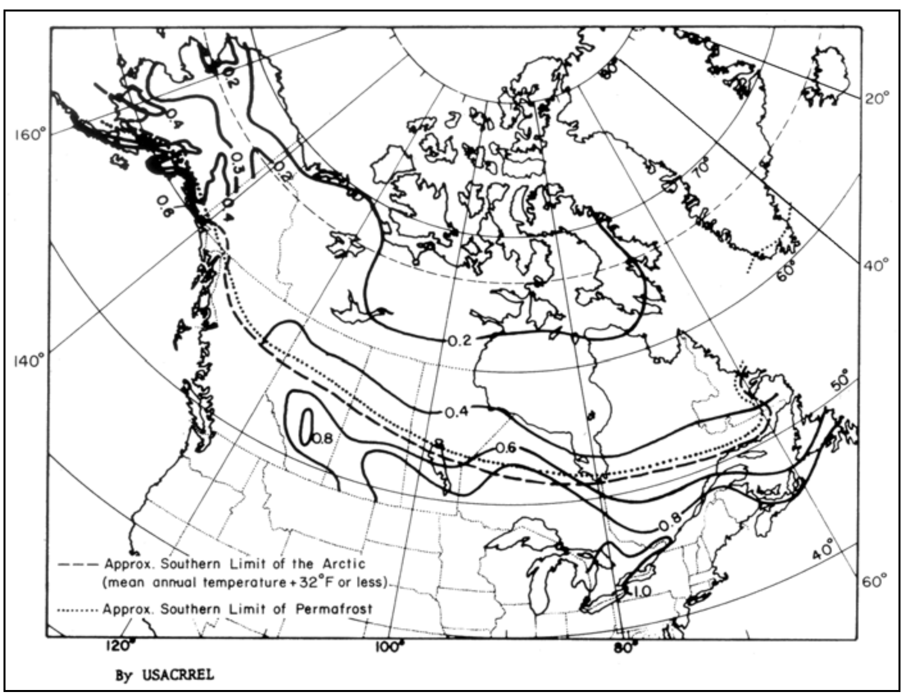
CHAPTER 2

SURFACE HYDROLOGY

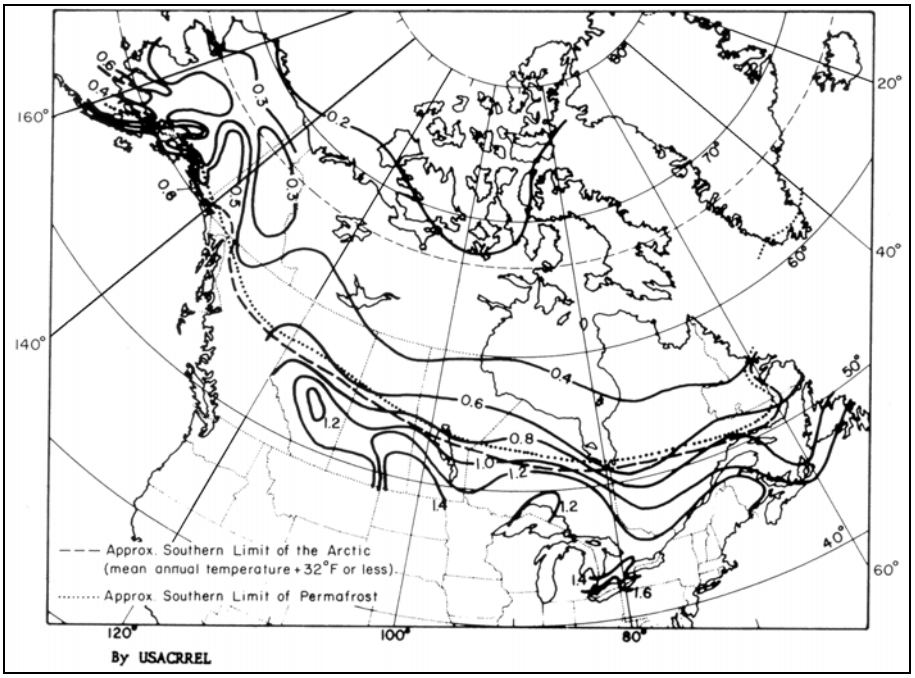
2-1 **PURPOSE AND SCOPE.** This chapter presents discussions and examples to give a better understanding of problems in the design of drainage facilities, and outlines convenient methods of estimating design capacities for airfield and heliport drainage facilities in arctic and subarctic regions. Although the design data herein have been developed primarily for drainage conditions in North America, the data are also generally applicable to other arctic and subarctic regions. For roads and built-over areas, different methods and design rates of rainfall are used in computing required runoff amounts and in determining the size of storm drains, culverts and other drainage facilities. However, the general information in this chapter on icings and special design considerations for arctic and subarctic conditions are applicable. Criteria in Sections 4-4.10 through 4-4.14, together with design storm indexes as determined from Figure 2-1, will be used for design of drainage facilities for other than airfields and heliports.

Figure 2-1. Design Storm Index for Alaska and Canada: Isolines of maximum 1-hour rainfall (inches) occurring once in 2, 5, 10 and 25 years. Lines correspond to the intensity-duration curves in Figure 2-3. Data from US National Weather Service, the Canadian Department of Transportation, Meteorological Branch, and Quartermaster Research and Development Center

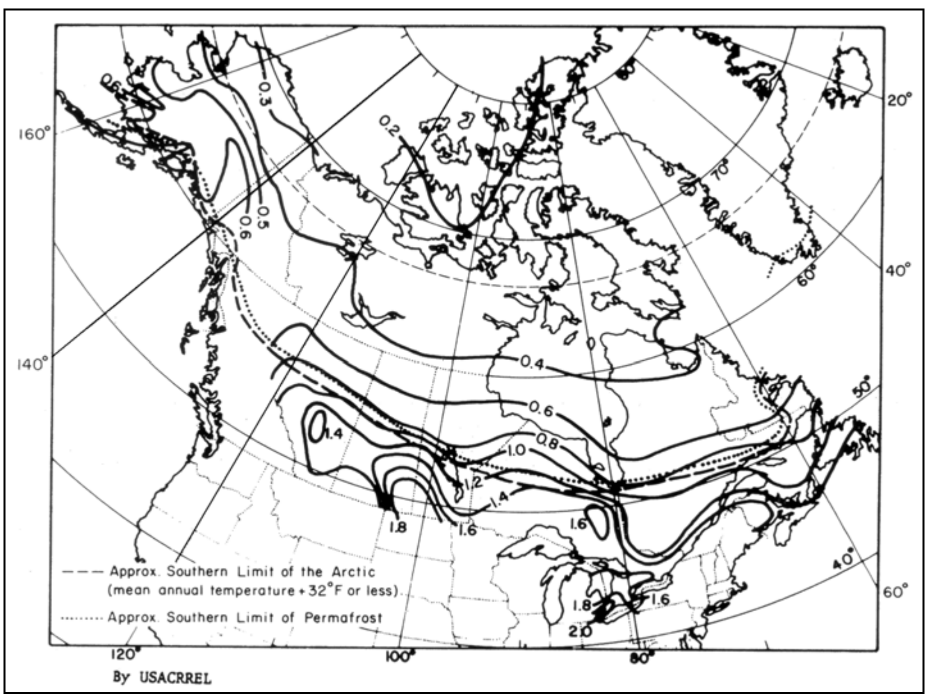
a. Once in 2 yr



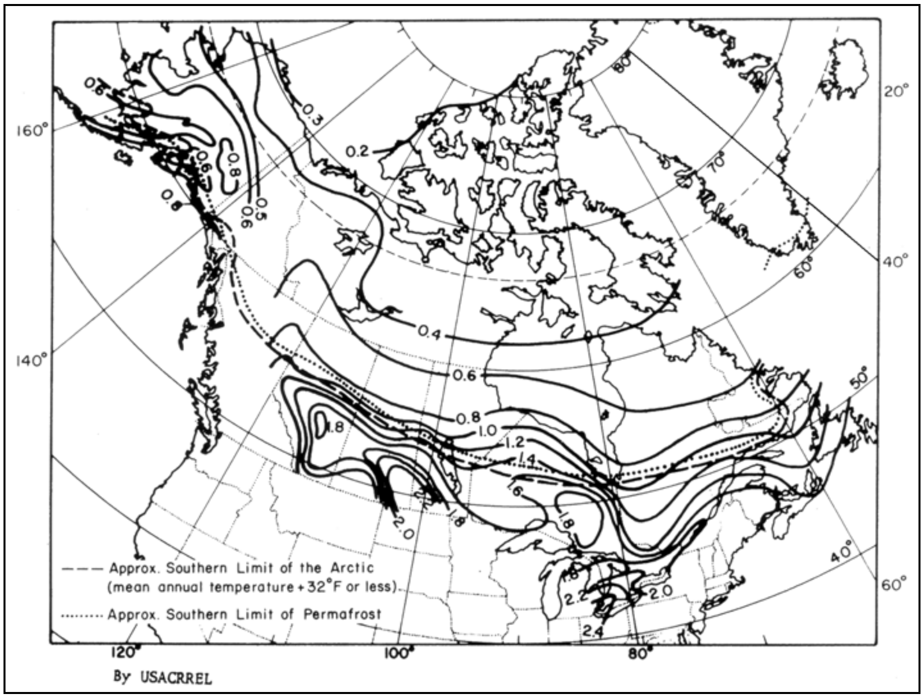
b. Once in 5 yr



c. Once in 10 yr



d. Once in 25 yr



2-2 **HYDROLOGIC CRITERIA.** The Rational Method, developed over 100 yr ago, is widely used for estimating design runoff from urban areas. The Rational Formula, popular because of its simplicity in application, is described in Chapter 4. It is suited mainly to sizing culverts, storm drains or channels to accommodate drainage from small areas, general less than 50 acres. Selection of appropriate values of runoff coefficients in the formula depends on the experience of the designers and the designers' knowledge of local rainfall-runoff relationships. Use of the Rational Method in the design of military airfield drainage systems, with their large, generally level contributory drainage areas, is not recommended. The development of hydrologic criteria in this manual closely follows the procedure outlined in Chapter 3. "Investigation of Airfield Drainage, Arctic and Subarctic Regions, Part I, Field Reconnaissance Report," by L. G. Straub and L. A. Johnson, is one of several confirming that this procedure accurately determines required hydraulic capacity of airfield drainage facilities with lessened dependence on arbitrary assumptions of design factors. Although judgment is important in any engineering design, guesswork is minimized in use of this procedure which is based on theoretical concepts which have been verified in carefully controlled natural and simulated rainfall and runoff tests under widely varying hydrologic and topographic conditions. In the design of drainage facilities for the Arctic and Subarctic, additional capacity must in many cases be provided to compensate for that lost due to icings. This is discussed in Section 2-8.

2-2.1 **Definitions.** The following specialized terms are used in this chapter.

2-2.1.1 **Arctic.** The northern region in which the mean temperature for the warmest month is less than 50 degrees F and the mean annual temperature is below 32 degrees F. In general, the Arctic coincides with the tundra region north of the limit of trees.

2-2.1.2 **Subarctic.** The region adjacent to the Arctic in which the mean temperature for the coldest month is below 32 degrees F, the mean temperature for the warmest month is above 50 degrees F, and in which there are less than 4 months having a mean temperature above 50 degrees F. In general, the subarctic land areas coincide with the circumpolar belt of dominant coniferous forests.

2-2.2 **Design Objectives.** The design capacity of the airfield or heliport surface drainage system should be adequate to accomplish the following objectives as satisfactorily as is economically feasible and with due consideration of the mission and importance of the particular airfield or heliport, effects of icings, and environmental impact.

2-2.2.1 **Surface runoff from design storm.** Surface runoff from the selected design storm will be disposed of without damage to facilities, undue saturation of the subsoil, or significant interruption of normal traffic.

2-2.2.2 **Surface runoff from storms exceeding design storm.** Surface runoff from storms greater than the design storm will be disposed of with the minimum damage to the airfield for heliport. The center 50 percent of runways; the center 50 percent of taxiways serving these runways; and helipad surfaces shall be free from ponding resulting from storms of one hour duration, 25-yr frequency and intensity determined by the graphic location.

2-2.2.3 **Reliability of operation.** The drainage system will have the maximum reliability of operation practicable under all conditions, with due consideration given to abnormal requirements during annual periods of snowmelt and ice jam breakup.

