AC NO: 150/5320-5B

DATE: 7/1/70

ADVISORY CIRCULAR

DEPARTMENT OF TRANSPORTATION

FEDERAL AVIATION ADMINISTRATION

SUBJECT: AIRPORT DRAINAGE

1. PURPOSE.

This circular provides guidance for engineers, airport managers, and the public in the design and maintenance of airport drainage systems.

2. CANCELLATION.

This publication cancels "Airport Drainage," AC 150/5320-5A, dated 1965.

3. REFERENCES.

The publications listed in the Bibliography, page 79, provide further guidance and technical information as may be required.

4. EXPLANATION OF REVISIONS.

In addition to minor changes in the text and figures, this advisory circular includes:

a. Reference to more detailed rainfall frequency charts now being published by the Weather Bureau for eleven Western States.

b. Addition of other fundamentals for use of Weather Bureau Technical Paper No. 40 and other technical papers and charts.

c. Emphasis on designing for direct runoff.

d. Information on grates and frames versus aircraft types, weights, and tire pressures.

e. A new section on flow in open channels.

f. Consideration of low head situations for grates in aprons.

g. A new section on culverts.

h. New recommendations for loads on structures in view of very heavy aircraft.

i. Additional emphasis on erosion control.

j. Information on permeability factors in subsurface drainage.

k. Addition of information on use of plastic filter cloths in subsurface drainage.

l. Some corrections to sample drainage system design data and to computations for ponding examples.

m. Revision of minimum pipe cover table including addition of corrugated aluminum alloy pipe.

n. Mention of need for pollution control.

o. Addition of a bibliography of reference material.

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Chapter 1. INTRODUCTION

1. CHARACTERISTICS OF AIRPORT DRAINAGE.

a. An airport should have smooth, well-drained operational areas with sufficient stability to permit the safe movement of aircraft under all weather conditions. The design of adequate drainage is important because it affects the stability and usability of extensive areas; yet, these areas are subject to varying soil and drainage conditions, and also have relatively flat grades.

b. The drainage system should be built before or during the grading operations because draining and grading are interrelated. A drainage system cannot be expected to function properly unless the airport area has been correctly graded to divert the surface runoff into the system. In the absence of adequate stabilization or pavement, drainage does not assure an all-weather airport, but it does shorten the interval of nonuse.

c. The large area that must be drained on an average airport requires an economically designed drainage system to realize the full value of the investment made. Sound engineering principles must be applied in the utilization of all available data, such as: topographic maps; soil reports; determinations of water tables; intensity, frequency, and duration of precipitation; climate and temperature reports; and nature of the area surrounding the particular site.

d. The topography of the site and the off-site areas affect the final layout of the runways, taxiways, aprons, and buildings. The location and size of these facilities will control the grading and the extent of drainage required. It is important that the grading of the airport be such that all shoulders and slopes drain away from runways, taxiways, and all paved areas. After final elevations on the airport have been determined, all surface flow of water onto the site must be intercepted and disposed of, any depressed or low spots on the site must be drained, and all surface runoff must be accumulated and directed into adequate outfalls.

e. Enough tests should be taken to identify all soil types because texture, permeability, and capillarity have a pronounced effect upon their drainability. Because of its effect on the stability of soils and on the ultimate design of the airport, the water table should be accurately determined over the entire area. When a high water table does exist, provision should be made for controlling or lowering it—or alternatively raising the pavement grades—see paragraph 21.

f. In designing a drainage system, it is important to determine expected precipitation at the airport site. Intensity-frequency or precipitation data may be obtained and developed from information in several publications as noted in paragraph 3.

g. Localized climatological data should be studied and advice sought on average frost penetration and recommended minimum depth of storm sewer installations for the area. For some localities, records of average accumulated snowfall would be pertinent to the drainage design.

2. PURPOSE OF AIRPORT DRAINAGE.

a. The purpose of airport drainage is to dispose of water which may hinder any activity necessary to the safe and efficient operation of the airport. The drainage system should collect and remove surface water runoff from each area, remove excess underground water, lower the water table, and protect all slopes from erosion.

b. Natural drainage normally does not meet these requirements. Constructed drainage facilities must be sufficient to provide for present requirements and any future enlargements of the system. This may mean the installation of a portion of a drainage system to supplement the natural drainage on the site or it may call for a complete system to drain the entire airport area. A proper understanding of all contributing drainage factors determines the extent of the facilities required on each particular airport.

c. An inadequate drainage system can cause serious hazards to air traffic at airports. The most dangerous consequences of inadequate drainage systems are saturation of the subgrade and subbase, damage to slopes by erosion, loss of load-bearing capacity of the paved surfaces, and excessive ponding of water.

d. Aprons and other pavement should be sloped away from buildings so that there will be no possibility of fuel spillage flowing toward buildings.

Chapter 2. HYDROLOGY

3. RAINFALL.

a. The determination of the amounts of rainfall and runoff to be used as a basis for design of a drainage system is the primary step to be considered by the designer. The rate of storm runoff which will flow into the system must be established in the preliminary design stage. At some locations this may involve rainfall plus melting snow or ice.

b. The importance of the rainfall-intensity factor is well known to drainage engineers, particularly in its relationship to total runoff. Investigations have shown that results of studies regarding the probable intensity, frequency, and duration of rainfall in particular locations are more likely to be correct and conservative if they are obtained from the records of many stations rather than from the record of one station. Single stations seldom give a true picture of the rainfall-frequency regime for various locations. The use of many stations in a region to determine the pattern and value of the rainfall-frequency values tends to minimize the limitations of a small sample in both time and space.

c. Many investigations and studies have been conducted to find a basis for making reasonable estimates of the intensities, frequencies, and durations of rainfall for different locations. A previously used publication by D. L. Yarnell, "Rainfall Intensity-Frequency Data," has now been replaced by more recent studies. Rainfall-frequency data for the United States, Puerto Rico, the Virgin Islands, Hawaii, and Alaska can be obtained from a series of Weather Bureau Technical Papers. Data for the conterminous United States are given in ESSA-Weather Bureau Technical Paper No. 40, "Rainfall Frequency Atlas of the United States," dated May 1961. Technical Paper No. 40 is intended as a convenient summary of empirical relationships, working guides, and maps, useful in practical problems requiring rainfall-frequency data. It is an outgrowth of several previous Weather Bureau publications on this subject and contains an expression and generalization of the ideas and results in earlier papers. It is divided into two parts:

(1) The first part presents the rainfall analyses. Included are measures of the quality of the various relationships, comparisons with previous works of a similar nature, numerical examples, discussions of the limitations of the results, transformation from point to areal frequency, and seasonal variation.

(2) The second part presents 49 rainfall-frequency maps based on a comprehensive and integrated collection of up-to-date statistics, several related maps, and seasonal variation diagrams. The rainfall-frequency (isopluvial) maps are for selected durations from 30 minutes to 24 hours and return periods from 1 to 100 years.

Rainfall-frequency data for Puerto Rico and the Virgin Islands, Hawaii and Alaska can be obtained from ESSA-Weather Bureau Technical Papers No. 42, "Probable Maximum Precipitation and Rainfall-Frequency Data for Puerto Rico and the Virgin Islands, "dated 1961; No. 43, "Rainfall-Frequency Atlas of the Hawaiian Islands," dated 1962; and No. 47, "Probable Maximum Precipitation and Rainfall-Frequency Data for Alaska," dated 1962, respectively.

d. The preferred source of precipitation-frequency information for the eastern two-thirds of the conterminous United States is ESSA-Weather Bureau Technical Paper No. 40. Rainfall-frequency data for the eleven Western States can be obtained from the physiographically adjusted precipitation-frequency maps currently being prepared by the ESSA-Weather Bureau. These maps have been completed for the States of Arizona, New Mexico, Colorado, Utah, Wyoming, Montana, Idaho, Washington, and Oregon. Maps for Nevada and California are currently being prepared and should be available shortly. The maps for the Western United States are only for the 6- and 24-hour duration values and relations given in Technical Paper No. 40 must be used to obtain the values for the shorter durations (see paragraph g below). Rainfall-frequency values for Puerto Rico and the Virgin Islands, Hawaii and Alaska can be obtained from ESSA-Weather Bureau Technical Papers Nos. 42, 43, and 47, respectively. It should be recognized that there may be some locations where the values from the generalized charts may be either an over or an underestimate. If the engineer suspects this to be the case, he should investigate all possible sources of data for additional information to verify or revise the published values. Possible data sources are the local ESSA-Weather Bureau Office, the State Highway Office, State Hydrographers Office, City Engineer's Office, and perhaps local drainage districts or utility companies. Such locally derived data should not be used to override the data from the Weather Bureau Technical Papers unless there is ample evidence that the latter data is clearly not applicable to the local situation.

e. The rainfall intensity-duration curves required for design purposes, in the conterminous United States, can be derived from the charts in Technical Paper No. 40. Figures 1 to 5 are examples of the charts in that paper. It is not intended that these figures be used for determining intensity as the scale was kept quite small for illustration purposes. Return periods of 2, 5, and 10 years are sufficient for airport drainage calculations and sufficient for comparisons between such periods. To construct intensity-duration curves such as shown in Figure 6, begin by spotting the airport location on the 30-minute, 1-hour, and 2-hour charts for 2, 5, and 10 years in Technical Paper No. 40. Then read the intensities by scaling between the isolines, linear interpolation between adjoining isolines is sufficient. For example, the 5-year, 30-minute rainfall chart (Chart 3) reveals that the intensity at Chicago would be 1.37 inches. This must be converted to a 1-hour basis for curve plotting purposes, therefore, the scaled quantity must be multiplied by the ratio between 1-hour and the ½-hour duration ($1.37 \times (60 \div 30) = 2.74''$). Plot this intensity on coordinate paper (using inches per hour as ordinates and duration in minutes as abscissas). Similarly, scale the intensity for Chicago for a 1-hour rainfall to be expected once in 5 years from Chart 10, for a 2-hour rainfall from Chart 17, and convert the latter value to a 1-hour basis. As Technical Paper No. 40 does not have short-duration rainfall charts, i.e., for 5, 10, and 15 minutes, it is necessary to use the Weather Bureau developed relationship between a 30-minute rainfall on the one hand and 5-, 10-, and 15-minute amounts on the other. This relationship is as follows:

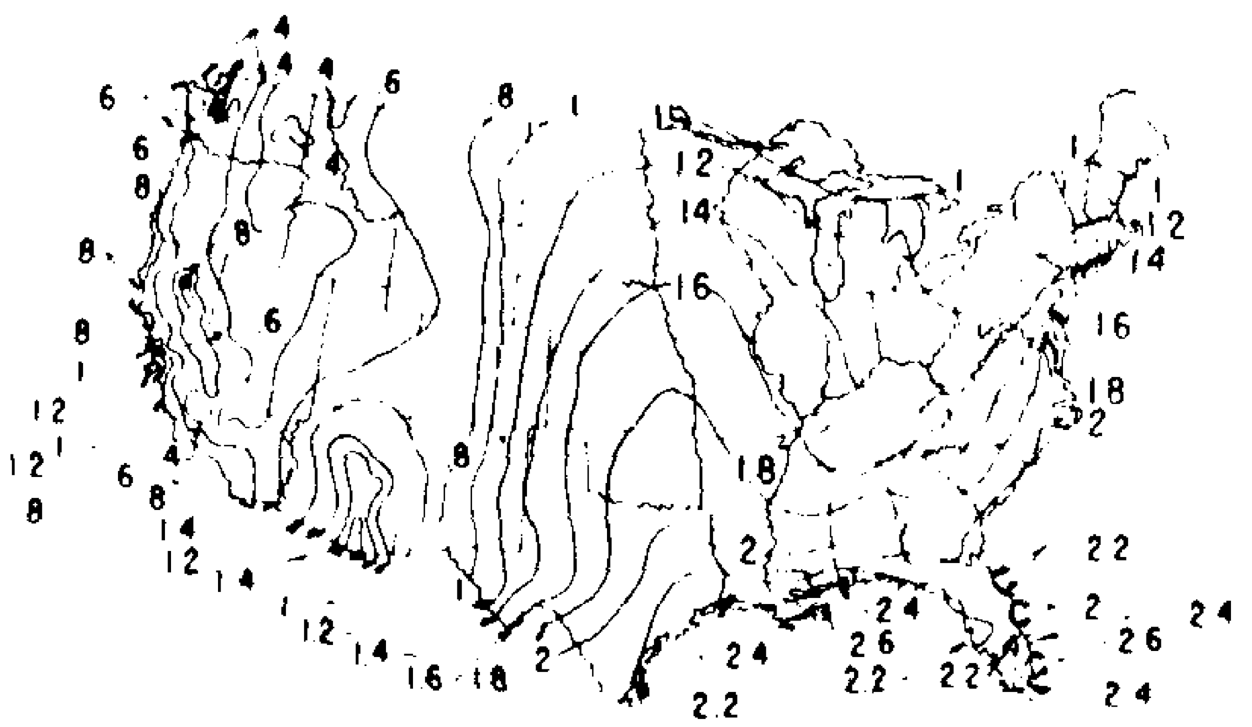
Duration (minutes)	5	10	15
Ratio	0.37	0.57	0.72

Thus, the Chicago 30-minute amount of 1.37 can be reduced to a 5-minute amount by multiplying 1.37 by 0.37 or $1.37 \times 0.37 = 0.51''$; similarly, $1.37 \times 0.57 = 0.78''$ for the 10-minute amount, and $1.37 \times 0.72 = 0.99''$ for the 15-minute amount. Then convert these values to a 1-hour basis as described above. Accordingly, the 5-, 10-, 15-, and 30-minute, 1-hour, and 2-hour intensities give 6 points on the coordinate paper and a smooth curve can be drawn through the points to construct the 5-year curve. This curve will indicate the intensity of rainfall to be expected for any time interval from 5 minutes to 2 hours for a storm that might occur once in 5 years. The same procedure should be followed to construct curves for 2 and 10 years. Figure 6 is a graph exemplifying this procedure.

f. Similar procedures can be followed in Puerto Rico and the Virgin Islands, Hawaii, and Alaska, using charts from the appropriate Weather Bureau Technical Papers to obtain values for 30 minutes, 1, and 2 hours. The 5-, 10-, and 15-minute values can be approximated using the ratios with 30-minute rainfall values cited earlier.

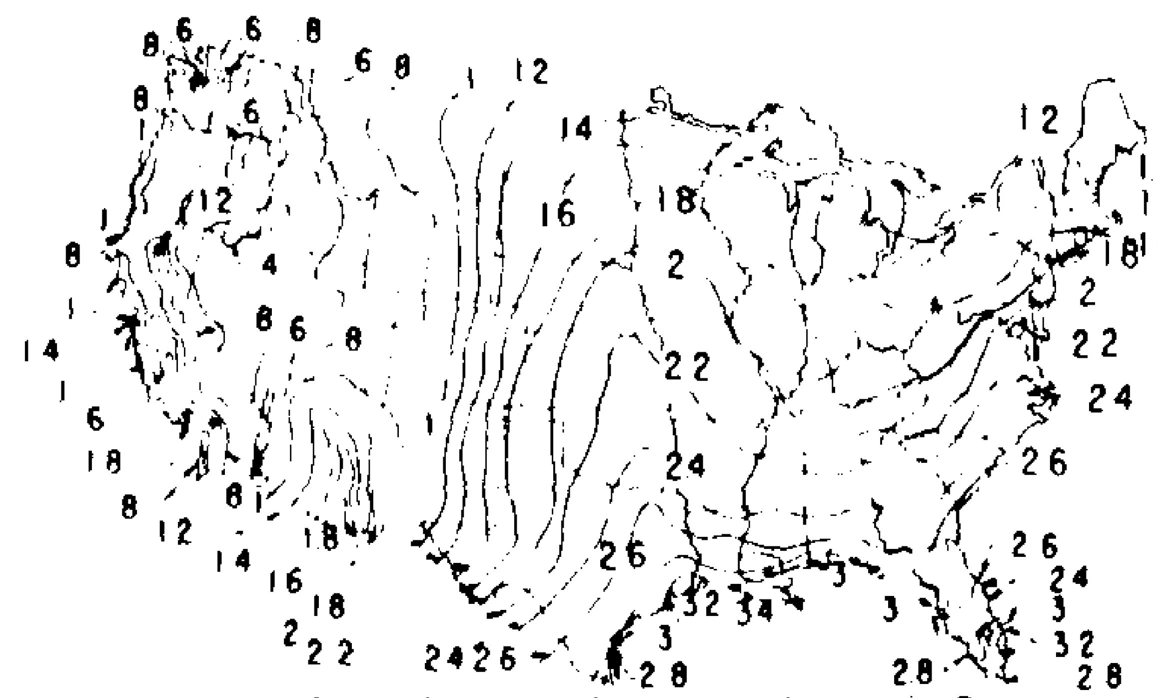
g. The more detailed rainfall-frequency charts for the 11 Western States, referred to in paragraph d above, should be used to obtain the rainfall intensities for the more variable conditions found in the mountainous areas of Western United States. These studies provide detailed maps which depict the variation in rainfall-frequency values in this portion of the country. These maps were developed to depict the rainfall-frequency values for average conditions along orographic barriers and in mountain valleys. At some locations, where the topography departs significantly from average conditions, values determined from the generalized chart may be either an under or overestimate. Locally available data for these locations could be considered by the engineer to modify values obtained from the generalized charts. Such locally derived data should not be used, however, unless there is ample evidence to show that the local situation insists that such data are more applicable than that in the generalized charts. The following procedure will need to be used to convert the information in the charts for the 11 Western States to a more useable form. As the charts provide values for only the 6- and 24-hour durations, it is necessary to compute values for the shorter durations for our purposes. This can b edone by using Figure 2 of Technical Paper No. 40. (First note, however, that these charts show values in tenths of an inch, so they must be converted to inches.) For example, read the 5-year, 6- and 24-hour amounts for the airport location from the charts and plot these amounts on Figure 2. Lay a straight-edge across the two points and read the values for 1- and 2-hour durations at the proper intersections. Estimate the 30-minute rainfall by multiplying the one-hour value by 0.79. Then proceed to convert the 30-minute value to 5-, 10-, and 15-minute values by using the ratios given in paragraph e above. Then convert the values to a 1-hour intensity and construct a 5-year curve in the manner described in paragraph e.

h. The use of the data, developed as described in either paragraph e or g above, is taken up later in Chapter 5 and is the basis for estimating runoff.



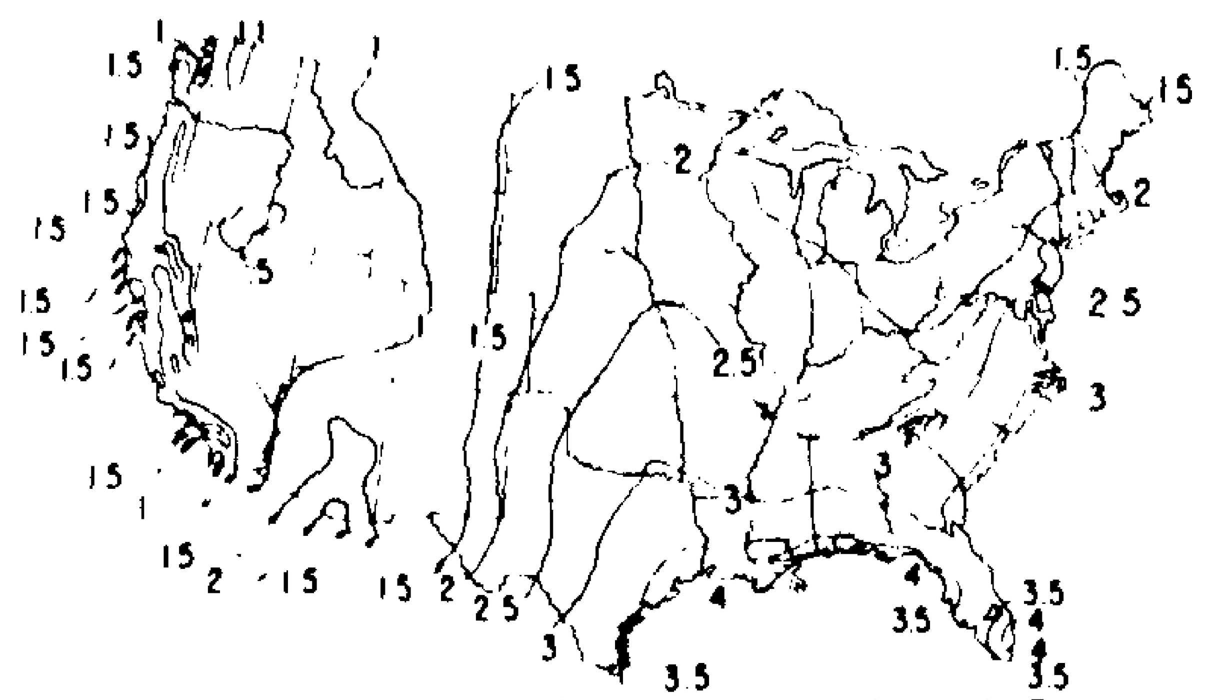
Thirty-minute rainfall, in inches, to be expected once in 5 years.

Figure 1



One-hour rainfall, in inches, to be expected once in 5 years

Figure 2



Two-hour rainfall, in inches, to be expected once in 5 years

Figure 3

NOTES

THE WEATHER BUREAU DEVELOPED RELATIONSHIPS FOR SHORT DURATION RAINFALLS. THE AVERAGE RELATIONSHIP BETWEEN 30 MINUTE RAINFALL ON THE ONE HAND AND 5-, 10-, AND 15-MINUTE RAINFALL ON THE OTHER CAN BE OBTAINED FROM THE FOLLOWING

DURATION (MIN)	5	10	15
RATIO	0.37	0.57	0.72

THUS A GEOGRAPHICAL POINT ON FIGURE 1 WHICH INDICATES A THIRTY MINUTE RAINFALL OF 1.6 INCHES ONCE IN 5 YEARS MAY BE REDUCED TO 5 MINUTES BY MULTIPLYING 1.6 BY 0.37 = .592 INCHES OR 10 MINUTES AS 1.6 BY 0.57 = .912 INCHES OR TO 15 MINUTES AS 1.6 X 0.72 = 1.152 INCHES.

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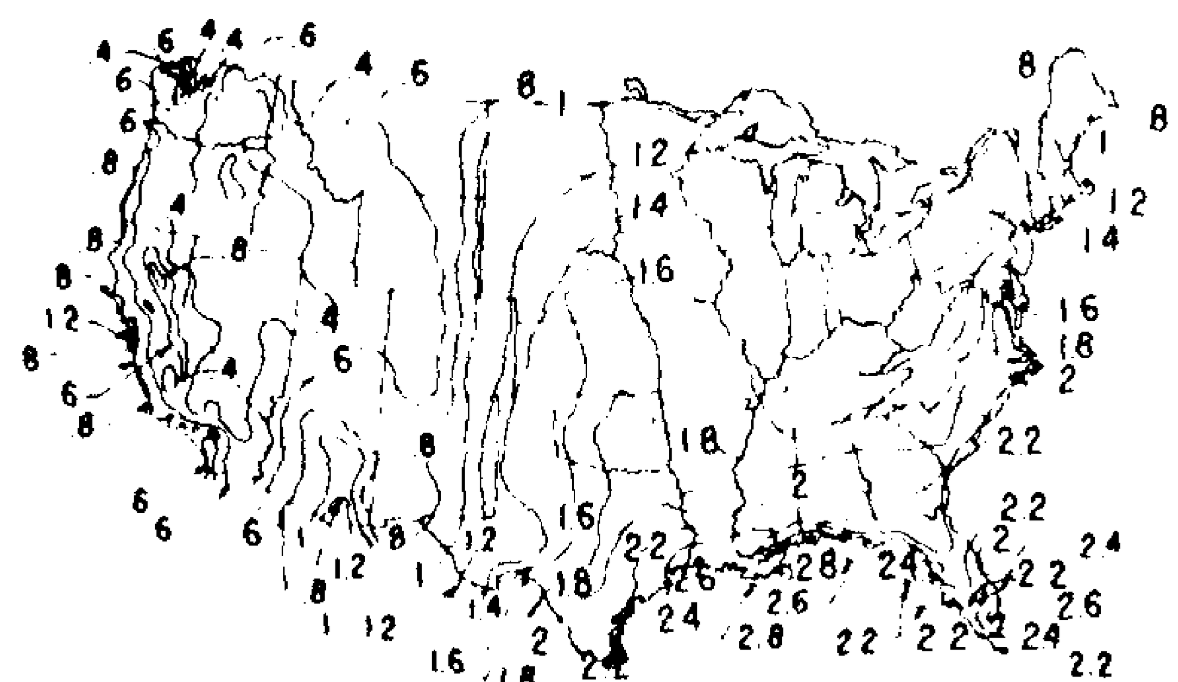
IF ACTUAL WEATHER BUREAU RECORDS OR SUPPLEMENTAL RECORDS ARE NOT AVAILABLE FOR THE POINT IN QUESTION THEN THE CHARTS IN THE TECHNICAL PAPER NUMBER 40 "RAINFALL FREQUENCY ATLAS OF THE UNITED STATES" MAY BE USED TO CONSTRUCT INTENSITY CURVES SUCH AS SHOWN IN FIGURE 6. INDEED FIGURES 1, 2, 3, 4, 5 WERE CONSTRUCTED FROM TECHNICAL PAPER NUMBER 40 AND SERVE TO ILLUSTRATE THE METHOD OF DEVELOPING AN INTENSITY-DURATION CURVE OR CURVES. FIRST SPOT THE AIRPORT LOCATION UNDER CONSIDERATION ON FIGURE 1, (30 MINUTE RAINFALL, INCHES, TO BE EXPECTED ONCE IN 5 YEARS,) LOCATING THE POINT AS ACCURATELY AS POSSIBLE THEN SCALE THE INTENSITY FOR THE POINT IN QUESTION BY INTERPOLATION BETWEEN THE TWO ISOHYETAL LINES ON EITHER SIDE OF THE POINT. TO ILLUSTRATE, THE INTENSITY SCALED FROM FIGURE 1 IS FOUND TO BE 1.36 INCHES AT CHICAGO; THIS IS FOR A 30 MINUTE RAINFALL TO BE EXPECTED ONCE IN 5 YEARS. THIS INTENSITY NEEDS TO BE CONVERTED TO A 1-HOUR BASIS FOR PLOTTING PURPOSES.

THUS THE SCALED QUANTITY OF 1.36 INCHES MUST BE MULTIPLIED BY 60 ÷ 30 OR 2 RESULTING IN 1.36" X 2 = 2.72". PLOT THIS INTENSITY ON COORDINATE PAPER (USING INCHES PER HOUR AS ORDINATES AND TIMES IN MINUTES AS ABSCISSAS). SIMILARLY SCALE THE INTENSITY FOR CHICAGO FOR 1-HOUR RAINFALL TO BE EXPECTED ONCE IN 5 YEARS FROM FIGURE 2 AND FOLLOW THE ABOVE PROCEDURE, THEN USE FIGURE 3 FOR TWO HOUR RAINFALL.

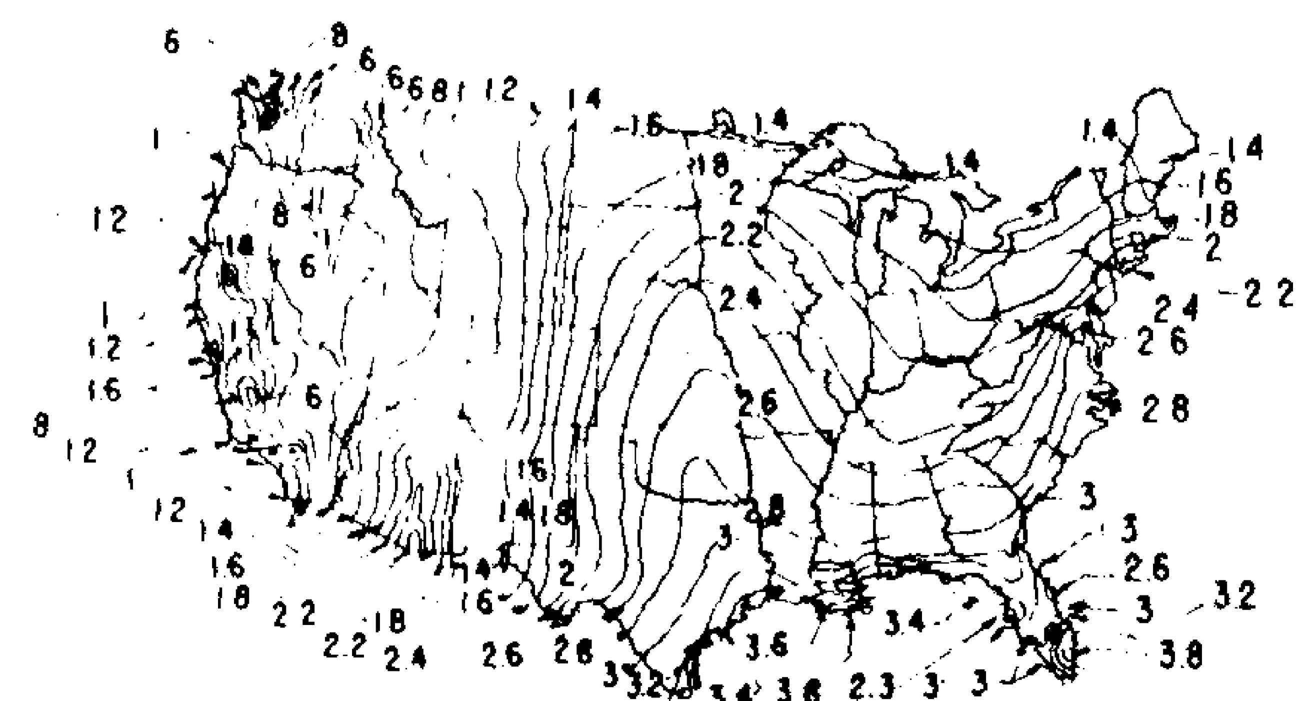
ALSO, CONVERT THE 30 MINUTE INTENSITY TO 5-, 10-, AND 15 MINUTE INTENSITIES AS INDICATED ABOVE, CONVERT TO 1 HOUR BASIS AND PLOT. FIGURE 6 ILLUSTRATES 2, 5 AND 10 YEAR INTENSITY DURATION CURVES CONSTRUCTED IN THIS MANNER.

FIGURES 1, 2, 3. Rainfall frequency maps.



One-hour rainfall, in inches, to be expected once in 2 years

Figure 4



One-hour rainfall, in inches, to be expected once in 10 years.

Figure 5

FIGURES 4 and 5. Rainfall frequency maps.

INTENSITY CURVES FOR STORMS IN VICINITY OF EXAMPLE SITE

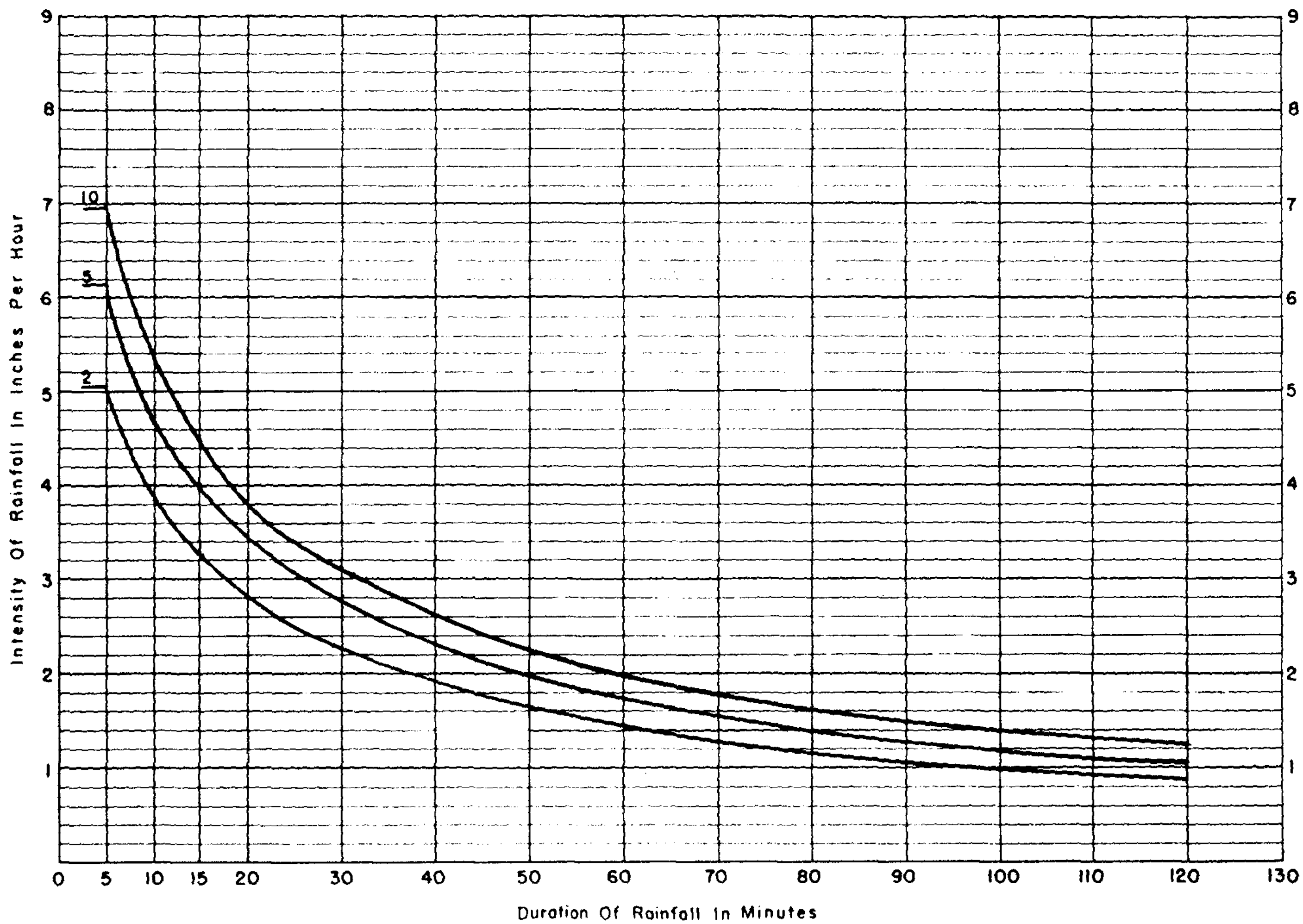


FIGURE 6. Intensity curves for storms in vicinity of example site.

