CHAPTER 3

SURFACE DRAINAGE FACILITIES FOR AIRFIELDS AND HELIPORTS

3-1 **PURPOSE AND SCOPE.** This chapter prescribes standards of design of surface drainage of airfields and heliports. Problems involved in the design of drainage facilities are discussed, and convenient methods of estimating design capacities are outlined. These standards can be altered when necessary to meet special problems or unusual conditions on the basis of good engineering practice. Design of drainage facilities for arctic or subarctic regions is discussed in Chapter 8 (see Appendix A for referenced publications).

3-2 **DESIGN OBJECTIVES FOR AIRFIELD AND HELIPORT SURFACE DRAINAGE.** Surface drainage facilities will be designed to suit the mission and the importance of airfields or heliports; the design capacity will be adequate to accomplish the following objectives:

3-2.1 **Surface Runoff from the Design Storm**. Surface runoff from the selected design storm will be disposed of without damage to the airfield facilities or significant interruption of normal traffic.

3-2.2 **Surface Runoff from Storms Exceeding the Design Storm**. Surface runoff from storm exceeding the design storm will be disposed of with minimum damage to the airfield facilities and with the shortest practicable interruption of normal traffic. The primary runway will remain operational under all conditions.

3-2.3 **Reliability of Operation**. The drainage system will provide maximum practicable reliability of operation under all climatic conditions.

3-2.4 **Maintenance**. The drainage system in the immediate vicinity of operational facilities will require minimum maintenance.

3-2.5 **Coordination**. Basic data obtained during preliminary field investigations will be coordinated with the facility master plan and with other agencies having jurisdiction over conservation, flood control, drainage, and irrigation.

3-2.6 **Safety Requirement**. Separate drainage and containment should be provided in areas with a high potential for fuel spills. This provision will allow spilled fuel to be promptly separated, collected, and removed from the rest of the drainage system.

3-2.7 **Future Expansion**. Drainage design should allow for future expansion with a minimum of expense and traffic interruption.

3-2.8 **Environmental Impact**. Drainage facilities will be constructed with minimal impact on the environment.

3-3 DRAINAGE PROTECTION REQUIRED

3-3.1 **Degree of Drainage Protection**. The degree of drainage protection depends largely on the importance of the airfield or heliport, the mission and volume of traffic to be accommodated, and the necessity for uninterrupted service. Within certain limits the degree of drainage protection should be sufficient so that hazards can be avoided during operation.

3-3.2 **Frequency of the Design Storm**. Drainage for military airfields and heliports will be based on a 2-yr design frequency, unless exceptional circumstances require greater protection. Temporary ponding will be permitted on graded areas adjacent to runway and taxiway aprons, or airfield or heliport pavements other than primary runways. Ponding will not be permitted on primary runways under any condition. To determine the extent of ponding permissible on areas where ponding is allowed, possible damage of pavement subgrades and base courses as a result of occasional flooding must be considered. In addition, ponding basins must conform to grading standards.

3-4 HYDROLOGIC CONSIDERATIONS

3-4.1 **Definitions**. The following definitions are used in the development of hydrologic concepts.

3-4.1.1 **Design frequency**. The average frequency with which the design event, rainfall or runoff, is equaled or exceeded. The reciprocal of frequency is the annual probability of occurrence. Design frequency is selected to afford the degree of protection deemed necessary. Except in special circumstances, the 2-yr frequency, that is, an annual probability of occurrence of 0.5, is considered satisfactory for most airfields.

3-4.1.2 **Design storm**. The standard rainfall intensity-frequency relation, lasting for various durations of supply. The design storm is used to compute the runoff to be carried in drainage facilities.

3-4.1.3 **Rainfall-excess**. The amount of rainfall which appears as surface runoff. Rainfall-excess is rainfall less losses to infiltration or other abstractions.

3-4.1.4 **Standard supply**. The standard intensity-frequency-duration relationship of the selected design storm less losses for infiltration. Standard supply is usually designated by the average rainfall intensity in inches per hour at the 1-hr duration.

3-4.2 **Design Methods**. The design procedures for drainage facilities involve computations to convert the rainfall intensities expected from 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: direct estimates of the proportion of the average rainfall intensity which will appear as the peak rate of runoff and hydrographic methods which account for losses such as infiltration and for the effects of flow over the surface to the point of design. The first approach is exemplified by the "Rational Method," which is used in most engineering offices in the United States. This approach can be used successfully by experienced designers for drainage areas up to 1 square mile in size. ASCE Manual of Practice No. 37 and FAA AC 150/5320-5B explain and illustrate the use of the Rational Method. Chapter 4 presents a modified Rational Method. The second approach includes techniques to synthesize hydrography of runoff. Where studies of large drainage areas or complex conditions of storage require hydrography, the designer should refer to the sources listed in the Bibliography and other publications on these subjects. The method described in Sections 3-5 through 3-9 and developed and illustrated in Section 3-11 and Design Example C-3 combines features from both basic approaches to determine runoff.

3-5 RAINFALL

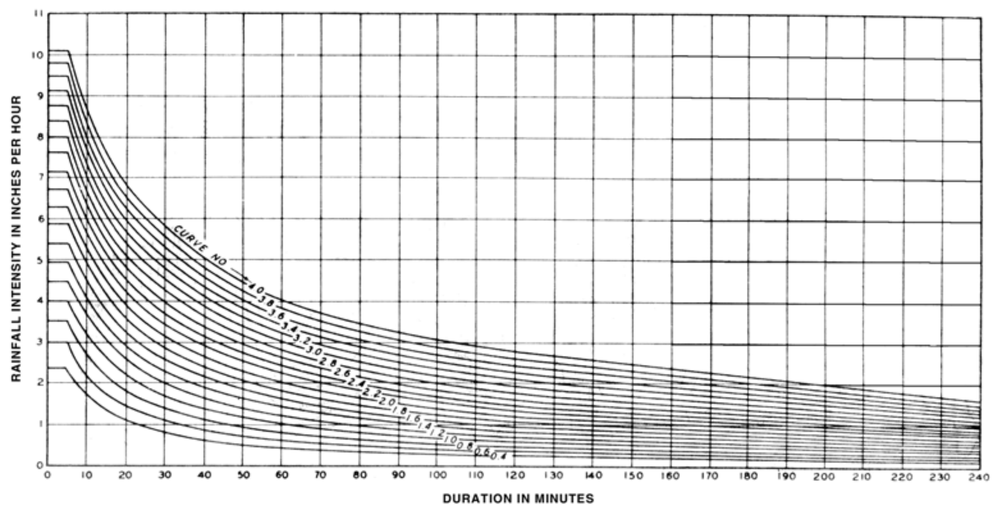
3-5.1 **Intensity-Frequency Data.** Studies of rainfall intensity-frequency data indicate a fairly consistent relation between the average intensities of rainfall for a period of 1 hr and the average intensities at the same frequency for periods less than 1 hr, regardless of the geographical location of the stations. The average rainfall for a 1-hr period at various frequencies for the continental United States, excluding Alaska, may be determined from Figure 2-2. Data for other locations are available from the Office, Chief of Engineers, and the National Oceanic and Atmospheric Administration, National Weather Service (formerly the U.S. Weather Bureau). For Alaska, data may be obtained from Figure 2-1 and U.S. Weather Bureau Technical Paper No. 47. Data for Puerto Rico and the Virgin Islands and for Hawaii may be obtained from U.S. Weather Bureau Technical Papers No. 42 and 43, respectively. For any frequency, the 1-hr rainfall intensity is considered a design-storm index for all average intensities and duration of storms with the same frequency.

3-5.2 **Standard Rainfall Intensity-Duration Curves.** Figure 3-1 shows the standard curves that have been compiled to express the rainfall intensity-duration relationships and the standard supply (infiltration subtracted) which are satisfactory for the design of airfield drainage systems in the continental United States. The curves may be used for all locations until standard curves are developed for any region under consideration. As an example, assume the average rainfall intensity is required for a 40-min design storm based on a 2-yr frequency in central Kentucky. From Figure 2-2 the 2-yr 1-hr rainfall is found to be 1.4 in./hr. In Figure 3-1, supply curve No. 1.4 is used with the 40-min duration of storm to determine a rainfall intensity of 1.9 in./hr.

3-5.3 **Incomplete Data.** In areas where rainfall data are incomplete or unavailable, the methods described in Section 3-11 can be used to develop design rainfall information.

3-5.4 **Design Frequency.** Drainage systems are normally designed for the maximum runoff from rainfall with a certain frequency of occurrence. The design frequency indicates the average frequency at which some portions or all of the drainage system will be taxed to capacity. After the design frequency is selected, computations must be made to determine the critical duration of rainfall necessary to produce the maximum rate of runoff for the specific areas involved. Ordinarily, the maximum rate of runoff occurs when all tributary areas are contributing to the system. However, in cases of odd-shaped areas and areas containing both paved and turfed areas, peak runoff rates may occur before all areas are contributing. Factors affecting the critical duration of rainfall are primarily the length of overland flow, extent of surface detention, ponding, and characteristics of the runoff surfaces.

Figure 3-1. Standard Rainfall Intensity-Duration Curves or Standard Supply Curves



NOTE: CURVE NUMBERS CORRESPOND TO 1-HOUR VALUES OF RAINFALL OR SUPPLY INDICATED BY RESPECTIVE CURVES. ALL POINTS ON THE SAME CURVE ARE ASSUMED TO HAVE THE SAME AVERAGE FREQUENCY OF OCCURRENCE.

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3-5.5 **Storms of Greater Severity than the Design Storm.** The design storm alone is not a completely reliable criterion for the adequacy of drainage facilities. Often storms more severe than the design storm can cause excessive damage and affect operations. Therefore, the probable consequences of storms greater than the design storm should be considered before deciding on the adequacy of facilities designed to handle only the design storm.

3-6 **INFILTRATION.** Infiltration refers to the rate of absorption of rainfall into the ground during a design storm, which is assumed to occur after a 1-hr period of antecedent rainfall. Wherever possible, determine average infiltration rates from a study of runoff records near the airfield from infiltrometer studies or from similar acceptable information. Suggested mean values of infiltration for classifications are shown in Table 2-3. The soil group symbols are those given in generalized soil MIL-STD-619. Infiltration values are for uncompacted soils. Studies indicate that where soils are compacted, infiltration values decrease; the percentage decrease ranges from 25 to 75 percent, depending on the degree of compaction and the types of soil. Vegetation generally decreases infiltration capacity of coarse soils and increases that of clayey soils. The infiltration rate after 1 hr of rainfall for turfed areas is approximately 0.5 in./hr and seldom exceeds 1.0 in./hr. The infiltration rate for paved or roofed areas, blast protective surfaces, and impervious dust-palliative-treated areas is zero.

3-7 **RATE OF SUPPLY.** Rate of supply refers to the difference between the rainfall intensity and the infiltration capacity at the same instant for a particular storm. To simplify computations, the rainfall intensity and the infiltration capacity are assumed to be uniform during any specific storm. Thus the rate of supply during the design storm will also be uniform.

3-7.1 **Average Rate of Supply.** Average rates of supply corresponding to storms of different lengths and the same average frequency of occurrence may be computed by subtracting estimated infiltration capacities from rainfall intensities represented by the selected standard rainfall intensity-duration curve in Figure 3-1. For convenience and since no appreciable error results, standard supply curves are assumed to have the same shapes as those of the standard rainfall intensity-duration curves shown in Figure 3-1. For example, if supply curve No. 2.2 in Figure 3-1 were selected as the design storm and the infiltration loss during a 1-hr storm were estimated as 0.6 in., supply curve No. 1.6 would be adopted as the standard supply curve for the given areas.

3-7.2 **Weighted Standard Rate of Supply Curves.** Drainage areas usually consist of combinations of paved and unpaved areas having different infiltration capacities. A weighted standard supply should be established for the composite drainage areas by weighting the standard supply curve numbers adopted for paved and unpaved surfaces in proportion to their respective tributary area.

3-8 **RUNOFF.** The method of runoff determination described herein is based on an overland flow model. Details are given in Section 3-11.3.

3-8.1 **Overland Flow.** The surface runoff resulting from a uniform rate of supply is termed “overland flow.” If the rate of supply were to continue indefinitely, the runoff would rise to a peak rate and remain constant. Ordinarily, the peak rate is established after all parts of the drainage surface are contributing to runoff. However, in cases of odd-shaped areas and areas containing both paved and turfed areas, peak runoff rates may occur before all areas are contributing. The elapsed time for runoff to build to a peak is termed the “time of concentration,” which depends primarily on the coefficient of roughness, the slope, and the effective length of the surface. When the supply terminates, the runoff rate diminishes, but continues until the excess stored on the surface drains away.

3-8.2 **Effective Length.** The effective length to the point under consideration must account for the effects of overland and channel flow and for the differences in roughness and slope of the drainage surface. Methods for determining effective length are presented in Section 3-11.2.

3-8.3 **Maximum Rate of Runoff.** Figure 2-4 shows the results of overland flow computations using standard supply curves No. 2.0 and 2.2. Curves for other supply rates are given in Figures 3-2 through 3-9). Figure 2-4 depicts the relationships between the rate of supply, σ, in inches per hour; critical duration of supply or time of concentration, t_c; the effective length of overland flow, L; and the resulting maximum rate of runoff. The curves are not complete hydrography for any specific design storm, but are peak rates of runoff from individual storm events of various durations, all having the same frequency of occurrence. Use of the curves can be illustrated by using supply curve No. 2.0, as follows:

3-8.3.1 Assume the effective length of overland flow is 300 ft:

3-8.3.1.1 The critical duration of supply, that is, the time of concentration, to provide maximum runoff is obtained by reading vertically downward from the point where t_c and L = 300 ft curves intersect. This value is found to be 24 min.

3-8.3.1.2 The maximum rate of runoff from overland flow is obtained by reading horizontally across from the point where t_c and L = 300 ft curves intersect. This value is found to be 2.5 in./hr or 2.5 cubic feet per second per acre (cfs/acre).

3-8.3.1.3 The average rate of supply over the area is obtained by reading vertically upward from the point where the t_c and L = 300 ft curve intersect to the σ curve and then reading horizontally across from this point. This value is found to be 3.6 in./hr or 3.6 cfs/acre.

3-8.3.2 Assume the critical duration of supply is 30 min:

3-8.3.2.1 The average rate of supply is obtained by reading horizontally across from the point where the duration of supply = 30 min and σ intersect. This value is found to be 3.2 in./hr or 3.2 cfs/acre.

3-8.3.2.2 The effective length is obtained by reading the point where t_c and the duration of supply = 30 min intersect. This is found to be 500 ft.

3-8.3.2.3 The maximum rate of runoff is obtained by reading horizontally across from this point. This is found to be 2.0 in./hr or 2.0 cfs/acre.

3-9 **STORAGE.** The supply curves in Figure 2-4 assume no surface storage. Where surface storage or ponding is permitted, the overland flow will be stored temporarily and released as the pond drains. The discharge rate from the pond will depend on the volume of storage provided and the extent to which the surface area of the pond reduces the effective length of overland flow. Methods for designing with temporary storage or ponding are given in Section 3-11.4.