2-2.2.4 **Maintenance.** The drainage system will require minimum maintenance which will be accomplished quickly and economically. Particular reliance will be placed on maintenance of drainage components serving operational facilities.

2-2.2.5 **Future expansion.** Future expansion of drainage facilities will be feasible with the minimum of expense and interruption to normal traffic.

2-2.3 **Degree of Drainage Required.** The degree of protection to be provided by the drainage system depends largely on the importance of the facility as determined by the type and volume of traffic to be accommodated, the necessity for uninterrupted service, and similar factors. Although the degree of protection should increase with the importance of the airfield or heliport, minimum requirements must be adequate to avoid hazards to operation. One severe accident chargeable to inadequate drainage can offset any difference between the cost of reasonably adequate and inadequate drainage facilities. Drainage for military airfields or heliports will be based on a 2-yr design storm frequency, unless exceptional circumstances require greater protection. For design purposes, a minimum supply rate of 0.2 in./hr of rainfall plus snowmelt is to be used, even where intensity frequency studies for the Arctic indicate somewhat lower values. In mountainous areas subject to orographic precipitation, maps showing local variations of the design storm index will prove useful for drainage designs provided that adequate long-term precipitation records are available to warrant such refinements. In some cases one can justify use of design storm frequencies appreciably higher than the 2-yr rate to protect important facilities. In some U.S. designs, portions of the drainage system have been based on as high as a 50-yr design frequency to reduce likelihood of flooding a facility essential to operations and to prevent loss of life. Many designers find that using the 2-yr design with this Corps of Engineers method will usually yield results comparable with use of a 10-yr design based on the Rational Method.

2-3 **RAINFALL.** A study of rainfall intensity-frequency data recorded at arctic and subarctic stations indicates significant variance between the average intensity of rainfall for a period of 1 hr and the average precipitation rates of comparable frequency for shorter intervals. This is also evident when compared with similar rainfall data in the continental United States. Even within the area of Alaska, there is noticeable difference between the orographic rains of Juneau and the convergent and convective precipitation at Fairbanks. The higher values for rainfall intensity were used to develop design intensity-duration (supply) curves. Similar curves for the continental United States are shown in Figure 2-2.

Figure 2.2 Design Storm Index, 1-hour Rainfall Intensity-Frequency Data for Continental United States Excluding Alaska

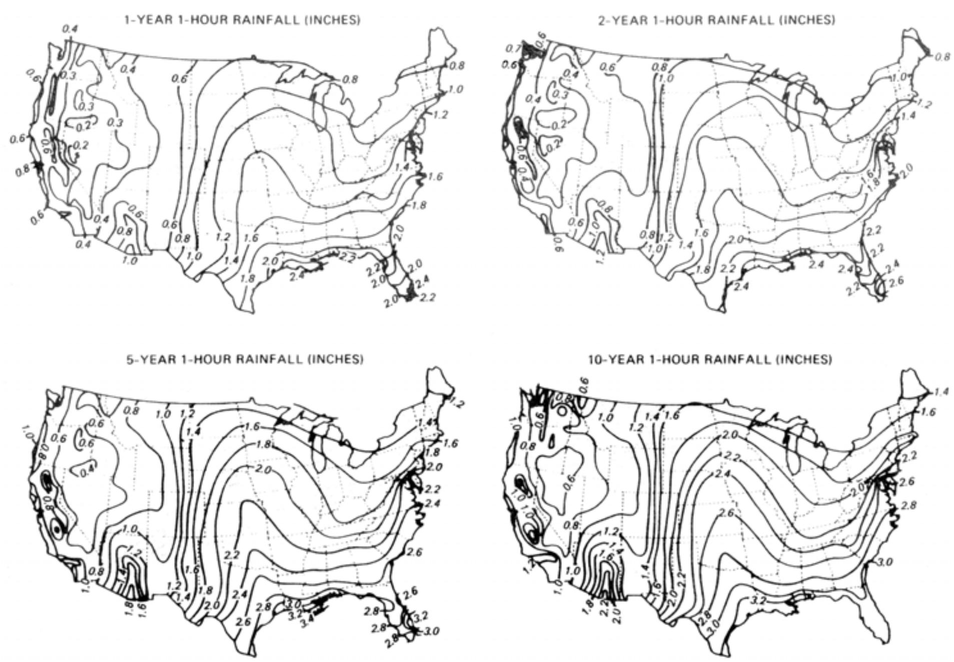


Chart reproduced from US Weather Bureau, Technical Paper No. 40, Rainfall Frequency Atlas of the United States, Washington, DC, May 1961.

2-3.1 **Design Storm Frequency.** Design storm frequencies are normally stated in engineering instructions for the specific project. For airfields and heliports, the 2-yr design storm frequency is most often used. It should be noted that after this design storm frequency is specified, computations must be made to determine the critical duration of rainfall required to produce the maximum rate of runoff for each area. This will depend primarily on the slope and length of overland flow.

2-3.2 **Storms of Greater Severity Than Design Storm.** The design storm frequency alone is not a reliable criterion of the adequacy of storm drain facilities. Under some circumstances, storms much more severe than the design storm may cause very little damage or inconvenience, whereas under other circumstances flooding of important areas may result. It is advisable to investigate the probable consequences of storms more severe and less frequent than the design storm before making final decisions regarding the adequacy of proposed drain-inlet capacities. Additional requirements necessitated by the effects of icings on drainage facilities in arctic and subarctic regions are discussed in Section 2-8.

2-3.3 **Design Storm Index.** One-hour rainfall intensities having various average frequencies of occurrence in the artic and subarctic regions of Alaska and Canada are shown in Figure 2-1. This figure, on which rainfall depth curves are superimposed, is known as a design storm index and is based on reports by the U.S. National Weather Service and the Canadian Department of Transport, Meteorological Branch. The curves are labeled according to the 1-hour amounts of rainfall and are coordinated with the supply curves of Figure 2-3. Figures 2-1 and 2-3 used in combination provide a sufficiently accurate means for determining rainfall intensities for runoff computations for any duration and geographic location. Where data are incomplete for a specific foreign area under study, a generalized method for estimating the 2-yr 1-hr value has been developed using usually available climatic data. This method uses a diagram (Figure 2-4) which relates the 2-yr 1-hr rainfall to the following more commonly known climatic data: mean annual precipitation, mean annual number of days of precipitation, mean annual thunderstorm days, and mean of the annual maximum observational-day rainfall amounts. The diagram gives maximum 60-min, not clock-hour, rainfall for the 2-yr frequency.

Figure 2-3. Supply Curves for Arctic and Subarctic Regions

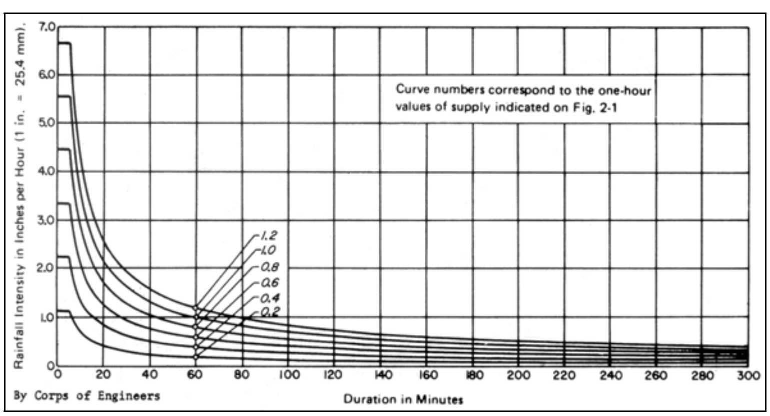
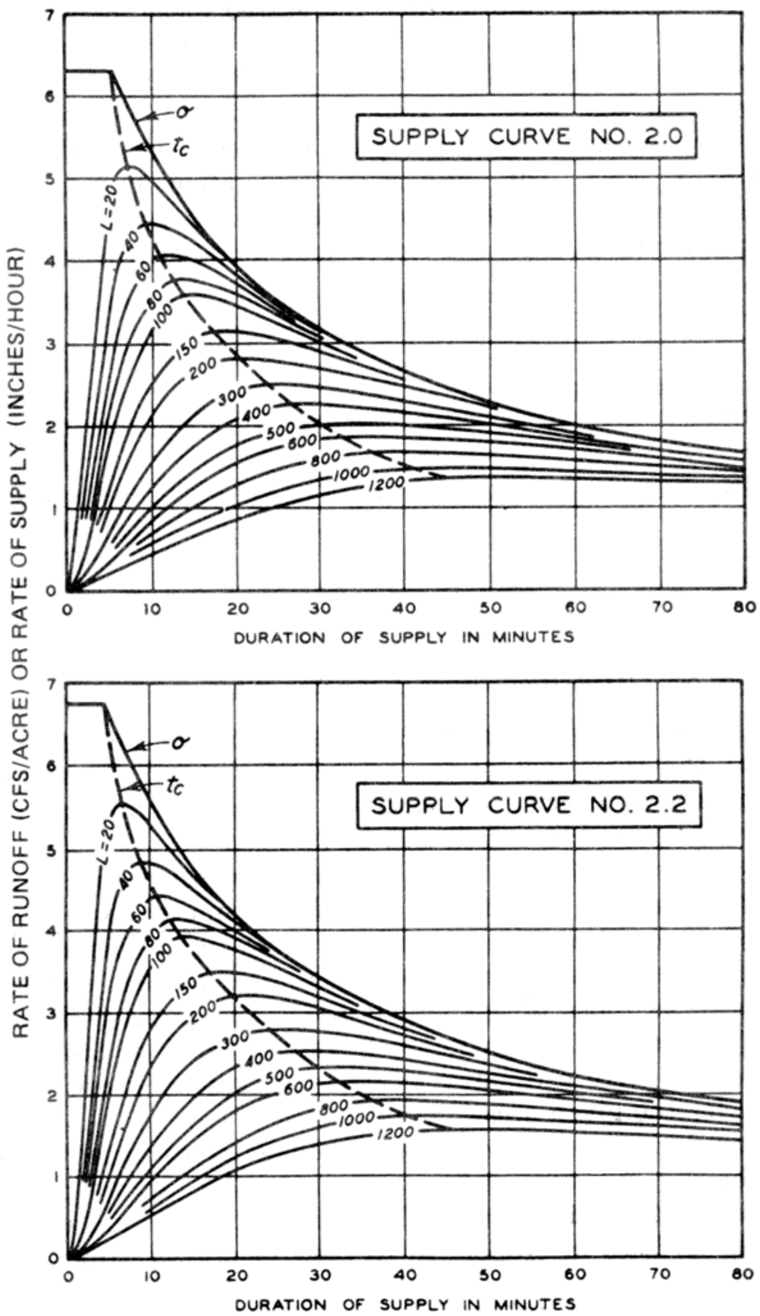


Figure 2-4. Rates of runoff and Rates of Supply Corresponding to Standard Supply Curves No. 2.0 and 2.2; n = 0.40 and S = 1 percent



NOTE: L = EFFECTIVE LENGTH OF OVERLAND OR CHANNEL FLOW, IN FEET.

t_c = CRITICAL DURATION OF SUPPLY, IN MINUTES, ASSUMING SURFACE STORAGE AS NEGLIGIBLE.

σ = RATE OF SUPPLY, IN INCHES PER HOUR.

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2-4 INFILTRATION

2-4.1 **Definition.** As used herein, the term "infiltration" refers to the absorption of rainfall by the ground during a design storm. The infiltration capacity, or ability of a soil to absorb precipitation, normally decreases as the duration of rainfall increases, until a fairly definite minimum rate is reached. Variations in the degree of compaction, soil moisture deficiencies at the beginning of rainfall, and the depth to the groundwater table may greatly influence the infiltration capacity of a particular soil.