Chapter 3. COMPUTATION, COLLECTION, AND DISPOSITION OF RUNOFF

4. RUNOFF.

a. After rainfall rates have been studied, there remains a problem of determining what portion of the rainfall must be accounted for as surface runoff. The runoff rate depends on a number of conditions and is seldom constant for any given area during a single period of precipitation. The following factors have a pronounced influence on the rate of runoff from an area:

(1) Intensity and duration of the rainfall.

(2) Type and moisture content of the soil affecting infiltration.

(3) Perviousness or imperviousness of surfaces.

(4) Slope or irregularity of surfaces.

(5) Extent and condition of vegetative cover.

(6) Snow cover.

(7) Temperature of air, water, and soil.

b. Many studies have been conducted during the last few decades in attempts to derive a method for estimating the amount of runoff when affected by the varying factors actually met under field conditions. The studies have covered infiltration of soils; runoff from pavement, turf areas, different length and slopes; rainfall characteristics as related to soil erosion; and numerous other conditions. Some studies have contributed valuable data toward a more comprehensive understanding of the complex problem. Until a more valid method is developed for determining the amount of runoff from given areas, the following is considered to be the practical course.

c. The Rational Method of calculating runoff is most universally applied and recommended by engineers in drainage practice. The method has come into favor because it enables the engineer to apply judgment directly to specific determinations which are subject to analysis after consideration of local conditions.

(1) The Rational Method is based on the direct relationship between rainfall and runoff. It is expressed by the equation $Q = CIA$, in which:

Q = the runoff in cu. f.p.s. from a given area;

C = a runoff coefficient depending upon the character of the drainage area;

I = the intensity of rainfall in inches per hour;

A = the drainage area in acres.

The value of C to be used must be based on a study of the soil, the slope and condition of the surface, the imperviousness of the surface, and the consideration of probable future changes in the surface within the area. The value of I to be selected depends upon the curves for the intensity of rainfall plotted for the local vicinity and the assumed period of return or recurrence, as well as the period of concentration required for surface runoff to flow from the most distant point in the area under study to the inlet structure or point of collection being considered. Design should be governed by the greatest intensity of rainfall during this period of concentration and not by some intensity for a shorter period. The value of A is measured and can be accurately determined.

(2) A maximum rainfall expected once in 5 years is generally recommended for estimating runoff for airports; thus, a 5-year curve similar to that shown in Figure 6 will usually be used. The damage or inconvenience which may be caused by greater storms is insufficient to warrant the increased cost of a drainage system large enough to accommodate a storm expected once in a period longer than 5 years.

Also, it is the recommended design procedure to provide capacity in the drainage system for direct runoff. The calculation of and provision for ponding between runways, taxiways, and aprons should usually be considered as a safety factor—for temporary accommodation of runoff from storm return periods longer than 5 years.

Ponding of more than a temporary nature may be acceptable on the airport site other than between runways, taxiways, and aprons. Such ponding may indeed be essential because of limitations in offsite outfalls.

5. RUNOFF COEFFICIENT.

a. The runoff coefficient or factor as it is sometimes designated, is the percentage of rainfall on a given area that flows off as free water. This percentage will seldom reach 100 percent, even with steep slopes, because impervious surfaces absorb some moisture and small depressions and irregularities hold back additional amounts. During a storm, the percentage of runoff will increase gradually as the soil becomes saturated, the impervious areas become thoroughly wet, and all depressions become filled. Then the percentage will remain fairly constant, varying directly with the intensity of the rainfall. The composite effect of all those factors must be taken into consideration.

b. Many authorities have presented estimates for "values of relative imperviousness" for different types of urban surfaces, to be used in conjunction with their various formulas. These estimates cover conditions applicable to the design of drainage systems for large areas, usually within urban surroundings where the character of surface is different generally from those on airports.

c. From these studies and other information pertaining to relative imperviousness of different surfaces, Table I has been compiled which is applicable to the conditions found on airports. The appropriate runoff coefficient should be selected from Table I for use in the formula Q=CIA.

d. If the drainage area contributing to a certain inlet is composed of several surfaces for which different coefficients from this table must be assigned, the coefficient used in the formula should be a weighted average in accordance with the respective areas. For example, if a drainage area to an inlet consists of ½ acre of asphalt pavement having a coefficient of 0.90 and 2 acres of impervious soil with turf having a coefficient of 0.35, the average coefficient for the total area is equal to $[(0.90 \times 0.5) + (0.35 \times 2.0)] \div (0.5 + 2.0)$ or 0.46.

TABLE I. Value of factor "C"

Type of surface	Factor "C"
For all watertight roof surfaces	.75 to .95
For asphalt runway pavements	.80 to .95
For concrete runway pavements	.70 to .90
For gravel or macadam pavements	.35 to .70
For impervious soils (heavy)*	.40 to .65
For impervious soils, with turf*	.30 to .55
For slightly pervious soils*	.15 to .40
For slightly pervious soils, with turf*	.10 to .30
For moderately pervious soils*	.05 to .20
For moderately pervious soils, with turf*	.00 to .10

*For slopes from 1 percent to 2 percent.

6. TIME OF CONCENTRATION.

a. According to the theory underlying the Rational Method, maximum discharge at any point in a drainage system occurs when:

(1) The entire area tributary to that point is contributing to the flow.

(2) The rainfall intensity producing such flow is based upon the rate of rainfall which can be expected to fall in the time required for water to flow from the most remote point of the area to the point being investigated. The "most remote point" is the point from which the time of flow is greatest. It may not be at the greatest linear distance from the point under investigation.

b. The time at which maximum discharge occurs is referred to as the time of concentration. It is composed of two components referred to as the "inlet time" and "time of flow". The "inlet time" is the time required for water to flow overland from the most remote point in the drainage subarea to the inlet. The "time of flow" is the time during which water flows through the drainage system to any point

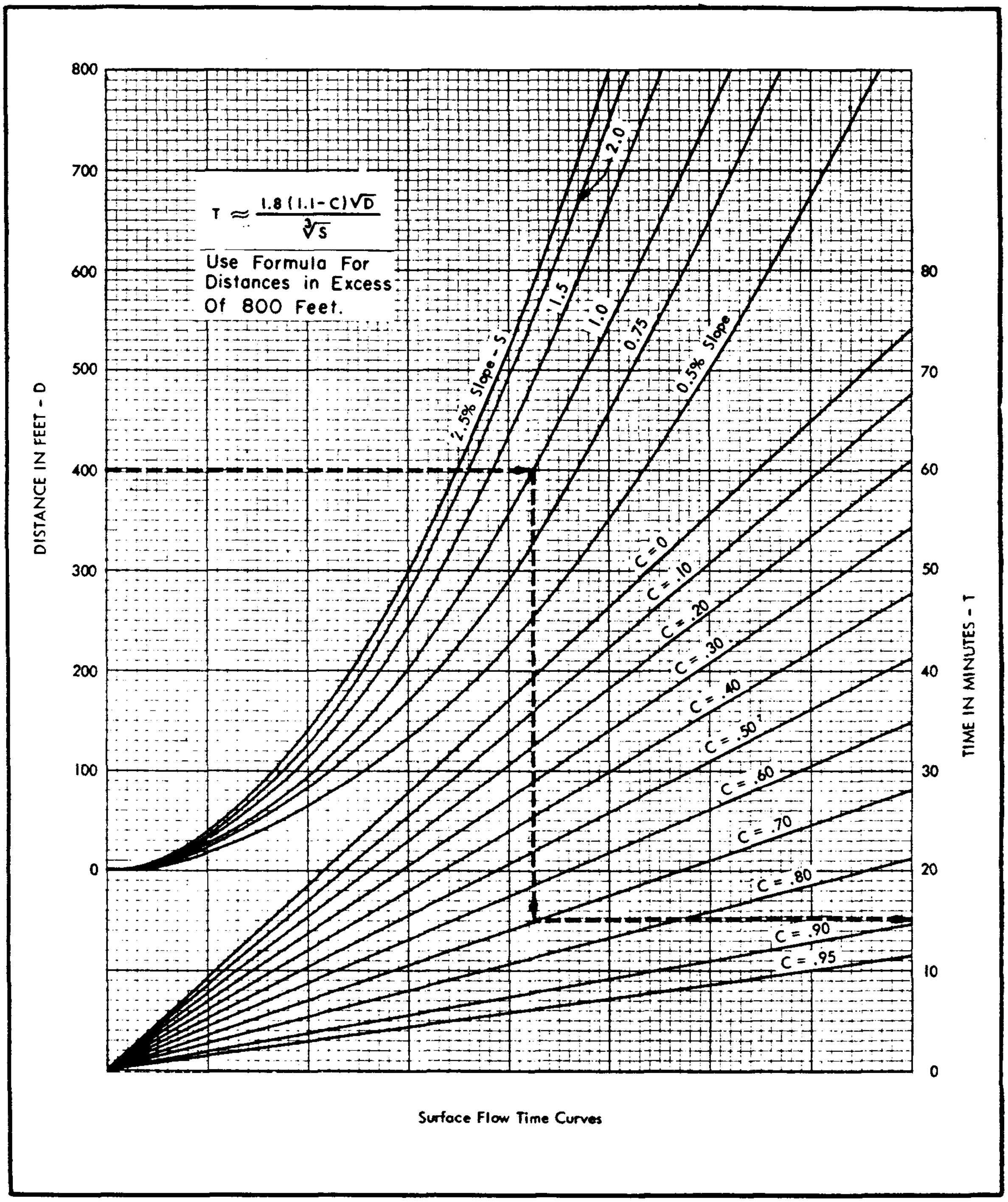


FIGURE 7. Surface flow time curves.

being investigated. In some instances the "inlet time" will be the time of concentration. Such is the case for an inlet at the upper end of a drainage line.

(1) Furthermore, a condition may exist where the "inlet time" to a structure along the line may exceed the time required for water falling on a more distant subarea to reach that

inlet. All areas tributary to the particular structure are not contributing until such time as water is entering the inlet from the most remote part of the individual subarea which it serves. The time of concentration, therefore, will be the "inlet time." Problems which arise in this regard will have to be investigated and resolved individually to determine under what conditions of time and flow the maximum volume of water can be expected at the point studied.

(2) "Time of flow" can be determined by hydraulic computation, i.e., by dividing the length of pipe by the velocity of flow.

(3) The "inlet time," considered one of the most important factors in determining runoff, will vary with surface characteristics of the drainage area. The curves or formula in Figure 7 will provide adequate estimates of "inlet time" for the designer. Use the formula for distances in excess of 800 feet. Where the particular drainage area consists of several types of surfaces, the "inlet time" must be determined by adding the respective times established for flow over the length of the several surfaces along the path from the most remote point to the inlet.

7. COLLECTION AND DISPOSAL OF RUNOFF.

a. Before any definite computations can be made toward the actual design of the drainage system, a topographical map will have to be prepared showing actual ground contours existing on the airport area. The contours preferably should be drawn to a 2-foot interval. This map should be extensive enough to show the areas surrounding the airport boundaries with all natural watercourses, swales, draws, drainage structures, ditches, slopes, ridges and configurations. It should also show all improvements that might have a bearing on the runoff and drainage of the immediate area, such as railroads, highways, canals, and irrigation and drainage installations.

b. An additional detailed plan is necessary to show the proposed and ultimate layout of the runways, taxiways, aprons, and building area with the finished contours drawn to a 1-foot interval or less. This plan can be the "drainage working drawing." The entire system should be sketched upon it, with the outline and identification of each single subarea, all main and lateral storm pipelines, pipe sizes, direction of flow, gradients, catch basins, inlets, manholes, gutters when required, surface channels, peripheral and outfall ditches, and other essential drainage features.

c. The finished grades in conjunction with the drainage design are very important. The location of the runways, taxiways, aprons, and building area is usually fixed by the time the drainage design is started; but the cross sections of the paved areas, their profiles, and all the grades of intermediate areas should be carefully studied for their influence on the drainage layout. It is important that all finished grades be established so that every area is drainable and so that the runoff can be collected by some drainage facility.

d. The primary consideration, therefore, is the determination of a satisfactory drainage arrangement at a reasonable cost, involving the location of the shallow channels, inlets, catch basins, manholes, selection of grades, etc. Trial computations of several different drainage layouts should be made in arriving at the most practical design. When all arrangements for location of the inlets, shallow drainage swales, and storm pipes have been plotted upon the drainage working drawing, a tabulation of data and computations should be made. Such a table is further discussed in Chapter 5.

e. Normally, the inlets should be located at least 75 feet from the edge of the pavement at airports with scheduled air carrier operations and 25 feet from the edge of the pavement at airports used exclusively by general aviation. If inlets are placed close to the pavement edges, they may be bypassed by the flow of water. Also, no ponding would be possible because the impounded water could back up to the edge of the pavement and cause saturation of the subgrade. The grading should be planned so that the inlets can be placed normally at the edges of the runway safety area or in the area midway between the runways and the parallel taxiways. The runways and taxiways should be crowned. Beyond the paved edges, the slopes should be in accordance with design recommendations. In establishing grades out-

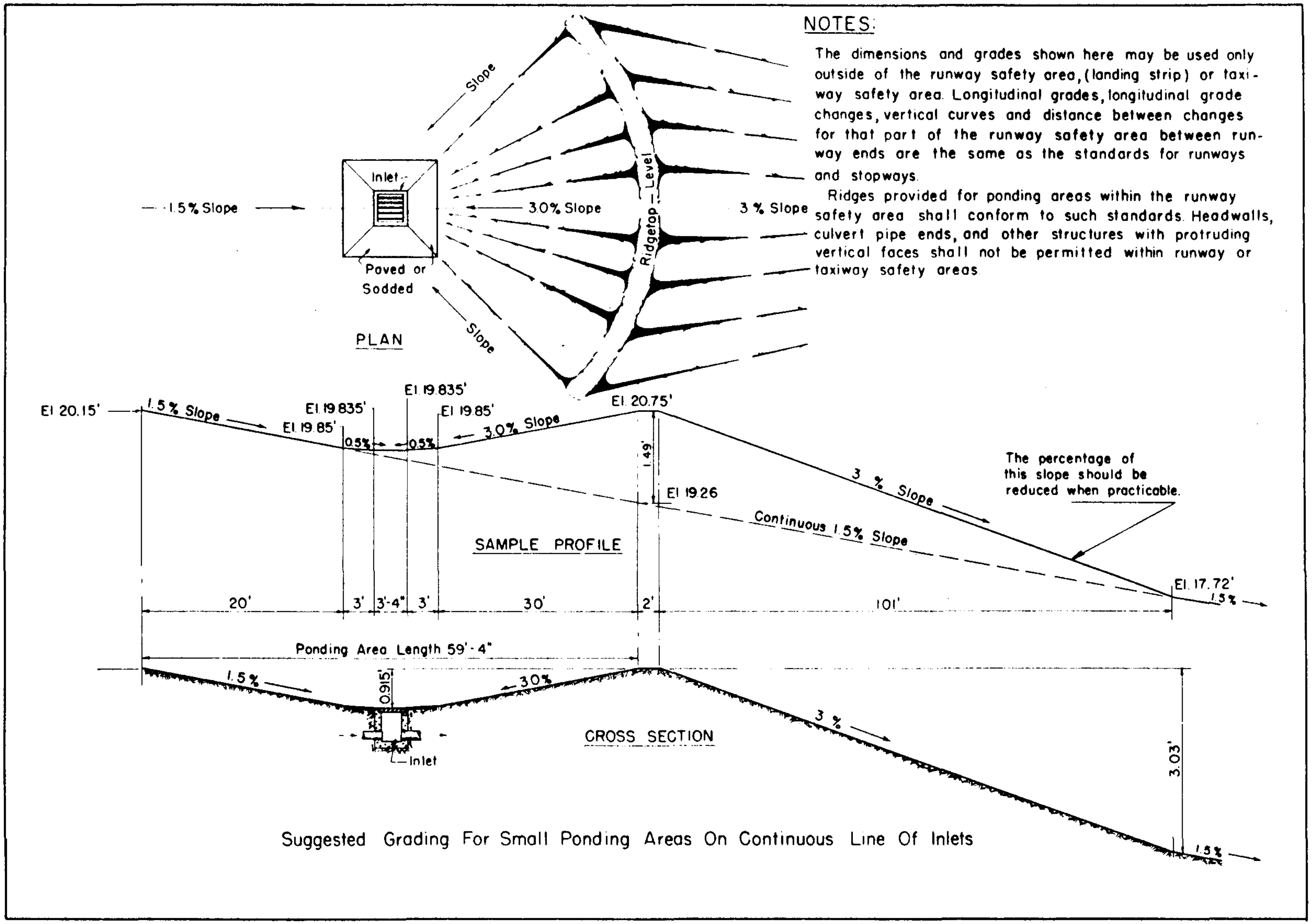


FIGURE 8. Suggested grading for small ponding areas on continuous line of inlets.

side of pavements, the soil characteristics should be considered so as to avoid erosion and promote infiltration. Less grade is used for sandy soils than for other soils. A slope of 5 percent should be used for a 10-foot width adjacent to pavement edges to facilitate runoff.

f. It is desirable to provide for ponding areas around inlets as temporary storage for runoff from the occasional storm which exceeds the design storm. If several inlets are in the same graded area, it is appropriate to design a ridge between the inlets to prevent runoff from bypassing the upper inlet. These ridges and ponding areas must be designed to avoid unacceptable grades and grade changes within runway and taxiway safety areas (see Figure 8).

g. Inlets should be placed at all intermediate low points created by grading the airport. In the case of a long run of surface drainage where the fall is all in one direction, the inlets should be spaced so that the runoff will not travel excessive distances before reaching a structure. Normally, inlets should be spaced so that the flow from the most remote point of the drainage area is not more than 400 feet.

h. Manholes, or combination manholes and inlets, should be provided where necessary: their spacing should approximate that for inlets. In good drainage practice, manholes should be placed at all changes in pipe grades, changes in pipe sizes, changes in direction, junctures of pipe runs, and at reasonable intervals (approximately 300 to 500 feet) for cleanout and inspection purposes. When it is impractical to have manholes approximately 400 feet apart on subsurface systems, then lamphole risers should be installed to allow access for observation and flushing. (It may be noted that the drainage design example of paragraph 19, Figure 33, and Tables IV and V contradict the principles of this and the preceding paragraph to spacing and location of inlets and manholes. The contradiction is deliberate and was made in the interest of simplifying the example).

i. All natural watercourses, draws, and outfalls should be accurately spotted on the drainage working drawing, and the drainage system should be planned so that as many of these watercourses as possible can be used for outfall and rapid removal of the runoff from the airport area. This procedure is necessary to prevent concentration of all the airport runoff in one or two outfalls and flooding of property below the airport site. By use of several outfalls when they are available, the cost of the system can be held to a minimum by reducing pipe sizes and by shortening the discharge pipe runs.

j. Open peripheral ditches should be used, whenever practicable, to receive outfall flow from the drainage system, to collect surface flow from the airport site and adjacent areas, to intercept ground water flow from adjacent higher terrain, and in many cases to aid in lowering the ground water table. These open peripheral ditches should not be constructed where they will cross the runway safety area (landing strip) or extended safety area. The flow across this section should be placed in conduit for at least the width of the safety area. Before a system of peripheral ditches is planned for an airport site, the soil should be examined to determine whether the soil will erode. Open ditches have a tendency to erode because of the concentrated flow. Ditches should not be constructed where the airport is located on sand unless they are absolutely necessary, and even then, they should be shallow ditches with flat slope and immediately lined with sod or otherwise protected to prevent erosion.

k. If the outfall drainage cannot be emptied into existing watercourses or natural drainage channels, or if the quantity of water is greatly increased over normal flow, easements or agreements should be obtained from the affected property owners to avoid future controversy.

8. FLOW IN CONDUITS.

a. After the locations of the inlets, manholes, pipe runs, and outfalls have been determined and the design runoff for all subareas has been computed, the next step in the design will be the computation (by appropriate hydraulic formulas) of the size and gradient of the pipe drains. Also, the "flow time" in the pipes from the various inlets can be computed according to the hydraulic characteristics of the pipe.

(1) Several formulas are used by engineers to determine the flow characteristics in pipes. Many of them give practically the same results, but the Manning formula is the most widely used and is recommended for use and is as follows:

$$Q = A \frac{1.486\ R^{2/3}\ S^{1/2}}{n}$$

in which:

Q=discharge in cubic feet per second

A=cross-sectional area of flow in square feet

R=hydraulic radius in ft. $= \frac{\text{area of section}}{\text{wetted perimeter}}$

S=slope of pipe invert in ft. per foot

n=coefficient of roughness of pipe

(2) Charts have been compiled for the solution of the Manning formula. They usually are used instead of the formula to determine the size of pipe required. Figures 9 to 12 inclusive show these charts based on Manning's formula for discharge of circular pipes flowing full, with slopes from 0.0001 to 0.1 feet per foot, and values of "n" = 0.012, 0.013, 0.021, 0.023, 0.024, 0.027, and 0.031. The selection of the value of "n" in Table II is also a matter of judgment. The value selected should represent conditions which will prevail during the useful life of the line.

b. The design engineer should keep in mind that it is important to maintain sufficient velocity within the pipes to prevent depositing of suspended matter washed into the system through the inlets. The velocity of flow in pipes depends on the head or slope and the resistance to flow of the wetted portion of the pipe interior. The head or slope used in design always refers to the position of the hydraulic gradient, which is the line assumed by the top surface of the flowing water when free to rise vertically. The wetted portion of the pipe interior is used in determining the hydraulic radius, which is the area of the inside of the pipe divided by the wetted perimeter. The mean velocity of flow is used in determining the size of drains.

(1) Some engineers, when designing drainage systems, do not differentiate between the slope of the invert of the pipe and the hydraulic gradient of the pipe run. The hydraulic gradient should be considered in the design of storm drains because it is used in the solution of velocity and discharge. The hydraulic gradient at the upper end of the line should be established near the elevation of the inlet grate. The ponding volume may produce at times a higher elevation of the hydraulic gradient at this point.

(2) Past experience shows that a mean velocity of 2.5 feet per second will normally prevent the depositing of suspended matter in the pipes. Economy of design and topography will control the velocities. When lower velocities are used, special care should be taken in the construction of the system to assure good alignment, straight grades, smooth well- constructed joints, and proper installation of structures. The pipelines and slopes should be designed, wherever possible and when topographical conditions permit, so that the velocity of flow will increase progressively or be maintained uniformly from inlets to outfall. Thus the suspended matter will be carried through the system and out the outfall end.

c. The conduits in the drainage system may be constructed of reinforced concrete, concrete vitrified clay, corrugated steel, corrugated aluminum alloy, or asbestos-cement pipe. The pipes should be of conventional standard sizes and provided with either bell-and-spigot or tongue-and-groove joints in the precast pipes, adequate metal bands for the corrugated metal pipe, and couplings for the asbestos-cement pipe.

d. The chemical characteristics of water and soil which might affect the durability of drainage pipes should be investigated. The type of pipe least affected by those chemicals should then be recommended for installation.

e. Bituminous coated, partially or fully paved corrugated metal pipe should not be installed where fuel spillage, wash-rack wastes, or solvents can be expected to enter the pipe. Such coating or paving is soluble in aircraft fuel and some aircraft washing liquids. Such liquids are also flammable, thus a fire carried into the pipe would damage the pipe severely. Design of the drainage system for aircraft fuelling aprons should:

(1) Prevent spread of a fuel spill to structures, passenger loading fingers, or concourses, which could result in the fuel or vapors therefrom reaching a sourse of ignition or might release dangerous or toxic vapors within the structure itself.

(2) Prevent spread of a fuel spill over large areas of the apron surface and the transmission of vapors, which may expose a number of aircraft or other equipment on the apron.

(3) Provide for the safe disposal of fuel spillage.

These objectives may require consideration of one or several means. For example, the apron should slope away from buildings. Inlets should be located to allow reasonable flexibility in parking of aircraft without the likelihood of aircraft being positioned over inlets. Sections of drainage systems should be isolated at intervals through use of water seal traps or interceptors or separators to prevent transmission of flame or vapor through the system.

TABLE II. Coefficient of roughness

PIPE	Coeffic. "n"
Clay, concrete, and asbestos cement	0.012
Corrugated metal	
Fully paved	0.012
25% Paved, 2⅔ x ½ inch corr	0.021
3 x 1 or 6 x 1 inch corr	0.023
6 x 2 or 9 x 2½ inch corr	0.026
Unpaved, 2⅔ x ½ inch corr	0.024
3 x 1 or 6 x 1 inch corr	0.027
6 x 2 or 9 x 2½ inch corr	0.031
2 x ½ inch helical corr	
12 inch diameter	0.012
18 inch diameter	0.015
24 inch diameter	0.017
30 inch diameter to 48 inch diameter	0.018 to 0.021

OPEN CHANNELS	Maximum Permissible Velocity in Feet/Second	Coeffic. "n"
Paved		
Concrete	20 to 30+	0.011 to 0.020
Asphalt	12 to 15+	0.013 to 0.017
Rubble or Riprap	20 to 25	0.017 to 0.030
Earth		
Bare, sandy silt, weathered	2.0	0.020
Silt clay or soft shale	3.5	0.020
Clay	6.0	0.020
Soft sandstone	8.0	0.020
Clean gravelly soil	6.0	0.025
Natural earth, with vegetation	6.0	0.030 to 0.150*
Turf		
Shallow flow	6.0	0.06 to 0.08
Depth of flow over 1 foot	6.0	0.04 to 0.06

*Will vary with straightness of alignment, smoothness of bed and side slopes, and whether channel has light vegetation or is choked with weeds and brush.

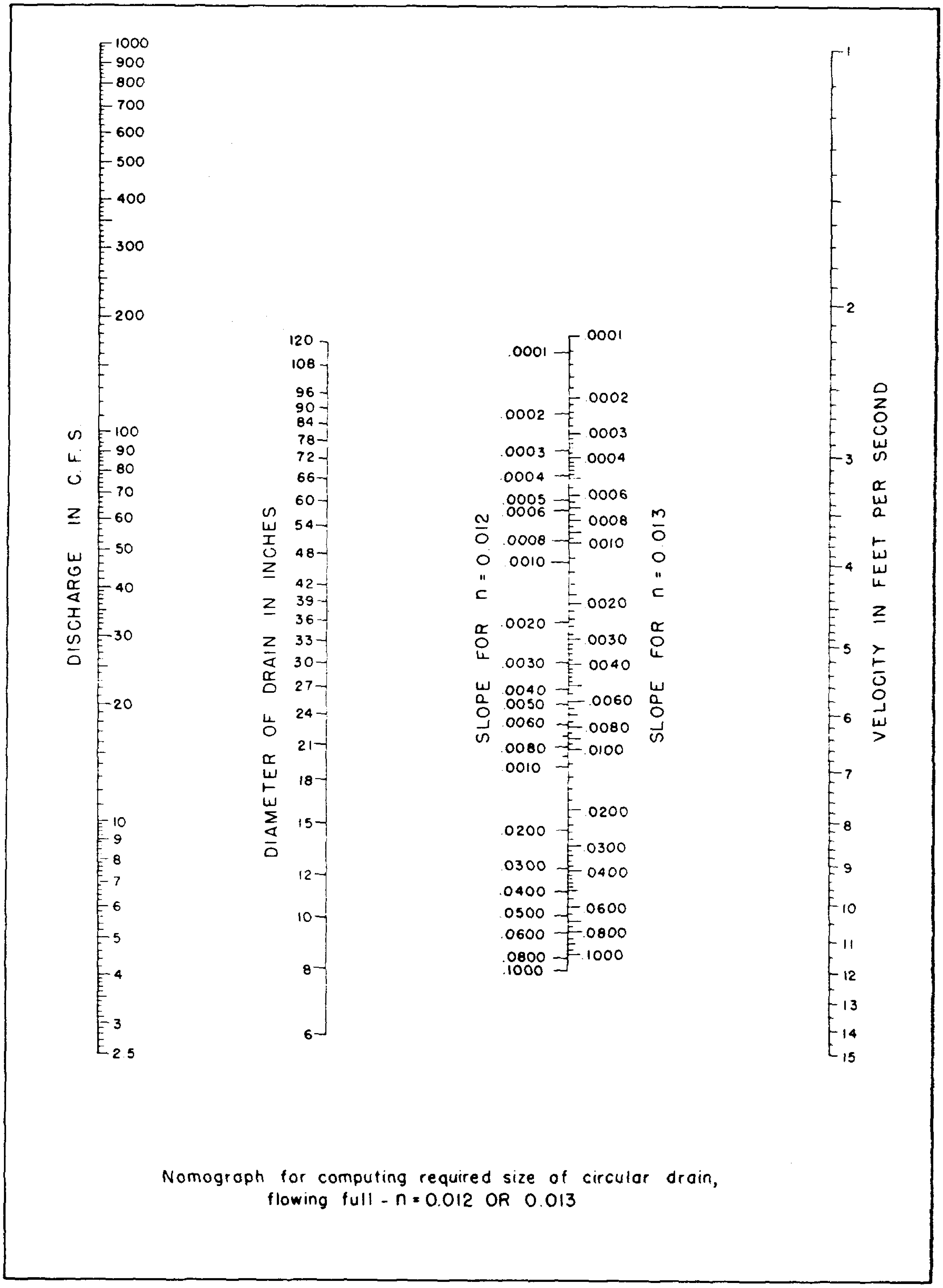


FIGURE 9. Nomograph for computing required size of circular drain for n 0.012 or 0.013.

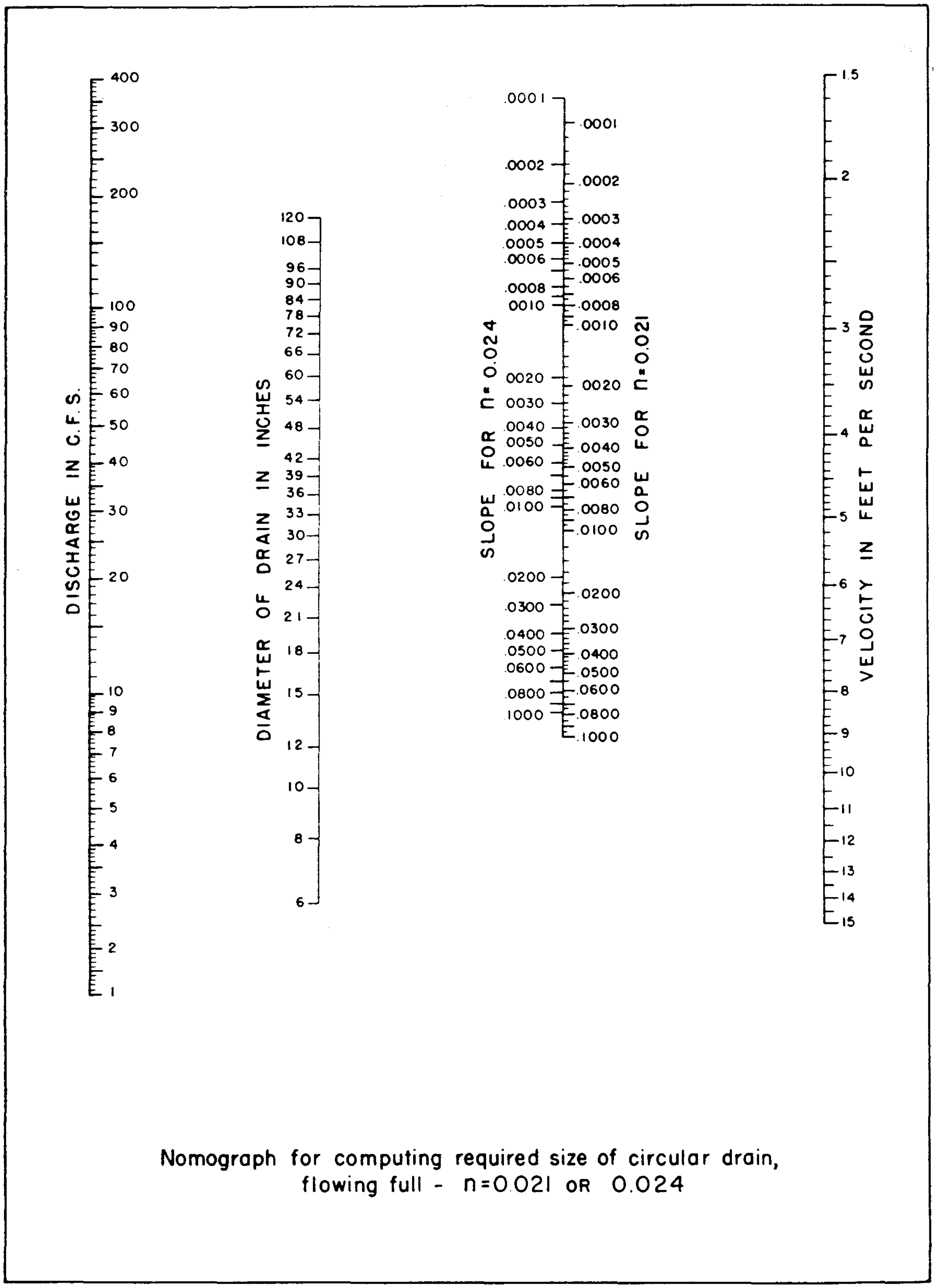


FIGURE 10. Nomograph for computing required size of circular drain for n 0.021 or 0.024.

