ARMY TM 5-820-1
AIR FORCE AFM 88-5, Chap. 1

DEPARTMENTS OF THE ARMY AND THE
AIR FORCE TECHNICAL MANUAL

SURFACE DRAINAGE FACILITIES FOR AIRFIELDS AND HELIPORTS

DEPARTMENTS OF THE ARMY, AND THE AIR FORCE
AUGUST 1987

REPRODUCTION AUTHORIZATION/RESTRICTIONS

TECHNICAL MANUAL
NO. 5-820-1
AIR FORCE MANUAL
NO. 88-5, VOLUME 1

HEADQUARTERS
DEPARTMENTS OF THE ARMY
AND THE AIR FORCE
WASHINGTON, DC, *20 August 1987*

SURFACE DRAINAGE FACILITIES FOR AIRFIELDS AND HELIPORTS

LIST OF FIGURES

***This manual supersedes TM 5-820-1/AFM 88-5, Chap. 1, dated April 1977.**

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SURFACE DRAINAGE FACILITIES FOR AIRFIELDS AND HELIPORTS

1. Purpose and scope. This manual 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 TM 5-852-7/AFM 88-19, Chap. 7 (see app A for referenced publications).

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:

a. Surface runoff from the design s term. Surface runoff from the selected design storm will be disposed of without damage to the airfield facilities or significant interruption of normal traffic.

b. 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.

c. *Reliability of operation.* The drainage system will provide maximum practicable reliability of operation under all climatic conditions.

d. Maintenance. The drainage system in the immediate vicinity of operational facilities will require minimum maintenance.

e. 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.

f. 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.

g. *Future expansion.* Drainage design should allow for future expansion with a minimum of expense and traffic interruption.

h. Environmental impact. Drainage facilities will be constructed with minimal impact on the environment.

3. Drainage protection required.

a. 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.

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

4. Hydrologic considerations.

a. Definitions. The following definitions are used in the development of hydrologic concepts.

(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-year frequency, that is, an annual probability y of occurrence of 0.5, is considered satisfactory for most airfields.

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

(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 l-hour duration.

b. 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. TM 5-820-4/AFM 88-5, Chap. 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 paragraphs 5 through 9 of this manual and developed and illustrated in appendixes B and C combines features from both basic approaches to determine runoff.

5. Rainfall.

a. 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 hour and the average intensities at the same frequency for periods less than 1 hour, regardless of the geographical location of the stations. The average rainfall for a l-hour period at various frequencies for the continental United States, excluding Alaska, may be determined from figure 1. Data for other locations are available from the Office, Chief of Engineers, and the National Oceanic and Atmospheric Administration, National Weather Service (formerly the US Weather Bureau). For Alaska, data may be obtained from TM 5-852-7/AFM 88-19, Chap. 7, and US Weather Bureau Technical Paper No. 47. Data for Puerto Rico and the Virgin Islands and for Hawaii may be obtained from US Weather Bureau Technical Papers No. 42 and 43, respectively. For any frequency, the l-hour rainfall intensity is considered a design-storm index for all average intensities and duration of storms with the same frequency.

b. Standard rainfall intensity-duration curves. Figure 2 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-minute design storm based on a 2-year frequency in central Kentucky. From figure 1 the 2-year l-hour rainfall is found to be 1.4 inches per hour. In figure 2, supply curve No. 1.4 is used with the 40-minute duration of storm to determine a rainfall intensity of 1.9 inches per hour.

c. Incomplete data. In areas where rainfall data are incomplete or unavailable, the methods described in appendix B can be used to develop design rainfall information.

d. 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, pending, and characteristics of the runoff surfaces.

e. 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.

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-hour period of antecedent rainfall. Wherever possible, determine average infiltration rates from a study of runoff records near the airfield from infiltrometer

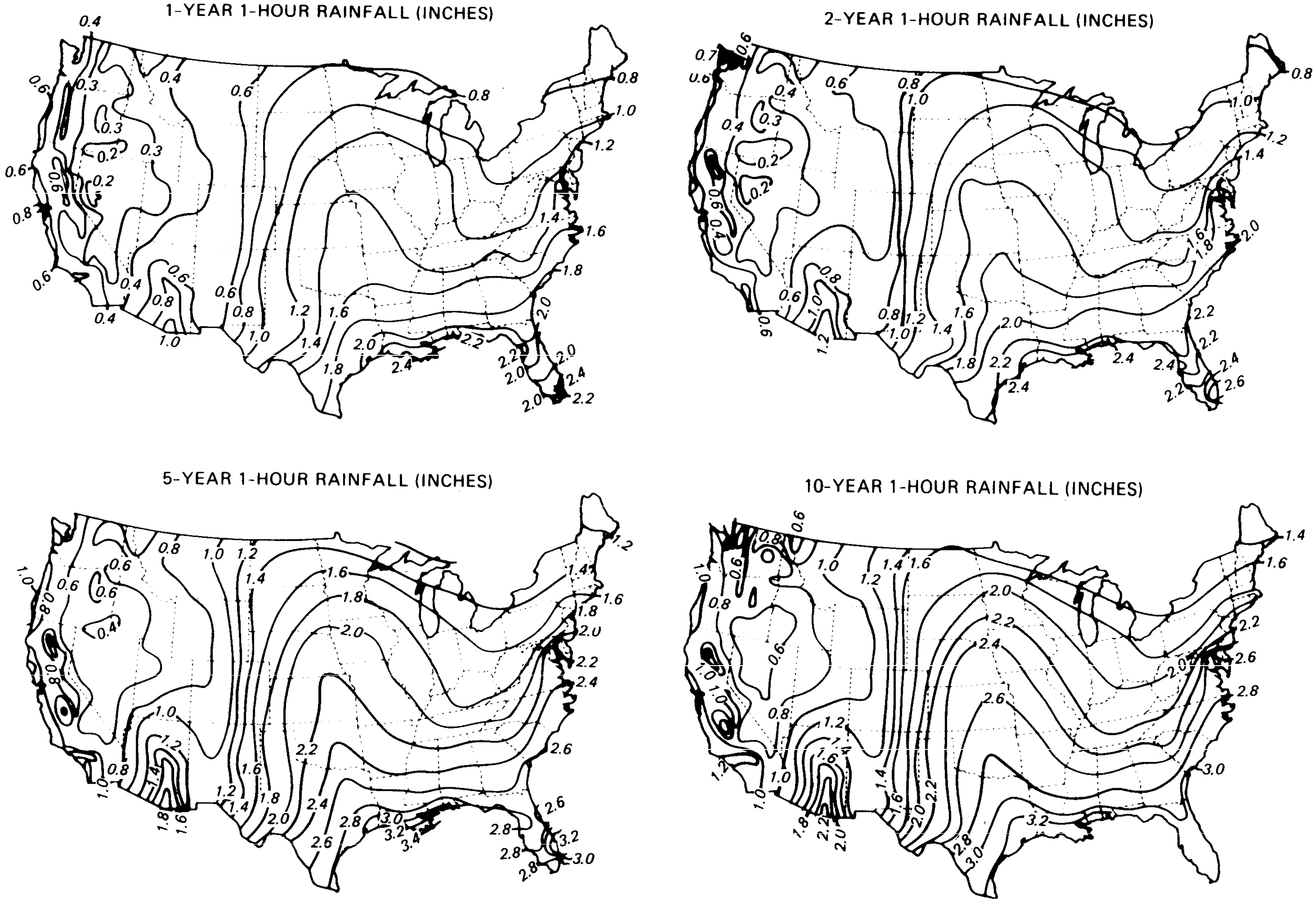
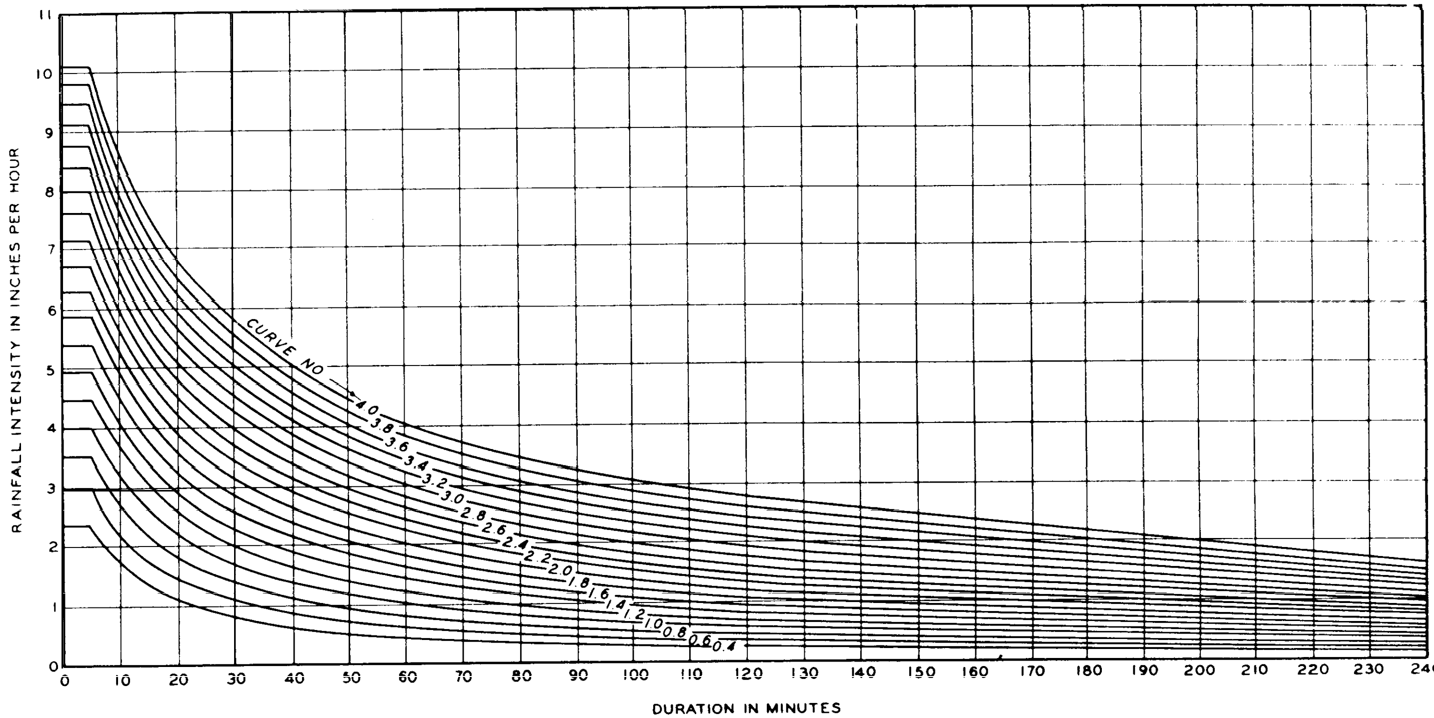


Chart reproduced from US Weather Bureau, Technical Paper No. 40, Rainfall Frequency Atlas of the United States, Washington, DC, May 1961.

Figure 1. Design storm index: 1-hour rainfall intensity-frequency data for continental United States excluding Alaska.



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.

Reproduced from "Design of Drainage Facilities," by G. A. Hathaway, Transactions, Vol 110, with permission of the American Society of Civil Engineers.

Figure 2. Standard rainfall intensity-duration curves or standard supply curves.

studies or from similar acceptable information. Suggested mean values of infiltration for generalized soil classifications are shown in table 1. The soil group symbols are those given in 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 hour of rainfall for turfed areas is approximately 0.5 inch per hour and seldom exceeds 1.0 inch per hour. The infiltration rate for paved or roofed areas, blast protective surfaces, and impervious dust-palliative-treated areas is zero.

Table 1. Infiltration rate for generalized soil classifications (uncompacted)

Description	Soil Group Symbol*	Infiltration, inch per hour
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

*Classified by the Unified Soil Classification system (MIL–STD–619).

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.

a. 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 2. 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 2. For example, if supply curve No. 2.2 in figure 2 were selected as the design storm and the infiltration loss during a l-hour storm were estimated as 0.6 inch, supply curve No. 1.6 would be adopted as the standard supply curve for the given areas.

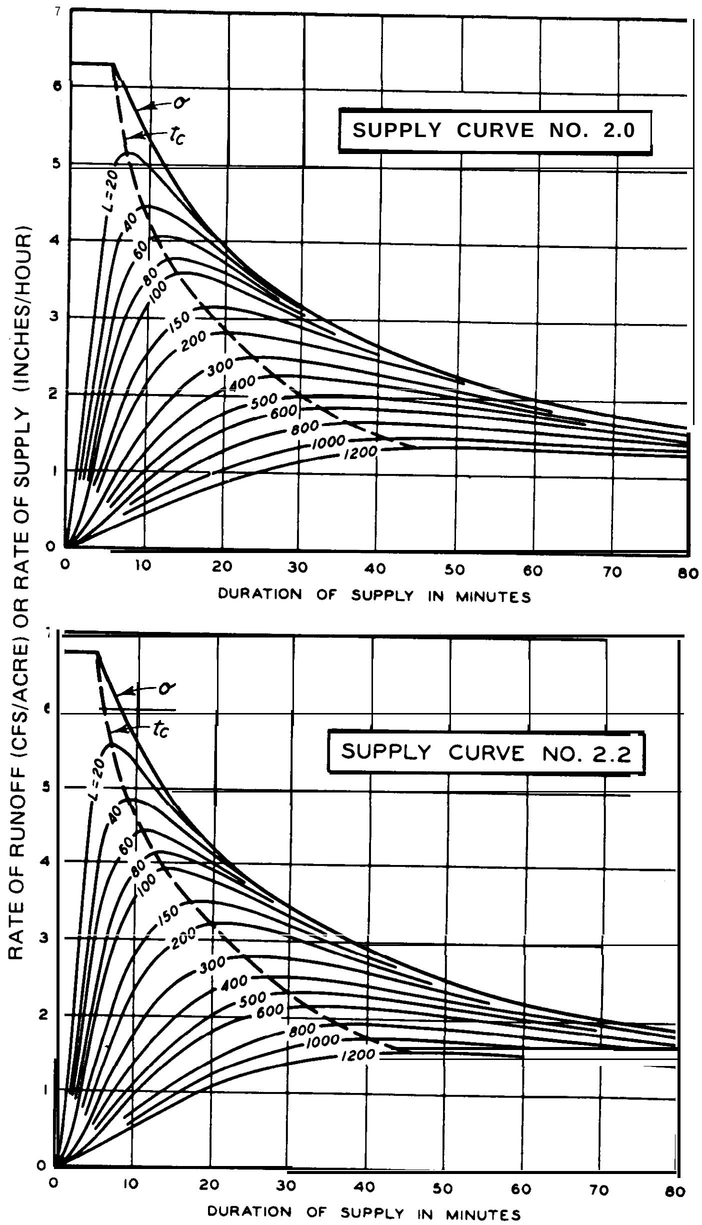
b. Weighted standard rate of supply curves. Drainage areas usually consist of combinations of paved and unpaved areas having different infiltration capacities. A weighted standard supply should be established for the composite drainage areas by weighting the standard supply curve numbers adopted for paved and unpaved surfaces in proportion to their respective tributary area. An example is given in appendix B.

8. Runoff. The method of runoff determination described herein is based on an overland flow model. Details are given in appendix B.

a. 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.

b. 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 appendix B.

c. Maximum rate of runoff. Figure 3 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 appendix B (figs B-7 through B–14). Figure 3 depicts the relationships between 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



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.

US Army Corps of Engineers

Figure 3. 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.

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:

(1) Assume the effective length of overland flow is 300 feet:

(*a*) 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 feet curves intersect. This value is found to be 24 minutes.

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

(c) The average rate of supply over the area is obtained by reading vertically upward from the point where the t_c and L = 300 feet curve intersect to the σ curve and then reading horizontally across from this point. This value is found to be 3.6 inches per hour or 3.6 cfs/acre.

(2) Assume the critical duration of supply is 30 minutes:

(a) The average rate of supply is obtained by reading horizontally across from the point where the duration of supply = 30 minutes and a intersect. This value is found to be 3.2 inches per hour or 3.2 cfs/acre.

(b) The effective length is obtained by reading the point where t_c and the duration of supply = 30 minutes intersect. This is found to be 500 feet.

(c) The maximum rate of runoff is obtained by reading horizontally across from this point. This is found to be 2.0 inches per hour or 2.0 cfs/acre.

9. Storage. The supply curves in figure 3 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 appendix B.

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 pending; 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 TM 5-820-2/AFM 88-5, Chap. 2, 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.

a. Information required. Before proceeding with the design calculations, as illustrated in appendixes B and C, certain additional information and data must be developed. These include:

(1) A topographic map.

(2) A layout of the helipad, runways, taxiways, aprons, and other hardstands with tentative finished grading contours at l-foot intervals.

(3) Profiles of runways, taxiways, apron areas, and other hardstands.

(4) Soil profiles based on soil tests to include, whenever possible, infiltration properties of local soils to be encountered.

(5) Groundwater elevation and fluctuation if known or obtainable.

(6) A summary of climatic conditions including temperature ranges, freezing and thawing patterns, and depth of frost penetration.

(7) Snowfall records, snow cover depths, and convertibility factors to inches of rainfall.

(8) Runoff records for drainage areas in the same locality having similar characteristics and soil conditions.

b. 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.

(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.

(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-foot 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-foot 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) *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.

(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.

c. Classification of storm drains. Storm drains for airfields and heliports may be classified in two groups, primary and auxiliary.

(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 pending basins above the drain inlets. To lessen construction requirements for drainage facilities, maximum use of pending 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.

(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.

d. Storm-drain layout. The principal procedures in the determination of the storm-drain layout follow:

(1) *Preliminary layout.* Prepare a preliminary map (scale 1 inch = 200 feet 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 pending basins around primary drain inlets, need not be shown more accurately than with l-foot contour intervals.

(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) *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.

(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.

(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.

(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.

(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.

(8) *Maximum pending 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 feet 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.

(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.

(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.

e. *Typical design procedures.* The procedures in paragraphs 2 through 10 are illustrated and annotated in the design computations contained in appendix C. Comparative designs with and without provisions for temporary ponding have been prepared for the airfield shown.