Figure 3-2. Rates of Runoff Corresponding to Supply Curves No. 0.4 and 0.6; n = 0.40 and S = 1 percent

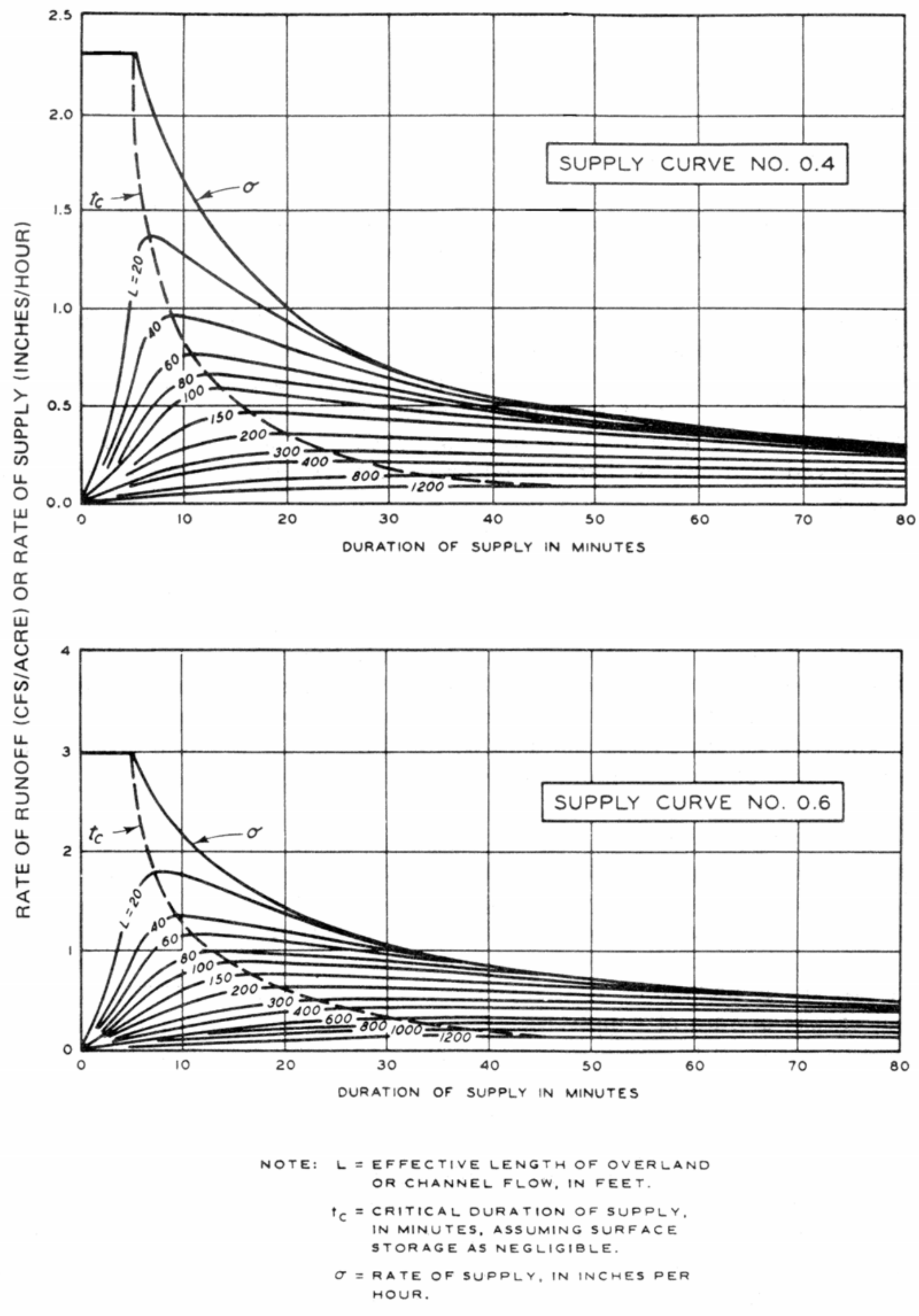

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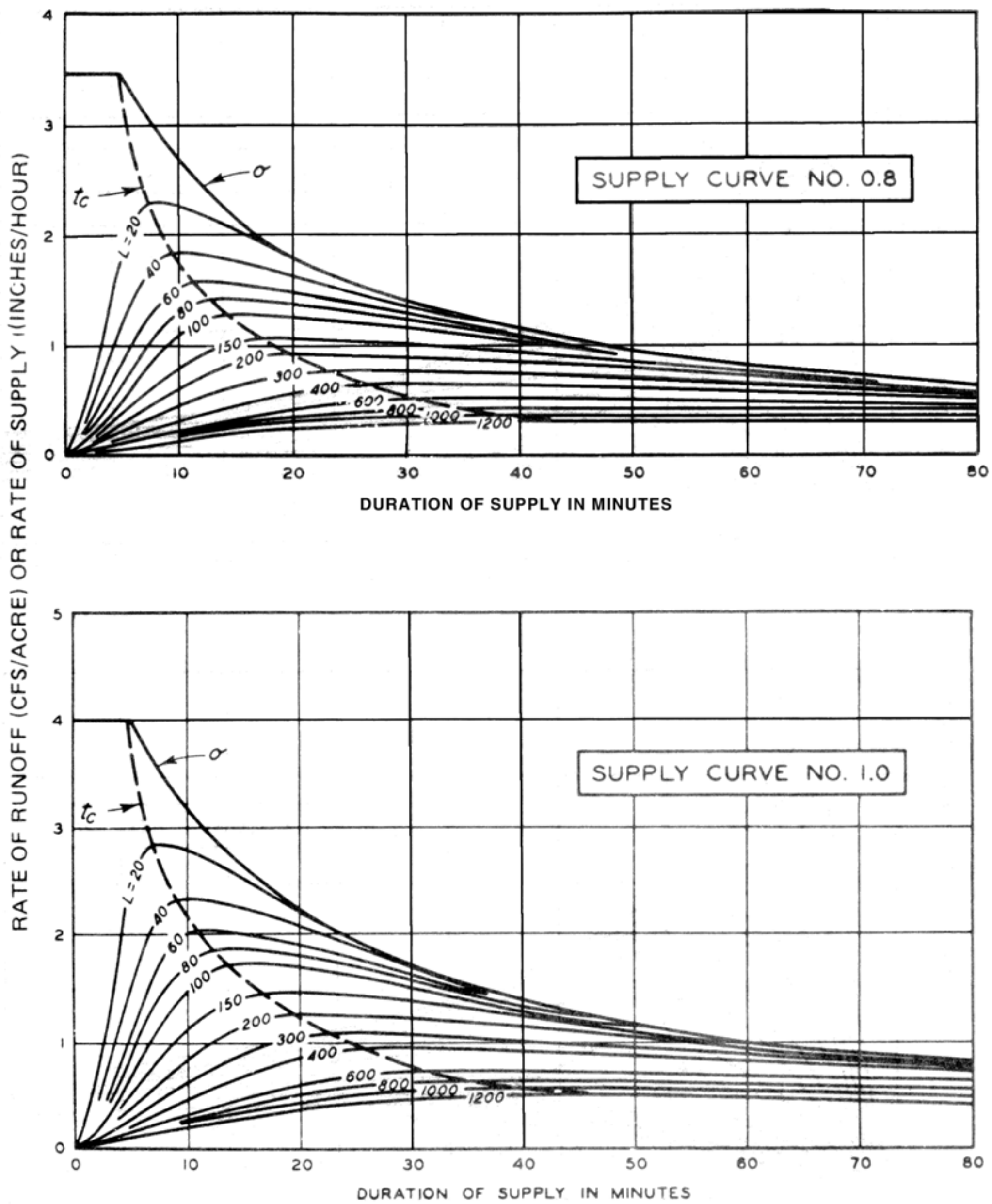
Figure 3-3. Rates of Runoff Corresponding to Supply Curves No. 0.8 and 1.0; n = 0.40 and S = 1 percent



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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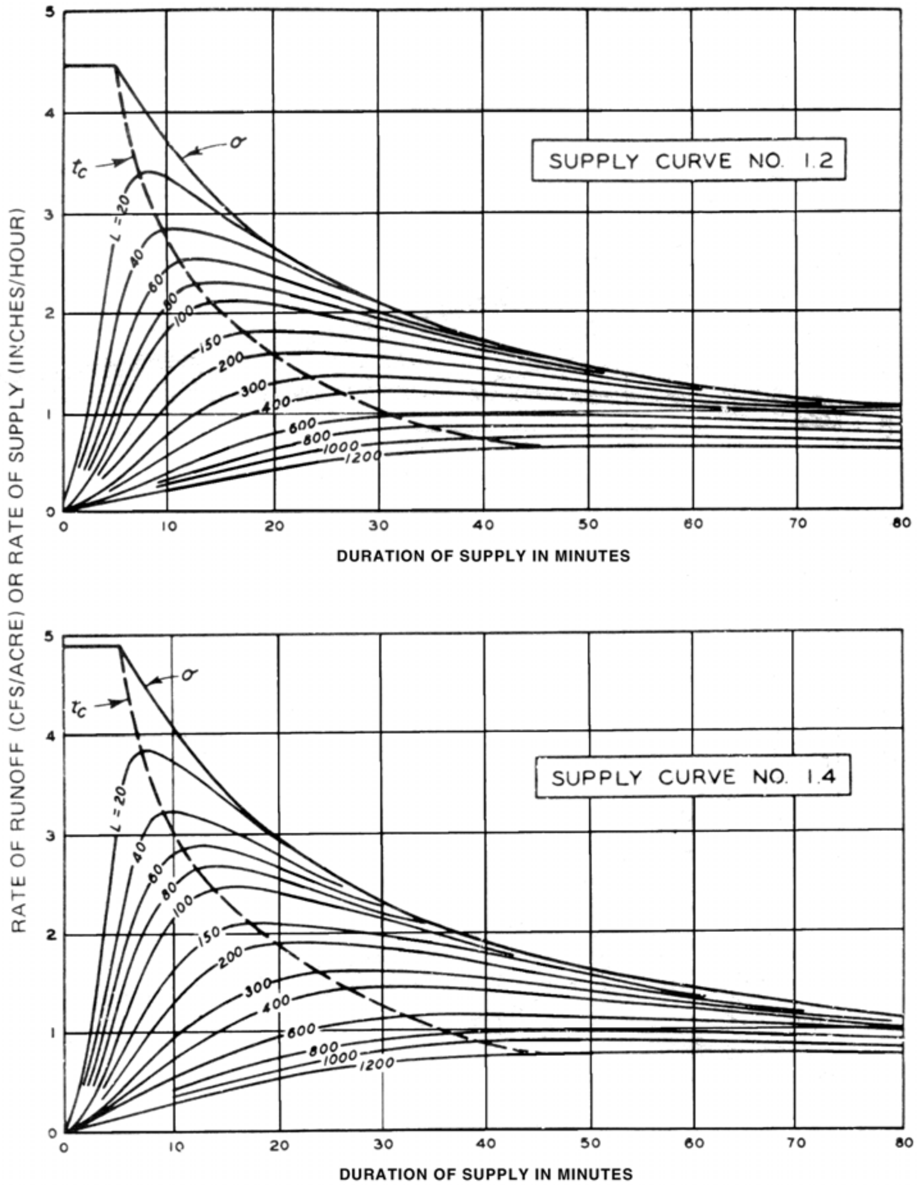
Figure 3-4. Rates of Runoff Corresponding to Supply Curves No. 1.2 and 1.4; n = 0.40 and S = 1 percent



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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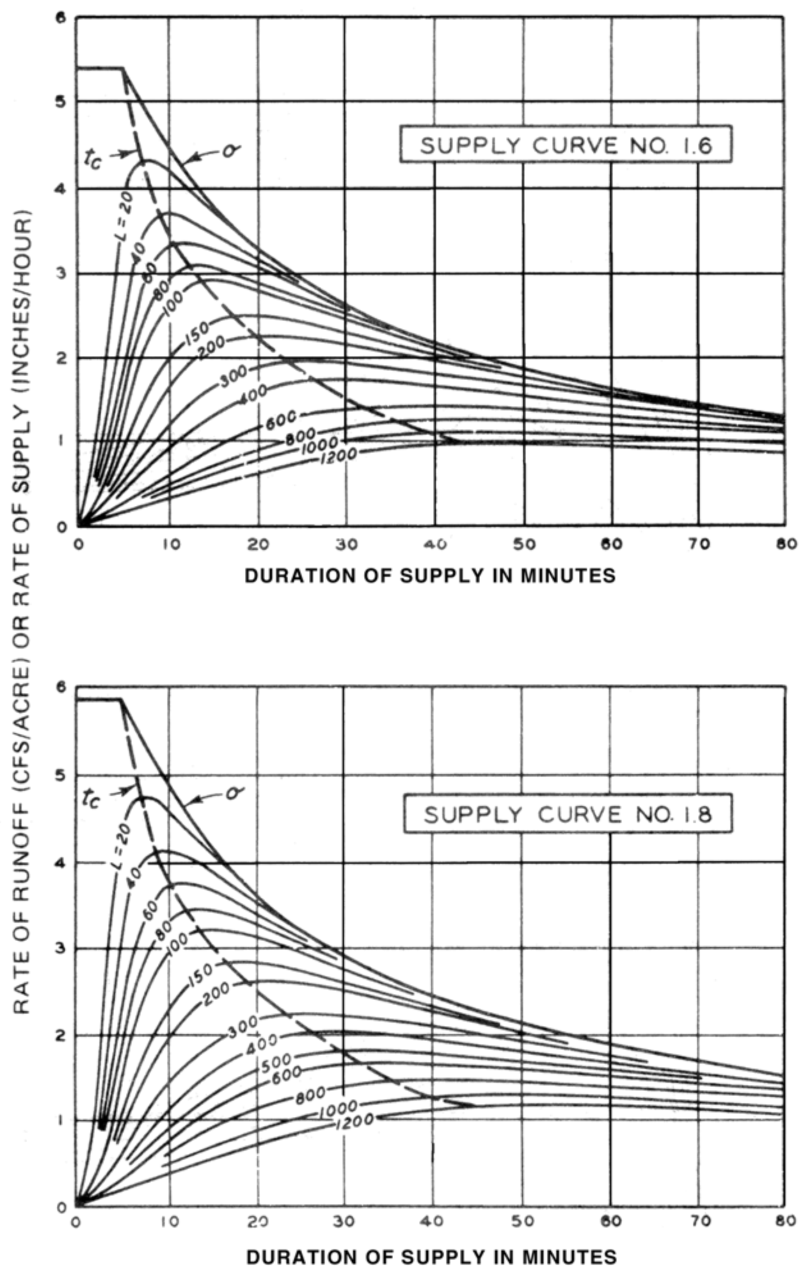
Figure 3-5. Rates of Runoff Corresponding to Supply Curves No. 1.6 and 1.8; n = 0.40 and S = 1 percent



