DOT/FAA/RD-90/24
Research and Development Service
Washington, D.C. 20591

Airport Pavement Drainage

Jeanette A. Hare
Richard A. Pur
Barry J. Dempsey

Department of Civil Engineering
University of Illinois
Urbana, Illinois 61801

June 1990

Synthesis Report

This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161.

U.S. Department of Transportation
Federal Aviation Administration

NOTICE

Technical Report Documentation Page

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
DOT/FAA/RD-90/24		

4. Title and Subtitle	5. Report Date
AIRPORT PAVEMENT DRAINAGE - SYNTHESIS REPORT	June 1990
	6. Performing Organization Code
	8. Performing Organization Report No.

7. Author(s)
Jeanette A. Hare, Richard A. Pur, Barry J. Dempsey

9. Performing Organization Name and Address
Department of Civil Engineering
University of Illinois
Urbana, Illinois 61801

10. Work Unit No. (TRAIS)

11. Contract or Grant No.

12. Sponsoring Agency Name and Address
U. S. Department of Transportation
Federal Aviation Administration
Research and Development Service
Washington, D. C. 20591

13. Type of Report and Period Covered
Synthesis Report

14. Sponsoring Agency Code
ARD-200

15. Supplementary Notes

16. Abstract

This report provides a literature review of the state-of-the-art for airport drainage. The report reviews the literature concerning the climatic parameters which relate to airport drainage. A summary of the past practices for both surface and subsurface drainage for airports is provided which describes drainage structures and design procedures. The components of a subsurface drainage system which are applicable to airports are described in the report.

17. Key Words	18. Distribution Statement
Drainage Subdrainage Airport Pavements Climatic Factors	This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	85	

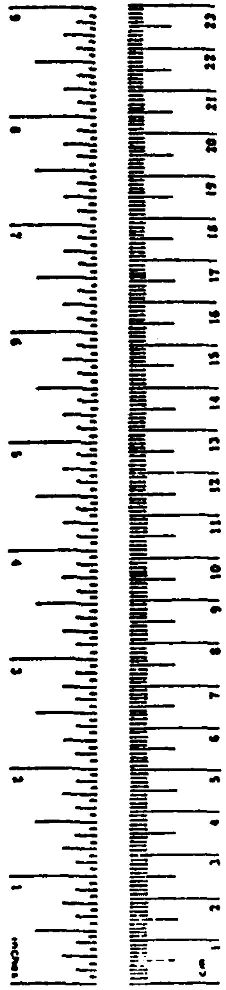
Form DOT F 1700.7 (8-72) Reproduction of completed page authorized

METRIC CONVERSION FACTORS

Approximate Conversions to Metric Measures

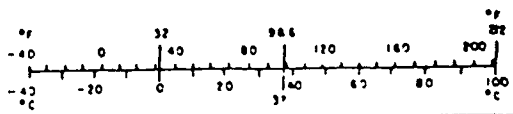
Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
in	inches	*2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
		AREA		
in^2	square inches	6.5	square centimeters	cm^2
ft^2	square feet	0.09	square meters	m^2
yd^2	square yards	0.8	square meters	m^2
mi^2	square miles	2.6	square kilometers	km^2
	acres	0.4	hectares	ha
		MASS (weight)		
oz	ounces	28	grams	g
lb	pounds	0.45	kilograms	kg
	short tons (2000 lb)	0.9	tonnes	t
		VOLUME		
tsp	teaspoons	5	milliliters	ml
Tbsp	tablespoons	15	milliliters	ml
fl oz	fluid ounces	30	milliliters	ml
c	cups	0.24	liters	l
pt	pints	0.47	liters	l
qt	quarts	0.95	liters	l
gal	gallons	3.8	liters	l
ft^3	cubic feet	0.03	cubic meters	m^3
yd^3	cubic yards	0.76	cubic meters	m^3
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C

* 1 in. = 2.54 (exactly). For other exact conversions and more detailed tables, see NBC Misc. Publ. 286, Units of Weights and Measures, Price $2.25, SD Catalog No. C13.10 286.



Approximate Conversions to Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
mm	millimeters	0.04	inches	in
cm	centimeters	0.4	inches	in
m	meters	3.3	feet	ft
m	meters	1.1	yards	yd
km	kilometers	0.6	miles	mi
		AREA		
cm^2	square centimeters	0.16	square inches	in^2
m^2	square meters	1.2	square yards	yd^2
km^2	square kilometers	0.4	square miles	mi^2
ha	hectares (10,000 m^2)	2.5	acres	
		MASS (weight)		
g	grams	0.035	ounces	oz
kg	kilograms	2.2	pounds	lb
t	tonnes (1000 kg)	1.1	short tons	
		VOLUME		
ml	milliliters	0.03	fluid ounces	fl oz
l	liters	2.1	pints	pt
l	liters	1.06	quarts	qt
l	liters	0.26	gallons	gal
m^3	cubic meters	35	cubic feet	ft^3
m^3	cubic meters	1.3	cubic yards	yd^3
		TEMPERATURE (exact)		
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



PREFACE

This synthesis report on airport pavement drainage was prepared for the U.S. Department of Transportation Federal Aviation Administration with the direct supervision of the U.S. Army Corps of Engineers Construction Engineering Research Laboratory, Champaign, Illinois 61821, under contract Numbers DACA 88-85-M-0271, DACA 88-85-M-0786, DACW 88-85-D-0004-11 and DACW 88-85-D-0004-12 by the Department of Civil Engineering, University of Illinois, Urbana-Champaign, Illinois. Dr. Mohamed Shahin was the project coordinator for the U.S. Army Corps of Engineers.

This report is the first of two reports prepared under the specified contracts. This report provides background information of the state-of-the-art for airport pavement drainage. A second report entitled "Guidelines for Design, Construction, and Evaluation of Airport Pavement Drainage," will provide detailed procedures for airport pavement drainage design and construction.

TABLE OF CONTENTS

TABLE OF CONTENTS CONTINUED

LIST OF TABLES

LIST OF FIGURES

Figure		Page

CHAPTER 1

INTRODUCTION

1.1 PROBLEM STATEMENT

Three forms of drainage need to be considered when designing an airport. Surface drainage is needed to direct the flow of water away from pavements and buildings and to eventually remove it from the airfield. Another form is subsurface drainage which is needed to remove the water from beneath the pavement. The third form of drainage, pavement surface drainage, is needed to prevent the build up of water which causes hydroplaning. Serious accidents can result when aircraft lose steering and braking control. All of these forms of drainage will be covered in the following report which summarizes the state-of-the-art in airport drainage systems.

In chapter two, rainfall analysis is discussed because it is important for the designer to estimate the amount of rainfall in the area and the runoff produced on the airfield. The calculated runoff is used in determining the number and size of inlets and other structures needed. The use and placement of these structures is discussed in chapter three.

Poor subsurface drainage in a pavement can lead to failure from slope instability or a rapid decrease in the level of serviceability by causing rutting, cracking, and faulting. Methods of removing water from beneath airport pavements are discussed in Chapter Four.

Much of the subsurface drainage is adapted from highway subdrainage design. Airport runways and taxiways are similar to highways in all drainage aspects except for the distance water has to flow to reach the edge of the pavement.

Unsaturated flow will not be covered in this report. Freeze-thaw and swell problems resulting from unsaturated flow are well documented in other works by Dempsey (1) and Dempsey, Darter, and Carpenter (2).

Pavement surface drainage is covered in chapter five of this report. Both grooving and porous friction courses can be used to remove water rapidly from the pavement surface.

1.2 OBJECTIVES

The objective of this synthesis report is to summarize the present literature and state-of-the-art concerning surface and subsurface drainage for airport pavements. The specific objectives of this report are as follows:

1. Review the literature concerning the climatic parameters which relate to airport drainage.
2. Summarize the present practices for pavement surface drainage evaluation and design.
3. Describe some of the procedures presently being used to promote pavement subsurface drainage.
4. Make recommendations for areas of further study.

REFERENCES

1. Dempsey, B.J., "Climatic Effects on Airport Pavement Systems: State of the Art," Contract Report S-76-12, U.S. Army Corps of Engineers and Federal Aviation Administration, Washington, D.C., 1976.
2. Dempsey, B.J., Darter, M.I., and Carpenter, S.H., "Improving Subdrainage and Shoulders of Existing Pavements - State of the Art," Report No. FHWA/RD-81/077, Federal Highway Administration, Washington, D.C., 1982.

CHAPTER 2

RAINFALL ANALYSIS

2.1 INTRODUCTION

In designing a drainage system, the designer should determine the amounts of rainfall (design precipitation rate) which are likely to occur in the area and consequently, the runoff produced by various precipitation events (storms). It is important to know or calculate how much water can be present in the drainage system after a storm so that the correct types and sizes of aggregate subbases and pipes are chosen for the drainage design. The drainage system must be able to adequately drain the design infiltration rate (to which the design precipitation rate contributes) and maintain an adequate margin of safety.

2.2 FACTORS INFLUENCING RATE OF RUNOFF

In determining the rate of runoff, consideration must be given to many factors. Probable frequency and duration of the design storm are helpful in determining the rainfall intensity for that storm. The type of soil and the moisture content affect the rate of infiltration and therefore the amount of runoff. The perviousness, slope, and irregularities (joints, cracks, depressions, etc.) in the pavement and the surrounding area also effect the runoff rate.

A relationship between rainfall intensity (in./hr) and duration can be derived as shown in Figure 2.1. The curves in Figure 2.1 were developed using rainfall frequency maps similar to those in Figure 2.2. The FAA Advisory Circular on Airport Drainage shows how this procedure was accomplished (1). In Figure 2.1, each curve represents a different storm

return period. Typically, a return period of 5 years is used in estimating the runoff for airfields. Once the intensity-duration graph is derived, the intensity of the design storm can be determined.

There are many intensity, duration, and frequency models used in rainfall analysis. Some probability distribution models for quantities of precipitation are presented by Kattegoda (2). The Gamma distribution is another theoretical model used for frequency distribution of precipitation (3). Some frequency models for rainfall, such as the Markov Chain Method, estimate the probability distributions of the lengths of sequences of dry days and wet days of the pavement system (4,5). Thus, there is no one set method for analyzing rainfall frequency.

2.3 CALCULATION OF RUNOFF

The Rational Method is the most widely used method for calculating runoff. It is based on a direct relationship between runoff and rainfall. The method uses the equation:

$$Q = CIA \qquad \text{(Eq. 2.1)}$$

where:

Q = runoff in ft^3/sec for a given area,

C = runoff coefficient depending on the character of the drainage,

I = intensity of rainfall in in./hr., and

A = drainage area in acres.

If there is a combination of areas with different runoff coefficients, a composite runoff coefficient can be calculated using the following equation:

$$C_t = \frac{C_1A_1 + C_2A_2 + \ldots + C_nA_n}{A_1 + A_2 + \ldots + A_n} \qquad \text{(Eq. 2.2)}$$

where:

C_t = composite runoff coefficient,

C_n = runoff coefficient for each individual area,

A_n = area of each individual study section, and

n = number of areas being combined.

Typical ranges of values for runoff coefficients are given in Table 2.1. (6).

REFERENCES

1. Airport Drainage, Advisory Circular AC 150/5320-5B, Federal Aviation Administration, Washington, D. C., 1970.
2. Kottegoda, K.T., "Stochastic Water Resources Technology," Wiley, New York, 1980.
3. Suzuki, E., "A Summarized Review of Theoretical Distributions Fitted to Climatic Factors and Markov Models of Weather Sequences, with Some Examples," Statistical Climatology, Elsevier, Amsterdam, Netherlands, 1980.
4. Gabriel, K.R. and Neumann, J., "A Markov Chain Model for Daily Rainfall Occurrence at Tel Aviv," Quart. J. Roy. Met. Soc. 88, 1962.
5. Katz, R.W., "Computing Probabilities Associated with the Markov Chain Model for Precipitation." Journal of Applied Meteorology 13, 1974.
6. Modern Sewer Design, American Iron and Steel Institute, Washington, D.C., 1980.

