, then the cutting depth is the water depth plus 6 inches of free board.

Cutting depth = d + free board

In the example, the cutting depth equals 1.6 feet plus 0.5 feet (6 inches) or 2.1 feet (see Figure 25).

Final Cross-Section

NOTE: It is very important for the engineer to communicate the right design to the equipment operator. This is accomplished by providing a good sketch.

See Figure 25 for an example of a designed ditch.

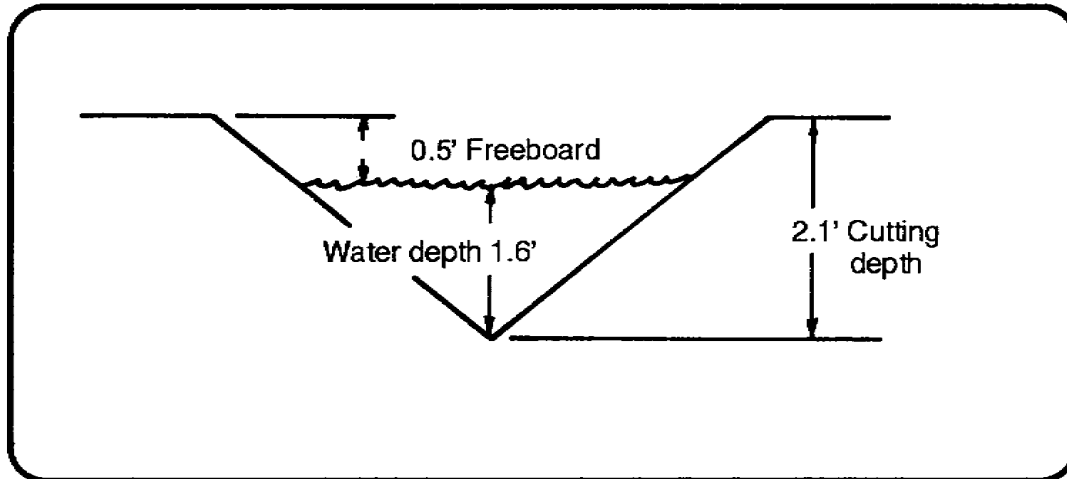


Figure 25. Example of a designed ditch

If the velocity for the ditch had been too high, then another slope ditch with shallower side slopes could be used; say 4:1 or 5:1 to get lower velocities. On the other hand, if the velocity were too low (for example, less than 3 fps), we could raise the velocity by decreasing the side slope to 2:1 or 1:1. If no ditch can be made to work, then a change to some artificial type of channel lining should be considered. If you change the design of the ditch by installing a lining, the size of the ditch must be recalculated by calculating the hydraulic radius and finding a new cross-sectional area and depth.

DESIGN CLOSED CULVERTS

The purpose of this section is to discuss, explain, and illustrate the factors that enter into the design, placement, and construction of culverts.

Before proceeding, it is necessary to define the terms found in general use in culvert practice. Look at Figure 26 for the terms used.

- *Culvert* -- A culvert is an enclosed waterway placed through an embankment or fill. Culverts are used in many ways; to continue natural streams through an intercepting structure, to provide cross drainage in a fill section, to provide ditch relief and to continue side ditches at road intersections. Their efficiency in conveying water, strength, and ease of construction often make them more desirable than small bridges.
- *Fill or embankment* -- The soil mass through which the culvert carries the flow of water.
- *Headwall* -- Retaining wall always employed at the upstream end of culvert; can be used downstream. Its purpose is to support and protect the soil mass at the end of the culvert.
- *Wingwalls* -- Retaining walls set at an angle to the head wall to further support the fill and direct the flow of the water.
- *Crown* -- Outside top of the culvert. When dealing with corrugated metal pipe culverts, the inside and outside elevations of the culvert are assumed to be identical.
- *Invert* -- Inside bottom of pipe.
- *Fill depth* -- The depth of fill from the top of structure to the invert (inside bottom) of the culvert.
- *Cover* -- The depth of fill from the top of structure to the crown outside top of the culvert.

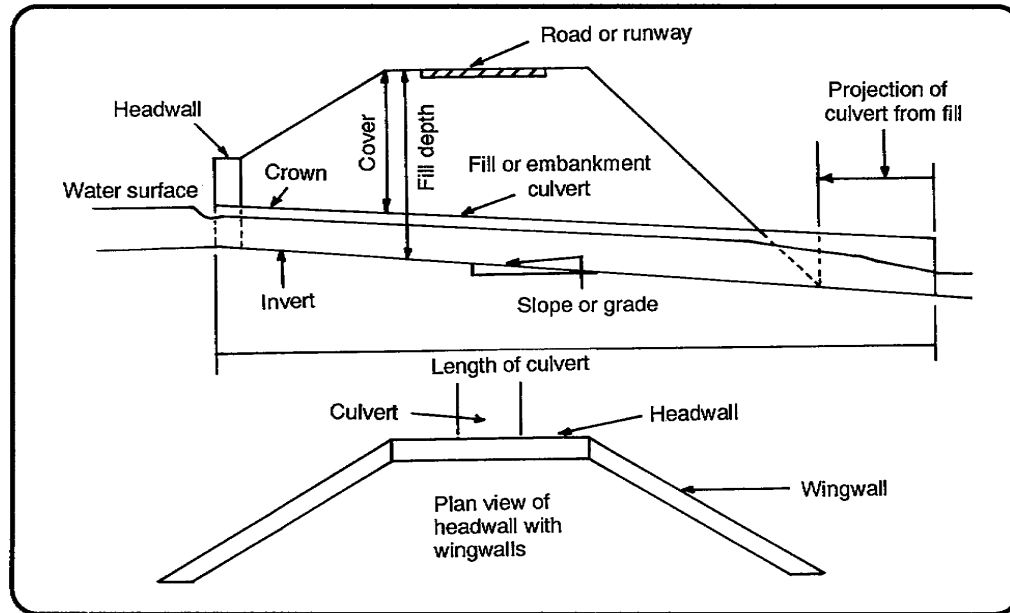


Figure 26. Cross-section of culvert in place

THE THEORY OF CLOSED CHANNEL FLOW

The flow or discharge capacity (Q) of a culvert is the amount of water which the culvert will carry in a unit of time, expressed in cfs. For a particular culvert, Q is controlled by one or more of the following factors:

- The elevation of the culvert inlet.
- The slope or hydraulic gradient (S) of the culvert.
- The length (L) of the culvert.
- The elevation of the tailwater at the culvert outlet.
- The type of inlet. Except for drop inlets, the type of inlet is generally considered in culvert design.

Remember that the discharge capacity of a culvert will be increased, particularly in short culverts on steep slopes, by a smooth-transition type of inlet.