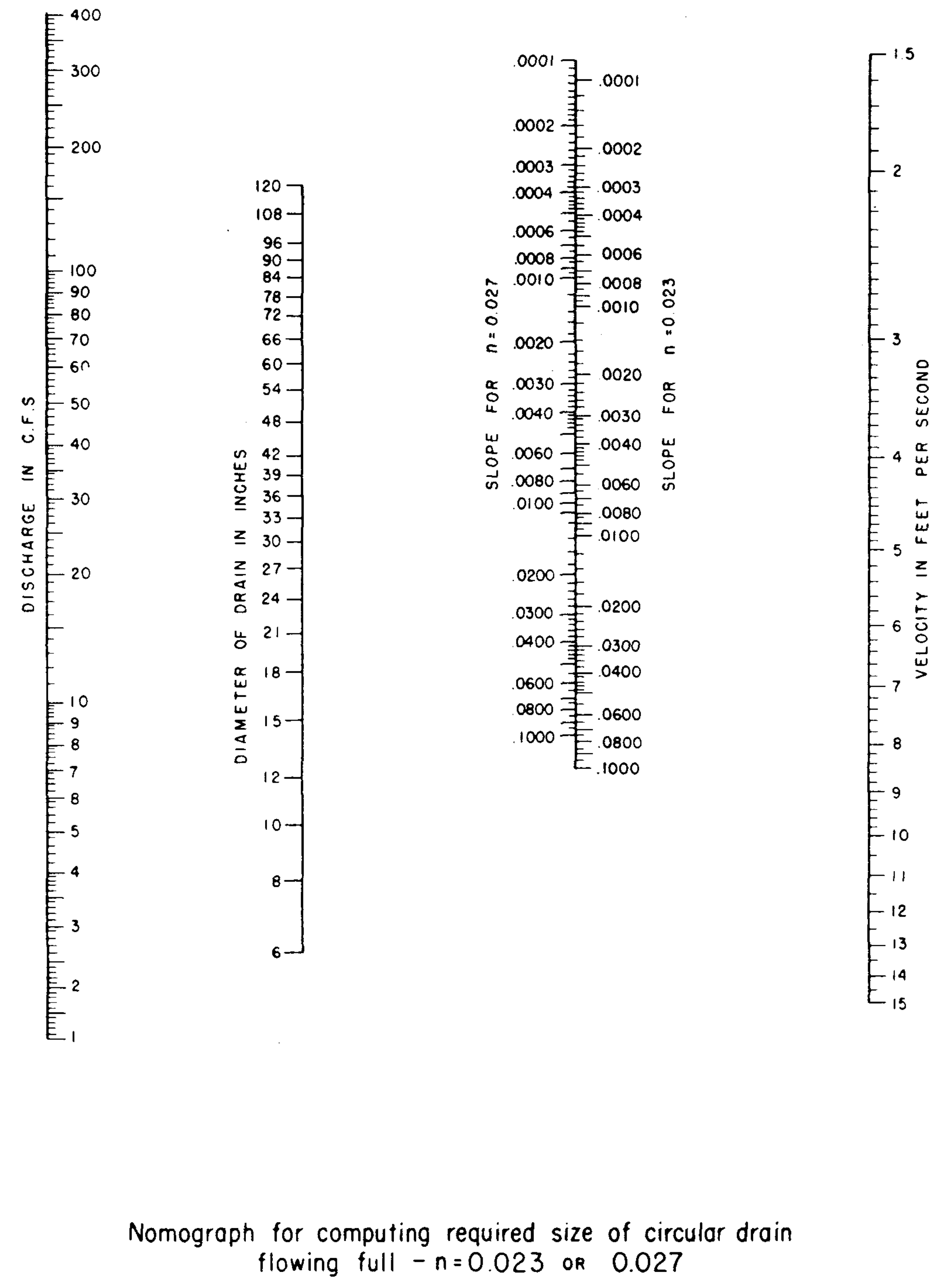
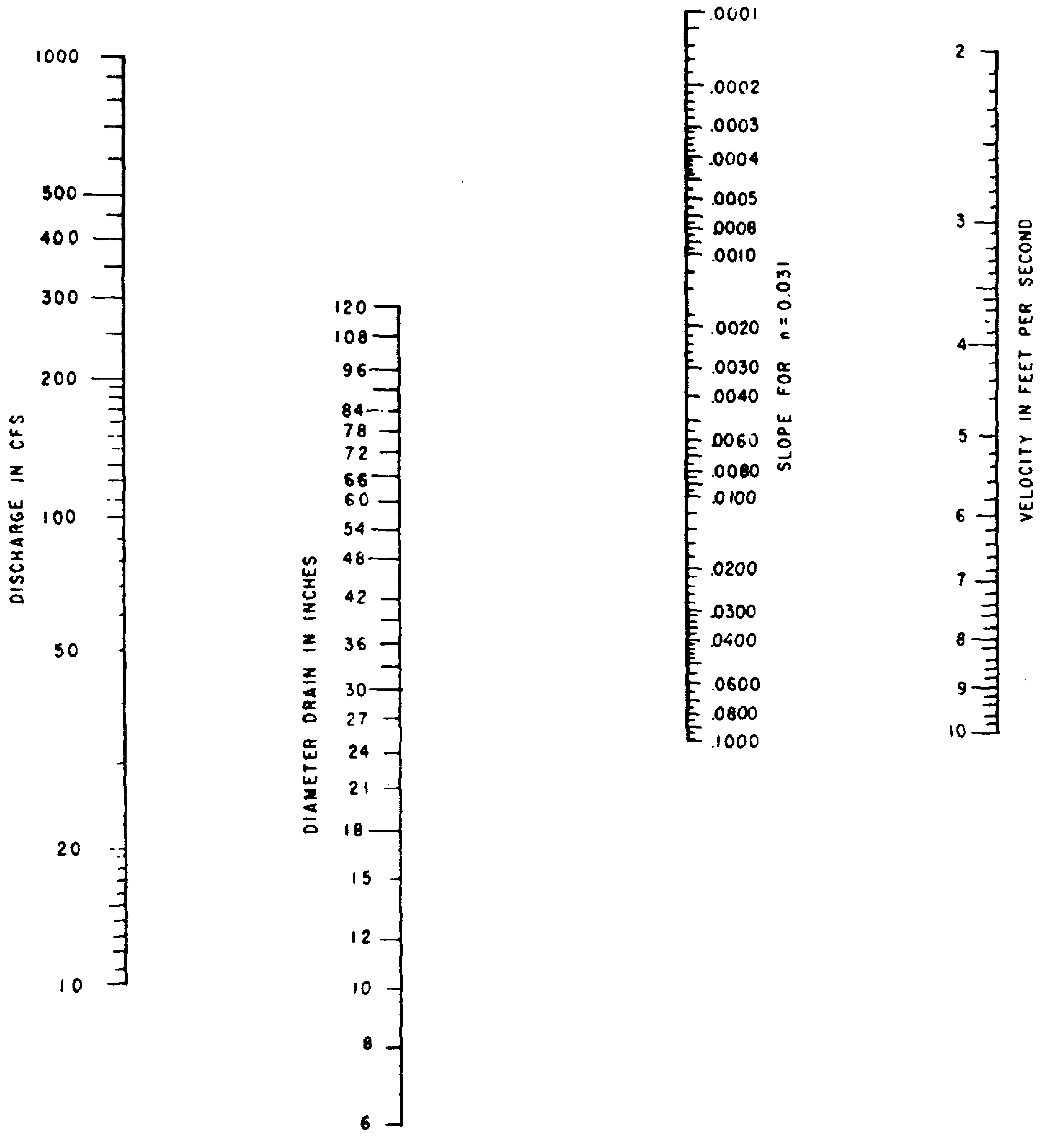


FIGURE 11. Nomograph for computing required size of circular drain for n 0.023 or 0.027.



NOMOGRAPH FOR COMPUTING REQUIRED
SIZE OF CIRCULAR DRAIN
FLOWING FULL

FIGURE 12. Nomograph for computing required size of circular drain for n 0.031.

9. FLOW IN OPEN CHANNELS.

a. For our purposes, an open channel is defied as any conduit in which water flows with a free-water surface such as an open ditch, creek, river, or canal. Although pipe drains act as open channels when flowing partially full, they are considered separately in paragraph 8 and elsewhere in this publication.

b. It is recommended that maximum use be made of open channels because of the relatively low cost thereof as compared with capacity attained. Obviously, open channels need to be consistent with safety and operational requirements of the airport. Channels shall be of adequate capacity and free from excessive maintenance which could result from erosion, silting, or steep back slopes. In the latter connection, it is noted that mowers cannot normally be used on slopes steeper than 2.5 to 1. Slopes steeper than 2.5 to 1 will usually need to be cut by hand or cut by mowers restrained by other equipment at the top of the slope. Some areas of the country, where soil is generally noncohesive and vegetation is likely to be sparse, use 3 to 1 as maximum.

c. The Manning formula may be applied directly to hard surfaced channels of concrete, corrugated metal, etc., and to earth channels using coefficient of roughness values for "n" as shown in Table II. That table also lists maximum permissible velocities for such channels.

d. A nomographic solution of the Manning formula can be made by using Figure 13. The channel-dimension diagrams for trapezoidal, triangular, and parabolic shaped channels, Figures 14 through 19 permit rapid solution of many ordinary channel problems.

e. The Manning formula is $Q = AV$ where:

Q = rate of discharge or flow in cubic feet per second.

A = cross sectional area of the flow in square feet.

$V = \frac{1.486\ R^{2/3}\ S^{1/2}}{n}$ = velocity of flow in feet per second.

n = coefficient of roughness.

R = hydraulic radius in feet or cross sectional area A divided by the wetted perimeter.

S = slope of energy gradient in feet per foot, considered equal to slope of channel bed for most situations.

f. The following indicates the use of the channel-dimension diagrams in Figures 14 to 19.

(1) For example, given $Q = 100$ cubic feet per second and $S = 0.0002$ feet per foot.

Find: The bottom width, depth of flow, and velocity of a drainage channel with 2.5 to 1 side slope. Assume channel will be maintained regularly and kept free of excessive vegetal growth. Estimate "n" to be 0.04. (See Table II for open channel "n" values).

Solution: Requires one or more trial solutions. The steps are: (a) select a velocity, (b) compute the area required from Q/V, (c) determine the required R from the nomograph of Figure 13, and (d) determine the bottom width and depth from the 2.5 to 1 dimension diagram of Figure 14.

Item	*Trial Solution* 1	2	3
V=feet per second (selected)	1.0	0.9	0.95
A=square feet (from Q/V)	100	111.1	105.6
R=feet (from Figure 13)	2.60	2.20	2.40
b=bottom width in feet (from Figure 14)	20	36	28
d=depth of flow (from Figure 14)	3.6	2.6	3.1

The solution chosen will depend upon depth and botom width of channel desired. If still not satisfactory, change *V* slightly and redetermine dimensions. Final selection of channel dimensions must consider addition of reasonable freeboard to depth of flow.

(2) For example, given $Q = 50$ c.f.s. and trapezoidal channel with side slopes of 3 to 1 and bottom width of 10 feet.

Find: The bed slope required to maintain a velocity of 2.5 f.p.s. Use "n" = 0.04.

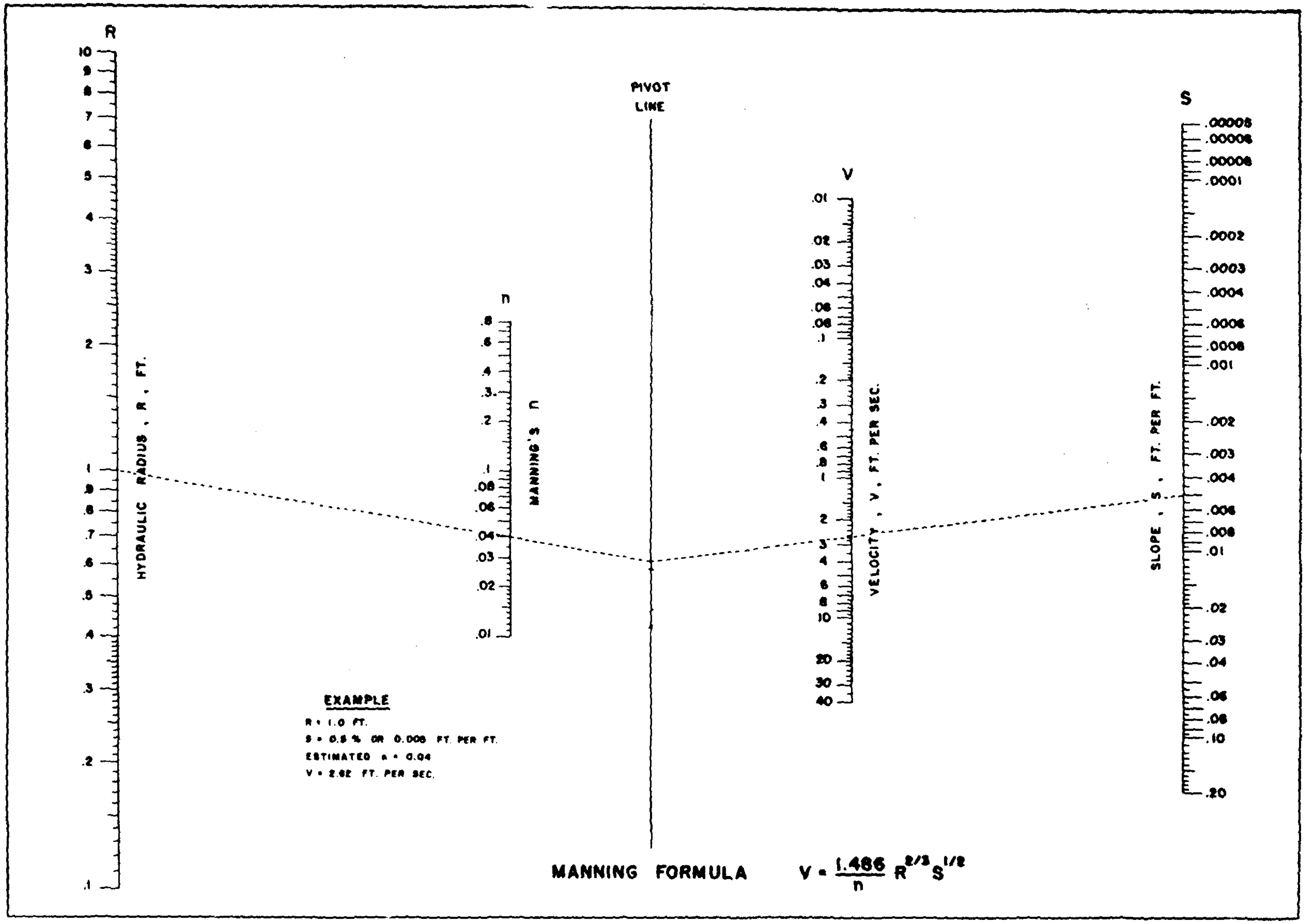


FIGURE 13. Solution of the Manning formula.

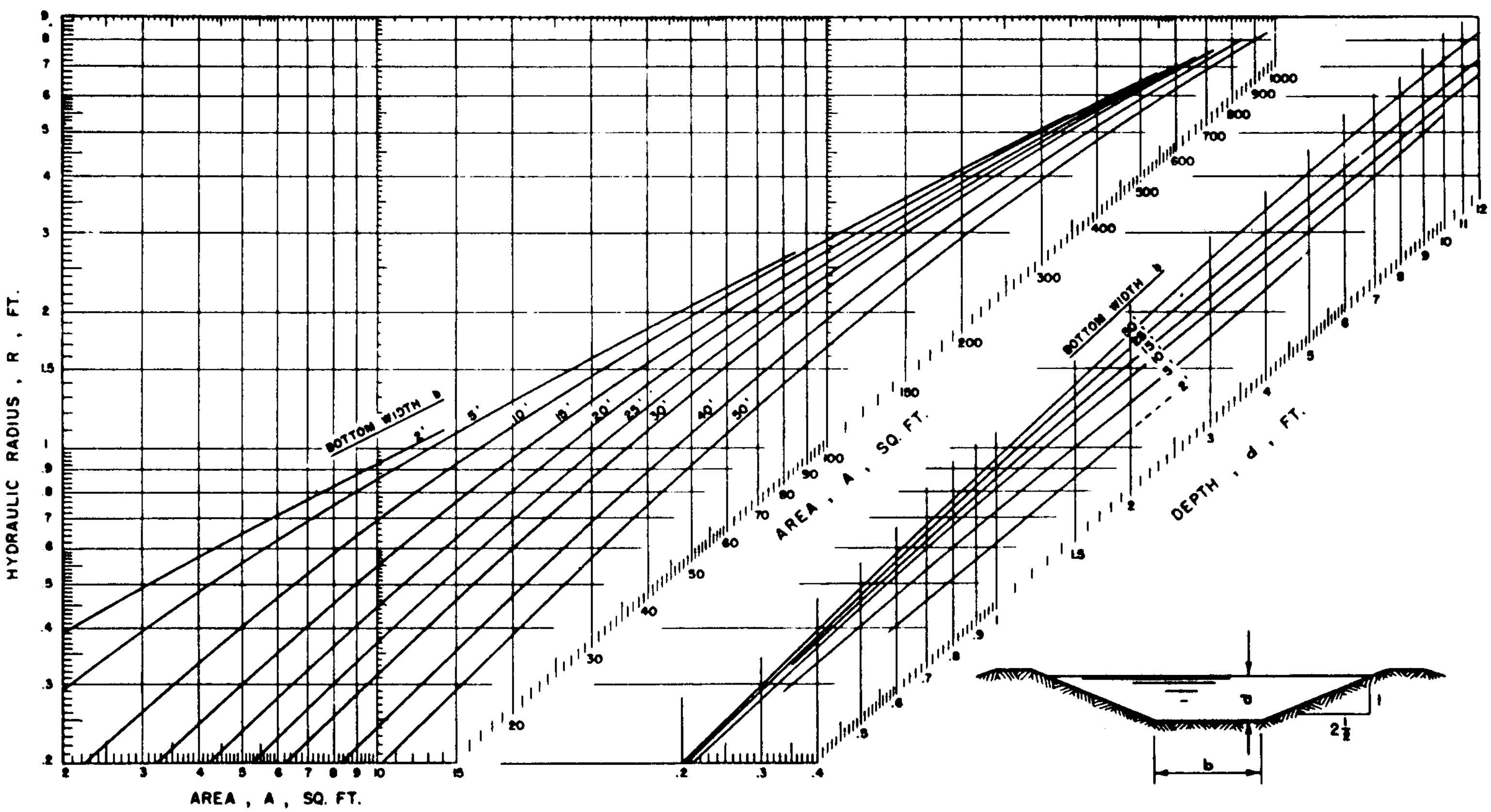


FIGURE 14. Dimensions of trapezoidal channels with 2½ to 1 side slope.

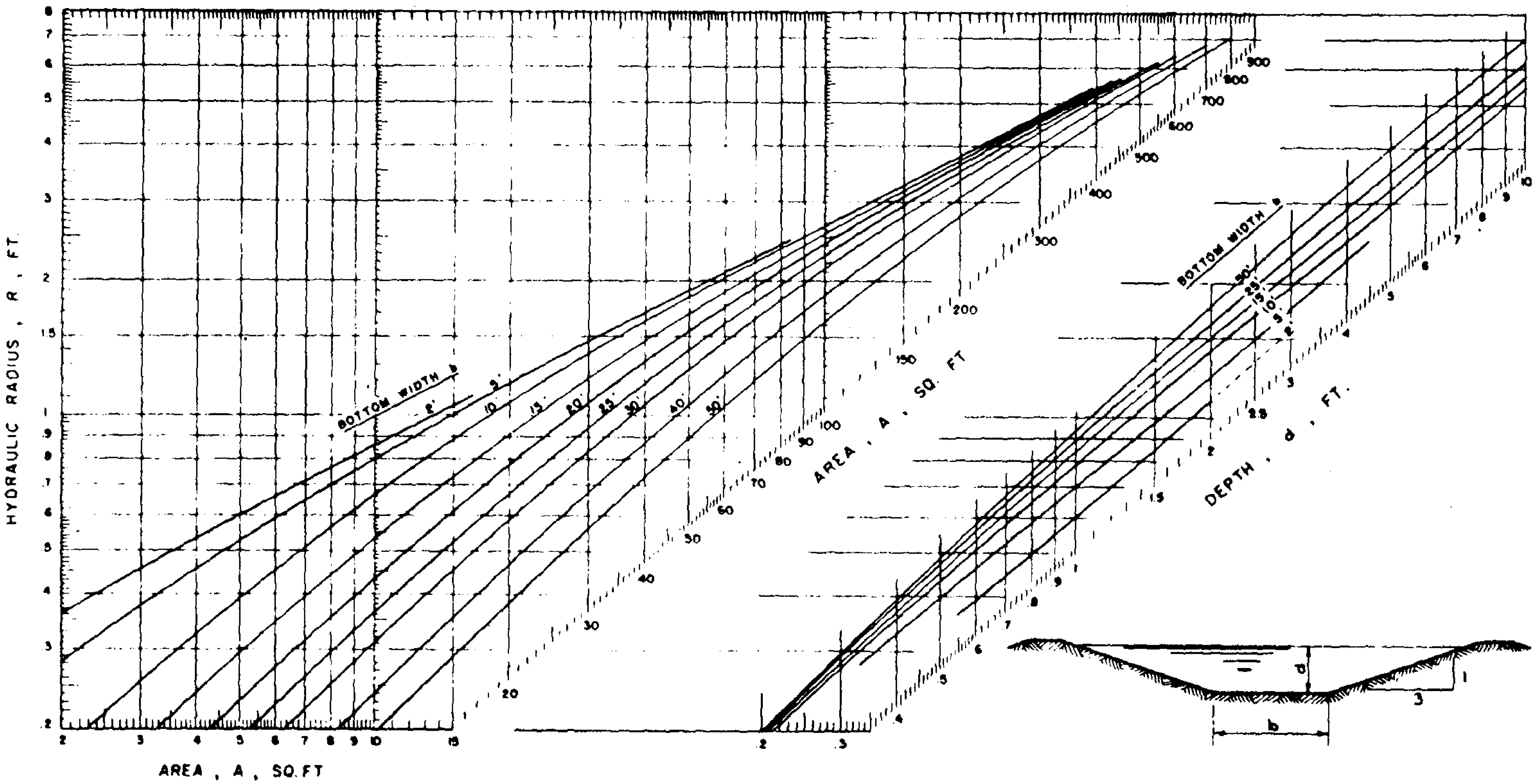


FIGURE 15. Dimensions of trapezoidal channels with 3 to 1 side slopes.

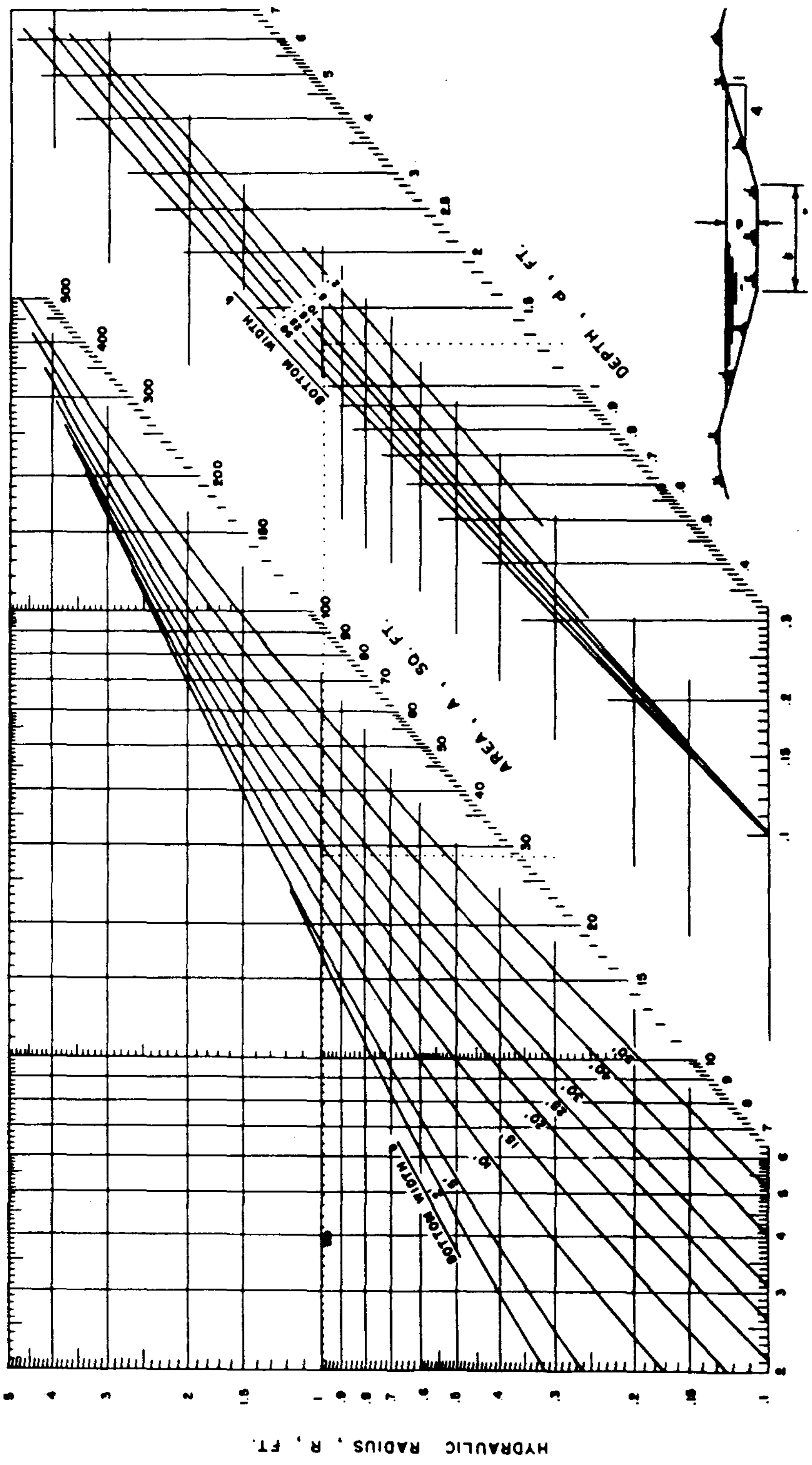


FIGURE 16. Dimensions of trapezoidal channels with 4 to 1 side slopes.

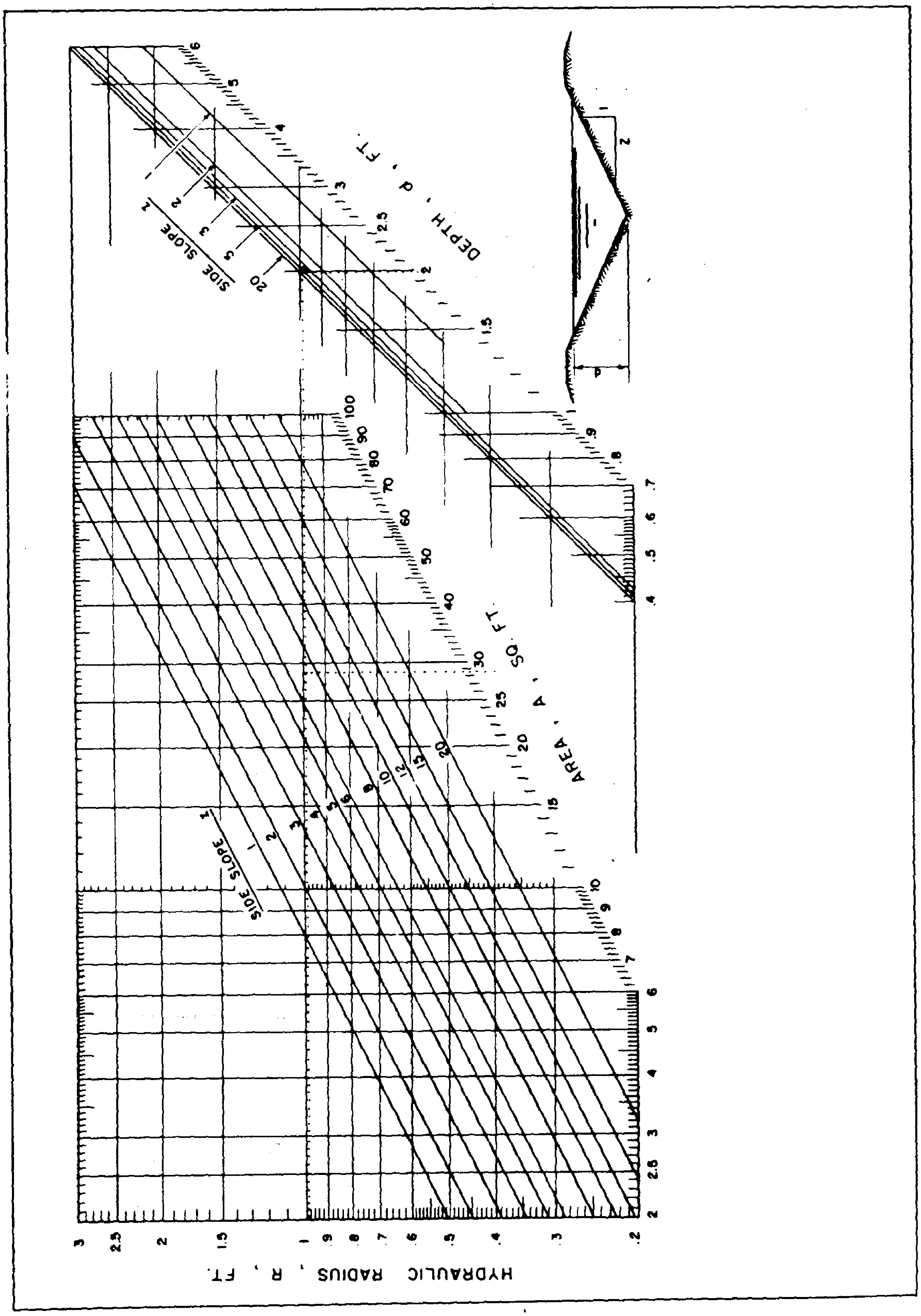


FIGURE 17. Dimensions of triangular channels.