Table 2.1 Typical Runoff Coefficients for the Rational Method (Ref. 6).

Description of Area	Runoff Coefficients
Business	
Downtown	0.70 to 0.95
Neighborhood	0.50 to 0.70
Residential	
Single-family	0.30 to 0.50
Multi-units, detached	0.40 to 0.60
Multi-units, attached	0.60 to 0.75
Residential (suburban)	0.25 to 0.40
Apartment	0.50 to 0.70
Industrial	
Light	0.50 to 0.80
Heavy	0.60 to 0.90
Parks, cemeteries	0.10 to 0.25
Playgrounds	0.20 to 0.35
Railroad yard	0.20 to 0.35
Unimproved	0.10 to 0.30

It often is desirable to develop a composite runoff based on the percentage of different types of surface in the drainage area. This procedure often is applied to typical "sample" blocks as a guide to selecton of reasonable values of the coefficient for an entire area. Coefficients with respect to surface type currently in use are:

Character of Surface	Runoff Coefficients
Pavement	
Asphalt and Concrete	0.70 to 0.95
Brick	0.70 to 0.85
Roofs	0.75 to 0.95
Lawns, sandy soil	
Flat, 2 percent	0.13 to 0.17
Average, 2 to 7 percent	0.18 to 0.22
Steep, 7 percent	0.25 to 0.35

The coefficients in these two tabulations are applicable for storms of 5-to 10-yr frequencies. Less frequent, higher intensity storms will require the use of higher coefficients because infiltration and other losses have a proportionally smaller effect on runoff. The coefficients are based on the assumption that the design storm does not occur when the ground surface is frozen.

INTENSITY CURVES FOR STORMS IN VICINITY OF EXAMPLE SITE

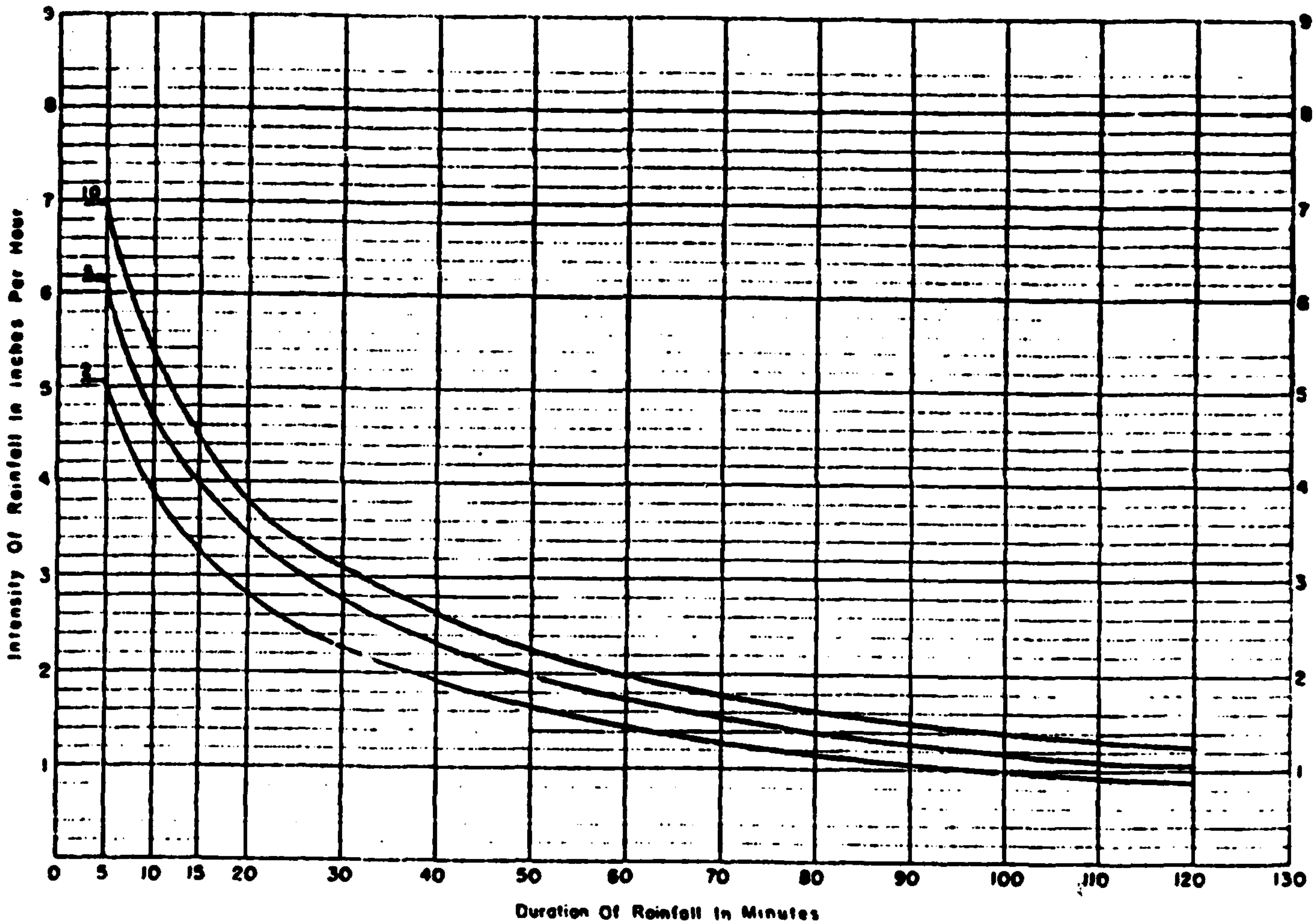
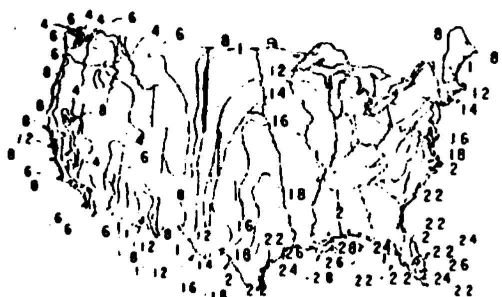


Figure 2.1 Relationship Between Rainfall Intensity and Duration (Ref. 1).



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

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

Figure 2.2 Rainfall Frequency Maps (Ref. 1).

CHAPTER 3

SURFACE DRAINAGE SYSTEMS

3.1 GENERAL

An example of the general surface drainage design process can be found in the FAA Advisory Circular on Airport Drainage (1). In order to design the surface drainage system for an airport a contour map of the airport and adjacent areas including the layout of runways, taxiways, and aprons is needed. This working drawing should have contour intervals of one foot. The general directions of flow and any natural watercourses should first be noted. Surface and interceptor ditches can be located around the periphery of the airport to prevent water from flowing onto pavement areas. Figure 3.1 shows typical interceptor ditches. Inlet structures are then located at the lowest points in the field area. Inlets should be spaced so that the flow from the farthest point in the drainage area is not more than 400 ft. Each of the inlet structures must be connected by pipelines leading to the major outfalls. All surface flow should be away from the pavements and not directed across them.

It is good practice to place manholes at all changes in pipe grades, sizes, changes in direction and junctures of pipe runs for inspection and cleanout purposes. A reasonable interval where these features are not present is 300 ft to 500 ft. Where manholes are impractical drop inlets can be used to allow access for observation and flushing.

Ponding can provide capacity in the drainage system for direct runoff. The provision for ponding between runways, taxiways, and aprons will insure a safety factor and provide an area to temporarily hold runoff from storm return periods longer than 5 years. Ponding areas should be kept at least 75 feet away from pavement edges. This will prevent the ponded water from

saturating the pavement base or subbase. Ponding on a more permanent basis is acceptable away from the paved areas when there is no convenient outfall offsite.

After all of these features have been located the next step is to compute the size and gradients of the pipes. An example of these calculations can be found in Chapter 3 of the FAA Advisory Circular on Airport Drainage (1). Manning's formula, which is the most widely used for this purpose, is as follows:

$$Q = \frac{1.486 R^{2/3} S^{1/2} A}{n} \qquad \text{(Eq. 3.1)}$$

where:
- Q = discharge in cfs,
- R = hydraulic radius (area of section/wetted perimeter) in ft,
- S = slope of pipe invert in ft/ft,
- A = cross sectional area, ft^2, and
- n = coefficient of pipe roughness.

A nomograph for solving Manning's formula is shown in Figure 3.2.

Profiles of the 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 pipe. Flow lines through the pipe will be uniform if the pipe size doesn't change. Drop inlets can be installed to prevent the pipeline gradient from becoming too steep.

Drainage of aircraft fueling aprons should provide for the safe disposal of fuel spillage. The aprons should slope away from buildings to properly drain the fuel. Interceptors, separators, or water seal traps can be used to isolate the drains and to prevent the transmission of flame or vapor from fuel spillage.

3.2 STRUCTURES

3.2.1 Introduction

In general the structures in an airport drainage system are similar to those used in municipal construction. Structures in the usable area of an airport should not extend above ground level. They should be 0.1 ft to 0.2 ft below the ground level to allow for possible settlement around the structure, to permit unobstructed use of the area by equipment, and to facilitate entrance of surface water. The structures used most often are inlets, catch basins, manholes, and headwalls. Some suggested headwall details are shown in Figure 3.3. Embankment protection structures are also used at some airports. Examples of these structures are shown in Figure 3.4.

3.2.2 Grates

Grates are used where the surface water is admitted into the system. These may be cast in steel, iron, or ductile iron. Figures 3.5 and 3.6 show examples of grates and inlet structures respectively which are used on airports. These grates should be strong enough to support the load from the aircraft and maintenance equipment in the area. The number and capacity of grates is determined by the depth of head at the grate and the quantity of runoff. The general weir formula is used to calculate capacity in low head situations. For medium and high heads the orifice formula is used. These formulas and the transition between them are described in Figure 3.7.

A slotted grate, such as that shown in Figure 3.8, could be used for some airport drainage applications (2). This slotted drain, made of cast iron, can capture large quantities of water when placed perpendicular to the flow. Installation is accomplished by sawing a slot in the top of a drainage pipe,

placing the slotted grate in place, then placing concrete around the system, Figure 3.8. Figures 3.9 and 3.10 show typical performance relationships for the slotted grate when compared to a conventional grate for varying longitudinal slopes. For these flow rates and slopes the grate captured all of the flow. These grates could have applications on airport aprons, taxiways, and runways if their load carrying capacity and strength meet requirements for airports.

3.2.3 Inlet Structures

Inlet structures may be constructed of reinforced concrete, brick, concrete block, precast concrete, or rubble masonry. Whatever material is chosen must be strong enough to withstand any applied loads. Inside barrel dimensions are commonly 3 1/2 ft in diameter and 4 ft in height, Figure 3.11.

The backfill around pavement inlet structures should be compacted with particular care to prevent differential settlement. In rigid pavements the inlet is normally isolated by expansion joints placed around its frame.

Catch basins are not necessary for airport drainage if the drains are laid on self-cleaning grades. Under certain conditions they might be necessary to prevents solids and debris from washing into the system.

Manholes are basically standardized to type and come in round, oval, square, or rectangular shapes. They are usually constructured of reinforced concrete, brick, concrete block, precast concrete, corrugated metal, or precast pipe sections.

REFERENCES

1. "Airport Drainage," Advisory Circular AC 150/5320-5B, Federal Aviation Administration, Washington, D.C., 1970.