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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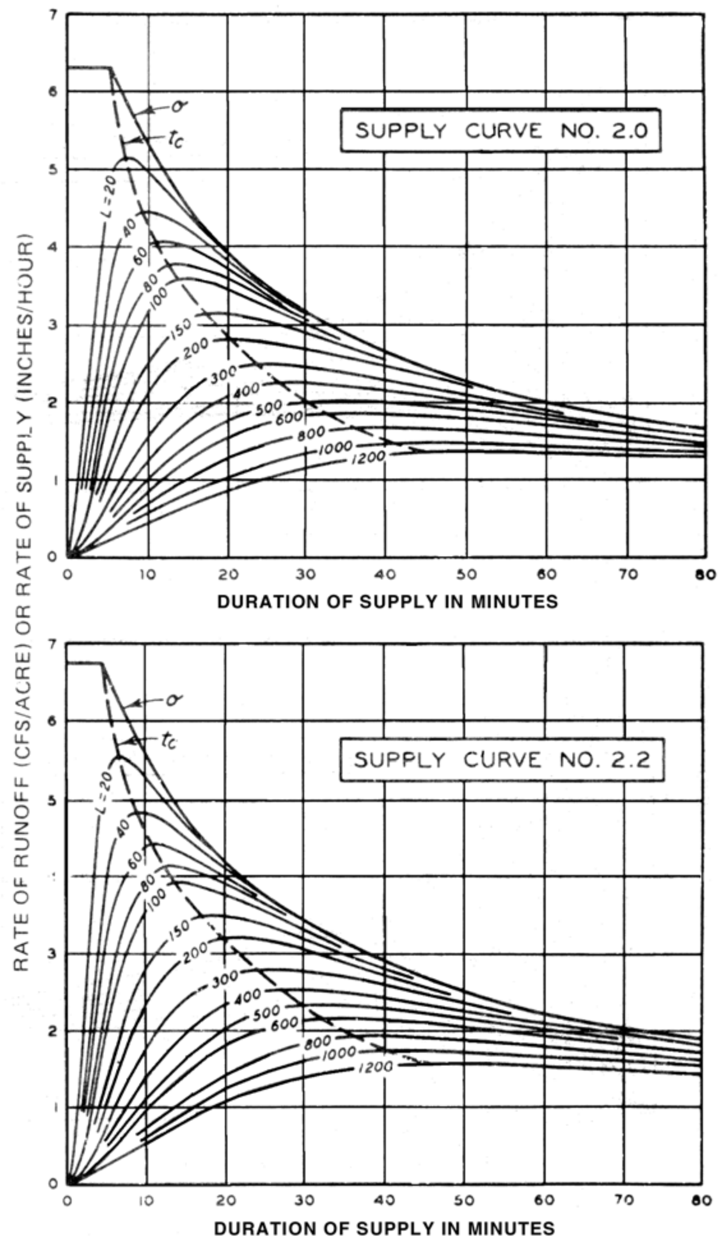
Figure 3-6. Rates of Runoff Corresponding to Supply Curves No. 2.0 and 2.2; n = 0.40 and S = 1 percent



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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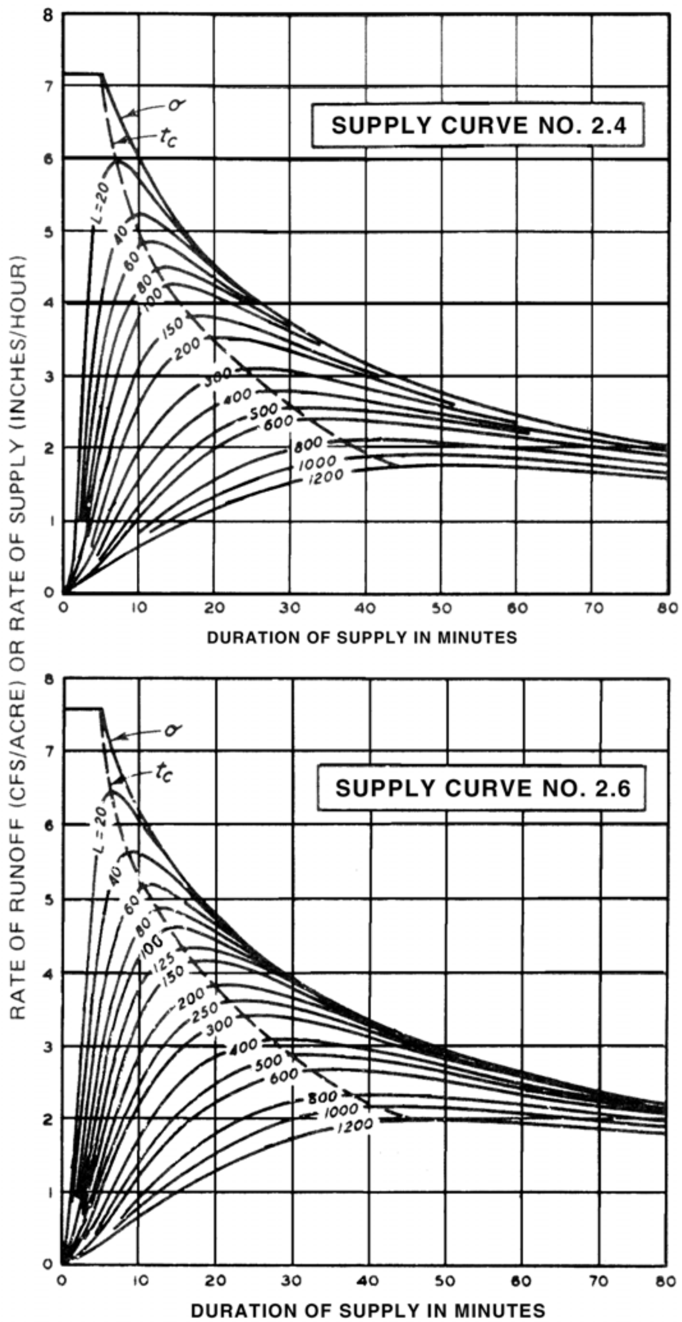
Figure 3-7. Rates of Runoff Corresponding to Supply Curves No. 2.4 and 2.6; n = 0.40 and S = 1 percent



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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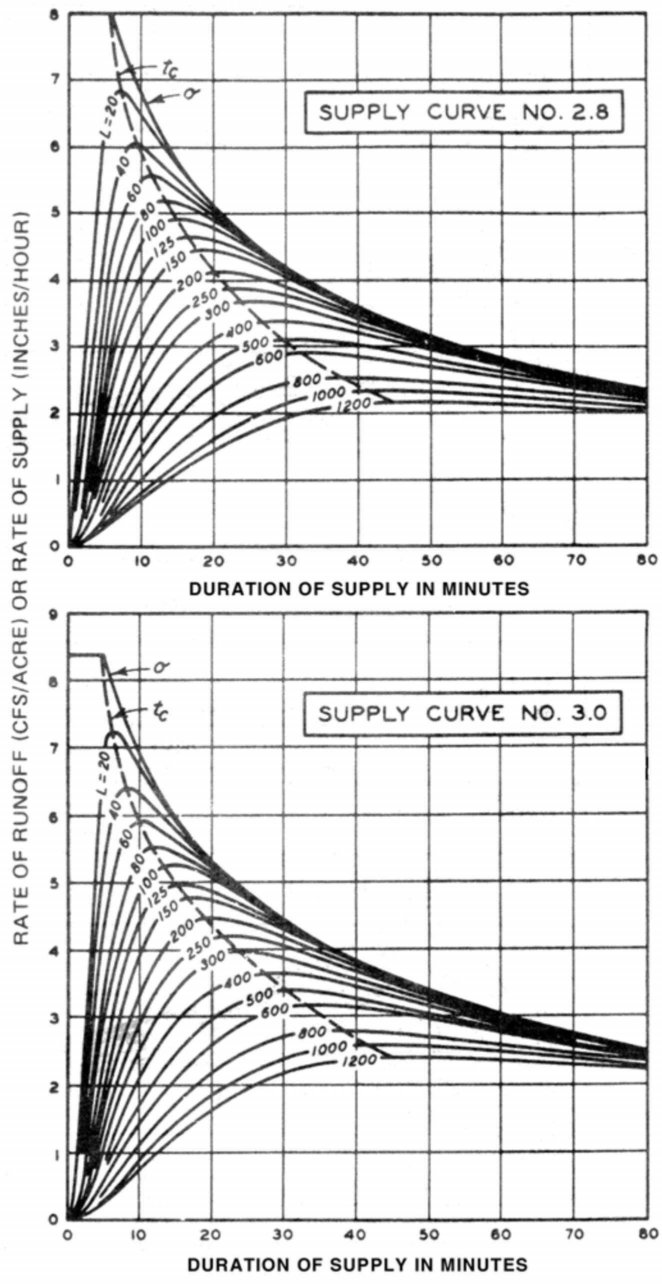
Figure 3-8. Rates of Runoff Corresponding to Supply Curves No. 2.8 and 3.0; n = 0.40 and S = 1 percent



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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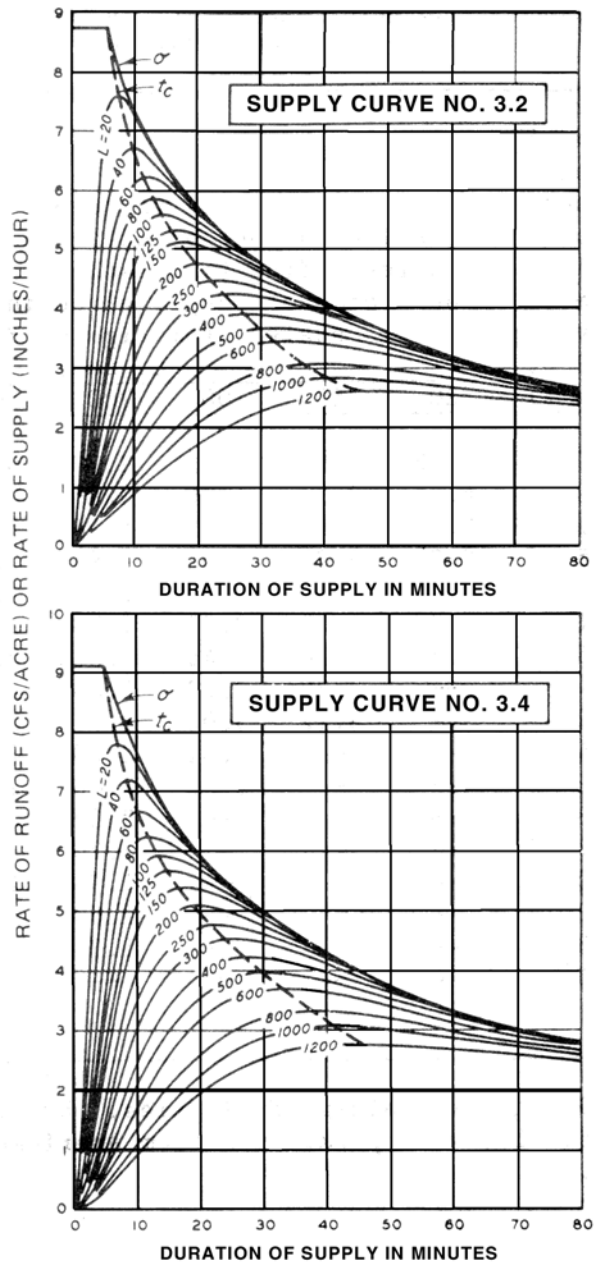
Figure 3-9. Rates of Runoff Corresponding to Supply Curves No. 3.2 and 3.4; n = 0.40 and S = 1 percent



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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3-10 **DESIGN PROCEDURES FOR THE DRAINAGE SYSTEM.** Design-storm runoff must be efficiently removed from airfields and heliports to avoid interruption of operations during or following storms and to prevent temporary or permanent damage to pavement subgrades. Removal is accomplished by a drainage system unique to each airfield and heliport site. Drainage systems will vary in design and extent depending upon local soil conditions and topography; size of the physical facility; vegetation cover or its absence; the anticipated presence or absence of ponding; and most importantly, upon local storm intensity and frequency patterns. The drainage system should function with a minimum of maintenance difficulties and expense and should be adaptable to future expansion. Open channels or natural water courses are permitted only at the periphery of the airfield or heliport facility and must be well removed from the landing strips and traffic areas. Provisions for subsurface drainage, the requirements for which are provided in Chapter 6, may necessitate careful consideration. Subdrains are used to drain the base material, lower the water table, or drain perched water tables. Fluctuations of the water table must be considered in the initial design of the airfield or heliport facility.

3-10.1 **Information Required.** Before proceeding with the design calculations, as illustrated in Section 3-11 and Example C-3, certain additional information and data must be developed. These include:

3-10.1.1 A topographic map.

3-10.1.2 A layout of the helipad, runways, taxiways, aprons, and other hardstands with tentative finished grading contours at 1-ft intervals.

3-10.1.3 Profiles of runways, taxiways, apron areas, and other hardstands.

3-10.1.4 Soil profiles based on soil tests to include, whenever possible, infiltration properties of local soils to be encountered.

3-10.1.5 Groundwater elevation and fluctuation if known or obtainable.

3-10.1.6 A summary of climatic conditions including temperature ranges, freezing and thawing patterns and depth of frost penetration.

3-10.1.7 Snowfall records, snow cover depths, and convertibility factors to inches of rainfall.

3-10.1.8 Runoff records for drainage areas in the same locality having similar characteristics and soil conditions.

3-10.2 **Grading.** Proper grading is the most important single factor contributing to the success of the drainage system. Development of grading and drainage plans must be fully coordinated. Grading criteria in AFR 86-14 for Air Force facilities and TM 5-803–4 for Army airfields and heliports provide adequate grading standards to insure effective drainage.

3-10.2.1 **Minimum slopes.** For satisfactory drainage of airfield pavements, a minimum gradient of 1.5 percent in the direction of drainage is recommended except for rigid pavements where 1.0 percent is adequate. In some cases, gradients less than 1.5 percent are adequate because of existing grades; arid or semiarid climatic conditions; presence of noncohesive, free-draining subgrades; and locations of existing drainage structures. Such factors may allow a lesser transverse slope; thus, construction economies are effected and preferred operational grades are obtained.

3-10.2.2 **Shoulder slopes.** In Attachment 5 of AFR 86-14, transverse grades of shoulders are specified for runways, taxiways, and aprons. In areas of moderate or heavy rainfall or excessive turf encroachment, use of a steeper transition shoulder section immediately adjacent to the airfield pavement is permitted. In designing shoulders, the first 10-ft strip of shoulder adjacent to the pavement edges of runways, taxiways, or aprons should have a 5 percent slope. The elevation of the pavement edge and the shoulder will coincide. The shoulder gradient beyond the 10-ft strip will conform to the minimum 2 percent and maximum 3 percent specified in AFR 86-14. Waivers will not be required for the 5 percent slope discussed above. Paved shoulders will normally have the same transverse slope as that of the contiguous runways and taxiways.

3-10.2.3 **Determination of drainage area.** Use the completed grading plan as a guide and sketch the boundaries of specific drainage areas tributary to their respective drain inlets. Compute the area of paved and unpaved areas tributary to the respective inlets by planimetering.

3-10.2.4 **Drainage patterns.** Drainage patterns consisting of closely spaced interior inlets in pavements with intervening ridges are to be avoided. Such grading may cause taxiing problems including bumping or scraping of wing tanks. Crowned sections are the standard cross sections for runways, taxiways, and safety areas. Crowned sections generally slope each way from the center line of the runway on a transverse grade to the pavement. Although crowned grading patterns result in most economical drainage, adjacent pavements, topographic considerations, or other matters may necessitate other pavement grading.

3-10.3 **Classification of Storm Drains.** Storm drains for airfields and heliports may be classified in two groups, primary and auxiliary.

3-10.3.1 **Primary drains.** Primary drains consist of main drains and laterals that have sufficient capacity to accommodate the project design storm, either with or without supplementary storage in ponding basins above the drain inlets. To lessen construction requirements for drainage facilities, maximum use of ponding consistent with operational and grading requirements will be considered. The location and elevation of the drain inlets are determined in the development of the grading plans.

3-10.3.2 **Auxiliary drains.** Auxiliary drains normally consist of any type or size drains provided to facilitate the removal of storm runoff, but lacking sufficient capacity to remove the project design storm without excessive flooding or overflow. Auxiliary storm drains may be used in certain airfields to provide positive drainage of long flat swales located adjacent to runways or in unpaved adjacent areas. During less frequent storms of high intensity, excess runoff should flow overland to the primary drain system or other suitable outlet with a minimum of erosion. An auxiliary drain may also be installed to convey runoff from pavement gutters wherever a gutter capacity of less than design discharge is provided.

