SRC-BOOKS

Erosion and Sediment Control Handbook

Steven J. Goldman
California Tahoe Conservancy

Katharine Jackson
Consultant

Taras A. Bursztynsky, P.E.
Association of Bay Area Governments

McGraw-Hill Publishing Company

New York St. Louis San Francisco Auckland Bogotá
Caracas Hamburg Lisbon London Madrid Mexico Milan
Montreal New Delhi Oklahoma City Paris San Juan
São Paulo Singapore Sydney Tokyo Toronto

12/11/91

Library of Congress Cataloging-in-Publication Data

Goldman, Steven J.
Erosion and sediment control handbook.

Includes bibliographies and index.
1.Sediment control—United States—Handbooks, manuals, etc. 2.Soil erosion—United States—Handbooks, manuals, etc. 3.Soil conservation—United States—Handbooks, manuals, etc. I.Jackson, Katharine. II.Bursztynsky, Taras A. III.Title.
TC423.G645 1986 627'.52 85-15371
ISBN 0-07-023655-0

234567890 HDHD 99876543210

ISBN 0-07-023655-0

The editors for this book were Joan Zseleczky and Galen H. Fleck; the designer was Mark E. Safran; and the production supervisor was Thomas G. Kowalczyk. It was set in Century Schoolbook by University Graphics, Inc.

Printed and bound by Halliday Lithograph

chapter 4

Estimating Runoff

Estimating runoff and soil loss is a necessary first step in the design of erosion control facilities. Proper sizing of runoff conveyance facilities and sediment retention structures depends on knowing both the amount and rate of runoff and the amount of sediment expected to be carried in the runoff. Erosion and sediment control measures can then be designed to handle the anticipated flows adequately and without unnecessary investment in oversized facilities.

Erosion control facilities form part of the drainage system in a project. It is expected that the civil engineer designing the roads and storm drains will also be responsible for sediment basin and temporary channel design. A soils engineer should participate if considerable earth movement is involved and, in any event, should be asked to evaluate the entire erosion control plan. If a large settling basin is proposed, the soils engineer's opinion is most important to ensure that the structure is properly designed for the geologic conditions at the site.

This chapter describes a simple method for calculating runoff by using the rational method. Soil loss prediction is described in Chap. 5. Hydraulics of open channel flow and design of permanent drainage facilities are not covered. These subjects are the responsibility of the civil engineer. Portions of the drainage system design process relevant to erosion control are discussed in Chaps. 7 and 8. For more thorough treatment, consult appropriate engineering handbooks such as those listed in the references at end of this chapter.

4.1 ESTIMATING RUNOFF BY THE RATIONAL METHOD

In erosion control planning, there are several reasons for calculating runoff. They include:

- Sizing conveyance structures
- Selection of channel linings
- Sizing outlet protection (e.g., riprap aprons)
- Sizing retention structures (e.g., sediment basins)

This section discusses the rational method as a simple formula for finding peak and average runoff. Any other proven method is also acceptable and can be applied to an erosion control project.

4.1a The Equation

In the rational method a simple equation is used to compute discharge from small areas. Although the method has been applied to areas as large as 5 mi^2 (13 km^2), it is strongly recommended that it not be applied to areas larger than 200 acres (81 ha). (14) The equation is

$$Q = C \times i \times A$$

where Q = runoff rate, ft^3/sec
C = runoff coefficient, a factor chosen to reflect such watershed characteristics as topography, soil type, vegetation, and land use
i = precipitation intensity, in/hr
A = watershed area, acres

In English units, the dimensions of the various factors do not match but the conversion factor needed to make the units on both sides of the equation the same is nearly 1. A runoff rate of 1 acre·in/hr equals 1.008 ft^3/sec. The use of a conversion factor of 1/1.008 is unnecessary in so approximate a relation as the rational formula.

In metric units

$$Q\ (\mathrm{m^3/sec}) = \frac{C \times i\ (\mathrm{mm/hr}) \times A\ (\mathrm{ha})}{360}$$

The factor 1/360 is necessary to make the metric units match:

$$\frac{1\ \mathrm{m}}{1000\ \mathrm{mm}} \times \frac{1\ \mathrm{hr}}{3600\ \mathrm{sec}} \times \frac{10{,}000\ \mathrm{m^2}}{\mathrm{ha}} = \frac{1}{360}$$

At the beginning of a storm, runoff from distant parts of a watershed will not have reached the discharge point where the watershed's runoff Q is monitored.

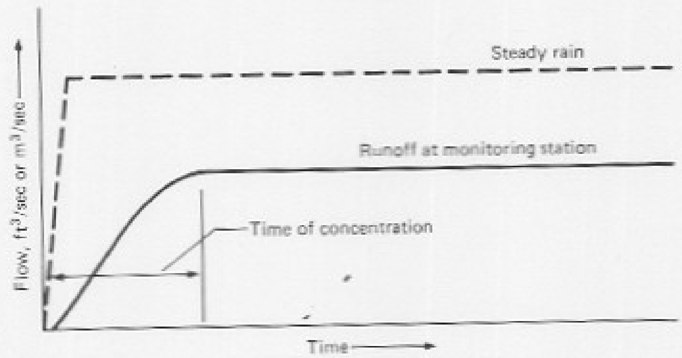


Fig. 4.1 Graphic representation of time of concentration.

After a period that is specific to each watershed, termed "time of concentration," a steady-state flow will occur as shown in Fig. 4.1. Time of concentration is defined as the time required for the runoff from the most remote part of the drainage area to reach the point under consideration. It is used to find the precipitation intensity i, and it predicts the peak runoff rate Q_{peak}.

The rational formula can also be used to find an average flow rate from a watershed. When an average flow rate is required, as in the design of sediment basins, the average rainfall intensity throughout the duration of a storm is used to find i. Using average intensity produces an estimate of average flow rate Q_{avg}, which is not related to the time of concentration.

4.1b Design Storm

We know from experience that rainfall amounts vary considerably from year to year. Every few years a large storm occurs. The *return period* is defined as the average number of years between storms of given duration and intensity. For example, at Fairfield, California, the rainfall records show that every 2 years you can expect a 6-hr rainfall that drops 1.16 in (29 mm) of rain. Every 100 years you can expect a 6-hr rainfall that drops 2.91 in (74 mm) of rain. Two years and 100 years are the return periods for the 6-hr rainfalls *of those magnitudes*. It is usually not desirable to design a structure to handle the greatest rainfall that has ever occurred.

It is often more economical to allow a periodic overflow than to design a very large structure that will never overflow. However, if human life would be threatened by an overflow, the structure should be sized to handle the largest storm expected. The rainfall duration and intensity (such as the 24-hr, 6-in storm) used to size a drainage facility is called the *design storm*. Many jurisdictions have specified storms for designing various structures. Those specifications usually apply to permanent structures such as storm drains.

Erosion control structures may be temporary or permanent. Concrete channels are often a permanent part of the drainage system of a fill slope. Sediment basins are usually temporary, although the risers may discharge directly into permanent storm drains. Unlined channels are frequently temporary; they are used when grading continues during the rainy season. Permanent structures must be designed according to the local standards. But when designing temporary structures, the planner can often choose a design storm. This choice should be based on a comparison of the risks and costs of hydraulic failure with the expense of building a structure sized to a larger design storm. If the cost of failure will not be high and the local jurisdiction has not specified otherwise, a storm with a 10-year return interval is recommended for sizing temporary erosion control measures.

The costs associated with failed systems vary with site factors such as the topography, climate, and geology, the amount of grading (cost of repairing washed-out fill slopes), the presence of natural drainageways on or near the site, and surrounding land uses, as well as local policy toward sedimentation of waterways and lakes. For example, a very thin layer of sediment deposited on grass may have no adverse affects, but a similar deposit on a roadway may cause hazardous driving conditions. Sediments deposited downslope on a construction site may present a minor problem, but if the sediment fills the newly built storm drain system and causes overflow onto a street, additional cleanup costs are imposed on both the developer and the local public works department. And if a sediment basin discharges into a natural creek, the local flood control district or public works department probably will be very concerned about siltation of the channel. In any location where the consequence of failure is high, temporary erosion and sediment control structures should be sized for a design storm larger than the 10-year return period storm.

One consideration favoring a more conservative choice of design storm is that, in many permanent structures, the effects of failure are mitigated by secondary systems. When a storm drain overflows, the excess can usually be contained by the curb and gutter system. If a terrace drain overflows, the runoff may flow over a vegetatively stabilized slope with only minor damage. But temporary measures often have no backup systems to limit damage if a failure does occur. If an interceptor swale at the top of a new fill slope fails, the recently planted ground cover will be washed away. Gullies can form to such an extent that the fill slope may have to be completely rebuilt. Collapse of an undersized temporary sediment basin could release a flood of water and a considerable amount of sediment to adjacent properties. Both public and private owners could sustain substantial damage under such circumstances, and the developer and the agency that approved the project could be held responsible.

Finally, it is especially important to prevent erosion in areas of high erosion potential. Such areas are likely to be associated with unstable slopes or with drainage problems which are already recognized as hazards by the soils engineer. Not only can erosion eventually threaten the stability of foundations and roadways, but loss of topsoil makes it much more difficult to establish permanent vegetation that will prevent erosion over the long term. A consistent local policy in favor of preventing erosion during development will reduce long-term maintenance and repair costs. Such a policy should be based on whether and where such high-risk areas exist. An appropriately longer storm return interval is advised in high-risk areas.

4.1c Use of Q_{peak}

The peak runoff, which is normally used to size drainage systems, is calculated when the capacity of a channel or other conveyance structure must be sufficient to carry all of the flow. In erosion control work, Q_{peak} is important not only to size conveyance facilities but also to:

- Check for potentially erosive velocities in unlined channels
- Select channel linings that will not erode
- Design outlet protection

In these cases, the rational method is applied by using a peak precipitation intensity:

$$Q_{peak} = C \times i_{peak} \times A$$

Peak precipitation intensity i_{peak} is determined by estimating the time of concentration for the drainage area and then finding the maximum rainfall intensity for that time duration and design storm return interval. For example, if the time of concentration for a watershed is 1 hr, you should use the peak 1-hr rainfall intensity in your calculations. The procedure for determining this time is explained in Sec. 4.1g.

4.1d Use of Q_{avg}

An average flow Q_{avg}, rather than peak flow, is used to find the required surface area of sediment basins and traps. The rational formula is still applied, except that an average precipitation intensity instead of the peak intensity is used:

$$Q_{avg} = C \times i_{avg} \times A$$

Average precipitation intensity i_{avg} is determined by taking the total rainfall for a specified storm return period and duration (e.g., 10-year, 6-hr storm) and dividing that total by the number of hours of duration:

$$i_{avg} = \frac{\text{total 6-hr rain}}{6}$$

A 6-hr storm duration is suggested. Sediment basins designed with a 6-hr storm strike a reasonable compromise between being somewhat undersized during storm peaks and being somewhat oversized during the rest of the storm.

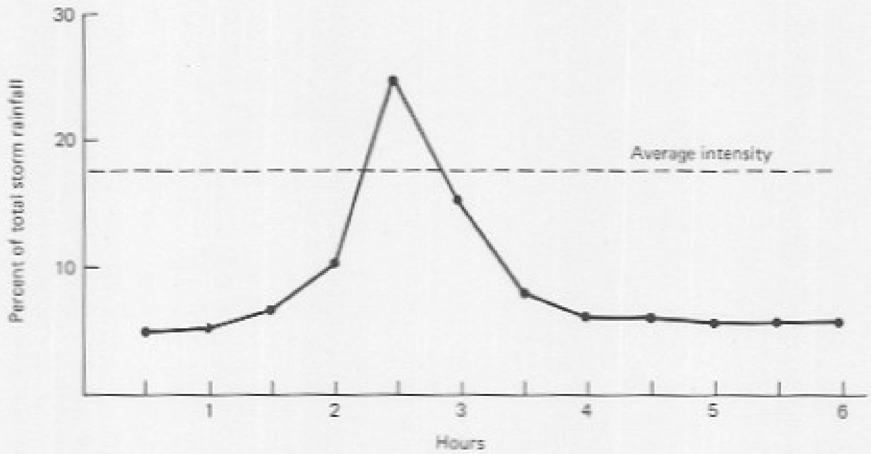


Fig. 4.2 Typical rainfall distribution and storm runoff during a 6-hr storm.

Sediment trapping efficiency is lower during the portion of a storm in which the immediate flow exceeds the average flow. The graph in Fig. 4.2 illustrates a typical rainfall distribution and storm runoff for a 6-hr storm. Peak intensity occurs at the top of the distribution curve; average intensity is indicated by a dashed line. Only during a small portion of the storm will the flow exceed average design flow. The graph was generated by placing the ½-hr incremental rainfall for a 6-hr storm in the sequence suggested by the U.S. Soil Conservation Service (SCS), 6-hr design storm distribution. (11, 12)

Use of average flow is a cost-saving measure. For the same design storm return interval, peak flow, which is based on a short time of concentration, is much larger than average flow of longer duration. A basin sized by using peak flow will thus be much larger than one sized by using the average flow. The sizing procedure is described in Chap. 8.

4.1e Runoff Coefficient C

The runoff coefficient C determines the portion of rainfall that will run off the watershed. It is based on the permeability and water-holding capacity of the various surfaces in the watershed. The C value can vary from close to zero to up to 1.0. A low C value indicates that most of the water is retained for a time on the site, as by soaking into the ground or forming puddles, whereas a high C value means that most of the rain runs off rapidly. Well-vegetated areas have low C values. Developed land, with its pavement, rooftops, and other impermeable surfaces, has a high C value. A high runoff coefficient produces higher runoff than does a low C value, and Q is directly proportional to C.