2. Neenah Grate Information, Neenah Foundry Company, Neenah, Wisconsin, 1985.

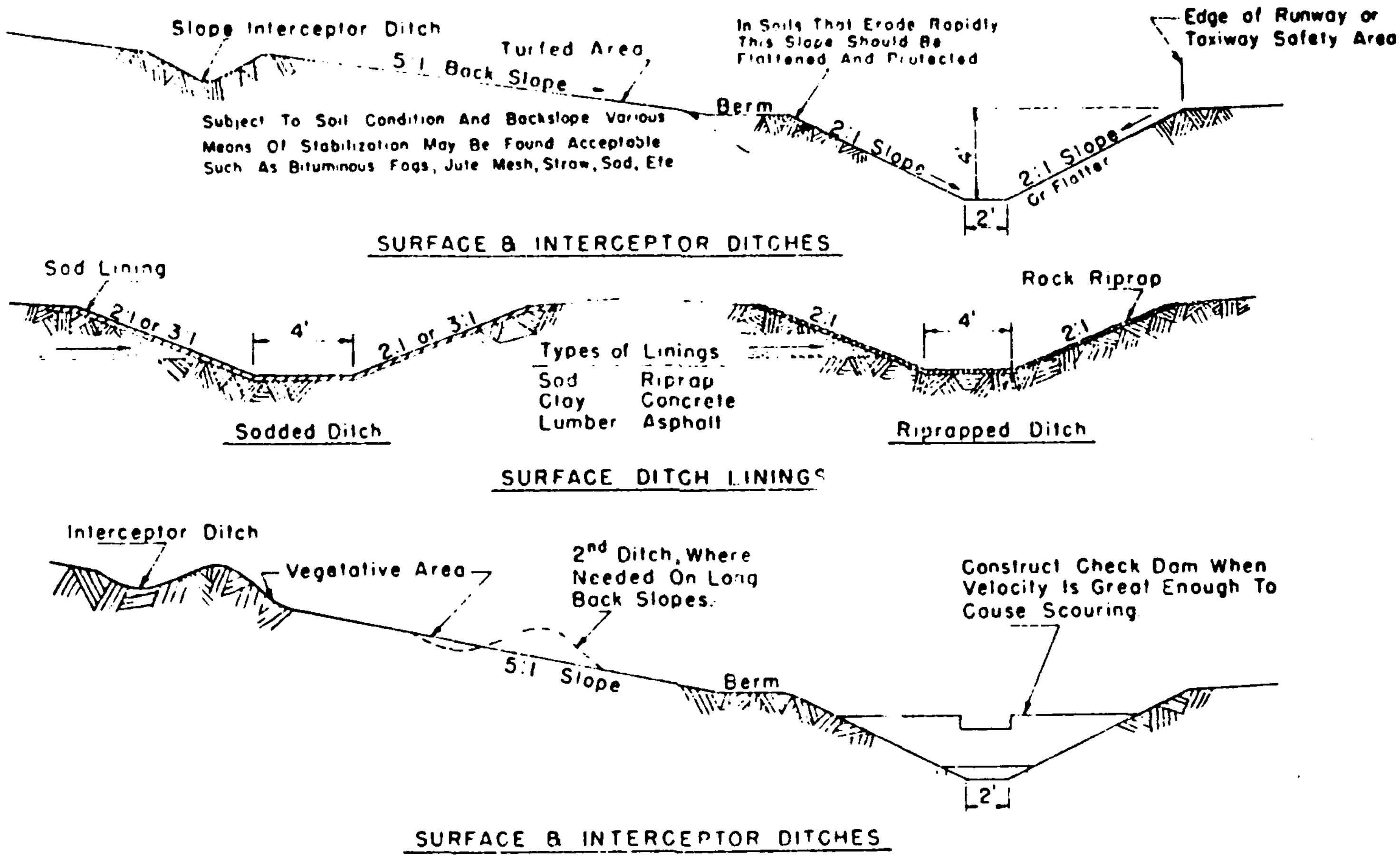


Figure 3.1 Types of Surface and Interceptor Ditches for Airports (Ref. 1)

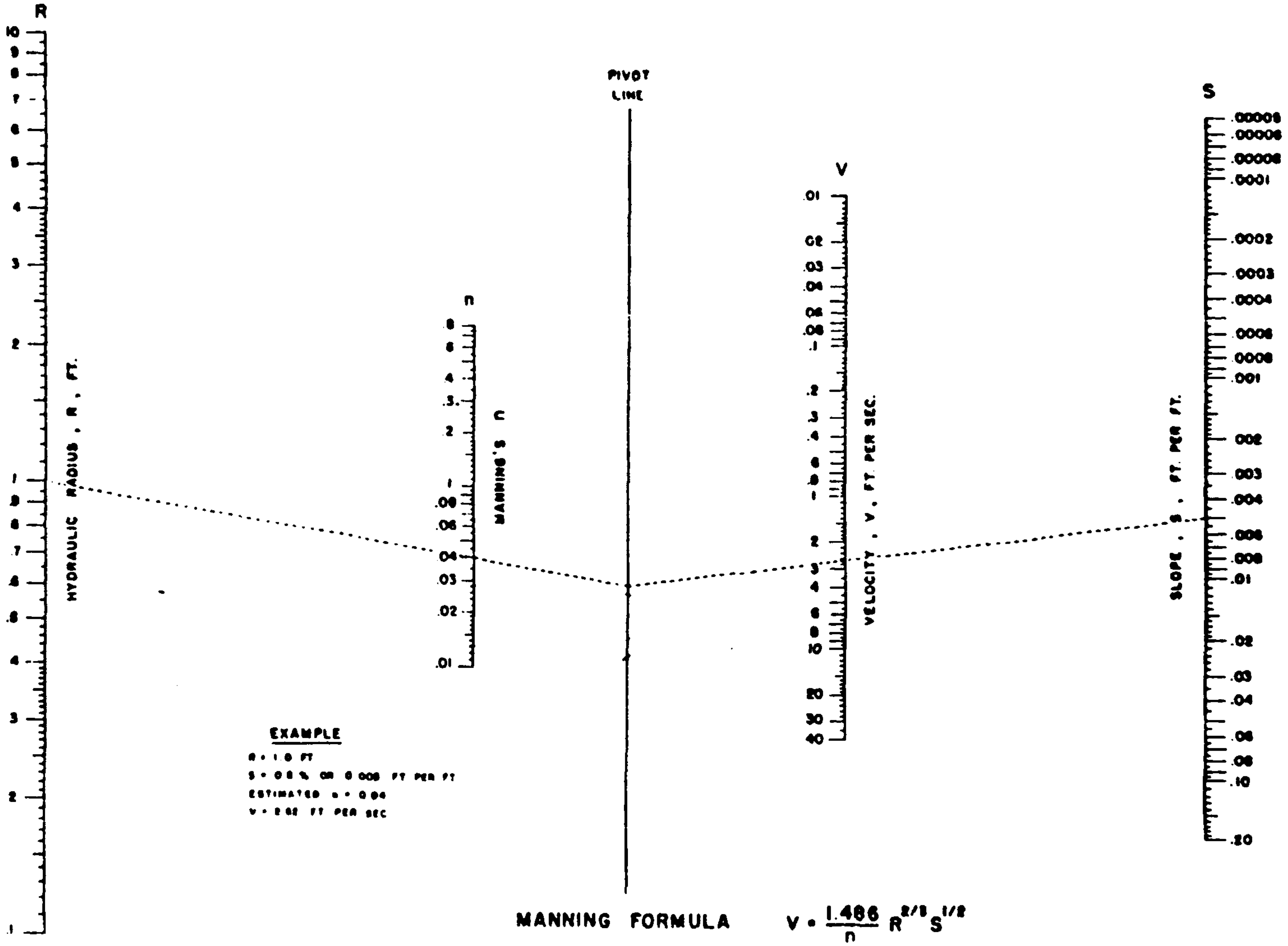
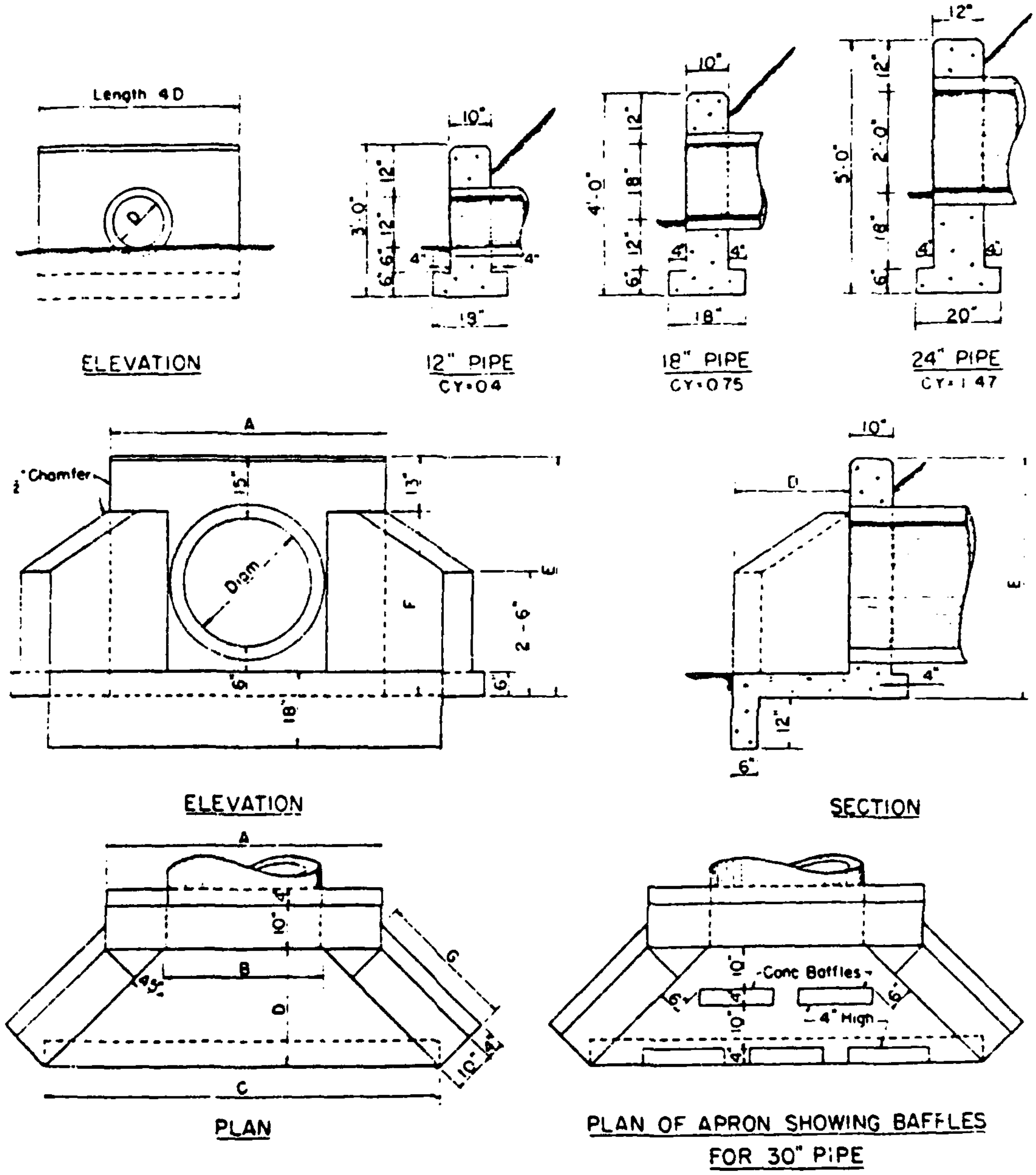


Figure 3.2 Nomograph for Solving Manning's Formula (Ref. 1).



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'-5¾"
36"	2.24	6'-0"	3'-8"	9'-10"	3'-1"	5'-3"	4'-2"	3'-6½"
42"	2.88	7'-0"	4'-2"	11'-10"	3'-10"	5'-9"	4'-8"	4'-5½"
48"	3.59	7'-6"	4'-8"	13'-10"	4'-7"	6'-3"	5'-2"	5'-6¾"

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

Figure 3.3 Typical Headwall Details for Drainage (Ref. 1).