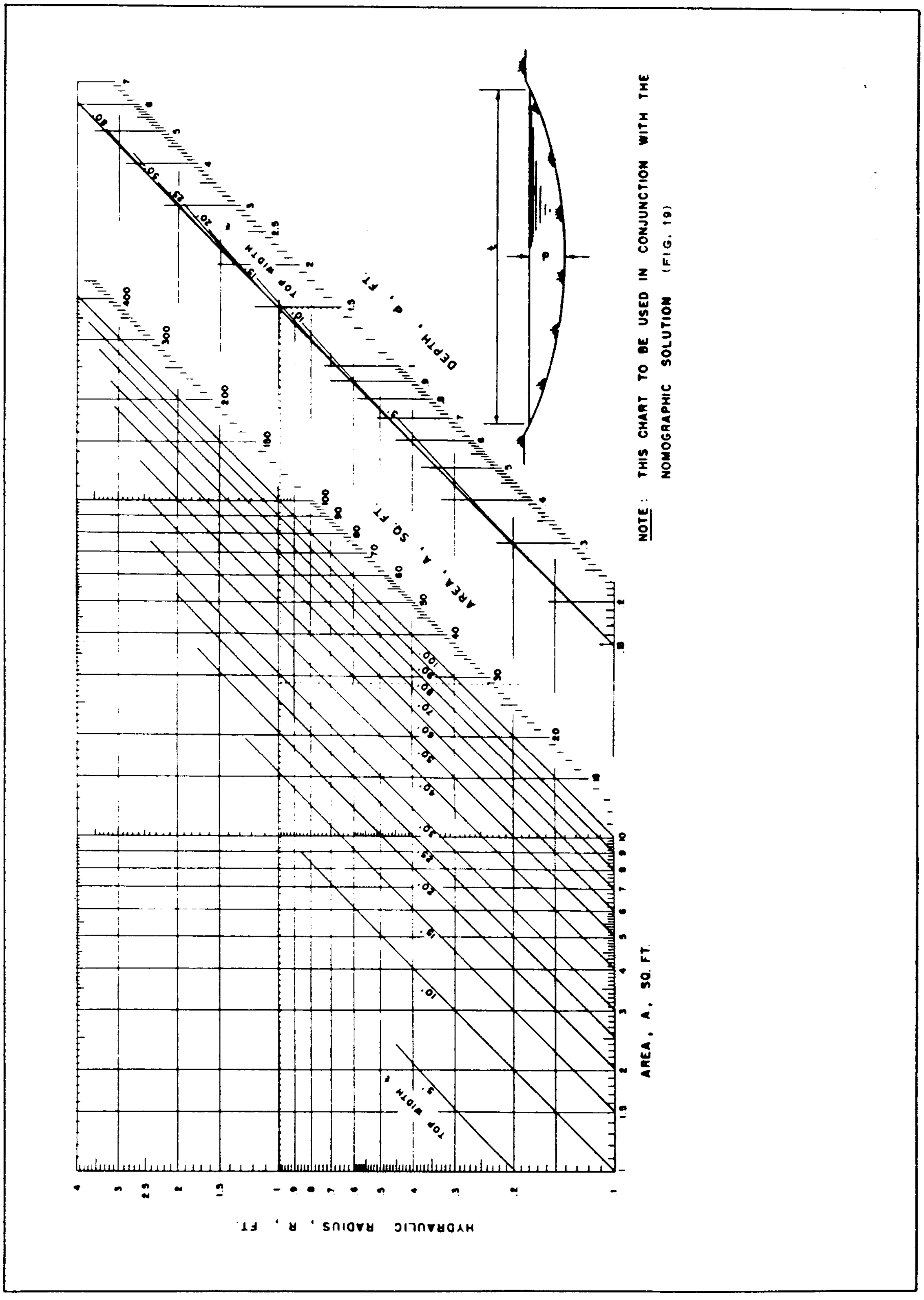
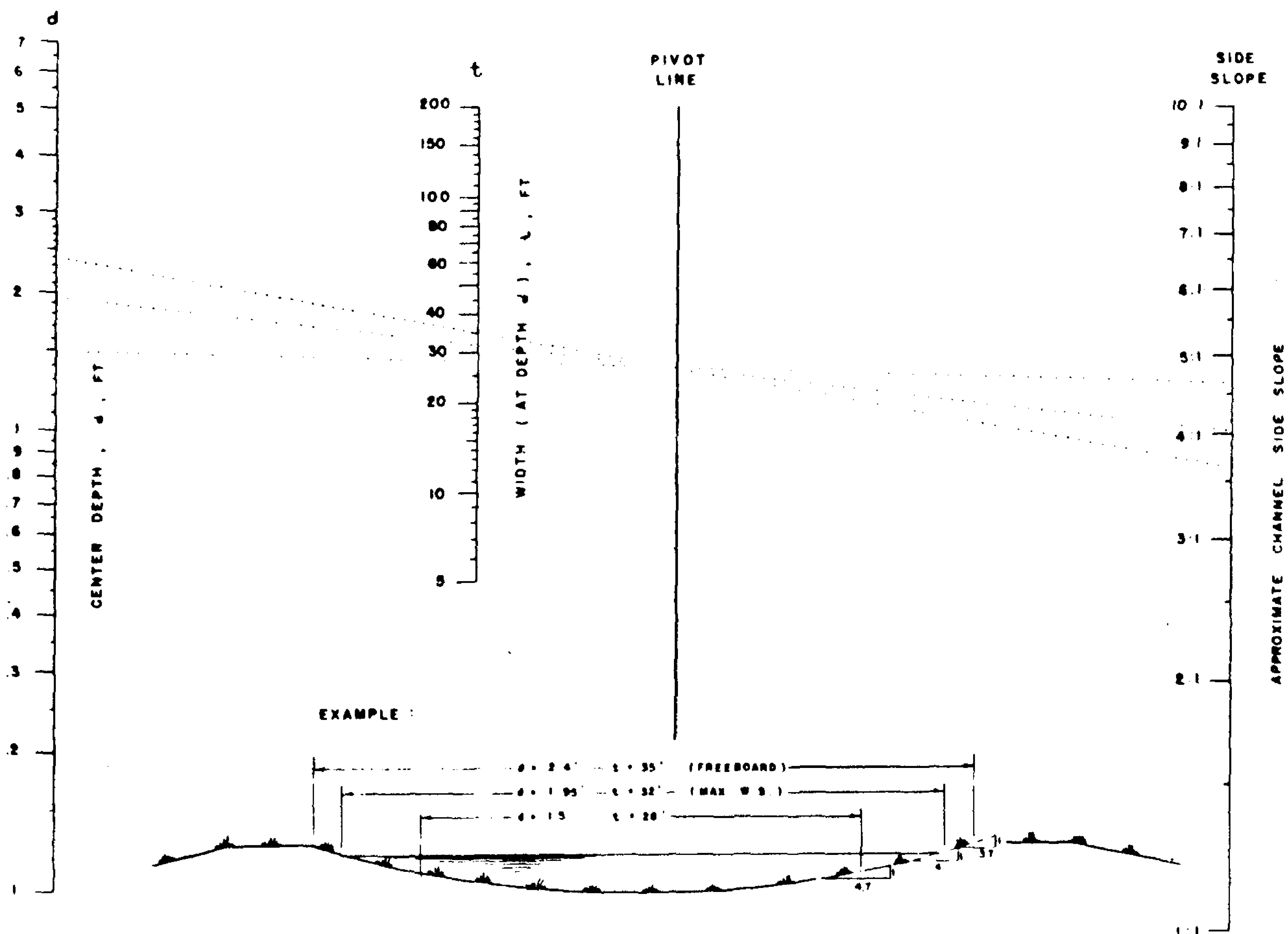
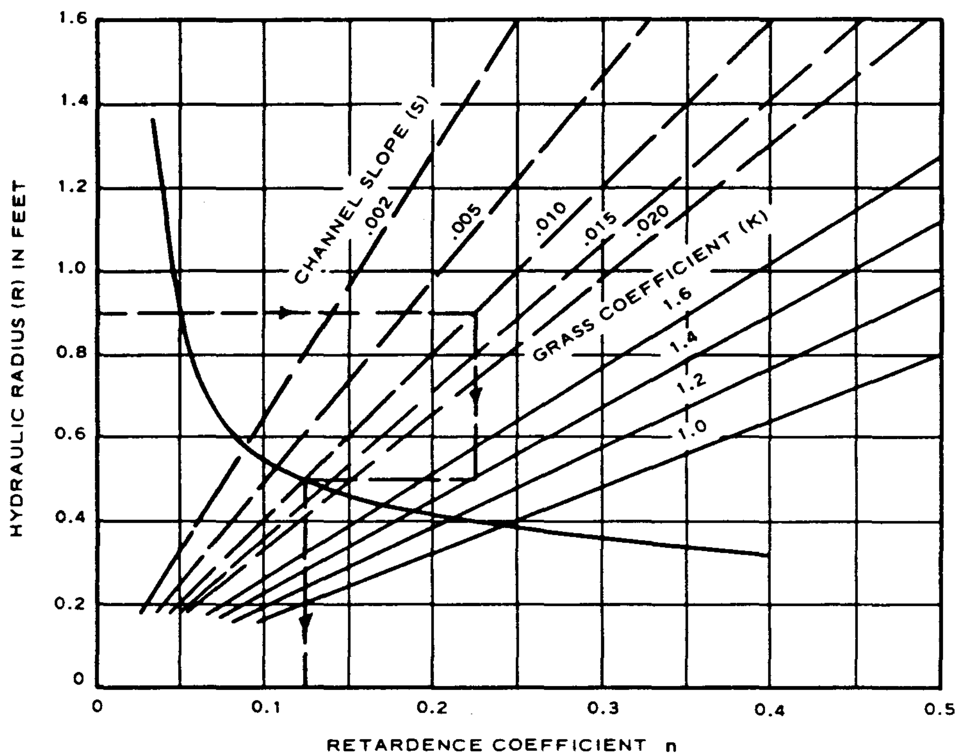


FIGURE 18. Dimensions of parabolic channels.



NOTE THIS CHART TO BE USED TO OBTAIN OTHER DIMENSIONS AFTER t AND d FOR MAXIMUM FLOW HAVE BEEN DETERMINED (FROM FIG. 18)

FIGURE 19. Solution for dimensions of parabolic channels.



GRASS COEFFICIENTS (K) FOR DENSE AIRFIELD TURF

GRASS SPECIES	AVG LENGTH OF GRASS IN INCHES		
	<6	6-12	>12
BUFFALO	1.6	--	--
BLUE GRAMMA	1.5	1.4	1.3
BLUE GRASS	1.4	1.3	1.2
BERMUDA	1.4	1.3	1.2
LESPEDEZA SERICEA	1.3	1.2	1.1

EXAMPLE:

DETERMINE n FOR 4-INCH BERMUDA GRASS CHANNEL WITH R = 0.9 AND S = .010.

FROM TABLE K = 1.4 AND FROM GRAPH, FOLLOWING DASHED LINE, n IS EQUAL TO 0.125.

FIGURE 20. Retardence coefficients for flow in turfed channels.

Solution: The area required is Q/V or 20 sq. ft. Enter Figure 15 with A = and b = 10, and find that R = 1.08 feet. Then in the nomograph, Figure 13, extend a line from R = 1.08 through n = 0.04 to the pivot line, extend a line through the velocity scale at 2.5 to the slope line. The slope required is found to be 0.004 feet per foot. Depth of flow from Figure 15 is found to be 1.45 feet.

(3) For example, given an existing channel with side slopes of 3:1, bottom width 12 feet, depth 3.5 feet, bed slope 0.0008 feet per foot. Channel has not been well maintained and some willows and other vegetal growth exists, therefore, estimate "n" to be 0.06.

Find: Present capacity allowing a 0.5-foot freeboard.

Solution: Depth of flow will be 3.5–0.5 or 3.0 feet. From Figure 15, R = 2.05 feet and A = 63 square feet. From Figure 13, V = 1.15 feet per second. Then Q, as channel capacity, = 63 × 1.15 = 72.45 c.f.s.

g. When the channel is to be lined with vegetation, the design problem is complicated by a vegetal retardence element, which is a function of both the turf characteristics and the depth and velocity of flow. This retardal element or coefficient of roughness varies with VR, the product of velocity and hydraulic radius. The degree of vegetal retardence depends on the height and density of cover, particularly the height.

Most drainage references include 4 or 5 degrees of retardence, which are related to type and height of grass in the channel. As shown by Figure 20, having a known hydraulic radius, slope of channel, and selecting a grass coefficient, one can obtain a retardence or roughness coefficient.

10. STRUCTURES.

a. The structures usually built in connection with airport drainage are quite similar to those used in municipal construction. Generally speaking, the standard types are adequate, but occasionally a special type of structure will be needed. Structures located in the usable areas on airports should be so designed that they do not extend above the ground level. The tops of such structures should be one or two-tenths of a foot below the ground line to allow for possible settlement around the structure, to permit unobstructed use of the area by equipment, and to facilitate entrance of surface water.

b. The structures most generally used are inlets, manholes, combination manholes and inlets, catch basins, lampholes, and headwalls. Some of these structures will be covered with a grate when it is necessary to admit the surface water into the system. The grates may be of cast iron, steel, or ductile iron. Several suggested designs of grates and inlets are shown in Figures 21 and 22. For suggested headwall details, see Figure 23. Precast and manufactured headwalls are available.

c. The general designs of drainage structures used by the municipalities are quite alike; however, almost every large city has its own special standards which vary in details according to the desires and ideas of the design engineer. These structures all vary as to the design load they will support and should be thoroughly checked for load-carrying capacities.

d. In aircraft traffic areas, grates and frames should usually be specified to support loads from aircraft which will use the facility as these loads normally exceed those imposed by maintenance equipment or other traffic. Although roadway type grates used by the municipality or highway department will certainly be adequate for all single gear and some dual gear aircraft, it is apparent that most dual and dual tandem gear aircraft weigh far more than highway vehicles and have higher tire pressures. Representative takeoff weights and tire pressure are:

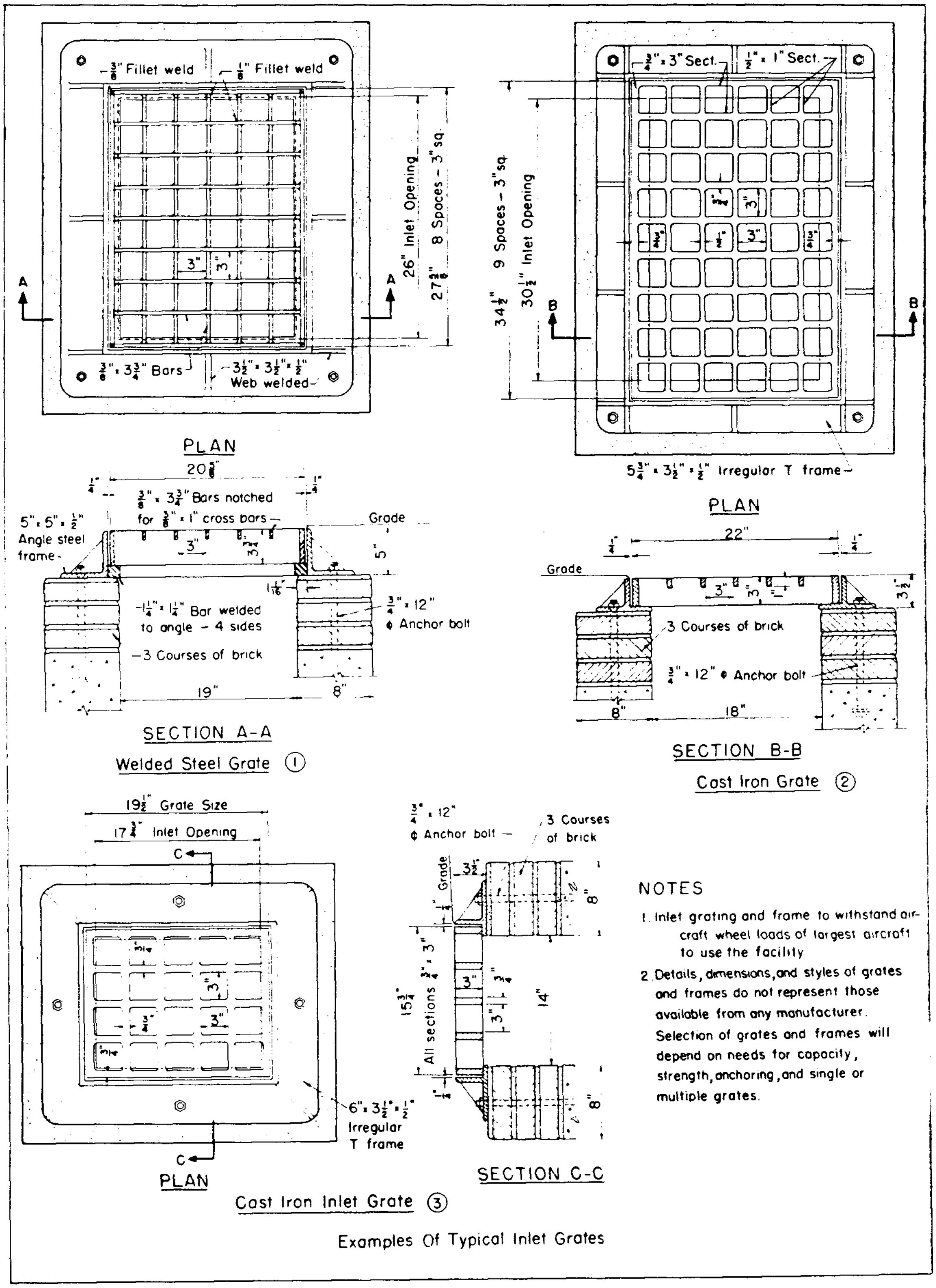


FIGURE 21. Examples of typical inlet grates.

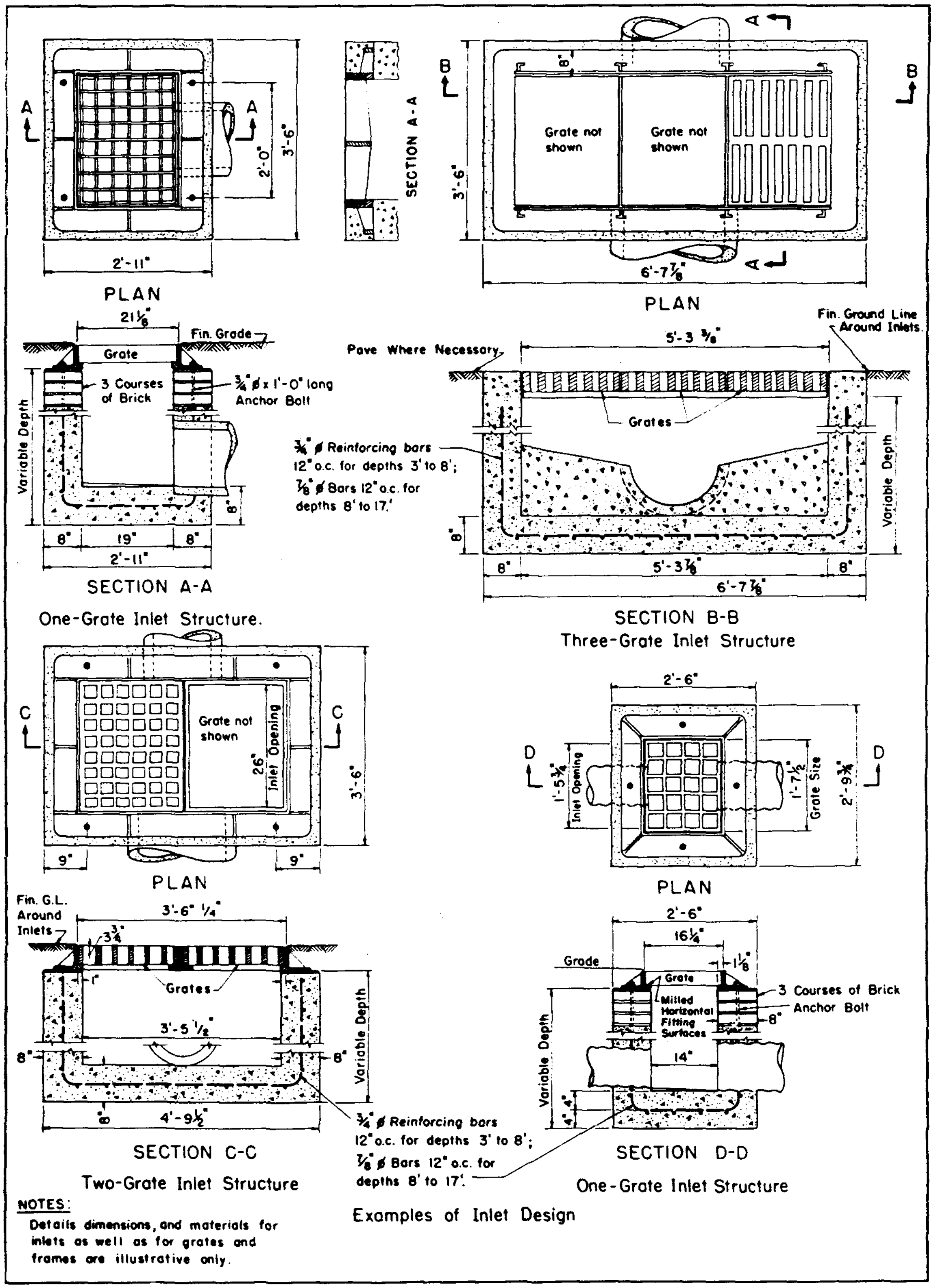
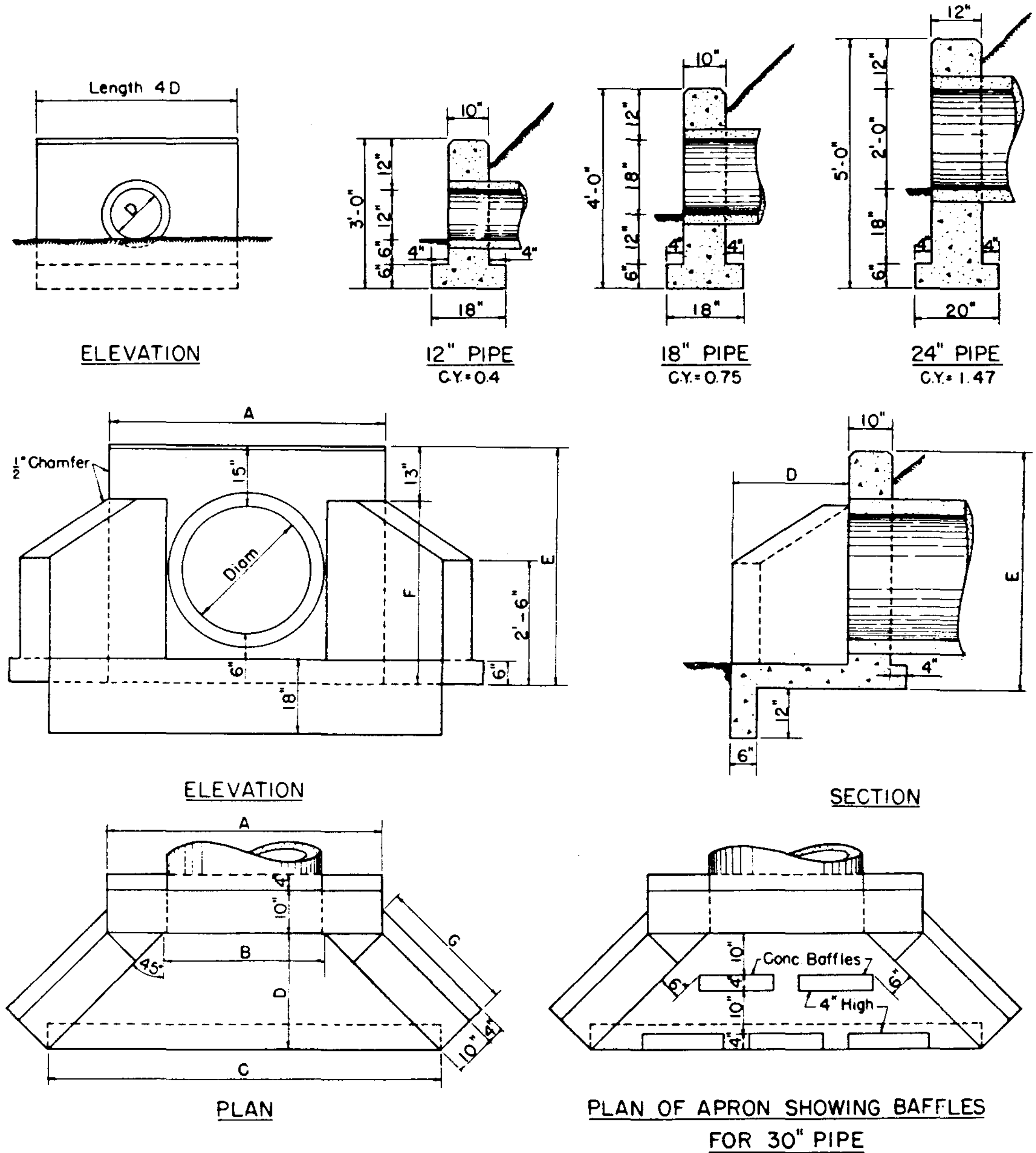


FIGURE 22. Examples of inlet design.



Diam. of Pipe	C.Y Conc. Headwall	A	B	C	D	E	F	G
30"	1.67	5'-6"	3'-2"	7'-10"	2'-4"	4'-9"	3'-8"	2'-5 3/4"
36"	2.24	6'-0"	3'-8"	9'-10"	3'-1"	5'-3"	4'-2"	3'-6 1/2"
42"	2.88	7'-0"	4'-2"	11'-10"	3'-10"	5'-9"	4'-8"	4'-5 1/2"
48"	3.59	7'-6"	4'-8"	13'-10"	4'-7"	6'-3"	5'-2"	5'-6 3/4"

NOTES

1. Reinforcing material may be installed in headwalls whenever necessary.
2. Baffles may be installed in headwall aprons to break up excess velocity of water.

Suggested Types of Headwalls

FIGURE 23. Suggested types of headwalls.

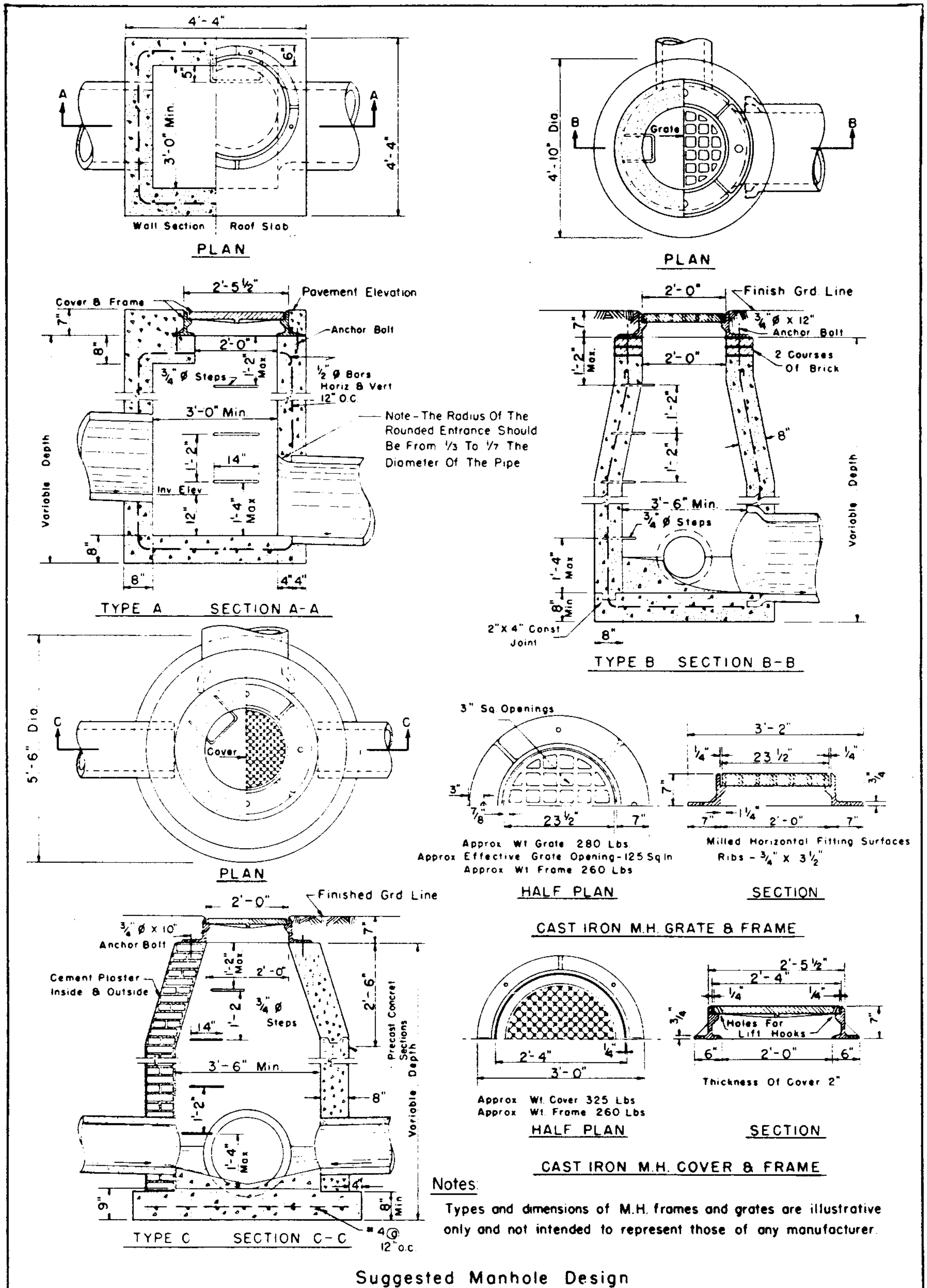
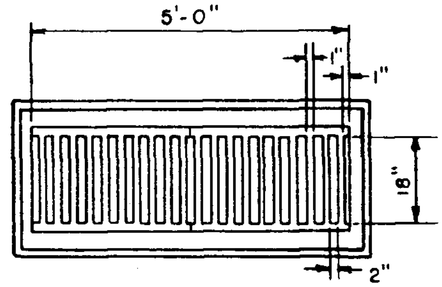


FIGURE 24. Suggested manhole design.



TYPICAL PLAN OF DOUBLE INLET GRATING

WATERWAY OPENING = 5.0 SQ. FT. (DOUBLE GRATING)

ASSUME GRATING IS PLACED SO THAT FLOW WILL OCCUR FROM ALL SIDES OF INLET. FOR LOW HEADS DISCHARGE WILL CONFORM WITH GENERAL WEIR EQUATION.

$Q = CLH^{3/2}$

WHERE

C = 3.0

L = 13.0 FT. GROSS PERIMETER OF GRATE OPENING (OMITTING BARS) FOR GRATE ILLUSTRATED

H = HEAD IN FEET

FOR HIGH HEADS DISCHARGE WILL CONFORM WITH ORIFICE FORMULA:

$Q = CA\sqrt{2gH}$

WHERE

C = 0.6

A = 5.0 SQ. FT.

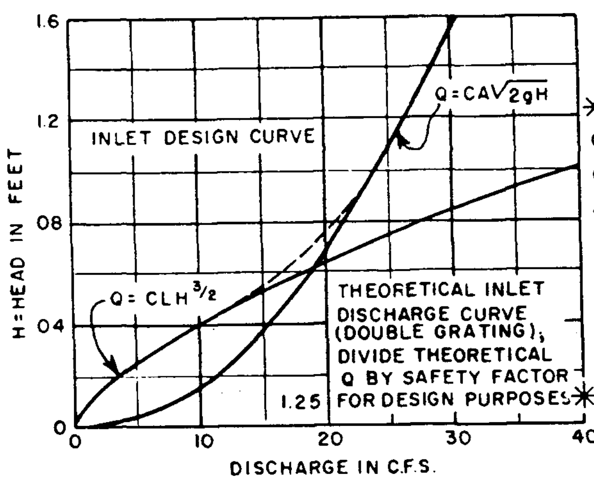
g = ACCELERATION OF GRAVITY IN FEET PER SECOND²

H = HEAD IN FEET

THEORETICAL DISCHARGE RELATION TO BE MODIFIED BY 1.25 SAFETY FACTOR

COEFFICIENTS BASED ON MODEL TEST OF SIMILAR GRATES WITH RATIO:

NET WIDTH OF GRATE OPENING TO GROSS WIDTH = 2:3



*USE SAFETY FACTOR OF 1.5 TO 2. FOR GRATES IN TURFED AREAS.

DETERMINATION OF TYPICAL INLET GRATING DISCHARGE CURVE

FIGURE 25. Determination of typical inlet grating discharge curve.

Aircraft type	Maximum takeoff weight (lb.)	Tire pressure (p.s.i.)	Gear type
Martin 202, 404; F-27, FH-227; Convairs 240, 340, 440	39,900 to 53,200	60 to 95	Dual
Viscount; BAC-111; DC-4	64,500 to 87,000	75 to 120	Dual
Constellations; DC-9; B-737; DC-7; B-727	90,700 to 170,000	95 to 168	Dual
Caravelle	110,200	124	Dual tandem
Convair 880, 990; B-720, B-707	184,500 to 312,000	120 to 172	Dual tandem
DC-8	273,000 to 355,000	148 to 168	Dual tandem
Concorde	367,000	189	Dual tandem
DC-10; L-1011	411,000 to 518,000	170 to175	DC-10=Dual tandem +2 L-1011=Dual tandem
B-SST	750,000	220	12 Wheel
B-747; L-500	713,000 to 861,500	185 to 210	Dual, dual tandem
Future heavy aircraft	1,500,000	250	(See App. 2 of AC 150/5320-6A)

Grates and frames for Utility and Basic Transport airports may be those used by the municipality or for highway loadings. Those for other airports should be selected to accept aircraft then using or expected to use the airport in the future.

Some manufacturers of grates and frames produce types specifically designed for airport loadings such as 100, 150, 200, and 250 p.s.i. Hold-down bolts, hooks, or other devices should be provided for the grates to prevent displacement by traffic. The engineer should specify the design load to be carried by the grates and frames and require the vendor to certify that the items furnished have that capacity.

e. The inlet structures may be constructed of reinforced concrete, brick, concrete block, precast concrete, or rubble masonry. They should be strong enough to withstand the loads to which they will be subjected.

f. Catch basins for airport drainage are not usually considered necessary particularly when drainage lines are laid on self-cleaning grades. Under certain conditions, catch basins are needed to prevent solids and debris from washing into the system. They should be cleaned out frequently and involve an additional maintenance problem.

g. Manholes are more or less standardized as to type and can be round, oval, square, or rectangular design. They are usually made of reinforced concrete, brick, concrete block, precast concrete, corrugated metal, or precast pipe sections (Figure 24). The design will depend on the stresses to which they will be subjected. Adequate unobstructed space must be provided within the manhole to enable workmen to clean out the line when necessary. Inside barrel dimensions equivalent to a diameter of 3-½ feet and a height of 4 feet are usually considered sufficient, but they can be varied to suit particular situations.

h. A gutter is not permissible along a runway or a taxiway as it could be hazardous to aircraft operations and would interrupt the runoff which should flow unobstructed transversely off the pavement and across the safety area to the field inlets. In wide apron areas, the inlets should be placed in the valley of the pavement at proper intervals to collect runoff.

i. Inlets in paved areas frequently cause trouble due to differential settlement of the drainage structure and the adjacent pavement. The resulting depressions around the inlets may result in pavement failure through softening of the subgrade. Particular care in compaction of backfill around inlets will prevent settlement. In rigid pavement, the structure is normally protected by expansion joints placed around the inlet frame. Also, construction joints are installed at a distance from the structure which either matches or is equal to the normal spacing for joints. The slab thus formed around the structure should include steel reinforcement to prevent cracking outward from each corner of the inlet.

j. The number and capacity of grates selected will be dictated by the quantity of runoff to be handled as well as by the depth of head at the grate. In low head situations, capacity (discharge) of the grate(s) will conform to the general weir formula, while in higher head situations the orifice formula is applicable. These formulas and the transition between them are described in Figure 25.

Medium or high head ponding will be unacceptable to personnel servicing aircraft with baggage, fuel, food, etc. Moreover, such ponding could obscure pavement markings and thus inhibit parking of aircraft at gate positions. In some cases, flooding of refueling pits would occur. Although inlets should be located beyond gate positions, controlling elevations and apron configuration may result in location of inlets fairly close to the field side of the airplane. For these reasons, the weir formula and low head is appropriate for use at some locations.

In general, aprons serving instrument or all weather air carrier operations warrant a head limitation of 0.4 feet. Also, airports where continuing operations are regularly conducted in severe weather conditions would warrant use of a similar limitation. This presumes that apron widths and configurations at other locations will allow runoff to flow directly to turfed areas or that the comparatively light traffic in severe weather conditions could adapt to deeper ponding. It is recommended that apron areas drain away from buildings, whenever possible.

The orifice formula portion of Figure 25 is applicable to grates in turfed areas, except for the unique case where ponding must be severely limited, such as small turfed areas between aprons and closeby taxiways. The safety factor of 1.25 mentioned in the diagram in Figure 25 is intended for use in paved areas. That factor should be increased to 1.5 to 2.0 for grates in turfed areas to compensate for grass cuttings. Replacement of drainage lines or structures in or under pavement would be expensive and would disrupt traffic, therefore, the drainage system should be designed with a capacity sufficient for the ultimate pavement configuration. Terminal apron drainage design should be coordinated with those responsible for the fueling methods and systems to be employed, as well as with those responsible for fire and rescue services.

In large paved areas, such as aprons, sometimes it is feasible to design several segments of the system without ridges between adjacent inlets, thus avoiding grade changes and attendant inhibitions to traffic. In these cases, the efficiency of the grate and its placement should be considered. For such efficiency, all rectangular bars should be parallel with the flow and the openings should cover at least 50 percent of the width of the grate. Multiple grates should be placed normal to the direction of the flow.