2-4.2 **Variability.** Because of the several variables that affect the infiltration capacity of a given soil, it is impracticable to determine accurately the infiltration capacities assumed to apply during storms. The rate of infiltration changes not only during the course of a storm but also during a season. The infiltration rate also varies with the type of soil structure, the soil cover, the temperature of air, soil, and water, the moisture content of soil, turbidity of the water, and the amount of organic matter in the soil. The total porosity of a soil determines to a considerable extent the total amount of water that may filter into it. Available data indicate that the rate of infiltration increases with a rise in the temperature of the air, soil and water, and conversely, the rate of infiltration lessens with an increase in the moisture content of the soil. Soils with a high organic matter content also have high infiltration rates. Vegetative cover serves as a protection from the impact of rain, retards the rate of runoff, and thereby reduces the velocity of overland flow and turbidity, and permits greater infiltration of water into the soil. Rates of infiltration on bare soil can be expected to be considerably less than those for turfed areas. For use in the design of storm drains for a particular airfield or heliport, the infiltration capacity that is estimated to be characteristic of the given soil, *following a rainfall of 1 hr*, serves as the most convenient index to the probable volume of loss through infiltration during the design storm. Antecedent rainfall conditions such as those ordinarily occurring during seasons in which the adopted design storm is likely to occur will be assumed in estimating the 1-hr infiltration rate referred to above.

2-4.3 **Rate.** In permafrost regions, groundwater percolation rates are much lower than in thawed soils and the rate of infiltration for design purposes should be considered zero. In other areas, a good guide can be obtained when test borings are made. Rates would normally not exceed about 0.5 in./hr for clayey soils with low permeability.

2-5 **SNOWMELT.** Airfields, heliports, and other pavement areas in the Arctic and Subarctic are subjected to their most critical drainage requirements during spring thaw and other periods of snow and ice melting. Initial periods of higher temperatures and longer days result in densification or "ripening" of snow, subsequently converted to snowmelt runoff. With banked water-laden snow on or adjacent to pavements, inlets and drainage ditches, a maximum rate of runoff from snowmelt, exclusive of rainfall, is about 0.1 in./hr. In regions of lesser snowfall accumulation, snowmelt runoff at half this rate, 0.05 in./hr, would be expected. Accordingly, an amount of 0.05 to 0.1 in/hr for snowmelt will be added to the design rainfall intensity rate for drainage facilities in the Arctic and Subarctic.

2-6 **SUPPLY.** The term “rate of supply” refers to the rainfall intensity plus snowmelt minus the infiltration capacity at the same instant of a particular storm. To simplify computation procedures, the rainfall intensity, rate of snowmelt and infiltration capacity are assumed to be constant during any specific storm. On this premise, the rate of supply during a particular storm would also be uniform.

2-6.1 **Average Rates of Supply.** Average rates of supply corresponding to storms of different durations and the same average frequency of occurrence can be computed by subtracting estimated infiltration capacities from rainfall plus snowmelt intensities represented by the proper standard rainfall intensity-duration curve in Figure 2-3. For convenience, standard supply curves are assumed to have same shape as the rainfall intensity-duration curves. For example, if curve 0.8 in Figure 2-3 was indicated by Figure 2-1 as the design rainfall plus snowmelt, and infiltration loss at the rate of 0.2 in./hr was estimated to be applicable, curve 0.6 would be adopted as the supply curve for that area.

2-6.2 **Weighted Standard Supply Curves.** In most cases, drainage areas consist of combinations of paved and unpaved areas having different infiltration capacities. To simplify computations, weighted standard supply curves should be estimated for composite tributary drainage areas by weighting the standard supply curve numbers adopted for paved and unpaved surfaces in proportion to their respective areas.

2-7 **RUNOFF**

2-7.1 **Notation.** Symbols used in equations and discussions contained in the following paragraphs are defined below:

L = effective length of overland flow, ft (See discussion of effective length in 2-7.3 and 2-7.5 below.)
n = retardance coefficient
Q = discharge capacity, ft^3/sec, at a designated point
Q_d = drain-inlet capacity, ft^3/sec
q = rate of overland flow at the lower end of an elemental strip of turfed, bare, frozen or paved surface, in./hr or in ft^3/sec per acre of drainage area
q_d = drain-inlet capacity, or maximum rate of outflow from a ponding area, ft^3/sec per acre of tributary drainage area
q_p = peak runoff rate, in./hr or ft^3/sec per acre of drainage area
S = slope of surface, or hydraulic gradient
t = time, or duration, min; time from beginning of supply
t_c = critical duration or supply, min; that is, the duration of rainfall plus snowmelt excess (rate of supply) for a given standard supply curve that would produce the maximum rate of outflow from a given drainage area, taking into account surface detention and surface runoff characteristics

t_d = time required for water to travel from a specified inlet to a given point in the drainage system, min
t_r = duration of supply, min
σ = rate of supply or rainfall plus snowmelt in excess of the rate of infiltration, in./hr
tanh = hyperbolic tangent (defined as the quotient of the hyperbolic sine divided by the hyperbolic cosine, i.e., tanh

$$x = \frac{\sinh x}{\cosh x},$$

the hyperbolic functions having the same relationship to the equilateral hyperbola as the trigonometric functions do to the circle).

2-7.2 **Overland Flow Equation.** The term “overland flow” as used herein relates to surface runoff, resembling sheet flow, before it has reached a defined channel or ponding basin. Horton developed an equation for the rate of overland flow to be expected from a uniform rate of rainfall excess, or rate of supply, which in a form modified for this manual is as follows:

$$q = \sigma \tanh^2 \left[0.922t \ \ (\sigma / nL)^{0.50} S^{0.25} \right]$$

2-7.3 **Effective Length.** In the basic derivation of the above equation, the term L, effective length, represents the length of overland sheet flow measured in a direction parallel to the maximum slope, before the runoff has reached a defined channel. In actuality, particularly in large drainage areas and under many conditions of grading, considerable channelized flow will occur during the design storm conditions. Investigation of many runoff records for watersheds similar to typical airfield and heliport areas in the continental United States indicates that by modifying the determination of effective length, satisfactory reproduction of runoff by hydrographs can be obtained regardless of channelization of flow. The effective length L is the sum of the channelized flow length and the overland flow length, each converted to an equivalent length for n = 0.40 and S = 1.0 percent by means of Figure 2-5. The length of channel flow is measured along the proposed collecting channel or swale for that section in which appreciable depth of flow may occur during the design storm. Length of overland flow is the average distance from the end of the effective channel, if any, or the drain to the outer periphery of the drainage area. Even with excellent grading, overland flow lengths seldom exceed a few hundred feet before channelization occurs. Typical values of the retardance coefficient n for use in determining equivalent length of overland flow are shown in Table 2-1. A guide to selection of n values in the case of channelized flow is shown in Figure 2-6. A more detailed description of the procedure for selecting “n” value is contained in Chapter 3 and Section 4-2.1.3.

2-7.4 **Ponding.** Although provision of ponding areas is advantageous in temperate zone drainage designs, ponding on or alongside paved areas should be avoided in permafrost regions. There, water accumulated alongside airfield or roadway pavement embankments can cause thermal as well as mechanical erosion. Saturation of fine-grained soil and subsoil shortly before freezeup in the fall may greatly increase subsequent destructive frost heaving damage.

Figure 2-5. Airfield Drainage-Overland Flow Relations. Modification in L Required to Compensate for Differences in n and S

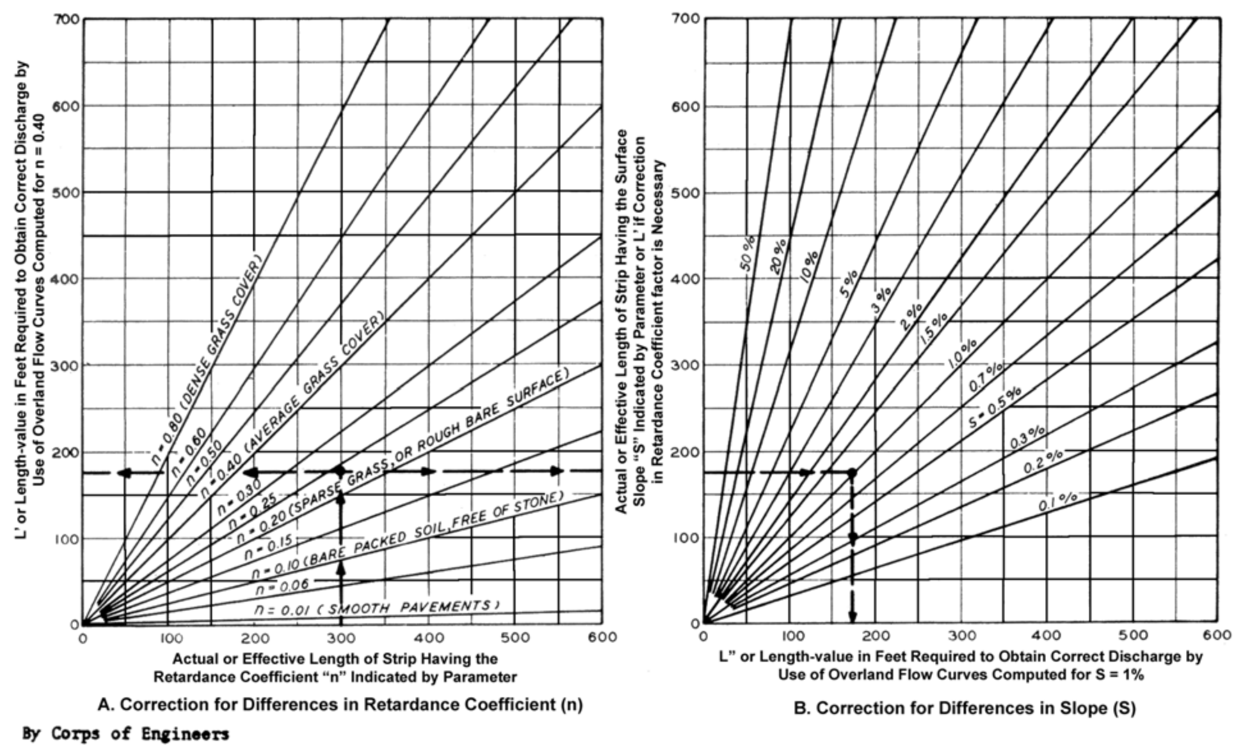

Table 2-1. Retardance Coefficients for Overland Flow

Surface	Value of n
Pavements and frozen ground	0.01
Bare packed soil free of stone	0.10
Sparse grass cover, tundra, or moderately rough bare surface	0.20
Average grass cover	0.40
Dense grass cover	0.80

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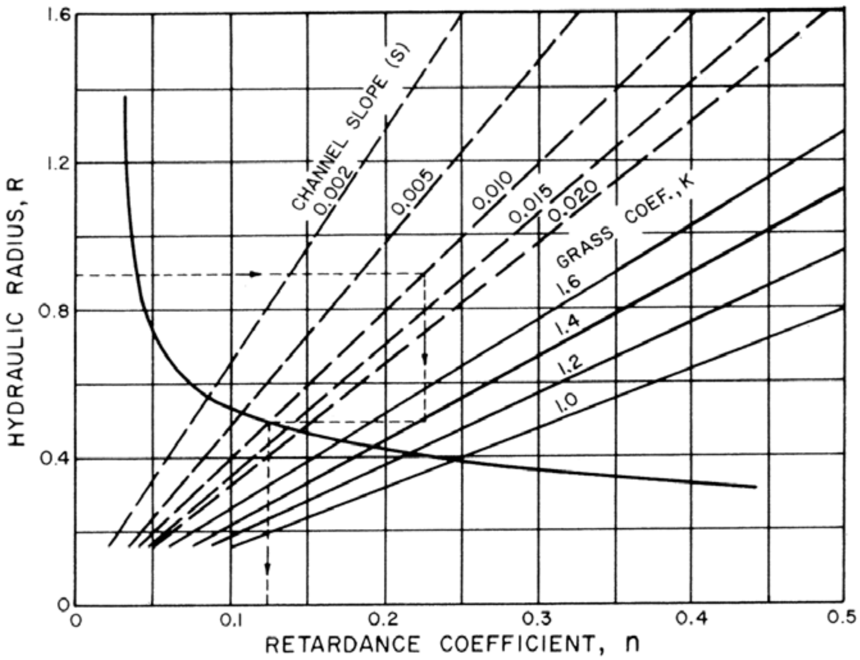
2-7.5 **Effect of Paved Area on Determination of Effective Length.** The time required for water to run off the average paved or ice-covered area is normally very short. Consequently, the length of the paved areas need be given little weight in estimating the effective length L for a composite area. As q is inversely proportional to L, it is helpful to grade the slopes so that the drain inlet is located as far as practicable from the watershed center. In a rectangular area, a drain inlet located near a corner would require less discharge capacity than one located in or near the center of the plot.