APPENDIX A
REFERENCES

Government Publications.

Department of Defense

Military Standards

MIL-STD-619	Unified Soil Classification System for Roads, Airfields, Embankments, and Foundations

Departments of the Army and the Air Force

AFR 86-14	Airfield and Heliport Planning Criteria
TM 5-803-4	Planning of Army Aviation Facilities
TM 5-820-21AFM 88-5, Chap. 2	Drainage and Erosion Control, Subsurface Drainage Facilities for Airfields
TM 5-820-3/AFM 88-5, Chap. 3	Drainage and Erosion Control, Structures for Airfields and Heliports
TM 5-820-4/AFM 88-5, Chap. 4	Drainage for Areas Other than Airfields
TM 5-852-71AFM 88-19, Chap. 7	Surface Drainage Design for Airfields and Heliports in Arctic and Subarctic Regions

Department of Commerce

National Oceanic and Atmospheric Administration, Environmental Data and Information Center, FederaI Building, Asheville, NC 28801

US Weather Bureau Reports

Technical Paper No. 40	Rainfall-Frequency Atlas of the United States (1963)
Technical Paper No. 42	Generalized Estimates of Probable Maximum Precipitation and Rainfall-Frequency Data for Puerto Rico and Virgin Islands (1961)
Technical Paper No. 43	Rainfall-Frequency Atlas of the Hawaiian Islands (1962)
Technical Paper No. 47	Probable Maximum Precipitation and Rainfall-Frequency Data for Alaska (1963)

Department of Transportation

Federal Aviation Administration, M-443.1, 400 7th Street S. W., Washington, DC 20590

AC 150/5320-5B	Airport Drainage (1970)

Nongovernment Publications.

American Society of Civil Engineers (ASCE), United Engineering Center, 345 E. 47th Street, New York, NY 10017

Manual of Practice No. 37	Design and Construction of Sanitary and Storm Sewers (1969, reprinted in 1974)
Transactions, Vol. 110	Design of Drainage Facilities (1945)

APPENDIX B
DESIGN PROCEDURE

B-1. Rainfall.

a. Intensity-frequency data. In areas where intensity-frequency data are incomplete or unavailable, the 2-year l-hour 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 inch 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-year 1-hour rainfall with these four climatic parameters appears in figure B–1.

(1) When daily rainfall data are not available, the 2-year l-hour 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.

(2) As an example of the use of figure B-1, assume the mean annual precipitation is 60 inches, 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-year l-hour precipitation value (first estimate). In this example, the first estimate for the 2-year l-hour precipitation is approximately 1.4 inches. Now assume the fourth parameter, the mean of annual series of maximum daily precipitation, is 4.3 inches. 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-year l-hour precipitation value (second estimate). In this example, the second estimate would be 2.0 inches. The second estimate is preferable, if four parameters are available.

(3) For frequencies other than 2 years, the following factors in table B-1 can be used to approximate intensity-frequency values, using the 2-year 1-hour value as a base:

Table B–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

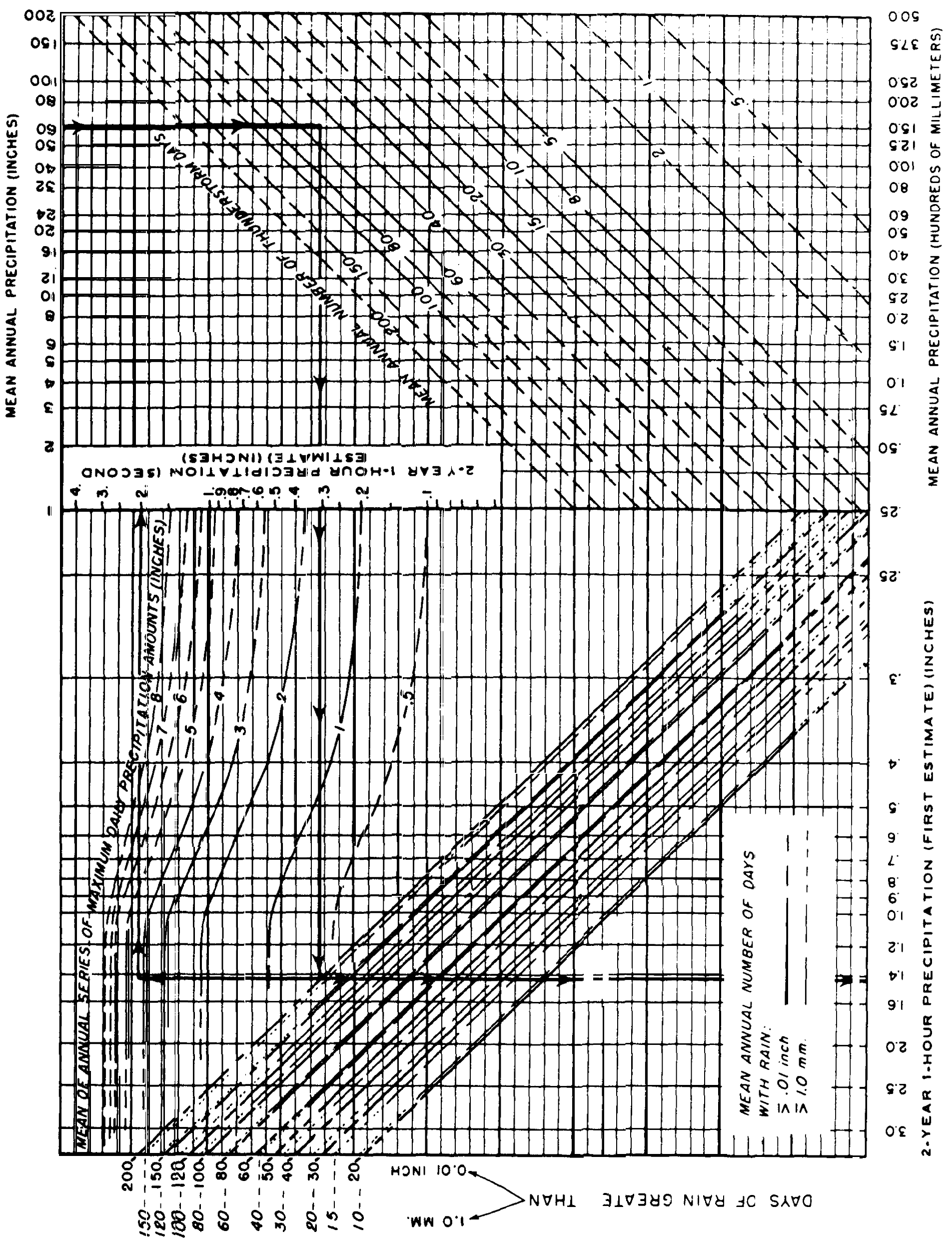
b. Standard rate of supply curves. Standard supply curves for areas with zero infiltration loss will be the same as the standard rainfall intensity curves in figure 2 (see main text). Where infiltration losses occur, the standard supply curve number corresponding to a given standard rainfall curve number is computed by subtracting the estimated l-hour infiltration value from the 1-hour rainfall quantity.

c. *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:

$$SC_w = \frac{[(SC_1 \times A_1) + (SC_2 \times A_2) + \ldots (SC_n \times A_n)]}{A_1 + A_2 + \ldots + A_n} \qquad \text{(eq B–1)}$$

where the SC's are standard supply rates for the various areas, A. For example, if the drainage area under consideration has a 1-hour rainfall intensity of 2.5 inches; estimated infiltration values of 0.0 for paved area Al, 0.6 for turfed area AZ, and 0.2 for bare clay area A3; and drainage area Al is 1.5 acres, A2 is 5.0 acres, and A3 is 6.5 acres; then the weighted standard supply curve for the composite drainage area would be:

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



NOTE: DASHED LINES ARE UNSUPPORTED BY DATA.

Joint Committee of US Army Corps of Engineers and US Weather Bureau

Figure B-1. Diagram for estimating 2-year 1-hour rainfall.

$SC_w = 2.2$

d. 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 B –2 as interpreted by G. A. Hathaway (American Society of Civil Engineers, *Transactions*, Vol 110):

$$q = \sigma \tanh^2 [0.922t(\sigma/nL)^{0.50}S^{0.25}] \quad \text{(eq B–2)}$$

where

q = rate of overland flow at the lower end of an elemental strip, inches per hour or cubic feet per second per acre.

σ = rate of supply or intensity of rainfall excess, inches per hour.

t = time, or duration, from beginning of supply, minutes.

n = coefficient of roughness of the surface.

L effective length of overland, or channel flow, feet.

S = slope of the surface (absolute, that is, 1 percent = 0.01).

tanh = hyperbolic tangent.

(1) The curves shown in figures B-2 through B-4 were computed using equation B-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 B-5. Hydrography 1 and 1-A in figure B-5 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 minutes 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).

(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 B-2 through B–4. The necessary conversions to get an effective length may be accomplished by substituting the quantity $nL\ \sqrt{0.4}\ \sqrt{S}$ for L or by using figure B–6 as explained in paragraph B–2.

B-2. Effective length.

a. General. In equation B –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 pending 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 B-6. The length of channel flow is measured along the proposed collecting channel for that section in which appreciable depth of flow may be 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 B-6. 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 B-2. TM 5-820–3/AFM 88-5, Chap. 3 gives additional n values for turfed channels. For example, to find the effective length of overland flow for an actual length of 900 feet 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-foot actual length by 2 and enter graph A of figure B-6 with 450 feet 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 feet, which must be multiplied by 2, resulting in a total effective length of overland flow of 550 feet.

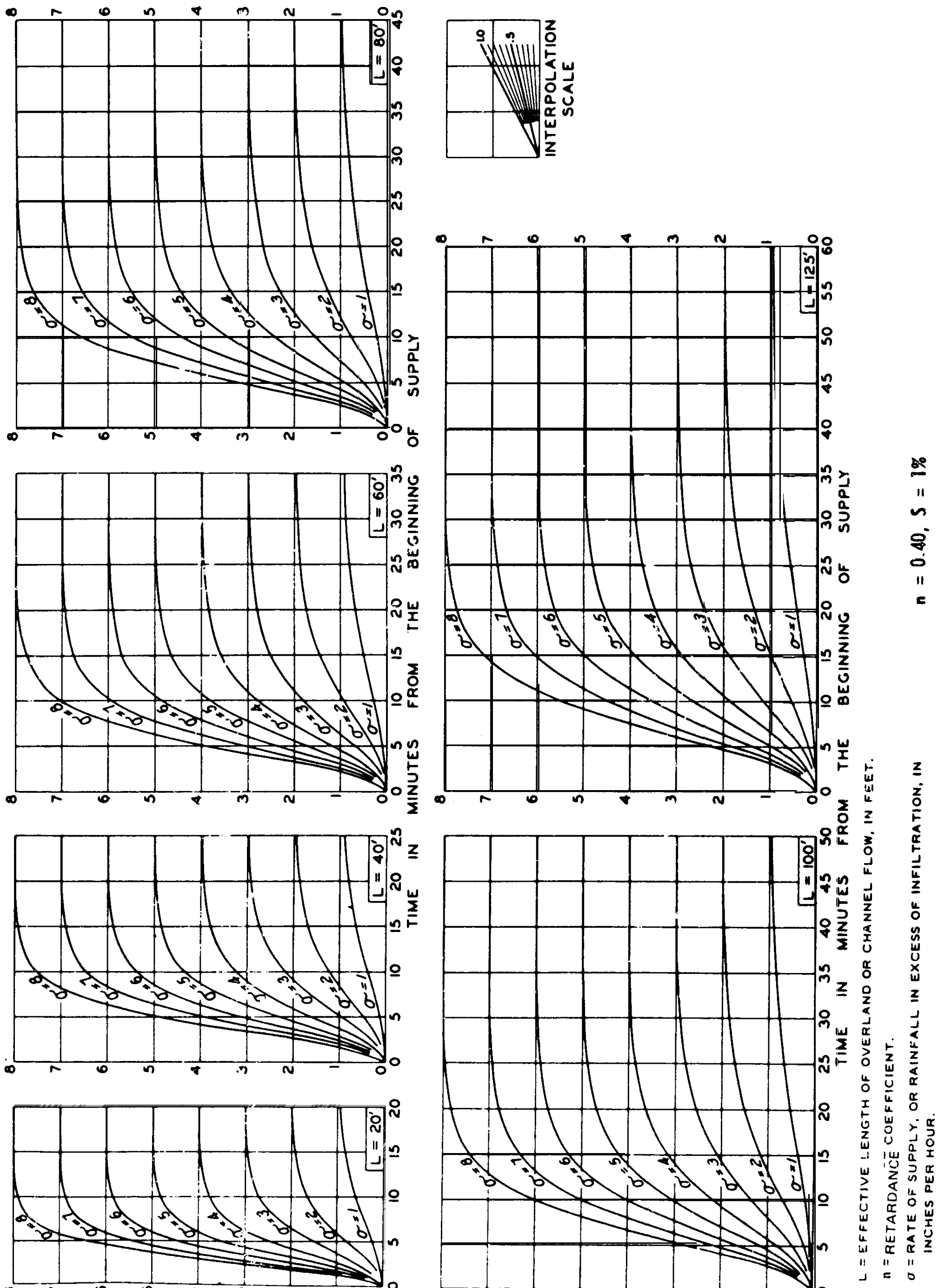


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Figure B-2. Rates of runoff versus duration of supply for turfed areas; L = 20, 40, 60, 80, 100, and 125 feet.

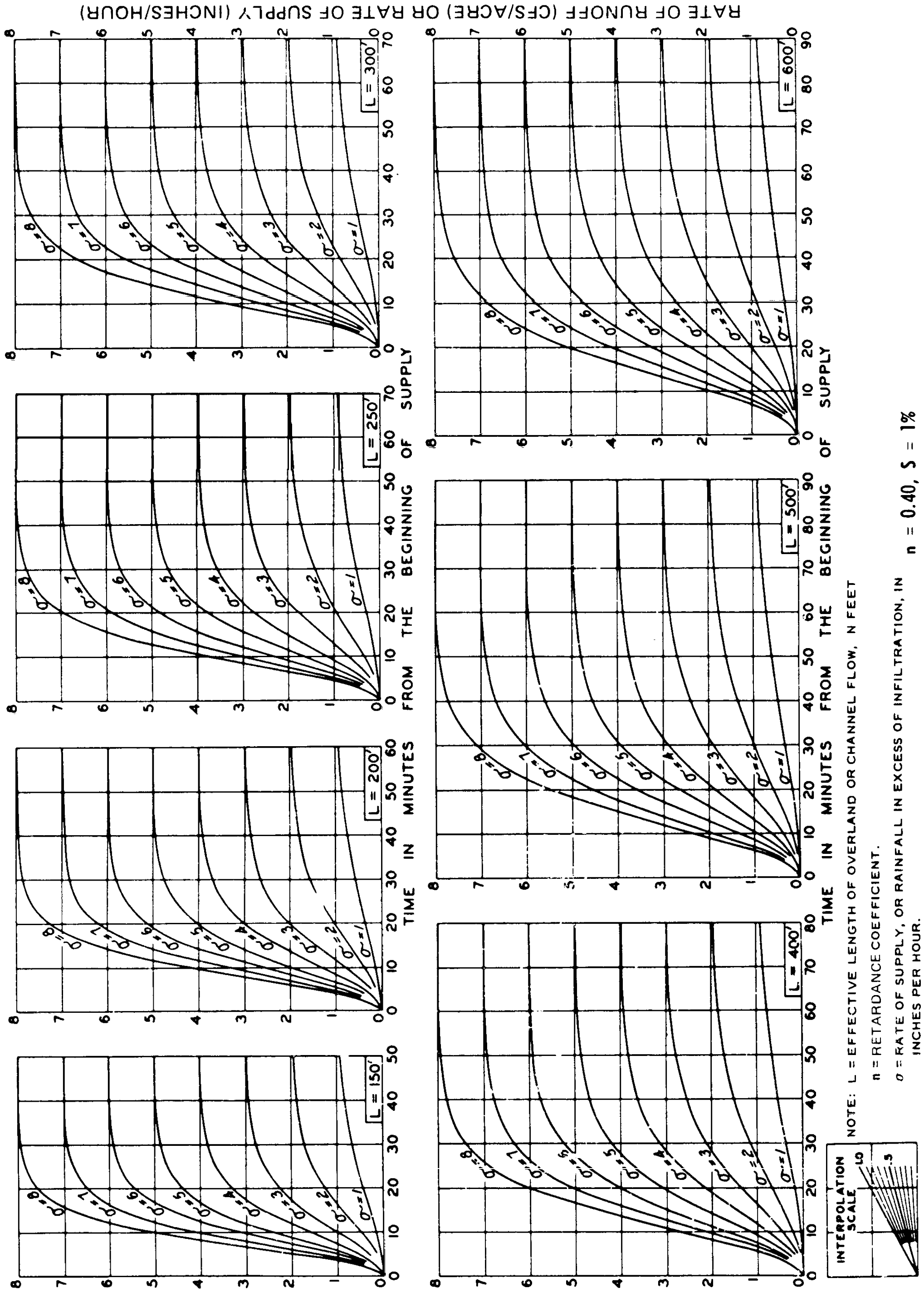

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Figure B-3. Rates of runoff versus duration of supply for turfed areas; L = 150, 200, 250, 300, 400, 500, and 600 feet.