The *Theory of Closed Channel Flow*, like the principles of open channel flow, holds with the conservation of mass; that is, it assumes that the flow is continuous, that there is no ponding, and that the flow occurs at crown elevation (free flowing). Therefore, neither the inlet nor outlet can be submerged. To design a pipe with a given Q the engineer must take into consideration four factors:

- Friction (Manning's roughness coefficient, "n")
- Area
- Slope
- Head (the difference in elevation between each end of the culvert)

FOUR TYPES OF CULVERTS

Culverts are classified as either permanent or expedient (improvised).

Permanent Structures

Concrete box
Concrete pipe
Corrugated metal pipe (CMP)
Pipe arch
Clay or other pipes

Expedient Structures

Lumber
Log
Oil drum
Sand bag with pierced-steel
Plank or logs

In this lesson we are concerned with four types of pipe: CMP, concrete pipes, and concrete box culverts.

CMP

CMP is available in the diameters and gauge listed below:

Gage	Thickness (inches)
8	0.179
10	0.135
12	0.105
14	0.075
16	0.0598

If the soil of the fill has excessive fines, all joints of the CMP should be sealed with tar or asphalt. This will prevent the fine material being carried into the pipe by seepage flowing in the fill. If this action is not taken, large voids can be created that will cause the fill to collapse.

CONCRETE PIPE

Consider using concrete pipe when it is available. Concrete pipe has three advantages over CMP. First, it is stronger, and requires less cover than CMP to support the same load. Second, the interior surface is smoother (Manning's "n" = 0.013). The same diameter concrete pipe set on the same slope as the corrugated metal will carry a higher Q. Third, concrete pipes are easily made and are more readily available in most areas. There are two factors to keep in mind when constructing concrete pipe culverts.

- Concrete pipes require headwalls both upstream and downstream to prevent the pipe sections from separating. A supporting bench or cradle may be used in lieu of the downstream headwall.
- All the joints between the pipes should be sealed with a bituminous material or grout to prevent seepage into and out of the pipe.

CONCRETE BOX

Concrete box culverts have good hydraulic capacity and therefore can be considered for use where it is necessary to utilize the full area of the waterway. Additionally, they can be designed to withstand external loads with little or no cover. They are especially adaptable to culvert sites in rock cuts where bedding may be a problem restricting the use of other types of culverts. A disadvantage is that the design of concrete boxes requires knowledge of reinforced concrete design. The construction requires close supervision because the concrete form work, the placement of the reinforcing steel, and the concrete mix itself is critical.

CULVERT PLACEMENT IN DITCH DESIGN

ALIGNMENT

The relationship of the culvert to the bed of the stream is of major importance. Improper location can cause the stream to seek a path other than through the culvert, and the project may be destroyed. Certain techniques can be used to lessen the probability of this occurring, such as the following:

Place the culvert directly in the stream bed, (see Figure 27(a)) even though it may be diagonal to the structure. If no change is made in the hydraulics of the channel, the stream will not change its direction.

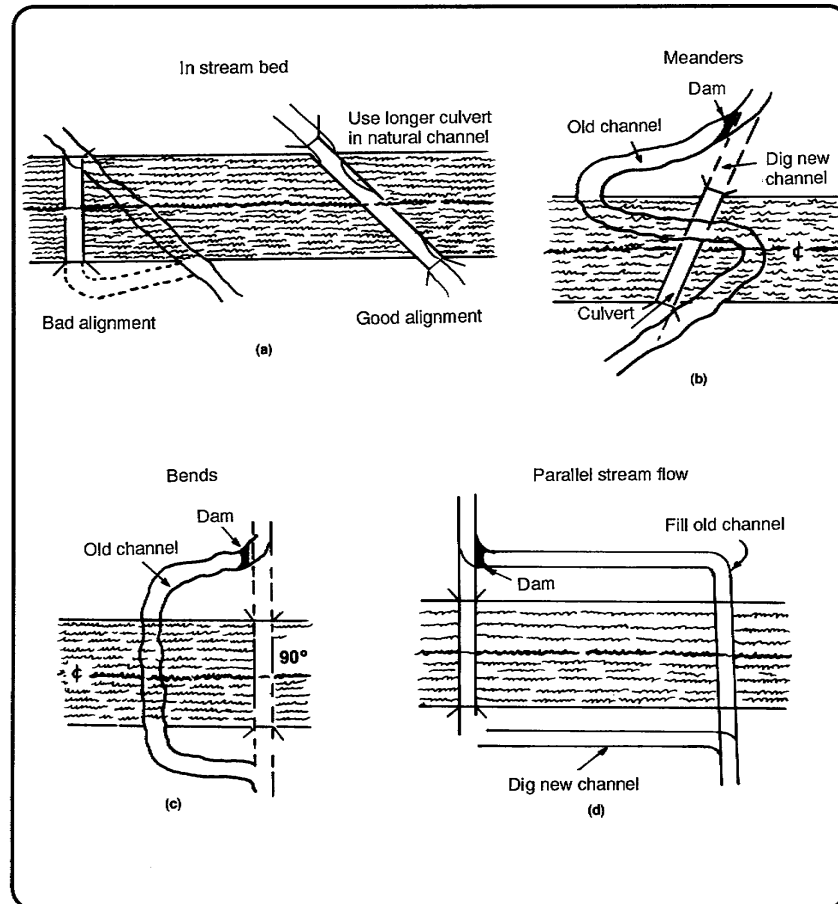


Figure 27. Culvert placement

- Sometimes the structure will cut across a stream meander, as shown in Figure 27(b). In this case, it is best to recut the channel to divert the stream away from the structure. It will be necessary to fill in the old stream bed and place a dam faced with erosion-resistant material (such as rock or rubble) at the junction of the old and the new channel.
- The structure may cut across a bend of the stream, as shown in Figure 27(c), with a straight run of the stream through the structure. The distance from the bend to the structure would determine if there is a problem. If it is far enough that erosion at the bend will not affect the structure, then the culvert can be laid in a straight line. If not, the old channel should be filled with erosion-resistant material at the junction between the old and new channels.
- A bad situation exists when the stream flows close to and parallel to the structure before the crossing as shown in Figure 27(d). The problems in this situation are manifold. Erosion at the bends can cause problems, as noted above. The stream may develop new meander loops and attack the structure. If any ponding occurs, the subgrade may become saturated over a large area,

causing the structure to fail under loads. The best solution is to rechannel the stream as shown. Again, care must be taken to fill in the old stream bed and to provide a faced dam with erosion-resistant material at the junction point.

GRADE (SLOPE)

Normally, culverts are placed on the same slope as the material and artificial drainage channel. However, for culverts to be self-cleaning the velocity of flow should be at least three fps. To achieve this velocity, culverts should not, if practicable, be set on a slope less than 0.4 percent. To prevent excessive velocity, which could cause outlet erosion, the culvert grade should not exceed the critical slope for the particular pipe size being used. Critical slope is the minimum slope of the hydraulic gradient that will permit the maximum Q (flow or discharge).