11. CULVERTS.

A drainage culvert is designed as a structure (other than a bridge) to convey water through or under a roadway, runway, taxiway, or other obstruction. The choice of circular, oval, elliptical, arch, or box cross section and single or multiple installation will depend on capacity, headroom, economy and occasionally be dictated by local rules or requirements.

The development of a new airport site, or the interruption of natural stream channels by airport facilities, or the increase in runoff caused by airport development may require the design of culverts or consideration of the capacity of existing culverts. In some cases, the airport storm sewer system will need to accept off-site runoff and culverts under roads on the airport perimeter could indicate the volume of runoff to be accommodated.

Local highway departments or drainage districts normally have jurisdiction over design and construction of culverts and channels. Indeed, such governmental units may insist on use of certain design criteria. Accordingly, proposed modification or improvement of drainage facilities under roadways or off-site should be reviewed and approved by the responsible government agency.

The objective in the design of an economical culvert is to provide a waterway opening adequate for the passage of floods, and selection of a culvert meeting that objective is based on the culvert's hydraulic capacity and site conditions.

Many formulas have been developed over the years to estimate runoff/flood discharge and to determine the size of a culvert. The magnitude of the design flood is a function of its frequency, therefore, it is important to use the appropriate frequency.

Because of the potential risk to adjacent property and to automobile traffic, the design flood frequency used for culverts under roads is for a greater recurrence interval than that used for airport storm sewer systems. The local highway department or drainage district may establish the flood frequency to be used: common practice is to design culverts for minor roads for 10- to 25-year flood frequency and for major highways for a recurrence interval of 50 years or more.

The rate of discharge or flow through a culvert involves either of two major types of culvert flow; (1) flow with inlet control or (2) flow with outlet control. Under inlet control, the cross sectional area of the culvert, the inlet geometry, and the amount of headwater or ponding at the entrance are of primary importance. Outlet control involves the additional consideration of the elevation of the tailwater in the outlet channel and the slope, roughness, and length of the culvert barrel.

Field inspection and evaluation of downstream controls or obstructions would indicate the potential depth of tailwater and thus indicate when outlet control will be applicable. Usually channels are wide as compared to the culvert and the depth of water in the channel is considerably less than critical depth and thus inlet control is applicable.

It has been demonstrated that the capacity of a culvert in inlet control can be increased by providing a rounded, bevelled, or tapered entrance. In other words, the capacity of a thin edge projecting metal pipe can be enhanced by the addition of an attachment or the building of a rounded, bevelled, or tapered entrance into a headwall. Similarly, the capacity of other types of pipe or box culverts can be increased at little cost by incorporating a bevel into the headwall.

The procedure for selection of the culvert size and type for most design conditions is well covered in several publications. These publications include charts or nomographs allowing comparison of the capacities of various types and sizes of culverts, therefore, the most economical choice will be apparent.

Current issues of several, widely used publications are listed in the bibliography as references 9, 14, 15, 17, 18, 21, and 24.

Obviously, the foregoing is not intended to be a complete treatment of culvert design, but is included to emphasize some of the fundamentals and to recognize that the airport designer may need to consider and include culverts in the total design.

Chapter 4. GRADING CRITERIA

12. SELECTIVE GRADING.

a. In developing an airport, proper grading is the most important single factor contributing to the success of the drainage system. Grading and drainage plans should be most carefully coordinated. Cross sections for runways and taxiways should be developed with sloping shoulders so that the surface water is directed away from the pavements and into areas for collection and disposal. The life of the pavements and the functions of drainage can generally be improved by selective grading.

b. Before grading activities are started, the engineer should have complete soil test data of the different soils encountered on the site and data on materials from any borrow sources. As determined from the soil profile and the soil characteristics, the best types of available excavated materials should be so selected and placed to form the strongest and most drainable soil structures beneath and adjacent to the pavement. The more undesirable soils should be placed in the intermediate areas as far removed from the pavements as possible.

c. Gradient design standards are found in:

(1) AC 150/5300-2A, Airport Design Standards—Site Requirements for Terminal Navigational Facilities.

(2) AC 150/5300-4A, Utility Airports,

(3) AC 150/5325-2B, Airport Design Standards—Air Carrier Airports—Surface Gradient and Line of Sight.

13. SOIL CONDITIONS.

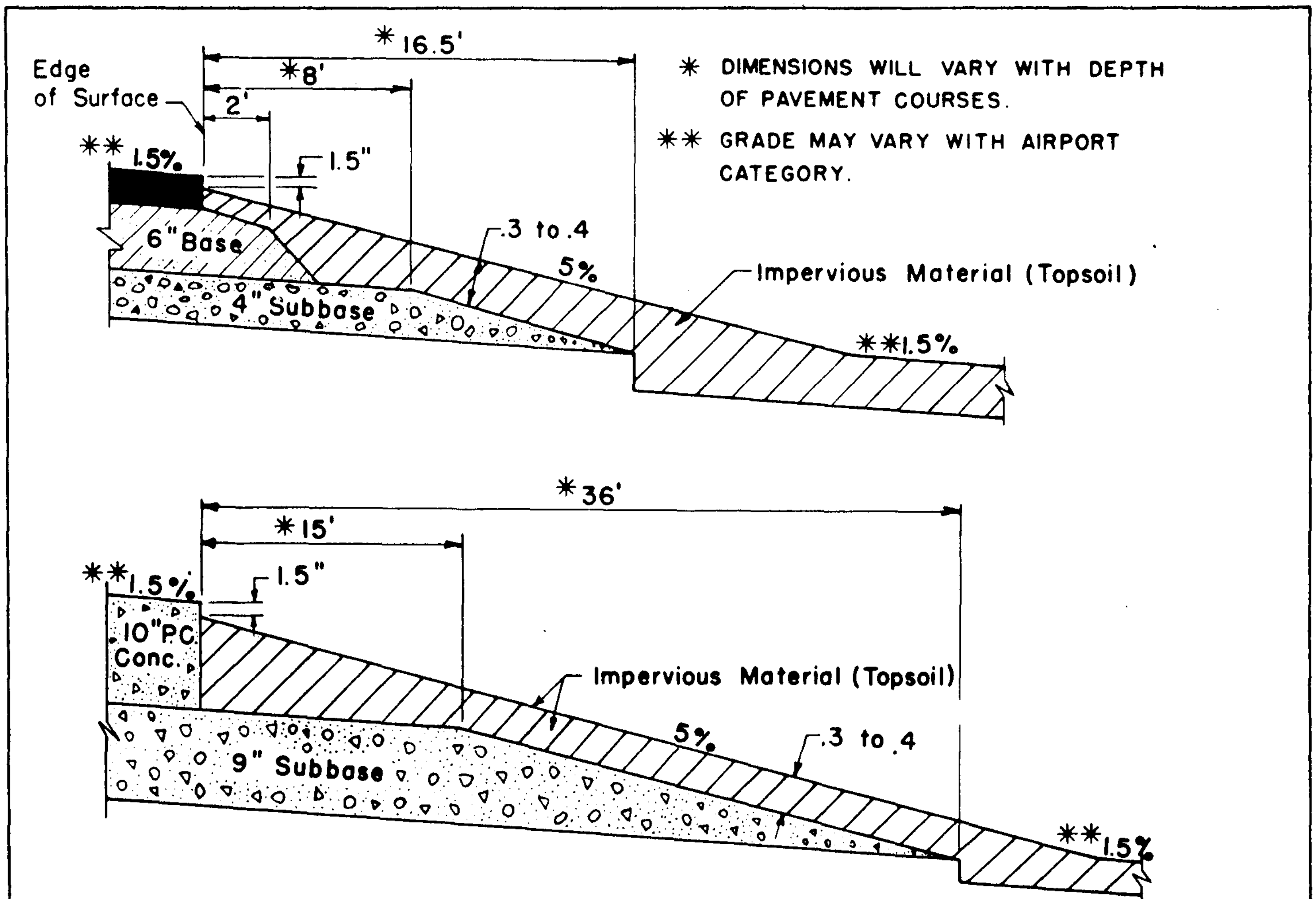
a. When the soil survey discloses different types and strata of soils on the site, different methods and procedures in the grading and drainage construction should be considered. In grading, fills are made of the material obtained from cuts and other excavation. Therefore, a basis of design is an understanding of the nature of the soils that will be encountered.

(1) On sites where the soils are of a good pervious type and are drainable, the drainage problem is greatly simplified. This type of soil is generally the contributing factor for natural drainage. The major consideration of such a site is to determine whether an impervious strata, which might pocket the water as it percolates downward, underlies the pervious surface soils. If so, provisions must be made to remove the trapped water. Usually, though, the only consideration necessary is proper grading of the area to provide for surface runoff. The slope of the graded areas must be carefully controlled because such soils may tend to erode.

(2) Sites with impervious soils are a different drainage problem. By their nature, very little precipitation will infiltrate into impervious soil. In such cases, there is little need for any subsurface drainage. Surface drainage is required, however, and will have to be designed to take care of the estimated runoff. Some impervious soils are also subject to erosion, and this characteristic should be considered.

Some clay type soils may give the appearance of being impervious, but they actually allow a very high capillary rise from the water table, which can saturate the subbase or base course. In such cases, subsurface drains at the pavement edge will help keep the subbase or base dry.

(3) At sites where pervious soils are superimposed on impervious soils, tests should be made to determine the extent and the profile of the top of the underlying layer. Some surface drainage will be needed and may be provided by proper grading with occasional inlets in the low areas, but some system of sub-



Example of Widened Subbase Sections for Extreme Snow Conditions

FIGURE 26. Example of widened subbase sections for extreme snow conditions.

drainage is definitely required to remove the water from the top of the impervious layer. If the layer is not too far below the surface, the subdrainage pipe trenches should extend slightly into the impervious layer (approximately 6 inches) and be backfilled with a granular material. The granular backfill material should be placed around and adjacent to the pipe.

(4) There are cases where an airport will be located on a site in which an impervious layer of soil is on the surface, with a pervious stratum below. Surface drainage will always be needed for draining such a site, and the system should be designed to remove all of the estimated runoff. Again, a thorough understanding of the types and extent of the soil with their respective profiles will be needed because grading operations may open up or pocket the underlying porous stratum. The underlying porous stratum often introduces underground water onto a site, requiring intercepting ditches along the edges of the airport or an intercepting drain line to cut off and divert this flow. Surface runoff sometimes may be directed into the porous layer, if it is extensive enough, by tapping through the top impervious layer with proper structures, and allowing the surface water to enter the porous layer.

(5) Drainage engineers frequently find the situation in which there are irregular strata of pervious and impervious materials. The most important thing in this situation is to locate all of the pockets existing beneath the surface and to provide sufficient drains to remove the water from them. Drainage from those pockets can be piped directly from the site or fed into the surface drainage system by proper connections. In some cases it may be necessary to remove the undesirable material from the pockets, especially under and adjacent to the pavement, and to backfill with desirable material.

b. In seasonal frost areas it is important to determine the frost penetration, since the drainage pipelines should be placed below this depth whenever possible. A serious condition could develop if the drainage lines were laid above the frost penertation line. The system could become inoperative when water freezes upon contact with the drainpipes. Field determinations of frost penetration show that the depth of penetration for various soils are fairly consistent for the same location.

(1) In granular soils, frost enters the ground quicker, penetrates deeper at an earlier stage, and leaves the ground more rapidly in the spring than in a tighter clay soil. In a clay soil the frost gradually leaves in the early spring from both the top and the bottom of the frozen stratum. It finally thaws out at a point somewhere near the midsection between the ground surface and the point of deepest penetration. During this thawing-out period, the soil becomes saturated and very unstable.

(2) Frost heaving may also occur and cause damage to the drainage system. Frost heaving is the result of freezing of capillary moisture that cannot be removed by drainage from certain soils of a silty or silty sand texture. The best way to eliminate frost heave is to remove the unfavorable soil to a sufficient depth and to replace it with a suitable material not subject to frost heave. In some localities, low temperatures and snowfalls result in deep frost penertation and deep accumulations of snow on the pavement shoulder areas and adjacent thereto. As a consequence, the thaw period is prolonged and may result in saturation of the subgrade and instability of the base or subbase.

Installation of edge subdrains is the usual solution, however, it may be found that a widened subbase will be as effective and of reasonable cost. The examples of widened subbase sections shown in Figure 26 are not intended to be standard, as the dimensions and some of the grades will vary with the pavement depth and the airport category. This method of subsurface drainage is not advocated for airports unless it has been shown to be practical in the general soil and climatological situation applicable to the site.

14. LOADS ON CONDUITS.

a. In the design and construction of drainage system conduits under pavement, the maximum anticipated wheel loads should re-

TABLE III. Minimum depth of cover in feet for pipe under flexible pavement (Part 1)

CORRUGATED ALUMINUM 2 2/3" x 1/2" or 2" x 1/2" CORRUGATIONS

AIRCRAFT WHEEL LOAD—Up to 30,000 lb. single and up to 40,000 lb. dual

Metal thickness (in.)	Pipe diameter (in.)								
	12	18	24	36	48	60	72	84	96
0.060	2.0	2.5	2.5						
0.075	1.5	2.0	2.5	2.5	3.0				
0.105		1.5	1.5	1.5	2.0	2.5	3.0		
0.135			1.0	1.0	1.5	1.5	1.5		
0.165					1.0	1.5	1.5	2.0	2.0

AIRCRAFT WHEEL LOAD—40,000 lb. dual to 110,000 lb. dual

Metal thickness (in.)	Pipe diameter (in.)								
	12	18	24	36	48	60	72	84	96
0.060	2.0	2.5	2.5						
0.075	1.5	2.0	2.5	2.5	3.0				
0.105		1.5	1.5	1.5	2.0	2.5	3.0		
0.135				1.5	1.5	2.0	2.5	3.0	
0.165					1.5	1.5	2.0	2.0	2.5

AIRCRAFT WHEEL LOAD—110,000 lb. dual to 200,000 lb. dual; 190,000 lb. dt. to 350,000 lb. dt; up to 750,000 lb. ddt & 1,500,000 lb

Metal thickness (in.)	Pipe diameter (in.)								
	12	18	24	36	48	60	72	84	96
0.060	3.0	3.0	3.0						
0.075	3.0	3.0	3.0	3.5	5.0				
0.105		2.0	2.0	2.5	3.5	4.5			
0.135				2.0	3.0	4.0	4.5	5.5	
0.165					2.5	3.5	4.0	5.0	5.5

CORRUGATED ALUMINUM 6" x 1" CORRUGATIONS

AIRCRAFT WHEEL LOAD—up to 30,000 lb. single and up to 40,000 lb. dual

Metal thickness (in.)	Pipe diameter (in.)							
	36	48	60	72	84	96	108	120
0.060	2.0	2.0	2.5	3.0				
0.075	1.0	1.5	2.0	2.5	3.5			
0.105	1.0	1.0	1.5	2.0	3.0	3.5		
0.135			1.5	2.0	2.5	3.0	4.0	
0.165					2.0	2.5	3.5	4.5

AIRCRAFT WHEEL LOAD—40,000 lb. dual to 110,000 lb. dual

Metal thickness (in.)	Pipe diameter (in.)							
	36	48	60	72	84	96	108	120
0.060	2.5	3.0	3.5	4.0				
0.075	1.5	2.0	2.5	3.0	4.0			
0.105	1.5	1.5	2.0	2.5	3.5	4.0		
0.135			2.0	2.5	3.0	3.5	4.5	
0.165					2.5	3.0	4.0	5.0

AIRCRAFT WHEEL LOAD—110,000 lb. d. to 200,000 lb. d; 190,000 lb. dt. to 350,000 lb. dt.; up to 750,000 lb. ddt. & 1,500,000 lb.

Metal thickness (in.)	Pipe diameter (in.)							
	36	48	60	72	84	96	108	120
0.060	4.0	4.5	5.0	5.0				
0.075	3.0	3.5	3.5	4.0	4.0			
0.105	2.0	2.0	3.0	2.5	4.0	4.5		
0.135			2.5	3.0	3.5	4.0	5.0	
0.165					3.0	3.5	4.5	5.5

CLAY

AIRCRAFT WHEEL LOAD—up to 30,000 lb. single and up to 40,000 lb. dual

Pipe type	Pipe diameter (in.)								
	6	10	12	15	18	21	24	30	36
Std. strength clay	2.0	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5
Extra strength clay	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0

AIRCRAFT WHEEL LOAD—40,000 lb. dual to 110,000 lb. dual

Pipe type	Pipe diameter (in.)								
	6	10	12	15	18	21	24	30	36
Std. strength clay	4.0	5.5	6.0	6.0	6.0	6.0	6.0	6.0	6.0
Extra strength clay	2.0	3.5	3.5	3.5	3.5	3.5	3.5	3.5	3.5

ASBESTOS CEMENT

AIRCRAFT WHEEL LOAD—up to 30,000 lb. single and up to 40,000 lb. dual

Asbestos cement-class	Pipe diameter (in.)								
	6	10	12	16	18	24	30	36	42
1500	2.5	2.5	2.5	2.5					
2400	2.5	2.5	2.5	2.5	2.5	2.5			
3300	1.5	1.5	1.5	1.5	1.5	1.5	1.5		
4000		1.5	1.5	1.5	1.5	1.5	1.5	1.5	
5000		1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
6000								1.0	1.0
7000								1.0	1.0

AIRCRAFT WHEEL LOAD—40,000 lb. dual to 110,000 lb. dual

Asbestos cement-class	Pipe diameter (in.)								
	6	10	12	16	18	24	30	36	42
1500	5.5	5.5	5.5	5.5					
2400	6.0	6.0	6.0	6.0	6.0	6.0			
3300	3.5	3.5	3.5	3.5	3.5	3.5			
4000		3.5	3.5	3.5	3.5	3.5	3.5	3.5	
5000		3.5	3.5	3.5	3.5	3.5	3.5	3.5	3.5
6000								2.5	2.5
7000								2.5	2.5

TABLE III.—Minimum depth of cover in feet for pipe under flexible pavement (Part 2)

CORRUGATED STEEL 2 2/3" x 1/2" CORRUGATIONS

AIRCRAFT WHEEL LOAD—Up to 30,000 lb. single and up to 40,000 lb. dual

Metal thickness (in.)	Pipe diameter (in.) 12	18	24	36	48	60	72	84	96
0.052	1.0	1.0	1.5	1.5					
0.064	1.0	1.0	1.0	1.5	1.5				
0.079	1.0	1.0	1.0	1.5	1.5	1.5			
0.109			1.0	1.0	1.0	1.0	1.5		
0.138				1.0	1.0	1.0	1.0	1.5	
0.168				1.0	1.0	1.0	1.0	1.5	1.5

AIRCRAFT WHEEL LOAD—40,000 lb. dual to 110,000 lb. dual

Metal thickness (in.)	Pipe diameter (in.) 12	18	24	36	48	60	72	84	96
0.052	1.5	2.0	2.0	2.5					
0.064	1.5	1.5	2.0	2.5	2.5				
0.079	1.5	1.5	2.0	2.5	2.5	2.5			
0.109			1.5	2.0	2.0	2.0	2.5		
0.138				2.0	2.0	2.0	2.0	2.5	
0.168				2.0	1.5	2.0	2.0	2.0	2.5

AIRCRAFT WHEEL LOAD—110,000 lb. dual to 200,000 lb. dual; 190,000 lb. dt. to 350,000 lb. dt.; up to 750,000 lb. ddt.

Metal thickness (in.)	Pipe diameter (in.) 12	18	24	36	48	60	72	84	96
0.052	2.0	2.5	3.0	3.0					
0.064	2.0	2.5	2.5	3.0	3.0				
0.079	2.0	2.0	2.5	2.5	2.5	3.0			
0.109			2.0	2.5	2.5	2.5	3.0		
0.138				2.0	2.0	2.5	3.0	3.0	
0.168				2.0	2.0	2.5	3.0	3.0	3.0

AIRCRAFT WHEEL LOAD—Up to 1,500,000 lb.

Metal thickness (in.)	Pipe diameter (in.) 12	18	24	36	48	60	72	84	96
0.052	2.5	2.5	3.0	3.0					
0.064	2.5	2.5	2.5	3.0	3.0				
0.079	2.5	2.5	2.5	2.5	2.5	3.0			
0.109			2.5	2.5	2.5	2.5	3.0		
0.138				2.5	2.5	2.5	3.0	3.0	
0.168					2.5	2.5	3.0	3.0	3.0

CORRUGATED STEEL 3" x 1" CORRUGATIONS

AIRCRAFT WHEEL LOAD—Up to 30,000 lb. single and up to 40,000 lb. dual

Metal thickness (in.)	Pipe diameter (in.) 36	48	60	72	84	96	108	120
0.052	1.5	2.0	2.0	2.0				
0.064	1.0	1.5	1.5	2.0	2.0	2.0		
0.079	1.0	1.0	1.5	1.5	2.0	2.0	2.0	
0.109	1.0	1.0	1.0	1.0	1.5	1.5	2.0	2.0
0.138	1.0	1.0	1.0	1.0	1.0	1.5	2.0	2.0
0.168	1.0	1.0	1.0	1.0	1.0	1.5	2.0	2.0

AIRCRAFT WHEEL LOAD—40,000 lb. dual to 110,000 lb. dual

Metal thickness (in.)	Pipe diameter (in.) 36	48	60	72	84	96	108	120
0.052	2.5	3.0	3.0	3.0				
0.064	2.0	2.5	2.5	3.0	3.0	3.0		
0.079	1.5	2.0	2.5	2.5	3.0	3.0	3.0	
0.109	1.5	1.5	2.0	2.0	2.0	2.5	3.0	3.0
0.138	1.5	1.5	1.5	2.0	2.0	2.0	2.5	2.5
0.168	1.5	1.5	1.5	1.5	2.0	2.0	2.0	2.5

AIRCRAFT WHEEL LOAD—110,000 lb. dual to 200,000 lb. dual; 190,000 lb. dt. to 350,000 lb. dt; up to 750,000 lb. ddt.

Metal thickness (in.)	Pipe diameter (in.) 36	48	60	72	84	96	108	120
0.052	3.0	3.5	3.5					
0.064	2.5	3.0	3.5	3.5	3.5			
0.079	2.0	2.5	3.0	3.0	3.5	3.5		
0.109	2.0	2.0	2.5	2.5	3.0	3.5	3.5	3.5
0.138	2.0	2.0	2.0	2.5	3.0	3.0	3.5	3.5
0.168	2.0	2.0	2.0	2.0	2.5	2.5	3.0	3.0

AIRCRAFT WHEEL LOAD—Up to 1,500,000 lb.

Metal thickness (in.)	Pipe diameter (in.) 36	48	60	72	84	96	108	120
0.052	3.0	3.5	3.5					
0.064	2.5	3.0	3.5	3.5	3.5			
0.079	2.5	2.5	3.0	3.0	3.5	3.5		
0.109	2.5	2.5	2.5	2.5	3.0	3.5	3.5	3.5
0.138	2.5	2.5	2.5	2.5	3.0	3.0	3.5	3.5
0.168	2.5	2.5	2.5	2.5	2.5	2.5	3.0	3.0

STRUCTURAL PLATE PIPE—9" x 2 1/2" CORR. FOR ALUMINUM; 6" x 2" CORRUGATIONS FOR STEEL

AIRCRAFT WHEEL LOAD—Up to 30,000 lb. s. or 40,000 lb. d.	AIRCRAFT WHEEL LOAD—40,000 lb. d. to 110,000 lb. d.	AIRCRAFT WHEEL LOAD—110 k.d. to 200k.d.; 190 k d.t. to 350 k. d.t.; to 750 k. d.d.t.	AIRCRAFT WHEEL LOAD—Up to 1,500,000 lb.
Pipe dia. ÷ 8 but not less than 1.0'	Pipe dia ÷ 6 but not less than 1.5'	Pipe dia. ÷ 5 but not less than 2.0'	Pipe dia ÷ 4 but not less than 2.5'

TABLE III.—Minimum depth of cover in feet for pipe under flexible pavement (Part 3)

NONREINFORCED CONCRETE

AIRCRAFT WHEEL LOAD—Up to 30,000 lb. single and up to 40,000 lb. dual

Pipe type	Pipe diameter (in.) 4	6	8	10	12	15	18	21	24
Std. strength	2.0	2.0	2.0	2.0	2.5	2.5	2.5	2.5	2.5
Extra strength	1.0	1.0	1.5	1.5	1.5	1.5	1.5	1.5	1.5

AIRCRAFT WHEEL LOAD—40,000 lb. dual to 110,000 lb. dual

Pipe type	Pipe diameter (in.) 4	6	8	10	12	15	18	21	24
Std. strength	3.5	4.0	4.0	4.5	5.5	6.0	6.0	6.0	6.0
Extra strength	1.5	2.0	2.5	3.0	3.5	3.5	3.5	3.5	3.5

REINFORCED CONCRETE

AIRCRAFT WHEEL LOAD—Up to 30,000 lb. single and up to 40,000 lb. dual

Reinf. concrete 0.01" crack D-load	Pipe diameter (in.) 12	15	18	21	24	27	30	33	36	42	48	54	60	72	84	96	108	120	132	144
800													1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
1000	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	1.5	1.5	1.5	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
1350	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
2000	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
3000	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0

AIRCRAFT WHEEL LOAD—40,000 lb. dual to 110,000 lb. dual

Reinf. concrete 0.01" crack D-load	Pipe diameter (in.) 12	15	18	21	24	27	30	33	36	42	48	54	60	72	84	96	108	120	132	144
800													6.5	5.5	4.5	3.5	2.0	1.5	1.5	1.0
1000	5.5	5.5	5.5	5.5	5.5	5.0	5.0	5.0	4.5	4.5	4.0	4.0	3.5	3.0	2.0	1.5	1.0	1.0	1.0	1.0
1350	4.0	4.0	4.0	4.0	3.5	3.5	3.5	3.5	3.0	3.0	2.5	2.0	2.0	1.5	1.0	1.0	1.0	1.0	1.0	1.0
2000	3.0	3.0	2.5	2.5	2.5	2.0	2.0	2.0	1.5	1.5	1.5	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
3000	2.0	2.0	1.5	1.5	1.5	1.5	1.5	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0

AIRCRAFT WHEEL LOAD—110,000 lb. dual to 200,000 lb. dual; 190,000 lb. dual tandem to 350,000 lb. dual tandem; up to 750,000 lb. d.d.t.

Reinf. concrete 0.01" crack D-load	Pipe diameter (in.) 12	15	18	21	24	27	30	33	36	42	48	54	60	72	84	96	108	120	132	144
800																				
1000																				
1350	7.0	7.0	7.0	7.0	7.0	6.5	6.5	6.5	6.0	6.0	6.0	6.0	6.0	6.0	6.0	5.5	5.5	5.0	4.5	4.0
2000	4.0	4.0	4.0	4.0	4.0	4.0	3.5	3.5	3.5	3.5	3.0	2.5	2.0	2.0	2.5	2.5	2.0	2.0	2.0	1.5
3000	3.0	3.0	2.5	2.5	2.0	2.0	2.0	2.0	2.0	1.5	1.5	1.0	1.0	1.0	1.5	1.5	1.0	1.0	1.0	1.0

AIRCRAFT WHEEL LOAD—Up to 1,500,000 lb.

Reinf. concrete 0.01" crack D-load	Pipe diameter (in.) 12	15	18	21	24	27	30	33	36	42	48	54	60	72	84	96	108	120	132	144
2000	7.0	7.0	7.0	7.0	7.0	6.5	6.5	6.5	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0
3000	4.0	4.0	4.0	4.0	4.0	4.0	3.5	3.5	3.5	3.5	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0

1. Cover depths are measured from top of flexible pavement, however, provide at least 1 foot between bottom of pavement structure and top of pipe.
2. The types of pipe shown are available in intermediate sizes, such as 6", 8", 15", 27", 33", etc.
3. For pipe installation in turfed areas use cover depths shown for 30,000 pound single; 40,000 pound dual.
4. Cover depths shown do not provide for freezing conditions. Usually the pipe invert should be below maximum frost penetration.
5. Blanks in tables indicate that pipe will not meet strength requirements.
6. Minimum cover depths shown for flexible pipe are based on use of excellent backfill.
7. Minimum cover depths shown for rigid pipe are based on use of class B bedding.
8. Minimum cover requirements for concrete arch or elliptical pipe may be taken from tables for reinforced concrete circular pipe, providing the outside horizontal span of the arch or elliptical pipe is matched to outside diameter of the circular pipe (assumes that classes of the pipes are the same).
9. Pipe cover requirements for "up to 1,500,000 pounds" are theoretical as gear configuration is not known.

RIGID PAVEMENT

For all types and sizes of pipe use 1.5 foot as minimum cover under rigid pavement (measure from bottom of slab, providing pipe is kept below subbase course). Rigid pipe for loads categorized as "up to 1,500,000 lb." must, however, be either class IV or class V reinforced concrete.

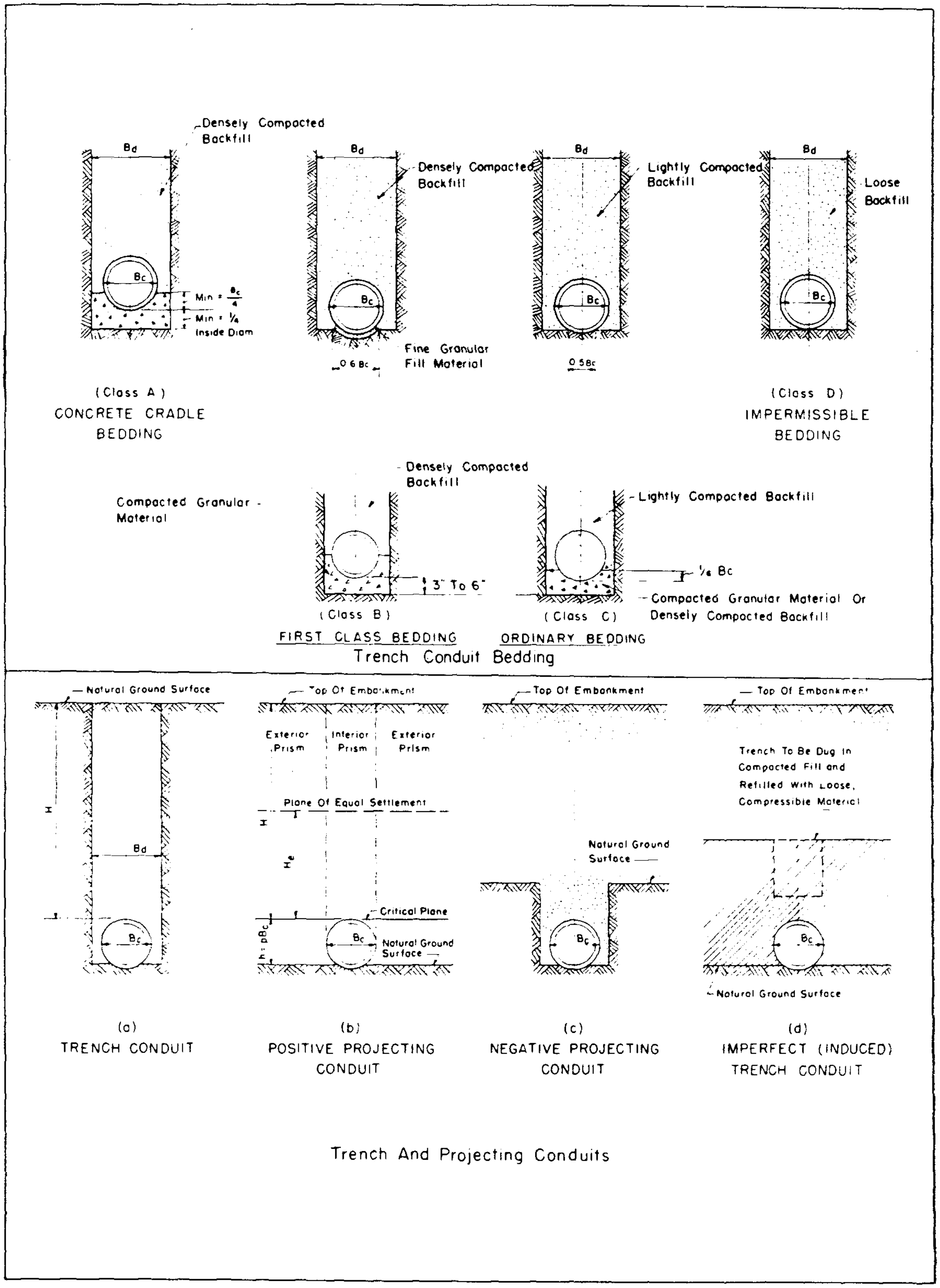


FIGURE 27. Trench and projecting conduits.

ceive consideration. The pipe grades should be established to provide the necessary depth of cover, that is, the distance between the top of the pipe and the pavement. A safe design requires consideration of the probable maximum wheel load, the inherent strength of the pipe, the details of construction conditions, the type and bearing strength of the pavement, and a factor of safety. The design of airport pavements is predicated on gross aircraft weights as applied to the type of landing gear geometry, i.e., single, dual, and dual-tandem wheels. The recommended minimum depths of cover given in Table III should be used. Note that cover under flexible pavement varies with the aircraft wheel load and type of pipe, whereas cover under rigid pavement does not involve such variables (see rigid pavement requirements in Table III (Part 3)).

b. The manufacturers of rigid and flexible pipe use different factors and procedures for calculation of loads on pipe. Table III includes recommendations of the manufacturers' organizations for minimum cover requiremets. This publication does not include recommendations for other than minimum cover situations—for other than minimum covers it is suggested that the literature listed in the bibliography be used. That literature permits determination of either the maximum allowable fill height or of pipe types suitabe for depths beyond the influence of live loads. The control and method of placing pipe under high embankments affect the magnitude of the resultant load. When the pipe is installed in a trench of specified width, the resultant load on the conduit is less severe or critical. When the installation does not involve a trench or where the trench width is not controlled—that is, very wide in proportion to width of pipe—the magnitude of the load becomes more critical. Because of the influence of the above installation conditions, underground conduits of the rigid type are classified into two major groups: trench conduits (Figure 27(a)) and embankment conduits (Figure 27(b), (c), and (d)). Embankment conduits are further subddivided into positive and negative projecting subgroups, depending on whether the conduits, as installed, are above or below the existing ground surface.