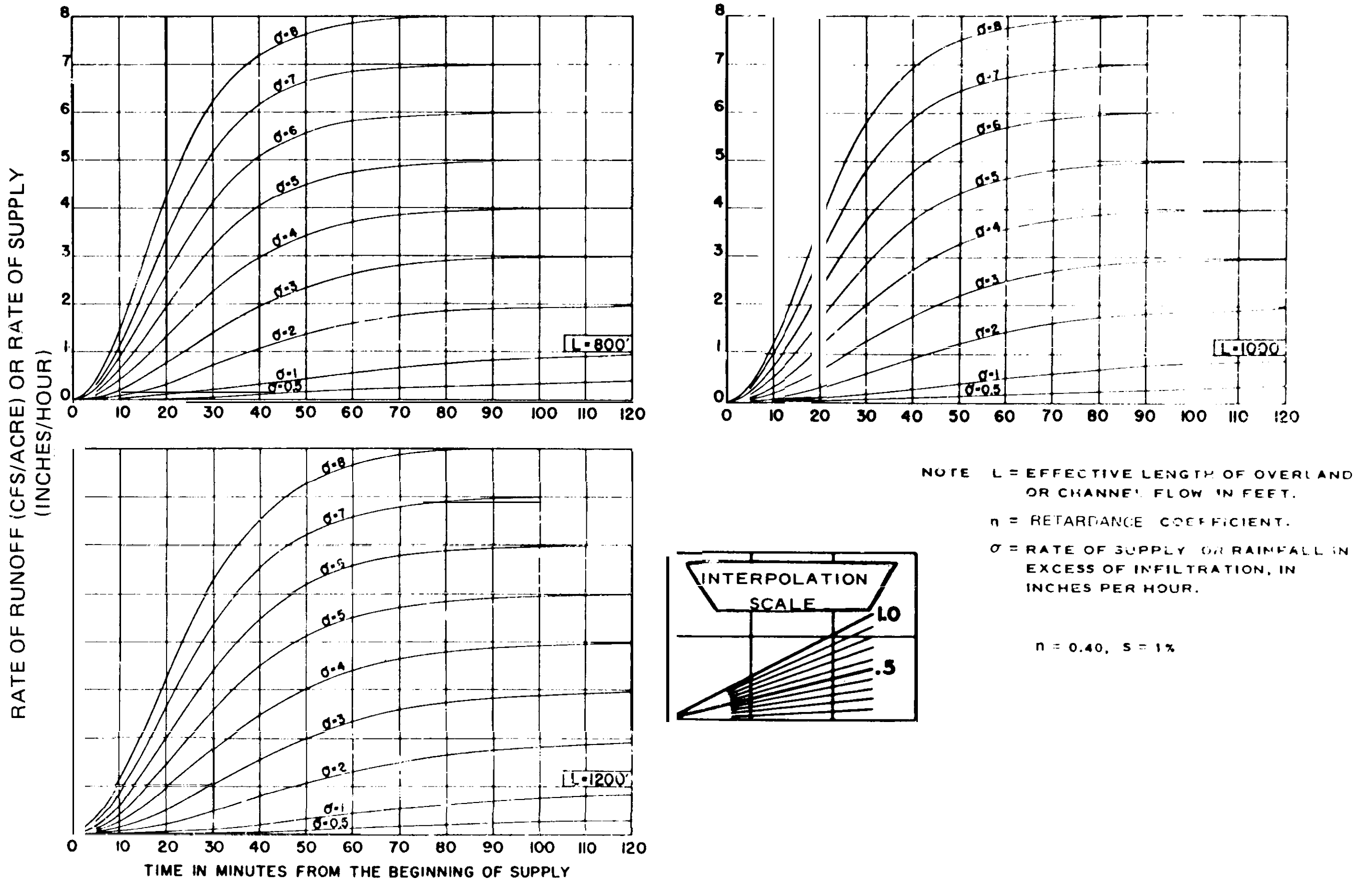

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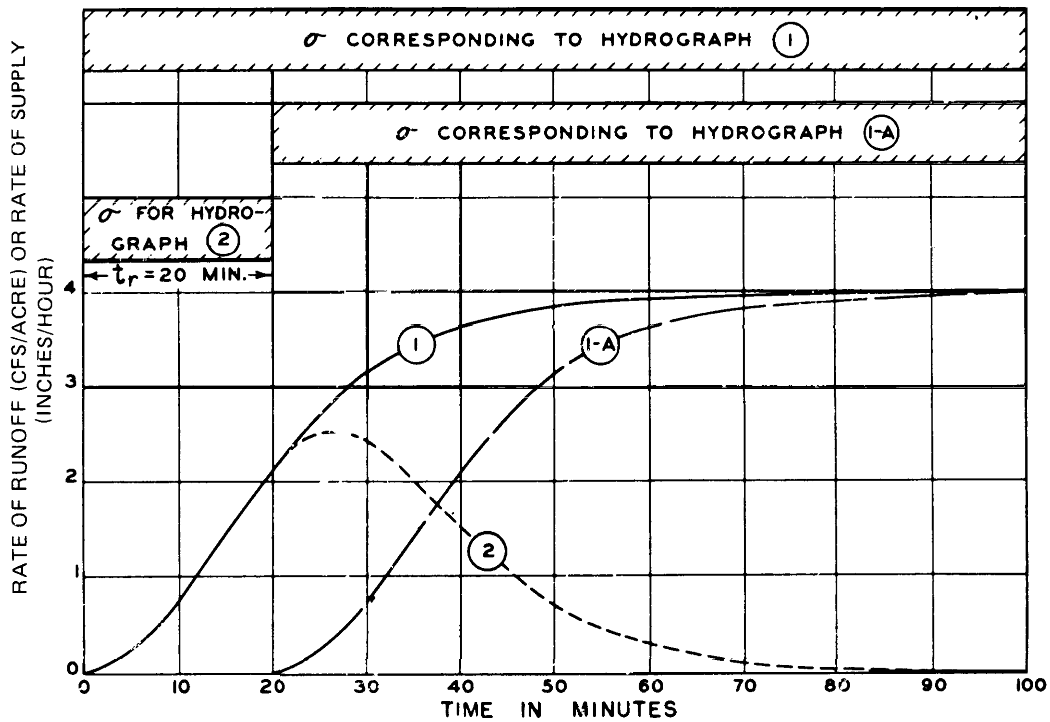
Figure B-4. Rates of runoff versus duration of supply for turfed areas; L = 800, 1,000, and 1,200 feet.



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. B-3).

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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Figure B-5. Computation of hydrograph to represent runoff from supply of specified duration.

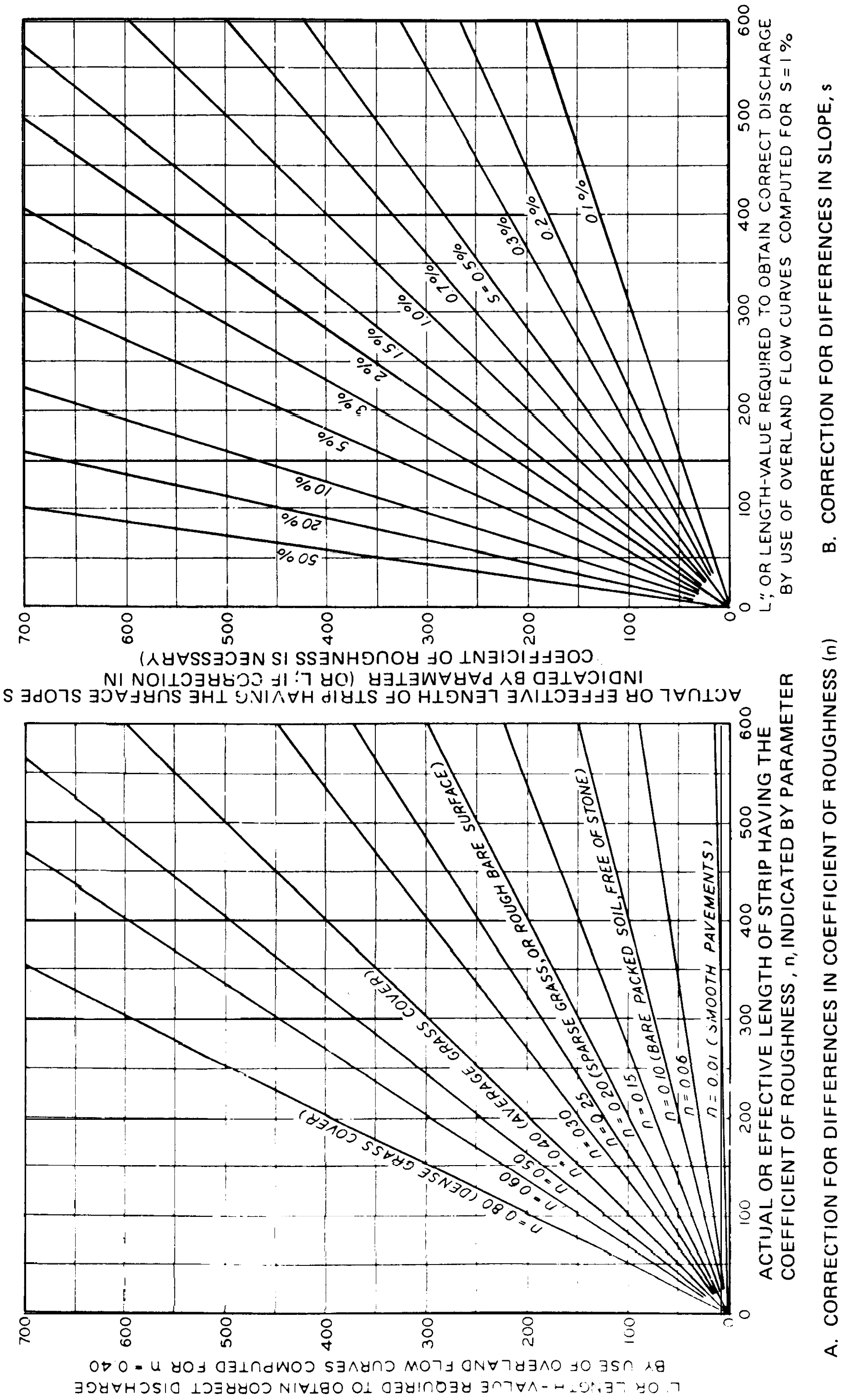

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Figure B-6. Modification in L required to compensate for differences in n and S.

Table B-2. Coefficients of roughness for overland flow

Surface	Value of n
Pavements and paved shoulders	0.01 (0.02)
Bare, packed soil, free of stone	0.10
Sparse grass cover, or moderately rough bare surface	0.20
Average grass cover	0.40
Dense grass cover	0.80

Reproduced from "Design of Drainage Facilities," by G. A. Hathaway, *Transactions*, Vol 110, with permission of the American Society of Civil Engineers.

b. Effect of paved area on determination of effective length. Pending 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 pending 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 minutes), the length of the paved area can be disregarded or given very little weight in estimating the value of L for a composite area.

c. Determination of effective length for pending 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 pending 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 pending area, which corresponds to a depth equal to two-thirds of the maximum depth caused by the design storm.

B-3. Runoff.

a. General. The curves shown in figures 3 (main text) and B-7 through B-14 describe the relationship between rate of supply, σ; critical duration of supply, t_c; effective length of overland flow, L; and maximum rate of runoff for the various supply curves presented in figure 2. The curves portray the data presented in the flow curves shown in figure B-2 through B-4 in another format. Table B-3 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.

b. Peak runoff rates. Figures 3 and B-7 through B-14 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 minutes for paved areas and 20 minutes 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:

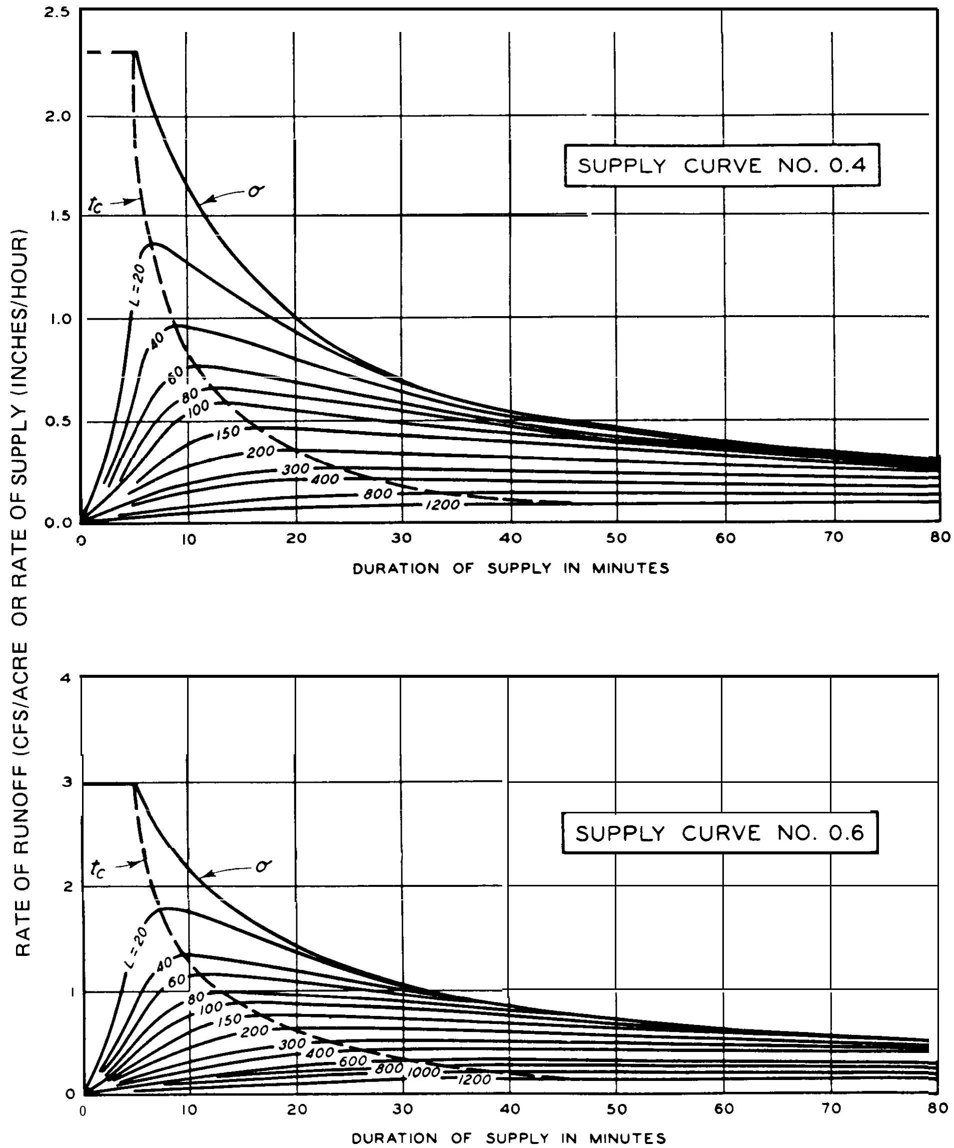
$$t_c = (10A_p + 20A_t) / (A_p + A_t) \qquad \text{(eq B-3)}$$

where

A_p = area paved, acres

A_t = area turfed, acres

c. *Consolidated design curve.* The data presented in figures 3 and B-7 through B-14 with respect to peak runoff rates and critical durations of supply have been consolidated into one diagram, figure B-15. Use of figure B-15 is not as precise as using figures 3 and B-7 through B-14, but figure B-15 may be applied to most drainage problems. The following example is provided to illustrate the use of figure B-15. Assume an effective length of overland flow of 315 feet and a rate of supply of 1.0 inch per hour. 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



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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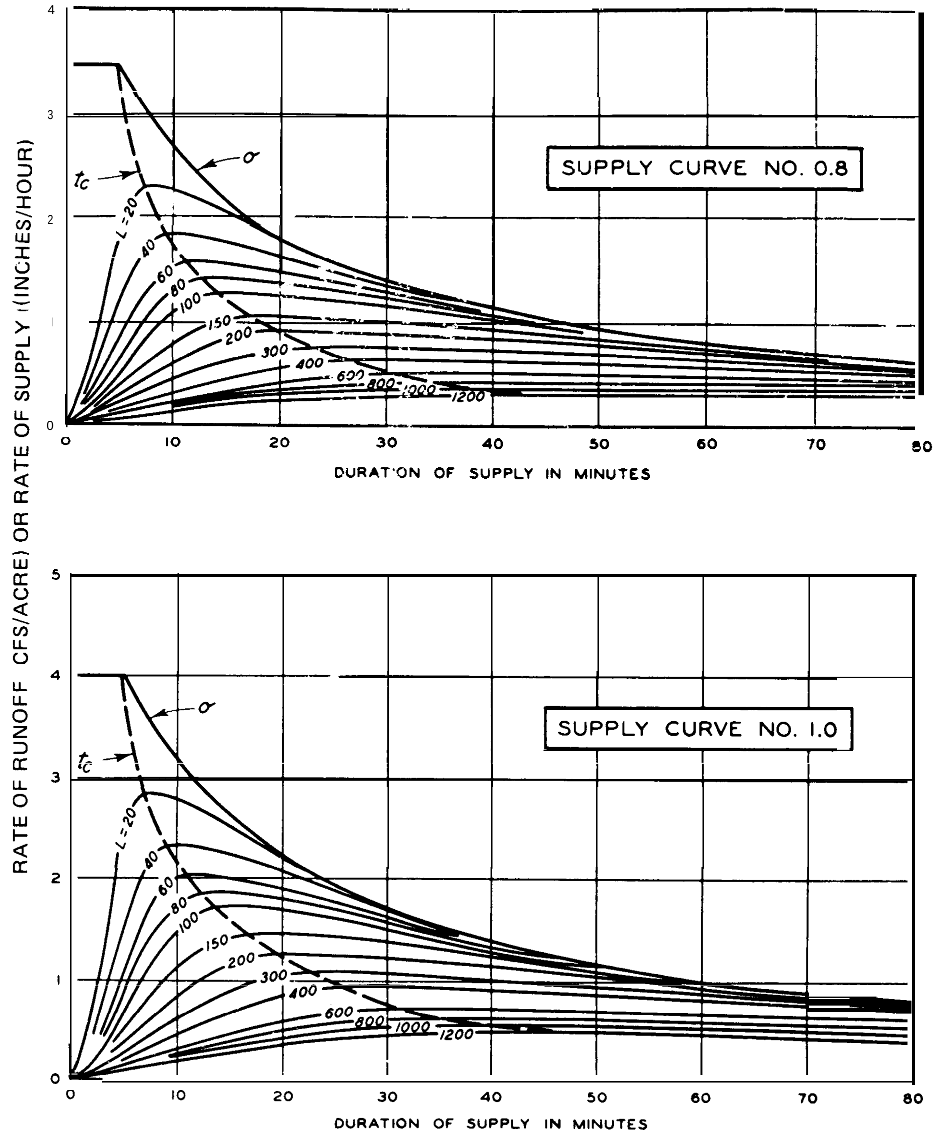
Figure B-7. Rates of runoff corresponding to supply curves No. 0.4 and 0.6; n = 0.40 and S = 1 percent.