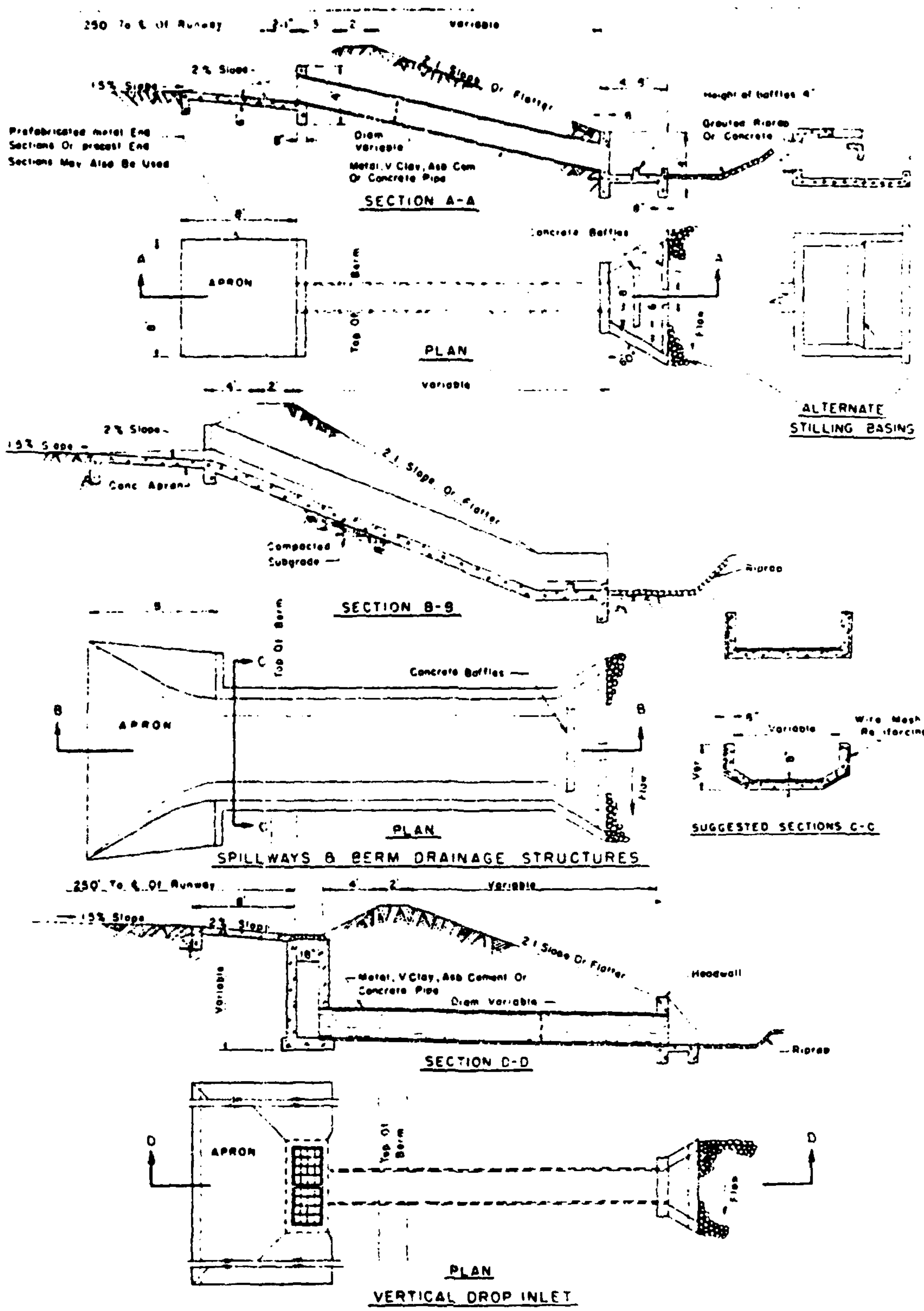


Figure 3.4 Typical Embankment Protection Structures (Ref. 1).

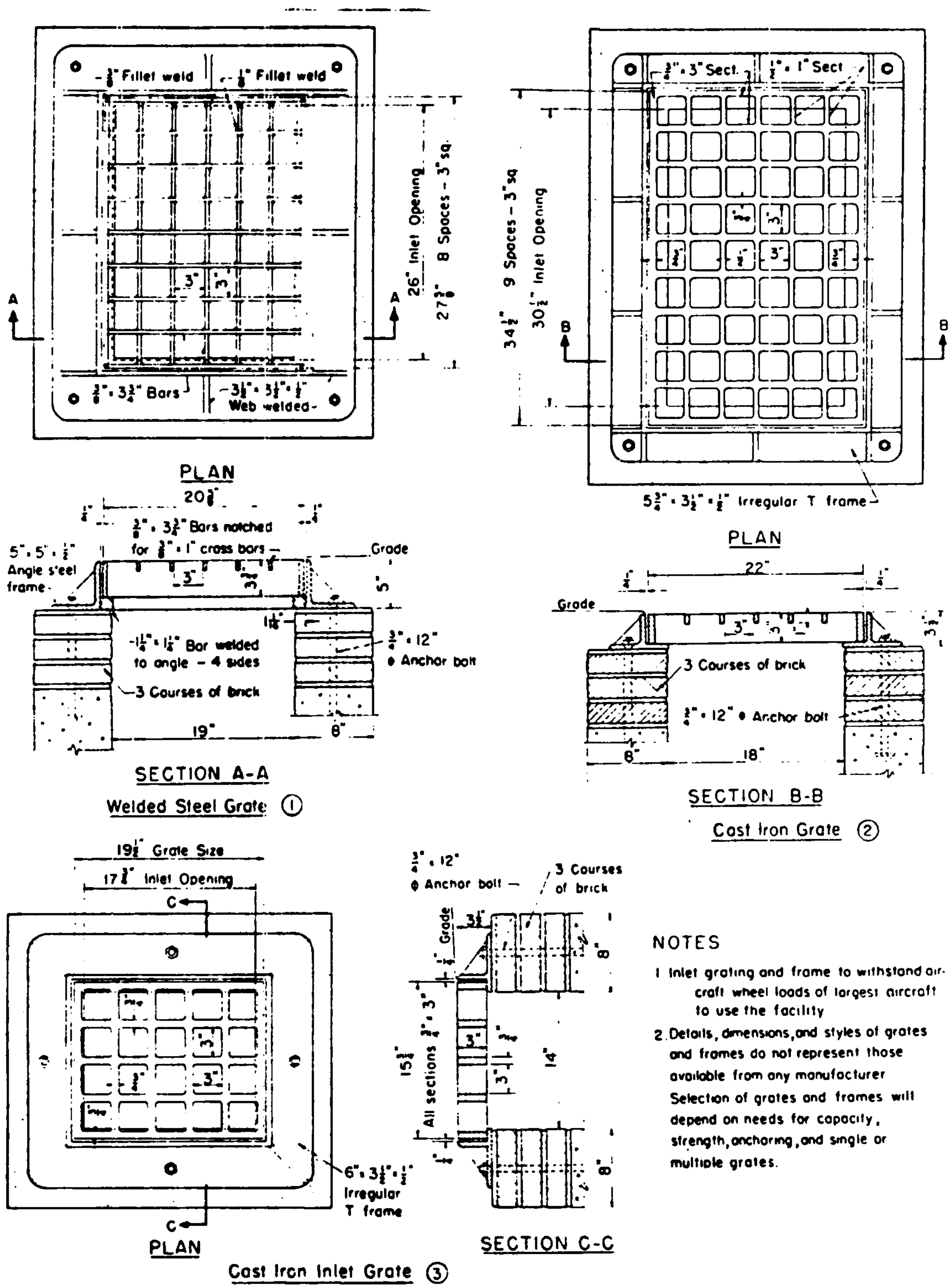
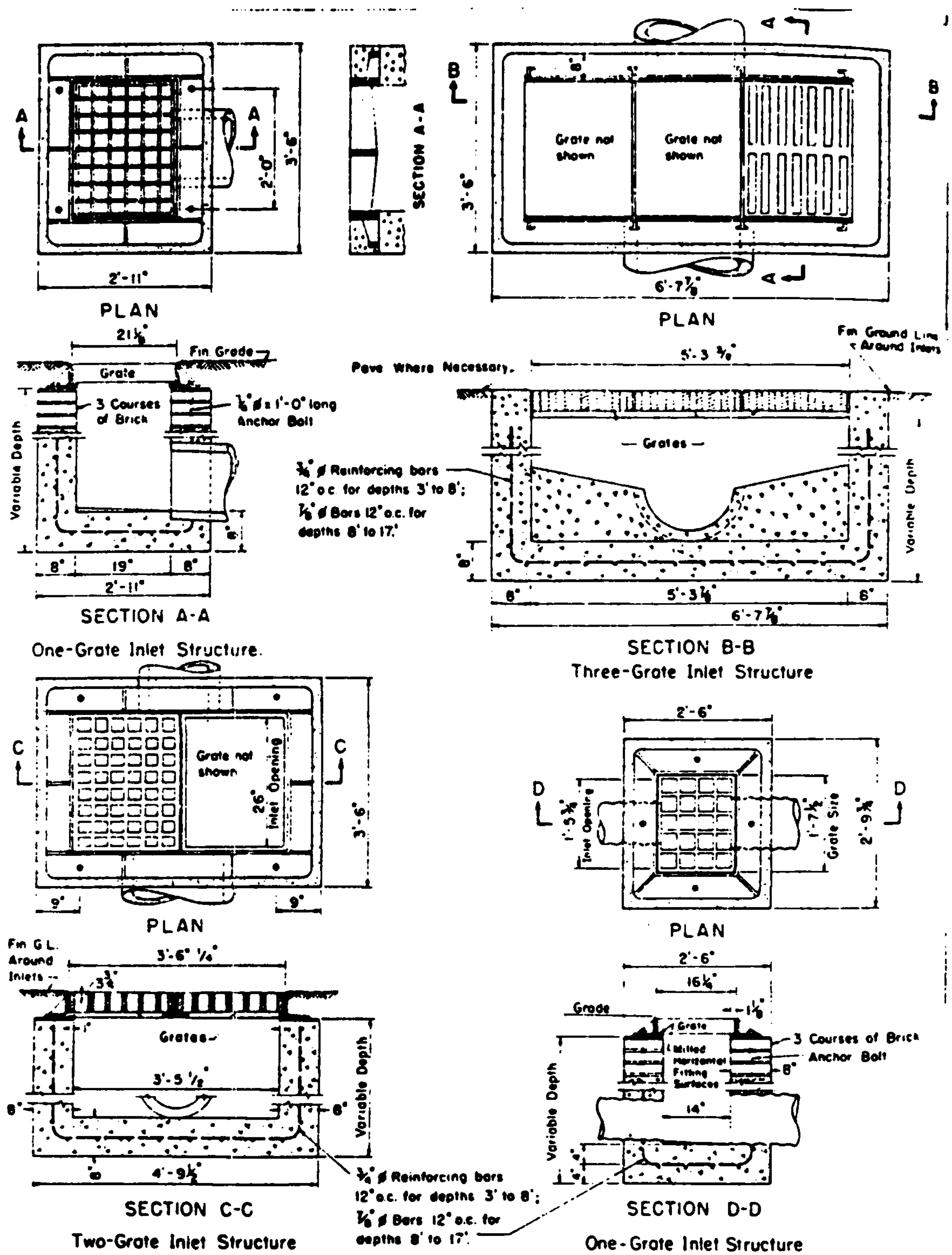


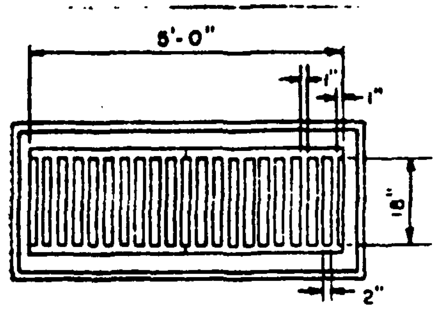
Figure 3.5 Examples of Typical Inlet Grates (Ref. 1).