(1) Trench conduits are those which are installed in relatively narrow trenches dug in passive or undisturbed soil and then covered with earth backfill which extends to the original ground surface or for some distance above the pipe. When the conduit is placed in a trench not wider than two times its outside width and covered with earth, the backfill will tend to settle downward. This downward movement of the backfill in the trench above the conduit is retarded by frictional forces along the sides of the trench which act upward and help support the backfill and thus reduce the dead load.

(2) Computation of actual loads on pipe installed as shown in Figure 27(b)—positive projecting conduit with the pipe placed on the natural ground elevation—shows that the results are very similar to those for pipe installed in trenches (measured at top of conduit), wider than several times the maximum outside conduit width. For both these situations, the load on the conduit could be as much as three times greater than the load on a pipe installed in a narrow trench.

(3) Between these extremes are many variations. The closer the engineer can come to producing a trench type of installation, the more favorable are the loading conditions. There are two methods of construction that would tend to reduce some of the load factors normally found in projection conduits. These are the negative projecting conduits and the imperfect or induced trench, (see Figure 27(c) and (d)).

(a) The negative projecting conduits are those installed in shallow trenches of such depth that the top of the conduit is below the natural ground surface. They are then covered with an embankment which extends some distance above the ground elevation.

(b) The imperfect trench (induced trench) is the method of construction in which a cushion of compressible material is placed in a purposely constructed trench directly above the pipe in the interior of the embankment.

These two installations act to relieve the load on the conduit. Many designers have adopted them for placing conduits under high embankment with remarkable success.

c. The supporting strength of a rigid conduit depends mainly on the width and quality of the contact between the pipe and the bedding, as this affects the distribution of the vertical reaction. Subsidence of conduits is a reflection of the load bearing capacity of the in-place soil and distribution of the load—such capacity should be determined by soil tests. Four classes of bedding are used for installing rigid conduits. They are listed in the order of their relative load distribution capability (see Figure 27).

(1) *Class A:* This method consists of placing the lower part of the conduit in a cradle of concrete having a minimum thickness under the pipe of one-fourth the nominal internal diameter and extending up the sides of the pipe to a height equal to at least one-fourth the outside diameter. The minimum compressive strength of such concrete shall be 200 p.s.i.

(2) *Class B:* This method provides that the conduit be set on fine granular material in an earth foundation carefully shaped to fit the lower part of the pipe exterior for a width of at least 60 percent of the outside diameter of the pipe. Alternatively, the pipe may be bedded in compacted granular material which extends up to the midpoint of the pipe diameter. The remainder of the pipe is entirely surrounded by thoroughly compacted granular materials.

(3) *Class C:* This method requires that the earth foundation be shaped to fit the lower part of the pipe exterior with reasonable closeness for at least 50 percent of the outside diameter of the pipe. Alternatively, the pipe may be bedded in compacted granular material or densely compacted backfill. The remainder of the pipe should be surrounded by compacted granular or fine-grained material.

(4) *Class D:* This method requires little or no care either in shaping the foundation surface to fit the lower part of the pipe exterior or in filling and compacting all spaces under and around the pipe. This method is not recommended.

Experimental data indicate that the four classes of bedding, in the order listed above, have load factors of approximately 2.8, 1.9, 1.5, and 1.1. The load factor is the ratio of the supporting strength of rigid pipe in the field to the strength in the three-edge bearing test (D-load).

d. The term "D-load" is used to express the allowable load on reinforced concrete pipe in pounds per linear foot per foot of internal diameter. Thus, field loads expressed in pounds per linear foot may be converted to D-load by dividing by the nominal pipe diameter in feet. The advantage of the D-load designation is that all sizes of different types and classes of pipe of a given D-load in similar bedding and installation conditions generally will support the same earth load.

e. Conduits installed at a depth somewhat greater than the minimum cover depths of Table III are subjected only to dead loads. The dead loads may be greater or less than the actual weight of the column of material above the conduit, however, the weight of the material is a primary factor in the maximum allowable depth of backfill. The associations representing producers of different types of pipe have prepared tables of allowable depths of backfill for the various sizes, strengths, and shapes of pipe. These tables also take into account such factors as the unit weight of backfill material, width of trench, class of bedding, whether in trench or embankment and for flexible pipe—the stiffness of the pipe wall. The tables are applicable to airport drainage design.

15. LOADS ON STRUCTURES.

a. At some airports it is necessary to install box culverts in lieu of conduits, in order to provide great drainage capacity. Although not related to drainage, utility tunnels are often similar in design to box culverts. These structures, as well as inlet grates and taxiway bridges, are subject to direct aircraft loadings at some airports. Large aircraft such as the B-747, DC-10, L-1011, etc., will impose loads substantially in excess of 350,000 pounds, i.e., up to 860,000 pounds. It is said that increasingly heavy aircraft will be developed so that a 1.5-million pound aircraft appears feasible and reasonably certain to materialize.

Obviously, point loading on some structures will be greatly increased over that used for design assumptions in the recent past. Accordingly, it is recommended that structures, which may need to accept loads such as mentioned here, be designed to withstand the following loadings:

(1) For spans of 2 feet or less in the least direction, a uniform live load of 250 p.s.i.

(2) For spans between 2 feet and 10 feet in the least direction, a uniform live load varying between 50 p.s.i. and 250 p.s.i., in direct proportion to the span length.

(3) For spans of 10 feet or greater in the least direction, the design should be based on the most critical loading condition which may be applied by gear configurations as illustrated in Appendix 2 to AC 150/5320-6A, Airport Paving.

b. The following elements of Appendix 2 to AC 150/5320-6A are also applicable to drainage:

(1) As always, footing design will vary with depth and soil type. For shallow structures or those subject to direct heavy aircraft loads, such as inlets and box culverts, concentrated load design may require heavier and more widely spread footings than heretofore provided.

(2) Locked wheel braking loads must be anticipated for structures subject to direct wheel loads.

(3) Because soil is relatively insensitive to increased loadings, a soil cover is recommended over structures where clearances permit.

16. EROSION CONTROL.

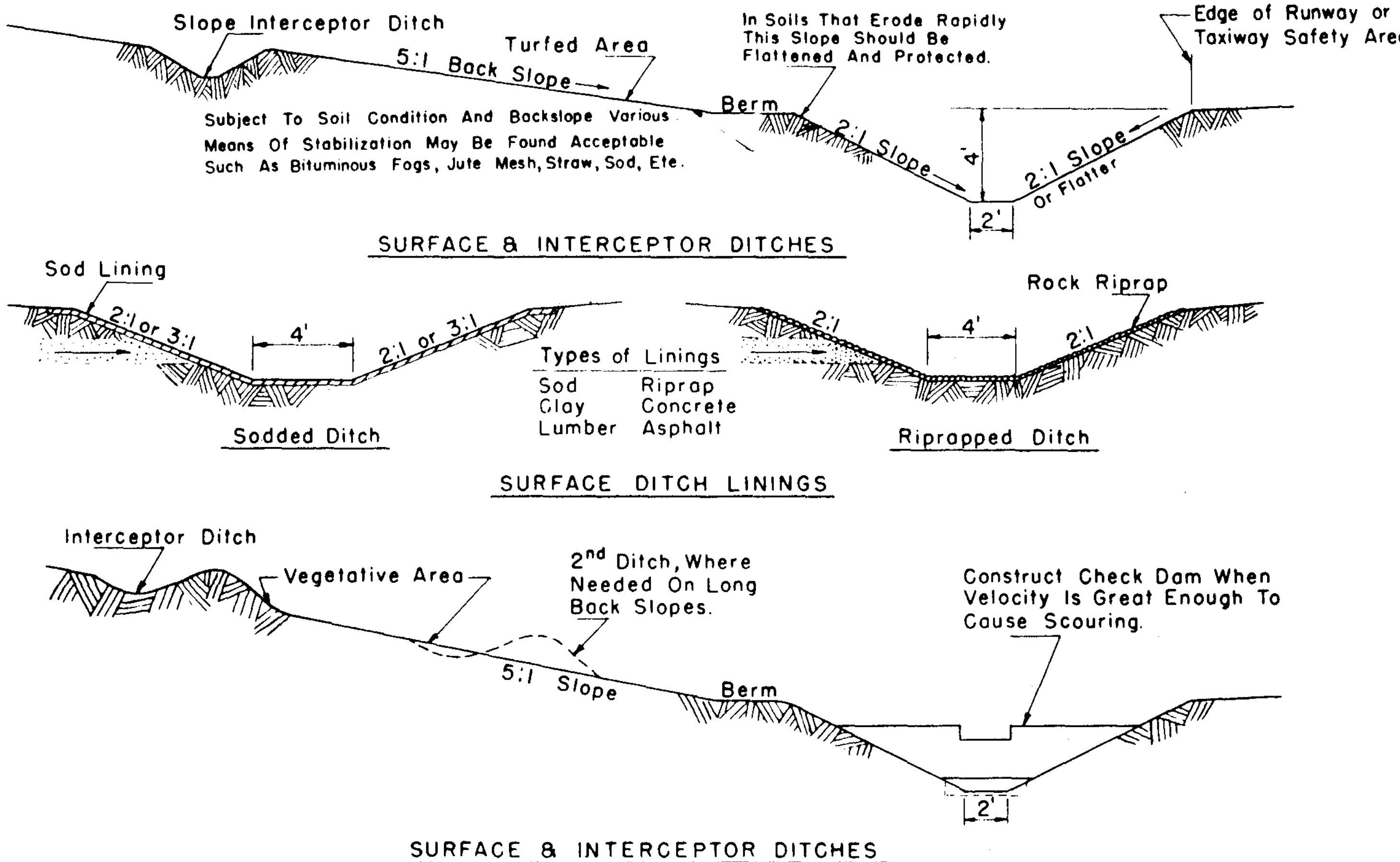
a. An important item in airport drainage is to provide for adequate protection of cut-and-fill slopes. Unless the slopes are correctly designed for the type of material contained in them, erosion will start during the first storm. The usual engineering practice is to establish a certain percentage of slope for the type of material encountered as shown on the soil profile and to maintain that slope throughout the particular section.

b. When cut-and-fill slopes are constructed to obtain the most economical section, some provision for their protection should be made. In airport construction, these slopes are usually made as flat as possible and vary from a 2:1 slope to one as flat as 10:1. Cut slopes more than 8 to 10 feet deep, with higher ground above them, should be provided with a cutoff surface ditch constructed several feet back from the top of the bank and running parallel to the top-of-cut line to intercept the surface water flowing down from the higher ground. To protect the cut slopes, it may be necessary to riprap, sod, sprig, or seed with rapid-growing grass or vegetation. It is good practice also to construct a ditch at the base of the bank to intercept the flow of runoff. Figure 28 illustrates several recommended types of interceptor ditches.

c. All fill-slopes that are more than 5 feet high should be protected against surface water erosion by building berms and gutters along the top of the slope to intercept the surface water and to prevent it from spilling down the slope. The surface water, thus intercepted, may be disposed of by properly constructed concrete spillways, vertical drop inlets, or other suitable means of conducting the water down the slope to proper outfall ditches. Several recommended types of embankment protection structures are shown in Figure 29. When a berm is placed along the top edge of the embankment, some method of protection is necesary, for example, by shooting the berm with a light asphaltic material, sodding the berm, or providing paved gutters. The method which most nearly satisfies local conditions should be used.

d. One oversight in the construction of the spillways has been the failure to provide an adequate cutoff wall beneath the apron at the entrance to the spillway. This cutoff is most important to prevent water from seeping under and along the spillway, and causing faiure from lack of support. It is desirable to construct either a series of baffies or a stilling basin at the base of the spillway to reduce the velocity of the flowing water. The elevation at the outlet should be the same as that of the ditch into which it empties. Where opentrough type spillways are constructed, their cross-sectional area should be larger than that required for the design storm, and provision should be made in the design for ample free-board.

e. The channel below culvert outlets or spillways should be protected against erosion. Usually headwall or stilling basin structures are provided as illustrated in Figure 29—together with stone riprap in the channel. If a headwall, stilling basin, or riprap is not provided—then erosion may be expected in most soil conditions and with other than minimum velocities. The erosion will occur as gully scour or as a scour hole. Gully scour may erode the bottom and sides of the channel so severely as to undermine the culvert, destroy or clog the channel, and cause loss of embankment. A scour hole can undermine the storm drain causing loss of some of the pipe sections. The required dimensions of the apron of the structure and/or riprap are predictable. For example, hydraulic laboratory investigation and field observations show that gully scour may be expected in a cohesionless soil if the Froude number of flow in the channel below a culvert outlet (without a structure or riprap) is 0.35 or greater. The Froude number (Fo) equals average velocity of flow at the culvert, f.p.s. ($\overline{V}o$) divided by the square root or acceleration due to gravity, ft./sec.^{2} (g) multiplied by the culvert diameter, ft. (Do) or $Fo = \overline{V}o\sqrt{g\ Do}$. Then the length of area to be protected, if maximum tailwater applies, equals Do (8 + 55 Log Fo). The beginning width of such area equals 3 Do and the area increases in width with a flare of 1 on 5. These and other relationships are explained in reference 33 of the bibliography and texts on drainage.



Types Of Surface And Interceptor Ditches

FIGURE 28. Types of surface and interceptor ditches.

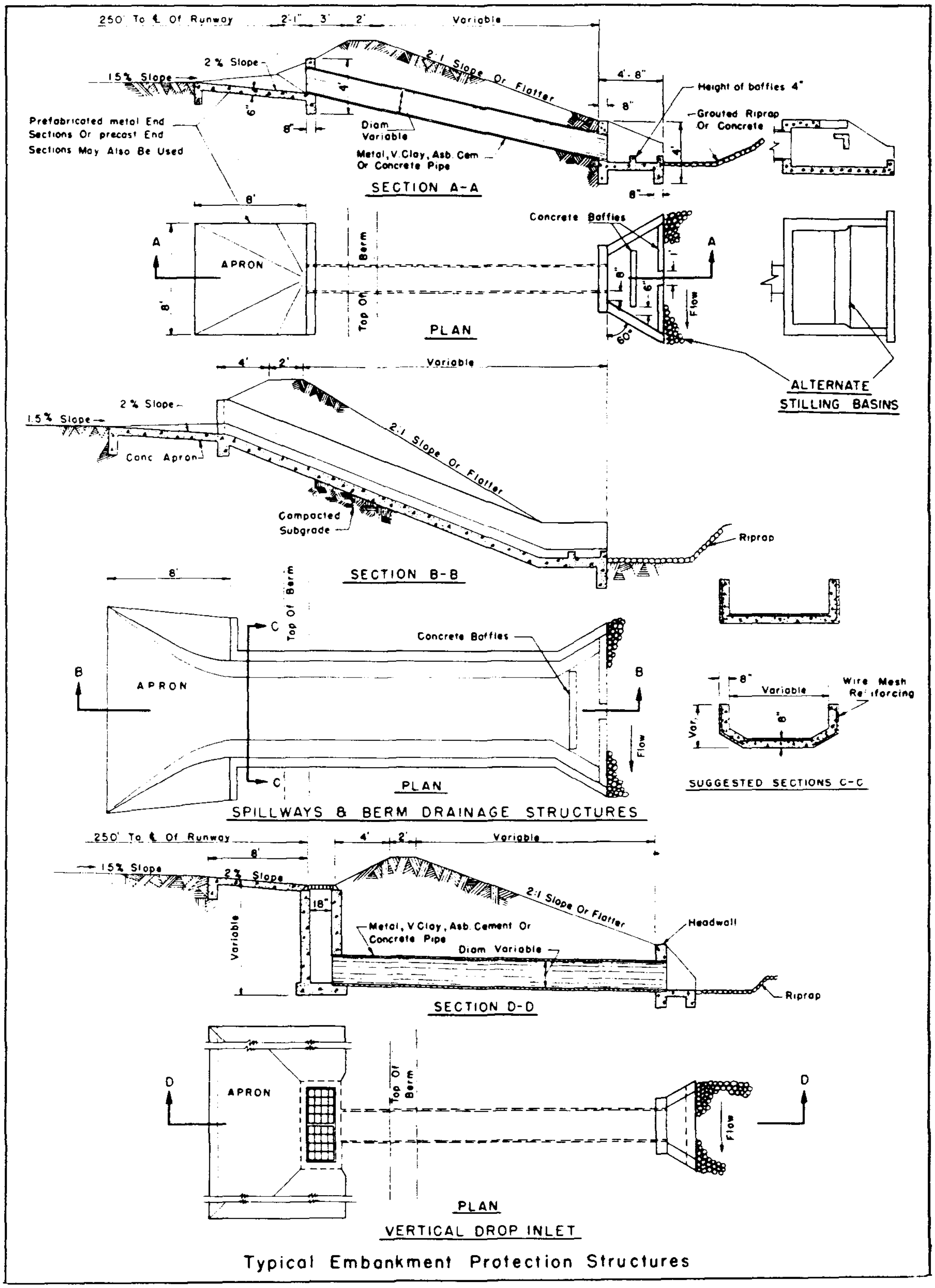


FIGURE 29. Typical embankment protection structures.

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Chapter 5. THE DRAINAGE SYSTEM

17. BASIC INFORMATION REQUIRED.

a. In this chapter, each of the steps considered pertinent to the actual design of an airport drainage system will be considered in their respective order. A typical layout plan and the drainage criteria described before will be used. Before any design can be undertaken, certain basic information and data must be available to develop and detail the drainage system. These data should consist primarily of the following:

(1) The contour map of the airport and adjacent areas.

(2) The "drainage working drawing" showing the layout of the runways, taxiways, aprons, and building areas.

(3) All rainfall data, such as frequency, intensity, and duration of storms. Rainfall intensity-duration curves should be plotted for storms of a 5-year frequency (considered adequate for airports) and the resultant graph used for runoff quantities in conjunction with the design. A frequency curve for 10 years should also be plotted for checking excess storms and verifying ponding capacity. A 2-year curve will illustrate intensity-duration for the shorter return period. These curves should be prepared using the appropriate ESSA-Weather Bureau Technical papers or charts (see Chapter 2, paragraphs 3c or d).

(4) Plotted centerline profiles of all the runways, taxiways, and apron areas, with necessary cross sections.

(5) Boring plans and soil profiles prepared on the basis of soil tests, including data on ground water elevation.

(6) Temperature data, especially records on maximum and minimum temperatures during seasons of freezing and thawing and on depth of frost penetration. Also, snowfall records indicating maximum and average depths of fall per month.

(7) Data, when obtainable, on the infiltration properties of soils encountered and any actual runoff records for drainage areas in the locality having similar characteristics and soils.

(8) Information on existing and future aircraft use for selection of appropriate strength for grates, covers, and frames for inlets and manholes.

b. In the actual design, the initial step is a comprehensive study of the topographic map that is extensive enough to include the areas surrounding the airport site, to permit identifying possible contributing surface or subsurface flow, to determine general direction of flow, and to locate natural watercourses or outfalls. The existence of any major local construction or improvement that could affect drainage disposal should be evident from the map. An example is Figure 30.

c. The outline of the boundary of the airport plus the location of the special airport features such as runways, taxiways, aprons, buildings, and roads have been superimposed on the map. Possible outfalls that can be utilized for runoff are shown in the southern section and to the west of the NW/SE runway. The airport is rather flat, without any nearby outstanding high areas; for this reason there should not be any outside flow towards the airport site. There is no development in the immediate neighborhood to cause any drainage problem. As noted from the contours, the outlet pipes can be daylighted within reasonable distances and ditches can be used for outfalls.

d. As the map shows, this particular site is higher than the surrounding terrain, a situation which simplifies the drainage objective because there is no possibility of flooding. In some other airport locations where the site elevation is relatively low, there may be problems with the outfall disposal. Thus, a careful study of the topographic map will disclose the characteristics of the area terrain and the general pattern of drainage design involved.

18. DRAINAGE LAYOUT.

a. With the general configuration of the terrain well in mind, actual layout of the drainage system can now be undertaken. This can best be done on the drainage working drawing (Figure 30), upon which have been placed the runway layout and the tentative finished grading by contours drawn to a 1-foot interval. The finished contours reveal that a crown section has been used which is the standard cross section for the runways, taxiways, and safety areas. This crowned section slopes each way from the centerline of the runway on a transverse grade to the edge of the pavement, except where it becomes necesary to warp the grade to provide a smooth transition at the intersection of pavements. As noted on the typical cross section of Figure 31, the intermediate areas of the runway safety area, each side of the runway pavement will be on a transverse grade away from the pavement. This grade may be varied slightly to properly design for drainage to inlets.

b. 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.

c. Before proceeding further, recheck the finished contours to ascertain 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 there are no excessively long distances for surface water to flow into the inlets. If there is a long gradual 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. Under such conditions, the ridge shown in Figure 8 will protect the area around the inlet, prevent by-passing, 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 advisory circulars. It is also essential for all ponding area edges to be kept at least 75 feet from the edges of the pavement. This prevents saturation of the base or subbase and of the ground adjacent to the pavement during periods of ponding.

d. After the field 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. Several types of interceptor ditches are shown in Figure 28. 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. All inlets, structures, and pipelines should be identified by numbers or letters for ready reference and for use in the computation sheets. It is customary to start numbering at the outlet end of the pipeline and to progress upgrade. The areas contributing to each inlet should be outlined and the acreage determined, differentation being made between the types of surfacing such as pavement, turf, earth, and so on. Profiles of the existing ground and final grades along the proposed drainlines should be observed and perhaps plotted; these data will be needed in determining the grades of the pipeline (see Figure 32).

f. Unless the pipe size changes, the flow line through the inlets should be uniform. Occasionally, drop inlets are installed to alleviate steep gradients on the pipeline.

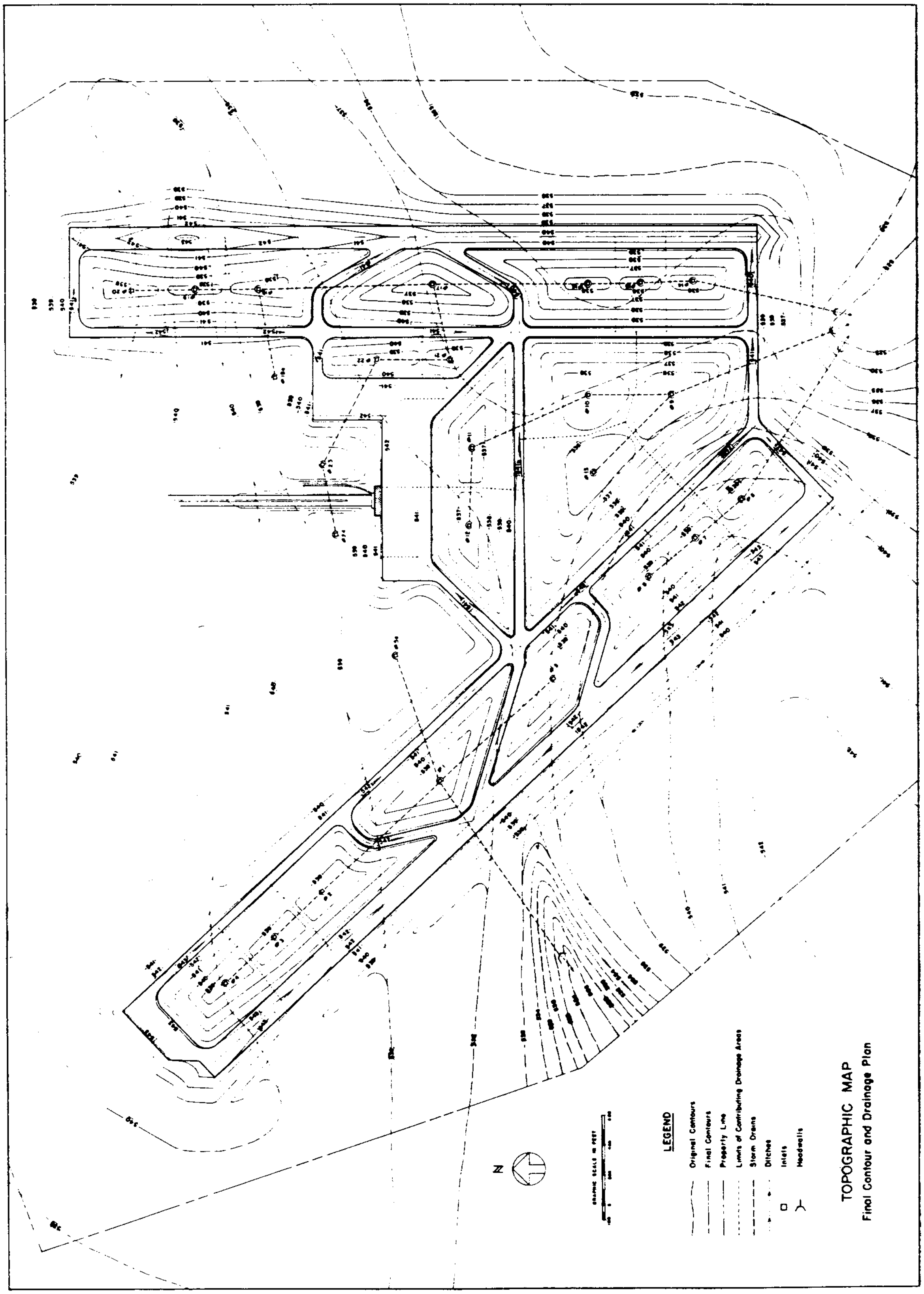


FIGURE 30. Topographic map.

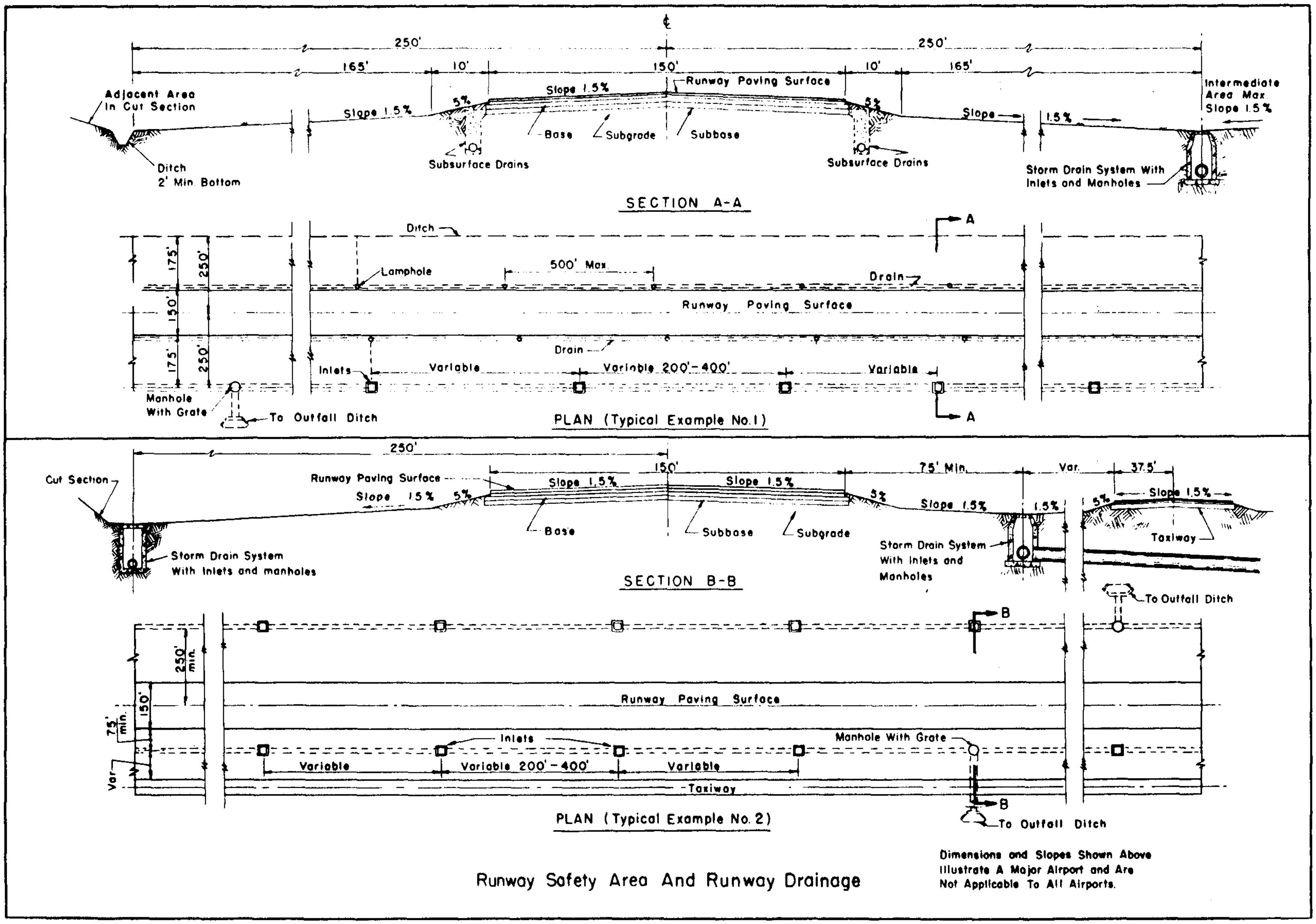


FIGURE 31. Runway safety area and runway drainage.

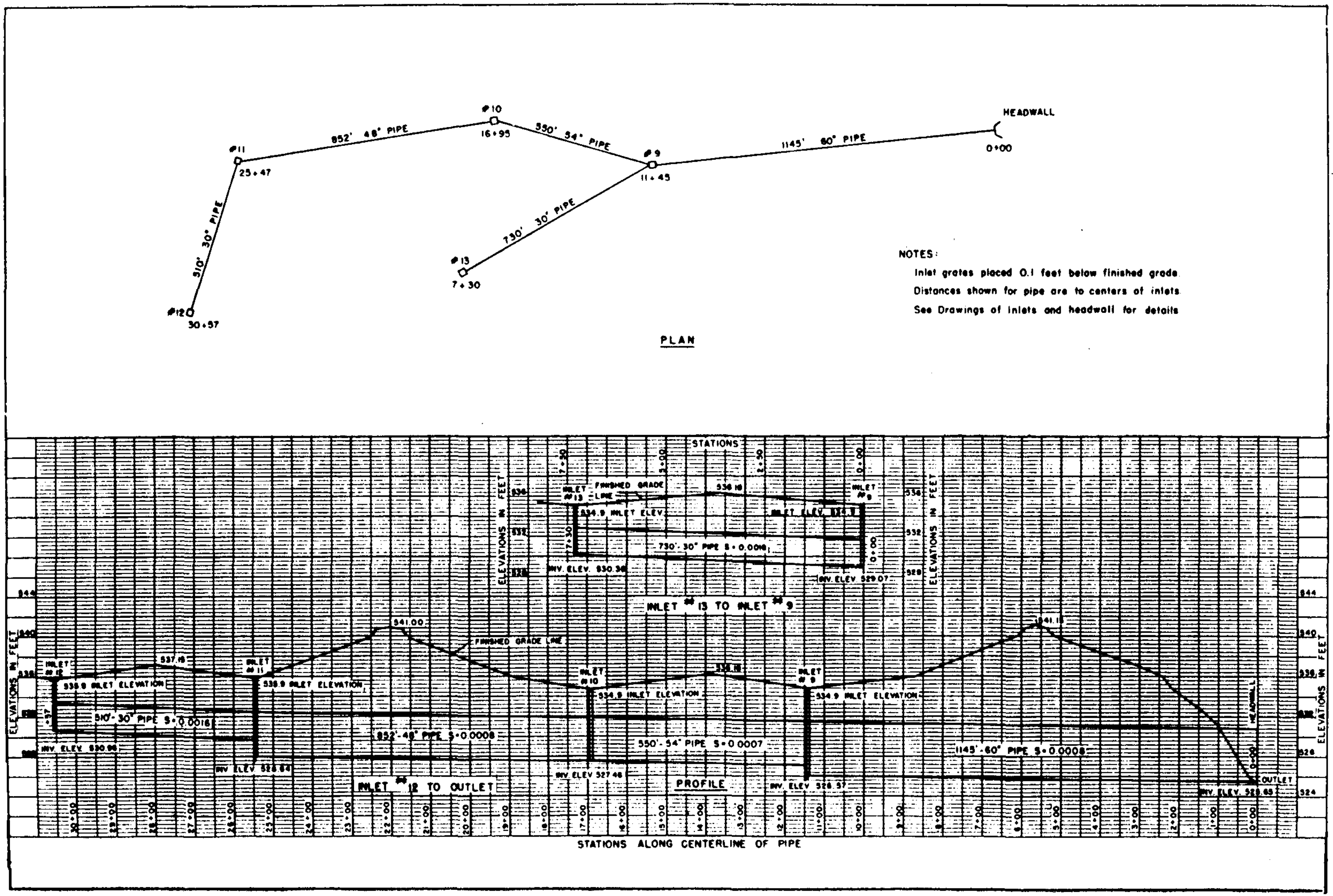


FIGURE 32. Plan and profile of drainage pipe.

g. Ditches form an integral part of the drainage system. The size of the ditches and their functions are quite variable. Some ditches serve to carry the outfall away from the pipe system and drainable areas into the natural drainage channels or into existing watercourses. Sometimes it becomes necessary to construct extensive peripheral ditches. Their purpose is to receive outfall flow from the drainage system, to collect surface flow from the airport site or adjacent areas, and to intercept possible ground water flow from higher adjacent terrain. Open ditches are liable to erode 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.

h. With the plans and data referred to in the preceding text, it is possible to design the drainage system. A step-by-step drainage procedure is as follows:

(1) Identify the structures and estabish the lengths of pipe segments between structures. Scaled dimensions are of sufficient accuracy at this stage.

(2) Select values for coefficients of runoff "C" for the several types of surfaces over which water will flow. Table I may be used as a guide in arriving at acceptable values for this factor.

(3) Compute a weighted value of "C", if required, as explained under "Runoff Coefficient," paragraph 5.