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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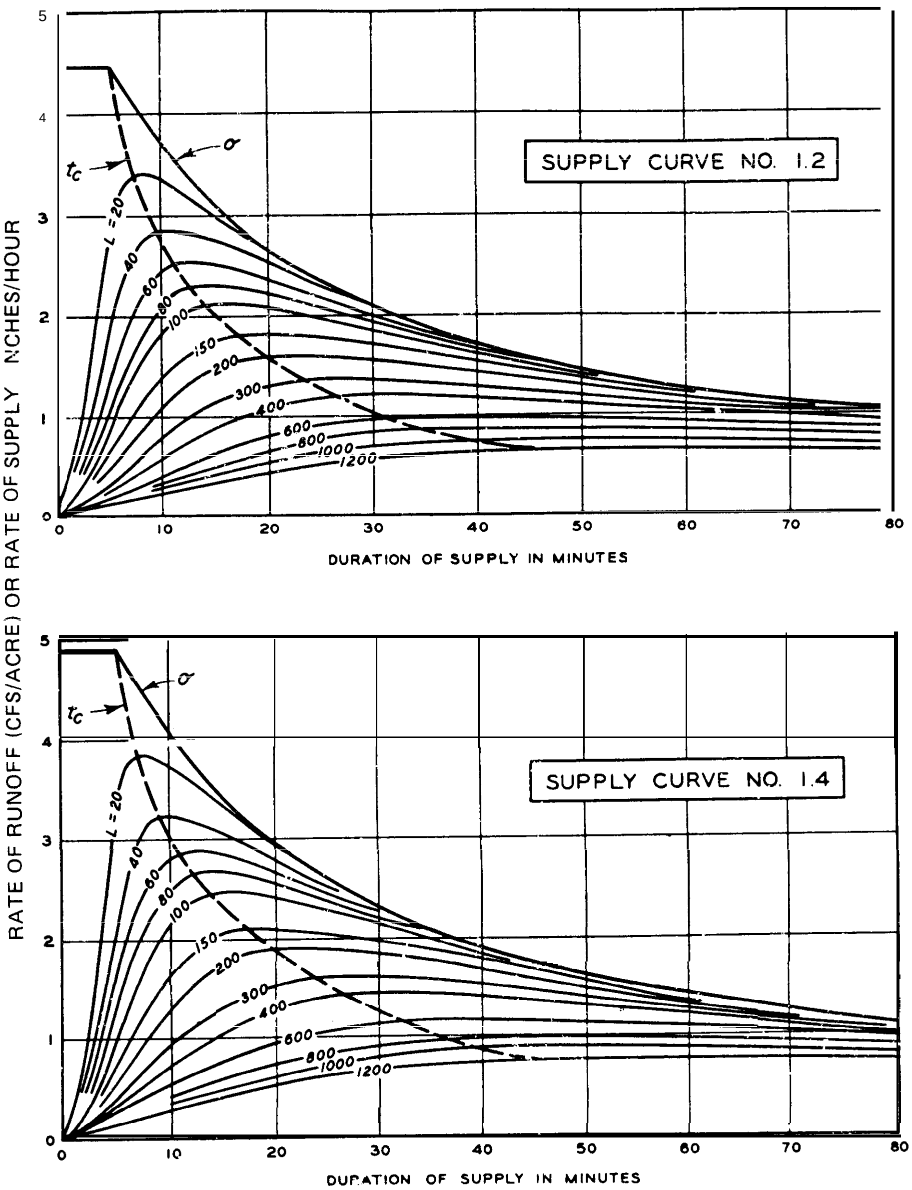
Figure B-8. Rates of runoff corresponding to supply curves No. 0.8 and 1.0; n = 0.40 and S = 1 percent.



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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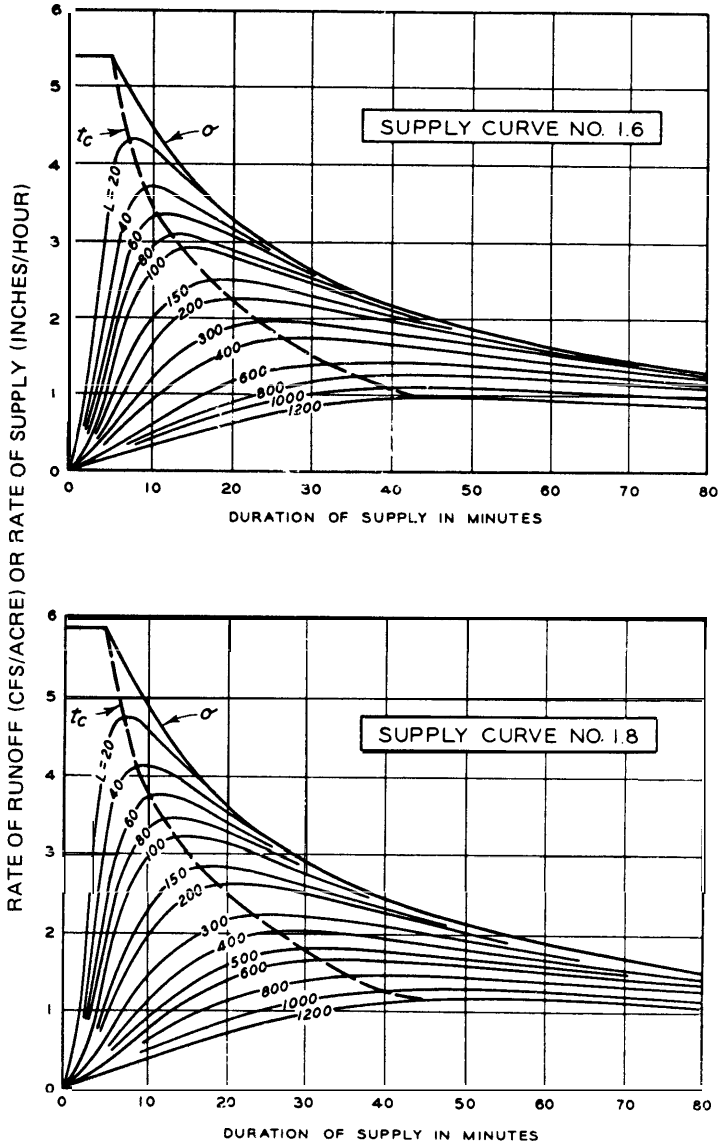
Figure B-9. Rates of runoff corresponding to supply curves No. 1.2 and 1.4; n = 0.40 and S = 1 percent.



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 B-10. *Rates of runoff corresponding to supply curves No. 1.6 and 1.8; n = 0.40 and S = 1 percent.*

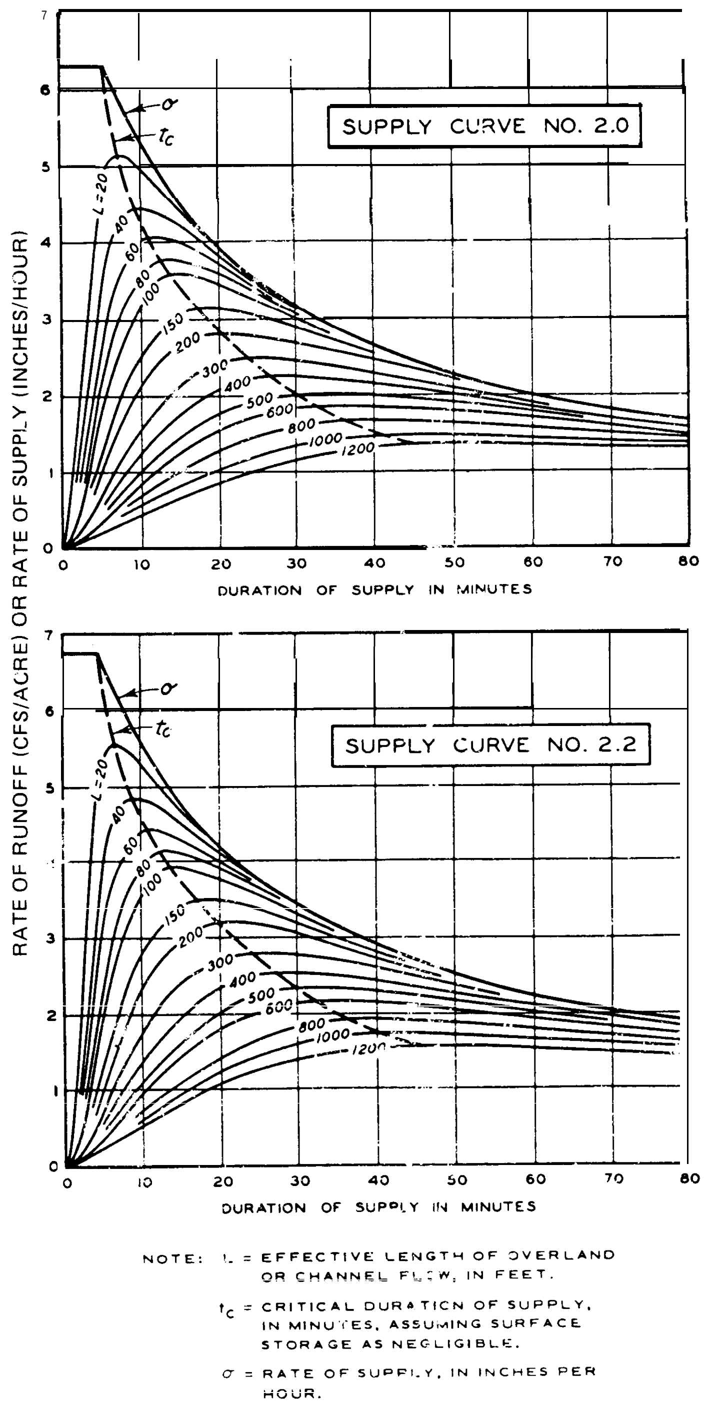

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Figure B-11. Rates of runoff corresponding to supply curves No. 2.0 and 2.2; n = 0.40 and S = 1 percent.

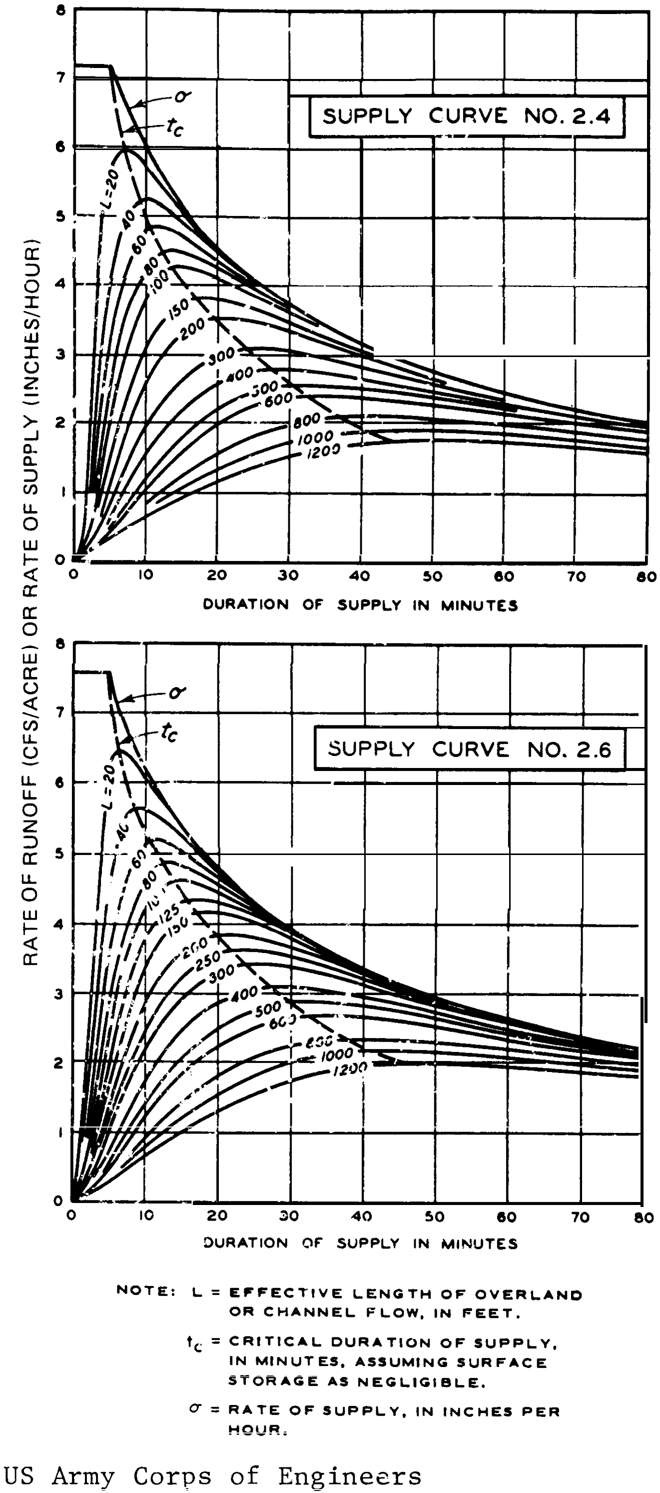
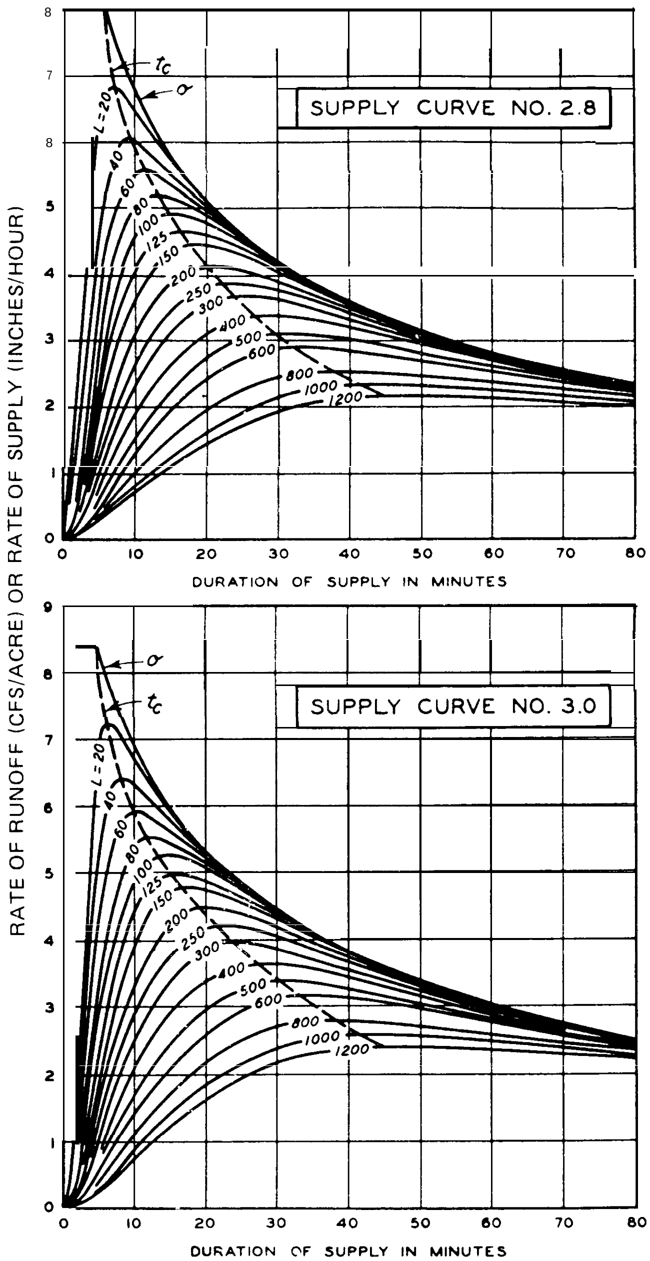


Figure B-12. Rates of runoff corresponding to supply curves No. 2.4 and 2.6; n = 0.40 and S = 1 percent.



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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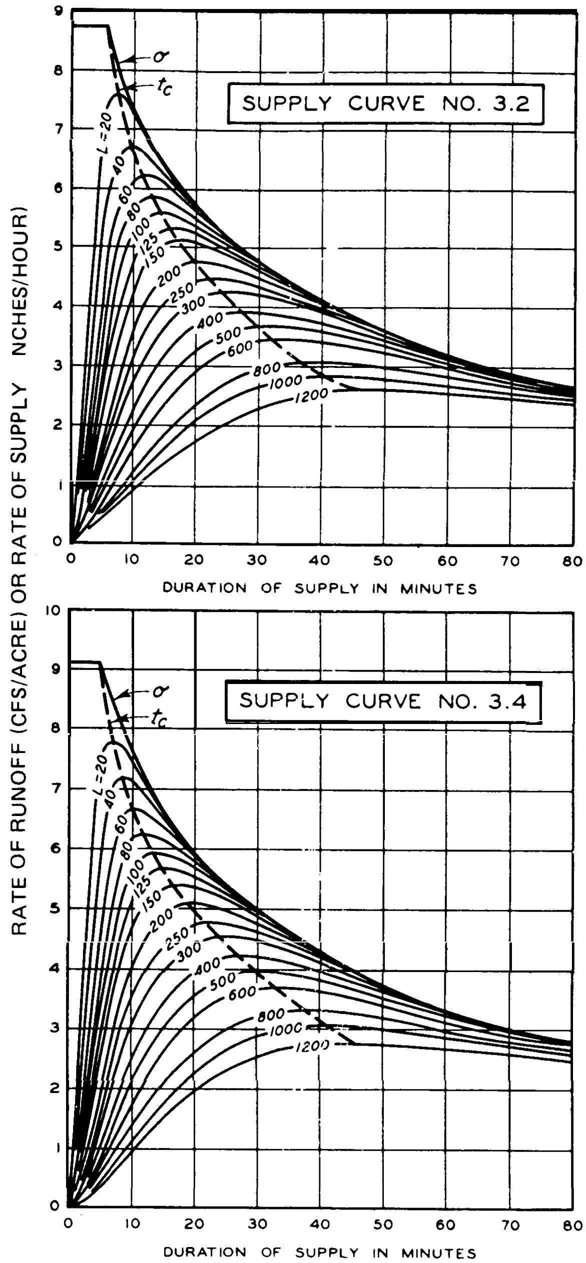
Figure B-13. Rates of runoff corresponding to supply curves No. 2.8 and 3.0; n = 0.40 and S = 1 percent.