Table 4.1 lists C values for use in the rational formula. Select a C value within the range for land use. The designer must exercise judgment in selecting C from

Estimating I

TABLE 4.1
Land
Business
Downtown
Neighborh
Residential
Single-fam
Multi units
Multi units
Suburban
Industrial
Light areas
Heavy area
Parks, cemet
Playgrounds
Railroad yar
Unimproved
Streets
Asphaltic
Concrete
Brick
Drives and w
Roofs
Note: The d
larger areas w
areas with de
values.
*From Portla

the range f
and vegeta
For con
cent, use t
density of
from the r
and shrub
and 0.30. I
choose a v
to imperm
is higher f
average of
If area

TABLE 4.1 Rational Method *C* Values (13)

Land use	*C*	Land use	*C*
Business		Lawns	
Downtown areas	0.70–0.95	Sandy soil, flat, 2%	0.05–0.10
Neighborhood areas	0.50–0.70	Sandy soil, average, 2–7%	0.10–0.15
Residential		Sandy soil, steep, 7%	0.15–0.20
Single-family areas	0.30–0.50	Heavy soil, flat, 2%	0.13–0.17
Multi units, detached	0.40–0.60	Heavy soil, average, 2–7%	0.18–0.22
Multi units, attached	0.60–0.75	Heavy soil, steep, 7%	0.25–0.35
Suburban	0.25–0.40	Agricultural land, 0–30%	
Industrial		Bare packed soil	
Light areas	0.50–0.80	Smooth	0.30–0.60
Heavy areas	0.60–0.90	Rough	0.20–0.50
Parks, cemeteries	0.10–0.25	Cultivated rows	
Playgrounds	0.20–0.35	Heavy, soil, no crop	0.30–0.60
Railroad yard areas	0.20–0.40	Heavy soil with crop	0.20–0.50
Unimproved areas	0.10–0.30	Sandy soil, no crop	0.20–0.40
Streets		Sandy soil with crop	0.10–0.25
Asphaltic	0.70–0.95	Pasture	
Concrete	0.80–0.95	Heavy soil	0.15–0.45
Brick	0.70–0.85	Sandy soil	0.05–0.25
Drives and walks	0.75–0.85	Woodlands	0.05–0.25
Roofs	0.75–0.95	Barren slopes, >30%*	
		Smooth, impervious	0.70–0.90
		Rough	0.50–0.70

Note: The designer must use judgment to select the appropriate *C* value within the range. Generally, larger areas with permeable soils, flat slopes, and dense vegetation should have lowest *C* values. Smaller areas with dense soils, moderate to steep slopes, and sparse vegetation should be assigned highest *C* values.

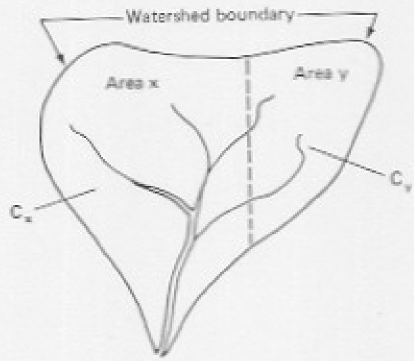
*From Portland Cement Association, *Handbook of Concrete Culvert Pipe Hydraulics*, 1964, p. 45.

the range given by considering factors such as permeability, soil type, steepness, and vegetation.

For construction sites, when the soil is bare and the slope is less than 30 percent, use the agricultural values in the table and consider soil conditions and density of vegetation. For areas with temporary vegetative cover, select a value from the ranges for "cultivated rows"; for undisturbed areas under natural grass and shrub cover assign an appropriate "unimproved areas" *C* value between 0.10 and 0.30. If the slope gradient is greater than 30 percent, for example, 3:1 or 2:1, choose a value in the range 0.50–0.90 under "barren slopes." Soil depth or depth to impermeable rock influences the choice within the ranges given; the *C* value is higher for shallower soils. For sites with mixed land uses, compute a weighted average of the individual *C* values, as follows:

If area $A = x + y$, then

$$C\ (\text{weighted}) = \frac{(x \times C_x) + (y \times C_y)}{A}$$



EXAMPLE 4.1

Given: A 15-acre (6-ha) site with clay soil and the slope and vegetative conditions shown below.

Find: The C value for each land use and a weighted C value for the entire site.

Solution:

Site condition	C
5 acres (2 ha): 3:1 gradient, fill (pervious), hydroseeded (rough surface)	0.50
10 acres (4 ha): 2% gradient, smooth, bare, packed soil no vegetation	0.30

The lower end of each range was selected because fill is usually more pervious than a cut slope in the case of the 5 acres (2 ha) and the slope is very flat on the 10-acre (5-ha) portion of the site.

$$C \text{ (weighted)} = \frac{(5 \text{ acres} \times 0.5) + (10 \text{ acres} \times 0.3)}{15 \text{ acres}} \quad \left[\frac{(2 \text{ ha} \times 0.5) + (4 \text{ ha} \times 0.3)}{6 \text{ ha}}\right]$$

$$= 0.37$$

4.1f Precipitation Intensity i

The i value, precipitation intensity, represents the rate of rainfall during a storm. It is calculated from depth and duration of rainfall, and it can vary tremendously with the average annual rainfall and the duration of the individual storm. Many states compile precipitation frequency, duration, and intensity data. In California, the Department of Water Resources (DWR) publishes precipitation data derived from records of rainfall data collected from more than 650 recording rain gauges throughout the state. (2)

TABLE 4.2 A
Address inquiries

ALABAMA
K. E. Johnson E
Center
The University o
Huntsville, AL 3

ALASKA
AEIDC/Universi
Alaska Climate
707 A Street
Anchorage, AK

ARIZONA
The Laboratory
Arizona State U
Tempe, AZ 852

ARKANSAS
Department of
Carnall Hall 10
University of A
Fayetteville, AR

CALIFORNIA
California Dept
Division of Flo
P.O. Box 388
Sacramento, C

COLORADO
Colorado Clim
Dept. of Atmos
Colorado State
Fort Collins, C

CONNECTIC
Dept. of Renev
University of
Storrs, CT 062

DELAWARE
Department of
University of
Newark, DE 1

TABLE 4.2 Addresses of State Climatologists in the United States
Address inquiries to "State Climatologist" at the addresses below.

Location	Type of data available*
ALABAMA K. E. Johnson Environmental & Energy Center The University of Alabama–Huntsville Huntsville, AL 35899	24-hr maximum rainfall for all cooperative observing sites, plus summaries of intensities for Mobile, Montgomery, Birmingham, and Huntsville.
ALASKA AEIDC/University of Alaska Alaska Climate Center 707 A Street Anchorage, AK 99501	U.S. Dept of Commerce Technical Papers Nos. 47 and 52. National Climatic Data Center tape with 15-min and hourly precipitation data for 1976–1978.
ARIZONA The Laboratory of Climatology Arizona State University Tempe, AZ 85287	NOAA publications, plus some additional data at county scales.
ARKANSAS Department of Geography Carnall Hall 104 University of Arkansas Fayetteville, AR 72701	Hourly precipitation and local climatological data.
CALIFORNIA California Dept. of Water Resources Division of Flood Management P.O. Box 388 Sacramento, CA 95802	5-, 10-, 15-, and 30-min, 1-, 2-, 3-, 5-, 12-, and 24-hr maximum precipitation for return periods of 2, 5, 10, 20, 25, 40, 50, 100, 200, 1000, and 10,000 years and PMP. Long-duration data is also available. The data, which is on microfiche, is from 689 recording rain gauges and 853 nonrecording gauges in California.
COLORADO Colorado Climate Center Dept. of Atmospheric Science Colorado State University Fort Collins, CO 80523	NOAA Atlas 2, Hydrometeorological Report Series—PMP, daily and hourly precipitation data.
CONNECTICUT Dept. of Renewable Natural Resources University of Connecticut, Box U-87 Storrs, CT 06268	Real-time data from automated weather stations around state. Statistical analyses of data from cooperating and first-order weather stations.
DELAWARE Department of Geography University of Deleware Newark, DE 19716	Office is unfunded; services are limited. Will answer questions and locate data sources if readily available.

Location	Type of data available*
FLORIDA Department of Meteorology Florida State University Tallahassee, FL 32306	Monthly and daily rainfall amounts only.
GEORGIA Institute of Natural Resources Ecology Building University of Georgia Athens, GA 30602	Monthly total rainfalls since earliest records for 23 places representative of statewide conditions. Moving averages and plots available on contract.
HAWAII Division of Water & Land Development Dept. of Land & Natural Resources P.O. Box 373 Honolulu, HI 96809	Hourly and daily rainfall data published and available from the National Climatic Data Center, Asheville, NC 28801.
IDAHO Agricultural Engineering Dept. University of Idaho Moscow, ID 83843	National Weather Service publications. Will prepare other data on a straight-time cost basis when available. Very short duration data is scarce.
ILLINOIS Illinois State Water Survey P.O. Box 5050, Station A Champaign, IL 61820	Annual data by section of state. Currently analyzing monthly data by region.
INDIANA Department of Agronomy Room 201-5 Poultry Science Bldg. Purdue University West Lafayette, IN 47907	All official climate data for Indiana from 1901 to date on computer disks and tapes, including hourly precipitation data.
IOWA Iowa Dept. of Agriculture Weather Service Municipal Airport, Room 10 Des Moines, IA 50321	*Climatology of Excessive Short Duration Rainfall in Iowa.* Climate of Iowa Series #6; *Iowa's Greatest 24-hr. Precipitation and Related Rainstorm Data*, Climate of Iowa Series #2 (an adaptation of U.S.W.B. Tech. Paper No. 40). These publications can be ordered for $2.50 per copy.
KANSAS Dept. of Physics—Caldwell Hall Kansas State University Manhattan, KS 66505	HPD publication and magnetic tape files for these data. Not funded as a service agency and will not provide off-campus service, but files are open to all who come to the office.
KENTUCKY Kentucky Climate Center Dept. of Geography & Geology Western Kentucky University Bowling Green, KY 42101	Hourly and daily rainfall data from which frequencies can be calculated.

Location	Type of data available*
LOUISIANA Dept. of Geography & Anthropology Lousiana State University Baton Rouge, LA 70803	
MAINE Pest Management Office Cooperative Extension Service 491 College Avenue Orono, ME 04473	
MARYLAND 1123A, Jull Hall University of Maryland College Park, MD 20742	
MASSACHUSETTS Dept. of Environmental Management Division of Water Resources 496 Park Street North Reading, MA 01864	
MICHIGAN MDA/Climatology 417 Natural Science Bldg. Michigan State University East Lansing, MI 48824	Hourly and excessive rainfall data for the Detroit metropolitan area (sponsored by South East Michigan Council of Governments) and for two agricultural watersheds near East Lansing. Also has federal publications TP-25, TP-40, and Hydro-35 for reference or to make a limited number of photocopies from.
MINNESOTA Minnesota Dept. of Natural Resources University of Minnesota 279 North Hall St. Paul, MN 55108	File copies of data from the National Climatic Data Center.
MISSISSIPPI Dept. of Geology & Geography Mississippi State Mississippi State, MS 39762	Expects to publish a weekly precipitation probability analysis in late 1985.
MISSOURI Dept. of Atmospheric Science University of Missouri–Columbia 701 Hitt Street Columbia, MO 65211	Daily rainfall values for 120 locations for the period 1948–1981.
MONTANA Plant & Soil Science Department Montana State University Bozeman, MT 59717	

Location	Type of data available*
NEBRASKA CAMAC 239 L. W. Chase Hall University of Nebraska Lincoln, NE 68583-0728	Hourly and daily rainfall data, probabilities of x amount of rain in a given time period, U.S. Weather Bureau maps of rainfall frequencies and intensities, National Climatic Data Center published summaries.
NEVADA Geography Dept. University of Nevada, Reno Reno, Nevada 89557	
NEW HAMPSHIRE Dept. of Geography University of New Hampshire Durham, NH 03824	
NEW JERSEY Dept. of Meteorology & Physical Oceanography Cook College, Rutgers University P.O. Box 231 New Brunswick, NJ 08903	
NEW MEXICO P.O. Box 5702 New Mexico Dept. of Agriculture Las Cruces, NM 88003	*Precipation-Frequency Atlas of the Western U.S., Volume IV—New Mexico*, precipitation summaries for various New Mexico locations, which include 24-hr rainfall by month.
NEW YORK Northeast Regional Climate Center Box 21, Bradfield Hall Cornell University Ithaca, NY 14853	Daily totals on tape, hourly totals on paper.
NORTH CAROLINA Dept. of Marine, Earth & Atmos. Sciences P.O. Box 8208 North Carolina State University Raleigh, NC 27695-8208	Monthly and hourly precipitation data for approximately 45 stations, published by NOAA. Data can be obtained by writing to state climatologist or calling (919) 737-3056.
NORTH DAKOTA Soils Department North Dakota State University Fargo, ND 58105	
OHIO Dept. of Geography Ohio State University 103 Bricker Hall Columbus, OH 43210-1361	U.S. Weather Bureau Technical Paper No. 25 (1955).