Figure 2-6. Retardance Coefficients for Flow in Turfed Channels



AVERAGE GRASS COEFFICIENT (K) FOR DENSE AIRFIELD TURF

AVERAGE LENGTH OF GRASS IN INCHES	Under 6 in. (0-15 cm)	6 to 12 in. (15-30 cm)	Over 12 in. (30 cm)
GRASS COEF. (K)	1.4	1.3	1.2

EXAMPLE:

Determine "n" for a channel with 4 inches of dense grass, R = 0.9, and S = 0.010.

From table, K = 1.4; from graph, by following dotted line, "n" is equal to 0.12.

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2-7.6 **Relation of Overland Flow is Standard Supply Curves.** The curves shown in Figures 2-7 through 2-12 were obtained by computing the rates of discharge, at appropriate time intervals that would result from various rates of supply, corresponding to the respective standard supply curves of Figure 2-3. The procedure is illustrated by the sample computations in Table 2-2. The curves shown are not hydrographs for any specific design storm but represent the peak rates of runoff from individual storm events of various durations, all of which have the same average frequency of occurrence. The duration of supply corresponding to the greatest discharge for a particular standard supply curve and value of L in Figures 2-7 through 2-12 is defined as the critical duration of supply t_c for runoff from an area.

Figure 2-7. Supply Curve No. 0.2

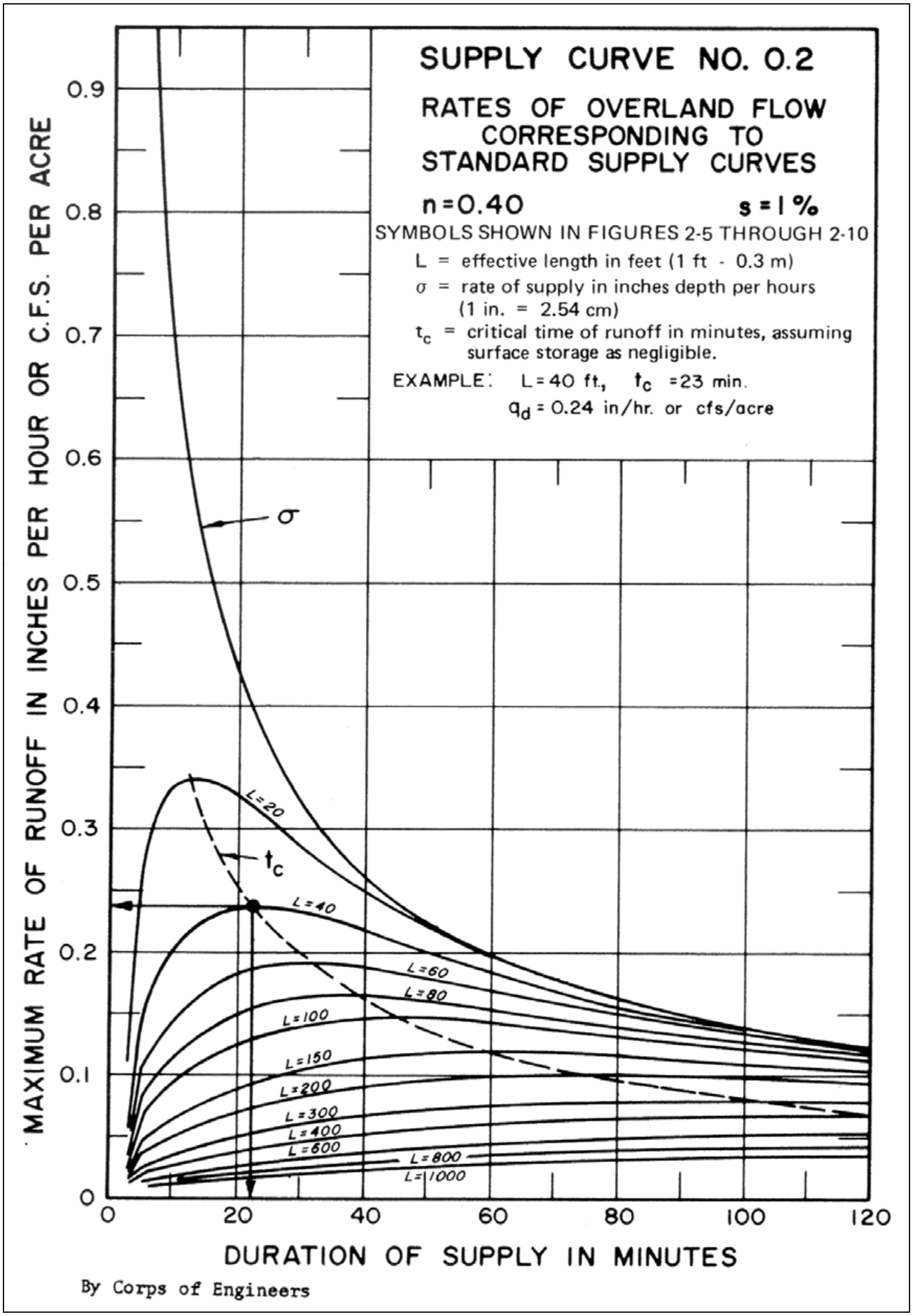

Figure 2-8. Supply Curve No. 0.4

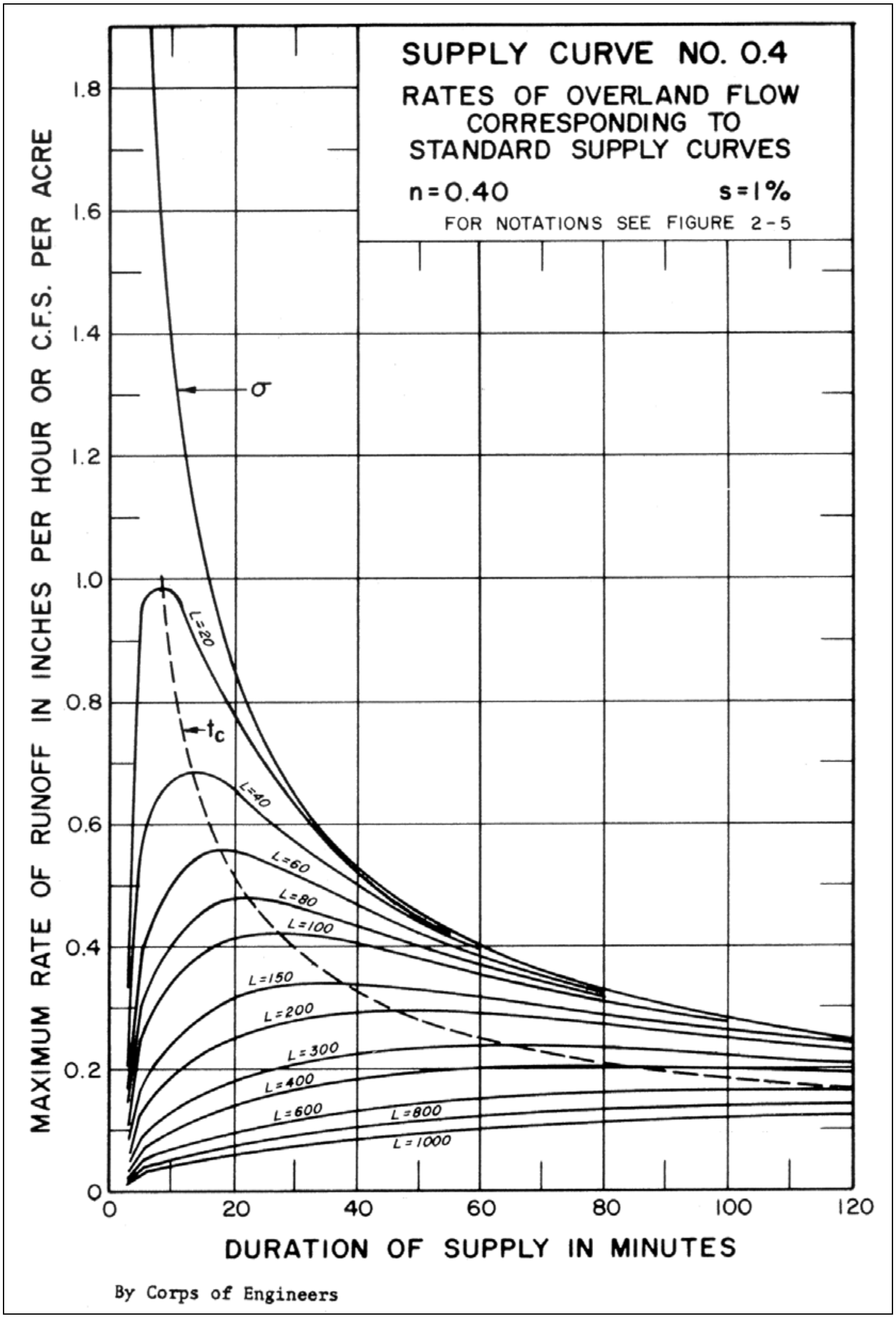

Figure 2-9. Supply Curve No. 0.6

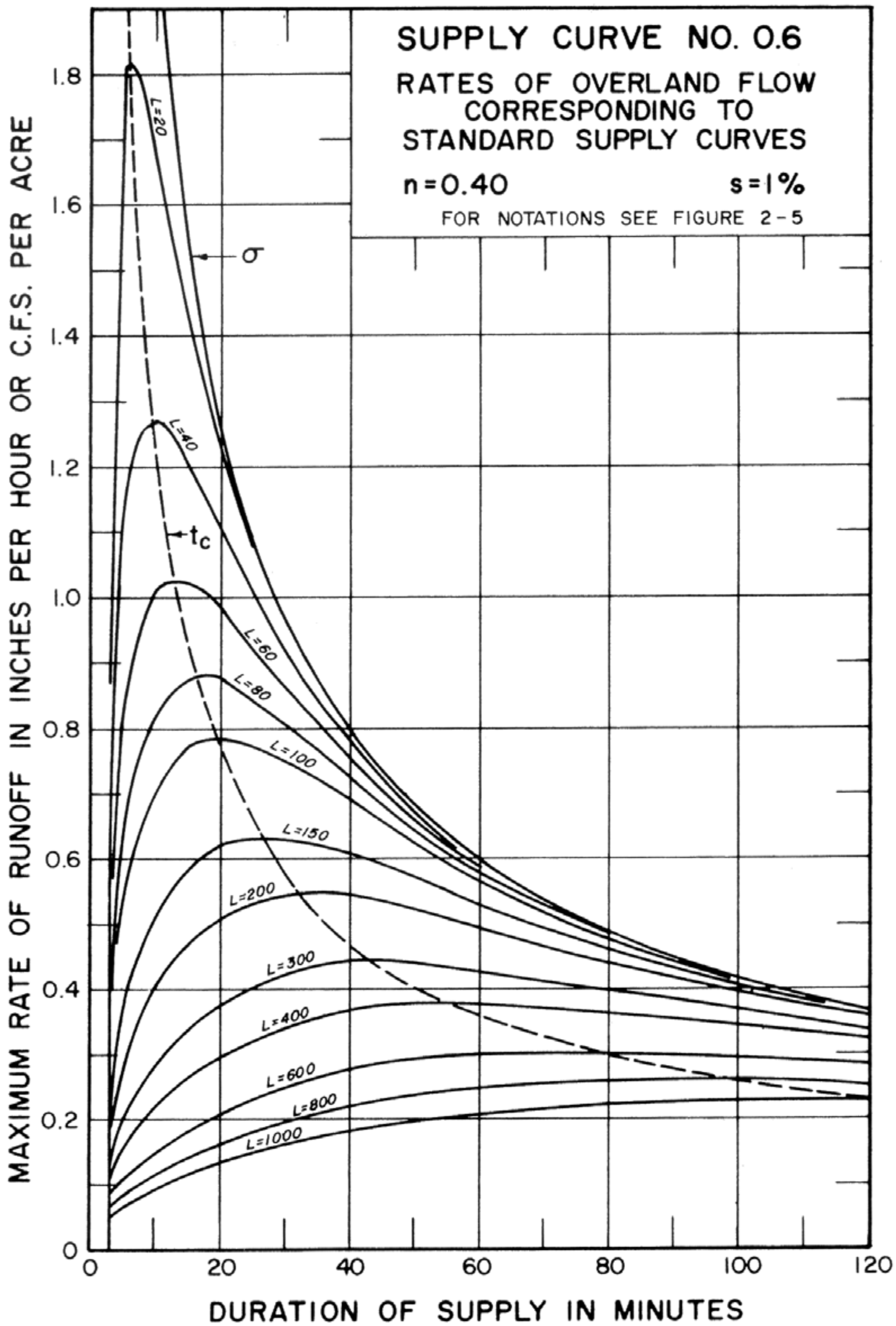

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Figure 2-10. Supply Curve No. 0.8