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 B-14. Rates of runoff corresponding to supply curves No. 3.2 and 3.4; n = 0.40 and S = 1 percent.

Table B-3. Rates of runoff corresponding to intensities and durations of supply represented by standard supply curve No. 2 in figure 2 (n = 0.40, S = 1 percent).

(1) Duration of supply minutes	(2) Rate of supply inches/hour (scaled from curve No. 2.0, figure 2)	(3) Rate of runoff (in inches/hour or cubic feet/second/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	(4) 60	(5) 100	(6) 200	(7) 300	(8) 400	(9) 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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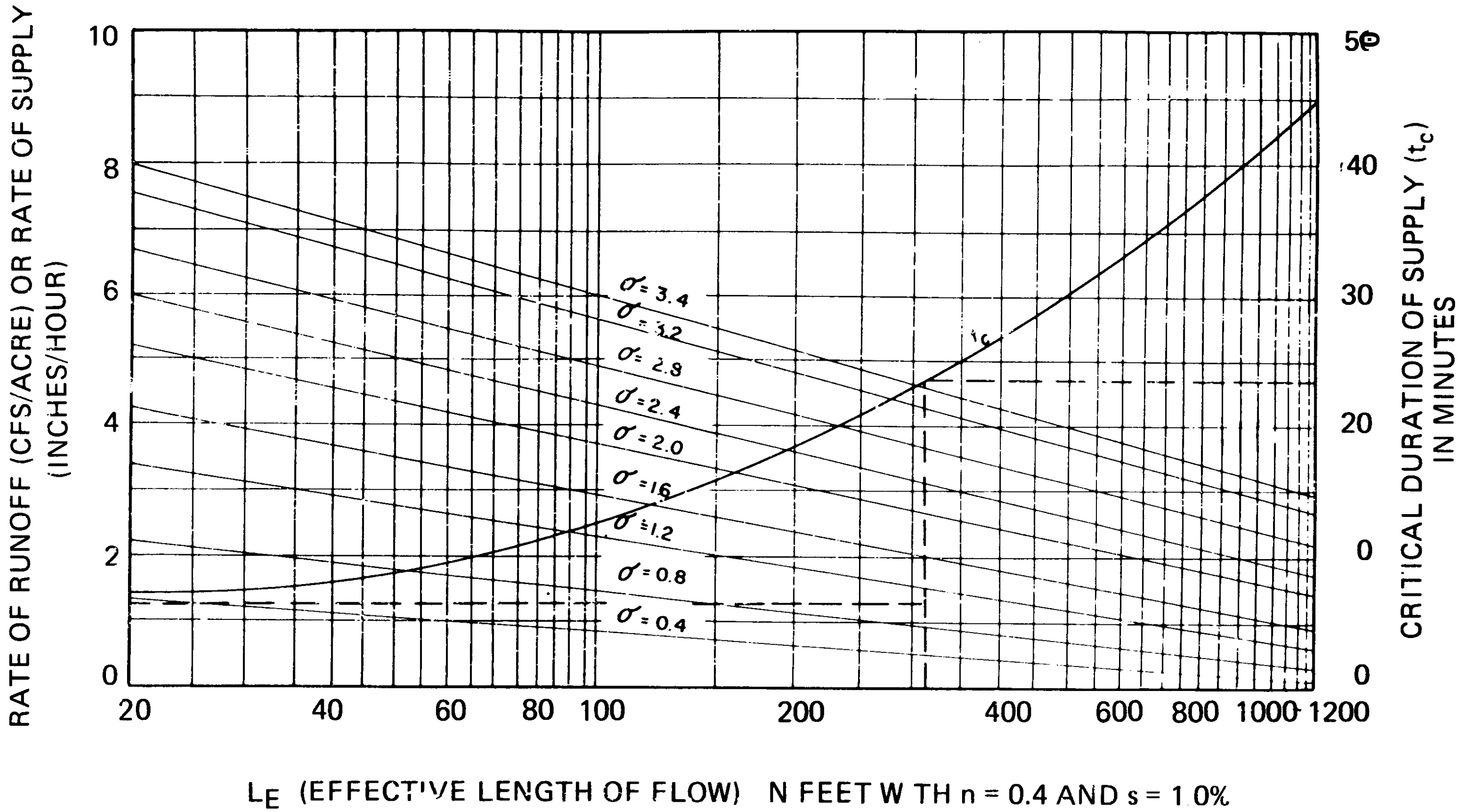

σ = Rate of Supply

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Figure B-15. Consolidated design curve composite of peak runoff rates and critical durations of supply shown in figures B-7 through B-14.

which, in this example, is 23 minutes. 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.

B-4. Storage.

a. Temporary storage or pending. 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 pending 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.

(1) The rate of outflow from a pending 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.

(2) To illustrate these assumptions, reference is made to the curves shown in figure B-16 and the computations in table B-4. Hydrography 1 and 2 are developed as for figure B-5. Hydrograph 3 of figure B-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 B– 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 minutes after the beginning of runoff.

b. Drain-inlet capacity-storage diagrams. The concepts presented by G. A. Hathaway (American Society of Civil Engineers, *Transactions,* Vol 110) and discussed in a(1) and (2) above have been included in the preparation of figures B-17 through B-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 B–17 through B-21 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 feet.

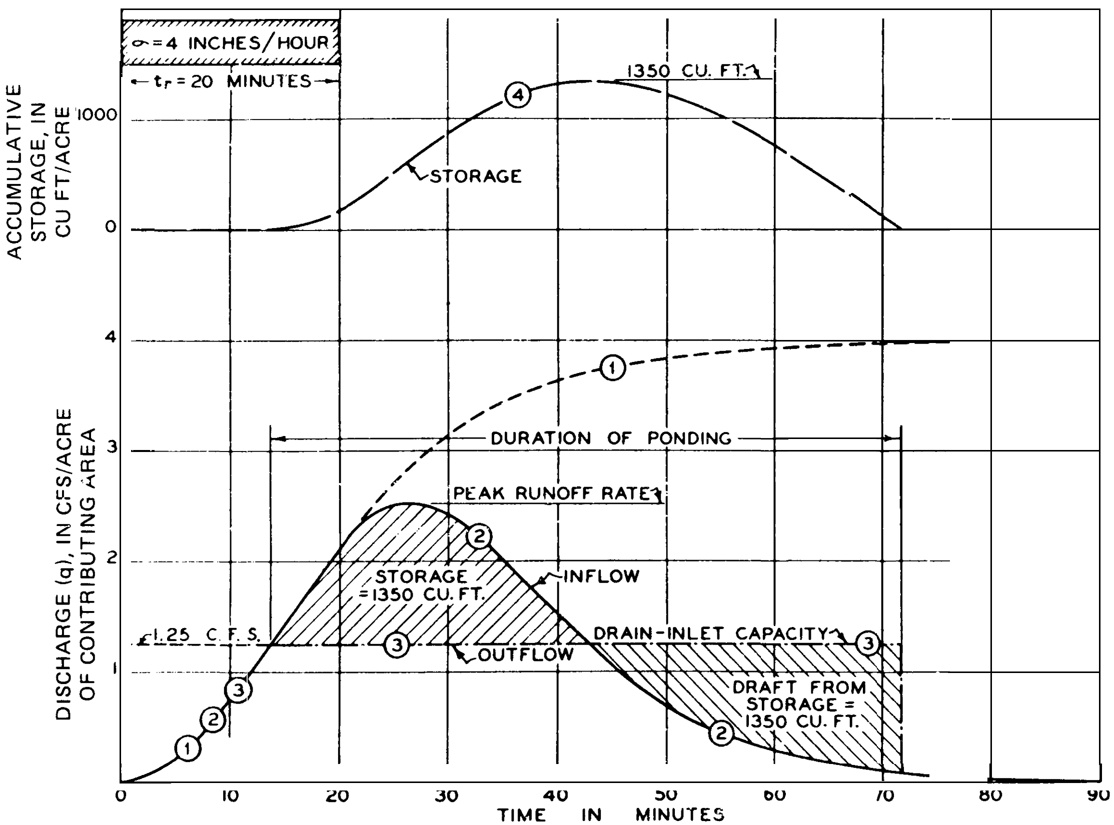
–Maximum storage allowable = 1,000 cubic feet per acre (cu ft/acre) of drainage area.

–Rate of supply = 3.0 inches per hour.

(1) From the 3.0 inches per hour line on the top portion of figure B-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=, 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 minutes.

(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 B–19, for an effective length of 400 feet, offers a quick check on the example presented in table B –4 and figure B-16.

c. *Minimum drain-inlet capacity.* Curve 4 in diagram A (figs B– 17 through B–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 hours on the standard supply curves given in figure 2. If the drain-inlet capacity indicated by curve 4 is adopted in a particular case, some storage may result in the pending basin during all storms less than 4 hours in duration that produce rates corresponding to the given standard supply curve.



NOTE: L = 400 FT., S = 1%, n = 0.40

LENGTH "L" WAS CHOSEN TO REPRESENT WEIGHTED MEAN DISTANCE FROM LIMITS OF DRAINAGE AREA TO AVERAGE LIMITS OF POND.

HYDROGRAPH 1 REPRESENTS RUNOFF THAT WOULD HAVE OCCURRED IF GIVEN RATE OF SUPPLY OF 4 IN. PER HR. HAD CONTINUED INDEFINITELY.

HYDROGRAPH 2 CORRESPONDS TO RUNOFF FROM SUPPLY OF 4 IN. PER HR. FOR DURATION OF 20 MIN.

HYDROGRAPH 3 REPRESENTS RATE OF OUTFLOW, ASSUMING INLET CAPACITY AS DESIGNATED AND CONSTANT HEAD-DISCHARGE RELATION.

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Figure B-16. Sample computations of storage required with selected drain-inlet capacity to provide for runoff from an acre of turf under assumptions designated.

Table B-4. Design example.

Duration of supply min	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 incre: ment cu ft 6	Total storage cu ft
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 B-3.

[b]Difference between columns 2 and 3.

[c]Example for 20- to 25-minute increment.

$$V = [(2.2 - 1.25) + (2.5 - 1.25)]/2 \times (5 \times 60) = 330 \text{ cubic feet.}$$

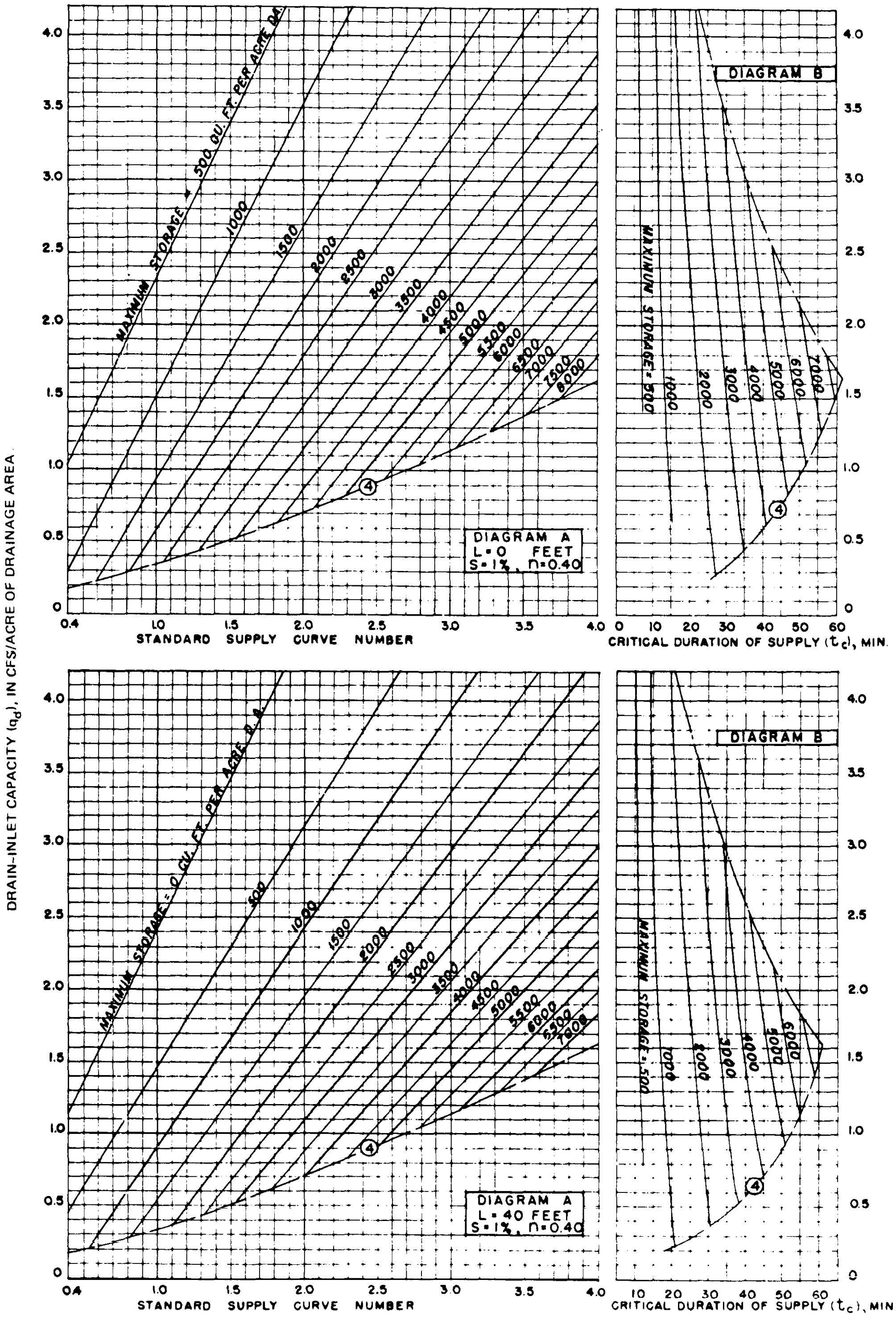

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Figure B-17. Drain-inlet capacity versus maximum surface storage; L = 0 and 40 feet.

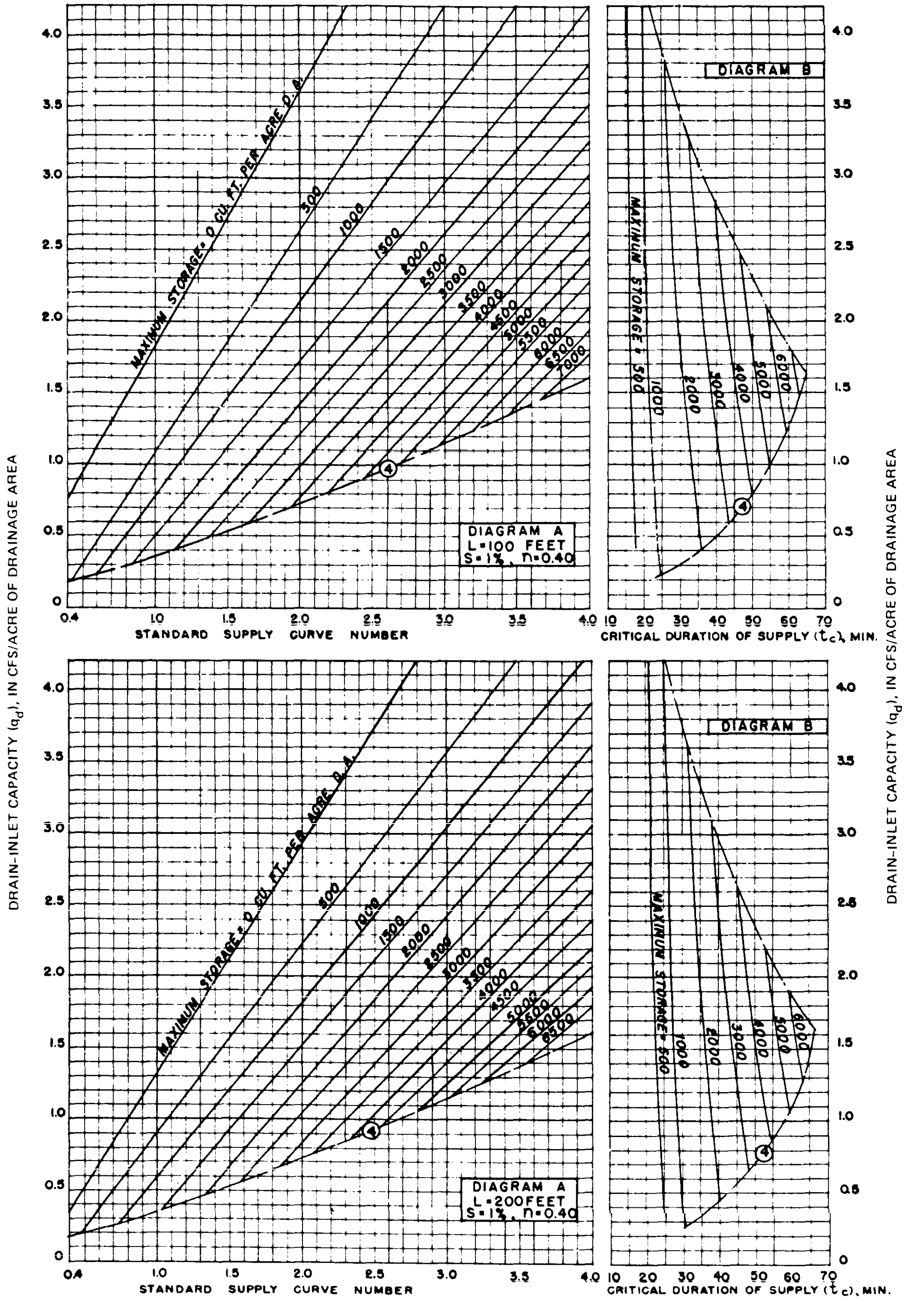

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Figure B-18. Drain-inlet capacity versus maximum surface storage; L = 100 and 200 feet.