3-10.4 **Storm-Drain Layout.** The principal procedures in the determination of the storm-drain layout follow:

3-10.4.1 **Preliminary layout.** Prepare a preliminary map (scale 1 in. = 200 ft or larger) showing the outlines of runways, taxiways, and parking aprons. Contours should represent approximately the finished grade for the airfield or heliport. Details of grading, including ponding basins around primary drain inlets, need not be shown more accurately than with 1-ft contour intervals.

3-10.4.2 **Profiles.** Plot profiles of all runways, taxiways, and aprons so that elevations controlling the grading of intermediate areas may be determined readily at any point.

3-10.4.3 **Drain outlets.** Consider the limiting grade elevations and feasible channels for the collection and disposition of the storm runoff. Select the most suitable locations for outlets of drains serving various portions of the field. Then select a tentative layout for primary storm drains. The most economical and most efficient design is generally obtained by maintaining the steepest hydraulic gradient attainable in the main drain and maintaining approximately equal lateral length on each side of the main drain.

3-10.4.4 **Cross-sectional profiles of intermediate areas.** Assume the location of cross-sectional profiles of intermediate areas. Plot data showing controlling elevations and indicate the tentatively selected locations for inlets by means of vertical lines. Projections of the runways, taxiways, or aprons for limited distances should be shown on the profiles, to facilitate a comparison of the elevations of intermediate areas with those of the paved areas. Generally, one cross-sectional profile should follow each line of the underground storm-drain system. Other profiles should pass through each of the inlets at approximately right angles to paved runways, taxiways, or aprons.

3-10.4.5 **Correlation of the controlling elevations and limiting grades.** Begin at points corresponding to the controlling elevations, such as the edges of runways, and sketch the ground profile from the given points to the respective drain inlets. Make the grades conform to the limiting slopes. Review the tentative grading and inlet elevations and make such adjustments in the locations of drain inlets and in grading details as necessary to obtain the most satisfactory general plan.

3-10.4.6 **Trial drainage layouts.** Several trial drainage layouts will be necessary before the most economical system can be selected. The first consideration will be the tentative layout serving all of the depressed areas in which overland flow will accumulate. The inlet structures will be located, during the initial step, at the lowest points within the field areas. The pipelines will be shown next. Each of the inlet structures will be connected to the field pipelines, which in turn will be connected to the major outfalls.

3-10.4.7 **Rechecking of finished contours.** Before proceeding further, recheck the finished contours to determine whether the surface flow is away from the paved areas, that the flow is not directed across them, that no field structures fall within the paved areas (except in aprons), that possible ponding areas are not adjacent to pavement edges, and that surface water will not have to travel excessively long distances to flow into the inlets. If there is a long, gradually sloping swale between a runway and its parallel taxiway (in which the longitudinal grade, for instance, is all in one direction), additional inlets should be placed at regular intervals down this swale. Should this be required, ridges may be provided to protect the area around the inlet, prevent bypassing, and facilitate the entry of the water into the structure. If the ridge area is within the runway safety area, the grades and grade changes will need to conform to the limitations established for runway safety areas in other pertinent publications.

3.10.4.8 **Maximum ponding area and volume.** Estimate the maximum elevation of storage permissible in the various ponding areas and indicate the elevations on the profiles referred to in (4) and (5) above. Scale the distances from the respective drain inlets to the point where the elevation of maximum permissible ponding intersects the ground line, transfer the scaled distances to the map prepared in (1) above, and sketch a line through the plotted points to represent the boundary of the maximum ponding area during the design storm. Determine the area within the various ponding areas and compute the volume of permissible storage at the respective drain inlets. All ponding area edges will be kept at least 75 ft from the edges of the pavement to prevent saturation of the base or subbase and of the ground adjacent to the pavement during periods of ponding.

3-10.4.9 **Ditches.** A system of extensive peripheral ditches may become an integral part of the drainage system. Ditch size and function are variable. Some ditches carry the outfall away from the pipe system and drainage areas into the natural drainage channels or into existing water courses. Others receive outfall flow from the airport site or adjacent terrain. Open ditches are subject to erosion if their gradients are steep and if the volume of flow is large. When necessary, the ditches may be turfed, sodded, stabilized, or lined to control erosion.

3-10.4.10 **Study of the contiguous areas.** After the storm drain system has been tentatively laid out and before the actual computations have been started, the areas contiguous to the graded portion of the airport which may contribute surface flow upon it should again be studied. A system of open channels, intercepting ditches, or storm drains should be designed where necessary to intercept this storm flow and conduct it away from the airport to convenient outfalls. A study of the soil profiles will assist in locating porous strata which may be conducting subsurface water into the airport. If this condition exists, the subsurface water should be intercepted and diverted.

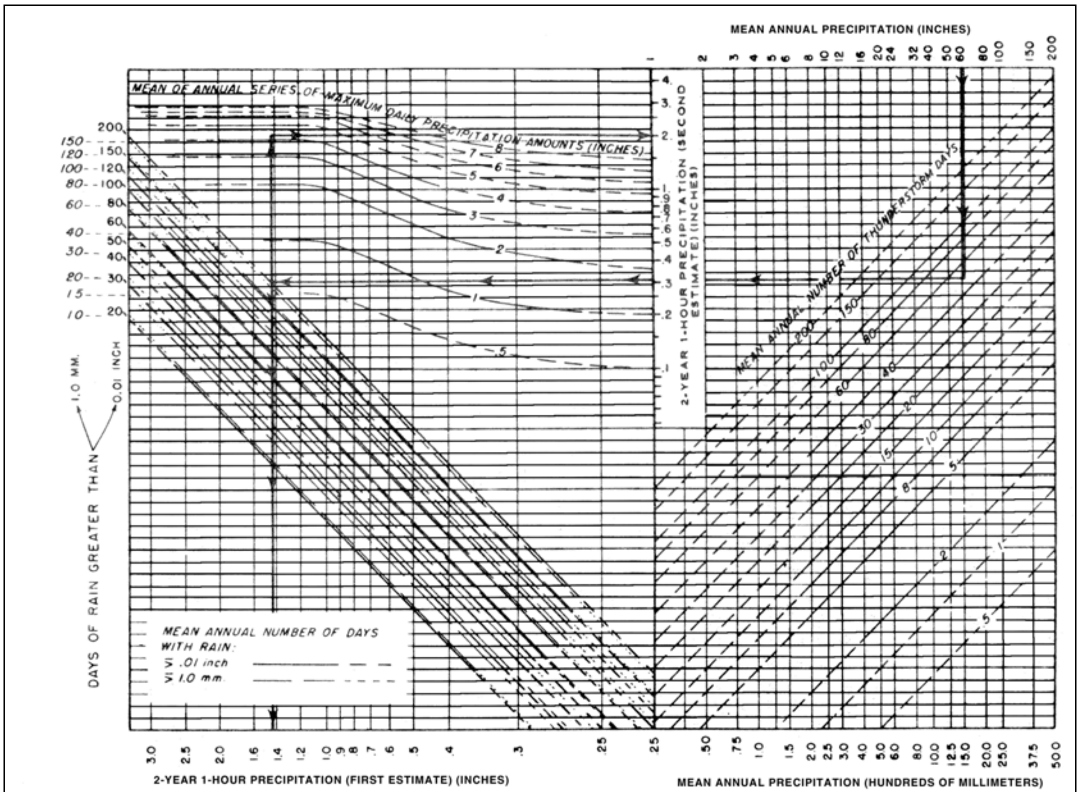
3-10.5 **Typical Design Procedures.** The procedures in Sections 3-2 through 3-10 are illustrated and annotated in the design computations contained in Example C-3. Comparative designs with and without provisions for temporary ponding have been prepared for the airfield shown.

3-11 DESIGN PROCEDURE

3-11.1 Rainfall

3-11.1.1 **Intensity-frequency data.** In areas where intensity-frequency data are incomplete or unavailable, the 2-yr 1-hr rainfall can be estimated from the following parameters: mean annual precipitation—the average of total yearly rainfall for a specified number of years; mean annual number of days of precipitation–the average number of days for a specified number of years in which greater than 0.01 in. of rain occurred; mean annual thunderstorm days—the average number of days for a specified number of years in which thunder was heard; and the mean of the annual maximum observational-day rainfall amounts—the average of the maximum rainfall on any calendar day within the year for a specified number of years. Correlation of the 2-yr 1-hr rainfall with these four climatic parameters appears in Figure 3-10.

Figure 3-10. Diagram for Estimating 2-yr 1-hr Rainfall



NOTE: DASHED LINES ARE UNSUPPORTED BY DATA.

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3-11.1.1.1 When daily rainfall data are not available, the 2-yr 1-hr value can be estimated using the other three parameters, namely, mean annual precipitation, mean annual number of precipitation days, and mean annual number of thunderstorm days. Three parameters are not as accurate as four, and the diagram should be supplemented wherever possible by correlation with other data.

3-11.1.1.2 As an example of the use of Figure 3-10, assume the mean annual precipitation is 60 in., the mean annual number of thunderstorm days is 50, and the mean annual number of precipitation days is 200. Enter the diagram at the upper right with the mean annual precipitation; proceed vertically down to the mean annual number of thunderstorm days; move horizontally to the left to the number of days of precipitation, and then vertically downward to the 2-yr 1-hr precipitation value (first estimate). In this example, the first estimate for the 2-yr 1-hr precipitation is approximately 1.4 in. Now assume the fourth parameter, the mean of annual series of maximum daily precipitation, is 4.3 in. The same procedure is followed to the mean annual days of precipitation; from there, move vertically upward to the mean of annual series of maximum daily precipitation value and then horizontally to the right to the 2-yr 1-hr precipitation value (second estimate). In this example, the second estimate would be 2.0 in. The second estimate is preferable, if four parameters are available.

3-11.1.1.3 For frequencies other than 2 years, the factors in Table 3-1 can be used to approximate intensity-frequency values, using the 2-yr 1-hr value as a base.

3-11.1.2 **Standard rate of supply curves.** Standard supply curves for areas with zero infiltration loss will be the same as the standard rainfall intensity curves in Figure 3-1. Where infiltration losses occur, the standard supply curve number corresponding to a given standard rainfall curve number is computed by subtracting the estimated 1-hr infiltration value from the 1-hr rainfall quantity.

Table 3-1. Approximate Intensity-Frequency Values

Factor	Intensity-Frequency Values
0.80	1-year 1-hour
1.00	2-year 1-hour
1.35	5-year 1-hour
1.60	10-year 1-hour
1.90	25-year 1-hour
2.10	50-year 1-hour
2.30	100-year 1-hour

3-11.1.3 **Weighted standard rates of supply.** For composite areas, the rate of supply should be the average weighted supply. Mathematically, the weighted supply curve, SC_W, can be expressed by the equation:

$$SC_W = \frac{\left[\left(SC_1 \times A_1\right) + \left(SC_2 \times A_2\right) + \ldots \left(SC_n \times A_n\right)\right]}{A_1 + A_2 + \ldots + A_n} \qquad \text{(eq. 3-1)}$$

where the SC's are standard supply rates for the various areas, A. For example, if the drainage area under consideration has a 1-hr rainfall intensity of 2.5 in.; estimated infiltration values of 0.0 for paved area A_1, 0.6 for turfed area A_2, and 0.2 for bare clay area A_3; and drainage area A_l is 1.5 acres, A_2 is 5.0 acres, and A_3 is 6.5 acres; then the weighted standard supply curve for the composite drainage area would be:

$$SC_W = \frac{(2.5-0.0)(1.5)+(2.5-0.6)(5.0)+(2.5-0.2)(6.5)}{1.5+5.0+6.5}$$

$$SC_W = 2.2$$

3-11.1.4 **Overland flow.** The rate of overland flow to be expected from a continuous and uniform rate of rainfall excess, or rate of supply, can be determined from Equation 3-2 as interpreted by G. A. Hathaway (American Society of Civil Engineers, Transactions, Vol. 110):

$$q = \sigma \tanh^2 \left[0.922t\,(\sigma /nL)^{0.50}\, S^{0.25}\right] \qquad \text{(eq. 3-2)}$$

where

q	=	rate of overland flow at the lower end of an elemental strip, in./hr or cfs/acre
Φ	=	rate of supply or intensity of rainfall excess, in./hr
t	=	time, or duration, from beginning of supply, min
n	=	coefficient of roughness of the surface
L	=	effective length of overland, or channel flow, ft
S	=	slope of the surface (absolute, that is, 1 percent = 0.01)
tanh	=	hyperbolic tangent

3-11.1.4.1 The curves shown in Figures 3-11 through 3-13 were computed using Equation 3-2, assuming n = 0.40 and S = 0.01. The overland flow curves are the hydrography that would result from continuous and uniform rates of rainfall-excess or rates of supply. From the curves, hydrography can be developed for any selected duration and rate of rainfall-excess by the procedure shown in Figure 3-14. Hydrography 1 and 1-A in Figure 3-14 represent rates of runoff under given conditions assuming supply continues indefinitely. However, by lagging the hydrography for a selected period of rainfall-excess, t_r (20 min in this example), and subtracting runoff in hydrography 1-A from hydrography 1, a hydrography can be obtained that represents the runoff pattern for the selected period of rainfall-excess (hydrography 2 in the example).