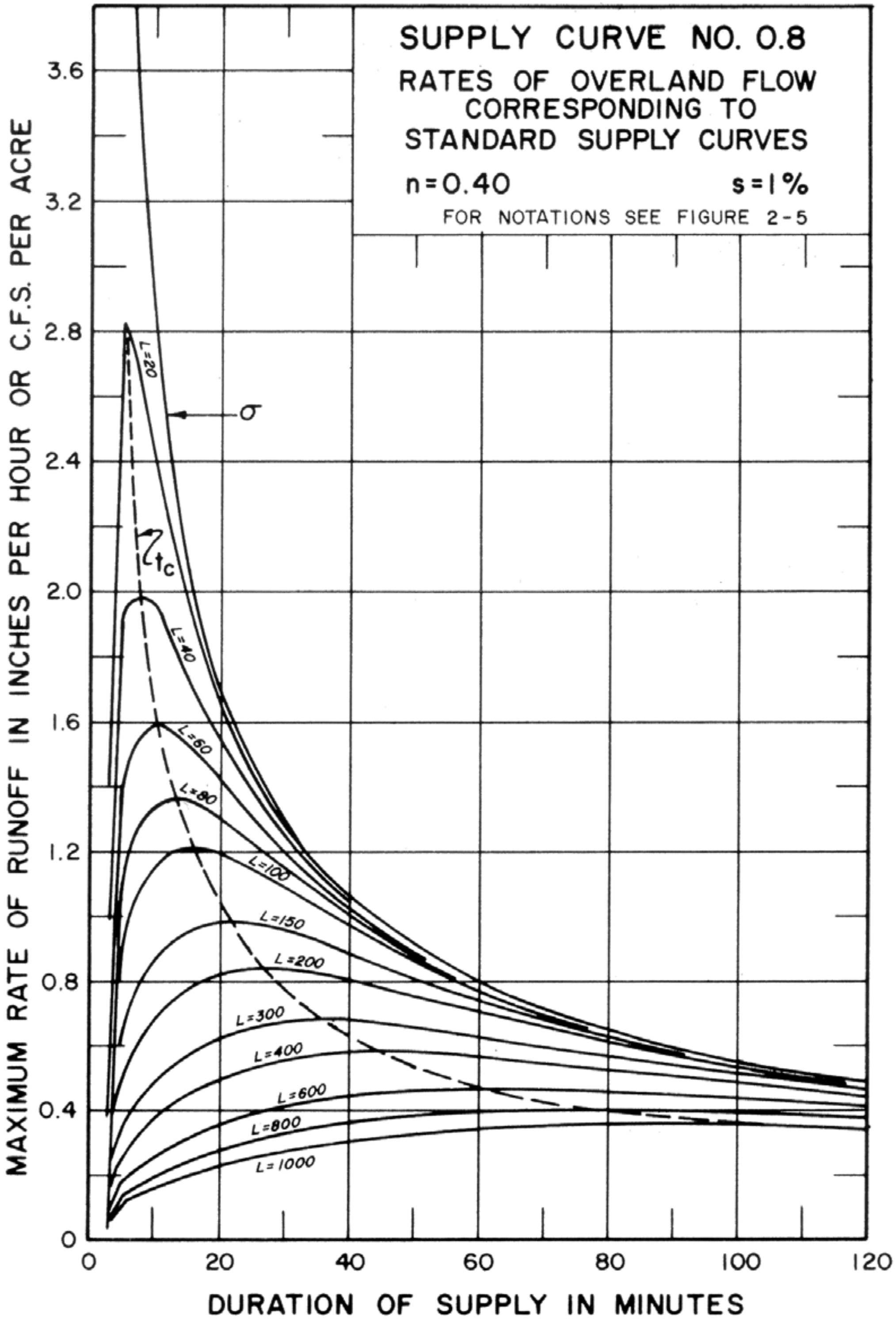

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Figure 2-11. Supply Curve No. 1.0

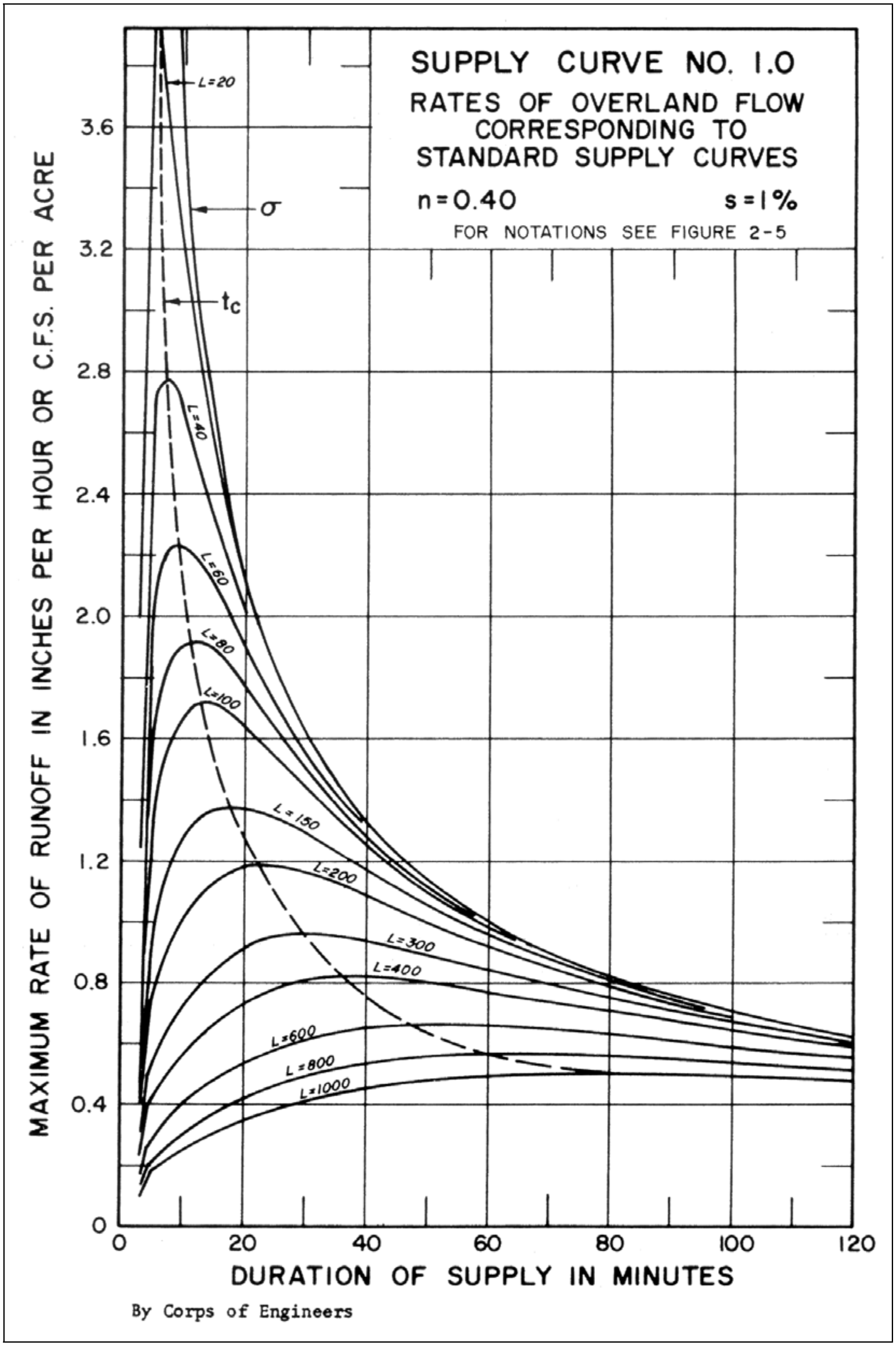

Figure 2-12. Supply Curve No. 1.2

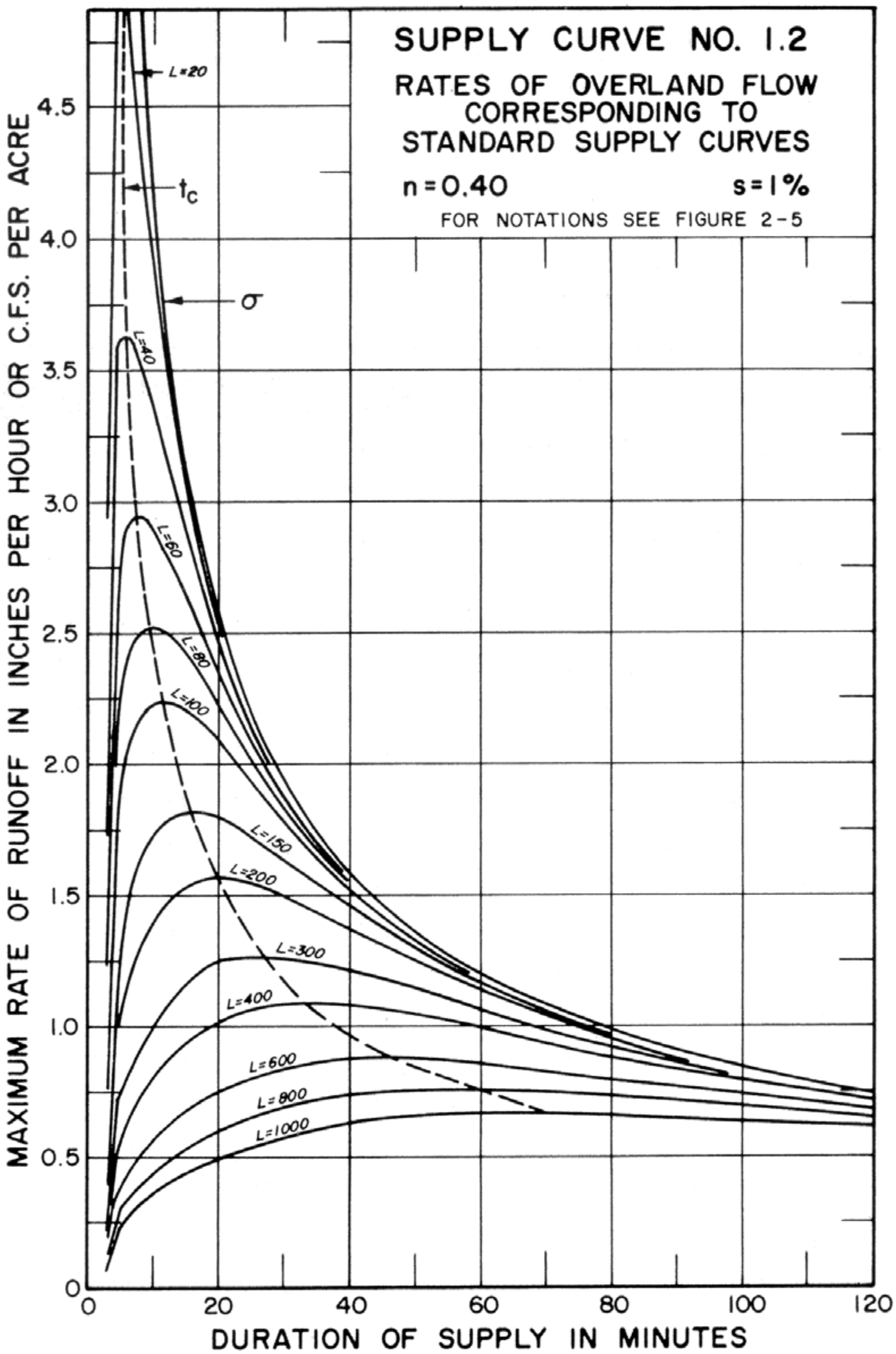

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Table 2-2. Rates of Overland Flow Corresponding to Intensities Shown on Supply Curve 0.2 in Figure 2-3