Location	Type of data available*
OKLAHOMA Oklahoma Climatological Survey University of Oklahoma 710 Asp, Suite 8 Norman, OK 73019	U.S. Weather Bureau Technical Paper No. 40.
OREGON Climatic Research Institute Oregon State University Corvallis, OR 97331	Hourly rainfall amounts 1948–1982 (approx. 180 stations), daily rainfall amounts 1928–1982 (approx. 450 stations), monthly rainfall amounts 1900–1982 (approx. 400 stations).
PENNSYLVANIA No state climatologist at this time.	
RHODE ISLAND Dept. of Plant Sciences Room 313, Woodward Hall University of Rhode Island Kingston, RI 02881	Daily precipitation—50 years. Distribution—2 years. Published in *Climatological Data—New England.*
SOUTH CAROLINA S.C. Water Resources Commission 3830 Forest Drive P.O. Box 4440 Columbia, SC 29240	5 min to 24 hr rainfall amounts for various locations in state, plus daily and monthly rainfalls.
SOUTH DAKOTA Agricultural Engineering Dept. South Dakota State University Brookings, SD 57007	U.S. Weather Bureau Technical Paper No. 40. The values in this publication do not apply in the Black Hills area, where intensities have proved to be higher.
TENNESSEE Tennessee Valley Authority 310 Evans Building Knoxville, TN 37902	Daily rainfall totals for approx. 150 gauges in Tennessee, western Virginia, western North Carolina and northern Alabama.
TEXAS Meteorology Department Texas A&M University College Station, TX 77843	
UTAH Utah State Climatologist Utah State University, UMC-48 Logan, UT 84322	Hourly rainfall data published by National Climatic Data Center; estimated return periods for short-duration precipitation.
VERMONT Hills Building University of Vermont Burlington, VT 05401	For data, write to Northeast Regional Climate Center, Box 21, Bradfield Hall, Cornell University, Ithaca, NY 14853.

Location	Type of data available*
VIRGINIA Dept. of Environmental Sciences Clark Hall University of Virginia Charlottesville, VA 22903	Rainfall frequency and intensity data for approximately 30 locations.
WASHINGTON Office of the State Climatologist Western Washington University Bellingham, WA 98225	
WEST VIRGINIA Dr. Stanley J. Tajchman Forestry P.O. Box 6125 Morgantown, WV 26505-6125	Data and graphs for specific project sites only. For more general information, contact the National Climatic Data Center, Asheville, NC 28801, (704) 259-0682
WISCONSIN University of Wisconsin Extension 1353 Meteorology & Space Science Bldg. 1225 West Dayton St. Madison, WI 53706	Hourly and daily precipitation data on magnetic tape for all Wisconsin stations. Reference library of federal precipitation atlases and publications.
WYOMING No state climatologist at this time.	

*Source: Responses to a survey conducted by the authors in 1985.

In other states, the office of the state climatologist compiles similar rainfall data. Table 4.2 lists the addresses of state climatologists in 48 states. (Pennsylvania and Wyoming did not have state climatologists at the time of writing.) Local flood control and public works agencies can often provide many more rain gauge stations, but the intensity information may be limited or nonexistent.

The California DWR presents the precipitation values in tabular form by rain gauge for different return intervals and rainfall durations. Table 4.3 shows the values for one station in California. Microfiche sets of the tables, the only practical way of presenting 3687 pages of data, are updated annually and are available from the DWR Publications Office in Sacramento. (2) Detailed rainfall intensity data for specific subregions of the state, such as the report prepared by Rantz covering the San Francisco Bay region (9) may also be available.

Tabular precipitation data such as that shown in Table 4.3, is the easiest form of data from which to determine i values for use in the rational formula. In areas east of the Rocky Mountains, for which detailed tabulations are not available, maximum rainfalls of ½-, 1-, 2-, 3-, 6-, 12-, and 24-hr duration can be estimated by using the *Rainfall Frequency Atlas of the United States.* (6) The atlas contains maps of total rainfall for storms of those durations and return periods of 1, 2, 5, 10, 24, 50, and 100 years. Figures 4.3 and 4.4 are simplified versions of the atlas maps of the 10-year, 30-min rainfall and the 10-year, 6-hr rainfall, respectively.

TABLE 4.3 Precipitation Values for Rain Gauge 2933, Fairfield, California (2)

Return period, years	Maximum precipitation (in) for indicated duration: m = minutes; h = hours										
	5m	10m	15m	30m	1h	2h	3h	6h	12h	24h	C–yr
2	0.17	0.23	0.27	0.36	0.48	0.68	0.83	1.16	1.54	2.07	20.47
5	0.23	0.33	0.38	0.50	0.67	0.95	1.17	1.64	2.17	2.92	26.70
10	0.28	0.39	0.45	$\boxed{0.60}$	$\boxed{0.80}$	1.14	1.60	$\boxed{1.95}$	2.59	3.49	30.29
20	0.32	0.46	0.52	$\boxed{0.69}$	$\boxed{0.93}$	1.31	1.62	2.25	2.98	4.02	33.45
25	0.34	0.47	0.55	0.72	0.97	1.37	1.68	2.35	3.11	4.19	34.40
40	0.36	0.51	0.59	0.78	1.05	1.48	1.82	2.54	3.36	4.53	36.32
50	0.38	0.53	0.61	0.81	1.08	1.53	1.89	2.63	3.48	4.70	37.20
100	0.42	0.59	0.68	0.90	1.20	1.69	2.09	2.91	3.85	5.19	39.82
200	0.46	0.64	0.74	0.98	1.31	1.85	2.28	3.18	4.21	5.68	42.30
1,000	0.54	0.77	0.88	1.17	1.57	2.21	2.73	3.80	5.03	6.78	47.68
10,000	0.67	0.94	1.08	1.44	1.92	2.71	3.35	4.66	6.17	8.32	54.74
PMP	1.28	1.81	2.08	2.75	3.68	5.20	6.41	8.93	11.83	15.95	125.87

PMP = probable maximum precipitation

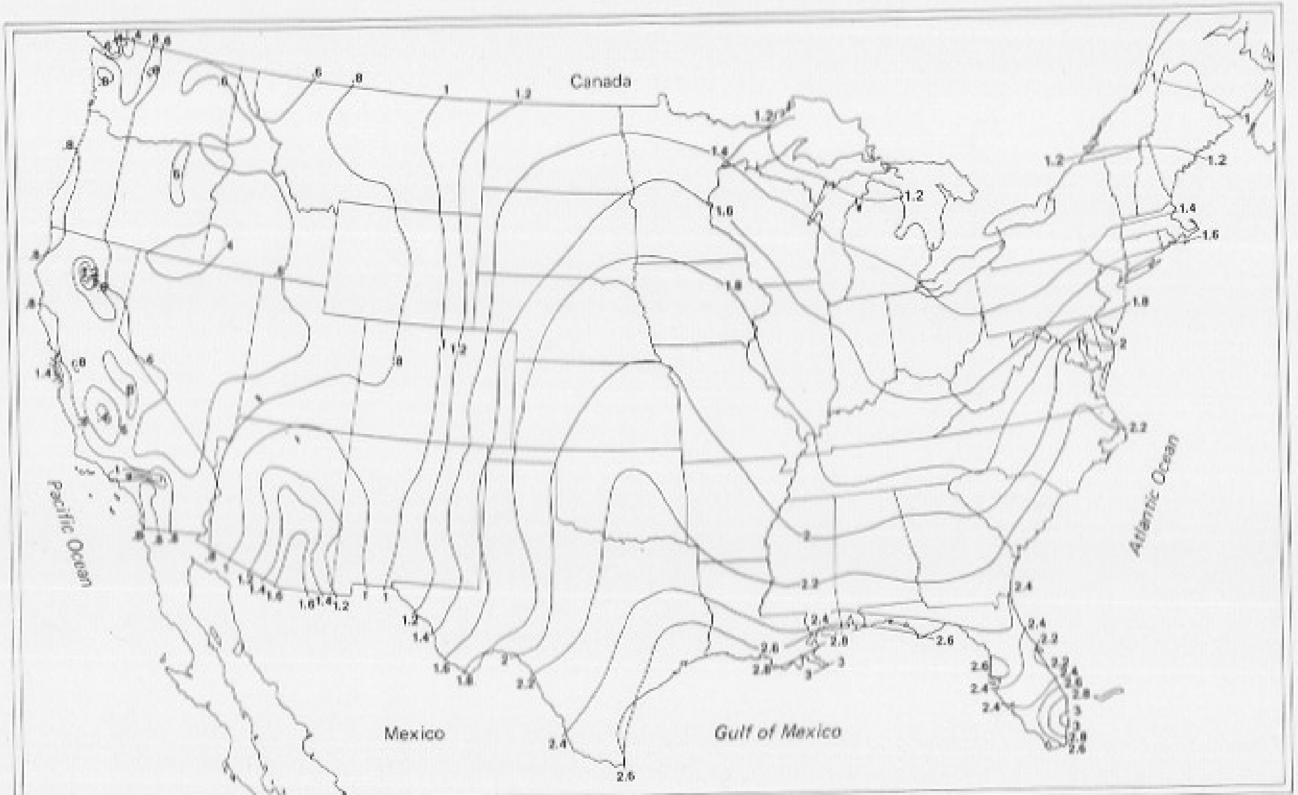


Fig. 4.3 Map of 10-year, 30-min rainfall, in inches. (Adapted from 6)

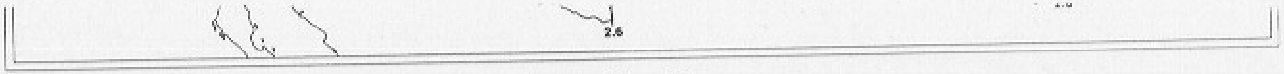


Fig. 4.3 Map of 10-year, 30-min rainfall, in inches. (Adapted from 6)

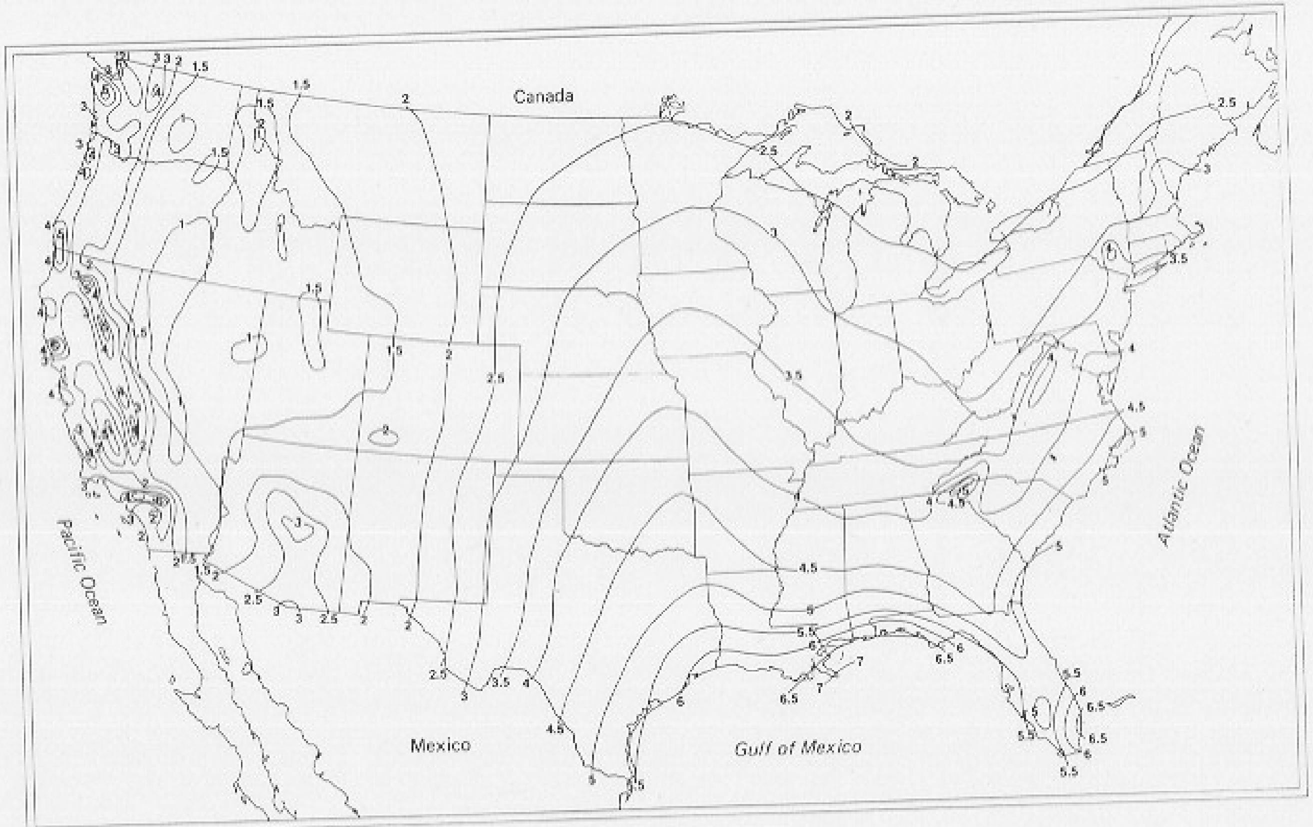


Fig. 4.4 Map of 10-year, 6-hr rainfall, in inches. (Adapted from 6)

Short-duration maximum rainfalls of 5 to 60 min in areas east of the Rocky Mountains can be estimated from NOAA Technical Memorandum NWS Hydro-35. (5) This document can be ordered from the National Technical Information Service, 5285 Port Royal Road, Springfield, VA 22161 (Publication No. PB 272-112). In the 11 western states, precipitation intensities can be estimated by using NOAA's *Precipitation Frequency Atlas of the Western States.* (8) This document, published in 11 separate volumes, can be ordered from the National Climatic Data Center, Federal Building, Asheville, NC 28801. It is available in both microfilm and paper versions, and the microfilm is substantially less expensive. Call the Data Center in Asheville, (704) 259-0682, for a price estimate before ordering.