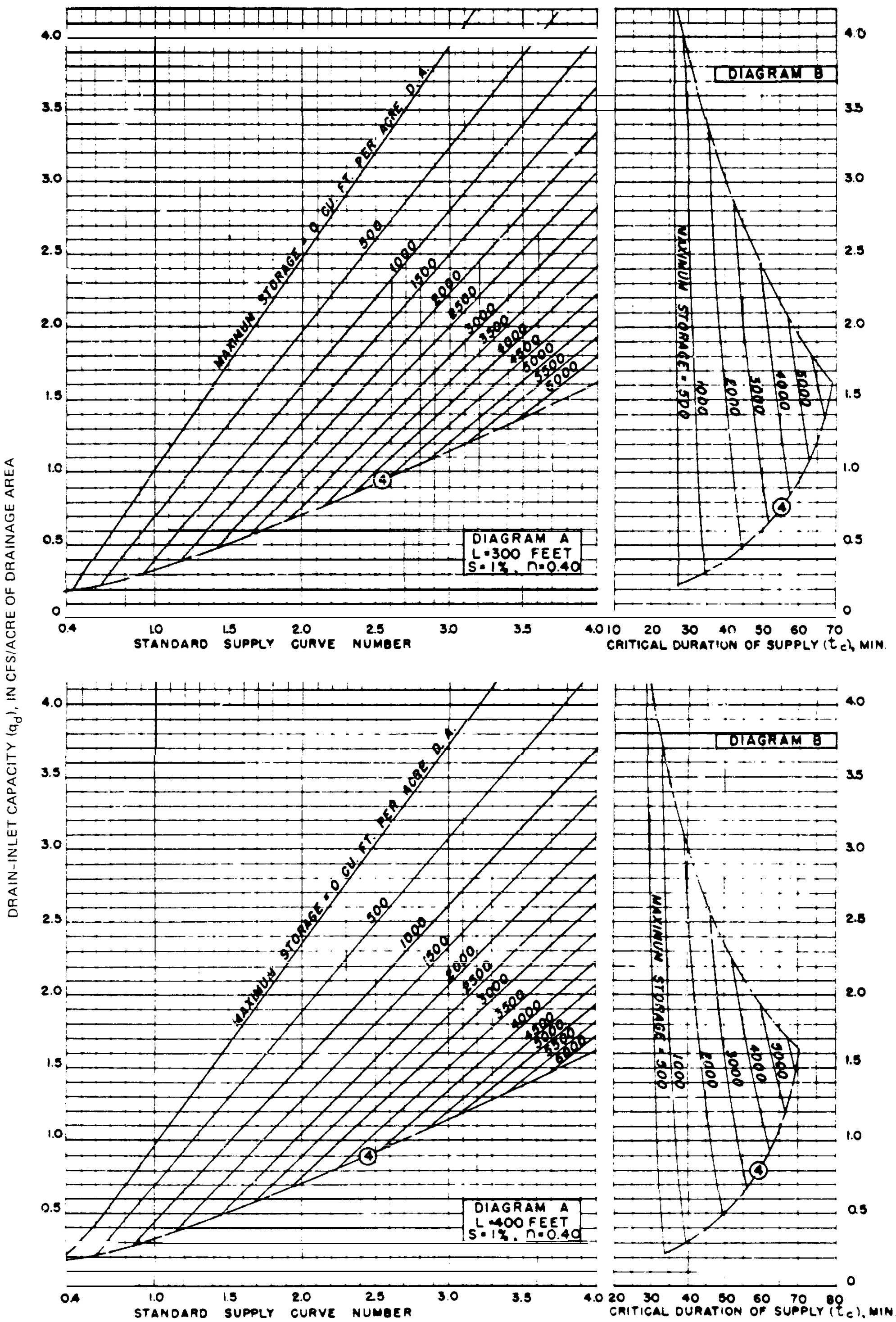

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Figure B-19. Drain-inlet capacity versus maximum surface storage; L = 300 and 400 feet.

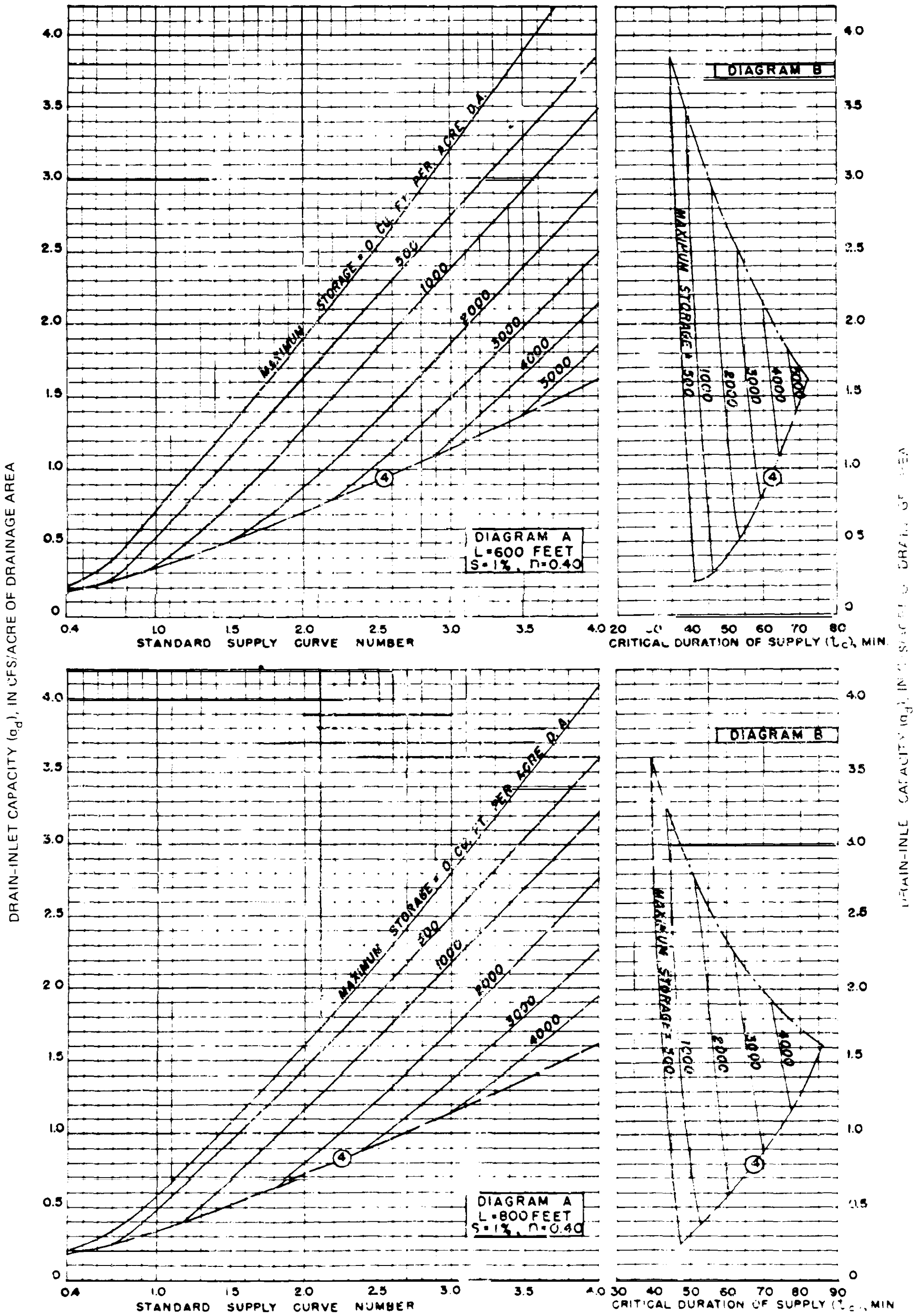

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Figure B-20. Drain-inlet capacity versus maximum surface storage; L = 600 and 800 feet.

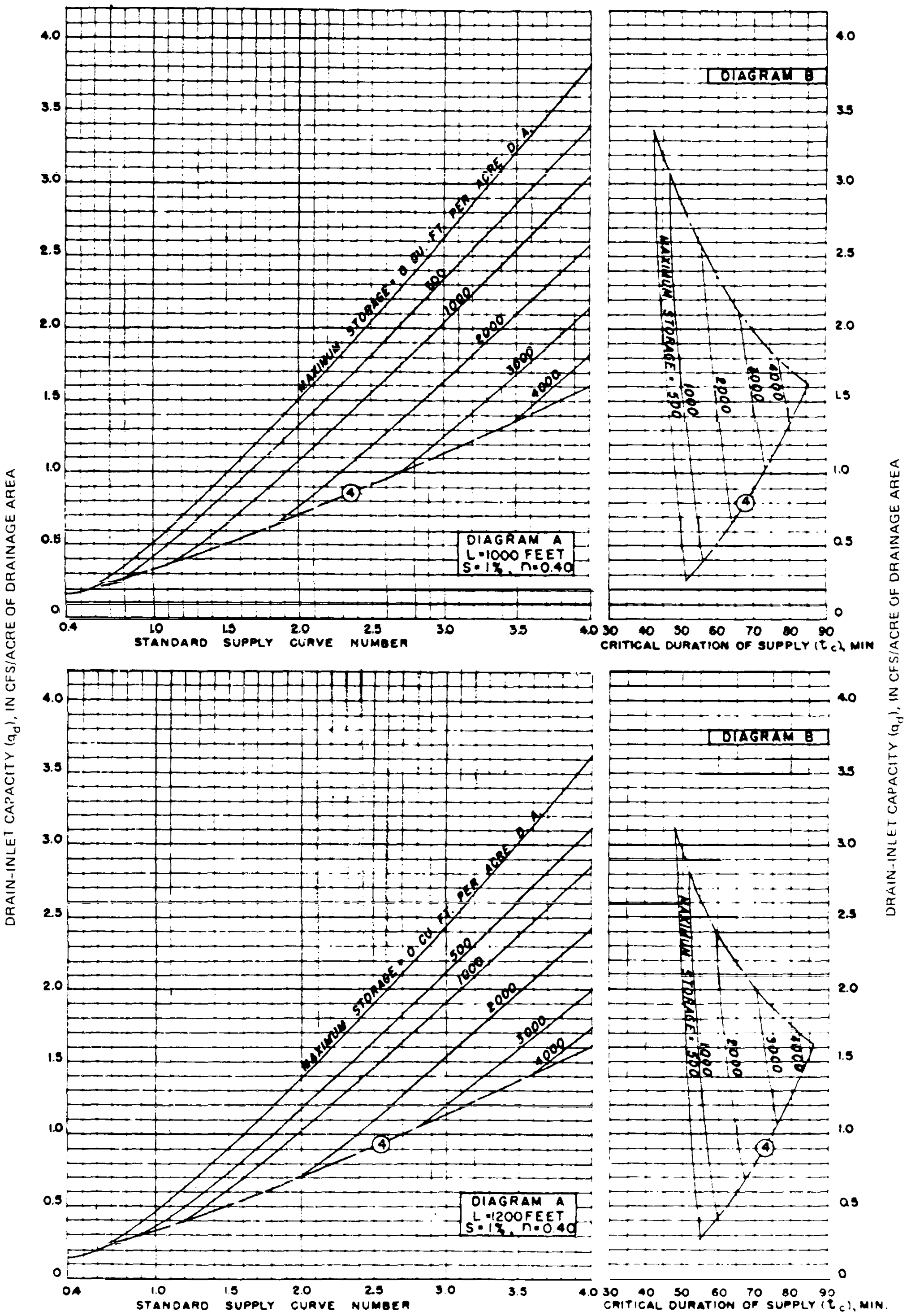

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Figure B-21. Drain-inlet capacity versus maximum surface storage; L = 1,000 and 1,200 feet.

B-5. Drain-inlet and drain capacities.

a. Determination of drain-inlet capacities without pending. From figures B-7 through B-14, 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 B-15 is used, the same data can be obtained by following the procedure described in paragraph B-3c.

(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

$$t_c' - t_c + t_d \qquad \text{(eq B-4)}$$

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.

(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 minutes. Inspection of figures B-7 through B-14 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 O storage capacity line of diagram A of figures B-20 and B-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.

b. Determination of drain-inlet capacities with temporary pending. From figures B-17 through B-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 pending. In a drainage system where pending 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 pending takes place as shown in figure B-16. Curve 4 in figures B-17 through B-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 hours on the respective standard supply curve given in figure 2. If the drain-inlet capacity indicated by curve 4 is adopted in a particular case, some storage may result in the pending basin during all storms less than 4 hours 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.

c. 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 B-22 through B-25. Storm drains will have a minimum diameter of 12 inches to lessen possibilities of clogging. Design of drain-inlet facilities is discussed in TM 5-820-3/AFM 88-5, Chap. 3.

(1) For conditions of instantaneous runoff the hydraulic gradient will be kept at the top of the pipe. Where temporary pending 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.

(2) Where flooding from a temporary pending 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 pending 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.

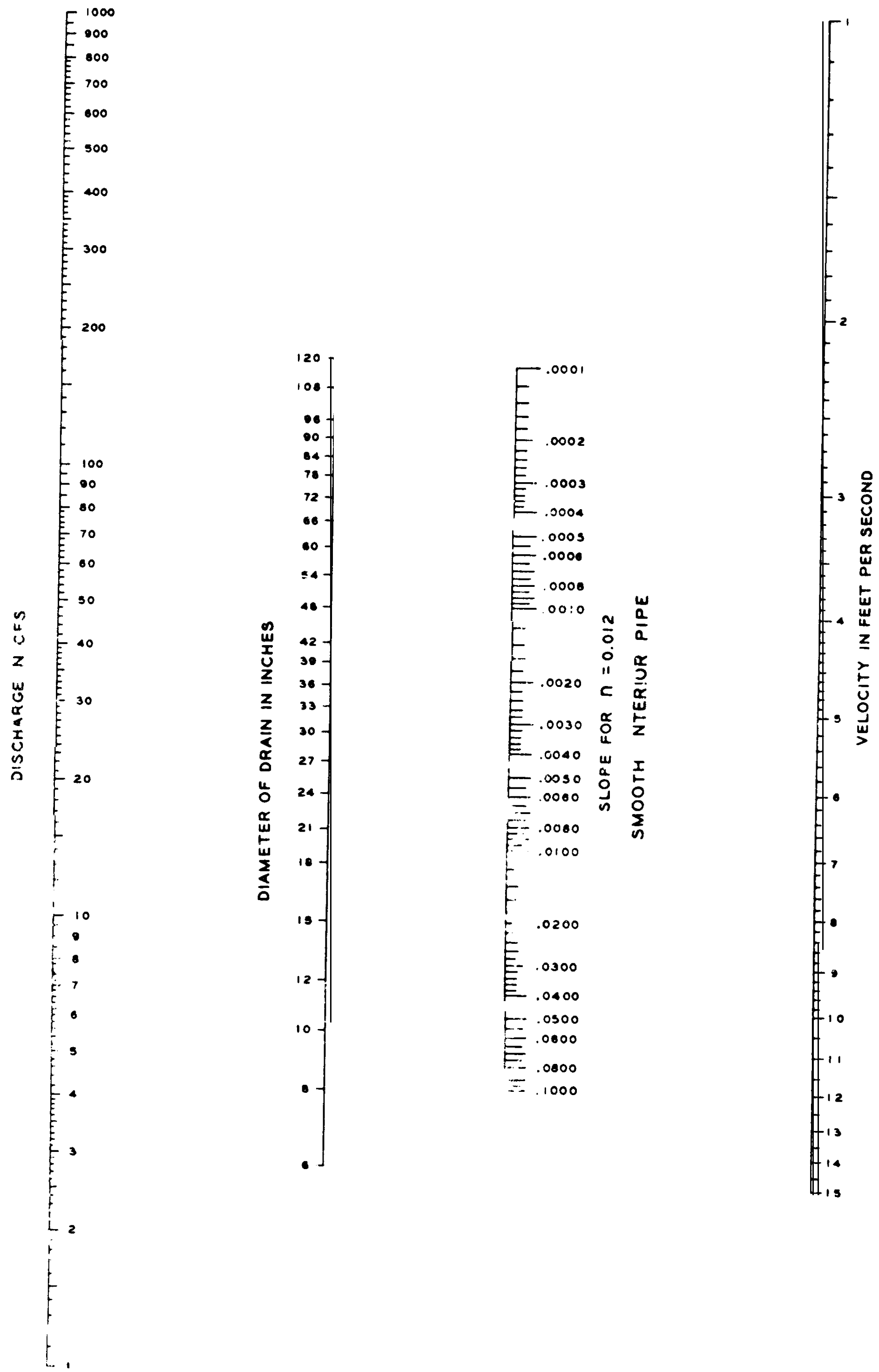

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Figure B-22. Nomograph for computing required size of circular drain smooth interior, flowing full; n = 0.012.