Figure 3-11. Rates of Runoff Versus Duration of Supply for Turfed Areas; L = 20, 40, 60, 80, 100, and 125 ft

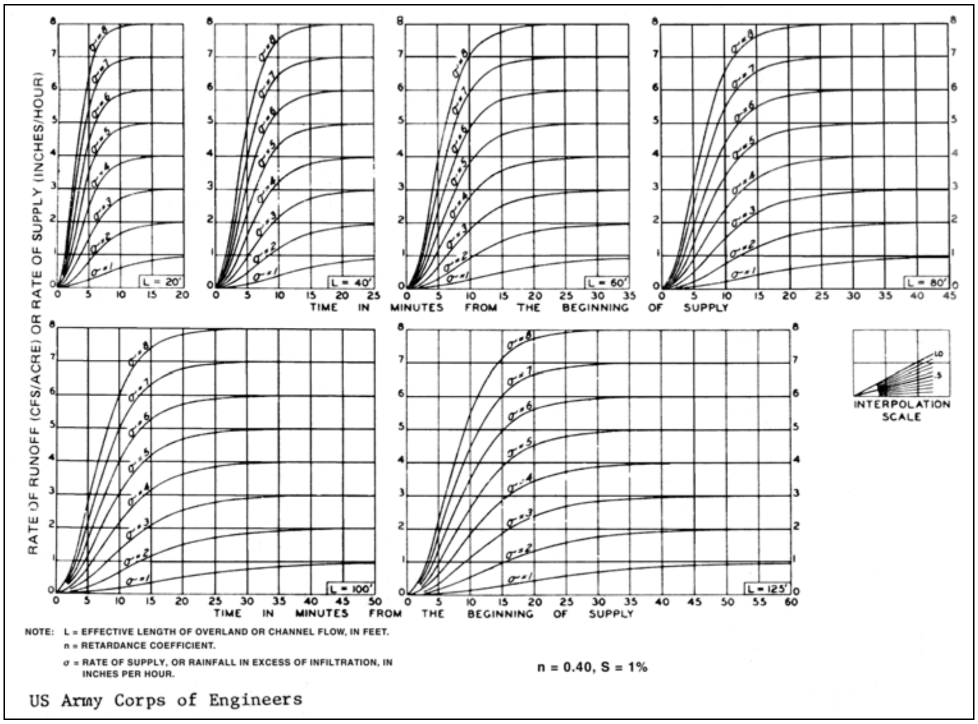


Figure 3-12. Rates of Runoff Versus Duration of Supply for Turfed Areas; L = 150, 200, 250, 300, 400, 500, and 600 ft

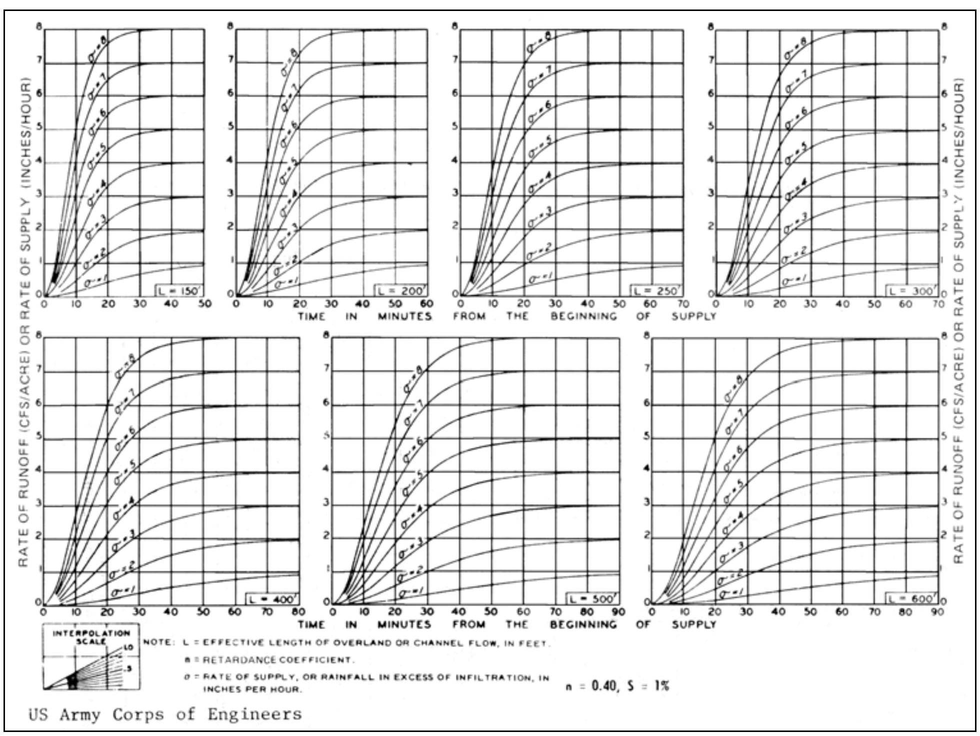
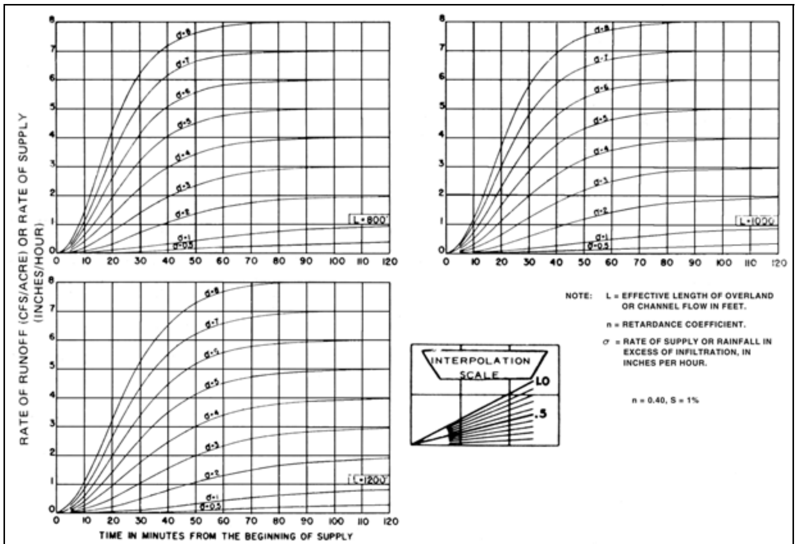
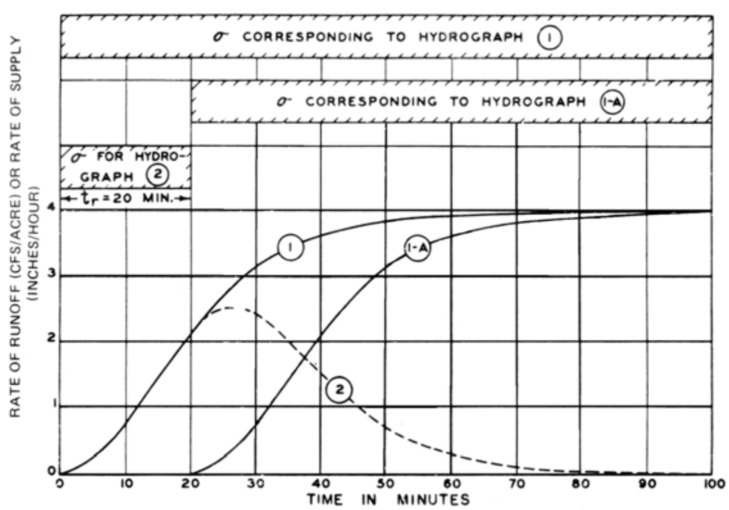


Figure 3-13. Rates of Runoff Versus Duration of Supply for Turfed Areas; L = 800, 1,000, and 1,200 ft



US Army Corps of Engineers

Figure 3-14. Computation of Hydrograph to Represent Runoff from a Supply of Specified Duration



EXPLANATION

EXAMPLE: L = 400 FT.; S = 1%; n = 0.40; σ = 4 IN. PER HR.; t_r = 20 MIN.

HYDROGRAPH 1 REPRESENTS RATE OF RUNOFF UNDER GIVEN CONDITIONS ASSUMING SUPPLY BEGINS AT TIME ZERO AND CONTINUES INDEFINITELY (SEE FIG. 3-12).

HYDROGRAPH 1-A IS IDENTICAL WITH HYDROGRAPH 1 EXCEPT THAT SUPPLY AND RUNOFF ARE ASSUMED TO BEGIN 20 MIN. LATER THAN HYDROGRAPH 1.

HYDROGRAPH 2, OBTAINED BY SUBTRACTING ORDINATES OF HYDROGRAPH 1-A FROM HYDROGRAPH 1, REPRESENTS APPROXIMATELY THE RUNOFF TO BE EXPECTED FROM A SUPPLY RATE OF 4 IN. PER HR. AND A DURATION OF 20 MIN.

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3-11.1.4.2 Overland flow curves may be used for surfaces having other coefficients of roughness or slopes by using, instead of actual length of the flow involved, a hypothetical length that is greater or less than the actual by a sufficient amount to compensate for the difference between the correct values of n and S and those used in preparing Figures 3-11 through 3-13. The necessary conversions to get an effective length may be accomplished by substituting the quantity nL $\sqrt{4}\,\sqrt{S}$ for L or by using Figure 3-4 as explained below (Effective Length).

3-11.2 **Effective Length**

3-11.2.1 **General.** In Equation 3-2, the effective length, L, represents the length of overland flow, measured in a direction parallel to the maximum slope, from the edge of the drainage area to a point where runoff has reached a defined channel or pOnding basin. In large drainage areas, considerable channelized flow will occur under design-storm conditions. Investigation of many runoff records for watersheds has indicated that by modifying the actual length, satisfactory reproduction of runoff hydrography may be obtained regardless of channelization of flow. The values for L are determined by summing the length of channel flow and the length of overland flow after each has been reduced to an effective 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 for that section in which appreciable depth of flow may reasonably be expected to occur during the design storm. Length of overland flow is the average distance from the end of the effective channel or from the drain inlet to the edge of the drainage area, measured in the direction of flow as indicated on the proposed grading plans. Airfield and heliport grading is such that overland flow will normally channelize in distances of 600 feet or less, although this distance may be exceeded. Whenever the distance is exceeded, the actual length may be divided by a number so that the quotient conveniently falls on the horizontal axis of graph A on Figure 2-5. The length derived from graph B on the figure would then be multiplied by this same number to determine the final effective length. Typical values of the coefficient of roughness, n, for use in determining effective length of overland flow are given in Table 2-1. Chapter 4 gives additional n values for turfed channels. For example, to find the effective length of overland flow for an actual length of 900 ft on a sparse grass ground cover where n = 0.20, and the overall slope is 0.7 percent, use the following procedure. Divide the 900-ft actual length by 2 and enter graph A of Figure 2-5 with 450 ft on the horizontal axis. Project a line vertically upward until it intersects the coefficient of roughness line; proceed horizontally to the intersection of the slope line equal to 0.7 percent on graph B, and proceed vertically down to obtain a length of 275 ft, which must be multiplied by 2, resulting in a total effective length of overland flow of 550 ft.

3-11.2.2 **Effect of paved area on determination of effective length.** Ponding areas are frequently located in intermediate turfed areas bordered by paved runways, taxiways, or aprons. Runoff from paved areas ultimately passes over turfed slopes to reach the ponding areas and drain inlets, and is retarded in a manner similar to runoff that results from precipitation falling directly on the turfed area. Inasmuch as the time required for water to flow from the average paved area is normally very short (5 to

10 min), the length of the paved area can be disregarded or given very little weight in estimating the value of L for a composite area.

3-11.2.3 **Determination of effective length for ponding conditions.** The true value of L applicable to a particular area varies as the size of the storage pond fluctuates during storm runoff. As water accumulates in the relatively flat storage area during storm runoff, the size of the pond increases rapidly and progressively reduces the distance from the edge of the pond to the outer limits of the drainage area. In the majority of cases, it is satisfactory to estimate the value of L as the distance from the outer limits of the drainage area to the average limits of the ponding area during the period of design-storm runoff. If the drain inlet is not located near the centroid of the drainage area, the value of L can be estimated approximately as the average distance to the limit of the ponding area, which corresponds to a depth equal to two-thirds of the maximum depth caused by the design storm.

3-11.3 **Runoff**

3-11.3.1 **General.** The curves shown in Figures 2-5 and 3-2 through 3-9 describe the relationship between rate of supply, Φ; critical duration of supply, t_c; effective length of overload flow, L; and maximum rate of runoff for the various supply curves presented in Figure 3-1. The curves portray the data presented in the flow curves shown in Figure 3-11 through 3-13 in another format. Table 3-2 illustrates the computational procedure. The runoff values obtained are assumed to be the maximum because surface storage is negligible. Actually, the maximum runoff would normally occur a short time after the rainfall excess or rate of supply ceases. For practical purposes, however, the maximum rate of overland flow can be assumed to occur at approximately the same time that the rate of supply ends.

3-11.3.2 **Peak runoff rates.** Figures 2-5 and 3-2 through 3-9 are not hydrography for any specified 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 these figures is defined as the critical duration of supply, t_c, for runoff from an area not affected by surface ponding. However, experience indicates that adopting minimum values for t_c of 10 min for paved areas and 20 min for turfed areas in the actual design of storm drains is feasible and practical. For combined turfed and paved areas, minimum values of t_c are to be used even though the calculated effective length of overland flow indicates a shorter critical duration of supply. For combined turfed and paved areas, where only the minimum values of t_c are of concern, the following equation should be used in selecting t_c:

$$t_c = (10A_p + 20A_t)/(A_p + A_t) \qquad \text{(eq 3-3)}$$

where

A_p = area paved, acres

A_t = area turfed, acres

Table 3-2. Rates of Runoff Corresponding to Intensities and Durations of Supply Represented by Standard Supply Curve No. 2 in Figure 3-11 (n = 0.40, S = 1 percent)

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
Duration of supply minutes	Rate of supply in./hr (scaled from curve No. 2.0, Figure 3-1)	Rate of runoff (in in./hr or cfs/acre) corresponding to durations shown in column 1 and rates of supply given in column 2 as scaled from Figures B-2 and B-3 L, feet						
		20	60	100	200	300	400	600
3	6.30	2.68	1.12	0.75	0.39	0.25	0.22	0.13
5	6.30	4.74	2.59	1.76	0.96	0.64	0.52	0.33
7	5.81	5.16	3.41	2.55	1.54	1.12	0.83	0.58
9	5.35	5.06	3.84	3.02	1.94	1.42	1.10	0.76
12	4.83	4.75	4.07	3.43	2.41	1.80	1.49	1.02
15	4.41	4.39	4.02	3.59	2.70	2.12	1.76	1.26
20	3.85	3.85	3.70	3.46	2.86	2.39	2.05	1.55
25	3.44	--	3.38	3.27	2.85	2.49	2.20	1.73
30	3.12	--	3.12	3.02	2.77	2.49	2.25	1.85
35	2.84	--	--	2.81	2.60	2.39	2.20	1.86
40	2.62	--	--	2.62	2.48	2.32	2.15	1.86
45	2.43	--	--	--	2.32	2.21	2.09	1.86
50	2.27	--	--	--	2.20	2.11	2.00	1.82
60	2.00	--	--	--	1.96	1.92	1.86	1.72
80	1.62	--	--	--	1.60	1.59	1.56	1.50
100	1.38	--	--	--	1.38	1.35	1.33	1.28
120	1.16	--	--	--	--	1.16	1.16	1.12
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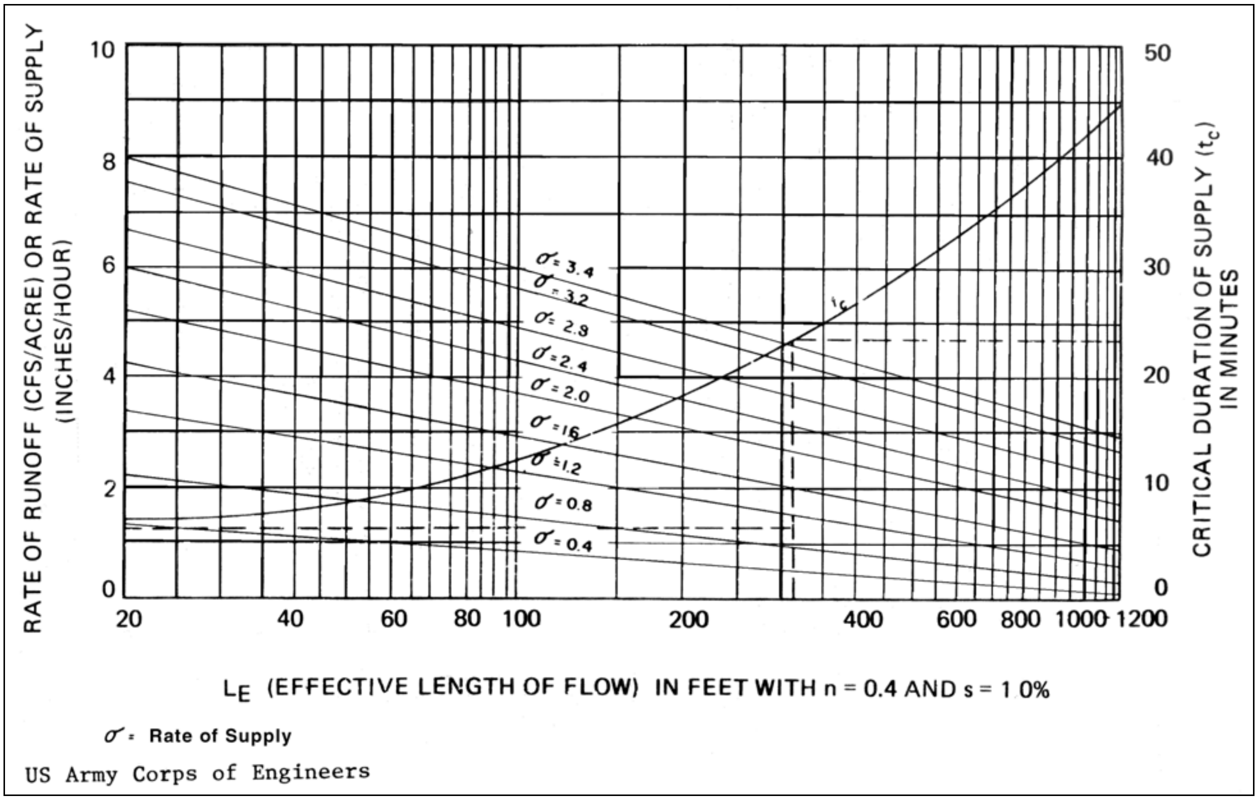
3-11.3.3 **Consolidated design curve.** The data presented in Figures 2-5 and 3-2 through 3-9 with respect to peak runoff rates and critical durations of supply have been consolidated into one diagram, Figure 3-15. Use of Figure 3-15 is not as precise as using Figures 2-5 and 3-2 through 3-9, but Figure 3-158 may be applied to most drainage problems. The following example is provided to illustrate the use of Figure 3-15. Assume an effective length of overland flow of 315 ft and a rate of supply of 1.0 in./hr. To determine the critical duration of supply, project a line vertically upward from the effective length to the intersection of the t_c curve and proceed horizontally to the right to the critical duration of supply which, in this example, is 23 min. To determine the maximum rate of runoff, proceed vertically upward from the effective length to the intersection of the rate of supply line and proceed horizontally to the left to the maximum rate of runoff, which is 1.2 cfs/acre of drainage area.