The National Environmental Data Referral Service (NEDRES) publishes a *Data Base User's Guide.* The guide describes various types of atmospheric data sets that have been developed in specific areas of the United States, lists the responsible agency for each area, and tells you how to order the data. To order the guide, contact the National Environmental Data Referral Service Office, 3300 White Haven Street, N.W., Room 533, Washington, DC 20235 [(202) 634-7722]. On-line access to the data base also is available.

The following examples illustrate the use of the two types of precipitation data (tables and maps) for computing i values.

EXAMPLE 4.2

Given: A site in Fairfield, California.

Find: The average precipitation intensity i for a 10-year, 6-hr storm by using Table 4.3.

Solution: The precipitation data table for the Fairfield rain gauge (Table 4.3) is obtained from the California DWR microfiche set. The table shows that 1.95 in (50 mm) of rain can be expected from the desired storm. Dividing 9.5 by 6 hr gives an average intensity of 0.33 in/hr (8.4 mm/hr).

EXAMPLE 4.3

Given: A site in central Wisconsin.

Find: The maximum rainfall to be expected from a 10-year, 30-min rainfall and the average intensity i.

Solution: In Fig. 4.3, the 10-year, 30-min rainfall is between 1.4 and 1.6 in (36 and 41 mm) for central Wisconsin. Interpolating between the isohyets produces a maximum rainfall of 1.5 in (38 mm).

The average intensity i for this storm duration and frequency is found by dividing i by the duration of 0.5 hr for an average i of 3 in/hr (76 mm/hr).

EXAMPLE 4.4

Given: A site in Fairfield, California.

Find: The average intensity i of a rainfall with a duration of 45 min and return interval of 15 years.

Solution: Using Table 4.3, we find the precipitation for a 10-year, 30-min event to be 0.60 in (15 mm); a 20-year, 30-min event to be 0.69 in (18 mm); a 10-year, 1-hr event to be 0.80 in (20 mm); and a 20-year, 1-hr event to be 0.93 in (24 mm). By interpolating to find the 10-year, 45-min rainfall, we get 0.70 in (18 mm), and the 20-year, 45-min rainfall is 0.81 in (21 mm). By interpolating again between the 10- and 20-year values, we get a 15-year, 45-min rainfall of 0.75 in (19 mm). This is sufficient precision for erosion control design.

4.1g Time of Concentration T_c

As discussed earlier, Q_{avg} is used to size water retention structures and is based upon the total precipitation during a storm event of selected length. It is a straightforward task to select a 6-hr storm period for design, find the average i over the 6 hr, and calculate the average runoff.

For sizing flow conveyance structures, the peak flow Q_{peak} must be selected on the basis of site-specific factors that affect the time of concentration. If a channel is sized for a peak i averaged over a shorter duration than the time of concentration T_c, the channel will be oversized. This is because, for any return frequency event, the shorter-duration events (such as 5 or 10 min) will produce a greater i than will a longer-duration event (such as 30 min or 1 hr). Conversely, a channel sized for a Q based upon a rainfall duration exceeding the T_c will be undersized.

Remember that, for a rainfall duration less than T_c, not all of the rain falling upon the earth will be reflected at the measuring point in the channel. Time must be allowed for overland flow. But for a rainfall event with intensity averaged over T_c, at T_c the runoff measured in the channel will reflect the rate of rainfall over the site in question.

To calculate Q_{peak} we need to use an intensity i based upon the storm frequency of choice and a duration equal to the T_c. The time of concentration consists of two parts:

- Time for overland flow to reach a drainage channel
- Channel flow time from the point of entry into the channel to the point under construction

Because channel flow is much faster than overland flow, the total time of concentration does not necessarily represent the greatest distance traveled; it represents the longest travel time. The overland flow time is usually the major portion of the time of concentration.

Overland Flow Time

Overland flow time is a function of the length of travel path, average slope, and the rational method runoff coefficient C. A nomograph relating those three criteria (Fig. 4.5) can be used to estimate overland travel time. To use the nomograph, follow these steps:

- Determine the distance from the most remote part of the drainage area to the intercepting channel.

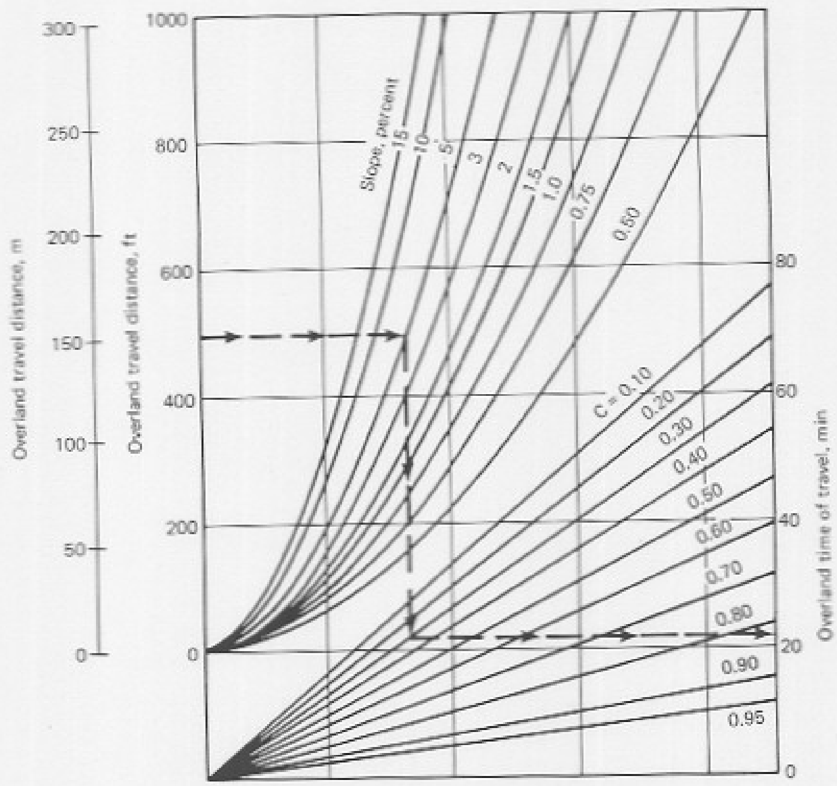


Fig. 4.5 A nomograph of overland flow time. (10) Enter left margin with slope length; move right to slope curve and down to C value; and find overland travel time on right margin.

- Calculate the average slope by computing the difference in altitude between the highest and lowest points of the flow path and dividing by the distance between those points.
- Find or compute the C value.
- Enter the graph on the left margin with the overland travel distance; move to the right to the correct slope curve; move down to the C value; and then move over to the right margin.
- Read the overland flow time from the right-hand scale.

EXAMPLE 4.5

Given: A site 500 ft (152 m) long with 5 percent average slope and a C value of 0.30.

Find: Overland flow time.

Solution: From Fig. 4.5, the estimated flow time is 22 min.

Channel Fl

Channel flo when a shor to a sedimer flow might b travel time: not erode t struction sit channel is u the soil typ damage on runoff can unprotected

Two equ equation an civil engine

where V =

n =

r =

s =

In metric u

where V is

The *cor*

where Q =

A =

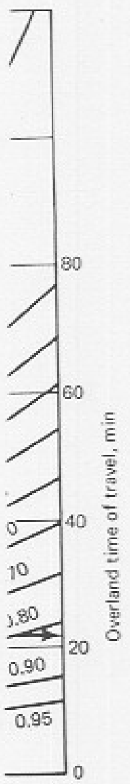
V =

The M Tables of Typical va lined chan

where n =

d_{50} =

Figure 7.2



ı with slope
l travel time

ıude between
the distance

ınce; move to
ıd then move

value of 0.30.

Channel Flow Time

Channel flow time can sometimes be ignored in calculating runoff. For example, when a short, paved channel collects water from a broad gentle slope and drains to a sediment basin, the channel travel time may be 2 min whereas the overland flow might be 20 min or more. There are two important reasons to know channel travel time: to assure adequate capacity and to be sure that the flow of water will not erode the channel. Frequently, temporary diversions and channels on construction sites either are not lined or are not sized for the expected flow. If the channel is unlined and the velocity exceeds the maximum permissible value for the soil type, the channel bed itself erodes, which further contributes to erosion damage on and off site. If the channel is not large enough to carry the flow, the runoff can overflow, form new channels that also erode, and severely damage unprotected slopes.

Two equations are used to calculate the flow in open channels: Manning's equation and the continuity equation. These equations should be familiar to all civil engineers. The *Manning equation* is:

$$V = \frac{1.49 \times r^{2/3} \times s^{1/2}}{n}$$

where V = velocity, ft/sec
n = Manning roughness coefficient, dimensionless
r = hydraulic radius, ft, at the depth of flow, i.e., channel cross-sectional area, ft^2, divided by wetted perimeter, ft, A/WP
s = average streambed slope, in decimals

In metric units

$$V = \frac{r^{2/3} \times s^{1/2}}{n}$$

where V is in m/sec, r is in m, and n and s are the same as above.

The *continuity equation* is:

$$Q = A \times V$$

where Q = flow in the channel, ft^3/sec (m^3/sec)
A = cross-sectional area of the flow, ft^2 (m^2)
V = velocity, ft/sec (m/sec)

The Manning roughness coefficient n reflects the condition of the channel. Tables of the coefficient can be found in handbooks used by drainage designers. Typical values useful in erosion control work are shown in Table 4.4. For riprap-lined channels, n can be determined from the following equation:

$$n = 0.0395 d_{50}^{1/6}$$

where n = Manning's roughness coefficient
d_{50} = median size stone in the mixture of riprap, ft (For d_{50} in meters, $n = 0.0481 d_{50}^{1/6}$)

Figure 7.29 solves this equation graphically.

TABLE 4.4 Manning Roughness Coefficients

n	Channel surface condition
0.013	Plastic sheet
0.015	Concrete or asphalt-lined channel
0.02	Ordinary earth, smoothly graded
0.025	Gravel-lined channel
0.030	Natural channel in good condition
0.040	Sod, depth of flow more than 6 in
0.10	Weed-choked natural channel

Note: The n value for sod, 0.040, is appropriate for a grassed waterway but changes slightly with the height of the grass (see Appendix B).

By far the easiest way to estimate channel travel time as a contributing part of time of concentration is to use the *Handbook of Hydraulics.* (1) Section 7 of the handbook covers steady uniform flow in open channels and contains numerous tables derived from the Manning equation.

The solution of a channel-sizing problem is an iterative process because a flow must be assumed before a depth and velocity can be calculated. However, the flow depends on the time of concentration, which, when it includes channel flow time, itself depends on flow. A two-part (or more, if channel size is altered) procedure is used to solve for T_c, V, and Q_{peak}:

- Find an initial flow Q_{peak} based on overland flow time.
- Use this initial Q_{peak}, the channel characteristics, and the tables in the *Handbook of Hydraulics* to solve for depth of flow.
- Use this value of depth of flow and Manning's equation to obtain an initial estimate of velocity.
- Find channel travel time by dividing the channel length by velocity:

$$\frac{L}{60V} = \text{travel time, min}$$

Now repeat this sequence; use the new total time of concentration equal to overland flow time plus channel travel time:

- With the new T_c, find new i value and Q_{peak}.
- Use the *Handbook of Hydraulics* to solve for depth of flow.
- Reapply Manning's equation to find the revised estimate of velocity in the channel.
- Check depth of flow and quantity against channel size and capacity. If channel is not adequate, redesign is necessary.
- Check velocity against maximum permissible velocity for the specific channel lining. Maximum velocities for unlined channels are listed in Table 7.1. Figures

7.36, 7.37, and 7.39 also allow a direct check for erosive velocities based on depth of flow and channel slope for several types of lining material.

If the *Handbook of Hydraulics* is not available, manual calculation is possible, although laborious. A calculator helps make the process somewhat less time-consuming. In this case, channel travel time is first calculated by assuming full flow depth, and actual depth and velocity are found by iterative applications of the equations. Section 4.1h contains examples of the use of the rational method to find Q_{peak} and examples of the ways discussed above to find the time of concentration T_c.

4.1h Examples of Use of the Rátional Method

The following examples build on those already presented to illustrate how the individual components of the rational equation combine to yield estimates of Q_{peak}. Examples 4.6 and 4.7 develop a Q_{peak} based entirely on overland flow. Example 4.8 is a simple average flow calculation. Example 4.9 is a complete calculation of the adequacy of a channel. It includes computing a weighted C value and overland flow time, approximating channel flow time and checking for capacity and susceptibility of the channel to erosion, and using the *Handbook of Hydraulics* to help solve Manning's equation. Example 4.10 repeats the computation in Example 4.8 without using the *Handbook of Hydraulics*.

EXAMPLE 4.6

Given: A 3-acre (1.2-ha) site in southern Tennessee.
Slope length of 500 ft (152 m)
Average slope of 5 percent
C value of 0.30
T_c of 22 min
Detailed rain gauge data not available

Find: Peak intensity i_{peak} for a 10-year storm for sizing of conveyance channels.