SPACING OF RELIEF CULVERTS

Culverts should be located wherever natural drainage channels are large enough to require cross drainage. On side-hill roads or where in-place roads intercept surface water, either in cut or in fill, the water is drained to the low side of the road, and, if possible, away from the road by ditch-relief culverts. On 8-percent grades, ditch-relief culverts should be placed about 300 feet apart; on 5-percent grades, 500 feet apart. The distance between the pipes in multiple-pipe culverts should be at least one-half the diameter of the pipe. See Figure 28.

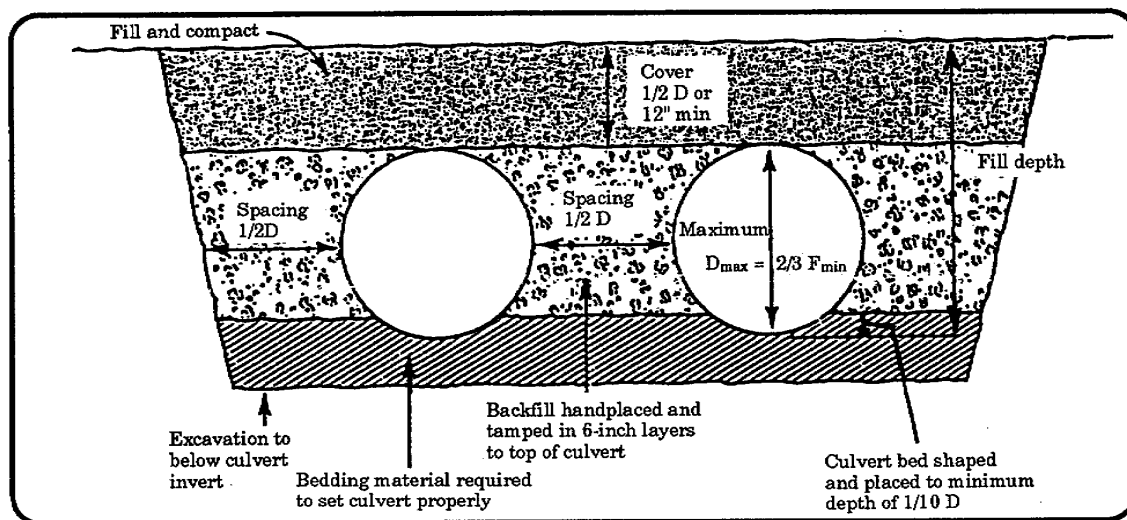


Figure 28. Maximum culvert specifications for CMP for vehicular traffic on roadways only

COVER, BEDDING, AND FILL FOR CMP

Cover

The function of the cover (figures 26, and Figure 28) over culverts is to prevent crushing of the pipe by distributing the live loads over a larger area. The depth of the cover required is a function of the live loads and the culvert diameter and material.

Cover requirements for CMP can be divided into two categories. The minimum amount of cover required to protect a CMP culvert against crushing by vehicular is:

- For vehicles, one-half the diameter of the culvert or 12 inches minimum, whichever is greater

(figure 28).

There is a maximum depth of cover for a particular pipe diameter and gauge and should be determined at the design stage. This requirement is necessary, since the weight of a large soil mass could crush or distort the culvert.

Bedding

Culverts are constructed on a firm, well-compacted soil foundation, except box or arch culverts which may be placed on a rock foundation when suitable rock is encountered. The foundation is always shaped to fit (or to bed) about one-fourth of the outside circumference of the pipe or to a minimum depth of one-tenth the diameter. In addition, foundations for pipe culverts are generally cambered or convexed upward.

The bed is laid so that the invert of the pipe is at grade. The culvert site should be checked to determine the composition of the soil. If the bedding is composed of a good granular material, only enough will be required to form a properly compacted and shaped bed. A poor bottom, composed of organic material, muck, silt, or material large enough to puncture the pipe, will require removal and replacement. The depth of the material that must be removed will depend upon the actual site conditions.

Backfill

Care must be taken in the compaction of the soil in the backfill operation of culvert placement, because the fill must support the loads to prevent the culvert from crushing. Compaction is accomplished in six-inch layers if done by hand mechanical tampers and not over four inches if by hand tampers. Backfilling and compaction are done by hand to a minimum of one foot above the top of the culvert. During this operation the layers should be kept at the same elevation on both sides of the pipe. When compacted fill is 12 inches over the top of the pipe, fill and compaction can be made using mechanized equipment. Special attention must be given to the backfill and compaction beneath the diameter of the pipe. This is the most difficult area to fill and compact, and the most often neglected.

HEADWALLS AND WINGWALLS

Headwalls and wingwalls are constructed to prevent and control erosion, guide water into the culvert, reduce seepage, and hold the end of the culvert in place. These structures are costly in both time and materials. When possible, to avoid expense, extend the outlet and use riprap and/or rubble at the toe of the slope. See Figure 29.

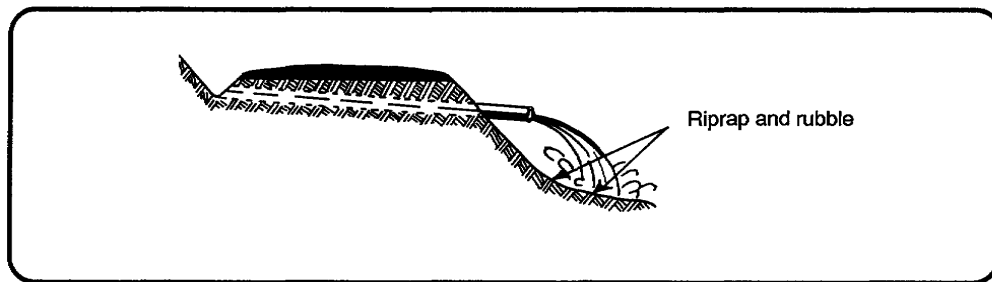


Figure 29. Placement of riprap and rubble

The inlet end of the pipe should be extended so that a minimum sized headwall is required. On steep grades, a headwall is necessary at the outlet side to hold the culvert sections in place. Headwalls should

not protrude above the shoulder grade and should be located at least two feet outside the shoulder to avoid being traffic hazards.

Wingwalls are required to channelize water and prevent washing out of headwalls. Their height should prevent embankment material from spilling into the waterway.

STEP BY STEP DESIGN OF PIPE CULVERTS WITH UNSUBMERGED INLETS

The factors to be applied to the design of these culverts are determined as follows:

STEP 1. Determine the rate of runoff. Use the area the culvert must drain. This will be the required capacity, Q_p , of the culvert.

STEP 2. Determine culvert use.

STEP 3. Calculate the critical dimensions from the cross sections. See Figure 30. Determine the length in place (LIP) and the fill critical (F_c).

STEP 4. Determine the largest pipe for the fill. Begin from the cross section at the outside edge of the shoulder on the inlet side of the culvert. Consider only pipes that are available and for which cover is adequate.

For roadway loadings, the maximum culvert diameter is equal to two-thirds of the minimum fill (FT from Figure 30). The cover required for culvert protection is equal to one-half the diameter of the culvert or 12 inches (whichever is greater).