Figure 3-15. Consolidated Design Curve Composite of Peak Runoff Rates and Critical Durations of Supply Shown in Figures 3-2 through 3-9



3-11.4 **Storage**

3-11.4.1 **Temporary storage or ponding.** If the rate of outflow from a drainage area is limited by the capacity of the drain serving the area, runoff rates exceeding the drain capacity must be stored temporarily. As soon as the rate of inflow into the ponding basin becomes less than the drain capacity, the accumulated storage may be drawn off at a rate equal to the difference between the drain capacity and the rate of inflow into the basin. The general relation between inflow, storage, and outflow is expressed as: outflow = inflow ∀ storage.

3-11.4.1.1 The rate of outflow from a ponding basin is affected by the elevation of the water surface at the drain inlet serving the area. The rate of outflow increases as the head on the inlet increases. However, because of the flat slopes of airfield areas, the surfaces of the storage ponds surrounding drain inlets are usually very large in comparison to the depth of water at the inlets. The rate of outflow through a particular drain inlet would be approximately constant as long as the rate of runoff and accumulated storage are sufficient to maintain the full discharge capacity of the drain inlet. The rate of outflow equals the rate of inflow into the pond until the full discharge capacity of the drain inlet is attained.

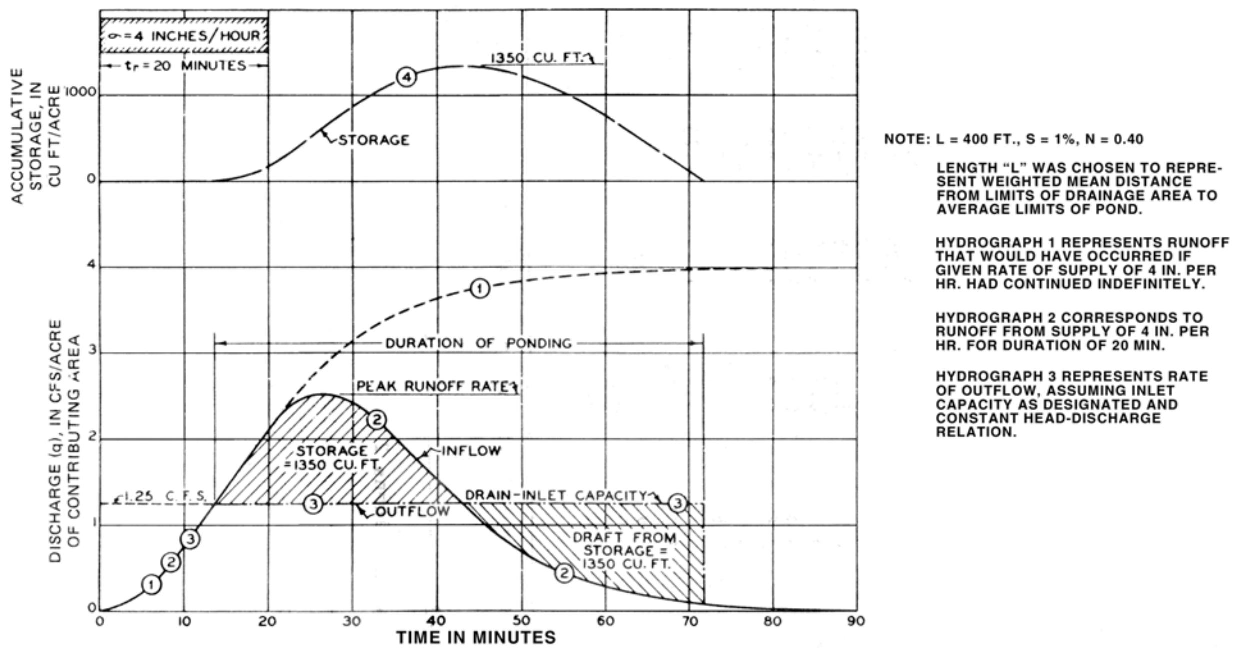
3-11.4.1.2 To illustrate these assumptions, reference is made to the curves shown in Figure 3-16 and the computations in Table 3-3. Hydrography 1 and 2 are developed as for Figure 3-14. Hydrograph 3 of Figure 3-16 represents the constant rate of outflow corresponding to inflow hydrography 2, when the drain-inlet capacity is assumed to be 1.25 cfs/acre of drainage area. Storage volume can be calculated from the area between curves 2 and 3. The volume of storage above outflow hydrography 3 and below hydrography 2 that would be accumulated at successive intervals of time under these conditions is indicated by curve 4 of Figure 3-16. The maximum storage that would accumulate under these particular conditions is 1,350 cu ft/acre of drainage area. The end of the accumulation period occurs approximately 43 min after the beginning of runoff.

Table 3-3. Design Example

Duration of supply min (1)	Rate of runoff cfs/acre (2)	Rate of runoff + 20 min cfs/acre (3)	Rate of runoff to inlet[b] cfs/acre (4)	Drain inlet capacity cfs (5)	Storage increment[c] cu ft (6)	Total storage cu ft (7)
0	0.0		0.0	0.0	0	0
5	0.2		0.2	1.25	0	0
10	0.8		0.8	1.25	0	0
13				1.25	0	0
15	1.5		1.5	1.25	+15	15
20	2.2	0.0	2.2	1.25	+180	195
25	2.7	0.2	2.5	1.25	+330	525
30	3.1	0.8	2.3	1.25	+345	870
35	3.5	1.5	2.0	1.25	+270	1,140
40	3.6	2.2	1.4	1.25	+165	1,305
43				1.25	+32	1,337
45	3.7	2.7	1.0	1.25	-15	1,322
50	3.8	3.1	0.7	1.25	-120	1,202
55	3.85	3.5	0.35	1.25	-218	984
60	3.9	3.6	0.3	1.25	-277	707
65	3.95	3.7	0.25	1.25	-292	415
70	4.0	3.8	0.2	1.25	-308	107
72				1.25	-125	0
75	4.0	3.85	0.15	1.25		
80	4.0	3.9	0.1	1.25		
85	4.0	3.95	0.05	1.25		
90	4.0	4.0	0.0			

Note: L = 400 feet; S = 1.0 percent; n = 0.40; σ = inches per hour; t_c = 20 minutes.
[a] From Figure 3-12.
[b] Difference between columns 2 and 3.
[c] Example for 20- to 25-minute increment.
V = [(2.2 – 1.25) + (2.5 – 1.25)]/2 × (5 × 60) = 330 cubic feet.

Figure 3-16. Sample Computations of Storage Required with Selected Drain-Inlet Capacity to Provide for Runoff from an Acre of Turf Under Assumptions Designated



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3-11.4.2 **Drain-inlet capacity-storage diagrams.** The concepts presented by G. A. Hathaway (American Society of Civil Engineers, Transactions, Vol 110) and discussed in Section 3-11.4 have been included in the preparation of Figures 3-17 through 3-21. These graphs are presented to facilitate the determination of the drain-inlet capacity (Diagram A) and the critical duration of supply (Diagram B) for drainage areas where temporary ponding can be permitted. Where temporary ponding is permitted, t_c reflects the time associated with both the overland flow and the time to obtain maximum temporary storage. The diagrams presented in Figures 3-20 through 3-24 have been prepared for use with effective lengths reduced to n = 0.40 and S = 1.0 percent. As an example of the use of these figures, assume:

- Effective length of overland flow = 300 ft.
- Maximum storage allowable = 1,000 cu ft/acre of drainage area.
- Rate of supply = 3.0 in./hr.

Figure 3-17. Drain-Inlet Capacity Versus Maximum Surface Storage; L = 0 and 40 ft

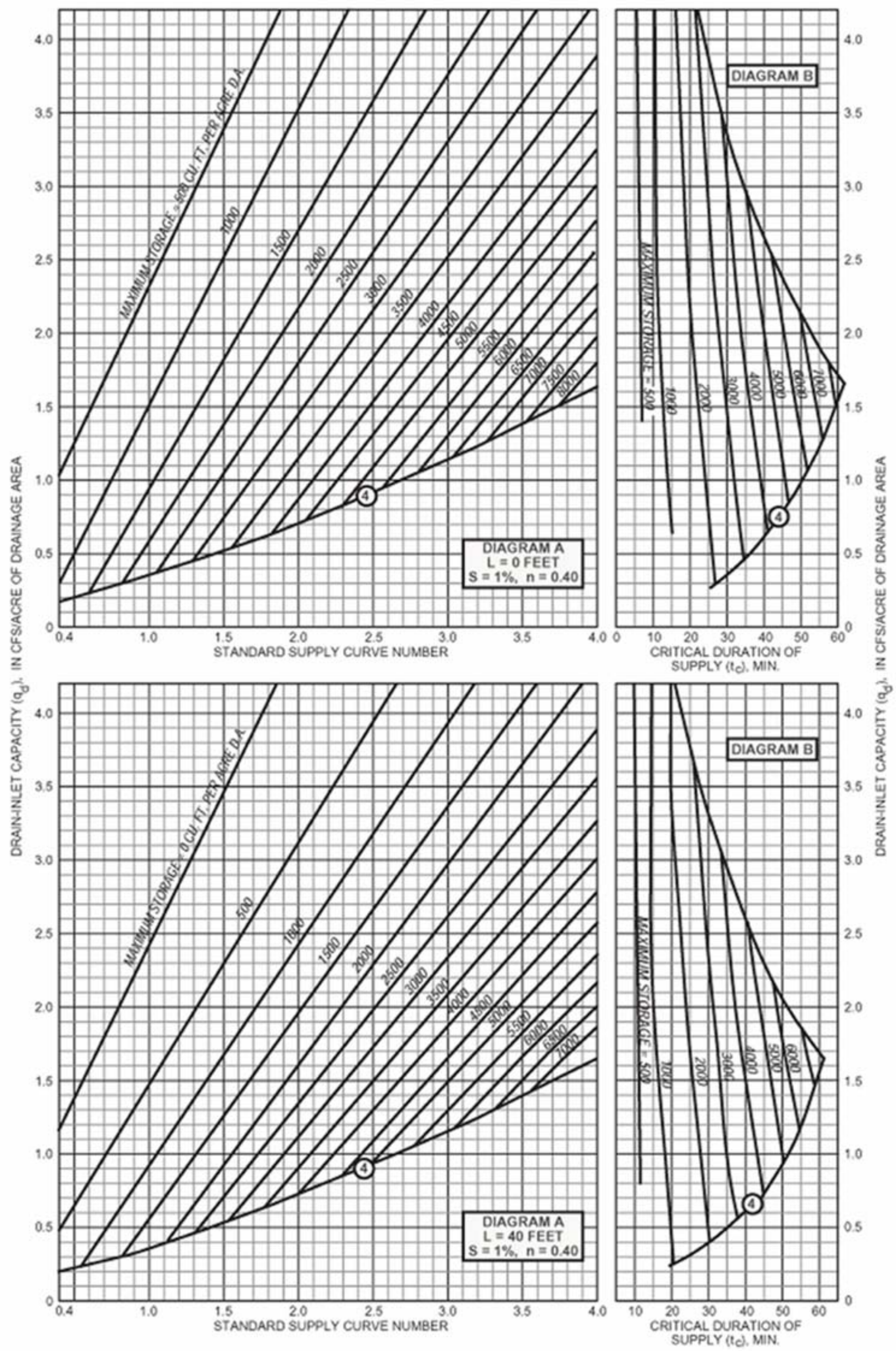


Figure 3-18. Drain-Inlet Capacity Versus Maximum Surface Storage; L = 100 and 200 ft

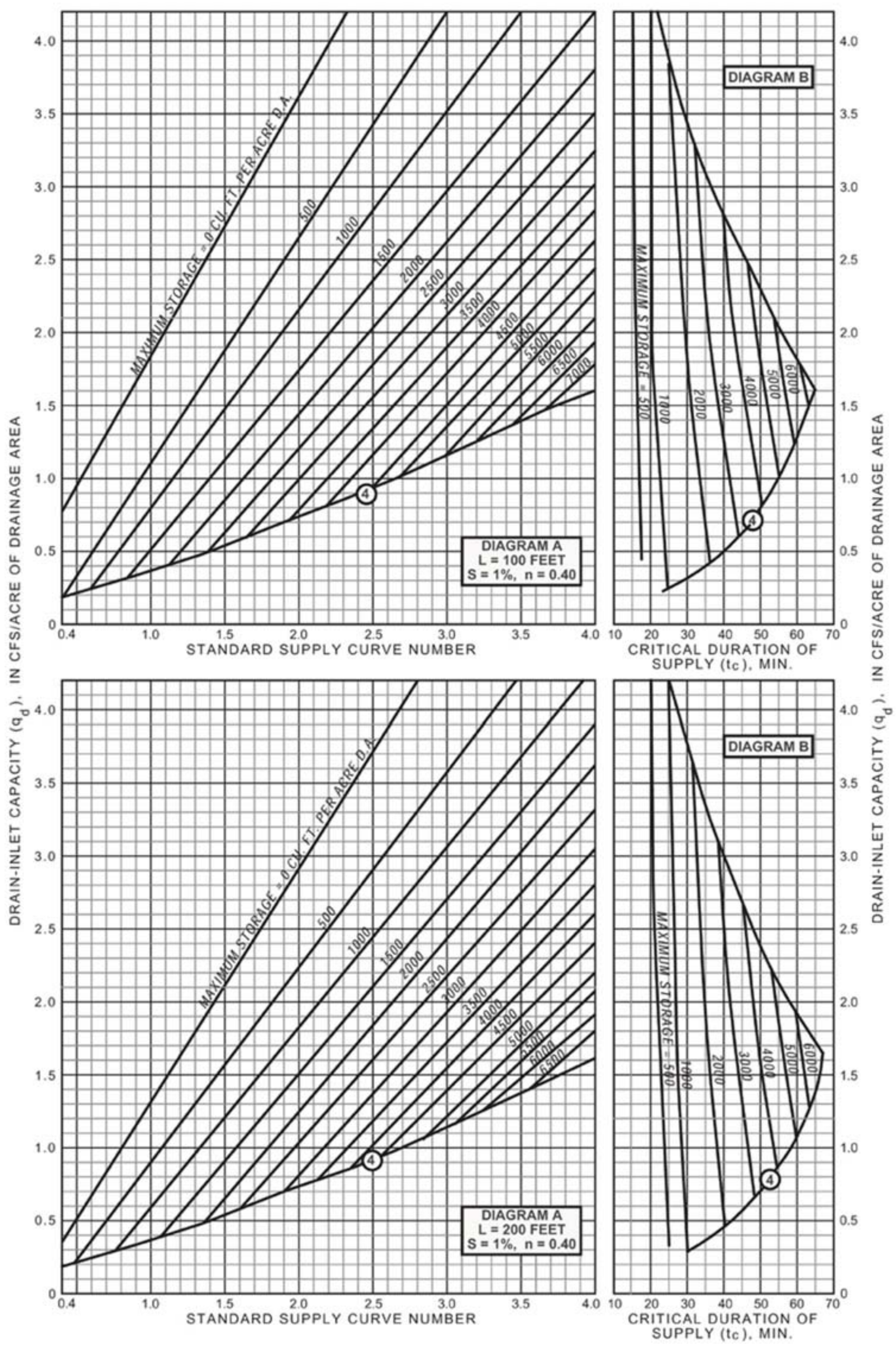


Figure 3-19. Drain-Inlet Capacity Versus Maximum Surface Storage; L = 300 and 400 ft