Solution: Detailed rain gauge data is generally available at major metropolitan centers. If this data has not been compiled or is not available, the *Rainfall Frequency Atlas* (6)can be employed. Refer to the 10-year, 30-min map and find 1.8 in (46 mm) of rainfall over 30 min. For short-duration events, at each halving of the time from 30 min, we recommend a conservative assumption of 90 percent of the preceding rainfall. Thus for a 15-min event, the design rainfall would be 0.90 × 1.8 in = 1.62 in (41 mm) and for a 7.5-min event it would be 0.90 × 1.62 = 1.46 in (37 mm).

The T_c for this problem is 22 min. Interpolation between 15 min = 1.62 in (41 mm) and 30 min = 1.8 in (46 mm) produces a design rainfall of 22 min = 1.70 in (43 mm). The result is

$$i_{peak} = \frac{1.70 \text{ in}}{22 \text{ min}} \times 60 \text{ min/hr} = 4.6 \text{ in/hr} \qquad \frac{43 \text{ mm}}{22 \text{ min}} \times \frac{60 \text{ min}}{\text{hr}} = 117 \text{ mm/hr}$$

This i_{peak} will usually result in a flow estimation higher than is likely to be encountered. However, a channel sized to handle a flow with such a short T_c will generally serve a small area, and the total excess cost to the builder is likely to be very minor.

EXAMPLE 4.7 The example is in two parts. In the first part we find Q_{peak}.

Given: The site in southern Tennessee of Example 4.6.
C value of 0.30
i_{peak} = 4.6 in/hr (117 mm/hr) (from Example 4.6)
Area of 3 acres (1.2 ha)

Find: Peak flow Q_{peak} for a 10-year storm.

Solution:

$$Q_{peak} = C \times i_{peak} \times A$$
$$= 0.30(4.6)(3) \qquad [0.30(117 \text{ mm/hr})(1.2 \text{ ha}/360)]$$
$$= 4.14 \text{ ft}^3/\text{sec} \qquad (0.117 \text{ m}^3/\text{sec})$$

Peak flow is used to size channels and risers for sediment basins and to check for potential erosion of the channel bed.

In this part of the example we find Q_{avg}.

Given: The southern Tennessee site of Example 4.6.

Find: Average flow Q_{avg} for 10-year, 6-hr storm event.

Solution: First, find i_{avg}. From Fig. 4.4, for 10-year, 6-hr precipitation intensity:
10-year, 6-hr rainfall = 3.7 in (94 mm)
i_{avg} = 3.7 in/6 hr = 0.62 in/hr (16 mm/hr)
$Q_{avg} = C \times i_{avg} \times A$
Q_{avg} = 0.30(0.62)(3) [0.30(16 mm/hr)(1.2 ha)/360)]
= 0.56 ft^3/sec (0.016 m^3/sec)
This average flow rate could be used to size a sediment basin.

Basic, Step-by-Step Procedure for Applying the Rational Method to a Channel Flow Problem

1. Determine drainage area.
2. Determine the proper C value for the site.
3. Determine overland flow time. Enter Fig. 4.5 with the slope length, slope percent, and the C value to find overland flow time.
4. Find an initial i value based on overland flow time. Use Fig. 4.3 and calculate i or, preferably, use rain gauge data from a nearby weather station. Remember to convert inches (millimeters) of rain for the storm duration to inches per hour (millimeters per hour) to obtain intensity i.
5. Compute initial Q_{peak}:

$$Q_{peak} = C \times i \times A$$

6. Use the tables in the *Handbook of Hydraulics* (1) and Manning's equation to determine initial estimates of the depth and velocity of flow in the channel.

7. Compute channel travel time by dividing the channel length by velocity of flow. Convert to minutes.
8. Determine total time of concentration: add overland flow time to channel travel time.
9. Repeat steps 4 through 6 for a second approximation of Q, depth of flow, and velocity, starting with a new i value based on the total time of concentration from step 8.
10. Determine adequacy of channel size and lining: Compare depth of flow with channel size and compare velocity with the maximum permissible velocity in an unlined channel for the soil type in the channel. If depth of flow plus minimum freeboard is deeper than the channel, choose a larger channel and repeat the calculations to verify the new size. If the capacity, but not the lining, is adequate, select a more erosion resistant channel lining.

EXAMPLE 4.8 Channel Flow Calculations Using the *Handbook of Hydraulics* (1)

Given: The sample site and the channel cross section shown in Fig. 4.6.
- Location, southeastern corner of Michigan
- 10-year, 30-min rainfall of 1.4 in (36 mm)
- 10-year, 1-hr rainfall of 1.6 in (41 mm)
- Drainage area of 20 acres (8.1 ha)
- Degree of development: rough graded bare earth on 70 percent of the watershed; the remaining 30 percent in natural condition
- Overland travel path characteristics
  - Overland travel distance, 850 ft (259 m)
  - Average slope of overland travel path, 5 percent

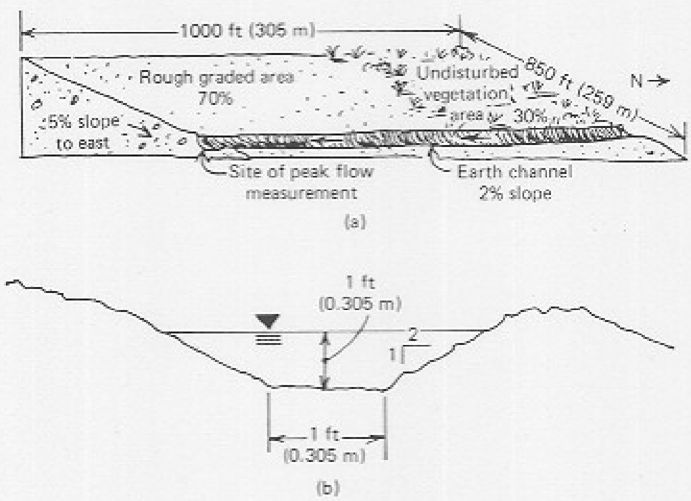


Fig. 4.6 Project area and channel geometry for Example 4.8.

Average channel characteristics
Length L, 1000 ft (305 m)
Slope s, 2 percent
Channel bed, bare earth
Roughness coefficient n, 0.02
Base width b, 1 ft (0.305 m)
Depth D, 1 ft (0.305 m)
Side slope z, 2:1

Channel section formulas are given in Appendix B. For trapezoidal channels with 2:1 side slopes,

Cross-sectional area: $A = bD + 2D^2$
Wetted perimeter: $\mathrm{WP} = b + 2D\sqrt{5}$
Hydraulic radius: $r = A/\mathrm{WP}$

Find: Peak discharge for a 10-year return interval storm, depth of flow in channel, and velocity of flow in channel.

Solution:

STEP 1. Drainage area is given; it is 20 acres (8.1 ha).

STEP 2. Weighted C value.
Using Table 4.1, a rough graded surface (70 percent of area) has a C of about 0.20.
Undisturbed areas (30 percent of area) have a C value of 0.10.
The weighted C value is

$$C = \frac{(0.7 \times 0.2) + (0.3 \times 0.1)}{1.0} = 0.17$$

STEP 3. Overland flow time. Overland flow time should be calculated on a long slope. In this example of a rectangular property, the proper slope length is 850 ft (259 m). The slowest overland flow will occur in the vegetated area, where the C value is 0.10. If the south end of this slope had been significantly larger than the vegetated north end, we would have chosen the south end to calculate overland flow time.

Enter Fig. 4.5 with a length of 850 ft (259 m), a slope of 5 percent, and a C value of 0.10 and read an overland travel time of 37 min.

STEP 4. Initial i value based on overland travel time only. From the *Rainfall Frequency Atlas* (6) we find the 10-year, 30-min rainfall to be 1.4 in (36 mm) and the 10-year 1-hr rainfall to be 1.6 in (41 mm). Interpolating between the numbers produces a rainfall of 1.45 in (36.8 mm) in 37 min.

$$i = \frac{1.45\text{ in}}{37\text{ min}} \times \frac{60\text{ min}}{\text{hr}} = 2.35\text{ in/hr} \qquad \left(\frac{36.8\text{ mm}}{37\text{ min}} \times \frac{60\text{ mm}}{\text{hr}} = 59.7\text{ mm/hr}\right)$$

STEP 5. Initial Q_{peak}.
$C = 0.17$ (from step 2)
$i = 2.35$ in/hr (59.7 mm/hr) (from step 4)
$A = 20$ acres (8.1 ha)

$$\begin{aligned} Q_{peak} &= C \times i \times A \\ &= 0.17(2.35)(20) \qquad \left[\frac{0.17(59.7)(8.1)}{360}\right] \\ &= 7.99\text{ ft}^3\text{/sec} \qquad (0.228\text{ m}^3\text{/sec}) \end{aligned}$$

STEP 6. Determine depth of flow. In the *Handbook of Hydraulics*, Table 7-11 (reproduced here as Table 4.5) provides the ratio of depth to base width after solving the following equation:

$$K' = \frac{Q\ (\text{in ft}^3/\text{sec}) \times n}{b^{8/3} \times s^{1/2}} \qquad \left[\frac{1.486 \times Q\ (\text{in m}^3/\text{sec}) \times n}{b^{8/3}\ (\text{in m}) \times s^{1/2}}\right]$$

where b = base width of trapezoidal channel = 1 ft (0.305 m)

n = Manning's roughness coefficient = 0.02
Q = 7.99 ft³/sec (0.228 m³/sec)

Therefore, K' = 1.13.

Find 1.13 in Table 4.5 in the column "2:1 side slope" and read D/b, where D = depth of water in channel and D/b = 0.64. Since b = 1 ft (0.305 m), D = 0.64 ft (0.20 m).

Initial estimate of velocity. First, Table 4.6 (Table 7.1 in the *Handbook of Hydraulics*) provides the hydraulic radius by the formula

$$r = C_r \times D$$

where C_r = tabulated value

For D/b = 0.64, find C_r = 0.590 in Table 4.6 in column "2:1 side slope."

$$r = 0.59(0.64\ \text{ft}) = 0.38\ \text{ft} \qquad [0.59(0.20\ \text{m}) = 0.12\ \text{m}]$$

Second, find velocity using Manning's equation:

$$V = \frac{1.49 \times r^{2/3}\, s^{1/2}}{n}$$

$$V = \frac{1.49(0.52\ \text{ft})(0.1414)}{0.02} \qquad \left[\frac{(0.24\ \text{m})(0.1414)}{0.02}\right]$$

$$= 5.47\ \text{ft/sec} \qquad (1.7\ \text{m/sec})$$

Step 7. Channel travel time, in minutes:

$$\frac{L}{60V} = \frac{1000\ \text{ft}}{(60\ \text{sec/min})(5.47\ \text{ft/sec})} = 3\ \text{min} \qquad \left[\frac{305\ \text{m}}{(60\ \text{sec/min})(1.7\ \text{m/sec})} = 3\ \text{min}\right]$$

Step 8. New time of concentration: overland plus channel travel time.

$$T_c = 37 + 3 = 40\ \text{min}$$

This new time of concentration is about 8 percent greater than the initial estimate used to determine the i value.

Step 9. Repeat steps 4, 5, and 6 to find second approximation of Q_{peak}, velocity, and depth of flow. Begin with a recalculation of i based upon the new T_c from step 8.

10-year, 30-min rainfall = 1.4 in (36 mm)
10-year, 1-hr rainfall = 1.6 in (41 mm)
By interpolation, 10-year, 40-min rainfall = 1.47 in (38 mm)

$$i = \frac{1.47\ \text{in}}{40\ \text{min}} \times \frac{60\ \text{min}}{\text{hr}} = 2.21\ \text{in/hr} \qquad \left(\frac{38\ \text{mm}}{40\ \text{min}} \times \frac{60\ \text{min}}{\text{hr}} = 57\ \text{mm}\right)$$

Revised Q_{peak}:

$$Q_{peak} = C \times i \times A$$

$$= 0.17(2.21\ \text{in})(20\ \text{acres}) \qquad \left[\frac{0.17(57\ \text{mm})(8.1\ \text{ha})}{360}\right]$$

$$= 7.5\ \text{ft}^3/\text{sec} \qquad (0.22\ \text{m}^3/\text{sec})$$

TABLE 4.5 Solution of Manning Equation for Trapezoidal Channels*

Values of K' in formula $Q = \frac{K'}{n} b^{8/3} S^{1/2}$ for Trapezoidal Channels† D = depth of water b = bottom width of channel