NOTES

Details, dimensions, and materials for inlets as well as for grates and frames are illustrative only.

Figure 3.6 Examples of Grate Inlet Structures (Ref. 1).



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

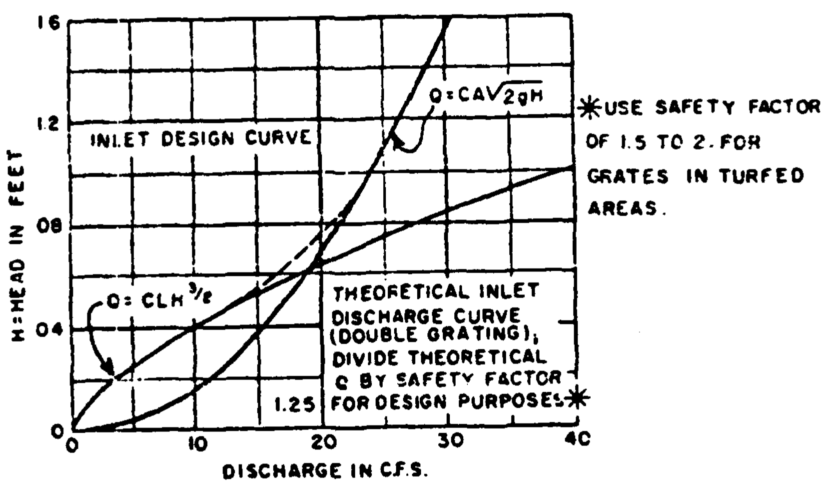


Figure 3.7 Determination of Typical Inlet Grating Discharge Curve (Ref. 1).

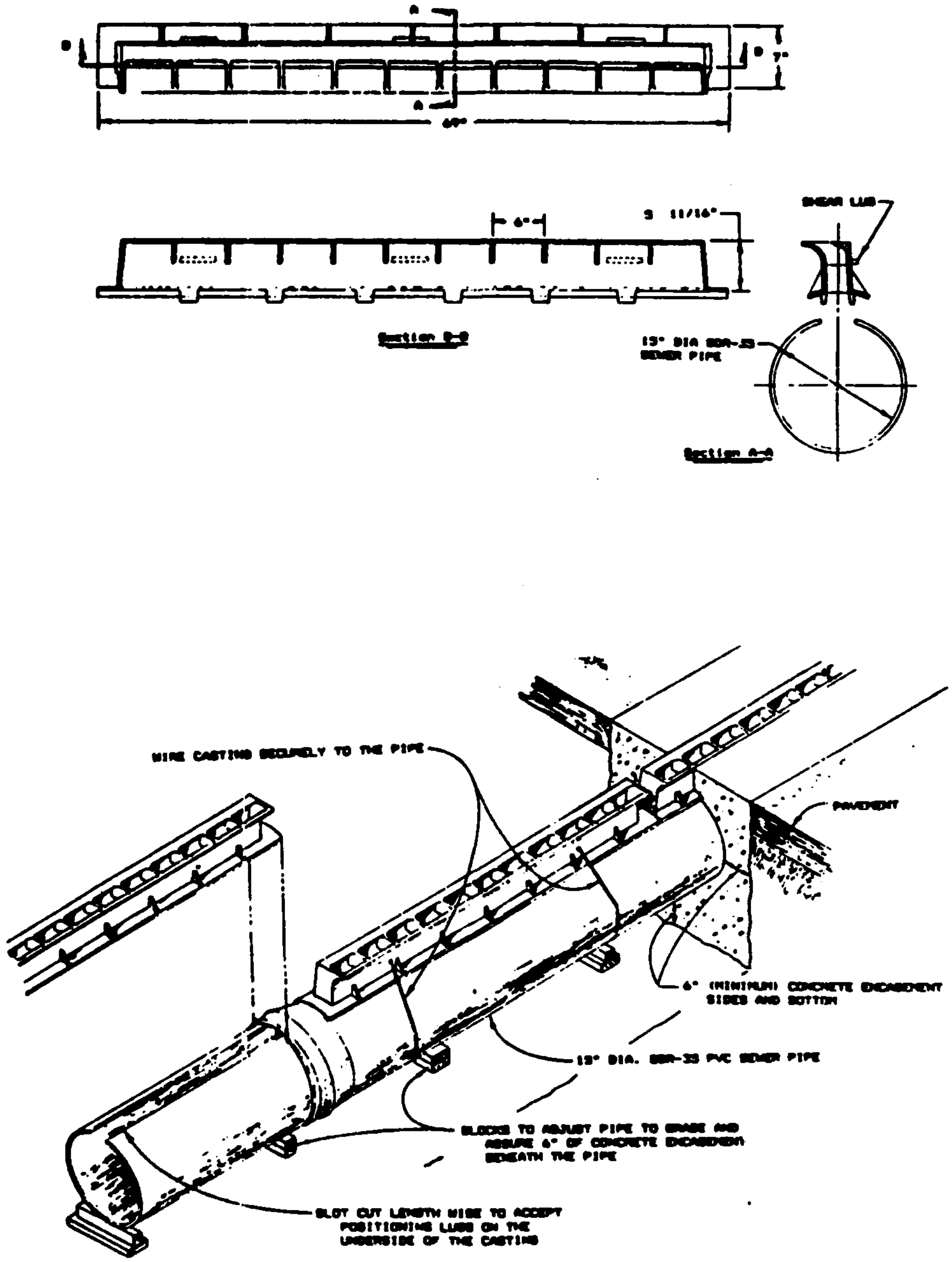


Figure 3.8 Slotted Grate and Collector Pipe (Ref. 2).

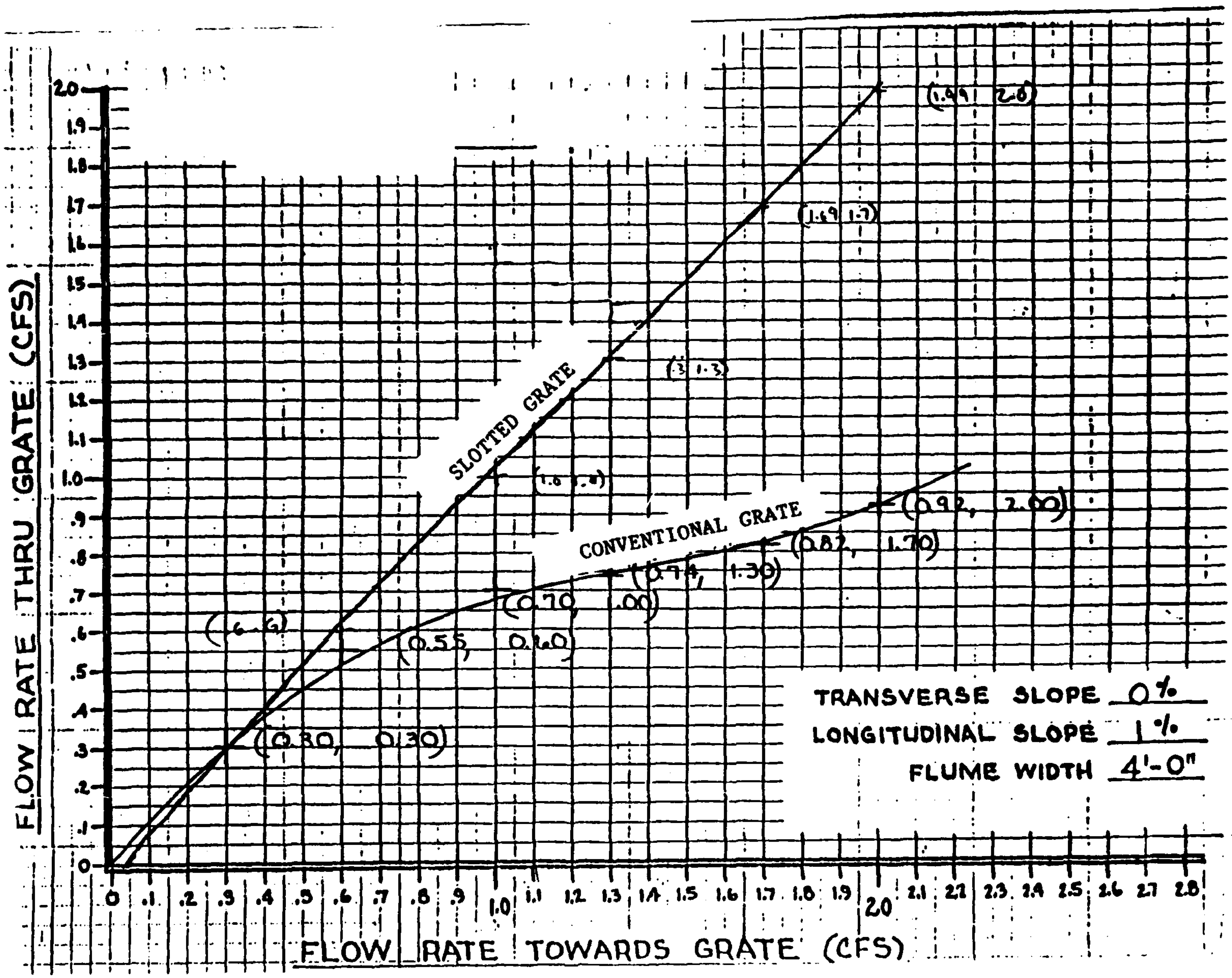


Figure 3.9 Comparison Between Slotted Grate and Conventional Grate for 1% Longitudinal Slope (Ref. 2).

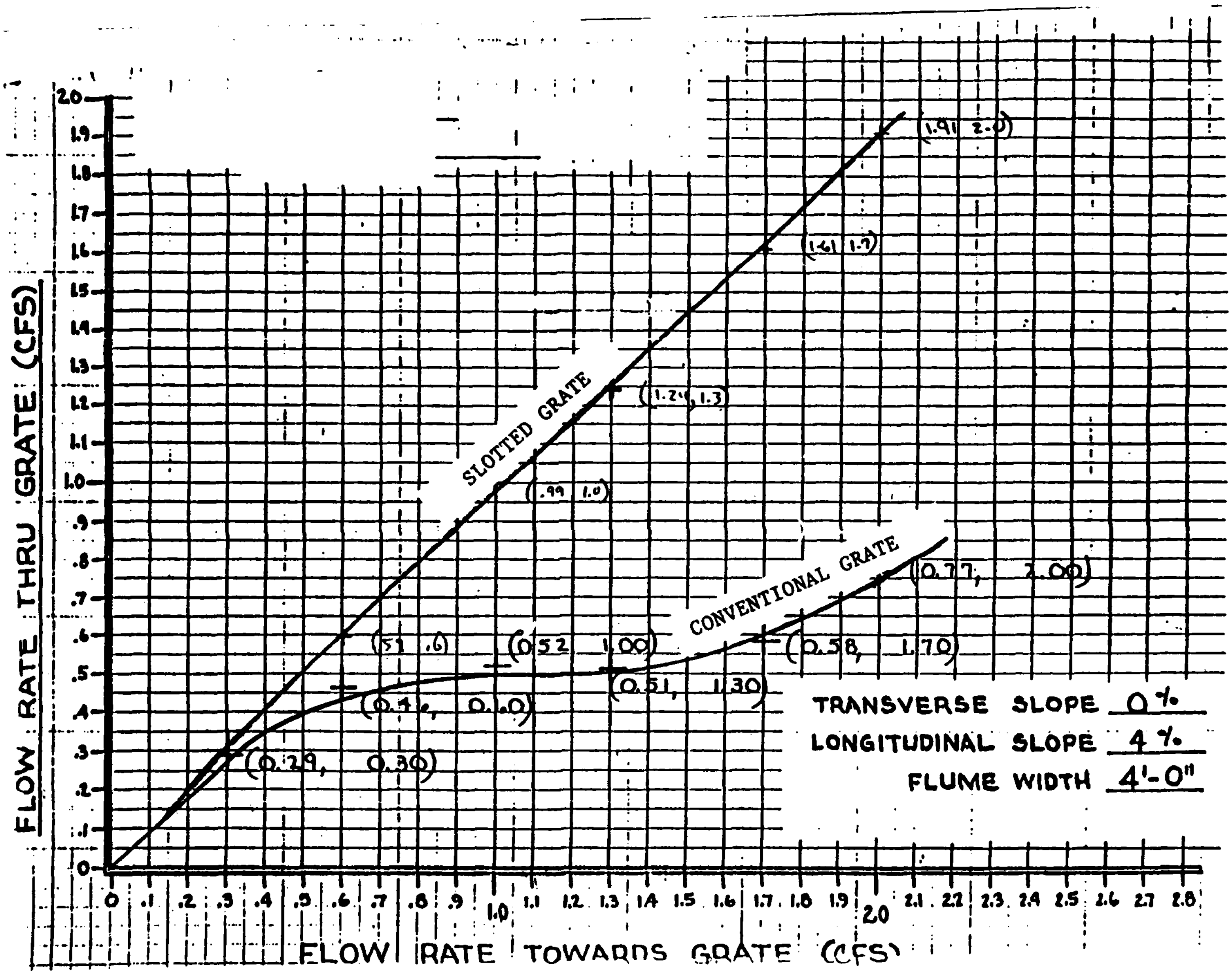


Figure 3.10 Comparison Between Slotted Grate and Conventional Grate for 4% Longitudinal Slope (Ref. 2).