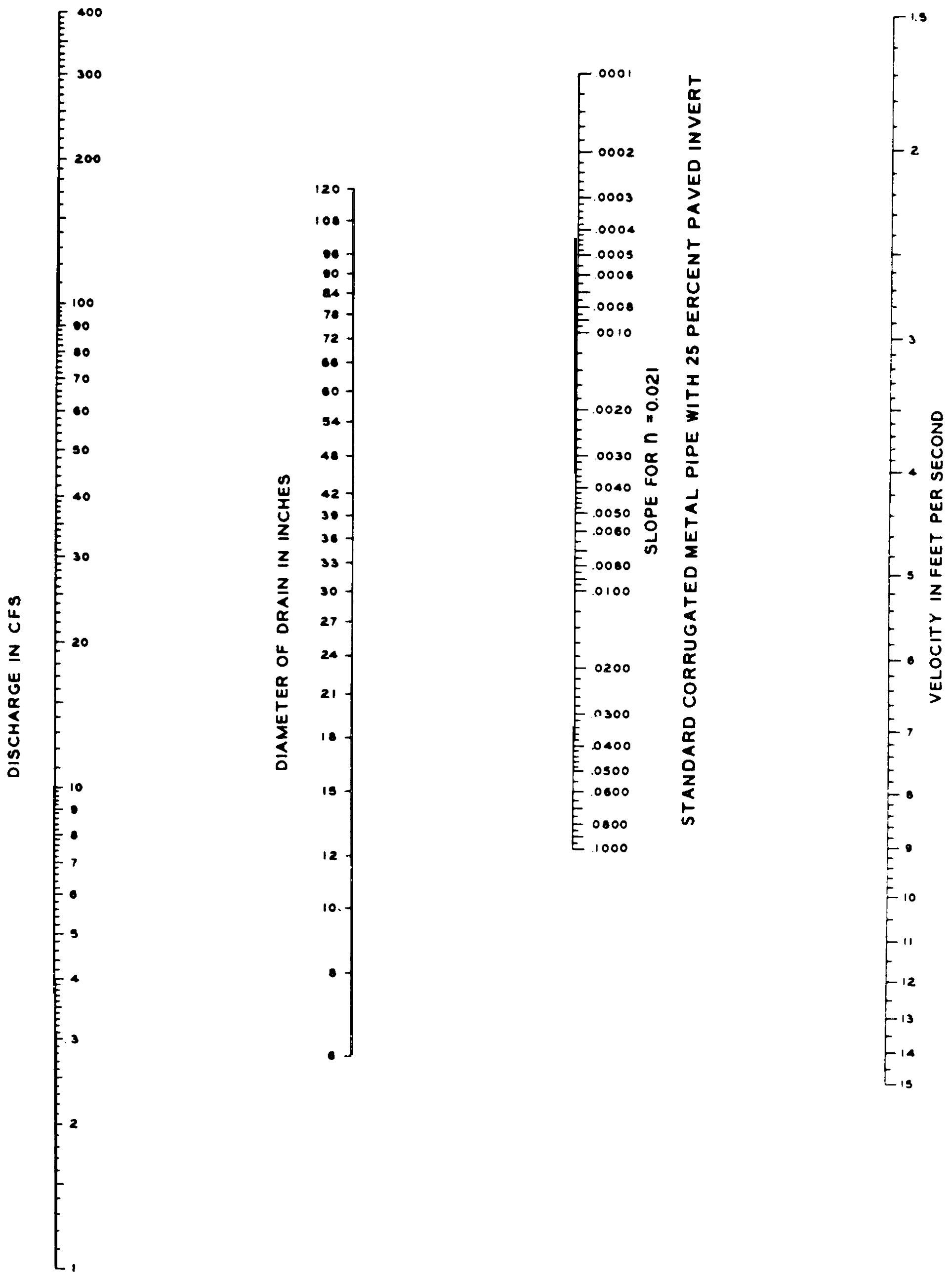

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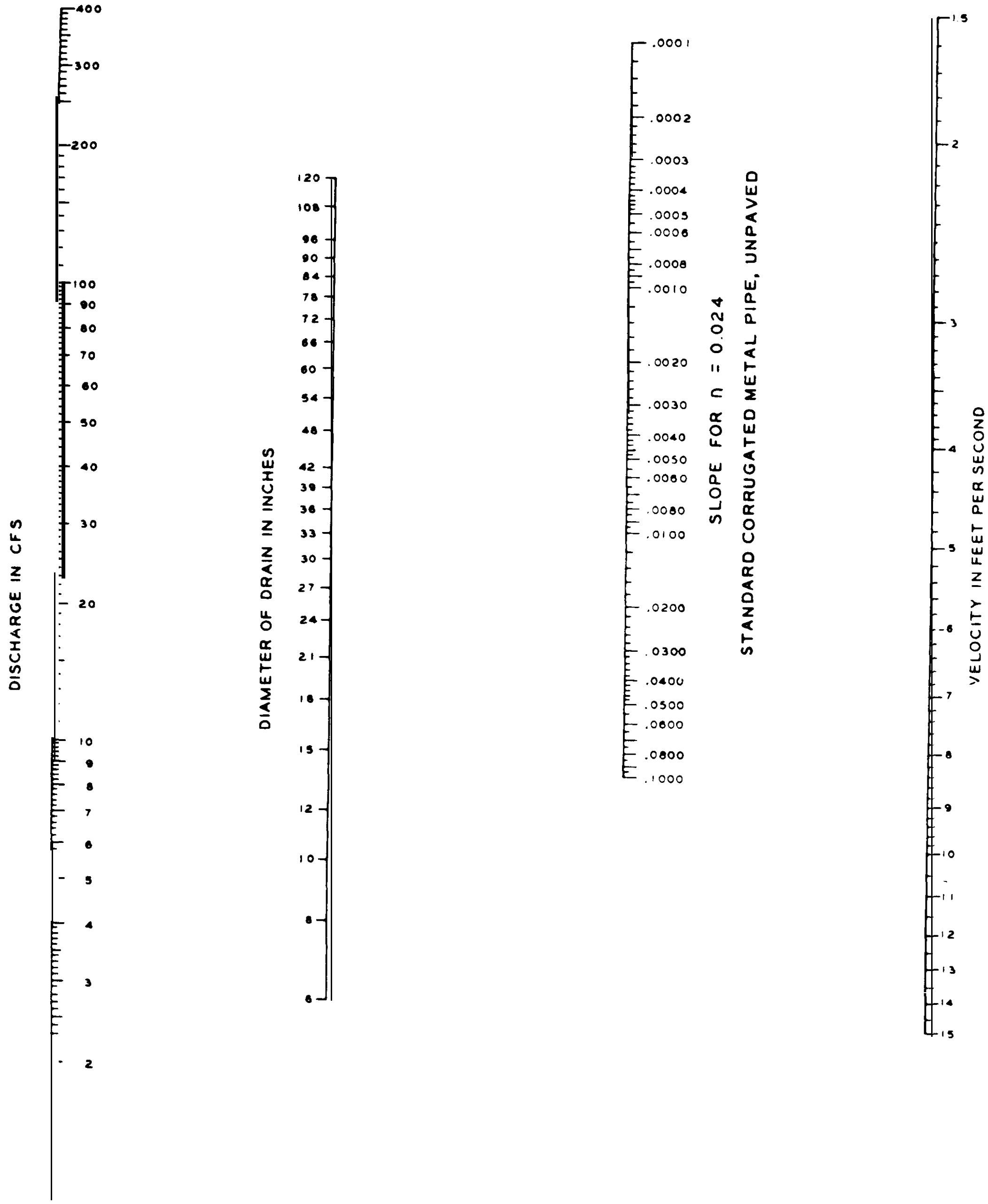
Figure B-23 Nomograph for computing required size of circular standard corrugated pipe, 25 percent paved invert, full; n = 0.021.



US Army Corps of Engineers

Figure B-24. Nomograph for computing required size of circular standard corrugated metal pipe, unpaved, flowing full; n = 0.024.

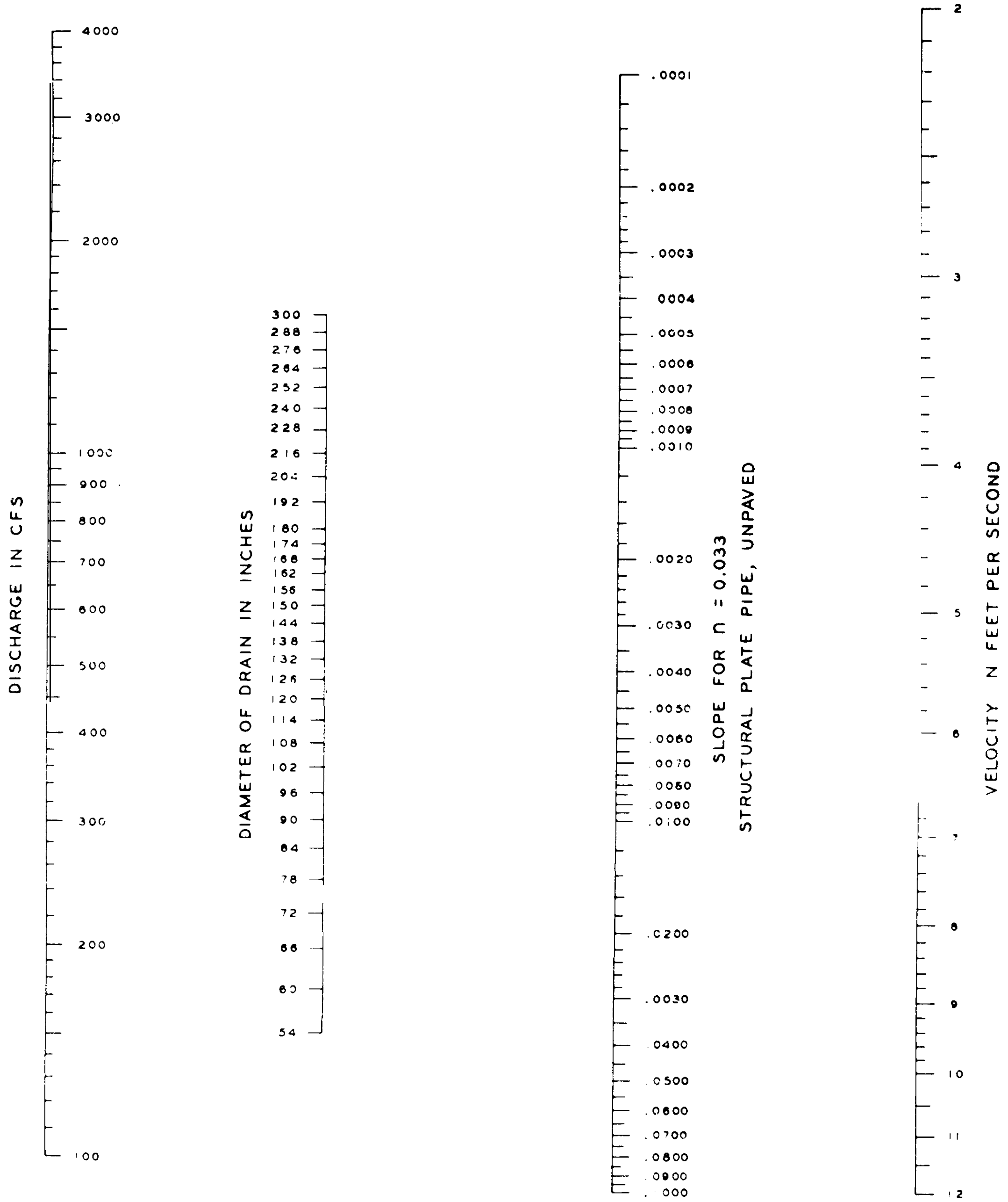


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Figure B-25. Nomograph for computing required size of circular structural plate pipe, unpaved, flowing full; n = 0.033.

APPENDIX C
DESIGN EXAMPLE

C-1. Design example.

a. The proposed layout for the primary storm drainage system for an airfield is depicted in figure C-1. This airfield is to be located in central Mississippi where the design storm index for a 2-year l-hour rainfall intensity, according to figure 1, is 2.0 inches per hour. The duration of storm being considered is 60 minutes; thus, figure 2 (see main text) need not initially be used. Infiltration values for the paved and turfed area are considered to be 0.0 and 0.5 inches per hour, respectively, according to paragraph 6 of the main text. The supply curves applicable to this airfield are No. 2.0 for paved areas (2.0–0.0) and No. 1.5 for turfed areas (2.0—0.5). These supply curves are provided in figure 2. Coefficients of roughness have been selected for the paved and turfed areas as 0.01 and 0.40, respectively, as suggested in table B-2.

b. In this example, two conditions are considered: where pending is permissible at inlets 4, 3, and 2, and where no pending is allowed at these inlets. The purpose of these examples is to portray the difference in pipe size requirements under these two imposing conditions. Tables C-1, C-2, and C-3 reflect the design where pending is permissible, and tables C-4, C-5, and C-6 reflect the design where pending is not acceptable.

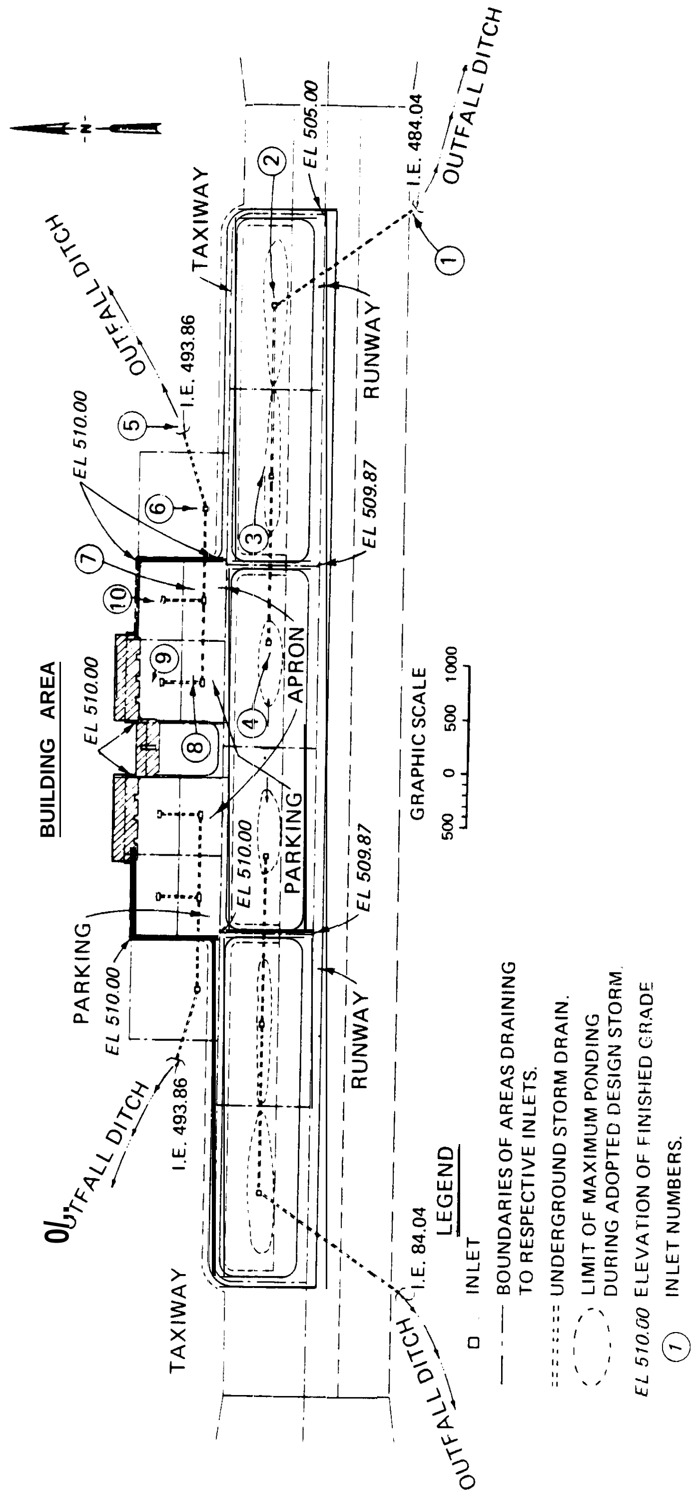

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Figure C-1. Sample computations of layout of primary storm drainage system.

Table C-1. Airfield drainage drain-inlet capacities required to limit ponding to permissible volumes with drainage section on east side of airfield and with supply curves No. 2.0 for paved areas and 1.5 for turfed areas.

INLET NO.	DRAINAGE AREA (D.A.) IN ACRES				PERMISSIBLE PONDING						LENGTH L (FT.)		
	Paved	Unpaved Bare	Unpaved Turf	Total	Depth at Inlet, ft.	Pond area, Thousands of Sq. Ft.	Volume Thousands of Cu. Ft.	Volume, Cu. ft. Per Acre D.A.	Average Coefficient of Roughness "n"	Average Slope "S" In Percent	Actual Length Ft.	Effective L for n = .40 and S = 1%	L Adopted for Selecting Diagrams
	n= 0.01	n=	n= 0.40										
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
10	5.93			5.93					.01	1.0	400	30	30
9	7.40			7.40					.01	1.0	425	30	30
				.93									
7	5.93			5.93					.01	1.0	400	30	30
6[a]	1.41		17.18	18.59					.37	1.5	775	580	580
4	5.97		26.81	32.78	3.00	138	207	6310	.33	2.0	525	310	310
3	5.69		25.54	31.23	1.73	145	125	4000	.33	2.8	340	170	170
2	5.69		25.54	31.23	2.73	270	369	11800	.33	2.8	340	170	170
Notes:	1		1	1	2	2	3	4	5	6	6	7	

[a]No ponding permitted.

(Continued)

(Sheet 1 of 4)

Table C-1. Airfield drainage drain-inlet capacities required to limit ponding to permissible volumes with drainage section on east side of airfield and with supply curves No. 2.0 for paved areas and 1.5 for turfed areas—continued.

STANDARD SUPPLY CURVE NO. x D.A.				Weighted Supply Curve (Col. 18 ÷ Col. 5)	DRAIN-INLET CAPACITY			CRITICAL CONTRIBUTION TO SYSTEM		
Paved Areas	Unpaved: Bare	Unpaved: Turf	Total		t_c In Minutes	q_d In c.f.s. Per Acre of D.A.	Q_d In c.f.s. (Col. 21 x Col. 5)	t'_c In Minutes	q_d In c.f.s. Per Acre of D.A.	Q_d In c.f.s. (Col. 24 x Col. 5)
(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)
11.86			11.86	2.0	10	4.7	27.9	15	4.3	25.5
								20	3.8	22.5
								30	3.2	19.0
14.80			14.80	2.0	10	4.7	34.8	15	4.3	31.8
								20	3.8	28.1
								30	3.2	23.7
11.86			11.86	2.0	10	4.7	27.9	15	4.3	25.5
								20	3.8	22.5
								30	3.2	19.0
11.86			11.86	2.0	10	4.7	27.9	15	4.3	25.5
								20	3.8	22.5
								30	3.2	19.0
2.82		25.77	28.59	1.5	32	1.3	24.2	20	1.2	22.3[a]
11.94		40.21	52.15	1.6	b	0.51	16.7			
11.38		38.31	49.69	1.6	b	0.51	15.9			
11.38		38.31	49.69	1.6	b	0.51	15.9			
Notes:					8	9		10	11	

[a] No ponding permitted.