Duration of Supply, min.	Rate of Supply, in./hr	Rate of overland flow in c.f.s. for various durations and rates of supply where L equals										
		20 ft	40 ft	60 ft	80 ft	100 ft	150 ft	200 ft	300 ft	400 ft	600 ft	800 ft
3	1.113	0.111	0.058	0.039	0.031	0.024	0.017	0.013	0.009	0.008	0.006	0.003
5	1.113	0.273	0.149	0.104	0.080	0.065	0.043	0.035	0.023	0.018	0.011	0.009
7	0.883	0.306	0.175	0.122	0.093	0.077	0.053	0.041	0.027	0.022	0.015	0.011
9	0.743	0.328	0.194	0.137	0.107	0.087	0.060	0.046	0.031	0.025	0.016	0.013
12	0.608	0.340	0.213	0.154	0.122	0.100	0.069	0.053	0.036	0.028	0.019	0.015
15	0.522	0.339	0.227	0.167	0.133	0.110	0.078	0.060	0.041	0.031	0.022	0.017
20	0.430	0.329	0.237	0.184	0.148	0.125	0.090	0.069	0.048	0.037	0.030	0.020
25	0.367	0.308	0.236	0.190	0.157	0.132	0.097	0.076	0.054	0.041	0.029	0.023
30	0.323	0.287	0.232	0.191	0.162	0.139	0.103	0.081	0.058	0.045	0.031	0.024
35	0.292	0.269	0.226	0.192	0.164	0.145	0.109	0.088	0.063	0.049	0.034	0.026
40	0.265	0.250	0.217	0.188	0.164	0.145	0.112	0.091	0.065	0.052	0.036	0.028
45	0.245	0.235	0.210	0.184	0.164	0.147	0.115	0.094	0.069	0.054	0.038	0.030
50	0.227	0.220	0.201	0.179	0.161	0.145	0.116	0.096	0.071	0.056	0.040	0.031
60	0.200	0.197	0.184	0.170	0.155	0.143	0.117	0.100	0.075	0.060	0.043	0.034
80	0.163	0.162	0.157	0.149	0.141	0.133	0.115	0.100	0.079	0.065	0.048	0.038
100	0.140	--	0.138	0.134	0.129	0.123	0.110	0.099	0.081	0.068	0.051	0.041
120	0.123	--	--	0.120	0.117	0.113	0.104	0.095	0.080	0.069	0.054	0.043

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2-8 ICING

2-8.1 **Description**. The term icing (sometimes misnamed “glaciering”) applies to a surface ice mass formed by the freezing of successive sheets of water, the source of which may be a river or stream, a spring, or seepage from the ground. When icing occurs at or near airfields, heliports, roadways or railroads, the drainage structures and channels gradually fill with ice, which may spread over pavements or structures, endangering and disrupting traffic and operations. Ice must be removed from pavements or structures, and drainage facilities must be cleared to avoid or limit the re-forming of icing. Obstruction of flow through drainage facilities—culverts, bridges, pipelines or channels—can lead to washout of pavement embankments or undermining of structures. The spring thaw period is most critical in this regard. Prevention or control of icing at or near drainage structures and the related effects on pavements and other facilities is a key objective of drainage design and maintenance in the Arctic and Subarctic. As icing can occur throughout both seasonal frost and permafrost areas, they are a widespread cause of recurring operational and maintenance problems. Drainage designs based only on conventional criteria will not fulfill the abnormal hydraulic conveyance requirements of icing-prone regions and will be subject to troublesome maintenance problems. Special design and maintenance concepts, based mainly on field experience under similar situations, are required.

2-8.2 **Types**. Icing is classed conveniently as river or stream icing, ground icing, or spring icing, although sometimes it is difficult to assign a specific type to a particular situation. The three general types of icing are discussed below.

2-8.2.1 **River or stream icing.** These occur more commonly on shallow streams with large width/depth ratios. Braided or meandering channels are more prone to icing formation than well defined single channels. River or stream icing normally begins to develop soon after normal ice cover forms on a stream surface, generally during October to December. The icing begins with the appearance of unfrozen water on the surface of the normal ice cover. This water may originate from cracks in the ice cover, from seepage through unfrozen portions of soil forming the channel banks, from adjacent springs which normally discharge into the channel, or other sources. This water, flowing in sheets of an inch or less in thickness to a foot or more, freezes in a layer. Each overflow even is followed by another, with new flow atop the previously frozen sheet, the icing growing higher layer upon layer with its boundaries extending laterally according to the topography. River icing may grow for only part of the winter or throughout the period of below-freezing temperatures. Icing behavior usually varies but little year by year, depending on availability of the feeding water. An icing surface is generally flat but can be gently terraced with each step marking the frozen edge of a thin overflow layer. Occasionally ice mounds form with cracks developing therein providing outlets for the confined water forming the mounds. The water flows out, continuing the growth of the icing for a limited period. Smaller icing is generally confined to the stream or drainage channel; larger ones may spread over floodplains or pavements. With onset of the spring thawing season, runoff cuts channels through the icing to the streambed. Channels are widened by thawing, collapse of the ice forming the sides, and erosion. Depending on the size of the icing and its geographic location, its remnants may last only until May or June, or in colder regions it may last all summer. In extreme locations, they never completely melt and are known as perennial icing. River or stream icing occurring at culverts is also objectionable in that fish migration is obstructed.

2-8.1.2 **Ground icing.** Unlike river or stream icing; ground icing, while developing on certain topographic features, does not have clearly defined areas of activity. These icings are commonly referred to as seepage icings, due to the way their feed waters appear on the ground surface. Seepage icings may develop on nearly level ground or at points of contact of two different types of relief (such as at the base of a slope) or as encrustations on slopes. Ground icing begins to form at different times of the year depending on the sources and modes of discharge of the feeding waters. Where water seeps from the ground often or continuously, icing may begin to form in September or October, in which case it might also be termed a spring icing. Those forming where water does not usually issue from the ground generally begin to form in November or December or even later in the winter. A characteristic of ground icing is that its development begins with unfrozen water appearing on the ground surface or with the saturation and subsequent freezing of snow on the ground. This water may seep from the soil or fractures in the bedrock, or it may travel along the roots of vegetation, or it may issue from frost-induced cracks in the ground. As the seepage flows are exposed to the cold atmosphere, they freeze; to be followed repeatedly by additional seepages onto the icing surface that also freeze, building up successive thin ice layers, seldom over an inch thick. Ground icings may grow during the winter, being extremely sensitive to weather and local hydrologic conditions of the winter and its preceding seasons. Normally ground icings are limited in size as compared with stream spring icing since their source of supply is limited. Some rapid growth may occur with advent of thawing weather. When general thawing occurs, the ground icing will slowly waste away. This disintegration is unlike that of stream icings where sizable runoff streams can rapidly erode icing.

2-8.1.3 **Spring icing.** Springs found in a variety of topographic situations sustain continuous discharge, leading to early winter formation of icing, generally prior to ground icing. Spring outlets generally remain fixed in location and continue to grow throughout the winter, ultimately reaching a larger size than ground icing. A flow of 1 cu ft/min can create a 1-ft-deep icing covering an acre in one month. Spring icing melts away slowly on all sides and these icings are also eroded by spring water channel flow.

2-8.3 **Natural Factors Conducive to Icing Formation**. These can be summarized as follows:

2-8.3.1 A rainy season prior to freeze-up producing an abundance of groundwater in the annual frost zone of the soil or in the ground above the permafrost.

2-8.3.2 Low air temperatures and little snow during the first half of the winter, that is through January. Early heavy snow minimizes occurrence of icing.

2-8.3.3 Nearness of an impervious horizon such as the permafrost table to the ground surface.

2-8.3.4 Heavy snow depth accumulations during the latter part of winter.

2-8.4 **Effects of Man’s Activities on Icing.** Airfields and heliports, in altering the natural physical environment, have profound effects on icing. The widespread clearing of vegetative cover, cutting and filling of soil, excavation of rock, and provisions for drainage, for example, greatly affect the natural thermal regime of the ground and the hydrologic regimes of both groundwater and surface water. Some of these effects are discussed below.

2-8.4.1 Removal of vegetation and organic soil with their generally higher insulation values than those of the construction materials replacing them results in increased seasonal frost penetration. This may create or aggravate nearby damming of groundwater flow and cause icing. Airfield and heliport pavement areas, kept clear of snow, lack its insulating value and are subject to deeper seasonal frost penetration, causing icing.

2-8.4.2 Cut faces may intersect the water table, and fill sections may block natural drainage channels. Construction compaction operations can reduce permeability of natural soils, blocking natural discharge openings.

2-8.4.3 In cut sections, water comes into contact with the cold atmosphere, forming ground icing where none occurred prior to the construction. Icing grows on the cut face, fills the adjacent drainage ditches with ice, and eventually reaches the pavement surface. In these conditions, deep snow on the slope and ditch insulates seepage from the cut face. Seepage water passes under the snow without freezing and reaches the snow-free pavement where it is sufficiently exposed to freeze. This type of man-made icing is the most common and troublesome type along pavements.

2-8.4.4 Snowplowing and storage of snow greatly affect the location and extent of icing by changing insulation values and damming seepage waters.

2-8.4.5 Channel realignment and grading into wider, more shallow sections, commonly done in airfield and heliport construction, renders the stream more susceptible to high heat losses and extensive freezing and formation of icing.

2-8.4.6 Drainage designers customarily size hydraulic structures to accommodate runoff from a specified design storm. In the Arctic and Subarctic, the size of hydraulic structures based solely on these well-founded hydrologic principles will usually result in inadequate capacity which will contribute or intensify icing formation. Culverts, small bridges, storm drains and inlets designed to accommodate peak design discharges are generally much too small to accommodate icing volumes before becoming completely blocked by ice. Once the drainage openings become blocked, icing upstream from the affected structures will grow markedly. The inadequacy of drainage facilities, both in capacity and number, because of failure to accommodate icing, leads to more serious effects of icing on engineering works.

2-8.5 **Methods of Counteracting Icing.** Several techniques are available for avoiding, controlling, or preventing icing. Although sound in principle, the methods are often applied without adequate understanding of the icing problems encountered, leading to unsuccessful or poor results. Selection of a particular method from the many that might be applied for the given set of conditions is based principally on economics. One must use a systems approach considering costs of installation plus costs of operation and maintenance, energy conservation, and environmental impact. Where feasible, methods requiring no fuel or electrical energy output or little or no service by maintenance personnel are preferred. The techniques for dealing with icings fall into two categories: *avoidance and control* and *prevention*. These are discussed below.

2-8.5.1 **Methods of icing avoidance and control**. These deal with the effects of the icing at the location being protected, so that the type of icing (river or stream, ground, or spring) is of little significance. Methods are as follows:

2-8.5.1.1 **Change of location.** Site facilities where icings do not occur. This is an economic consideration difficult to resolve in siting an airfield with its extensive area, grading and lateral clearance requirements.

2-8.5.1.2 **Raising grade.** This will deter or postpone icing formation but is costly and depends on availability of ample fill. There is also threat of embankment washouts resulting from ice-blocked facilities, and possibility of objectionable seepage effects.

2-8.5.1.3 **More and larger drainage structures.** Susceptibility to icing problems can be reduced by providing more and larger drainage facilities. Openings as much as 2 or 3 times as large as those required by conventional hydraulic design criteria will accommodate sizable icing volumes without encroaching on design flows. Culverts with large vertical dimensions, or small bridges in lieu of culverts, are advantageous. Provision for adequate drainage channels and conduits will facilitate diversion of meltwater runoff from icings, protecting the installation from washouts.

2-8.5.1.4 **Storage space**. This can be provided as a ponding basin or by shifting a cut face further back from the airfield or heliport. There, an icing can grow in an area where it will not encroach on operational facilities.

2-8.5.1.5 **Dams, dikes or barriers.** Known also as ice fences, these are often used to limit the horizontal extent of icings. Permanent barriers of earth, logs or lumber may be built between the source of the icing and the area to be protected. Temporary barriers may be erected of snow embankments, movable wooden fencing, corrugated metal, burlap, plastic sheeting, or expedient lumber construction. In some situations, a second or even third fence is required above the first as the icing grows higher.

2-8.5.1.6 **Culvert closures.** To prevent a culvert being filled with snow and ice, which requires a laborious spring clearing operation, closures are sometimes placed over the culvert ends in the fall. These can be of rocks to permit minor flows prior to freeze-up.