For runways and taxiways sustaining aircraft wheel loads, refer to Table 19 for the loads, and Table 20, for the minimum cover required.

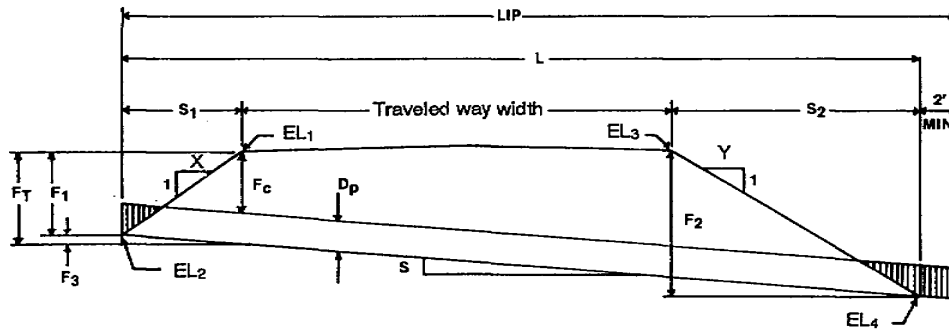
STEP 5. Determine the culvert capacity, Q_p , and outlet velocity using Table 21.

CMP ("n" = 0.024) (CMP)

STEP 6. Determine the number of culvert pipes required. Divide the area runoff, Q by the pipe capacity, Q_p . Round up to the next whole number. Use the smallest diameter pipe that can be used with the least number of pipes side by side.

STEP 7. Determine the order length (OL). The OL is calculated by multiplying the number of pipes (NP) times the LIP times a waste and grade correction factor of 1.15. (See Step 3 for the LIP and Step 6 for the NP).

STEP 8. Determine the maximum permissible discharge velocity, V_{max} . Use Table 8 to calculate V_{max} for the channel lining into which the culvert outlet will discharge. Determine the correct pipe to be used. Apply the following criteria in your calculations:



$$F_1 = EL_1 - EL_2 = \underline{\hspace{2cm}}$$

$$F_3 = S \times S_1 = \underline{\hspace{2cm}}$$

$$S_1 = F_1 \times X = \underline{\hspace{2cm}}$$

$$F_T = F_1 + F_3 = \underline{\hspace{2cm}}$$

$$F_2 = EL_3 - EL_4 = \underline{\hspace{2cm}}$$

$$LIP = S_1 + TW + S_2 + 2^1 = \underline{\hspace{2cm}}$$

$$S_2 = F_2 \times Y = \underline{\hspace{2cm}}$$

$$S = \frac{EL_2 - EL_4}{S_1 + TW + S_2} = \underline{\hspace{2cm}} = \underline{\hspace{2cm}} \quad F_C = F_T - D_P$$

- F_1 (Fill 1) = roadway elevation at the outer edge (EL_1) - the elevation at the toe of the slope (EL_2).
- S_1 (Slope horizontal distance on the inlet side of the road) = Fill 1 x the "X" value (horizontal component of the inlet side slope ratio).
- F_2 (Fill 2) = roadway elevation at the outer edge of the road on the outlet side (EL_3) - the elevation at the toe of the slope on the outlet end.
- S_2 (Horizontal distance of the roadside slope on the outlet end) = Fill 2 x the "Y" value (horizontal component of the outlet side slope ratio.)
- S (Slope) = the difference between EL_2 and EL_4 divided by the sum of $S_1 + TW$ (top width across the road) + S_2 .
- F_3 (Fill 3) = fill amount from the existing ground on the inlet side to the culvert inlet.
- F_T (Fill Total) = Fill 1 + Fill 3.
- LIP (Length in Place) = $S_1 + TW + S_2 + 2^1$.
- F_C (Fill Circle) = Fill total minus D_P (diameter of the pipe).

Figure 30. Calculating cover

Table 21. Capacity (Q_p) and outlet velocity (V) of CM ("n" = .024) (CMP)

S (%)	V (fps)	Diameter of Pipe (in)								
		12"	18"	24"	30"	36"	42"	48"	60"	72"
.4	2	1.3	3.9	8.4	15	25	37	53	96	160
.5		1.45	4.35	9.2	16.5	27.5	41	58.5	108	175
.6	3	1.6	4.8	10	18	30	45	64	120	190
.7		1.7	5.1	11	19.5	32	47.5	68	125	200
.8		1.8	5.4	12	21	34	50	72	130	210
.9		1.9	5.65	12.5	21.5	35	52	74.5	135	215
1.0		2.0	5.9	13	22	36	54	77	140	220
1.1		2.1	6.15	13	23	37	55.5	78.5	145	225
1.2	4	2.2	6.4	13	24	38	57	80	150	230
1.3		2.25	6.5	13	24.5	38.5	58	81	150	230
1.4		2.3	6.6	14	25	39	59	82	150	230
1.5		2.35	6.7	14	25	39	59	82		
1.6		2.4	6.8	14	25	40	59	83		
1.7		2.45	6.9	14	25	40				
1.8		2.5	7.0	14	26	40				
1.9		2.5	7.0	14.5	26					
2.0		2.5	7.0	15	26					
2.1		2.55	7.05	15						
2.2	2.6	7.1	15							
	5		6	7	8	9		10	11	

- Using Table 21, be sure the outlet velocity is equal to or less than the maximum velocity of the channel lining into which the culvert outlet will discharge. If outlet velocity exceeds the soils V_{max} , the outlet must be protected against erosion.
- Use the least number of culvert pipes possible to carry the total flow and still be consistent with the above criteria.

DESIGN EXAMPLE (UNSUBMERGED INLET)

Determine the most economical pipe size and the quantity of pipe required for a culvert located under a runway, with the general data cross section given below. The maximum using aircraft weight class for this example will be an SR-71C.

No headwall will be constructed downstream. The following are known conditions:

- Culvert weight type = 9.
- Q to be handled by culvert = 32 cfs.

- Soil type is bare SC; therefore, the maximum allowable outlet velocity = 3.4 fps.
- Pipe sizes available: 24-, 30-, 36-, and 42-inch CMP, 10 gauge.

Referring to Figure 31 note that the following information is required before design can be accomplished:

- Horizontal length of one culvert, L.
- Slope of culvert pipe needed to determine the flow characteristics of various pipe diameters (see Table 21) and F_c , which is a portion of the critical fill depth, F_t .
- Critical fill depth, F_t , in order to determine if the cover over the pipe meets the requirements of Table 20.

Solution:

STEP 1. Determine the runoff rate. Given for this example $Q = 32$ cfs.

STEP 2. Determine culvert use.

STEP 3. Calculate the critical dimensions. Computations should be made in the following sequence (see Figure 31).

- Determine the difference in elevation between the edge of the roadway and the culvert upstream invert.