[b] Not required when appreciable ponding is permissible.

(Continued)

(Sheet 2 of 4)

Table C-1. Airfield drainage drain-inlet capacities required to limit ponding to permissible volumes with drainage section on east side of airfield and with supply curves No. 2.0 for paved areas and 1.5 for turfed areas—continued.

Notes:

1. Both paved and turfed drainage areas have been previously computed and are provided for this example.
2. Permissible depth and area of temporary storage pond have been previously computed and are provided for this example.
3. Total ponding volume for inlet 4 is computed as follows:

$$v = (0 + 138{,}000)/2 \times 3.0 = 207{,}000 \text{ cu ft}$$

4. Ponding volume for inlet 4 per acre of drainage area is computed as follows:

$$v = 207{,}000/32.78 = 6315 \text{ cu ft/acre of drainage area}$$

5. Average coefficient of roughness "n" for inlet 4 is computed as follows:

$$n_w = [(0.01 \times 5.97) + (0.40 \times 26.81)]/(5.97 + 26.81) = 0.33$$

6. Average slope and actual length of drainage area to inlet have been previously determined and are provided for this example.
7. See figure B-6 for determination of this length.
8. For inlets 10, 9, 8, and 7, consult either figure 3 (main text) or B-14. The critical duration of supply t_c for L = 30 feet is about 8 minutes; however, paragraph B-3b specifies that minimum allowable t_c for paved areas is 10 minutes. For inlet 6, consult figures B-9 and B-10 or figure B-15 to determine the stated value.

(Continued) (Sheet 3 of 4)

Table C-1. Airfield drainage drain-inlet capacities required to limit ponding to permissible volumes with drainage section on east side of airfield and with supply curves No. 2.0 for paved areas and 1.5 for turfed areas—continued.

9. For inlets 10, 9, 8, and 7, the drain-inlet capacities q_d are determined from figure 3 or B-15. Figure B-17 could be used if the maximum storage line, 0 cu ft/acre were extended. The drain-inlet capacity for inlet 6 is determined from figures B-9 and B-10, B-15, or B-20. Drain-inlet capacities for inlets 4, 3, and 2 where ponding is permissible are determined from the 4-line in figures B-18 and B-19. The 4-line is used because the permissible volume of storage exceeds the net volume of runoff for a 4-hour storm having the stated rate of supply.

10. Values for duration from critical inlet t'_c are described in table C-2.

11. Values of q_d are determined for inlets 7, 8, 9, and 10 from figure 3, the weighted supply curve 2.0 (column 19), using the 15, 20, and 30-minute values of t'_c (column 23), and the effective length, L, of 30 feet (column 14). A similar procedure is followed for inlet 6 using only a 20-minute value of t'_c and the effective length, L, of 580 feet. (See note 5 for table C-2). The q_d values are read on the vertical axis labeled maximum rate of runoff. Figure B-15 cannot be used in determining these values.

(Sheet 4 of 4)

Table C-2. Airfield drainage size and profile of underground storm drains with drainage section on east side of airfield and with supply curves No. 2.0 for paved areas and 1.5 for turfed areas.

POINT OF DESIGN			Critical Runoff Time To Produce Maximum Flow In Undergound Drain						
	Distance, Feet					Drain Time, Min.			
Inlet or Junction Number	From Main outlet	From Preceding Inlet	Critical Inlet	Critical Inlet Time, t_c Min.	Assumed Velocity In Pipe, Ft./Sec.	From Preceding Inlet	Accum. Total	Approx. t'_c (Col. 5 + Col. 8)	Adopted t'_c In Minutes
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
9	2740		9	10				10	10
8	2355	385	9	10	3.0	2	2	12	10
7	1580	775	9	10	3.0	4	6	16	15
6	730	850	9	10	3.0	5	11	21	20
6	730	850	9						30
10	1965		10	10				10	10
4	4805								
3	3155	1650							
2	1505	1650							
Notes:	1	1	2	3		4			5

(Continued)

(Sheet 1 of 3)

Table C-2. Airfield drainage size and profile of underground storm drains with drainage section on east side of airfield and with supply curves No. 2.0 for paved areas and 1.5 for turfed areas—continued.

Rate of Inflow Into Underground Drains, In C.F.S., Corresponding to Adopted Value of t'_c (Column 10)

Inlet Number

9 (11)	8 (12)	7 (13)	10 (14)	6 (15)	(16)	4 (17)	3 (18)	2 (19)	(20)	(21)	(22)	Total (23)
34.8												34.8
34.8	7.8											62.6
31.8	5.5	25.5	25.5									108.3
28.1	2.5	22.5	22.5	22.3								117.9
23.7	9.0	19.0	19.0	24 2								104.9
			27.9									27.9
						16.7						16.7
						16.7	15.9					32.6
						16.7	15.9	15.9				48.5
Notes: 6	6	6	6	6		6	6	6				

(Continued)

(Sheet 2 of 3)

Table C-2. Airfield drainage size and profile of underground storm drains with drainage section on east side of airfield and with supply curves No. 2.0 for paved areas and 1.5 for turfed areas—continued.

Notes:

1. Distances between inlets and from any inlet to the main outlet have been previously determined and are provided for this example.
2. Normally the most distant inlet in each drainage line from the main outlet.
3. Refers to t_c for the most critical inlet and not the t_c for the numbered inlet.
4. Drain time from inlet 9 to 8 is calculated as follows:

$$385/(3 \times 60) \simeq 2 \text{ minutes}$$

5. Adopted t'_c values are rounded to the nearest 5 minutes in accordance with paragraph B-3b. These values reflect the total time required for a quantity of water to travel from the most remote portion of the critical inlet area to the inlet being considered. To determine the value of t'_c for inlet 6, the total flow at that point must be computed for each condition to ascertain which is the largest and governing rate of flow into the system. In this example, the maximum flow is associated with a time of concentration (20 minutes) from the upstream areas rather than the time of concentration (30 minutes) for the area contributing to inlet 6.
6. Values used are obtained from table C-1, column 25. These values reflect the total quantity of water contributing to the entire system at any given inlet in the system. At inlet 6 rates of inflow are provided based on the critical duration of supply time for both the critical inlet (inlet 9) and the inlet under consideration, because the t'_c for inlet 6 is greater than the t'_c for the entire system. These calculations indicate the rate of inflow for t'_c is the more critical or governing value.

(Sheet 3 of 3)

Table C-3. Hydraulic design data on underground storm drains where ponding is permissible.

Inlet or Junction Numbers		Distance Between Design Points, In Feet, Measured to ℄ of Inlet or Junction	Trial Design				
rom	To		Design Discharge Capacity In c.f.s.	Approx. Gradient Between Design Points, Ft./Ft.	Mean Velocity of Design Discharge In Ft./Sec.	Required Size of Pipe, In Inches	Selected Size of Pipe, In Inches
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1	2	1505	48.5	0.0020	5.1	41	42
						51	48[a]
2	3	1650	32.6	0.0020	4.6	36	36
						44	42[a]
3	4	1650	16.7	0.0014	3.4	29	30
						37	36[a]
5	6	730	117.9	0.0018	6.1	59	60
						74	72[a]
6	7	850	108.3	0.0026	6.8	53	54
						66	66[a]
7	8	775	62.6	0.0033	6.5	42	42
						51	48[a]
8	9	385	34.8	0.0024	4.9	36	36
						44	42[a]
7	10	385	27.9	0.0040	5.7	29	30
						37	36[a]
Notes:			1	2	3	4	5

[a]Pipe diameters required using "n" = 0.021 for 1/4d paved invert corrugated metal pipe. Size differential limited to 6 inches where feasible.

(Continued)

(Sheet 1 of 5)

Table C-3. Hydraulic design data on underground storm drains where ponding is permissible—continued.

Adopted Design						Construction Data			
Losses				Hydraulic Gradient At Inlet				Elevation of Invert at Design Point, (Col. 2) in Ft.	
Velocity Head $V^2/2g$	Inlet Coefficient K	Inlet Loss, In Feet (Col. 9 x Col. 10)	Friction Loss, In Feet	Outgoing Pipe	Incoming Pipe	Slope In Ft./Ft	Length of Pipe Between Design Points In Feet	Incoming Pipe	Outgoing Pipe
(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)
				487.54					484.04
0.39	0.12	0.05	3.00	490.59	490.09	0.0020			
0.33	0.12	0.04	3.55	493.68	493.18	0.0020			
0.18	0.12	0.02	2.33	495.53		0.0014			
				498.86					493.86
0.58	0.12	0.07	1.32	500.25	499.75	0.0018			
0.74	0.12	0.09	2.25	502.09	501.09	0.0026			
0.66	0.12	0.08	2.56	503.73	503.23	0.0033			
0.39	0.12	0.04	0.92	504.19		0.0024			
				499.09					
0.50	0.12	0.06	1.57	500.72		0.0040			
Notes:	6		7	8	9				

(Continued)

(Sheet 2 of 5)

Table C-3. Hydraulic design data on underground storm drains where ponding is permissible—continued.

Notes:

1. Values come from table C-2, column 23.
2. Values developed from profile of drainage line and have been provided for this example.
3. Values derived from use of Manning's formula in nomograph form (fig. B-22) for smooth interior pipe flowing full. The velocities indicated are higher than those assumed in table C-2, column 6, and in an actual problem these new values would be used in table C-2 for a second trial to develop new rates of inflow. Seldom are more than three iterations warranted.
4. Values derived from the use of figures B-22 and B-23.
5. Values in column 8 rounded off to nearest commercially available pipe size.
6. Inlet coefficients are available in many publications; however, the following general criteria are satisfactory for most airfield storm drainage systems:

Entrance loss coefficients, K, inlets

	Rectangular		Circular	
Slope percent	Socket of Pipe Projecting Slightly into Riser	Square-Edge Pipe Opening into Riser	0.25D Radius	Square Edge
0	0.12	1.2	-	-
10	0.13	0.87	-	-
20	0.13	0.82	0.50	0.66

(Continued)

(Sheet 3 of 5)

Table C-3. Hydraulic design data on underground storm drains where ponding is permissible—continued.

7. Friction loss can be computed by the formula:

$$H_f = 2.88n^2V^2L/D^{1.333} \quad \text{or} \quad H_f = fLV^2/(D \times 2g)$$

The factor f can be obtained from many texts; however, the following tabulation provides values sufficient for most cases:

Friction factors

	Values of f			
D (in.)	n = 0.012	n = 0.021	n = 0.024	n = 0.033
6	0.0336	0.1029	0.1344	0.2541
12	0.0267	0.0817	0.1067	0.2017
18	0.0233	0.0713	0.0932	0.1762
24	0.0212	0.0648	0.0847	0.1601
30	0.0196	0.0602	0.0786	0.1486
36	0.0185	0.0566	0.0740	0.1398
42	0.0176	0.0538	0.0703	0.1328
48	0.0168	0.0515	0.0672	0.1271
54	0.0162	0.0495	0.0646	0.1222
60	0.0156	0.0478	0.0624	0.1179
66	0.0151	0.0463	0.0604	0.1143
72	0.0147	0.0449	0.0587	0.1110
84	0.0139	0.0427	0.0558	0.1054

(Continued)

(Sheet 4 of 5)

Table C-3. Hydraulic design data on underground storm drains where ponding is permissible—continued.

8. Hydraulic gradient for the outgoing pipe at an inlet is computed by adding the friction and inlet losses to the hydraulic gradient of the inlet pipe at the previous inlet. The hydraulic gradient at the outfall of the drain is the elevation of the crest of this pipe. When computing hydraulic gradients by this method, start at the outfall end of the storm drain.

9. Hydraulic gradient for incoming pipe at an inlet is equal to the hydraulic gradient of the outgoing pipe plus or minus the change in pipe size. The bottom of the inlet is assumed to be level and the inverts of both pipes at the same elevation.

(Sheet 5 of 5)

Table C-4. Airfield drainage drain-inlet capacities required to limit ponding to permissible volumes with drainage section in which no ponding of runoff is assumed and with supply curves No. 2.0 for paved areas and 1.5 for turfed areas.

INLET NO.	DRAINAGE AREA (D.A.) IN ACRES				PERMISSIBLE PONDING						LENGTH (FT.)		
	Paved	Unpaved		Total									
		Bare	Turf										
	n= 0.01	n=	n= 0.40		Depth At Inlet, Ft.	Pond Area Thousands of Sq. Ft.	Volume Thousands of Cu. Ft.	Volume, Cu. Ft. Per Acre D. A.	Average Coefficient of Roughness "n"	Average Slope "S" In Percent	Actual Length, Ft.	Effective L for n = .40 and S = 1%	L Adopted for Selecting Diagrams
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
4	5.97		26.81	32.78					0.33	2.0	575	330	330
3	5.69		25.54	31.23					0.33	2.8	575	290	290
2	5.69		25.54	31.23					0.33	[illegible]	575	290	290
Notes											1		

Note: 1. Actual length of drainage area to an inlet has been previously determined and is provided for this example. Length is longer than in table C-1 because temporary ponding is not allowed resulting in a net increase in length of runoff.

(Continued)

Table C-4. Airfield drainage drain-inlet capacities required to limit ponding to permissible volumes with drainage section in which no ponding of runoff is assumed and with supply curves No. 2.0 for paved areas and 1.5 for turfed areas—continued.

STANDARD SUPPLY CURVE NO. x D.A.				Weighted Supply Curve (Col. 18 ÷ Col. 5)	DRAIN-INLET CAPACITY			CRITICAL CONTRIBUTION TO SYSTEM		
Paved Areas	Unpaved Bare	Unpaved Turf	Total		t_c In Minutes	q_d In c.f.s. Per Acre of D.A.	Q_d In c.f.s. (Col. 21 x Col. 5)	t'_c In Minutes	q_d In c.f.s. Per Acre of D.A.	Q_d In c.f.s. (Col. 24 x Col. 5)
(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)
11.94		40.21	52.15	1.6	24	1.9	62.3	25	1.9	62.3
								35	1.8	59.0
								40	1.7	55.7
11.38		38.31	49.69	1.6	22	2.0	62.5	25	2.0	62.5
								35	1.9	59.3
								40	1.8	56.2
11.38		38.31	49.69	1.6	22	2.0	62.5	25	2.0	62.5
								35	1.9	59.3
								40	1.8	56.2
Notes:					2	3		4	5	

Notes:
2. Consult either figure B-10 or B-14 to determine the stated value.
3. Drain-inlet capacities, q_d, are determined from figure B-18.
4. Values for duration of supply from critical inlet, t'_c, are described in table C-5.
5. Values of q_d are determined from figure B-10, supply curve No. 1.6, using the 25, 35, and 40-minute values of t'_c (column 23), the effective length L (column 14), and the weighted supply curve of 1.6 (column 19). The q_d values are read on the vertical axis labeled maximum rate of runoff. Figure B-14 cannot be used to determine these values.

(Sheet 2 of 2)

Table C-5. Airfield drainage size and profile of underground storm drains with drainage section in which no ponding of runoff is assumed and with supply curves No. 2.0 for paved areas and 1.5 for turfed areas.

POINT OF DESIGN			Critical Runoff Time To Produce Maximum Flow In Underground Drain						
	Distance, Feet					Drain Time, Min.			
INLET JUNCTION NUMBER	From Main Outlet	From Preceding Inlet	Critical Inlet	Critical Inlet Time, t_c Min.	Assumed Velocity In Pipe, Ft./Sec.	From Preceding Inlet	Accum. Total	Approx. t'_c (Col. 5 + Col. 8)	Adopted t'_c In Minutes
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
4	4805			24	3.0			24	25
3	3155	1650	4	24	3.0	9.2	9.2	33	35
2	1505	1650	4	24	3.0	9.2	18.4	42	40
Notes:									1

Note: 1. These values reflect the time required for a quantity of water to travel from the most remote portion of the critical inlet area to the inlet being considered.

(Continued)

Table C-5. Airfield drainage size and profile of underground storm drains with drainage section in which no ponding of runoff is assumed and with supply curves No. 2.0 for paved areas and 1.5 for turfed areas—continued.

Rate of Inflow Into Underground Drains, In C.F.S., Corresponding To Adopted Value of t'_c (Column 10)							
Inlet Numbers							
4	3	2					Total
(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)
62.3							62.3
59.0	59.3						118.3
55.7	56.2	56.2					168.1
Notes: 2	2	2					

Note: 2. These values reflect the total quantity of water contributing to the entire system at any given inlet in the system.

Table C-6. Hydraulic design data on underground storm drains where ponding is not acceptable.

Inlet or Junction Numbers		Distance Between Design Points, In Feet, Measured to ℄ of Inlet or Junction	Trial Design				
From	To		Design Discharge Capacity In c.f.s.	Approx. Gradient Between Design Points, Ft./Ft.	Mean Velocity of Design Discharge In Ft./Sec.	Required Size of Pipe, In Inches	Selected Size of Pipe, In Inches
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1	2	1505	168.1	0.0036	8.6	60	60
						73	72[a]
2	3	1650	118.3	0.0031	7.4	53	54
						66	66[a]
3	4	1650	62.3	0.0016	5.0	48	48
						59	60[a]

[a]Pipe diameters required using "n" = 0.021 for 1/4d paved invert corrugated metal pipe. Size differential limited to 6 inches where feasible.

(Continued)

Table C-6. Hydraulic design data on underground storm drains where ponding is not acceptable—continued.

Adopted Design						Construction Data			
Losses				Hydraulic Gradient At Inlet				Elevation of Invert at Design Point, (Col. 2) in Ft.	
Velocity Head $V^2/2g$	Inlet Coefficient K	Inlet Loss, In Feet	Friction Loss, In Feet	Outgoing Pipe	Incoming Pipe	Slope In Ft./Ft.	Length of Pipe Between Design Points In Feet	Incoming Pipe	Out-going Pipe
(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)
				489.04					
1.15	0.12	0.14	5.39	494.57	494.07	0.0036			484.04
0.85	0.12	0.10	5.01	499.18	498.68	0.0031			
0.39	0.12	0.05	2.70	501.43		0.0016			

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