2-8.5.1.7 **Staggered (or stacked) culverts.** This involves placement of two (or more) culverts, one at the usual location at the base of the fill, the other(s) higher in the fill. When the lower culvert becomes blocked by an icing accumulation, the higher ones carry initial spring runoff over the icing. As the spring thaw progresses, the lower one becomes cleared, eventually carrying the entire flow. In cases where there is limited height, the second culvert is placed to the side with its invert at a slightly higher elevation. The ponding area available for icing accumulations must be large enough to store an entire winter's ice without having the icing reach the upper culverts or the elevation of the area being protected.

2-8.5.1.8 **Heat.** Icing is commonly controlled by the application of heat in any of several ways, the objective being not to prevent icing but to establish and maintain thawed channels through it to minimize their growth and to pass spring runoff.

2-8.5.1.9 **Steam.** This method, common in North America, is used to thaw culvert openings and to thaw channels into icing for collecting icing feed water or early spring runoff. Steam, generated in truck-mounted boilers, is conducted through hoses to portable steam lances, or through hoses temporarily attached to permanently installed thaw pipes supported inside the tops of the culverts. Thaw pipes of 3/8- to 2-inch diameter have been used. The thaw pipe is terminated by a vertical riser at each end of the culvert, extending high enough to permit access above accumulated ice and snow. The pipe is filled with antifreeze, with the risers capped when not in use.

2-8.5.1.10 **Fuel oil heaters.** These heaters, known as firepots, are in common use. They consist of a 55-gallon oil drum, equipped with an oil burner unit (railroads often use coal or charcoal as fuel). The drum fed from a nearby fuel supply, is usually suspended from a tripod at the upstream end of the culvert. A continuous fire maintains a thaw pit in the icing. Fuel consumption varies, averaging about 30 gallons per day. Water, flowing over the icing, enters the pit where it receives heat, passes through the culvert, hopefully without refreezing before it flows beyond the area to be protected. While firepots are simple devices, they are inefficient energy sources due to loss of most heat to the atmosphere rather than to the water or icing. Firepots are in decreasing favor due to high maintenance requirements and difficulty in preventing theft of fuel in remote locations.

2-8.5.1.11 **Electrical heating.** Use of insulated heating cables to heat culverts is a recent adaptation successfully used where electrical power is available or, in important locations, where small generating stations would prove feasible. Heating cables have been used, not to prevent icing, but to create and maintain a thawed tunnel-like opening in an icing to minimize its growth and to provide for spring runoff. Cable can be strung in the fall within the culvert and, in some cases, along its upstream drainageway and removed in the spring. Cable can also be permanently installed in a small diameter metal pipe inside the culvert or buried at shallow depth under a drainage ditch or channel. Common heat output is 40 to 50 watts/lineal ft with minimum heat lost to the atmosphere. A tunnel about 2-3 ft wide and 4-5 ft high is achieved by later winter. Electrical heating requires much less attention by maintenance personnel than steam thawing.

2-8.5.1.12 **Breaking and removing accumulated ice.** This common technique, whether by manual or mechanical equipment, should be practiced only as an expedient or emergency measure. Timing of such operations, as for the following two methods, critically limits their effectiveness.

2-8.5.1.13 **Blasting.** This has a twofold objective—physical removal of ice and fracturing ice to provide paths for water flow deep in the icing. This flow can enlarge openings and still remain protected from the atmosphere and refreezing.

2-8.5.1.14 **Deicing chemicals.** Chemicals such as sodium or calcium chloride are sometimes used to prevent refreezing of a drainage facility, once it has been freed of ice by other means. A common practice is to place a burlap bag containing the salt at a culvert inlet, allowing the compound to be slowly dissolved by flow, the solution lowering the freezing point of the water. Objections are the detrimental effects on fish and wildlife, vegetation, and other downstream water uses and corrosive effects on metal pipe.

2-8.5.2 **Methods of icing prevention.** These preventive techniques are best classified according to the general type of icing (Section 2-8.2), as follows:

2-8.5.2.1 **River or stream icing**

a. **Channel modification.** Straightening and deepening a channel can prevent icing, although frequent maintenance is usually required to counteract the stream's tendency to resume natural configuration by erosion and deposition. Rock-fill gabions have been used to create a deep, narrow channel for low winter discharges. Such deepened channels permit formation of ice cover to normal thickness while providing adequate space beneath for flow. Deepening at riffles, rapids, or drop structures is especially important as icing is more likely to form in these shallow areas.

b. **Insulation of critical sections.** This icing may be prevented by insulating critical sections of the stream where high heat losses cause excessive thickening of the normal ice cover, to constrict or completely block flow and result in icing formation. These sections may be located under a bridge or taxiway or at riffles or rapids. The insulation which may be placed on the initial ice cover may consist of soil, snow, brush, peat, sawdust or other material, typically 1 to 2 ft thick. Another way is to cover the stream before ice forms, using logs, timber, or corrugated metal as a support for insulating material, later augmented by snowfall. Insulating covers, while beneficial in lessening heat losses from the stream, must be removed each spring before annual freshets. They may also be washed downstream to become obstructions if high water occurs prior to cover removal.

c. **Frost belts.** Known also as "permafrost belts," these are further discussed below under Ground Icing. A frost belt is essentially a ditch or cleared strip of land upstream or upslope from the icing problem area. If organic soil and vegetative cover are removed and the area is kept clear of snow during the first half of the winter, deep seasonal frost will act as a dam to water seeping through the ground, forcing it to the surface where it will form an icing upstream or upslope from the belt. In applying this technique to a drainage channel, a belt is formed by periodically cutting transversely into the ice to cause the bottom of the ice cover to lower and merge with the bed. In this way, the icing is induced to form away from the bridge or culvert entrance being protected.

2-8.5.2.2 **Ground icing.** The most successful methods of preventing ground icing involve drainage. Other procedures depend on preventing formation in one location by inducing formation elsewhere. Principal methods are cited below.

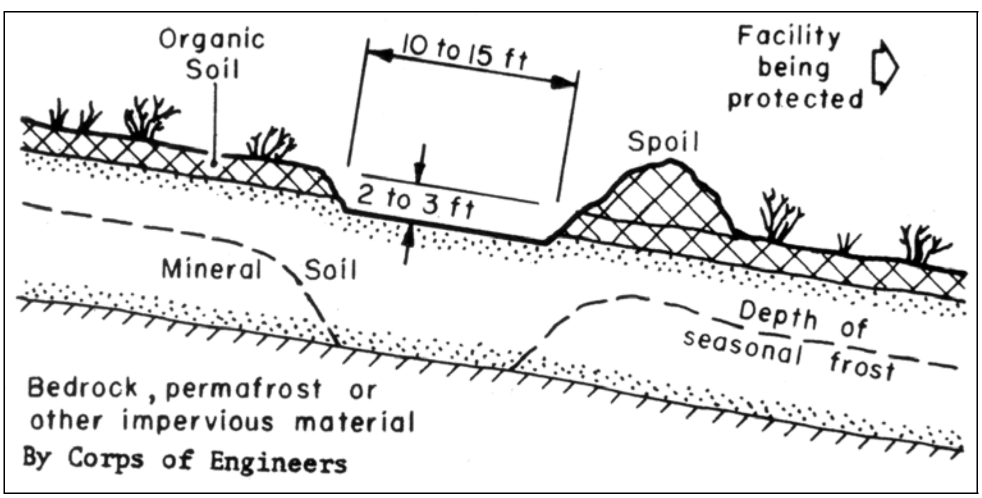
a. **Surface drainage.** This may be accomplished by a network of ditches located so as to drain the soil surface in the region of icing development. Ideally these ditches will be sited in compliance with airfield/heliport lateral safety clearance criteria and be narrow and deep so as to drain the soil to an appreciable depth and to expose only a small surface area to heat loss to the atmosphere. In some cases, these drainage ditches are covered and insulated to maintain flow in winter. Open ditches can be as narrow as 1 ft or, if insulated, about 3 ft wide by 3 ft deep.

b. **Subsurface drainage.** In seasonal frost areas, subsurface drainage systems are more suitable than surface drains because of their better resistance to freezing and ability to intercept more groundwater. They are not suitable for use in permafrost areas due to freezing. Subsurface drainage systems can use any of numerous types of perforated, slotted or open-jointed pipe materials most commonly in 6-in.-diameter size. Improved resistance to freezing can be obtained by placing an insulation layer above the usual granular backfill surrounding the subdrain but beneath the final native soil backfill. In any case, water collected must be conveyed to an outlet away from the area being protected even if it forms an icing at that point.

c. **Insulation of ground.** In some cases ground icings can be prevented by insulating the ground in areas where deep seasonal frost penetration forms a dam, blocking groundwater flow. Insulating material may be snow, soil, brush, or peat. This technique may merely shift the location where an impervious frost dam occurs. It is essential that the insulation of the ground extend under the pavement being protected to assure that ground water flow is maintained past it. Otherwise, seasonal frost penetration under a snow-free airfield pavement would act as a frost dam and cause an icing to form upslope from the area. Suitable insulation materials for pavements are available and have been used.

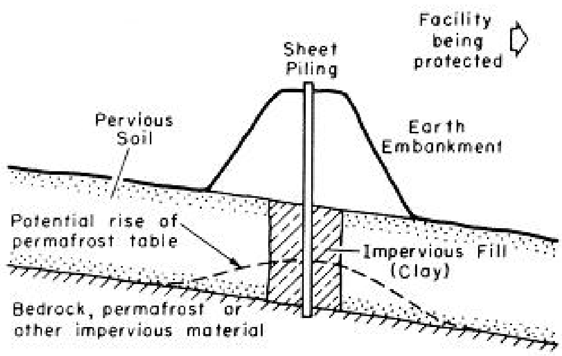
d. **Frost belts.** Successful use of frost belts requires careful siting, planning and maintenance. They may be either permanent or seasonal. The permanent type belt, as mentioned above for control of river or stream icing, is a strip of land cleared of organic soil and vegetation, extending across a slope normal to the direction of seepage flow. Seasonal frost beneath this belt, merging with or approaching some impervious base, causes an icing to form upslope from the belt location. The belt must be long enough to prevent the icing from extending around the ends of the belt and approaching the airfield or other areas being protected. Such a belt is usually about 2 to 3 ft deep and 10 to 15 ft wide. Spoil from the excavation is placed as a low ridge on the downslope side of the belt (Figure 2-13). The shape of the frost belt depends on the topography; often it is slightly convex downslope, or made of two straight segments meeting at an angle of 160-170 degrees on the upslope side of the belt. Sometimes more than one belt is needed, the belts being arranged parallel to each other with their spacing depending on the channel slope. Permanent frost belts require attention to avoid degradation of the permafrost table underneath as the insulation of the ground has been reduced by removing the organic soil and vegetative cover. After a few years, the permafrost table may lower so much that the seasonal frost penetration in the winter will not reach it. In such a case, seepage flow in the soil would not be stopped at the belt; an icing does not develop at the belt but occurs instead downslope at the airfield or other facility intended to be protected. This can be avoided by covering the belt area in the spring with an insulating material and removing it in the fall before the onset of winter frosts. The belt must be kept clear of snow through the first half of the winter to permit rapid and deep seasonal frost penetration. Seasonal type frost belts are free from most maintenance requirements associated with the permanent type and are much simpler and more economical to construct. Instead of preparing a ditch in the ground, one merely clears a strip of snow at the desired belt location and keeps it free of snow during the first half of the winter. The cleared snow is piled downslope of the belt, forming a ridge. The chief advantage of the seasonal belt is that it is less likely to degrade the underlying permafrost; this objective can be further assured by relocating the belt up- or downslope in successive winters. A disadvantage of the seasonal belt is that seasonal frost penetrates below it more slowly, owing to the high specific heat of the wet organic soil and the insulation afforded by the vegetation left in place. It therefore takes longer for a frost dam to form and stop the flow of seepage water. This may permit formation of some icing at the downslope protected area early in the winter before the seasonal frost belt attains full effectiveness. Frost belts have not been widely accepted because of neglect in placement of summer insulation and priority attention to snow removal from pavements rather than from frost belt areas in the winter. Frost belts are much easier to maintain in locations where the impervious base which restrict groundwater flow is other than permafrost, and thus is not subject to degradation.