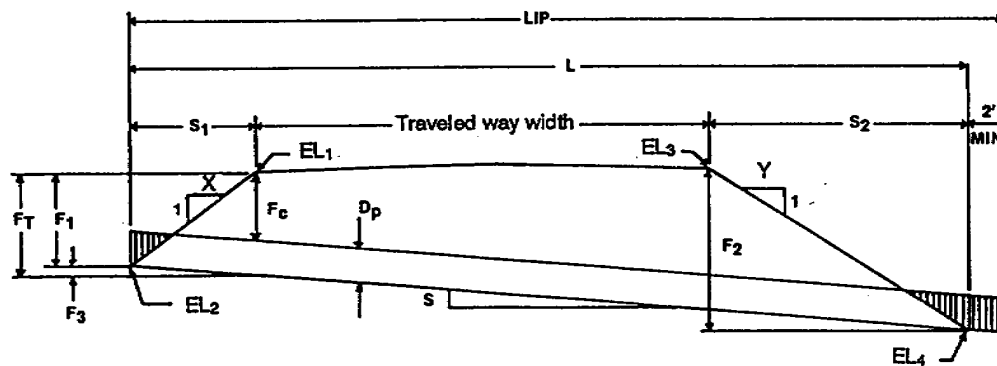
$$F_1 = EL_1 - EL_2$$

$$F_1 = 607.00 - 600.80 = 6.20 \text{ ft}$$

- Determine the horizontal distance between the culvert invert and the shoulder edge.

$$S_1 = F_1 \times X$$

$$S_1 = 6.20 \times 10 = 62 \text{ ft}$$



$$F_1 = EL_1 - EL_2 = 607.0' - 600.8' = 6.2'$$

$$F_3 = S \times S_1 = 0.005 \times 62' = .31'$$

$$S_1 = F_1 \times X = 6.2' \times 10 = 62'$$

$$F_T = F_1 + F_3 = 6.2' + .31' = 6.51'$$

$$F_2 = EL_3 - EL_4 = 607.0' - 599.62' = 7.38'$$

$$LIP = S_1 + TW + S_2 + 2' = 62' + 100' + 73.8' + 2 = 237.8' \text{ or } 238'$$

$$S_2 = F_2 \times Y = 7.38' \times 10 = 73.8'$$

$$S = \frac{EL_2 - EL_4}{S_1 + TW + S_2} = \frac{600.8 - 599.62}{62 + 100 + 73.8} = \frac{1.18}{235.8} = 0.005 \text{ or } 0.5\%$$

$$F_C = F_T - D_p$$

- F₁ (Fill 1) = roadway elevation at the outer edge (EL₁) - the elevation at the toe of the slope (EL₂).
- S₁ (Slope horizontal distance on the inlet side of the road) = Fill 1 x the "X" value (horizontal component of the inlet side slope ratio).
- F₂ (Fill 2) = roadway elevation at the outer edge of the road on the outlet side (EL₃) - the elevation at the toe of the slope on the outlet end.
- S₂ (Horizontal distance of the roadside slope on the outlet end) = Fill 2 x the "Y" value (horizontal component of the outlet side slope ratio.)
- S (Slope) = the difference between EL₂ and EL₄ divided by the sum of S₁ + TW (top width across the road) + S₂.
- F₃ (Fill 3) = fill amount from the existing ground on the inlet side to the culvert inlet.
- F_T (Fill Total) = Fill 1 + Fill 3.
- LIP (Length in Place) = S₁ + TW + S₂ + 2¹.
- F_C (Fill Circle) = Fill total minus D_p (diameter of the pipe).

Figure 31. Calculating cover (sample problem)

- Determine the difference in elevation between the edge of the runway and the culvert outlet and fill slope.

$$F_2 = EL_3 - EL_4$$

$$F_2 = 607.00 - 599.62 = 7.38 \text{ ft}$$

- Determine the horizontal distance between the edge of the runway and the intersection of the culvert and fill.

$$S_2 = F_2 \times Y$$

$$S_2 = 7.38 \times 10 = 73.8 \text{ ft}$$

- Determine the actual slope of the culvert.

$$S = \frac{EL_2 - EL_4}{S_1 + TW + S_2}$$

$$S = \frac{600.8 - 599.62}{62 + 100 + 73.8} = \frac{1.18}{235.8}$$

$$= 0.005 \text{ or } 0.5 \text{ percent}$$

- Determine the incremental elevation difference between the elevation of the upstream invert and the invert of culvert at critical fill section.

$$0.005 \text{ ft/ft} = 0.5 \text{ percent}$$

$$F_3 = S \times S_1$$

$$F_3 = 62 \times 0.005 = 0.31 \text{ ft}$$

- Determine the depth of critical fill section.

$$6.2 + .31 = 6.51'$$

$$F_t = F_1 + F_3$$

$F_t = 6.2 + 0.31 = 6.51 \text{ ft}$. (Length of horizontal projection of the culvert.) The upstream invert elevation to downstream invert elevation is-

$$LIP = S_1 + TW = S_2 + 2 \text{ ft}$$

$$LIP = 62 + 100 + 73.8 + 2 = 237.8 \text{ ft; (Round up to 238 ft)}$$

NOTE: These computations depict the cross section and Figure 31 summarizes the calculations.

From these calculations, it has been determined that-

- The actual length of the culvert is 235.8 feet. (2) A 2-foot projection is added because there is to be no downstream headwall. (3) The pipe must be rounded up to an even length of 238 feet. (4) This gives a culvert length of 238 feet for a single pipe.
- The slope of the culvert is 0.005 ft/ft or 0.5 percent.

- The depth of fill at the critical section is 65 feet (rounded to the nearest tenth of a foot).

STEP 4. Determine the largest pipe for the fill. Prepare a table as shown in Table 22 and fill in all known values. Start by entering the largest pipe available, D_P , into the table. Subtract D_P from total fill, F_T , to get fill critical, F_C . F_C represents the actual cover over the pipe at the critical section. Compare F_C to cover required, C_R .

To find C_R for aircraft, refer to Table 19, which indicates that an SR-71C aircraft has a culvert weight type (WT) of 9. The CMP available is 10 gauge. With this information, refer to Table 20, and select chart 9, corresponding to culvert WT 9. The diameters and cover are given in chart 9 under the 10-gauge line. Starting with the largest pipe available, 42-inch or 3.5 feet, C_R is 4.0 feet (interpolated between pipe diameters.) Enter this value in Table 22. The 42-inch pipe (3.5 feet) cannot be used because its actual cover is 3.01 feet against a required cover of 4.0 feet. Repeat the process using the next smaller pipe available, 36-inch pipe. It has a F_C of 3.51 feet and a C_R of 3.5 feet. This pipe will work.

STEP 5. Determine the culvert capacity and the number of pipes needed (side by side). Use Table 21, to find the capacity and velocity for the 36-inch pipe.