$\frac{D}{b}$	Side slopes of channel, ratio of horizontal to vertical									
	Vertical	¼–1	½–1	¾–1	1–1	1½–1	2–1	2½–1	3–1	4–1
.01	.00068	.00068	.00069	.00069	.00069	.00069	.00069	.00069	.00070	.00070
.02	.00213	.00215	.00216	.00217	.00218	.00220	.00221	.00222	.00223	.00225
.03	.00414	.00419	.00423	.00426	.00428	.00433	.00436	.00439	.00443	.00449
.04	.00660	.00670	.00679	.00685	.00691	.00700	.00708	.00716	.00723	.00736
.05	.00946	.00964	.00979	.00991	.01002	.01019	.01033	.01047	.01060	.01086
.06	.0127	.0130	.0132	.0134	.0136	.0138	.0141	.0143	.0145	.0150
.07	.0162	.0166	.0170	.0173	.0175	.0180	.0183	.0187	.0190	.0197
.08	.0200	.0206	.0211	.0215	.0219	.0225	.0231	.0236	.0240	.0250
.09	.0241	.0249	.0256	.0262	.0267	.0275	.0282	.0289	.0296	.0310
.10	.0284	.0294	.0304	.0311	.0318	.0329	.0339	.0348	.0358	.0376
.11	.0329	.0343	.0354	.0364	.0373	.0387	.0400	.0413	.0424	.0448
.12	.0376	.0393	.0408	.0420	.0431	.0450	.0466	0482	.0497	.0527
.13	.0425	.0446	.0464	.0480	.0493	.0516	.0537	.0556	.0575	.0613
.14	.0476	.0502	.0524	.0542	.0559	.0587	.0612	.0636	.0659	.0706
.15	.0528	.0559	.0585	.0608	.0627	.0662	.0692	.0721	.0749	.0805
.16	.0582	.0619	.0650	.0676	.0700	.0740	.0777	.0811	.0845	.0912
.17	.0638	.0680	.0716	.0748	.0775	.0823	.0866	.0907	.0947	.1026
.18	.0695	.0744	.0786	.0822	.0854	.0910	.0960	.1008	.1055	.1148
.19	.0753	.0809	.0857	.0899	.0936	.1001	.1059	.1115	.1169	.1277
.20	.0812	.0876	.0931	.0979	.1021	.1096	.1163	.1227	.1290	.1414
.21	.0873	.0945	.101	.106	.111	.120	.127	.135	.142	.156
.22	.0934	.1015	.109	.115	.120	.130	.139	.147	.155	.171
.23	.0997	.1087	.117	.124	.130	.141	.150	.160	.169	.187
.24	.1061	.1161	.125	.133	.140	.152	.163	.173	.184	.204
.25	.1125	.1236	.133	.142	.150	.163	.176	.188	.199	.222
.26	.119	.131	.142	.152	.160	.175	.189	.202	.215	.241
.27	.126	.139	.151	.162	.171	.188	.203	.218	.232	.260
.28	.132	.147	.160	.172	.182	.201	.217	.234	.249	.281
.29	.139	.155	.170	.182	.194	.214	.232	.250	.268	.302
.30	.146	.163	.179	.193	.205	.228	.248	.267	.287	.324
.31	.153	.172	.189	.204	.218	.242	.264	.285	.306	.347
.32	.160	.180	.199	.215	.230	.256	.281	.304	.327	.371
.33	.167	.189	.209	.227	.243	.271	.298	.323	.348	.396
.34	.174	.198	.219	.238	.256	.287	.316	.343	.370	.423
.35	.181	.207	.230	.251	.269	.303	.334	.363	.392	.450
.36	.189	.216	.241	.263	.283	.319	.353	.385	.416	.478
.37	.196	.225	.252	.275	.297	.336	.372	.406	.440	.507
.38	.203	.234	.263	.288	.312	.353	.392	.429	.465	.537
.39	.211	.244	.274	.301	.326	.371	.413	.452	.491	.568
.40	.218	.253	.286	.315	.341	.389	.434	.476	.518	.600
.41	.226	.263	.297	.328	.357	.408	.456	.501	.546	.633
.42	.233	.273	.309	.342	.373	.427	.478	.526	.574	.668
.43	.241	.283	.321	.357	.389	.447	.501	.553	.603	.703
.44	.248	.293	.334	.371	.405	.467	.525	.580	.633	.740
.45	.256	.303	.346	.386	.422	.488	.549	.607	.664	.777

$\frac{D}{b}$	Vertical
.46	.264
.47	.271
.48	.279
.49	.287
.50	.295
.51	.303
.52	.311
.53	.319
.54	.327
.55	.335
.56	.343
.57	.351
.58	.359
.59	.367
.60	.375
.61	.383
.62	.391
.63	.399
.64	.408
.65	.416
.66	.424
.67	.433
.68	.441
.69	.449
.70	.457
.71	.466
.72	.474
.73	.483
.74	.491
.75	.499
.76	.508
.77	.516
.78	.525
.79	.533
.80	.542
.81	.550
.82	.559
.83	.567
.84	.576
.85	.585
.86	.593
.87	.602
.88	.610
.89	.619
.90	.628

*E. F. Brat-
pany, 1976.

†K' values

h of

-1	4-1
0070	.00070
0223	.00225
0443	.00449
0723	.00736
1060	.01086
145	.0150
190	.0197
240	.0250
296	.0310
358	.0376
424	.0448
497	.0527
575	.0613
659	.0706
749	.0805
845	.0912
947	.1026
055	.1148
169	.1277
290	.1414
42	.156
55	.171
69	.187
84	.204
99	.222
215	.241
232	.260
249	.281
268	.302
287	.324
306	.347
327	.371
348	.396
370	.423
392	.450
416	.478
440	.507
465	.537
491	.568
518	.600
546	.633
574	.668
603	.703
633	.740
664	.777

$\frac{D}{b}$	Side slopes of channel, ratio of horizontal to vertical									
	Vertical	¼-1	½-1	¾-1	1-1	1½-1	2-1	2½-1	3-1	4-1
.46	.264	.313	.359	.401	.439	.509	.574	.636	.696	.816
.47	.271	.323	.372	.416	.457	.531	.599	.665	.729	.856
.48	.279	.334	.385	.432	.474	.553	.625	.695	.763	.897
.49	.287	.344	.398	.447	.493	.575	.652	.725	.797	.939
.50	.295	.355	.412	.463	.511	.598	.679	.757	.833	.983
.51	.303	.366	.425	.480	.530	.622	.707	.789	.869	1.03
.52	.311	.377	.439	.496	.549	.646	.736	.822	.907	1.07
.53	.319	.388	.453	.513	.569	.671	.765	.856	.945	1.12
.54	.327	.399	.467	.531	.589	.696	.795	.891	.984	1.17
.55	.335	.410	.482	.548	.609	.722	.826	.926	1.025	1.22
.56	.343	.422	.497	.566	.630	.748	.857	.963	1.07	1.27
.57	.351	.433	.511	.584	.651	.775	.889	1.000	1.11	1.32
.58	.359	.445	.526	.602	.673	.802	.922	1.038	1.15	1.37
.59	.367	.456	.542	.621	.694	.830	.956	1.077	1.20	1.43
.60	.375	.468	.557	.640	.717	.858	.990	1.117	1.24	1.49
.61	.383	.480	.573	.659	.739	.887	1.02	1.16	1.29	1.54
.62	.391	.492	.588	.678	.762	.916	1.06	1.20	1.33	1.60
.63	.399	.504	.604	.698	.785	.946	1.10	1.24	1.38	1.66
.64	.408	.516	.620	.718	.809	.977	1.13	1.28	1.43	1.72
.65	.416	.529	.637	.738	.833	1.008	1.17	1.33	1.48	1.79
.66	.424	.541	.653	.759	.857	1.04	1.21	1.37	1.53	1.85
.67	.433	.553	.670	.780	.882	1.07	1.25	1.42	1.59	1.91
.68	.441	.566	.687	.801	.907	1.10	1.29	1.47	1.64	1.98
.69	.449	.579	.704	.822	.933	1.14	1.33	1.51	1.69	2.05
.70	.457	.592	.722	.844	.959	1.17	1.37	1.56	1.75	2.12
.71	.466	.604	.739	.866	.985	1.21	1.41	1.61	1.81	2.19
.72	.474	.617	.757	.889	1.012	1.24	1.46	1.66	1.86	2.26
.73	.483	.631	.775	.911	1.039	1.28	1.50	1.71	1.92	2.34
.74	.491	.644	.793	.934	1.067	1.31	1.54	1.77	1.98	2.41
.75	.499	.657	.811	.957	1.095	1.35	1.59	1.82	2.05	2.49
.76	.508	.670	.830	.981	1.12	1.39	1.63	1.87	2.11	2.57
.77	.516	.684	.849	1.005	1.15	1.43	1.68	1.93	2.17	2.65
.78	.525	.698	.868	1.029	1.18	1.46	1.73	1.99	2.24	2.73
.79	.533	.711	.887	1.053	1.21	1.50	1.78	2.05	2.30	2.81
.80	.542	.725	.906	1.078	1.24	1.54	1.83	2.10	2.37	2.90
.81	.550	.739	.925	1.10	1.27	1.58	1.88	2.16	2.44	2.98
.82	.559	.753	.945	1.13	1.30	1.62	1.93	2.22	2.51	3.07
.83	.567	.767	.965	1.15	1.33	1.67	1.98	2.28	2.58	3.16
.84	.576	.781	.985	1.18	1.36	1.71	2.03	2.34	2.65	3.25
.85	.585	.796	1.006	1.21	1.40	1.75	2.08	2.41	2.72	3.35
.86	.593	.810	1.03	1.23	1.43	1.79	2.14	2.47	2.80	3.44
.87	.602	.825	1.05	1.26	1.46	1.84	2.19	2.54	2.87	3.54
.88	.610	.839	1.07	1.29	1.49	1.88	2.25	2.60	2.95	3.63
.89	.619	.854	1.09	1.31	1.53	1.93	2.31	2.67	3.03	3.73
.90	.628	.869	1.11	1.34	1.56	1.98	2.36	2.74	3.11	3.83

*E. F. Brater and H. W. King, *Handbook of Hydraulics*, 6th ed., copyright © McGraw-Hill Book Company, 1976. Used with the permission of the McGraw-Hill Book Company.

†K' values in the table are based on Q, cubic feet per second, and b, in feet. In metric units,

$$K' = 1.486 \frac{Q \text{ (in m}^3\text{/sec)}}{n} \, b \text{ (in m)}^{5/3} \times s^{1/2}$$

TABLE 4.6 Hydraulic Radius of Trapezoidal Channels*

For Determining Hydraulic Radius r for Trapezoidal Channels of Various Side Slopes

Let $\frac{\text{depth of water}}{\text{bottom width of channel}} = \frac{D}{b}$ and C_r = tabulated value. Then $r = C_rD$.

$\frac{D}{b}$	Side slopes of channel, ratio of horizontal to vertical									
	Vertical	¼-1	½-1	¾-1	1-1	1½-1	2-2	2½-1	3-1	4-1
.00	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
.01	.980	.982	.983	.983	.982	.980	.976	.973	.969	.961
.02	.962	.965	.967	.967	.965	.961	.955	.948	.941	.927
.03	.943	.949	.951	.951	.949	.943	.935	.926	.916	.898
.04	.926	.933	.936	.936	.934	.926	.916	.905	.894	.872
.05	.909	.918	.922	.922	.920	.911	.899	.886	.874	.850
.06	.893	.903	.908	.909	.906	.896	.883	.869	.856	.830
.07	.877	.889	.895	.896	.893	.882	.868	.853	.839	.812
.08	.862	.876	.882	.883	.881	.869	.854	.839	.823	.795
.09	.847	.863	.870	.871	.869	.857	.841	.825	.809	.781
.10	.833	.850	.858	.860	.858	.845	.829	.812	.797	.767
.11	.820	.838	.847	.849	.847	.834	.818	.801	.784	.755
.12	.806	.826	.836	.838	.836	.824	.807	.790	.773	.744
.13	.794	.814	.825	.828	.826	.814	.797	.779	.763	.734
.14	.781	.803	.815	.819	.817	.804	.787	.770	.753	.724
.15	.769	.793	.805	.809	.807	.795	.778	.761	.744	.715
.16	.758	.782	.795	.800	.799	.786	.769	.752	.736	.707
.17	.746	.772	.786	.791	.790	.778	.761	.744	.728	.700
.18	.735	.762	.777	.782	.782	.770	.753	.736	.720	.693
.19	.725	.752	.768	.774	.774	.763	.746	.729	.713	.686
.20	.714	.743	.760	.767	.766	.755	.739	.722	.706	.679
.21	.704	.734	.752	.759	.759	.748	.732	.716	.700	.674
.22	.694	.726	.744	.751	.752	.741	.726	.709	.694	.668
.23	.685	.717	.736	.744	.745	.735	.720	.704	.688	.663
.24	.676	.709	.729	.737	.739	.729	.714	.698	.683	.658
.25	.667	.701	.722	.730	.732	.723	.708	.693	.678	.653
.26	.658	.693	.715	.724	.726	.717	.703	.688	.673	.649
.27	.649	.686	.708	.717	.720	.712	.698	.683	.668	.645
.28	.641	.678	.701	.711	.714	.707	.693	.678	.664	.641
.29	.633	.671	.695	.706	.709	.702	.688	.673	.660	.637
.30	.625	.664	.688	.700	.703	.697	.683	.669	.656	.633
.31	.617	.657	.682	.694	.698	.692	.679	.665	.652	.630
.32	.610	.651	.676	.689	.693	.687	.675	.661	.648	.627
.33	.602	.644	.670	.684	.688	.683	.671	.657	.645	.624
.34	.595	.638	.665	.678	.683	.678	.667	.654	.641	.621
.35	.588	.632	.659	.673	.678	.674	.663	.650	.638	.618
.36	.581	.626	.654	.668	.674	.670	.659	.647	.635	.615
.37	.575	.620	.648	.664	.669	.666	.655	.643	.632	.612
.38	.568	.614	.643	.659	.665	.662	.652	.640	.629	.610
.39	.562	.608	.638	.654	.661	.658	.649	.637	.626	.607
.40	.556	.603	.633	.650	.657	.655	.645	.634	.623	.605
.41	.549	.598	.629	.646	.653	.652	.642	.631	.621	.603
.42	.543	.592	.624	.641	.649	.648	.639	.629	.618	.600

pes

4-1
1.000
.961
.927
.898
.872
.850
.830
.812
.795
.781
.767
.755
.744
.734
.724
.715
.707
.700
.693
.686
.679
.674
.668
.663
.658
.653
.649
.645
.641
.637
.633
.630
.627
.624
.621
.618
.615
.612
.610
.607
.605
.603
.600