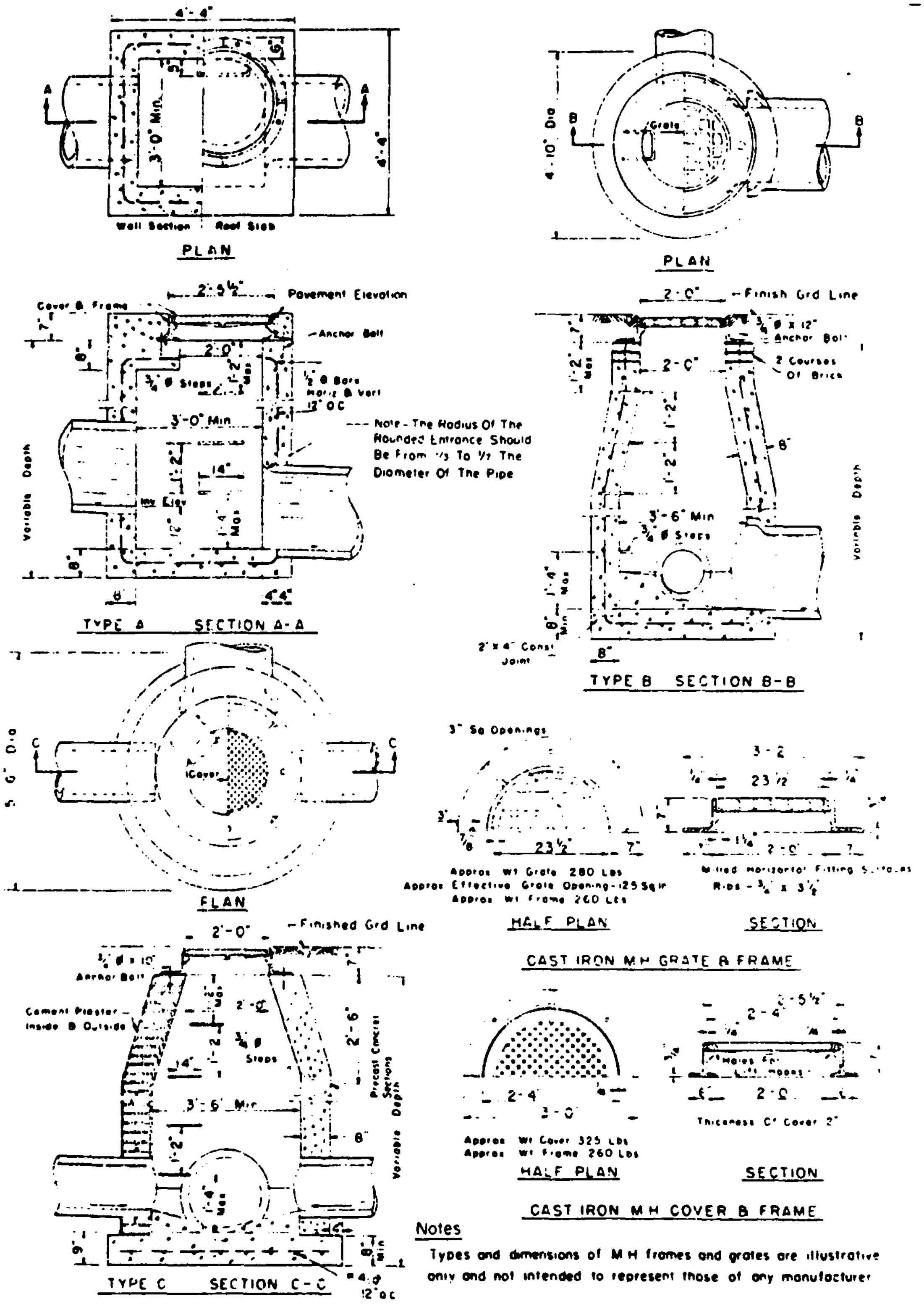


Figure 3.11 Typical Design Details for Manholes (Ref. 1).

CHAPTER 4

SUBSURFACE DRAINAGE

4.1 WATER INFILTRATION

4.1.1 Introduction

Airport pavements, like highway pavements, are very susceptible to the damaging effects of water. Jointed concrete pavements especially have trouble with water infiltration into the pavement structural section. Joint seals do not last very long, and unless the joints are regularly resealed, increasing amounts of water are allowed to enter the pavement structure. Extensive studies have been performed on water infiltration into pavements. Barenberg and Thompson (1), Ridgeway (2), Ring (3), Barksdale and Hicks (4), Dempsey et al (5), and Dempsey and Robnett (6) all have performed studies on the problems of water infiltration through cracks and joints in highway pavement systems. These study findings can also be carried over into the airfield pavement area. Fowler (7) provided recommended modifications to the FAA Advisory Circular on Airport Drainage which addressed the problem of subdrainage in the pavement structural section.

Less traffic on airfield pavements does not mean that less damage occurs. The same distresses occur on airport pavements as on highway pavements (pumping, D-cracking, frost heave, faulting, etc.). Carpenter et al (8,9) thoroughly discussed water-related distresses, or what they called Moisture Accelerated Distress (MAD). Once again, their discussion, which pertained to highway pavements, can be useful in airport pavement drainage analysis.

The damaging effects of water can be controlled if: (1) the water is kept out of the pavement structure, (2) the pavement materials are insensitive to

water, or (3) water which infiltrates into the pavement structure is effectively removed by drainage methods. Since it is very difficult to keep water from entering the pavement structure and to utilize materials insensitive to water, the latter of the three choices becomes very important in protecting the pavement from the distresses caused by water (8,9).

4.1.2 Sources of Water Inflow

One of the first steps in designing a drainage system is to determine the quantities of water the system will have to remove. This includes determining the inflow rates of water from various origins. Usually, the major source of water inflow is surface infiltration. However, there are other sources of inflow such as upward seepage from underlying groundwater and springs, capillary water from the watertable (usually minor), and water of hydrogenesis, which is usually negligible (10,11). All noticeable sources of inflow should be accounted for in the total inflow rate.

Surface infiltration is often the major source of water that enters the pavement structure. The amount of water infiltrating from the surface is controlled by either the design precipitation rate or the amount allowed into the pavement by the permeability of the surface course (including joints and cracks), whichever is smaller. The Federal Highway Administration Guidelines (11) states that for concrete pavement surfaces, the amount of water entering the pavement structure through the surface course (design infiltration rate) should be between 0.5 and 0.67 times the design precipitation rate. For asphalt concrete surfaces, the design infiltration rate is between 0.33 and 0.5 times the design precipitation rate. Ridgeway (2) suggests an infiltration rate of 0.11 ft^3/hr. per linear foot of crack for design purposes on asphalt concrete pavements. For cracks and joints in Portland Cement Concrete pavements, he suggests an infiltration rate of 0.03 ft^3/hr.

per linear foot of crack or joint. If data for joints and cracks are not provided, the alternative is to use procedures such as those provided by Dempsey and Robnett (6) which correlate pipe volume outflow and precipitation volume through the use of regression analysis. Nonetheless, more studies are needed in this area since the water infiltration rate into a pavement depends on many variables.

Any groundwater which penetrates into the pavement structure is added to the surface infiltration. These two sources are a major part of the inflows into the pavement. When upward groundwater seepage is expected to enter the pavement structure, Darcy's law can be applied to obtain the rate of inflow per square foot of drainage layer. Frost action water can sometimes be a part of upward groundwater seepage (11). The probable hydraulic gradient is needed and the best possible estimate of the subgrade permeability should be made. Flow nets, Figure 4.1, can be used to calculate the hydraulic gradient which is useful for determining inflow rates (12). By Darcy's law, the rate of inflow per square foot of drainage layer is the product of the hydraulic gradient and the subgrade permeability. Inflow rates can be taken off of Figure 4.2 for a range of hydraulic gradients and subgrade permeabilities. Some examples for estimating inflow rates for vertical seepage into the drainage layer and trench drains can be found in Cedergren (10).

4.2 COMPONENTS OF SUBSURFACE DRAINAGE SYSTEM

4.2.1 Outflow

Once the water has found its way into the structural section of the pavement, it should be rapidly drained. If the water remains in the pavement structure for extended amounts of time, the damaging effects of water will

begin to develop. Distresses such as pumping, faulting, frost heave, and others occur when there is a saturated base. Traffic test data from Barenberg and Thompson (1) showed that rate of damage with excess water present was 100 to 200 times greater than that without excess water. If there is no subsurface drainage or if the existing subdrainage system present in the pavement structure is inadequate, the drainage layer or base can remain saturated for extended periods of time.

There are several ways in which water can escape from the pavement structure:

1. surface evaporation,
2. loss by lateral seepage,
3. loss by subgrade percolation
4. loss through cracks and joints (bleeding and pumping), and
5. removal by subsurface drainage system.

The first four processes listed above are generally slow and do not contribute much to the drainage of water that has entered the pavement structure. If there is a dependence on any or all of these four processes for the drainage of water, the base will probably be saturated for weeks or even months after a significant rainfall.

If water is to be properly and quickly removed from the pavement structure, a subsurface drainage system is most likely required. This is especially true for airport pavements where runway half widths are typically 75 ft to 100 ft and rapid drainage of excess water in the pavement structure is necessary. Rapid drainage of water is very important for airport pavements in colder climates where frost action is present to significant depths (11). It is essential that the water be removed quickly in these regions so that the water does not have enough time to freeze while in the pavement structure.

A typical subsurface drainage system with all of the necessary components is shown in Figure 4.3. The four components of the system are:

1. an opened graded base drainage layer, which incorporates a subbase or filter layer (possibly a fabric) over the subgrade to protect the base from infiltrating subgrade particles,
2. an edge drain and possible intercept drain,
3. outlet pipes, and
4. outlet markers and protection of pipes from damage.