Table 22. Unsubmerged inlet sample data

		$F_C = F_T - D_P$			
Total Fill	F_T	6.51	6.51		
Pipe Diameter	D_P	3.01	3.0		
Fill Critical	F_C	3.01	3.51		
Cover Required	*C_R	4.0		3.5	

The slope of the culvert is 0.5 percent. Entering the graph from the top (pipe diameter), move down the column until you intersect with the slope in the left-hand column. This will state the quantity of flow in the pipe. The velocity of flow is shown by following the respective shaded or unshaded areas down and to the left until it ends in the velocity column, 4 feet per second.

The 36-inch pipe has a capacity of 27.5 cubic feet per second for the 0.5 percent slope. The velocity indicated is 4 feet per second.

STEP 6. Determine the number of pipe required to carry the flow by dividing the total (Q) of 32 cubic feet per second (Step 1) by the pipe capacity, 27.5 cubic feet per second (Step 5), and

rounding up to the next whole pipe. For this example, the number of pipe required is $32/27.5 = 1.2$ or two 36-inch diameter pipes.

STEP 7. The next step is determining the length of pipe to be ordered. The order length is calculated by multiplying NP (Step 6) times LIP (Step 3) times a waste factor (WF). Since pieces of material will be damaged in manufacturing, handling, transporting, and assembling, an additional amount over the actual in place length will be required. This value has been determined to be 15 percent of the total length of pipe required. For this project, the pipe selected will be 36 inches in diameter with a length in place of 238 feet. The length to order will be-

$NP \times LIP \times WF$

$2 \times 238 \times 1.15 = 547.4$ ft (round up to 548 ft)

The pipe comes in 2-foot increments; therefore, the value of 547.4 is rounded up to the next even value, or 548 feet of pipe.

CONTROL EROSION

Part of controlling erosion is an essential part of construction. Good erosion control practices will ensure the design life of the structure.

PROTECTION AGAINST SURFACE WATER EROSION

There are three commonly used methods to protect a construction area from surface erosion. They are:

- Interceptor ditches
- Terracing
- Vegetal cover

An interceptor ditch is an open channel. A engineer places the interceptor ditch on the slope above the construction area to catch the runoff and carry it to a discharge point.

Terracing is a control measure designed to dissipate the energy of overland flow before it reaches the construction area. A terrace consists of a low, broad-based earth levee constructed approximately parallel to the contours. It is designed to intercept and hold the water until it infiltrates into the soil or to conduct it as overland flow to a suitable discharge point. A hardy, vigorous turf should be established to hold the soil in place. Vertical spacing and longitudinal gradients of terrace are given in Figure 32.

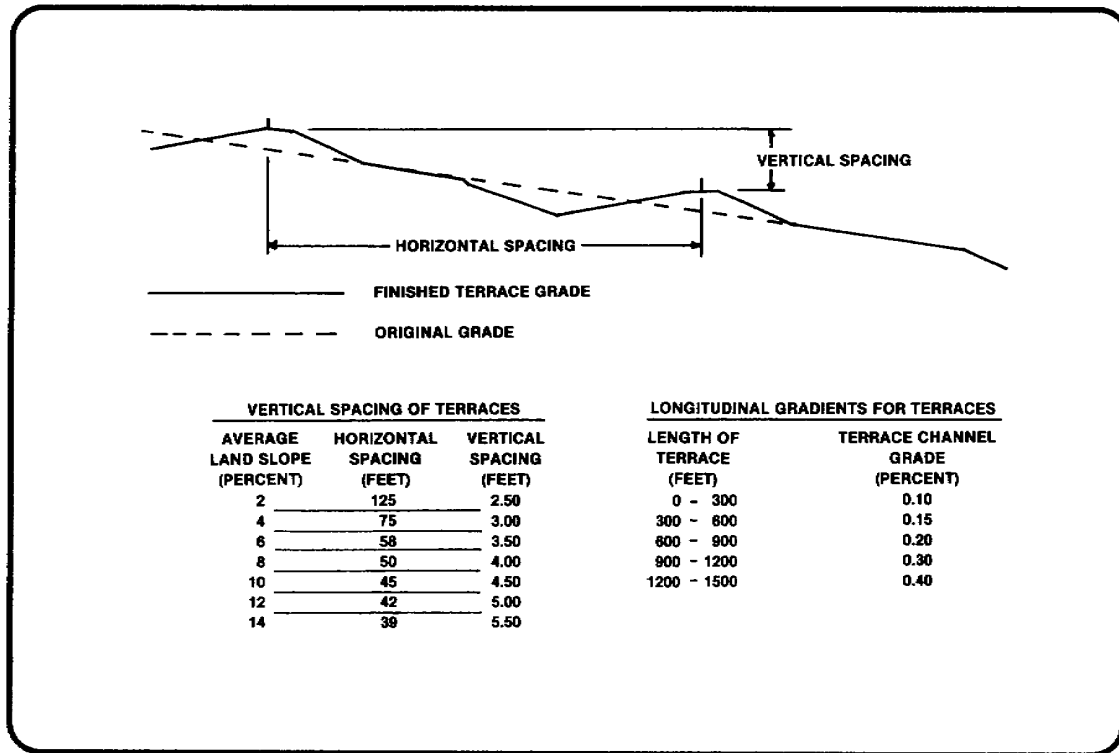


Figure 32. Vertical spacing and longitudinal gradients of terraces

This course has mentioned many times the value of turf and other vegetal covers. Turf is used to reduce the velocity of flow and it retains a certain amount of runoff through interception.

OPEN CHANNEL EROSION CONTROL

To review briefly, the means for protecting open channels from the damage caused by erosion are:

- Checkdams.
- Paving the ditch with asphalt or concrete.
- Ditch linings to increase the “n” factor, such as riprap and rubble.

One commonly used method is the *stilling basin*. This method is most often used with culverts. Its function is to prevent overloading the drainage system. *Stilling basins* or *ponding* will be discussed in detail in the next section.

CULVERT EROSION CONTROL

Erosion at the culvert outlet is caused by one of two conditions-turbulence or water velocity. Turbulence at the outlet will produce bottom and side-bank scour, as shown in Figure 33. Water velocity at the outlet flow exceeds the erosion velocity of the soil.