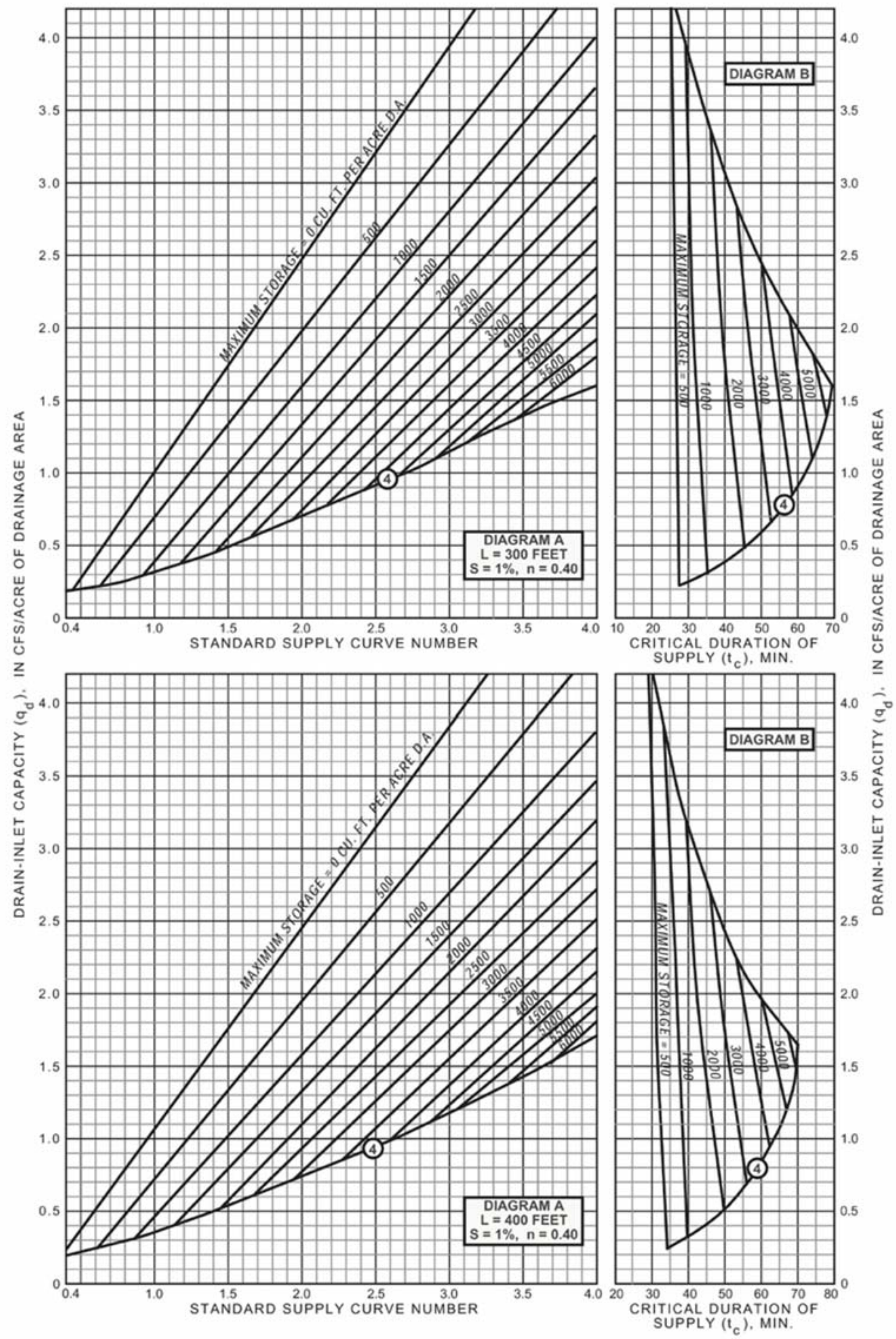


Figure 3-20. Drain-Inlet Capacity Versus Maximum Surface Storage; L = 600 and 800 ft

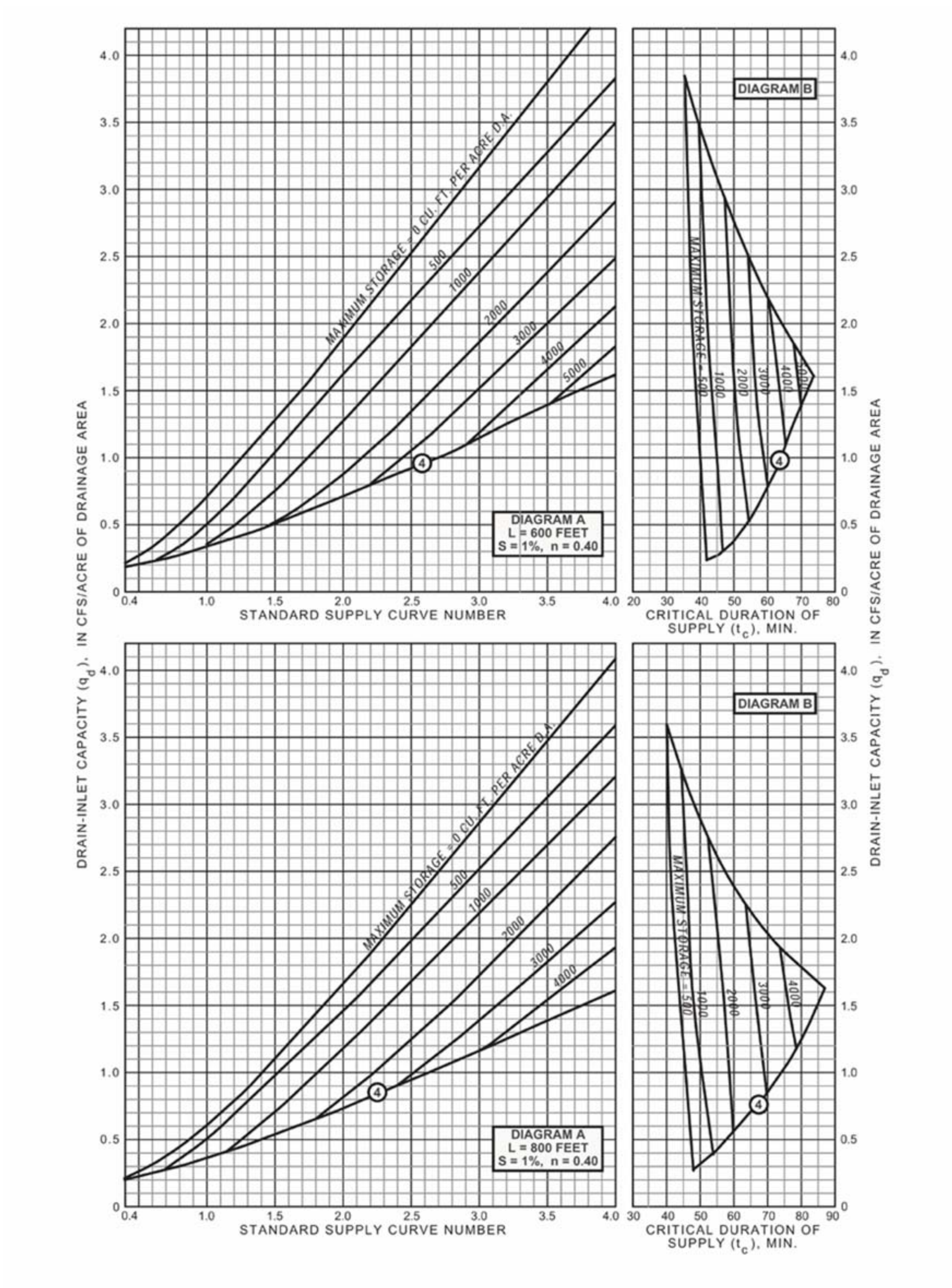
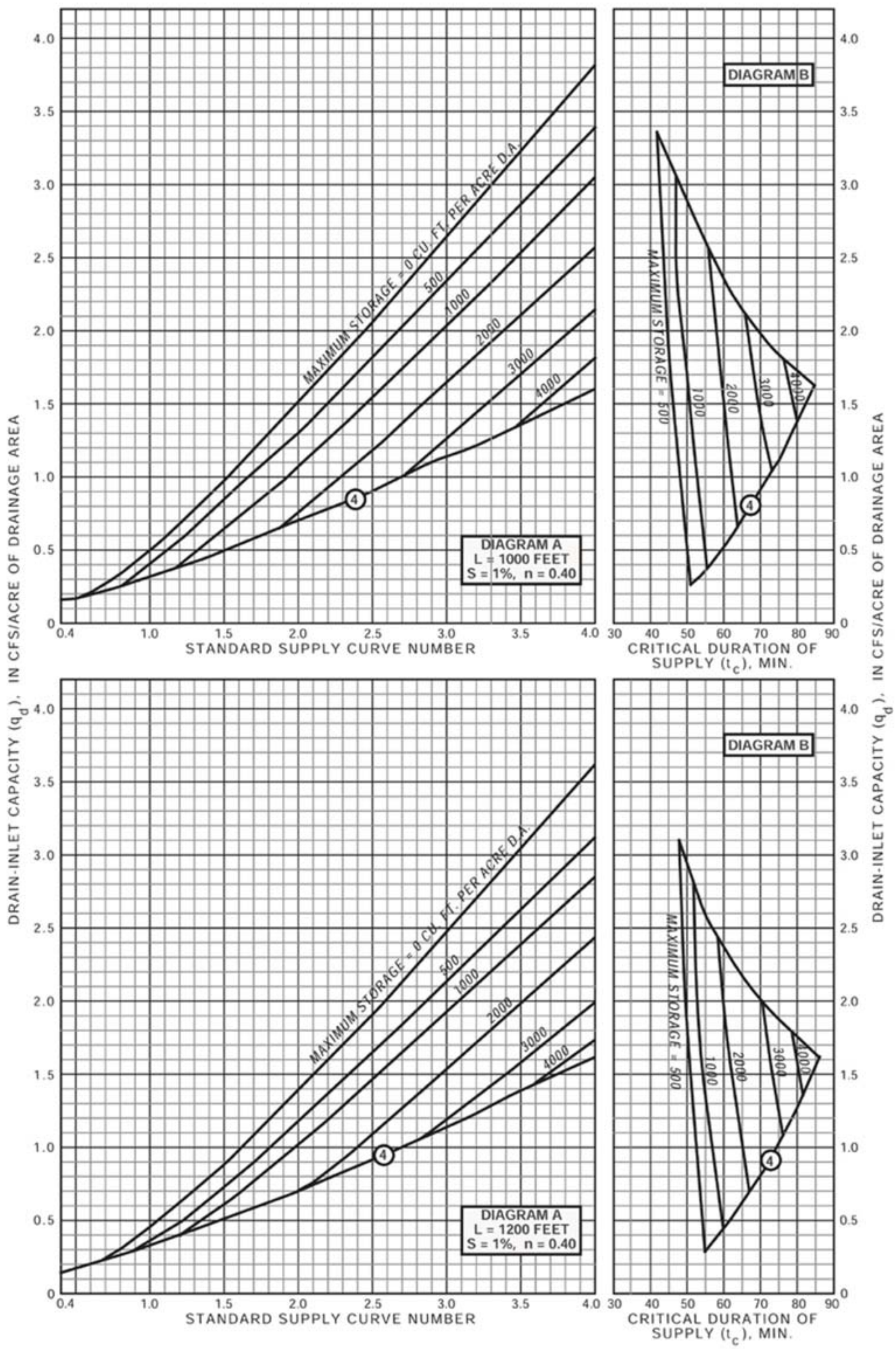


Figure 3-21. Drain-Inlet Capacity Versus Maximum Surface Storage; L = 1,000 and 1,200 ft



3-11.4.2.1 From the 3.0 in./hr line on the top portion of Figure 3-19, proceed vertically upward to the intersection of the 1,000 cu ft/acre of drainage area maximum storage capacity and then horizontally to the left to the intersection of the minimum design drain-inlet capacity of 2.8 cfs/acre of drainage area. To determine the critical duration of supply, t_c, proceed as before to the intersection of the maximum storage capacity on Diagram A; then move horizontally to the right to the intersection of the maximum, storage capacity on Diagram B, and then vertically downward to the intersection of t_c at 30 min.

3-11.4.2.2 If the drain-inlet capacity of an outlet has been previously established and the temporary ponding capacity is known, Diagram B can be entered directly to find t_c. Diagram B of Figure 3-19, for an effective length of 400 ft, offers a quick check on the example presented in Table 3-3 and Figure 3-16.

3-11.4.3 **Minimum drain-inlet capacity.** Curve 4 in Diagram A (Figures 3-17 through 3-21) represents the minimum drain-inlet capacities that are considered desirable, regardless of the volume of storage that may be permitted. The drain-inlet capacities represented by Curve 4 of Diagram A are equal to the rates of supply corresponding to durations of 4 hr on the standard supply curves given in Figure 3-1. If the drain-inlet capacity indicated by Curve 4 is adopted in a particular case, some storage may result in the ponding basin during all storms less than 4 hr in duration that produce rates corresponding to the given standard supply curve.

3-11.5. **Drain-Inlet and Drain Capacities**

3-11.5.1 **Determination of drain-inlet capacities without ponding.** From Figures 3-5 through 3-9 and 3-15 through 3-17, select the supply curve number corresponding to the weighted standard supply curve determined previously. The critical duration of supply, t_c, and the maximum rate of runoff, q_d, in cubic feet per second per acre, for the individual inlet drainage area can be read directly from the graph for the given value of effective length. If Figure 3-15 is used, the same data can be obtained by following the procedure described in Section 3-11.3.3.