The continuity of the water as it flows through the drainage system can also be seen in Figure 4.3. The water flows along the path A-B-C-D-E-F. The water first enters the pavement structure at A (a joint or crack) and flows to B, the surface course-base interface. It then flows to C, an interior point of the base drainage layer, on the way to D, the edge drain. The water then flows to E, the entrance to the the outlet pipe, and from there to F, where the water is disposed of properly. Thus, there are basically five segments of water flow through the drainage system, A-B, B-C, C-D, D-E, and E-F. As the water flows through these segments each segment should have a higher discharge capacity than the preceding segment to prevent any bottleneck effect occuring in the drainage system. For example, segment E-F, the outlet pipe, should have a higher discharge capacity than segment D-E, the edge drain.

These components can be used alone or in different combination to provide the drainage capacity needed. A short description of each drain and its uses has been described by Moulton (11) and will be summarized in the following paragraphs.

4.2.2 Longitudinal Edge Drains

Longitudinal drains are placed parallel to the pavement centerline in both the horizontal and vertical alignments. This type of drain consists of a trench with a perforated collector pipe surrounded by a protective filter. These drains are usually placed under the pavement edge joints where most water infiltrates the pavement; but, on wide pavements such as runways they might also be placed at the center and intermediate points to draw down the water table. In a cut slope a series of parallel drains may be used to lower the level of the water table.

4.2.3 Transverse Drains

Transverse and horizontal drains run laterally beneath the pavement either perpendicular to the centerline or skewed in a herringbone pattern. These drains are often used at pavement joints to drain infiltration and groundwater in bases and subbases. These drains are especially useful when the flow is in the longitudinal direction for they intercept the water and remove it from beneath the pavement. Most of the time these drains consist of a trench, a collector pipe, and a protective filter. A shallow trench filled with open graded aggregate can be used but the drainage provided will not be as good as that when a pipe is provided (11). In highway design horizontal drains can cause problems in areas which are susceptible to frost heave because the pavement heaves everywhere but at the drains and creates a rough surface. The general placement of both longitudinal and transverse drains is shown in Figure 4.4.

Horizontal drains are used when an underground spring threatens the stability of a cut or fill. Pipes are drilled into the side slopes to tap the spring and relieve pore water pressure. Usually these pipes drain directly into a drainage ditch which takes the water away from the pavement.

4.2.4 Drainage Blankets

A drainage blanket is a very permeable layer whose width and length in the direction of flow is large relative to its thickness. Drainage blankets can be used beneath the pavement structure or as an integral part of the pavement. Although base and subbase courses often consist of permeable granular material they will not act as drainage blankets unless designed to do so. Drainage blankets must have an adequate thickness of material with a very high coefficient of permeability (in the range of 1000 ft/day to 20,000 ft/day) and an outlet for the collected water. Filter layers sometimes need to be placed around the drainage blanket to prevent fines from the other layers to cause clogging. Typical permeability values for filter material range from about 10 ft/day to 100 ft/day. When properly designed, drainage blankets can be used to control both groundwater and infiltration. Drainage blankets can be used to prevent seepage from the surface of cut slopes and sidehill fills by controlling the flow of groundwater.

4.2.5 Vertical Well System

Vertical well systems, shown in Figure 4.5, are used to control groundwater and to relieve pore water pressures. These wells can be pumped to lower the water table during construction or they can be left to overflow for the relief of pore water pressures. They can be used to promote accelerated drainage of soft and compressible soils which are being consolidated under surcharge loading.

4.3 DESIGN OF DRAINAGE LAYERS

4.3.1 Permeability

The first essential component of a subsurface drainage system is the base course drainage layer. The outflow capabilities of the layer are very important. The discharge capacity should be substantially greater than the inflow rate of water into the layer to ensure a safety factor and thus, continuity of flow through the drainage system. Therefore, the aggregate used for the drainage layer should have a high permeability, with k values ranging from 1000 ft/day to greater than 20,000 ft/day. Studies indicate that for more drainability, a more uniformly graded aggregate (or open graded) is desired for the drainage layer. It is incorrect to assume that well graded blends of sand and gravel will provide beneficial drainage when used for a base course. An open graded aggregate has a much higher permeability than a well graded blend. Figure 4.6 shows that an open graded base course has the potential for removing a much greater amount of water inflow than a standard base course. According to Cedergren (10) the open graded aggregate can replace the normally used dense graded materials on an inch-for-inch basis. A main problem in using an open graded base course is that it does not make for a very stable working platform during construction. However the stability problem can be overcome by stabilizing the open graded base with a low percentage of cement or bituminous material.

Some gradation ranges that yield high permeabilities for open graded aggregates are 3/8 in. to No. 4, 3/4 in. to No. 4, 1 in. to 1/2 in., among others. To ensure that the particle sizes in the open graded aggregate are restricted to a narrow range, the 85 percent size (finer) of the aggregate should be less than 4 times the 15 percent size (finer), or $D(85) < 4D(15)$

(4). Also, to restrict the amount of fines in the aggregate, the 2 percent size should be greater than or equal to 0.1 in. in diameter, or $D(2) \geq 0.1$ in. If these constraints are satisfied, the permeability of the aggregate should be adequate.

Various charts and nomographs are available which are helpful in obtaining the permeability of an aggregate or to find out how the aggregate performs. A rough estimate of the permeability of an open graded material can be obtained from Figure 4.7 where the coefficient of permeability is related to the 15 percent size (D15) of the aggregate (13). Also, Moulton (11) provides a nomograph from which the permeability of a granular material can be obtained as shown in Figure 4.8. The product of the layer thickness, t, and the permeability, k, is known as the transmissibility of the drainage layer. This is a measure of the drainage capability of the layer per linear foot. The drainage layer must have a certain transmissibility to remove the net inflow. The net inflow is the sum of the inflows from all sources minus the outflow which naturally occurs. Figure 4.9 shows the relationship between k and t for various transmissibilities (10). Figure 4.10 can also be helpful in determining the thickness of open graded drainage layers for certain values for permeability and water inflow rate. Moulton also discusses thicknesses of drainage layers (11).

Another important factor to consider in the design or analysis of a drainage layer is the time required for the water to flow out of the layer. As mentioned earlier, this is especially crucial in cold regions where water can freeze while still in the base course. Long drainage paths on airfield pavements require long periods of time to drain and this problem has to be approached and analyzed.

After a significant rainfall occurs, it is quite probable that the base course will be saturated. The sooner there is a decrease in the saturation level, the better the pavement performance. For base drainage layers with collector pipes at the lower edges (edge drains), a formula developed by the U.S. Army Corps of Engineers is applicable:

$$t_{50} = \frac{n_e\ D^2}{2880k\ H_o} \qquad \text{(Eq. 4.1)}$$

where:

t_{50} = time for 50% drainage of a sloping base course with a drain at its lower edge,

n_e = effective porosity of the base course,

D = sloping width (ft),

k = coefficient of permeability of the base (ft/min),

H_o = H + sD where H is the thickness of the subbase (ft), and

s = cross slope.

Figure 4.11 is a graph of the solution of this formula for s = 0.01 and n_e = 0.30. Another graph, Figure 4.12, shows the relationship between the permeability of a drainage layer and the sloping width, w, for a maximum drainage time of 2 hours, given a certain cross slope and thickness of the drainage layer.

Casagrande and Shannon (14) performed theoretical analysis of base course drainage for one half of the cross section of the base course. In Figure 4.13 the analysis was divided up into two stages. The first stage shows the free surface as it gradually changes from CD to CA because of drainage through the free edge CD. In the second stage, the free surface rotates from CA to CB because of the loss of water through the face CD. An impervious subgrade is used throughout the entire flow calculations and the phreatic

surface is assumed to be a straight line. Casagrande and Shannon's equations are represented in the form of a chart by Barber and Sawyer (15) in Figure 4.14. Liu and Lytton (16) discussed Casagrande and Shannon's analysis and compared it with the Texas Transportation Institute's (TTI) model for base course drainage with an impermeable subgrade, Figure 4.15. The TTI model, however, uses a parabolic phreatic surface, Figure 4.16. Liu and Lytton (16) also compared the two models for permeable subgrades.

The Liu and Lytton (16) report describes all of the inputs to a computer simulation model for rainfall and drainage analysis. Among the inputs into the model are the physical features of the pavement system (length, height, slope, permeability), base course drainage computations using the TTI model, rainfall data and analysis, time of drainage, water infiltration, degree of saturation, elastic moduli of base course and subgrade, and other inputs. The output from the simulation model is the effect that rainfall has on the load carrying capacity of the pavement. It is a very complete model and makes use of many different subprogram models. A flow chart and a complete program and output printout is included in the report (16).

4.3.2 Filters

Filters are used to prevent loss of permeability in the drainage layer as a result of clogging. If fine soil is allowed to enter the drainage layer, the permeability of the drainage layer and the water removing capability will substantially be decreased. Often the gradation of the drainage layer does not satisfy certain filter criteria required to keep fines out of the layer. To prevent the infiltration of fines, filters are placed between the drainage layer and the underlying soil. Two types of filters commonly used in subsurface drainage are granular filters and geotextile filters.

Granular filters consist of a layer of granular material with the proper gradation to keep fines from working into the protected material. Figure 4.17 shows an example of the gradations of a subbase (drainage layer), the filter layer, and the native soil. The granular material in the filter layer must satisfy numerous gradation criteria which have been developed to guide the design of granular filters (13,17,18,19). The following criteria are used in the Highway Subdrainage Design Manual by Moulton (11):

$$D_{15} \text{ filter} \leq 5 \ D_{85} \text{ protected soil} \qquad \text{(Eq. 4.2)}$$

$$D_{15} \text{ filter} \geq 5 \ D_{15} \text{ protected soil} \qquad \text{(Eq. 4.3)}$$

$$D_{50} \text{ filter} \leq 25 \ D_{50} \text{ protected soil} \qquad \text{(Eq. 4.4)}$$

$$D_{5} \text{ filter} \geq 0.074 \text{ mm} \qquad \text{(Eq. 4.5)}$$

$$C_u \text{ filter} = \frac{D_{60} \text{ filter}}{D_{10} \text{ filter}} \leq 2 \qquad \text{(Eq. 4.6)}$$

If the fine material is uniformly graded then Eq. 4.2 should be the 15% size of the filter is less than or equal to 4 times the 85% size of the protected soil. The third criteria can be waived if the protected soil is a medium to high plasticity clay. When the soil to be protected contains a coarser fraction, the design should be based on the material which is finer than 1 in. in size.

Choosing the appropriate geotextile filter is more difficult. The properties which control soil retention and permeability are opening size and shape, permeability, structural rigidity, thickness, compressibility, and porosity. The following criteria are recommended for determining opening size to permit drainage and prevent clogging of geotextile filters in granular material with less than 50% by weight of fines:

$$\frac{\text{85\% passing size of soil}}{\text{opening size of EOS sieve}} \geq 1 \qquad \text{(Eq. 4.7)}$$

where:

EOS = equivalent opening size.

Geotextile filters should be used with caution for soils with 85% or more finer than the No. 200 sieve.

A Canadian materials laboratory recommends a geotextile filter to soil permeability ratio of 2 if the material is uniformly graded and 5 if it is well graded (11). A standard method of testing geotextile filter permeability has been developed in ASTM D4491-85 (20). Geotextile permeability is difficult to measure. When the geotextile becomes dirty, clogged with fines, or old and deteriorated the permeability changes. Janssen (21) has developed a dynamic permeability test which attempts to duplicate actual field conditions. In this test the parameters of loading, soil type, and hydraulic gradient can be varied to attain different field conditions.

The durability of the geotextile filter should be considered where it will be exposed to alkali or acidic soils, fuels, etc. Plastics can not be used where they will be exposed to ultraviolet rays or sunlight unless properly treated. When the material will be subjected to severity of service or harsh construction practices, its resistance to tear, puncture, burst, and tensile stresses must be considered. Ladd's (22) study included prototype tests as well as field investigations of geotextile filter fabrics. In recent years, many types of geotextile filter fabrics have been used for the protection of the drainage layer (10,23,24,25,26,27,28). Geotextile filters are not only used to protect the drainage layers, they are also used to protect the longitudinal edge drain from intrusion of fine soil which could

eventually work its way into the collector pipe and restrict its outflow capabilities. For examples of the placement of geotextile filters in edge drains see Figures 4.18 and 4.19.