(4) Determine the distance from the inlet to the most "time-remote" point in the tributary subarea. If in flowing from such point, water traverses different types of surfaces, the lengths of flow over each type of surface should be determined.

(5) Using the distances determined according to step (4), the time of flow to the inlet from the most "time-remote" point can be established. The time so determined is the "inlet time." It may be obtained by the use of the curves in Figure 7. If the distance exceeds the limits of the curves, use the formula in Figure 7. Keep in mind that the total length of overland flow may consist of several sublengths, each of different surface or slope.

(6) Determine the time of concentration for the inlet in accordance with the principles outlined under "Time of Concentration," paragraph 6.

(7) From the plotted rainfall curve for the design storm, find the rainfall intensity "I" for the corresponding time of concentration.

(8) Record the acreage of the subarea which is contributing to the inlet.

(9) Compute the quantity of runoff by the formula $Q = CIA$. This is the amount of water which must be accommodated by the drain pipe from this inlet.

(10) Select slope and determine the pipe size which will carry the runoff. Charts as shown in Figures 9 through 12 may be used.

(i) As the design progresses along the line, the runoff naturally accumulates. Each succeeding pipe run carries the water from the upper reaches of the system in addition to the water introduced through its immediate inlet structure. This accumulation, however, is not necessarily a straight arithmetic summation of flows from preceding inlets. Flow from influent lines may have to be adjusted to represent the amount of water which they are contributing at the time of concentration for the point being investigated.

19. SURFACE DRAINAGE.

a. A portion of the actual design of the system can now be considered in accordance with criteria and data given previously. For example, the area between the apron and taxiways of the airport layout has been selected for detail analysis in making the necessary calculations and determinations (see Figure 33).

b. The rainfall data for the location under study has been obtained from graphs found in the U.S. Weather Bureau Technical Paper No. 40. These data have been plotted and curves drawn (Figure 6) to indicate the intensity of rainfall. The curve of the intensity-duration for a 5-year frequency will be used in the computations. From this curve, the intensity for the corresponding time of concentration for each inlet can be readily determined and used in the system design.

c. After the drainage layout has been decided and sketched on drainage working drawings, the extent of the subarea contributing to each intake structure is measured and tabulated. The recording of the sizes of the subareas is shown in Table IV. Inspection of the areas will show that surface water will flow partly over pavements and partly over turfed areas. A runoff factor of 0.90 has been assumed for the paved areas and 0.30 for the turfed areas. A weighted value of the factor "C" or runoff coefficient was calculated as explained in Chapter 2 and is shown in Table IV. In working up the data shown in Table V, a bell-and-spigot type of pipe was used and a value of n = 0.015 was assumed.

d. For convenience in the computations and recording the results, a form such as that of Table V is suitable. Explanation of the various columns of this form is as follows:

(1) Column I identifies the inlet being investigated. All structures should be numbered, preferably starting with the first structure from the outfall and progressing along the line to the uppermost end.

(2) Column 2 identifies the particular segment of the drainage system being designed.

(3) Column 3 shows the length of that segment of the line.

(4) Column 4 gives the "inlet time" or time required for water to flow overland from the most time-remote point of the tributary subarea to the inlet being considered.

(5) Column 5 gives the "flow time" through the particular pipe segment. This is obtained by dividing the pipe length by the velocity of the drain. See column 12 for velocity.

(6) Column 6 shows the time of concentration. For inlets 12 and 13 in the example, time of concentration equals the "inlet time." Maximum flow does not occur at inlet 11 until all areas tributary to it are contributing to that inlet. All areas are contributing to inlet 11 in 55.4 minutes (see note in Table V).

(7) Column 7 shows the coefficient of runoff for the subarea contributing to the inlet. A method of determining the runoff factor is illustrated in Table IV.

(8) Column 8 gives the rainfall intensity based on the time of concentration and the design storm frequency (from Figure 6).

(9) Column 9 gives the acreage of the subarea immediately tributary to the inlet being investigated. (See Table IV.)

(10) Column 10 shows the amount of runoff from each tributary area as determined by the Rational Method formula Q = CIA.

(11) Column 11 gives the accumulated runoff which must be accommodated. In the example problem the maximum accumulated runoff to be discharged from inlet 11 consists of the runoff from the subarea tributary to inlet 12, plus the amount of runoff from the subarea tributary to inlet 11. The total accumulated runoff at inlet 11 is shown in the table.

(12) Column 12 gives the velocity of flow through the pipe, determined by dividing the pipe capacity by the area of the pipe. To be self-cleaning, drains should be designed to have a flow velocity of not less than 2.5 f.p.s.

(13) Column 13 gives the size of the pipe required to accommodate the flow.

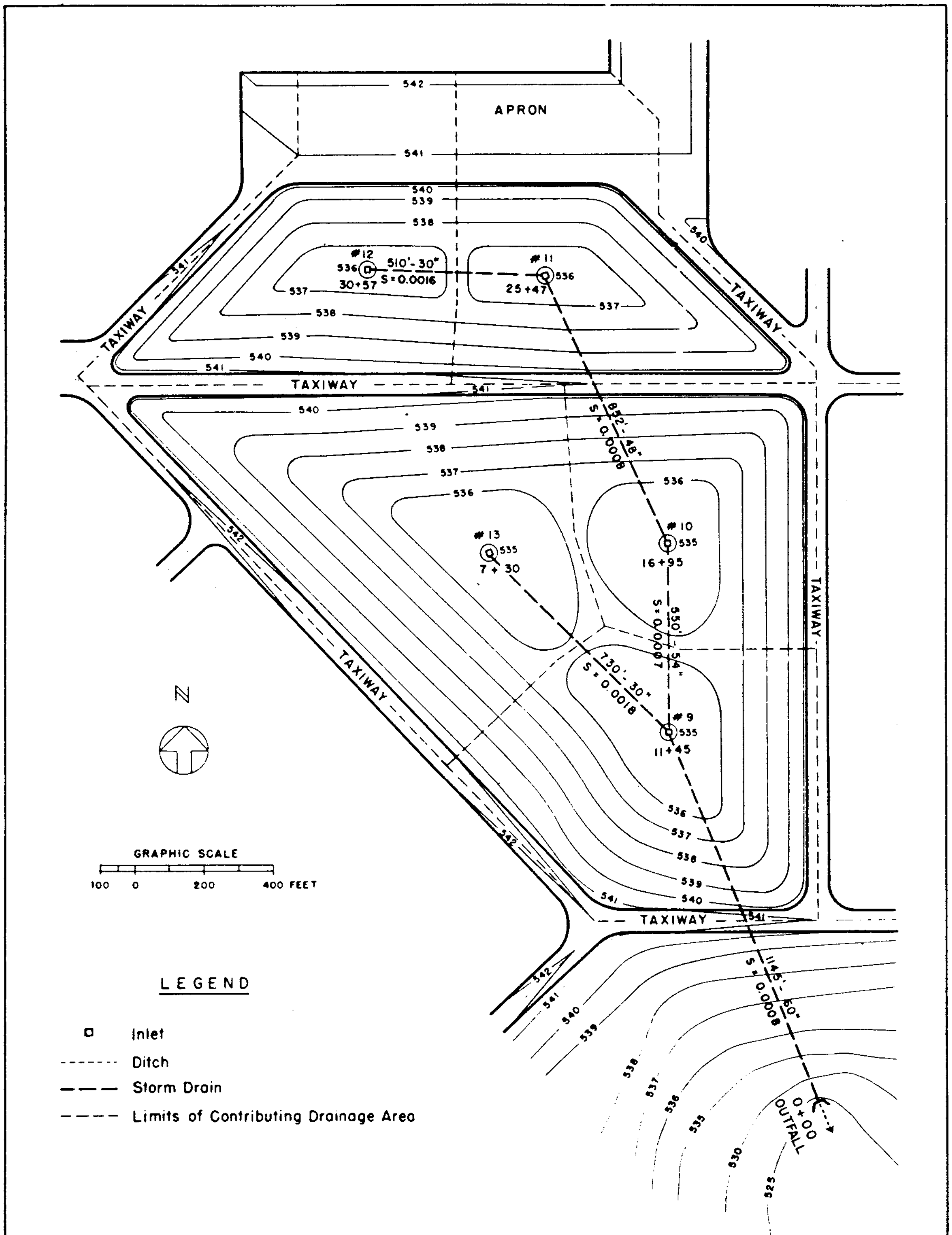
(14) Column 14 shows the slope of the pipe. Selection of the slope usually will be governed by such factors as topography, amount of cover, depth of excavation, desired discharge velocity and capacity, and elevation of discharge basin or channel.

(15) Column 15 shows the capacity of the pipe in cubic feet per second on the slope indicated. Obviously, the capacity must exceed the accumulated runoff if the system is to operate properly (use Figures 9 through 12).

(16) Column 16 gives the invert elevation of the structure identified in Column 1.

(17) Column 17 is available for any remarks pertinent to the design.

e. It is obvious that many combinations of pipe sizes and slopes can be selected which will provide the required pipe capacity. It is good practice to select the smallest size pipe, consistent with such considerations as economy of excavation and flow velocity, that will accommodate the desired discharge. Usually 12-inch pipe is the minimum size used to carry surface runoff. It is the general practice to increase pipe size as the volume of water to be



Portion of airport showing drainage design

FIGURE 33. Portion of airport showing drainage design.

TABLE IV. Design data for drainage example in Table V

DESIGN DATA FOR DRAINAGE EXAMPLE IN TABLE V

Inlet No.	Tributary Area To Inlets (in acres)				Distance Remote Point To Inlet (in ft)			Time For Overland Flow (in mins.)		
	Pavement	Turf	Both	Sub Total*	Pavement	Turf	Total	Pavement	Turf	Total
12	4.78	9.91	14.69	14.69	100	790	890	4	48	52
11	5.48	9.24	14.72	29.41	90	750	840	4	49	53
10	1.02	10.95	11.97	41.38	65	565	630	3	36	39
13	1.99	19.51	21.50	21.50	110	1140	1250	4	58	62
9	1.46	14.59	16.05	78.93	85	612	697	4	38	42
Totals	14.73	64.20	78.93							

Weighted Average For "C" For Tribulary Area To:

To Inlet 12

$$\frac{4.78}{14.69} \times 0.90 = 0.29$$

$$\frac{9.91}{14.69} \times 0.30 = \underline{0.20}$$

$$C = 0.49$$

To Inlet 10

$$\frac{1.02}{11.97} \times 0.90 = 0.08$$

$$\frac{10.95}{11.97} \times 0.30 = \underline{0.27}$$

$$C = 0.35$$

To Inlet 11

$$\frac{5.48}{14.72} \times 0.90 = 0.34$$

$$\frac{9.24}{14.72} \times 0.30 = \underline{0.19}$$

$$C = 0.53$$

To Inlet 13

$$\frac{1.99}{21.50} \times 0.90 = 0.08$$

$$\frac{19.51}{21.50} \times 0.30 = \underline{0.27}$$

$$C = 0.35$$

To Inlet 9

$$\frac{1.46}{16.05} \times 0.90 = 0.08$$

$$\frac{14.59}{16.05} \times 0.30 = \underline{0.27}$$

$$C = 0.35$$

Subtotals are shown to illustrate that the area contributing to downstream inlets is a summation of tributory areas above them plus the area contributing to the inlet itself. This prevails where time of concentration for the inlet in question is also a summation - see paragraph 6b (1).

In runoff calculations the accumulation is actually made by addition of the runoff from the line segments (as shown by Table V).

TABLE V. Drainage system design data

DRAINAGE SYSTEM DESIGN DATA

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
Inlet	Line Segment	Length of Segment	Inlet Time	Flow Time	Time of Concentration	Runoff Coefficient "C"	Rainfall Intensity "I"	Tributary Area "A"	Runoff "Q"	Accumulated Runoff	Velocity of Drain	Size of Pipe	Slope of Pipe	Capacity of Pipe	Invert Elevation	Remarks
		FT	MIN	MIN	MIN		IN/HR	ACRES	CFS	CFS	FT/SEC	IN	FT/FT	CFS		
12	12-11	510	52	3.4	52	0.49	1.98	14.69	14.25	14.25	2.8	30	.0016	14.25	530.96	(n = 0.015)
11	11-10	852	53	5.0	55.4	0.53	1.90	14.72	14.82	29.07	2.8	48	.0008	35.0	528.64	See note below
10	10-9	550	39	3.3	60.4	0.35	1.78	11.97	7.46	36.53	2.8	54	.0007	45.0	527.46	See note below
13	13-9	730	62	3.9	62	0.35	1.76	21.50	13.24	13.24	3.1	30	.0018	15.0	530.38	
9	9-OUT	1145	42	4.2	65.9	0.35	1.70	16.05	9.55	59.32	3.3	60	.0008	65.0	526.57	
OUT															525.65	

NOTE: Time of concentration for Inlet #11 is 55.4 minutes (52 + 3.4 = 55.4) which is the most time remote point for this inlet. Likewise time of concentration for Inlet #10 is 60.4 minutes (52 + 3.4 + 5.0 = 60.4) accommodated increases. The velocity in the entire system should be maintained or increased progressively along a line to prevent settlement of suspended solids. Care should be taken to avoid flow retardance or the creation of turbulence in the system as this also will cause settlement of suspended solids.

f. A form similar to that described may be used in the design of any of the several sections of the system. The desriability of using ponding areas should be studied and the system should be checked for its capability to take care of storms heavier than the design storm.

20. PONDING.

a. The rate of outflow from a drainage area is limited by the capacity of the drainage facility serving the area, usually a drainpipe. Whenever the rate of runoff at a structure such as an inlet exceeds the drain capacity, a temporary storage or ponding occurs. As soon as the rate of inflow into a ponding basin becomes less than the drain capacity, the accumulated storage will be drawn off at a rate equal to the difference between the capacity and the rate of inflow. The rate of outflow from a ponding basin is affected somewhat by the elevation of the water at the drain inlet, and it will increase as the head on the inlet increases. Because of the flat slopes on an airport, the surface areas of the storage basins surrounding the inlets are usually very large in comparison with water depths at the inlets. Although the hydraulic gradient at the inlet is raised slightly because of ponding, any increase in drain capacity should be considered a small factor of safety and not taken into account.

b. Figures 34 and 35 and Table VI have been prepared to illustrate the proposition of ponding. For example, the area to be drained is part of that shown in Figure 33 except that for simplification, the contours have been changed to create one large ponding area with only one drain to handle all the runoff. The size of the drain can be varied to compute the different time periods needed to discharge the volume of ponding accumulated.

c. A study of the cumulative rainfall for 5-year and 10-year frequency will be used as the rate of supply. The rainfall usually diminishes gradually in intensity after a couple of hours. Shown in the table is the tabulation of the hourly intensity in inches for various intervals for both the 5-year and 10-year frequency. Also shown are all the necessary data for the cumulative runoff for the two frequencies, and the discharge for a 30-inch diameter pipe. These data have been plotted in Figure 35. Also plotted are the discharge capacities for 21-inch, 24-inch, and 33-inch pipes.

d. Computations indicate that if the inlet is constructed to an elevation slightly below contour 534, there will be a ponding storage capacity between it and contour 536 of 243,300 cubic feet. From Figure 35, it can be seen that the 33-inch pipe will empty the area in 39 minutes after the start for the cumulative runoff from the 5-year frequency storm and will empty the area in 51 minutes after the start for the cumulative runoff from the 10-year frequency storm. The 21-inch pipe would provide sufficient discharge to keep the maximum ponding down to 82,985 cubic feet after 60 minutes after the start of the runoff for the 10-year frequency storm; however, this pipe would not empty the ponding area for an additional 3 hours or more.

e. One objective should be to limit the ponding to less than 243,300 cubic feet, and it will be noted that even the 21-inch pipe would accomplish that. Another objective should be to dispose of the ponded volume in a reasonable time so as not to have ponds in the runway safety area for long periods. Also, a prolonged storage of water may make the area unstable for some time after the storm and may kill the grass. For example, note that the 21-inch pipe would not empty the pond from a 5-year storm for more than 3 hours. The 30-inch pipe would, however, empty the pond in less than an hour. Under these circumstances, the 30-inch pipe would be a more acceptable selection, however, the 21-inch pipe might result in the hazards mentioned above.

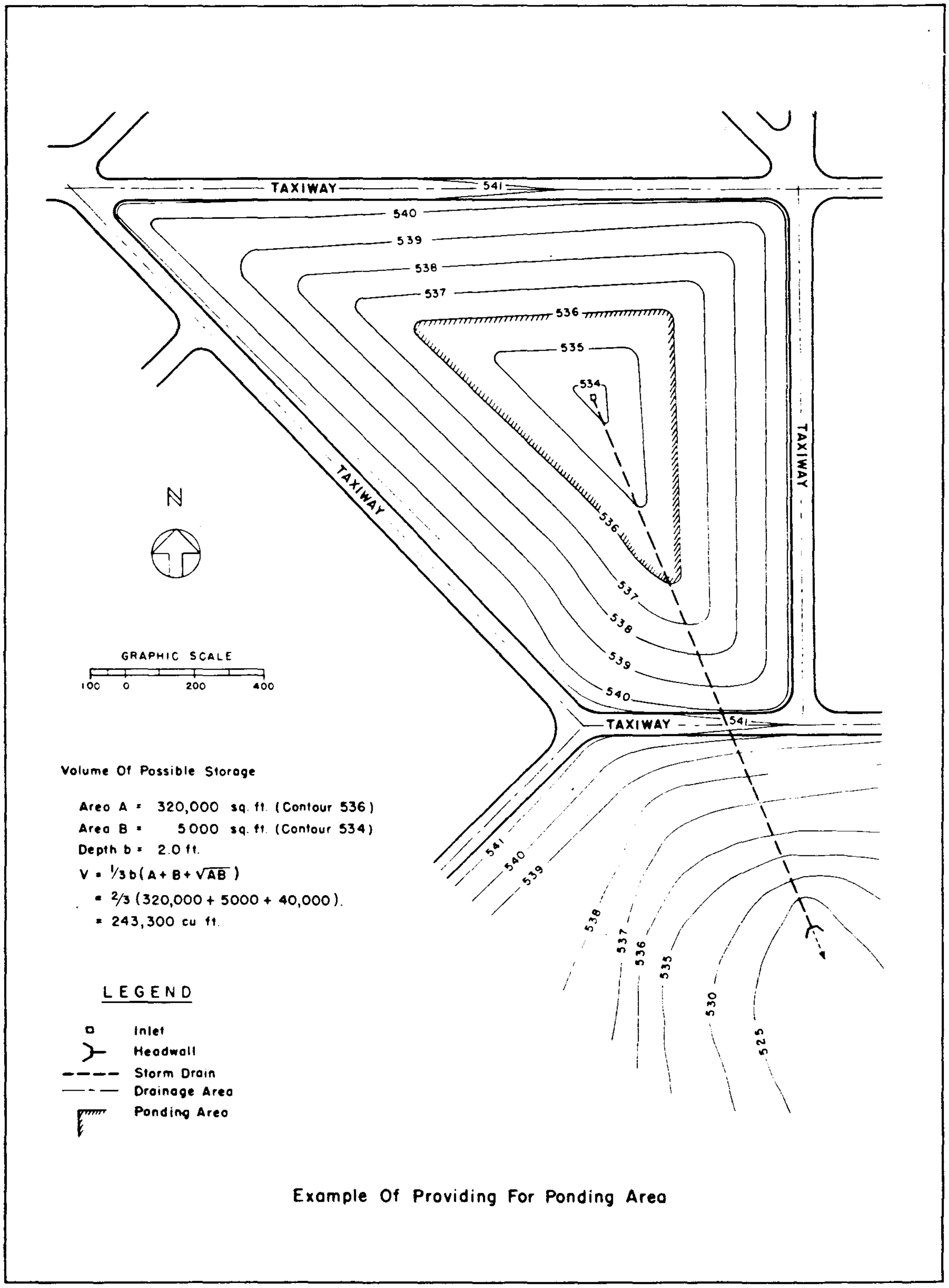


FIGURE 34. Example of providing for ponding area.

TABLE VI. Computations for ponding example in Figures 34 and 35

COMPUTATIONS FOR PONDING EXAMPLE IN FIGURES 34 AND 35

Hourly Intensities for Various Time Intervals from Figure 6

Time	5 yr. Frequency	10 yr. Frequency
5 min.	6.12	6.96
10 min.	4.68	5.34
15 min.	3.96	4.48
20 min.	3.40	3.90
30 min.	2.74	3.12
60 min.	1.76	1.97
90 min.	1.28	1.45
120 min.	1.05	1.23

Q = CIA

A = 4.47 Acres, Pavement
= 45.05 Acres, Turf
= 49.52 Acres, Total

C = 0.90 For Pavement
= 0.30 For Turf

I = 1.50 In. (From Fig. 6 for 75.2 min.)

Q = 0.354 x 1.60 x 49.52 = 26.29 c.f.s.

n = 0.015 S = 0.7% 30" pipe will carry 30 c.f.s.

Distance most remote point – 1600'
120' across pavement, 1480' across turf
Concentration Time: 4.5 + 70.7 = 75.2 minutes

$$\text{Average } C = \frac{4.47 \times 0.90}{49.52} + \frac{45.05 \times 0.30}{49.52} = 0.354$$

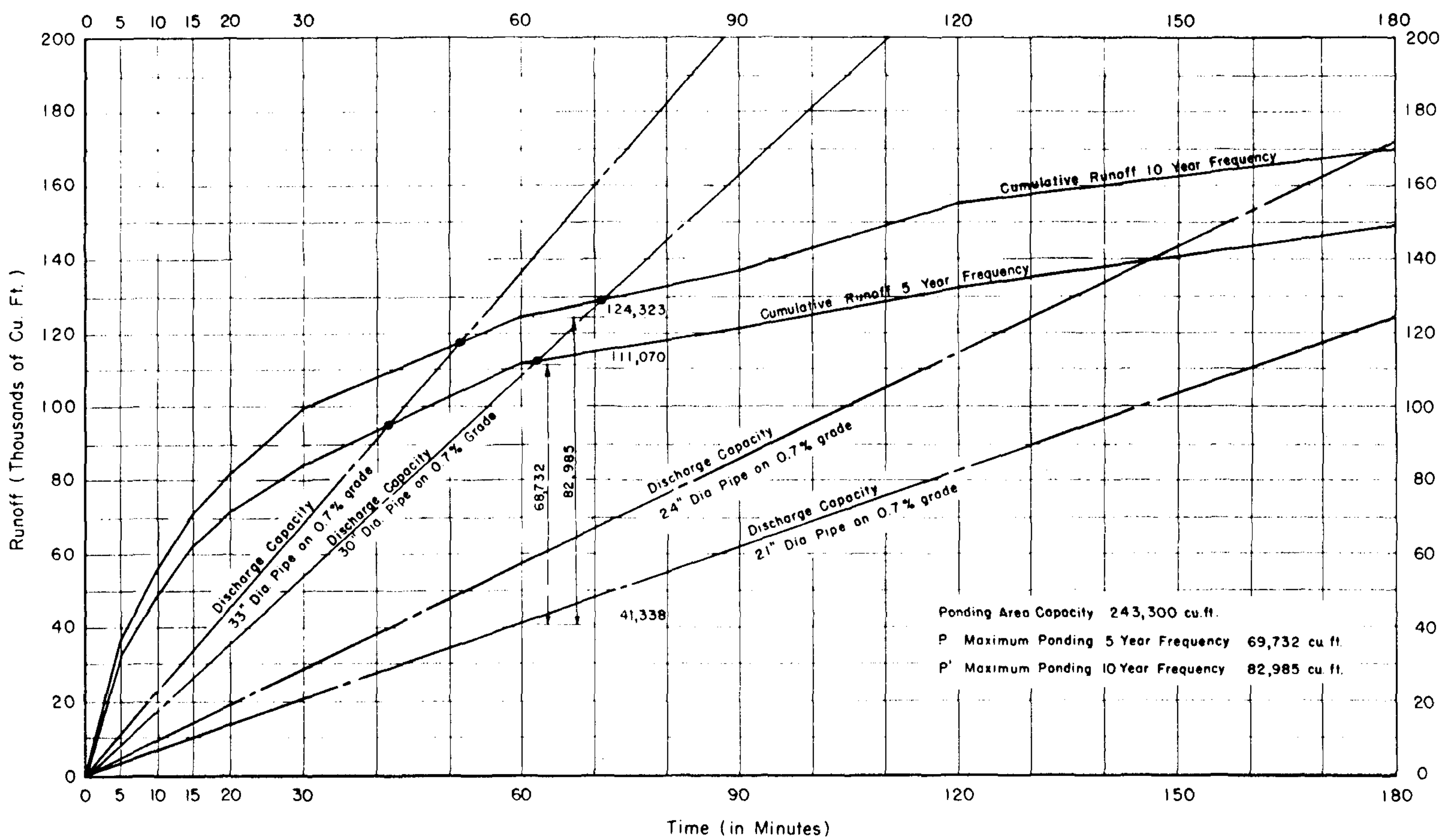
CA = 49.52 x 0.354 = 17.53
Runoff rate when all areas contributing
1 hr. = 3600 x 30 = 108,000 c.f.

Cumulative Runoff in cu. ft.
I = 6.12 (From above)
Q = CIA CA = 17.53
Q = 17.53 x 6.12 = 107.28 c.f.s.

For 5 min. for 5 yr. storm
5 min. = 300 seconds
107.28 x 300 = 32185 cu. ft.*

Thus:

Minutes	Cu. Ft. Supplied By 5 Yr Storm	Cu. Ft. Supplied By 10 Yr Storm
5	17.53 x 6.12 x 300 = 32185*	17.53 x 6.96 x 300 = 36603
10	17.53 x 4.68 x 600 = 49224	17.53 x 5.34 x 600 = 56166
15	17.53 x 3.96 x 900 = 62477	17.53 x 4.48 x 900 = 70681
20	17.53 x 3.40 x 1200 = 71522	17.53 x 3.90 x 1200 = 82040
30	17.53 x 2.74 x 1800 = 86458	17. 3 x 3.12 x 1800 = 98448
60	17.53 x 1.76 x 3600 = 111070	17.53 x 1.97 x 3600 = 124323
90	17.53 x 1.28 x 5400 = 121167	17.53 x 1.45 x 5400 = 137260
120	17.53 x 1.05 x 7200 = 132527	17.53 x 1.23 x 7200 = 155246



Cumulative runoff for ponding in Figure No 34

FIGURE 35. Cumulative runoff for ponding.

21. SUBSURFACE DRAINAGE.

a. Subsurface drainage to be considered on airports consists, in general, of providing intercepting drains to divert subterranean flows, draining wet masses or areas, controlling moisture in the base or subbase of pavement or any combination of these. Draining large field areas by subsurface drainage is not usually necessary on airports, since it can be done more efficiently by grading properly and installing surface drainage. Subdrains shoud be designed to function as subsurface drains only and should not operate to remove surface drainage.

b. The presence of a high-water table on an airport site calls for a thorough soil survey and a determination of the cause of such underground water. The details of a comprehensive soil survey are covered in Chapter 2 of AC 150/5320–6A. The water table may be extensive or be located in one or more isolated portions of the site. The soil horizons and types of soil will definitely reveal whether it is:

(1) pocketed in pervious soils over impervious stratum,

(2) In low areas of an undulated impervious stratum,

(3) confined within a porous waterbearing stratum, or

(4) within a high flood plane of a stream or watershed.

In many locations the water table fluctuates with the seasonal rainfall. This should be checked when making the soil survey. Conditions 1 and 2 can generally be best relieved by the use of subsurface drains placed within the actual areas having the high-water table. Conditions 3 and 4 are usually remedied by correctly placing intercepting surface ditches to cut across the porous water-bearing stratum, or to install intercepting drain lines, occasionally supplementing either with subsurface drains within the area affected. Figure 36 illustrates types of subsurface installation that have proved satisfactory.

c. Even though a very thorough soils survey of the site has been made, the presence of free-flowing water should be noted during construction. When encountered, action should be taken to collect and dispose of it. If free-flowing water is found in only a small area, the drain line may be carried to an appropriate outfall. If that solution is not found to be practical, the line may be connected to the sealed surface system by a connection similar to that shown in Figure 37. Care must be taken to prevent the water in the sealed system, when flowing full, from backing up into the subdrainage line and saturating the area contiguous to the subdrain.

d. Certain types of soils are self-draining, some can be drained by artificial means, and others are not drainable.

(1) Soils such as gravelly sand, silty sand, and some types of clay sands are often self-draining.

(2) Soils like sandy clay, clay silts, and certain sandy silts are drainable, and subsurface drains wil be effective. The percentage of sand in these soils determines their ability to be drained.

(3) Soils composed of silt or clay without a sand content such as silty clay, silt, and clay are difficult or impossible to drain.

(4) Although the above general soil classifications are helpful in deciding where subsurface drainage will be practical, the perviousness of a given soil, base, or subbase course is measured by the coefficient of permeability. There are several methods of determining the coefficient—both in the laboratory and in the field. The coefficient of permeability is expressed as centimeters per sec. (1 cm./sec. = 1.97 ft./min.) representing rate of flow. The range of coefficients is very large so they are usually expressed as values with exponents ranging from the most permeable at 10^{2} to very impermeable at 10^{-9}. The value 10^{-4} is a useful reference point as a soil with such a coefficient is just able to drain a very heavy rainfall. The coefficient of permeability of untreated base and subbase courses depends principally upon the percentage by weight of the fraction passing the 200-mesh sieve. For example, on that basis material meeting FAA construction Specifications P–154, P–208, or P–209 would have coefficients ranging from 10^{-1} to 10^{-4}.

A Corps of Engineers study shows, however, that the drainage characteristics of base course materials may also depend on the density of the material. Increases in density (compaction) cause large decreases in permeability and moderate decreases in effective porosity. Moreover, the drainability of a highly compacted material can approach zero when it contains as little as 5 percent fines (passing the 200-mesh sieve).

The investigation of need for subsurface drainage to protect the pavement subgrade should therefore include a determination of permeability of subgrade soils and pavement courses. Similarly, the practicality of intercepting drains could be indicated by the coefficient of permeability of the soil in the area selected for installation of the drain pipe.

Pavement courses that have been adequately stabilized with cement or bitumen are impermeable, therefore, a pavement edge drain system would be unnecessary. A high water table or other circumstances resulting in wet subgrade soils may, in these cases, best be handled by interceptor drains.

e. It is important, during grading operations, to place the best drainable type of soils available adjacent to or beneath the paved areas. This will form the strongest soil structure where it is most beneficial and, at the same time, will provide drainage away from the base and subbase. The poorer undrainable types of soils should be moved to non-traffic areas.

f. Figure 36 illustrates several different types of subdrainage systems often used on airports. These are only examples. The particular type to install will depend upon the actual conditions at each airport site. A review of the soil survey data during construction is the only safe way to determine the proper type of subdrainage system.

g. The design of a subsurface drainage system is somewhat similar to that of a surface drainage system. The flow from a subsurface system is considerably less than for other types, and the grades are usually flatter. The grades should not be less than 0.15 foot in 100 feet. The type of surface, the soil, the infiltration, the spacing and depth of the drains, the amount of precipitation of seepage, and other factors all affect the flow and, therefore, the size of the pipe needed.

(1) The rate of infiltration for subdrainage that is commonly used is 0.25 to 0.50 of an inch in 24 hours. A rate of 0.25 inch per acre in 24 hours is equal to 0.0105 cubic feet per second for each acre.

(2) When the rate of infiltration is known the proper size of pipe may be determined from Figures 9 through 12.

h. Generally, a single line of subsurface drains along the pavement edges is sufficient for the width of most runway and taxiway bases. It may be necessary on bases wider than 75 feet from crown to edge—such as on aprons—to install intermediate lines of drains. Item 7 of the bibliography is a source of design procedure and criteria for determining the spacing of such intermediate drains.

i. The types of pipe used for subdrains are: plain or perforated vitrified clay or concrete pipe, perforated corrugated aluminum alloy pipe, perforated corrugated steel pipe, cradle invert vitrified clay pipe, perforated asbestos-cement pipe, perforated bituminous-fiber pipe, and porous concrete pipe.

j. A type of subdrainage installation considered important in many localities for the protection of the base and subbase of the runways and taxiways is the intercepting drain. This drain should be placed across and at the lowest portion of the seepage stratum in order to cut off and divert the entire flow. The drain should seldom, if ever, be placed under the pavement proper.

k. The control of moisture under pavement is the principal reason for subsurface drainage along the pavement edges. Free water may collect below the pavement under several different conditions. The water table may rise into the base or subbase during an exceptionally wet season, or it may be high enough to supply capillary water to the top of the subgrade. Frost layers contribute free water when they thaw out and this water should be carried away by proper drains. This can be done by connecting the pervious base and subbase or the pavement with the backfill material in the subdrain system. The subsurface drains should be installed in accordance with "C" in Figure 37. As shown on the drawing, these drains need not be large; and, under normal conditions, a pipe 6 or 8 inches in diameter will suffice.

It is necessary to have access to subdrains for observation and flushing. Inspection/flushing holes are usually spaced no more than 500′ apart. They are constructed of the same type and size pipe as the subdrain with a grate or cover at the surface.

l. In some localities, low temperatures and snowfalls result in deep frost penetrations and deep accumulations of snow on the pavement shoulder area and adjacent thereto. As a consequence, the thaw period is prolonged and may result in saturation and thus, instability of the subgrade.

Although installation of subsurface drains at the pavement edges is the preferred solution, it has been found that, in some areas, a widened base or subbase works as well and is reasonable in cost (see paragraph 13b(3)).

m. When pervious bases and subbases are used with impervious subgrades, the low area in the longitudinal profile of the pavement can be a troublesome water collection basin. Normally, subdrainage pipes should be installed at the pavement edges to provide an outlet for that water. In some cases, however, french drains (trench sections filled with pervious material) leading from the base and/or subbase outward into the shoulder area will provide some relief.

n. The construction specifications should require backfilling the subsurface trenches with well compacted granular material to act as a filter. To prevent the possibility of large quantities of surface water entering these drains, the pervious backfill material surrounding the drains should not extend to the top of the trench.

o. The filter material requirement should be carefully considered because the quantity of water to be handled by these subdrains is relatively small and it is possible that the surrounding natural soil may filter into interstices of the filter material. The following should be considered in filter and underdrain design:

(1) A fine material will not wash through a filter material if the 15-percent size of the filter material is less than 5 times as large as the 85-percent size of the fine material and the fine material is well graded. If the fine material (natural foundation soil) is uniformly graded, then the 15-percent size of the filter (backfill) material should be less than 4 times as large as the 85-percent size of the fine material.