$\frac{D}{b}$	Side slopes of channel, ratio of horizontal to vertical									
	Vertical	¼-1	½-1	¾-1	1-1	1½-1	2-2	2½-1	3-1	4-1
.43	.538	.587	.619	.637	.645	.645	.636	.626	.616	.598
.44	.532	.582	.615	.633	.641	.642	.633	.623	.613	.596
.45	.526	.577	.611	.629	.638	.639	.631	.621	.611	.594
.46	.521	.572	.606	.626	.635	.636	.628	.618	.609	.592
.47	.515	.568	.602	.622	.631	.633	.625	.616	.607	.591
.48	.510	.563	.598	.618	.628	.630	.623	.614	.605	.589
.49	.505	.558	.594	.615	.625	.627	.620	.611	.603	.587
.50	.500	.554	.590	.611	.621	.624	.618	.609	.601	.586
.51	.495	.550	.587	.608	.618	.622	.616	.607	.599	.584
.52	.490	.545	.583	.604	.615	.619	.613	.605	.597	.583
.53	.485	.541	.579	.601	.612	.617	.611	.603	.595	.581
.54	.481	.537	.576	.598	.610	.614	.609	.601	.594	.580
.55	.476	.533	.572	.595	.607	.612	.607	.600	.592	.578
.56	.472	.529	.568	.592	.604	.610	.605	.598	.590	.577
.57	.467	.525	.505	.589	.601	.607	.603	.596	.589	.576
.58	.463	.521	.562	.586	.598	.605	.601	.594	.587	.574
.59	.459	.518	.558	.583	.595	.603	.599	.593	.586	.573
.60	.455	.514	.555	.580	.593	.601	.597	.591	.584	.572
.61	.450	.510	.552	.577	.591	.599	.596	.589	.583	.571
.62	.446	.507	.549	.575	.588	.597	.594	.588	.581	.569
.63	.442	.504	.546	.572	.586	.595	.592	.586	.580	.568
.64	.439	.500	.543	.569	.584	.593	.590	.585	.579	.567
.65	.435	.497	.540	.567	.581	.591	.589	.583	.577	.566
.66	.431	.494	.537	.564	.579	.589	.587	.582	.576	.565
.67	.427	.490	.534	.562	.577	.587	.586	.580	.575	.564
.68	.424	.487	.432	.559	.575	.585	.584	.579	.574	.563
.69	.420	.484	.529	.557	.573	.583	.583	.578	.573	.562
.70	.417	.481	.526	.555	.571	.582	.581	.577	.571	.561
.71	.413	.478	.524	.552	.569	.580	.580	.575	.570	.560
.72	.410	.475	.521	.550	.567	.578	.578	.574	.569	.559
.73	.407	.472	.518	.548	.565	.577	.577	.573	.568	.558
.74	.403	.469	.516	.546	.563	.575	.576	.572	.567	.558
.75	.400	.467	.514	.544	.561	.573	.574	.570	.566	.557
.76	.397	.464	.511	.542	.559	.572	.573	.569	.565	.556
.77	.394	.461	.509	.539	.557	.570	.572	.568	.564	.555
.78	.391	.458	.507	.537	.555	.569	.570	.567	.563	.554
.79	.388	.456	.504	.535	.554	.567	.569	.566	.562	.554
.80	.385	.453	.502	.533	.552	.566	.568	.565	.561	.553
.81	.382	.450	.500	.531	.550	.565	.567	.564	.560	.552
.82	.379	.448	.498	.530	.548	.564	.566	.563	.559	.551
.83	.376	.445	.495	.528	.547	.562	.565	.562	.558	.551

*E. F. Brater and H. W. King, *Handbook of Hydraulics*, 6th ed., copyright © McGraw-Hill Book Company, 1976. Used with the permission of the McGraw-Hill Book Company.

Revised K' and depth of flow D:

$$K' = \frac{Q \times n}{b^{8/3} \times s^{1/2}}$$

$$= \frac{(7.5\ \text{ft}^3/\text{sec})\ (0.02)}{(1\ \text{ft})(0.1414)} \qquad \left\{\frac{1.486(0.22\ \text{m}^3/\text{sec})(0.02)}{[(0.305\ \text{m})^{8/3}](0.1414)}\right\}$$

$$= 1.06 \qquad (1.10)^*$$

From Table 4.5, D/b = 0.62 (0.63)*; thus, D = 0.62 ft (0.19 m).

It is apparent that the revised depth of flow is not significantly different from the first approximation. The resulting velocity also will not be significantly different. This example illustrates that channel flow times are much shorter than overland flow times for comparable distances. Thus, *unless* the channel length is very large, channel flow time may be ignored in calculating T_c. However, if channel length is significant, or channel slope is very small, then channel flow time must be considered.

STEP 10. Adequacy of channel. It will be very easy to construct this channel, since a 1-ft (0.305-m) depth will allow a safe freeboard of 0.38 ft (0.12 m), or 4.6 in (12 cm) above the peak flow.

However, a velocity of 5.47 ft/sec (1.7m/sec) will be erosive. Maximum permissible velocity on ordinary firm loam, carrying silt-laden runoff, is 3.5 ft^3/sec (1.1 m/sec), whereas on stiff clay the maximum allowable velocity is 5.0 ft/sec (1.5 m/sec) (see Table 7.1 for maximum permissible velocities in unlined channels). In this case, a larger channel or a channel lining of coarse gravel is needed to prevent channel bed erosion.† If a coarse gravel lining is used, the channel roughness factor n in Manning's equation changes from 0.02 to 0.03 and the calculated velocity drops to 3.6 ft/sec (1.1 m/sec). The channel size must again be checked for adequacy with the new velocity.

Modified Procedure for Solving Channel Flow Problem without the Handbook of Hydraulics

If the *Handbook of Hydraulics* is not available, the following procedure based on simultaneous equations can be used to calculate velocity. The step-by-step procedure used in Example 4.8 is modified: At step 4, instead of determining Q_{peak} on the basis of overland travel time only, assume full flow in the channel and estimate a channel travel time before computing Q_{peak}. Steps 1, 2, and 3 are the same as given earlier.

STEP 4. As a first approximation, assume channel flowing full. Find the velocity by using Manning's equation. Compute channel travel time by dividing length by velocity.

STEP 5. Sum overland travel time and channel travel time to obtain total time of concentration.

STEP 6. Determine the precipitation intensity.

STEP 7. Compute the peak discharge.

---

*Because of round-off error, the values of K' and D/b are slightly different when metric units are used.

†Figure 7.36, which shows maximum permissible flow depths for unlined channels of various slopes, indicates that a lining is needed for the channel in this example, regardless of the soil type. Therefore, a lining should be installed (Sec. 7.2e).

STEP 8. Determine the second approximation of velocity in the channel to evaluate erosivity.

EXAMPLE 4.9 Channel Flow Calculations without the *Handbook of Hydraulics*

Given: This example uses the same site as Example 4.8 uses. However, let's now change the drainage channel slope to 0.1 percent and see what happens.

Solution:

STEP 4. Channel travel time. Select a reasonable initial channel size, 1 ft (0.305 m) deep. Assuming the channel is flowing full, use Manning's equation to find the velocity of flow in the channel: $n = 0.02$.

Area:

$$A = bD + 2D^2$$
$$= 1(1) + 2(1) \qquad [0.305(0.305) + 2(0.305^2)]$$
$$= 3\ \text{ft}^2 \qquad (0.28\ \text{m}^2)$$

Wetted perimeter:

$$\text{WP} = b + 2D\sqrt{2^2 + 1}$$
$$= 1 + 2(1)(\sqrt{5}) \qquad [0.305 + 2(0.305)(\sqrt{5})]$$
$$= 5.5\ \text{ft} \qquad (1.67\ \text{m})$$

$$r = \frac{A}{\text{WP}} = \frac{3}{5.5} = 0.55\ \text{ft} \qquad \left(\frac{0.28}{1.67} = 0.17\ \text{m}\right)$$

$$r^{2/3} = 0.67 \qquad (0.307)$$

$$s = 0.001$$

$$s^{1/2} = 0.032$$

$$V = \frac{1.49 \times r^{2/3} \times s^{1/2}}{n}$$
$$= \frac{1.49\,(0.67)(0.032)}{0.02} \qquad \left[\frac{0.307(0.032)}{0.02}\right]$$
$$= 1.6\ \text{ft/sec} \qquad (0.49\ \text{m/sec})$$

Channel travel time:

$$\frac{L}{60V} = \frac{1000\ \text{ft}}{(60\ \text{sec/min})(1.6\ \text{ft/sec})}$$
$$= 10.4\ \text{min} \qquad \left[\frac{305\ \text{m}}{(60\ \text{sec/min}) \times (0.49\ \text{m/sec})} = 10.4\ \text{min}\right]$$

STEP 5. Time of concentration:

$$T_c = \text{overland flow time plus channel travel time}$$
$$= 37 + 10.4$$
$$= 47.4\ \text{min}$$

STEP 6. Precipitation intensity. From the 10-year, 30-min and 10-year, 1-hr rainfalls given in the *Rainfall Frequency Atlas*, we interpolate to find the 10-year, 47.4-min rainfall:

10-year, 30-min = 1.4 in (36 mm)

10-year, 1-hr = 1.6 in (41 mm)

10-year, 47.4 min = 1.52 in (39 mm)

$$i = \frac{1.52 \text{ in}}{47.4 \text{ min}} \times \frac{60 \text{ min}}{\text{hr}} = 1.92 \text{ in/hr} \qquad \left(\frac{39 \text{ mm}}{47.4 \text{ min}} \times \frac{60 \text{ min}}{\text{hr}} = 49 \text{ mm/hr}\right)$$

STEP 7. Peak discharge:

$$\begin{aligned} Q_{\text{peak}} &= C \times i \times A \\ &= 0.17(1.92)(20) \qquad \left[\frac{0.17(49 \text{ mm/hr})(8.1 \text{ ha})}{360}\right] \\ &= 6.5 \text{ ft}^3\text{/sec} \qquad (0.19 \text{ m}^3\text{/sec}) \end{aligned}$$

The peak discharge from this drainage area is estimated to be 6.5 ft^3/sec (0.19 m^3/sec). By use of Manning's equation in step 4, the discharge through the full channel cross section was initially calculated to have a velocity of 1.6 ft/sec (0.49 m/sec). This maximum velocity provides a conservative estimate of peak discharge.

In actuality, a flow of 6.5 ft^3/sec (0.19 m^3/sec) through a channel of 3-ft^2 (0.028-m^2) cross-sectional area should have a velocity of 2.2 ft/sec (0.68 m/sec) [since $V = Q/A$ = 6.5 ft^3/sec ÷ 3 ft^2 = 2.2 ft/sec (0.19 m^3/sec ÷ 0.28 m^2 = 0.68 m/sec)]. This mismatch of velocities tells us that the channel would not flow full and the velocity would be somewhere between 1.6 and 2.2 ft/sec (0.49 and 0.68 m/sec).

STEP 8. In step 4 we assumed the channel was flowing full. The actual velocity in the channel can be estimated more precisely by reapplying Manning's equation. We need to determine the actual cross-sectional area of the channel when it is discharging 6.5 ft^3/sec (0.19 m^3/sec). Combining the equations gives us

$$V = \frac{1.49 \times r^{2/3} \times s^{1/2}}{n} \qquad Q = A \times V$$

results in

$$Q = A \times V = \frac{A \times 1.49 \times r^{2/3} \times s^{1/2}}{n} \qquad \left[Q \text{ (in m}^3\text{/sec)} = \frac{A \times r^{2/3} \times s^{1/2}}{n}\right]$$

From the preceding computations:

$$\begin{aligned} Q &= 6.5 \qquad (0.19 \text{ m}^3\text{/sec}) \\ \frac{1.49}{n} &= 74.5 \qquad \left(\frac{1}{0.02} = 50\right) \\ s^{1/2} &= 0.032 \end{aligned}$$

The equation now becomes

$$\begin{aligned} 6.5 &= A \times 74.5 \times r^{2/3} \times 0.032 \qquad (0.19 = A \times 50 \times r^{2/3} \times 0.032) \\ 6.5 &= A \times 2.38 \times r^{2/3} \qquad (0.09 = A \times 1.6 \times r^{2/3}) \\ 2.73 &= A \times r^{2/3} \qquad (0.12 = A \times r^{2/3}) \end{aligned}$$

Since $r = \dfrac{A}{\text{WP}}$ (from step 4)

$$2.73 = A \times \left(\frac{A}{\text{WP}}\right)^{2/3} \qquad \left\{0.12 = A \times \left(\frac{A}{\text{WP}}\right)^{2/3}\right\}$$

$$2.73 = \frac{A^{5/3}}{\text{WP}^{2/3}} \qquad \left(0.12 = \frac{A^{5/3}}{\text{WP}^{2/3}}\right)$$

Since both the cross-sectional area and the wetted perimeter can be expressed in terms of the channel depth, the equation can be solved algebraically. Alternatively, the equation can be solved by iteration: substitute a reasonable number and adjust it according to the results.