Figure 2-13. Typical Cross Section of a Frost Belt Installation

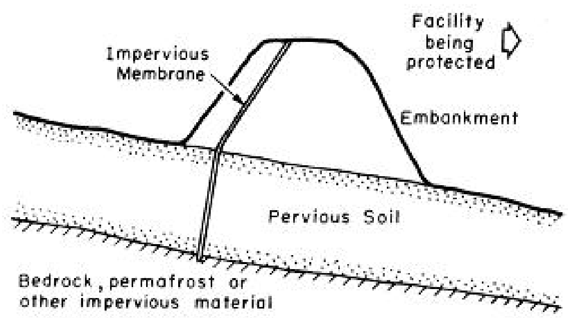


e. **Earth embankments and impervious barriers.** Ground icing formation can also be prevented by use of earth embankments combined with impervious barriers to groundwater flow. These are placed well away from the area to be protected and function similarly to frost belts in that they dam seepage flow through the soil, causing it to rise to the ground surface where it freezes to form an icing. In southern permafrost zones where permafrost is close to freezing temperatures, embankments may cause the permafrost to melt, leading to subsidence. Methods of developing the impervious barrier include trenching across the slope down to the impervious stratum, filling the trench with clay and then driving a row of sheet piling through it extending several feet above the surface to aid in ponding (Figure 2-14a). Other expedients include use of plastic membrane instead of piling (Figure 2-14b) or burial or horizontal air duct pipe (12 to 18 in.), located usually 4 to 6 ft below the bottom of the embankment.

Figure 2-14. Earth Embankments with Impervious Barriers



(a) Sheet Piling Barrier



(b) Plastic Membrane Barrier

By Corps of Engineers

Vertical air shafts from the horizontal ducts permit cold winter air to permeate the system, removing heat from the ground and freezing the soil beneath the embankment to create an impervious barrier. The vertical air shafts are sealed in the summer to prevent excessive thawing in the soil. A problem which has arisen in some duct installations is that if they are not completely watertight, infiltrated water will freeze in the duct, causing an obstruction, generally difficult to clear. As this type installation would obstruct seepage flow year-round, rather than just in winter, gated openings must be provided to allow accumulated water to flow downslope during the summer. The openings are closed all winter to assure that the icing will form upslope from the embankment. An innovation is use of a steel mesh grid with apertures 8 to 32 in. square. These permit passage of water when the air is warm, but gradually freeze until a blockage forms in subfreezing weather. Grids must be removed in the summer to avoid debris accumulation.

2-9. AREAS OTHER THAN AIRFIELDS

2-9.1 Design Storm

2-9.1.1 For such developed portions of military installations as administrative, industrial, and housing areas, the design storm will normally be based on rainfall of 10-yr frequency. Potential damage or operational requirements may warrant a more severe criterion; in certain storage and recreational areas, a lesser criterion may be appropriate. (With concurrence of the using Service, a lesser criterion may also be employed in regions where storms of an appreciable magnitude are infrequent and either damages or operational capabilities are such that large expenditures for drainage are not justified.)

2-9.1.2 The design of roadway culverts will normally be based on 10-yr rainfall. Examples of conditions where greater than 10-yr rainfall may be used are areas of steep slope in which overflows would cause severe erosion damage; high road fills that impound large quantities of water; and primary diversion structures, important bridges, and critical facilities where uninterrupted operation is imperative.

2-9.1.3 Protection of military installations against floodflows originating from areas exterior to the installation will normally be based on 25-yr or greater rainfall, again depending on $\frac{HW}{D}$ operational requirements, cost-benefit considerations, and nature and consequences of flood damage resulting from the failure of protective works. Justification for the selected design storm will be presented, and, if appropriate, comparative costs and damages for alternative designs should be included.

2-9.1.4 Rainfall intensity will be determined from the best available intensity-duration-frequency data. Basic information of this type will be taken from such publications as (see Appendix A for referenced publications):

Rainfall Frequency Atlas of the United States. Technical Paper No. 40.

Generalized Estimates of Probable Maximum Precipitation and Rainfall-Frequency Data for Puerto Rico and Virgin Islands. Technical Paper No. 42.

Rainfall-Frequency Atlas of the Hawaiian Islands. Technical Paper No. 43.

Probable Maximum Precipitation and Rainfall Frequency Data for Alaska. Technical Paper No. 47.

TM 5-785/AFM 88-29/NAVFAC P-89.

These publications may be supplemented as appropriate by more detailed publications of the Environmental Data and Information Center and by studies of local rainfall records. For large areas and in studies involving unit hydrography and flow-routing procedures, appropriate design storms must be synthesized from areal and time-distribution characteristics of typical regional rainfalls.

2-9.1.5 For some areas, it might reasonably be assumed that the ground would be covered with snow when the design rainfall occurs. If so, snowmelt would add to the runoff. Detailed procedures for estimating snowmelt runoff are given in Section 2-5. It should be noted, however, that the rate of snowmelt under the range of hydro-meteorological conditions normally encountered in military drainage design would seldom exceed 0.2 in. per hour and could be substantially less than that rate.

2-9.1.6 In selecting the design storm and making other design decisions, particular attention must be given to the hazard to life and other disastrous consequences resulting from the failure of protective works during a great flood. Potentially hazardous situations must be brought to the attention of the using service and others concerned so that appropriate steps can be taken.

2-9.2 Infiltration and Other Losses

2-9.2.1 Principal factors affecting the computation of runoff from rainfall for the design of military drainage systems comprise initial losses, infiltration, transitory storage, and, in some areas, percolation into natural streambeds. If necessary data are available, an excellent indication of the magnitudes of these factors can be derived from thorough analysis of past storms and recorded flows by the unit-hydrograph approach. At the onset of a storm, some rainfall is effectively retained in “wetting down” vegetation and other surfaces, in satisfying soil moisture deficiencies, and in filling surface depressions. Retention capacities vary considerably according to surface, soil type, cover, and antecedent moisture conditions. For high intensity design storms of the convective, thunderstorm type, a maximum initial loss of up to 1 in. may be assumed for the first hour of storm precipitation, but the usual values are in the range of 0.25 to 0.50 in./hr. If the design rainfall intensity is expected to occur during a storm of long duration, after substantial amounts of immediately prior rain, the retention capacity would have been satisfied by the prior rain and no further assumption of loss should be made.

2-9.2.2 Infiltration rates depend on type of soils, vegetal cover, and the use to which the areas are subjected. Also, the rates decrease as the duration of rainfall increases. Typical values of infiltration for generalized soil classifications are shown in Table 2-3. The soil group symbols are those given in MIL-STD-619, Unified Soil Classification System for Roads, Airfields, Embankments, and Foundations. These infiltration rates are for uncompacted soils. Studies indicate that compacted soils decrease infiltration values from 25 to 75 percent, the difference depending on the degree of compaction and the soil type. Vegetation generally decreases the infiltration capacity of coarse soils and increases that of clayey soils.

2-9.2.3 Peak rates of runoff are reduced by the effect of transitory storage in watercourses and minor ponds along the drainage route. The effects are reflected in the C factor of the Rational Formula (given below) or in the shape of the unit hydrography. Flow-routing techniques must be used to predict major storage effects caused by natural topography or man-made developments in the area.

Table 2-3. Typical Values of Infiltration Rates

Description	Soil Group Symbol	Infiltration, in./hr
Sand and gravel mixture	GW, GP SW, SP	0.8-1.0
Silty gravels and silty sands to inorganic silt, and well-developed loams	GM, SM ML, MH OL	0.3-0.6
Silty clay sand to sandy clay	SC, CL	0.2-0.3
Clays, inorganic and organic	CH, OH	0.1-0.2
Bare rock, not highly fractured	…	0.0-0.1
U.S. Army Corps of Engineers		

2-9.2.4 Streambed percolation losses to direct runoff need to be considered only for sandy, alluvial watercourses, such as those found in arid and semiarid regions. Rates of streambed percolation commonly range from 0.15 to 0.5 cfs/acre of wetted area.

2-9.3 **Runoff Computations**

2-9.3.1 Design procedures for drainage facilities involve computations to convert rainfall intensities expected during the design storm into runoff rates which can be used to size the various elements of the storm drainage system. There are two basic approaches: first, direct estimates of the proportion of average rainfall intensity that will appear as the peak runoff rate; and, second, hydrography methods that depict the time-distribution of runoff events after accounting for losses and attenuation of the flow over the surface to the point of design. The first approach is exemplified by the Rational Method which is used in the large majority of engineering offices in the United States. It can be employed successfully and consistently by experienced designers for drainage areas up to 1 square mile in size. *Design and Construction of Sanitary and Stem Sewers*, ASCE Manual No. 37, and *Airport Drainage*, FAA AC 150/5320-5B, explain and illustrate use of the method. A modified method is outlined below. The second approach encompasses the analysis of unit-hydrograph techniques to synthesize complete runoff hydrography.

2-9.3.2 To compute peak runoff the following empirical formula can be used

$$Q = C\,(I-F)\,A$$

where

Q = discharge or peak rate of runoff, cfs
C = coefficient
I = rainfall intensity, in./hr

A = drainage area, acres, total area of clear opening, or cross-sectional are of flow, ft^2

This equation is known as the modified rational method.

2-9.3.2.1 C is a coefficient expressing the percentage to which the peak runoff is reduced by losses (other than infiltration) and by attenuation owing to transitory storage. Its value depends primarily on surface slopes and irregularities of the tributary area, although accurate values of C cannot readily be determined. For most developed areas, the apparent values range from 0.6 to 1.0. However, values as low as 0.20 for C may be assumed in areas with low intensity design rainfall and high infiltration rates on flat terrain. A value of 0.6 may be assumed for areas left ungraded where meandering-flow and appreciable natural-ponding exists, slopes are 1 percent or less, and vegetal cover is relatively dense. A value of 1.0 may be assumed applicable to paved areas and to smooth areas of substantial slope with virtually no potential for surface storage and little or no vegetal cover.

2-9.3.2.2 The design intensity is selected from the appropriate intensity-duration-frequency relationship for the critical time of concentration and for the design storm frequency. Time of concentration is usually defined as the time required, under design storm conditions, for runoff to travel from the most remote point of the drainage area to the point in question. In computing time of concentration, it should be kept in mind that, even for uniformly graded bare or turfed ground, overland flow in “sheet” form will rarely travel more than 300 or 400 ft before becoming channelized and thence move relatively faster; a method which may be used for determining travel-time for sheet flow is given in Chapter 3. Also, for design, the practical minimum time of concentration for roofs or paved areas and for relatively small unpaved areas upstream of the uppermost inlet of a drainage system is 10 min; smaller values are rarely justifiable; values up to 20 min may be used if resulting runoff excesses will not cause appreciable damage. A minimum time of 20 min is generally applicable for turfed areas. Further, the configuration of the most remote portion of the drainage area may be such that the time of concentration would be lengthened markedly and thus design intensity and peak runoff would be decreased substantially. In such cases, the upper portion of the drainage areas should be ignored and the peak flow computation should be based only on the more efficient, downstream portion.

2-9.3.2.3 For all durations, the infiltration rate is assumed to be the constant amount that is established following a rainfall of 1 hour duration. Where F varies considerably within a given drainage area, a weighted rate may be used; it must be remembered, however, that previous portions may require individual consideration, because a weighted overall value for F is proper only if rainfall intensities are equal to or greater than the highest infiltration rate within the drainage area. In design of military construction drainage systems, factors such as initial rainfall losses and channel percolation rarely enter into runoff computations involving the Rational Method. Such losses are accounted for in the selection of the C coefficient.

2-9.3.3 Where basic hydrologic data on concurrent rainfall and runoff are adequate to determine unit hydrography for a drainage area, the uncertainties inherent in application of the Rational Method can largely be eliminated. Apparent loss rates determined from unit-hydrograph analyses of recorded floods provide a good basis for estimating loss rates for storms of design magnitude. Also, flow times and storage effects are accounted for in the shape of the unit-hydrograph. Where basic data are inadequate for direct determination of unit-hydrographs, use may be made of empirical methods for synthesis. Use of the unit-hydrograph method is particularly desirable where designs are being developed for ponds, detention reservoirs, and pump stations; where peak runoff from large tributary areas is involved in design; and where large-scale protective works are under consideration. Here, the volume and duration of storm runoff, as opposed to peak flow, may be the principal design criteria for determining the dimensions of hydraulic structures.

2-9.3.4 Procedures for routing storm runoff through reservoir-type storage and through stream channels can be found in publications listed in Appendix B and in the available publications on these subjects.