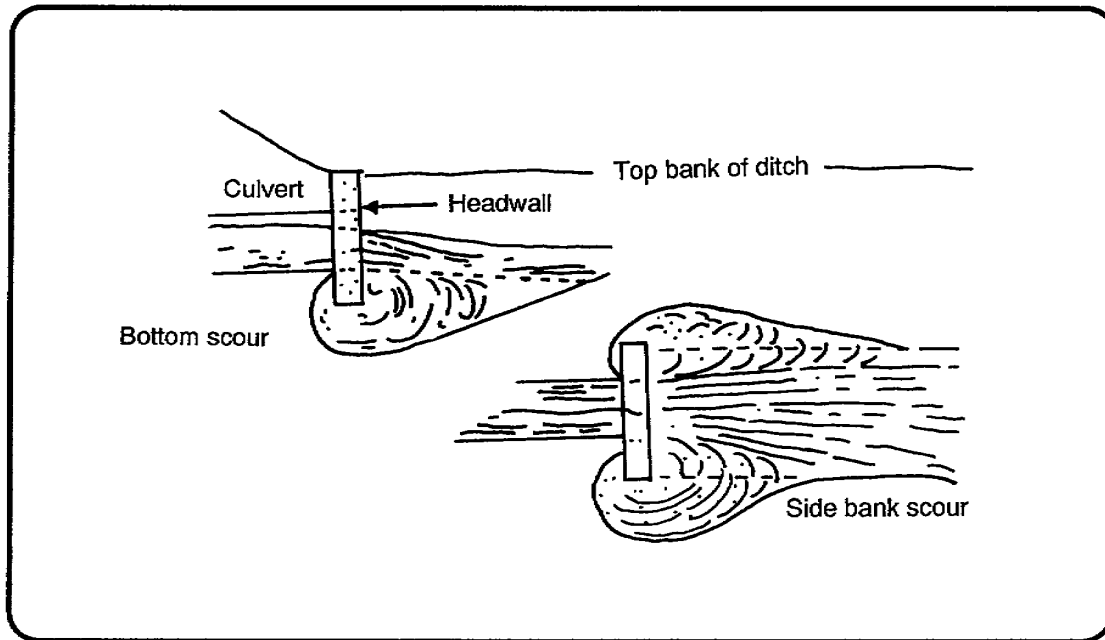


Figure 33. Culvert outlet turbulence

Side-bank and bottom scour can be controlled by laying erosion-resistant material at the affected points. The use of downstream walls will assist in the reduction of side-bank scour.

Remember-the last step in designing a culvert is to check the velocity to see if some form of erosion control will be necessary at the outlet. Erosion velocity of the soil is obtained from Table 8.

If headwalls and wingwalls are not used on the downstream end, the culvert should project a minimum distance of two feet beyond the toe of the fill. The area under the projecting culvert should be well protected by erosion-resistant material, such as sandbags or rock riprap. Rock riprap should be of rock not less than a 6-inch minimum dimension. On the sides of the channel, riprap should be two layers deep; under the culvert outfall, the depth of riprap should be at least three layers.

Changing the slope of the culvert is another way to decrease velocity and thus reduce the chance of erosion. The slope(s) of the culvert will generally be established by the stream bed. In the event the grade is steep, generating high outlet velocity, the slope of the culvert can be set by the engineer to lower the outlet velocity.

Often erosion of drainage structures occurs as a result of a very large storm. Drainage structures are seldom, if ever, designed to discharge the worst storm on record. For this reason it may be assumed that a storm more severe than the design storm may occur and overload the drainage system. Since most drainage structures will be overloaded at one time or another during their useful life, it is usually a good procedure to design the area around a culvert or drain inlet to take care of a certain amount of ponding. In some cases, for economy, a drainage engineer may deliberately specify a system which cannot immediately take care of even the design storm. When this procedure is followed, sufficient ponding areas must be included in the overall plan so that inundation of vital areas does not occur. In this case, the excess water is merely stored until the intensity of the storm decreases to the extent that the drains can handle the water. As a general rule, drainage systems are designed to take care of the runoff from the design storm without ponding. For reasons previously mentioned, however, some provisions should be made for ponding to take place in those areas where inundations for a period of time will not affect the

operational duties of the installation.

The following specifications are generally followed in the design of ponding areas for most installations:

- The edge of ponding areas must be at least 75 feet from the pavement edge.
- The pond must be drained before damaging infiltration of the subgrade can occur. The actual time during which ponding is allowable depends upon the condition and type of soils found in the ponding area. Generally speaking this period will be about four hours after the storm begins for most of the soils encountered in practice.

CONDITIONS CAUSING SUBSURFACE EROSION

Many construction projects fail because of unforeseen subsurface erosion. The subsurface conditions contributing to this problem are-

- Location (or proximity) of the groundwater table.
- Groundwater migration.
- Soil conditions.
- Perched water.
- Effluent streams.
- Side hill seepage.

Study Figure 34. All of the conditions promoting subsurface erosion are illustrated.

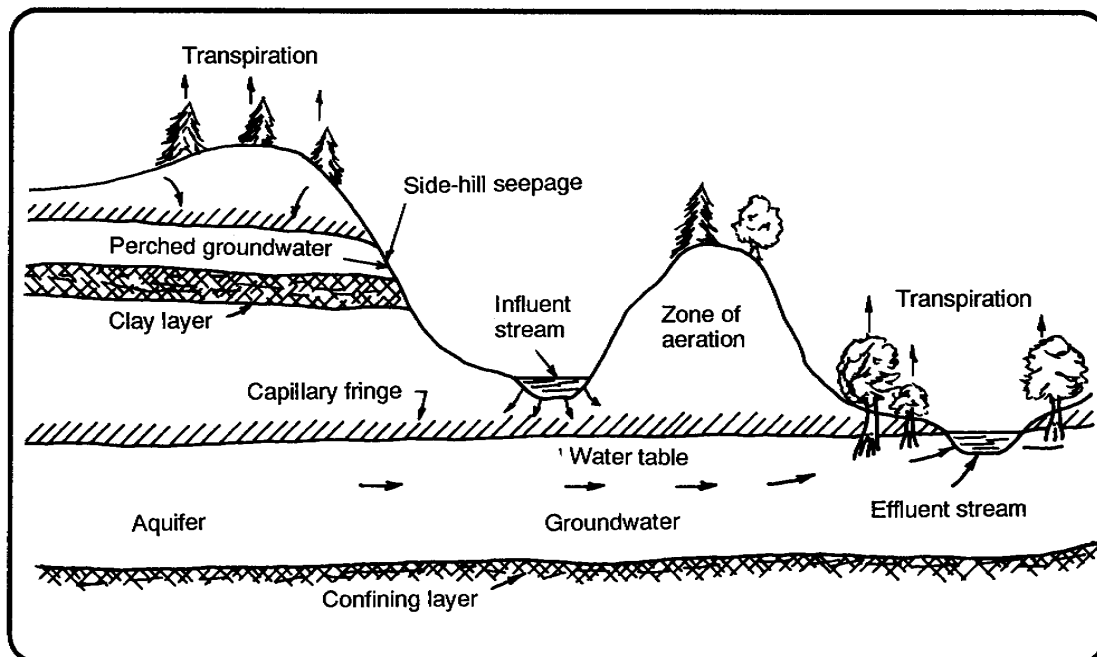


Figure 34. Schematic cross section showing the occurrence of groundwater

METHODS OF CONTROLLING SUBSURFACE EROSION

Subsurface water can be controlled using a combination of techniques which will vary according to the conditions existing in the area to be drained. Selected techniques which should be considered when planning and designing subsurface drainage follow.

DEEP DITCH

Where ditches will not interfere with operations or become a hazard to traffic, deep V-ditches with free outfall may be feasible. These ditches are easily built and readily enlarged. They provide positive interception and drainage of subsurface water before the water reaches the area to be protected. Erosion and subsequent maintenance are problems to be considered together with right-of-way and traffic problems.