For a trapezoidal channel with 2:1 side slopes, the area can be expressed in terms of the depth D as

$$A = bD + 2D^2 = 1 \times D + 2D^2 \qquad (0.305 \text{ m} \times D + 2D^2)$$

The wetted perimeter can also be expressed in terms of depth as

$$\text{WP} = b + 2D\sqrt{z^2 + 1} = 1 + 2D\sqrt{5} \qquad (0.305 \text{ m} + 2D\sqrt{5})$$

Choosing iteration as the solution method, we first try a depth of 0.8 ft (0.24 m).

$$A = 0.8 + 2(0.8^2) = 2.08 \qquad \{(0.305 \text{ m})(0.24 \text{ m}) + 2[(0.24 \text{ m})^2] = 0.188 \text{ m}^2\}$$

$$\text{WP} = 1 + 2(0.8)(\sqrt{5}) = 4.58 \qquad [0.305 \text{ m} + 2(0.24 \text{ m})(\sqrt{5}) = 1.38 \text{ m}]$$

and substituting into $2.73 \times \text{WP}^{2/3} = A^{5/3}$ $[0.12 \times \text{WP}^{2/3} \text{ (in m)} = A^{5/3} \text{ (in m)}]$ gives us

$$2.73(4.58^{2/3}) = 2.08^{5/3} \qquad \{0.12\,[(1.38 \text{ m})^{2/3}] = (0.188 \text{ m}^2)^{5/3}\}$$

$$7.53 > 3.39 \qquad (0.149 > 0.062)$$

Next, try a depth of 1.5 ft (0.46 m):

$$A = 1.5 + 2(1.5^2) = 6 \qquad \{0.46 \text{ m} + 2(0.46 \text{ m})^2 = 0.88 \text{ m}^2\}$$

$$\text{WP} = 1 + 2(1.5)(\sqrt{5}) = 7.7 \qquad [0.305 \text{ m} + 2(0.46 \text{ m})(\sqrt{5}) = 2.36 \text{ m}]$$

$$2.73(7.7^{2/3}) = 6^{5/3} \qquad \{0.12[(2.36 \text{ m})^{2/3}] = (0.88 \text{ m}^2)^{5/3}\}$$

$$10.6 < 19.8 \qquad (0.213 < 0.808)$$

Since the left side is now smaller rather than larger than the right side, we try several depths between 0.8 and 1.5 ft (0.24 and 0.46 m).

The actual channel depth when discharging 6.5 ft^3/sec (0.19 m^3/sec) would be 1.15 ft (0.35 m). The cross-sectional area with that depth is 3.79 ft^2 (0.35 m^2). The more precise channel velocity then is

$$Q = A \times V$$

$$6.5 = 3.79 \times V \qquad (0.19 \text{ m}^3/\text{sec} = 0.35 \text{ m}^2 \times V)$$

$$V = 1.7 \text{ ft/sec} \qquad (0.54 \text{ m/sec})$$

According to Table 7.1, unless the channel material were fine sand, this velocity would not be erosive to an unlined channel. However, Fig. 7.36, which was based on different experimental data and parameters, shows that a lining is probably needed in the channel. In deciding whether to install a lining, the prudent approach is to check both tables. If either table shows that a lining is needed, then a lining should be installed. Greater weight should be placed on the maximum depth of flow table (Fig. 7.36), since the maximum permissible velocity table (Table 7.1) assumes "aged" channels, and newly constructed channels on a construction site are unlikely to be in that relatively stable condition.

However, the 1-ft (0.305-m) channel depth assumed in Example 4.8 would not contain the peak flow with the flatter conditions of Example 4.9. Allowing for freeboard, the channel in this last example should be constructed with a depth of at least 1.5 ft (0.46 m).

4.2 OTHER METHODS FOR ESTIMATING RUNOFF

This handbook focuses on the use of the rational method to compute peak discharge from a small watershed. Brief descriptions of three other methods are presented below, and they are followed by a comparison of the methods. Any reliable method to compute runoff can be used for drainage system design.

4.2a Unit Hydrograph Method

The following description was taken from Rantz (10):

> The unit hydrograph is a widely used device for relating runoff to storm precipitation and is described in all standard hydrology texts. The unit hydrograph shows the time distribution of surface runoff resulting from a storm that produces 1 inch (25.4 mm) of rainfall excess over the watershed in some selected interval of time. Rainfall excess is defined as that part of the rainfall that is available to produce surface runoff, after the demands of infiltration and surface retention have been met. That part of the precipitation that infiltrates into the ground or is retained above ground is known as water loss.
>
> The time interval used for the 1 inch (25.4 mm) of rainfall excess varies with basin size and with the time response of runoff to rainfall. It may be as short as 1 minute for a small experimental plot or as long as 24 hours for a large slow-rising river, but in practice it generally ranges between 5 minutes and 6 hours. Given the unit hydrograph for a watershed and the precipitation distribution for a given storm, the hydrologist can produce the resulting hydrograph of surface runoff.
>
> The unit hydrograph for a gaged watershed can be derived from observed hydrographs and the record from a recording raingage in the basin. From the characteristics of the unit hydrographs for several gaged watersheds in a region, it is then possible to derive synthetic unit hydrographs for use with ungaged watersheds in the region.

Rantz continues with a discussion of design elements for synthetic unit hydrographs and develops some adjustments to the unit hydrograph for the effects of urbanization.

4.2b Hydrologic Basin Models

The use of hydrologic basin models for simulating runoff has been made possible by the widespread availability and sophistication of computers. This book does not address runoff models in detail. The reader may refer to references cited or to local engineering firms who use their own models. Models are more appropriate to large drainage areas and are associated more often with flood control projects than with individual construction sites.

Many hydrologic basin models for use in runoff simulation are under investigation, but the only ones in use at the time of the Rantz study (10) for computing storm runoff from small watersheds were the USGS Watershed Model (4) and various versions of the Stanford Watershed Model (3). Both the USGS and Stanford models:

- Use precipitation and pan evaporation as hydrometeorological inputs
- Maintain a water budget that is balanced at short intervals (usually every 15 min during storm periods)
- Require only a short period of runoff record for model calibration

Neither model requires the assumption of identical frequencies for peak discharge and peak precipitation rates, because both use historic sequences of storm precipitation over a period of years long enough to permit statistical frequency analysis of the derived discharge data.

4.2c SCS Method for Small Watersheds

The U.S. Soil Conservation Service (SCS) developed a set of charts for estimating the peak discharge from small areas. (7) The graphs were prepared by computer-processing national rainfall data and inputting the results into the runoff equation developed by Mockus in the SCS *National Engineering Handbook*. (11)

Peak discharges range from 5 to 2000 ft^3/sec (0.1 to 57 m^3/sec); drainage areas range from 5 to 2000 acres (2 to 809 ha); and 24-hr rainfalls range from 1 to 12 in (25 to 305 mm).

4.2d Comparison of Methods

The rational method is simple to apply. It provides a runoff rate for a small watershed no larger than 200 acres (81 ha) and it is applied separately to each drainage area on a site. Its principal disadvantage is that it provides only a peak or average discharge from a watershed, not a complete hydrograph, and therefore it cannot be used for routing multiple flows toward a single outlet. But erosion and sediment control structures are more effective when located throughout a drainage area rather than at a single collection point. So the rational method remains the simplest way to obtain estimates of peak discharge and average runoff rates for the design of erosion and sediment control structures.

The unit hydrograph method, although more complex in application than the rational method, has the advantage of providing a complete storm hydrograph rather than just the peak discharge. Thus, in dealing with a complex watershed, the storm hydrograph for each subwatershed can be computed independently by the unit hydrograph method for subsequent routing down the main channel. No such direct procedure for combining flood peaks from subwatersheds is possible with the rational method. The complete storm hydrograph is needed for flood control project design and for some water quality studies.

The chief weakness of the unit hydrograph method, according to Rantz, is that infiltration loss is difficult to determine. Rantz tentatively related infiltration to mean annual precipitation. He concluded that the unit hydrograph method would give better results than the rational method, especially on larger watersheds.

In erosion control, we are interested more in individual sites of limited area within a watershed than in the entire watershed. Although perhaps less accurate than the unit hydrograph method, the rational method does account for infiltration loss through the coefficient C. A complete storm hydrograph is not needed unless flood control or a water quality monitoring program is planned. Thus, in erosion control work, the rational method is usually sufficient.

Runoff simulation models are most useful for large-scale design projects. For erosion control planning, the site planner may apply an existing model to a construction site, but development of a new model just to obtain the peak discharge is impractical.

The SCS model for small watersheds is used by SCS offices nationwide. However, the factors used in this model (called curve numbers) do not provide the flexibility that the rational method C values do in evaluating site characteristics, since the SCS model was designed for rural areas. In addition, the SCS model cannot use precipitation depth-duration-frequency data, which, if available, improves the accuracy of the other methods.

In summary, the rational method provides sufficiently accurate, easy-to-obtain estimates of runoff from a construction site for use in erosion control planning. Other, more complex methods exist and may be used by those familiar with them.

REVIEW QUESTIONS

1. What is the rational method equation?
2. What are some of the advantages and disadvantages of the rational method equation?
3. What other methods for computing runoff are available?
4. Describe the three factors in the rational method equation.
5. How is design storm return interval chosen?
6. What is Q_{peak} used for? Q_{avg}?

7. What effect does slope steepness have on the C value? What effect does soil depth have?
8. What is the most reliable source of rainfall intensity data?
9. What local options for rainfall and intensity information may be available?
10. Describe how to determine the average and peak precipitation intensity. What information about the site do you need? What reference materials?
11. What is meant by the terms "time of concentration" and "overland flow"? For what computation are they necessary?
12. What are the two components of the time of concentration?
13. How is the overload flow time estimated? What site factors influence overland flow time?
14. How is channel flow time computed?
15. Why is it important to know the velocity in the channel? What are the alternatives if the calculated flow or velocity exceeds the capacity of a channel?
16. ***Given:*** A 4-acre (1.6-ha) site just north of New York City.
    Longest overland flow path length is 200 ft (61 m).
    Average slope is 10 percent.
    Bare, smooth, packed earth.

    Find: C value
    Overland flow time T_c
    Peak intensity i_{peak} for 10-year storm
    Q_{peak}, 10-year storm
17. For the site in Question 16, find the average intensity and average runoff for a 10-year, 6-hr storm.

REFERENCES

1. E. F. Brater and H. W. King, *Handbook of Hydraulics*, 6th ed., McGraw-Hill Book Company, New York, 1976.
2. California Department of Water Resources, *Rainfall Depth-Duration-Frequency for California*, microfiche, DWR, Sacramento, Calif., 1981.
3. N. H. Crawford and R. K. Linsley, *Digital Simulation in Hydrology, Stanford Watershed Model IV*, Stanford University, Department of Civil Engineering Technical Report No. 39, Palo Alto, Calif., 1966.
4. D. R. Dawdy, R. W. Lichty, and J. M. Bergmann, *A Rainfall-Runoff Simulation Model for Estimation of Flood Peaks for Small Drainage Basins—A Progress Report*, U.S. Geological Survey Open-File Report, Menlo Park, Calif., 1970.
5. R. H. Frederick, V. A. Myers, and E. P. Auciello, *Five- to Sixty-Minute Precipitation Frequency for the Eastern and Central United States*, NOAA Technical Memorandum NWS Hydro-35 (prepared for Engineering Division, U.S. Department of Agriculture, Soil Conservation Service), U.S. Department of Commerce, National Oceanic and Atmospheric Administration, National Weather Service, Office of Hydrology, Silver Springs, Md., 1977.

6. D. M. Hershfield, *Rainfall Frequency Atlas of the United States*, Technical Paper No. 40, U.S. Department of Commerce, Weather Bureau, Washington, D.C., 1961.

7. K. M. Kent, *A Method for Estimating Volume and Rate of Runoff in Small Watersheds*, SCS Technical Paper No. 149, U.S. Department of Agriculture, Soil Conservation Service, GPO, Washington, D.C., 1973.

8. J. F. Miller, R. H. Frederick, and R. J. Tracey, *Precipitation Frequency Atlas of the Western States* (11 volumes), NOAA Atlas 2 (prepared for Engineering Division, U.S. Department of Agriculture, Soil Conservation Service), U.S. Department of Commerce, National Oceanic and Atmospheric Administration, National Weather Service, Office of Hydrology, GPO, 1973.

9. S. E. Rantz, *Precipitation Depth-Duration-Frequency Relations for the San Francisco Bay Region*, U.S. Geological Survey, Basic Data Contribution No. 25, U.S.G.S., Menlo Park, Calif., 1971.

10. S. E. Rantz, *Suggested Criteria for Hydrologic Design of Storm Drainage Facilities in the San Francisco Bay Region, California*, U.S. Geological Survey, Menlo Park, Calif., 1971.

11. U.S. Department of Agriculture, Soil Conservation Service, *National Engineering Handbook, Section 4, Hydrology*, GPO, Washington, D.C., 1972.

12. U.S. Department of the Interior, Bureau of Reclamation, *Design of Small Dams*, GPO, Washington, D.C., 1973.

13. Virginia Soil and Water Conservation Commission, *Virginia Erosion and Sediment Control Handbook*, VSWCC, Richmond, Va., 1980.

14. Wright-McLaughlin Engineers, *Urban Storm Drainage Criteria Manual*, Denver Regional Council of Governments, Denver, Colo. 1969.