3-11.5.1.1 To obtain the maximum rate of runoff at a given point in a drainage system, during a supply of uniform intensity, the storm must continue long enough to produce the maximum rate of runoff into each upstream inlet and to permit the inflow to travel through the drain from the “critical inlet” to the point of design. “Critical inlet” is defined as the upstream inlet from which the critical duration of supply causes the maximum runoff to the point of design. The critical duration of supply necessary for these purposes is referred to as t'_c and is expressed as

$$t'_c - t_c + t_d \qquad \text{(eq 3-4)}$$

where t_c is the duration of supply that would provide the maximum design-storm runoff from the area tributary to the critical drain inlet, and t_d is the time required for water to flow from the critical drain inlet to the point of design. The critical drain inlet normally may be assumed to be the inlet located the greatest distance upstream from the given point. Care should be taken to check whether t_c to an inlet along a drainage line exceeds the time required for water falling on a more distant area to reach this same inlet. Problems which arise in this regard must be investigated individually to determine under what conditions of time and flow the maximum volume of water can be expected at the point of design.

3-11.5.1.2 In order to simplify the determination of drain-inlet capacities, the computed value of t'_c may be rounded off to the nearest 5 min. Inspection of Figures 3-2 through 3-9 will disclose that for large values of effective length and low values of supply curves the maximum rate of runoff is approximately constant after t_c duration of supply. In order to facilitate design computations, the drain-inlet capacity values, q_d, obtained from the 0 storage capacity line of Diagram A of Figures 3-20 and 3-21 should be used as a replacement for the maximum rate of runoff when the duration of supply is greater than t_c, when the values of effective length are large, and when low values of the supply curve are in effect.

3-11.5.2 **Determination of drain-inlet capacities with temporary ponding.** From Figures 3-17 through 3-21, select the graph corresponding to the effective length and determine the drain-inlet capacity from the given standard supply curve value and maximum permissible ponding. In a drainage system where ponding is used, the maximum rate of flow at any given point in the drainage system may be determined, in most cases, by the simple addition of the peak discharges for the upstream inlets based on drain-inlet capacities. This procedure is justified in view of the prolonged period where temporary ponding takes place as shown in Figure 3-16. Curve 4 in Figures 3-17 through 3-21 represents the minimum drain-inlet capacities that are considered desirable, regardless of the volume of flooding exceeding allowable limits. The drain-inlet capacities represented by curve 4, in cubic feet per second per acre of drainage area, are equal to the rates of supply corresponding to durations of 4 hr on the respective standard supply curve given in Figure 3-1. If the drain-inlet capacity indicated by curve 4 is adopted in a particular case, some storage may result in the ponding basin during all storms less than 4 hr in duration that produce supply rates corresponding to the given standard supply curve. The proper criteria to be followed in estimating minimum drain-inlet capacities depend largely on the extent of drainage desired and the characteristics of the soil involved.

3-11.5.3 **Computation of pipe sizes.** The size and gradient of storm drain required to discharge design-storm runoff may be determined by use of Manning's formula presented in nomograph form in Figures 3-22 through 3-25. Storm drains will have a minimum diameter of 12 in. to lessen possibilities of clogging. Design of drain-inlet facilities is discussed in Chapter 4.

3-11.5.3.1 For conditions of instantaneous runoff the hydraulic gradient will be kept at the top of the pipe. Where temporary ponding is proposed, considerable saving in pipe sizes may be accomplished by designing the pipeline under pressure, provided undesirable backflow does not result in some critical areas.

3-11.5.3.2 Where flooding from a temporary ponding area due to rates of supply greater than design will cause a hazard to the adjacent areas, special provisions must be made to assure adequate control. An auxiliary drainage system or a diversionary channel to another inlet or ponding area is a method that has been used successfully. The designer must consider each case individually to arrive at the most economical solution to provide the desired results.

Figure 3-22. Nomograph for Computing Required Size of Circular Drain Smooth Interior, Flowing Full; n = 0.012

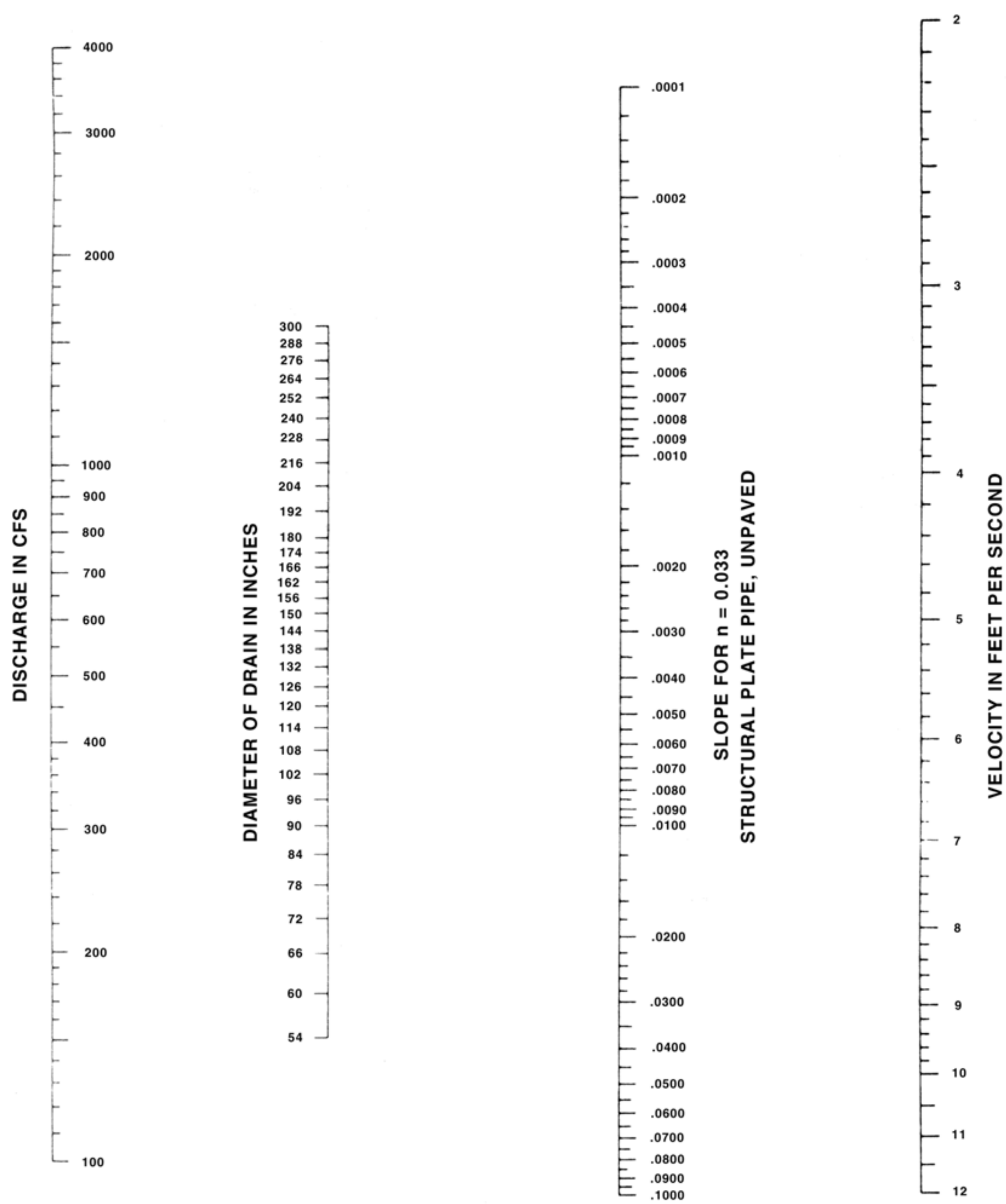


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Figure 3-23. Nomograph for Computing Required Size of Circular Standard Corrugated Metal Pipe, 25 percent Paved Invert, Flowing Full; n = 0.021

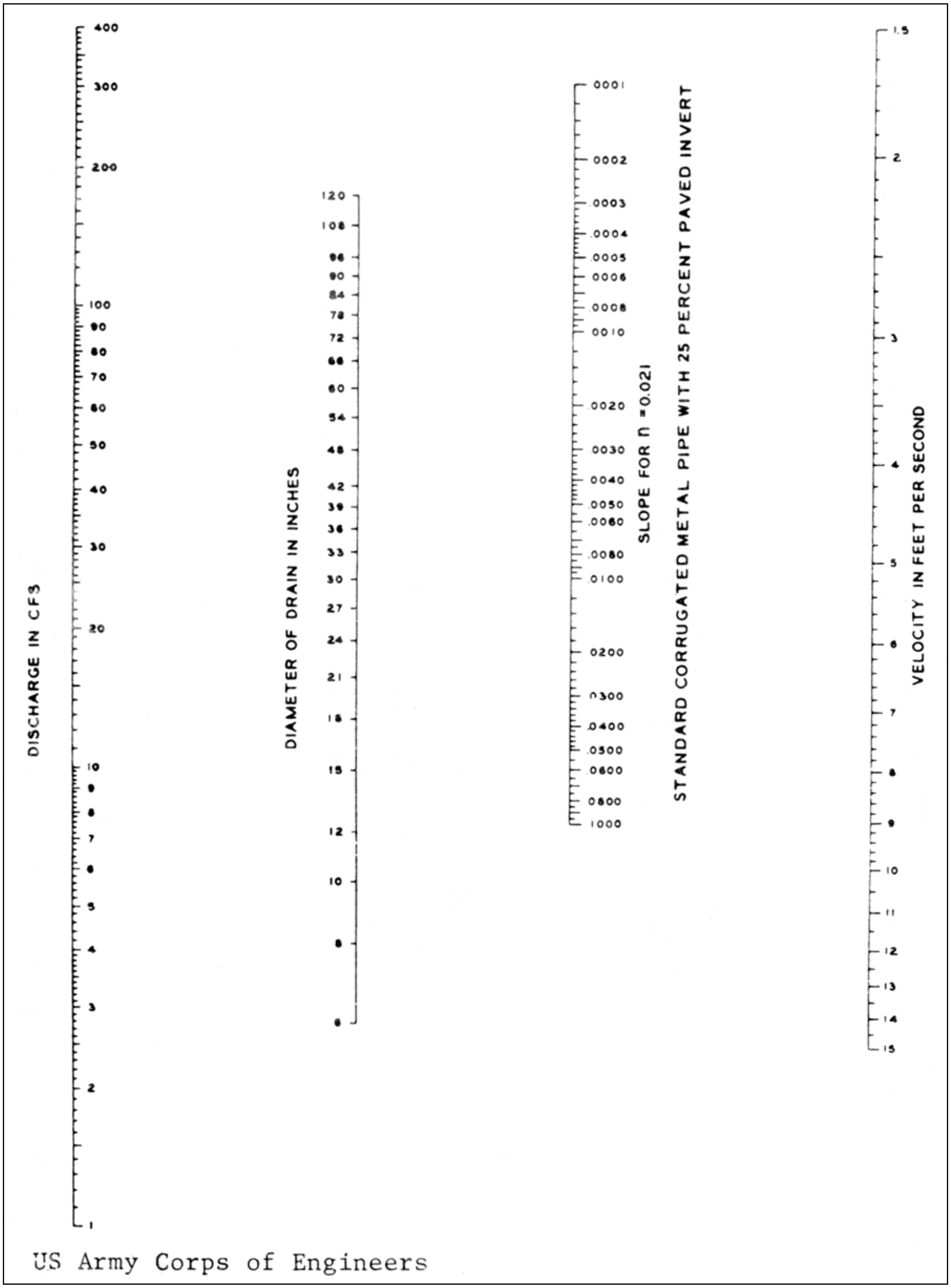

Figure 3-24. Nomograph for Computing Required Size of Circular Standard Corrugated Metal Pipe, Unpaved, Flowing Full; n = 0.021

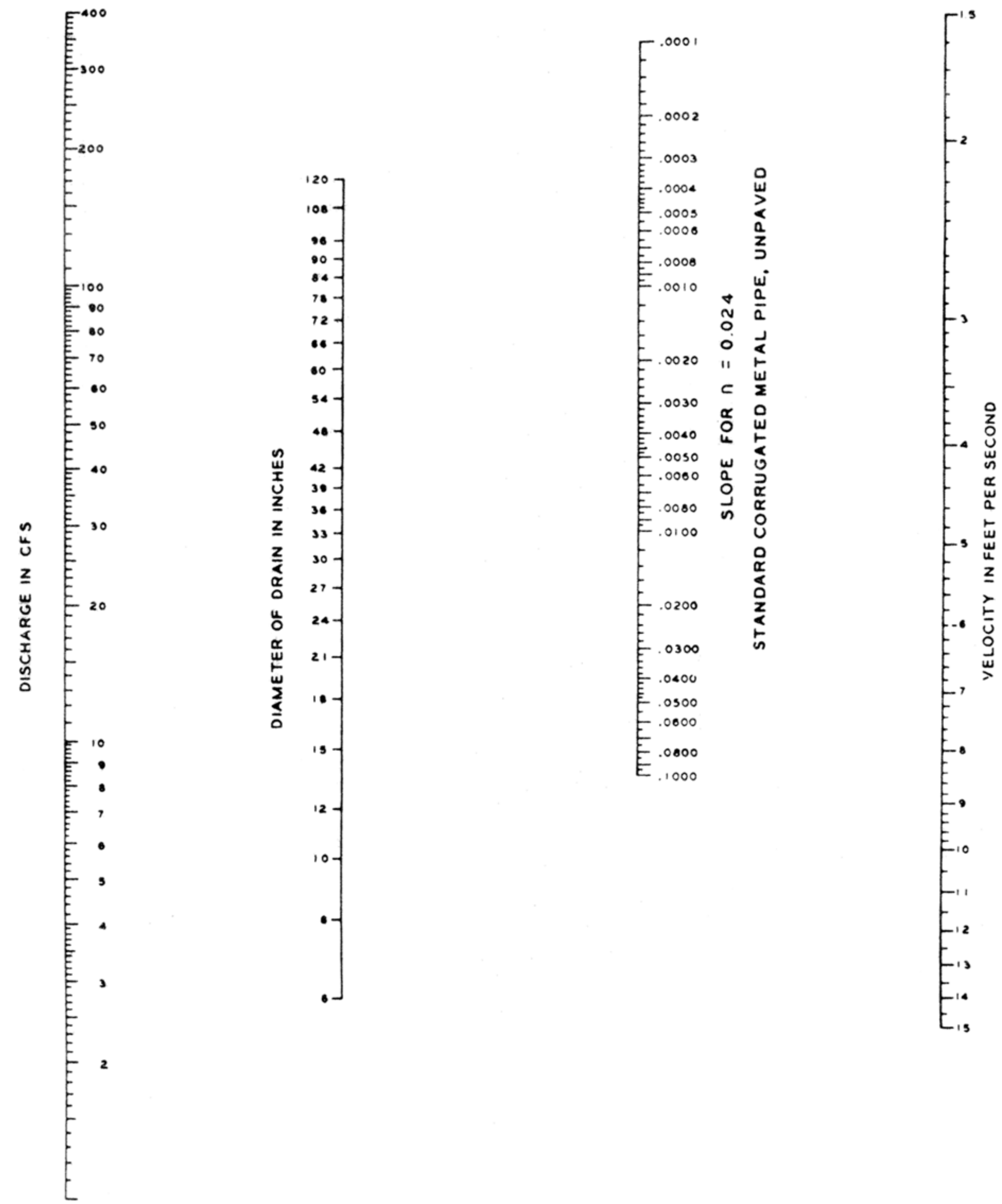

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Figure 3-25. Nomograph for Computing Required Size of Circular Structural Plate Pipe, Unpaved, Flowing Full; n = 0.033