(2) The ratio of the permeability of the filter material is also important to allow free water to reach the pipe. Appropriate permeability will be assured by conformance with the following equations:

$$\frac{15\% \text{ size filter material}}{15\% \text{ size foundation soil}} = 5.0 \text{ or greater and}$$

$$\frac{15\% \text{ size filter material}}{15\% \text{ size foundation soil}} = \text{not more than } 25.0.$$

(3) In addition to meeting the above size specification, the grain size curves for filter and fine material should be approximately parallel in order to minimize washing of the fine material into the filter material.

(4) Filter materials should be packed densely, to reduce the possibility that movement of the fines might cause any change in the gradation.

(5) A filter material is no more likely to fail when flow is upward than when flow is in some other direction, unless the seepage pressure becomes sufficient to cause flotation or a "quick" condition of the filter.

(6) A well-graded filter material is less susceptible to running through the drain pipe openings than a uniform material of the same average size. However, even a filter material having a wide range of gradation cannot be used successfully over a drain pipe having a large opening, since enough fine particles to cause serious clogging will move out of the well-graded filter into the pipe.

(7) Large openings in the drain pipe tend to increase the rate of infiltration, but also increase the tendency for filter material to collect in and clog the pipe.

(8) Where it is feasible to design and use two gradations of backfill consisting of a separate layers with coarse aggregate near the openings of the pipe, pipes with larger openings would probably operate satisfactorily.

p. Figure 38 is a graph of the gradation of a sample soil that is uniformly graded, and another that is well graded. It also shows the uniform filter material required for backfill to prevent infiltration of the uniform soil into the filter material. Also shown is the well-graded filter material required for backfill that will prevent infiltration. This graph is an example to illustrate the factors discussed.

(1) To use the graph, follow along the curve drawn for the well-graded soil to a point where 85-percent size passes the 0.25 millimeters. Then follow along the curve drawn for a well-graded backfill to where 15-percent size passes a certain sieve. It will be noted that it is the 1.25 millimeter size or 5 times the 0.25 millimeters. This also holds true for uniform material curves if the 85-percent size of the uniform soil is multiplied by 4 to check with the 15-percent size of the uniform backfill material. Thus these curves illustrate the piping ratio requirements. Similarly, they illustrate the permeability ratio requirements.

(2) To use a graph of this type, the natural soil should be screened for a mechanical analysis and the gradation curve plotted. Then establish the 15-percent size of the backfill material just less than 5 times the 85-percent size of the natural soil, and construct a curve for the backfill material parallel to the original soil curve. This will be the curve of the gradation of the backfill material desired.

(3) It will be noted from a study of Figure 38 that there will be a separate and distinct gradation curve for each type of soil analyzed. Consequently, there will be a separate gradation curve for the backfill material to use with each soil type. The limiting piping ratio for a uniform soil is 4, and for a well-graded soil is 5, and a backfill material with a parallel gradation curve not exceeding the piping ratio will prove satisfactory in preventing infiltration. If the specifications are written so that the gradation for the backfill material follows the exact curve drawn parallel to the soil gradation curve with its required piping ratio for each type of soil, it will be seen that the graded backfill material will be very difficult to produce commercially as many different soil gradations may be encountered. Figure 39 illustrates several soil gradation curves and also indicates the theoretical curve (No. 5) with a piping ratio of 5 for the backfill material for soil type No. 4. Also plotted on the graph are the specification limits for commercial size concrete sand and concrete coarse aggregate.

(4) A backfill material is safe to use if the gradation curve for that material indicates that the particle sizes are less than the plotted theoretical gradation curve with a piping ratio of 5 for any particular soil. Figure 39 indicates that the specification limits of commercial size concrete sand meet this requirement for the well-graded soil No. 4. However, the permeability ratio requirement of 15 percent size versus 15 percent size being greater than 5.0 would require that curve No. 8 be used. Accordingly, the concrete sand should be checked against curve No. 8. A preliminary investigation should be made in the locality of the site to determine the type and gradation of concrete sand available. If the sand falls within the limits shown, it should be used for backfill material.

(5) Certain installations will require the specifying of two separate sizes of backfill materials. The example shown in Figure 39 contemplates the use of the commercial size concrete sand for the backfill material adjacent to the well-graded soil No. 4 to prevent infiltration. Assuming that the openings in the drainpipe would allow the finer backfill material to enter the pipe if placed directly against it, a larger size material which will not enter the pipe nor allow displacement of the finer backfill material should be placed adjacent to the pipe.

(6) Using the same procedure as was used to establish curve No. 5, the safe gradation is determined for the coarse backfill material (larger size material) by using Figure 39 again. A theoretical gradation curve (No. 7) with a piping ratio of 5 is constructed for the finer backfill material by using the finer side of the gradation band for the concrete sand as the reference curve (No. 6) for this new gradation curve, since the percentage of the smallest

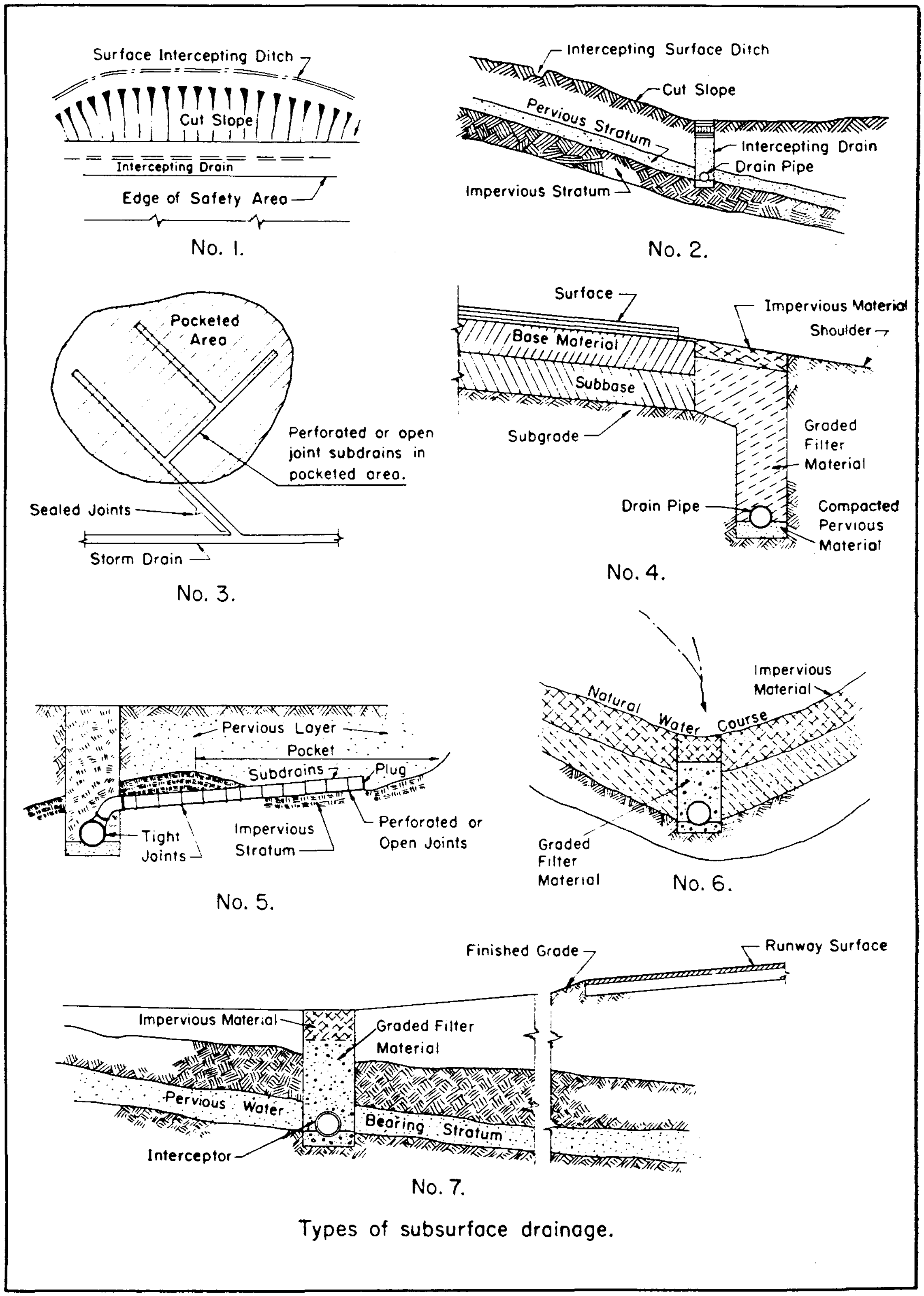


FIGURE 36. Types of subsurface drainage.

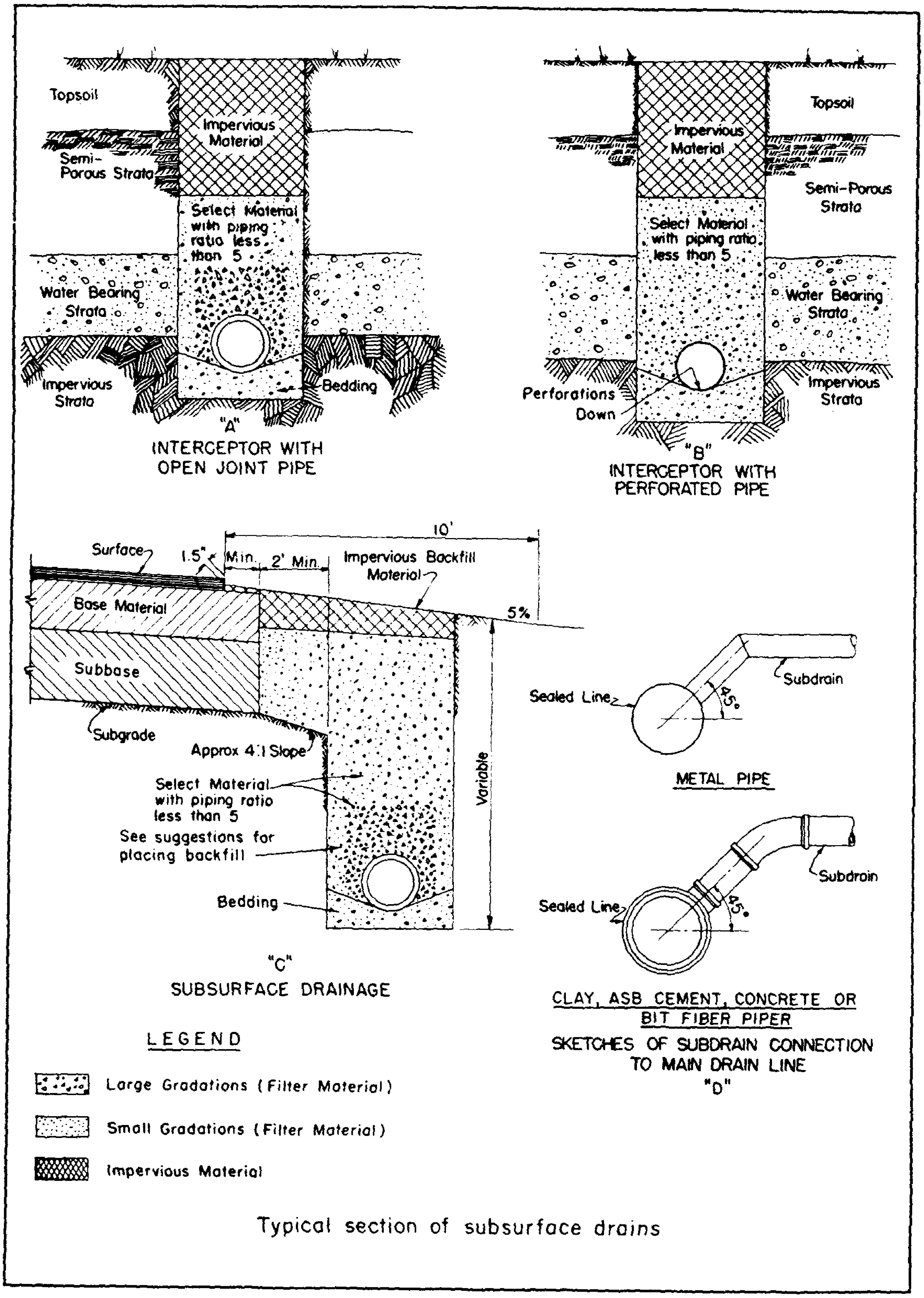
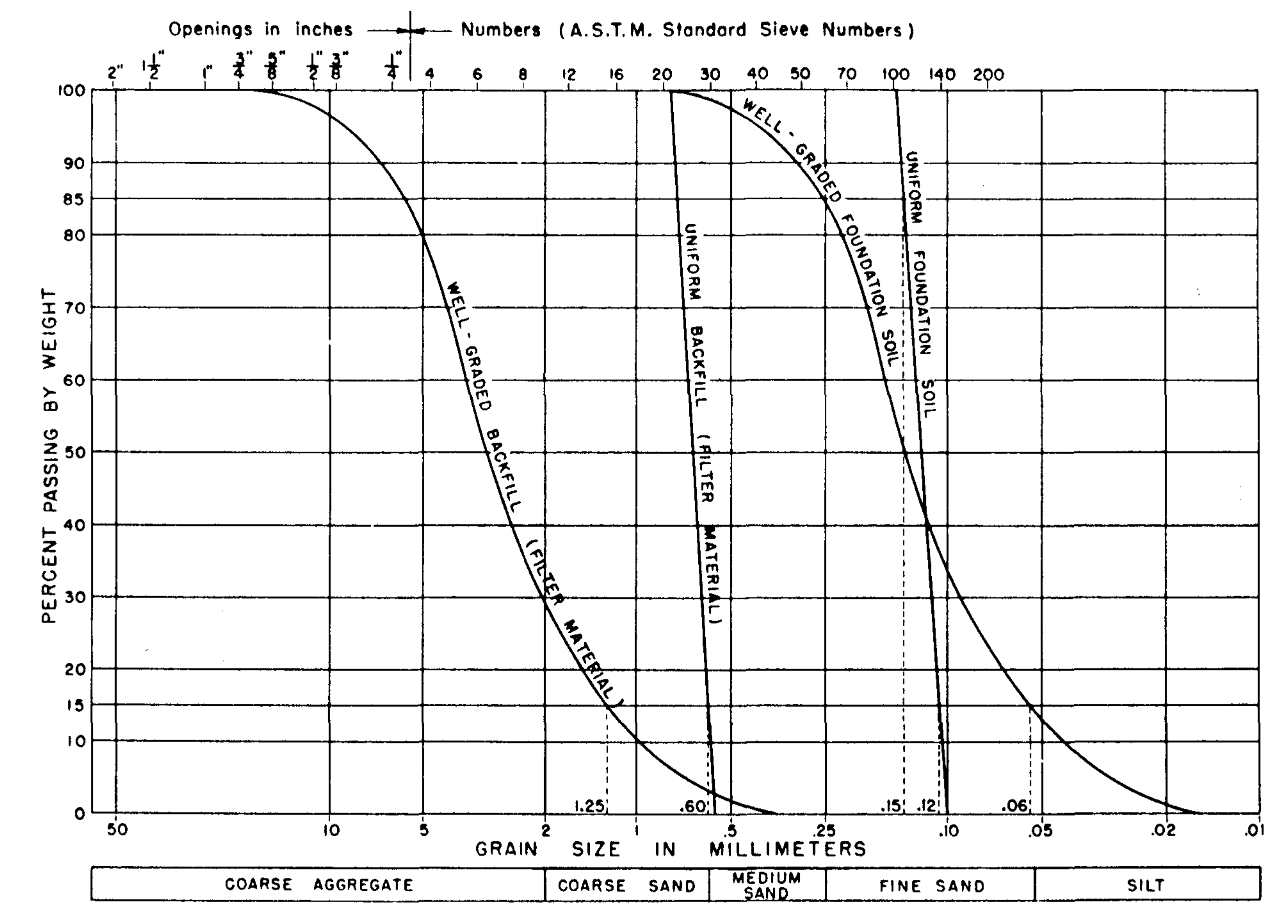


FIGURE 37. Typical section of subsurface drains.



PIPING RATIO:

$\frac{\text{15\% SIZE UNIFORM BACKFILL}}{\text{85\% SIZE UNIFORM FOUNDATION}}$, NOT GREATER THAN 4.0

$\frac{\text{15\% SIZE WELL GRADED BACKFILL}}{\text{85\% SIZE WELL GRADED FOUNDATION}}$, NOT GREATER THAN 5.0

PERMEABILITY RATIO:

$\frac{\text{15\% SIZE FILTER MATERIAL}}{\text{15\% SIZE FOUNDATION SOIL}}$, 5.0 OR GREATER

$\frac{\text{15\% SIZE FILTER MATERIAL}}{\text{15\% SIZE FOUNDATION SOIL}}$, NOT GREATER THAN 25

Limiting gradation for backfill surrounding pipe for subsurface drains.

FIGURE 38. Gradation of backfill for subsurface drains.

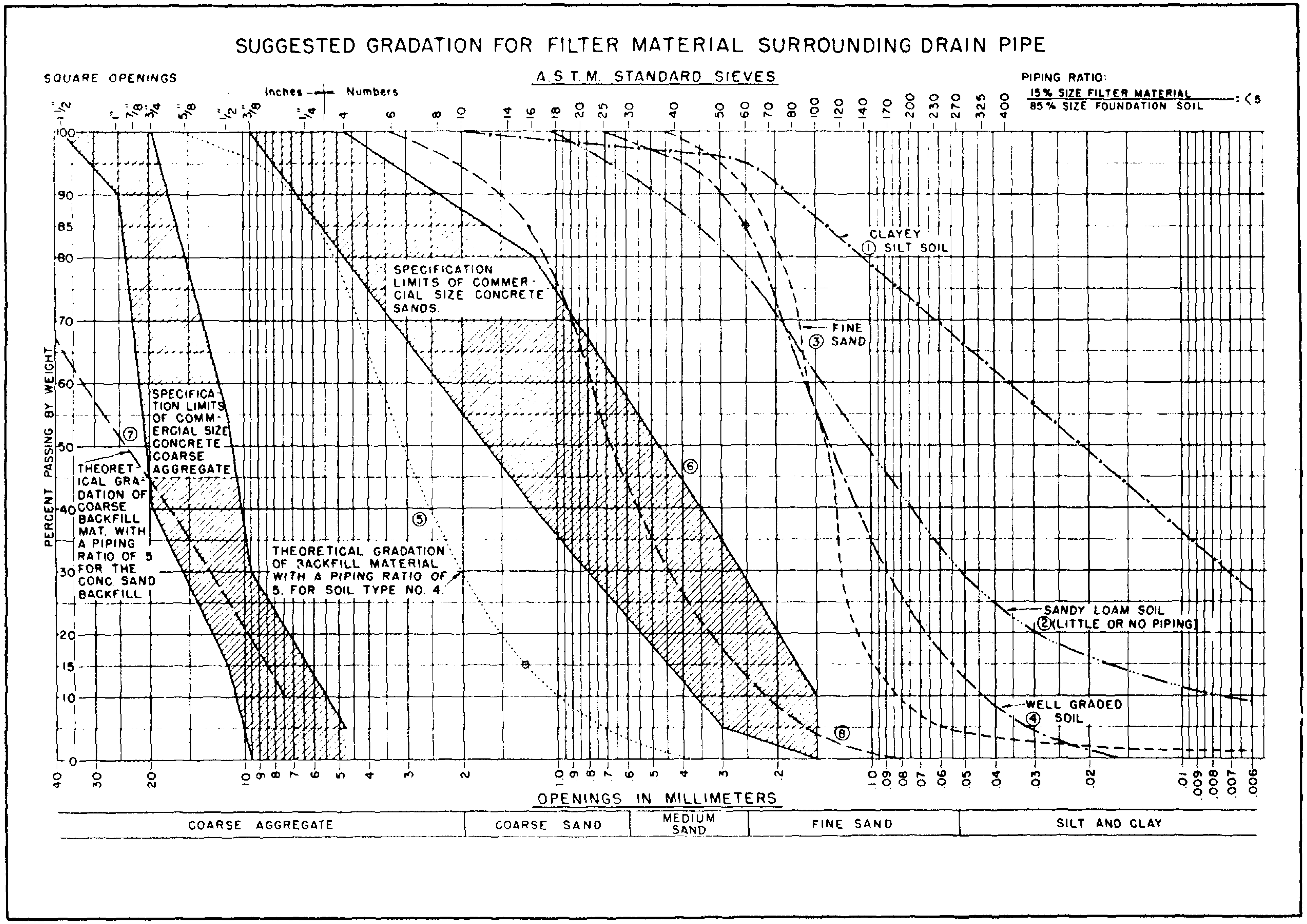


FIGURE 39. Gradation for filter material.

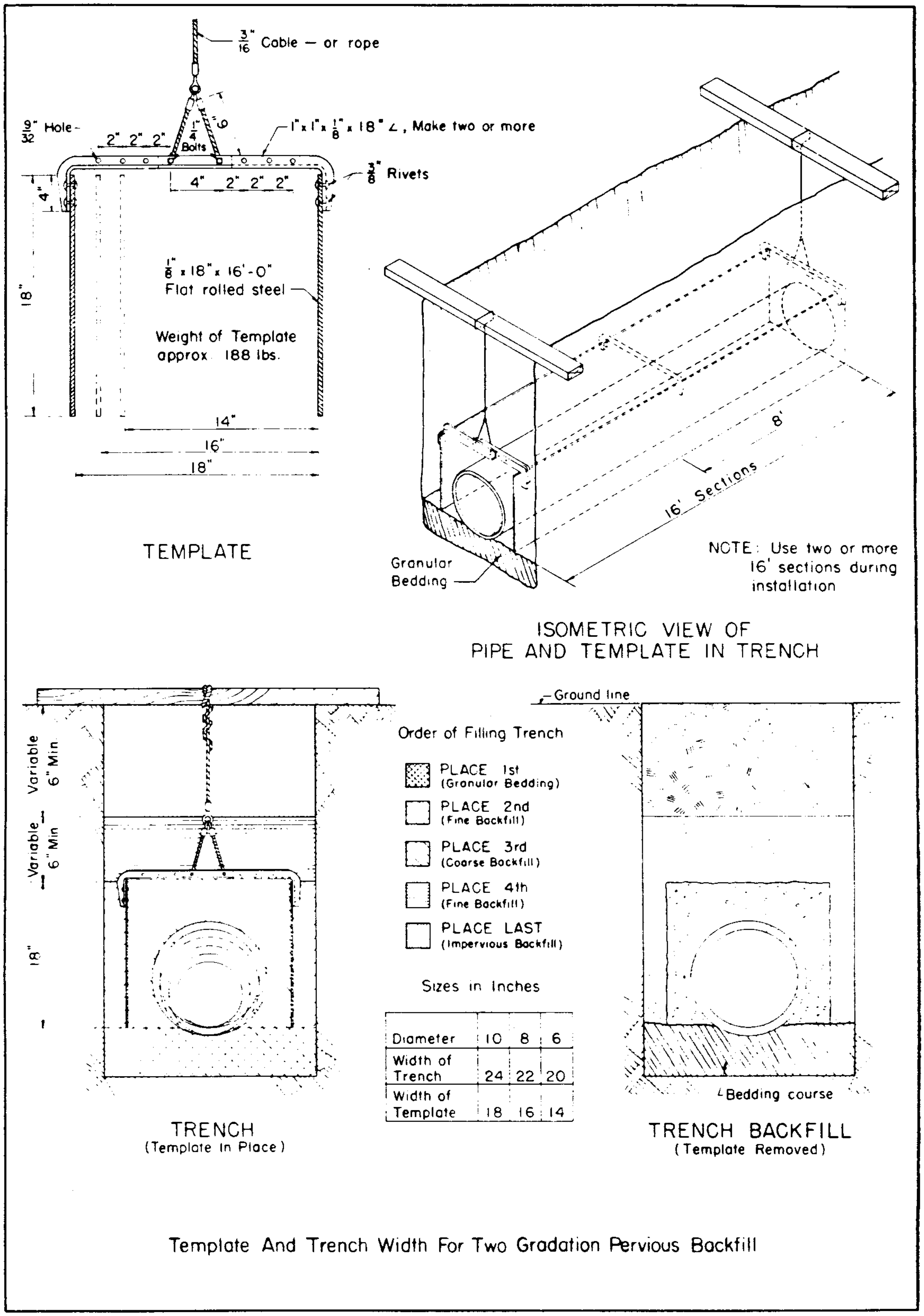


FIGURE 40. Template and trench width for pervious backfill.

size will indicate the material most difficult to control. This new curve No. 7 indicates that the courser side of the gradation band of the commercial size concrete coarse aggregate will satisfactorily prevent infiltration of the fine backfill material and will not enter the pipe. With openings in pipe rarely exceeding 1/4 inch and with the arbitrary piping ratio of 5, a factor of safety is provided for use in actual field conditions.

q. During the past few years, industry has developed woven plastic filter cloths which have been found to be satisfactory for protection against beach erosion, for protection against steep slope erosion and as a filter in subsurface drain installations.

In the latter connection, note that paragraph o above refers to the possibility of even well-graded granular filter materials moving into pipe openings or perforations. It is recognized too, that gradations of granular material, meeting the piping ratio and other requirements of paragraph n above, may not be economically available. In these cases, a single wrap of woven filter cloth around the pipe may be used in lieu of the coarser backfill illustrated in Figure 40. Filter cloths have openings with a size generally like the No. 40 sieve. The cloth may be wrapped around open joints of unperforated pipe or around the entire length of perforated pipe. Prefabricated filter cloth sleeves may also be used to encase the pipe. When the gradation of granular filter material is such that it satisfies requirements pertaining to material adjacent to joint openings or pipe perforations, but is too coarse to satisfy the filter criteria pertaining to the protected soil, a single layer of filter cloth may be used adjacent to the protected soil in lieu of a second filter material. This use, however, is restricted to situations where the protected soil is sand.

22. CONSTRUCTION.

The usual construction work associated with a drainage system includes such items as excavation, trenching and shoring; preparation of bedding, laying, aligning and jointing of pipe; (Figure 41, Methods of laying drainage pipe) backfilling and compacting; installing structures; and cleaning up. A successful and efficient airport drainage system should be well designed and should be constructed in accordance with the requirements of AC 150/5370-1A, Standard Specifications for Construction of Airports. Quality construction which is attained by consistently using proper and accepted construction methods and practices along with adequate inspection ensures a drainage system that functions properly. Poor construction leads to progressive deterioration and endless maintenance and reconstruction problems.

23. POLLUTION CONTROL.

Construction of drainage systems and grading as a contributory part of good drainage systems are to be accomplished in a way to control pollution. Sodding or some other form of stabilization of graded slopes and ditches should be done to minimize erosion and prevent pollution of streams and rivers.

Although this advisory circular is aimed at storm water drainage and subsurface drainage and not at sanitary sewer systems, it should be recognized that civil airports often support populations which are sufficiently large to contribute to the pollution of water resources if raw sewage is discharged from the airport structures such as the terminal building. Installation of sewage treatment plants, or diversion to such plants, should be required and should conform with local, county, or State sanitary codes. In either case, the sanitary sewer system should be separate from the storm sewer system.

24. MAINTENANCE OF THE SYSTEM.

a. Maintenance is essential to preserve and prolong the service and utility of all drainage facilities. All structures and visibe units of the system should be inspected frequently, and the malfunctionings should be immediately corrected. Several items that will need constant checking are: the inlet grates for clogging by grass cuttings, sticks, ice, and debris; the catch basins and pipelines for stoppage by sediment, and waste; settlement around pipes and structures from infiltration; stoppage in outfall ditches; erosion around structures in watercourses or embankments; high shoulders

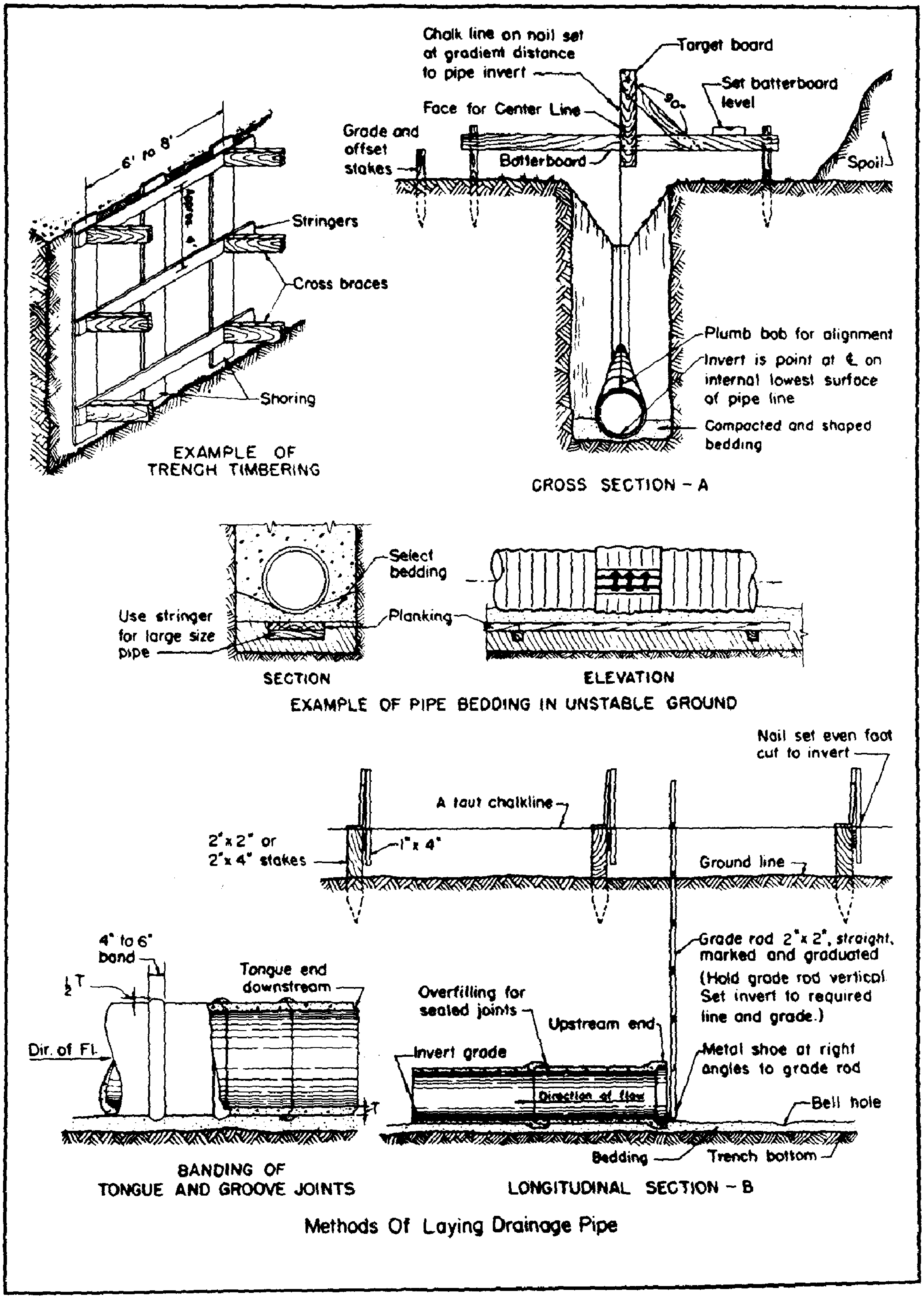


FIGURE 41. Methods of laying drainage pipe.

on pavements or structures; and any damage to the structures. A little maintenance at the right time may prevent major repairs later.

b. A qualified member of the airport personnel should be selected to be in charge of all drainage maintenance matters. He should be provided with sufficient and suitable equipment, tools, materials, supplies, and labor for necessary maintenance and repairs. Periodic inspections should be made, including a patrol of the system during or after a storm if conditions do not seem normal. As a minimum program, a complete inspection should be made in the fall in preparation for winter, and another in the spring to determine the extent of maintenance needed. Proper inspection and maintenance require familiarity with design, capacity, and location of drainage facilities.

c. Mechanical devices for cleaning drain lines of silt, sand, and other debris include various cutters, brushes, scoops, scrapers, and screws which are drawn through by hand or power-operated windlasses. These tools, some of which are adjustable, are available to fit all sizes of pipes. Sectional sewer rods with working and flushing heads can be used alone or with cutting devices. One flushing method often used is by blocking all openings in a manhole, filling it with water and then quickly removing the block at the outlet. The rapid flow of the released water usually will clean the pipe.

d. When ditches alone or in combination with natural watercourses comprise the surface drainage system, they should be properly maintained. Ditch slopes should be maintained to the original design slope. Where possible, a dense turf should be developed to stabilize open ditches. The dense turf should be mowed frequently as tall growth decreases flow. Ditches should be kept free of weeds, brush, logs, silt, and other debris which might divert or restrict the flow at any time.

e. When maintenance of an airport is being considered, the entire area within its boundary should be included. Any obstruction which could alter the designated flow should be changed, corrected, or removed. One item that will be objectionable and require periodic correction is high shoulders along the pavement edges. Also, formation of deep ruts may on occasion concentrate the runoff to an undesirable extent. Some surface obstruction may cause the flow to channelize and start erosion. Such conditions should be corrected. In patrolling the airport, attention should be given to the adequacy of the drainage design. Proper inspection might disclose that some portions of the waterways and structures could require enlarging, replacing, or additions. It is generally good practice and more economical to make minor corrections when the faults are detected, rather than to have major, expensive maintenance repairs later.

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