FRENCH DRAINS

French drains are constructed by filling a ditch or trench with broken or crushed rock. The top surface of the rock may be left exposed so the trench will act as a combination drain or the rock may be covered by a relatively impervious soil so that no surface water can penetrate. The latter is the general practice. In general, French drains are not recommended for permanent construction because they have a tendency to silt up with prolonged use. In TO construction, such drains are often used as a substitute for perforated or open joint pipes because of logistical limitations on piping or on filter materials suitable for use with such piping.

VERTICAL WELLS

Vertical wells are sometimes constructed to permit trapped subsurface water to pass through an impervious soil or rock layer to a lower, freely draining layer of soil. If drainage is obstructed, additional wells are driven, or the pocket is drained with an easily maintained lateral subdrain system. Vertical wells are often used in northern latitudes, where deep freezing is common, to permit fast runoff from melting snow to get through the frozen soil and reach a pervious stratum. Under such conditions the bottom of these wells are treated with calcium chloride or a layer of hay to prevent freezing.

SAND BLANKET

The sand blanket method of controlling subsurface erosion works by reducing the time of flow. Sand is placed in vertical columns dug into a marshy or muddy base. The whole area is then covered with a blanket of sand on which the roadway fill is placed. Figure 35 illustrates the sand blanket method before and after settlement.

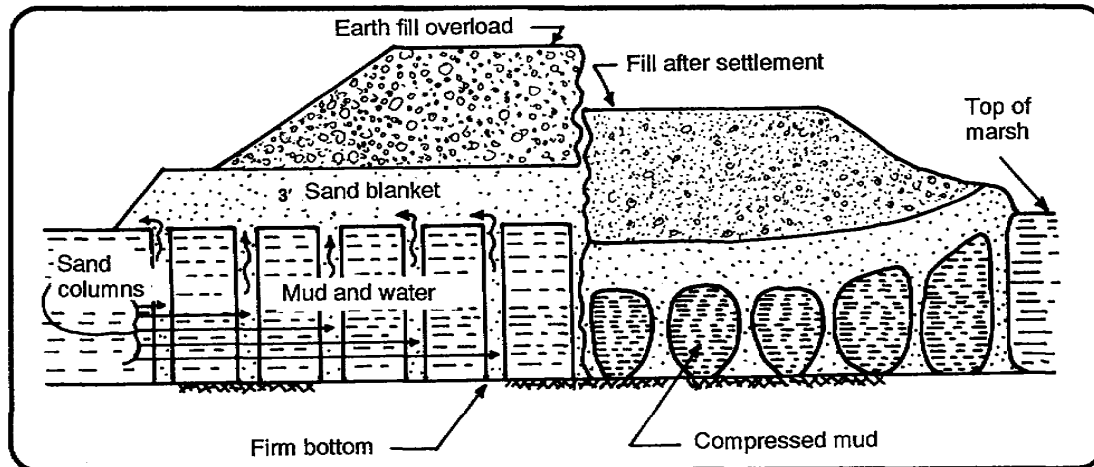


Figure 35. Sand blanket method

SUBSURFACE PIPE OR SUBGRADE DRAINAGE INSTALLATIONS

In cases where a V-type or other open-ditch type drainage system is not practical, it may be necessary to resort to a subsurface piping system utilizing one of the many forms of pipe currently available. Care in construction is essential during every stage of subsurface drainage construction. This prevents damage to base courses from seasonal fluctuations of ground water. Failure of the subsurface system stemming from improper control of grade, bedding, placement of pipe, placement of filter material, and other installation work gives no warning before failing and is extremely difficult to repair when failure is discovered.

The most common form of subsurface piping is perforated pipe. Where the perforations do not extend completely around the circumference of the pipe, the pipe is generally laid with the holes down and with the joints closed. Materials used in the manufacture of this type of pipe are corrugated metal, cast iron, vitrified clay, nonreinforced concrete, bituminized fiber, and asbestos-cement. Figure 36 shows how subgrade drainage lowers the ground water table in a construction area.

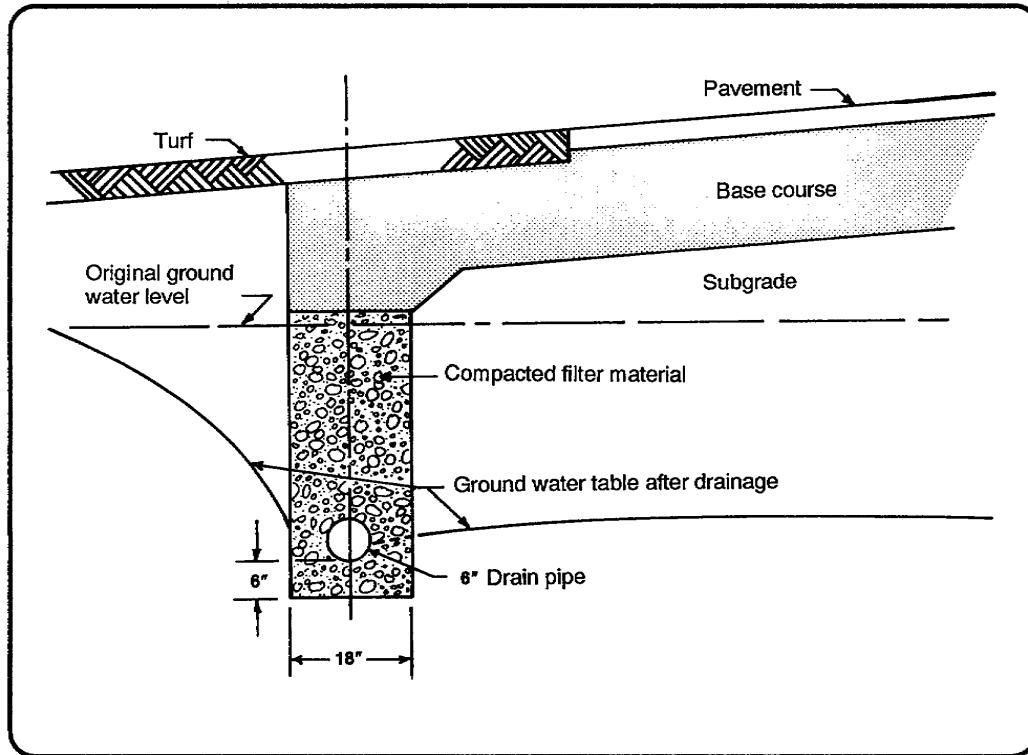


Figure 36. Subsurface drainage installations

MOVE THE PROJECT

Moving the project to a drier location is the last alternative. However, as the engineer on a construction project you may find that it is less costly to move the project than to install elaborate subsurface drainage systems.

GEOTEXTILES

The use of geotextiles or filter fabrics is another way of dealing with subsurface drainage problems. The use of the fabrics has proven to be very helpful in construction projects. The manufacturers of the textiles provide the necessary information to select and install the proper fabric to control subsurface drainage.