4.4 ANALYSIS AND DESIGN OF SUBDRAINAGE SYSTEMS

4.4.1 General

There are some general rules for selecting and placing the perforated or slotted pipe used in longitudinal and transverse subdrainage systems. When choosing the types and sizes of pipe to be used soil conditions, load requirements, durability, and environmental conditions should be considered. ASTM and AASHTO specifications and the manufacturers recommendations should be used to determine which types and sizes of pipe are sufficient.

A few of the different types of pipes that have been used for airport subdrainage are as follows (29):

1. Perforated metal, concrete, or vitrified clay pipe. The joints are sealed and the perforations usually extend over roughly one third of the pipe's circumference. The perforated area is generally placed next to the soil.
2. Bell-and-spigot pipes are placed with the joints open. These types of pipes are usually made from vitrified clay, cast iron, and plain concrete.
3. Porous concrete pipes which collect water by seepage through their walls. The joints are sealed.
4. Plastic pipes.

Recently, attention has been given to the use of plastic pipe in airport drainage systems. However, there is still reluctance to use plastic pipe

because of its lack of use in the field as well as its lack of standardized design, construction, and maintenance guidelines. In his report for the FAA, Harvey (30) presented a set of tentative recommended technical requirements and guidelines for plastic pipes that were based on a synthesis of information drawn from an extensive literature review, site inspections, and personal contacts with people in the drainage and plastic pipe industries. Also, Horn (31) reported on results of tests conducted on different types and sizes of plastic pipe which were installed under a circular test track. Resulting data included total pipe deflection at the conclusion of the testing, as well as deflections due to static loads, and a summary of dynamic load response of the pipes. Three types of plastic pipe were used in the field tests: (1) polyvinyl chloride(PVC), (2) polyethylene(PE),and (3) accrylonitrile-butadiene-styrene(ABS). Several different diameters were tested for each type of pipe.

The slope of the pipe is usually determined by the grade of the pavement with the pipe set at a constant depth beneath the surface. The minimum slope should not be less than 1% for smooth bore pipes and 2% for corrugated pipes. Sometimes a steeper gradient than the pavement grade is used to reduce the pipe size needed. The minimum pipe diameters recommended are 3 in. for PVC pipes and 4 in. for all other types. Typically, a 6-in. maximum diameter pipe is adequate unless extreme groundwater conditions are present.

An important factor to consider when selecting a certain type or size of pipe is the minimum depth of cover needed. Larger pipes need greater depths of cover and plastic pipes need larger depths of cover than concrete or other pipes. Horonjeff and McKelvey (29) recommend some minimum cover depths for various types and sizes of pipes. Smoothness of the pipe is also important.

A smaller diameter pipe with a smaller coefficient of roughness can have an outflow capability equal to or greater than that of a larger diameter and rougher pipe.

4.4.2 Longitudinal Drainage Systems

The position of longitudinal drainage systems is based on the depth of frost penetration and whether or not shoulder drainage is needed. When there is no significant frost penetration, the drainage pipe can be placed in shallow trenches, as shown in Figure 4.20 (11). If there is frost action, the trench should be deep enough so the drainage pipe is not frozen most of the time. Figure 4.21 shows some typical deep longitudinal drains. If drainage of the shoulder is not desired, the drainage system is placed at the joint between the pavement and the shoulder. This serves to prevent pumping by removing the water which collects at the joint along the pavement edge. A drainage pipe placed at the outside edge of the shoulder will drain the shoulder as well as the pavement. This should be considered carefully as it can significantly increase the length of the flow path in the granular material and may require added drainage layer thickness. Figure 4.22 is a nomograph relating the quantity of water, q_d, distance between outlets, Lo, and Manning's roughness coefficient, n_f, to pipe diameter and spacings. A filter may be needed to keep fines from moving into the trench backfill. If a shallow trench is used, the filter layer under the drainage blanket may be extended into the collector. A deep trench can be lined with a geotextile filter before the trench is backfilled, Figure 4.23, or filter aggregate can be placed just around the perforated pipe to prevent fines from entering, Figure 4.24 (11). Cedergren (10) suggests that collector pipes should be

laid with the openings down on compacted bedding material so that fines will be less likely to enter the pipe.

4.4.3 Transverse Drainage Systems

There are no set rules for positioning transverse drainage systems. Trial locations should be chosen to select flow path lengths which will maintain fairly consistent drainage layer thicknesses. When the longitudinal grade is steep relative to the cross slope, more closely spaced transverse drains are needed to remove the water. Transverse drains should also be placed at critical locations such as grade changes and the transition area before a superelevation. Rules for minimum pipe size and gradient, adequate depth to minimize freezing effects, and filter protection are the same as for longitudinal collectors.

4.4.4 Outlets

Outlets carry the water from the collectors out of the pavement, usually to a ditch, although some outlets are discharged directly into the sewer system. The oulet pipe is not perforated like the collectors. It is placed in a trench and backfilled with low permeability soil. This backfill material is to prevent piping around the outlet pipe. A device such as a "cutoff collar" may be used if this material is not available.

The location of outlets is controlled by the topographic and geometric features of the pavement, as well as the availability of good outlet points. The outflow should be free, unobstructed, and designed so as to preclude soil erosion and other possible drainage problems downstream. The spacing of outlets controls the size of longitudinal pipe needed so it should be considered carefully.

Screens are put on the pipe to prevent small animals or birds from entering the pipe to nest or deposit debris.

If the water level in the ditch is sometimes above the drain so that it is submerged, valves can be used to prevent backflow.

Outlet markers are necessary so that the outlets can be maintained. These markers should extend from 24 in. to 30 in. above the outlet.

REFERENCES

1. Barenberg, E.J., and Thompson, O.O., "Behavior and Performance of Flexible Pavements Evaluated in the University of Illinois Pavement Test Track," Highway Engineering Series No. 36, Illinois Cooperative Highway Research Program Series No.108, 1970.
2. Ridgeway, H.H., "Infiltration of Water Through the Pavement Surface," TRB, Transportation Research Record 616, Transportation Research Board, Washington, D.C. 1976.
3. Ring, G.W., "Drainage of Concrete Pavement Structures," Proceedings of International Conference on Concrete Pavement Design, Purdue University, West Lafayette, Indiana, 1977.
4. Barksdale, R.D., and Hicks, R.G., "Drainage Considerations to Minimize Distress at the Pavement-Shoulder Joint," Proceedings of International Conference on Concrete Pavement Design, Purdue University, West Lafayette, Indiana, 1977.
5. Dempsey, B.J., Darter, M.I., and Carpenter, S.H., "Improving Subdrainage and Shoulders of Existing Pavement-State-of-the-Art," Report No. FHWA/RD-81/077, Federal Highway Administration, Washington, D.C., 1982.
6. Dempsey, B.J., and Robnett, Q.L., "Influence of Precipitation, Joints, and Sealing on Pavement Drainage," Transportation Research Record 705, Transportation Research Board, Washington, D.C., 1975.
7. Fowler, J., "Improvements to Airport Drainage Criteria, Phase I," Report No. FAA-RD-76-59, U. S. Department of Transportation, Federal Aviation Administration, Washington, D. C., 1976.

8. Carpenter, S.H., Darter, M.I., Dempsey, B.J., and Herrin, S., "A Pavement Moisture Accelerated Distress (MAD) Identification System," Vol. 1, Report No. FHWA-RD-81/079, Federal Highway Administration, Washington, D.C., 1981.

9. Carpenter, S.H., Darter, M.I., and Dempsey, B.J.,"A Pavement Moisture Accelerated Distress (MAD) Identification System," Vol. 2, Report No. FHWA-RD-81/080, Federal Highway Administration, Washington, D.C., 1981.

10. Cedergren, H.R., "Drainage of Highway and Airfield Pavements," Wiley, New York, 1974.

11. Moulton, L.K., "Highway Subdrainage Design," FHWA Report, TS-80-224, Federal Highway Administration, Washington, D.C., 1980.

12. Cedergren, H.R., "Seepage, Drainage, and Flow Nets," Wiley, New York, 1967.

13. Bertram, G.E., "An Experimental Investigation of Protective Filters," Publication No. 267, Graduate School of Engineering, Harvard University, 1940.

14. Casagrande, A., and Shannon, W.L., "Base Course Drainage for Airport Pavement," Proceedings of the American Society of Civil Engineers, Vol. 77, 1951.

15. Barber, E.S., and Sawyer, C.L., "Highway Subdrainage," Public Roads, Vol. 26, February, 1952.

16. Liu, S.J., and Lytton, R.L., "A Simulation Model for Rainfall Infiltration, Drainage Analysis, and Load Carrying Capacity of Pavements," FHWA Report, DTFH-61-80-C-0013, Federal Highway Administration, Washington, D.C., 1984.

17. Corps of Engineers, U.S. Army, "Investigation of Filter Requirements for Underdrains," Technical Memorandum No. 183-1, Waterways Experiment Station, Vicksburg, Mississippi, 1941.

18. Karpoff, K.P., "The Use of Laboratory Tests to Develop Design Criteria for Protective Filters," Proceedings, Vol. 55, American Society for Testing Materials, Philadelphia, PA., 1955.

19. United States Department of Agriculture, Soil Conservation Service, "Tentative Guides for Determining the Gradation of Filter Materials," Soil Mechanics Note No. 1, May 1, 1968.

20. ASTM Standards on Geotextiles, ASTM Committee D-35, Philadelphia, PA, 1988.

21. Janssen, D.J., "A Dynamic Test to Predict the Field Behavior of Filter Fabrics Used in Pavement Subdrains," Transportation Research Record 916, Transportation Research Board, Washington, D.C., 1983.

22. Ladd, D.M., "A Review of Fabric Usage in Pavements Constructed on Low-Strength Soils," FAA-RD-79-93 U.S. Department of Transportation, Federal Aviation Administration, Washington, D.C., 1979.

23. Bell, J.R., Hicks, R.G., et al, "Evaluation of Test Methods and Use Criteria for Geotechnical Fabrics in Highway Applications," Interim Report, Prepared for the Federal Highway Administration by the Department of Civil Engineering and Research Institute, Oregon State University, 1979.

24. Calhoun, C.C., Jr., "Development of Design Criteria and Acceptance Specifications for Plastic Filter Cloths," Technical Report S-72-7, U.S. Army Waterways Experiment Station, Vicksburg, Mississippi, 1972.

25. Calhoun, C.C., Jr., Compton, J.R. and Strohm, W.E., "Performance of Plastic Filter Cloths as a Replacement for Granular Filter Materials," Highway Research Record No. 373, Highway Research Board, Washington, D.C., 1971.

26. Marks, B.D., "The Behavior of Aggregate and Fabric Filters in Subdrainage Applications," University of Tennessee, Knoxville, Tennessee, 1975.

27. Steward, J., Williamson, R. and Mohney, J., "Guidelines for Use of Fabrics in Construction and Maintenance of Low Volume Roads," Report No. FHWA-TS-78-205, Federal Highway Administration, Washington, D.C., 1977.

28. Miller, S.P., "Filter Fabrics for Airport Drainage," FAA-RD-79-29, U.S. Department of Transportation, Federal Aviation Administration, Washington, D.C., 1979.

29. Horonjeff, R. and McKelvey, F.X., "Planning and Design of Airports," New York, McGraw-Hill, 3rd ed., 1983.

30. Harvey, G.G., "Plastic Pipe in Airport Drainage Systems," FAA-RD-77-38, U.S. Department of Transportation, Federal Aviation Administration, Washington, D.C., 1977.

31. Horn, W.J., "Field Tests of Plastic Pipe for Airport Drainage Systems," FAA-RD-79-86, U.S. Department of Transportation, Federal Aviation Administration, Washington, D.C., 1979.

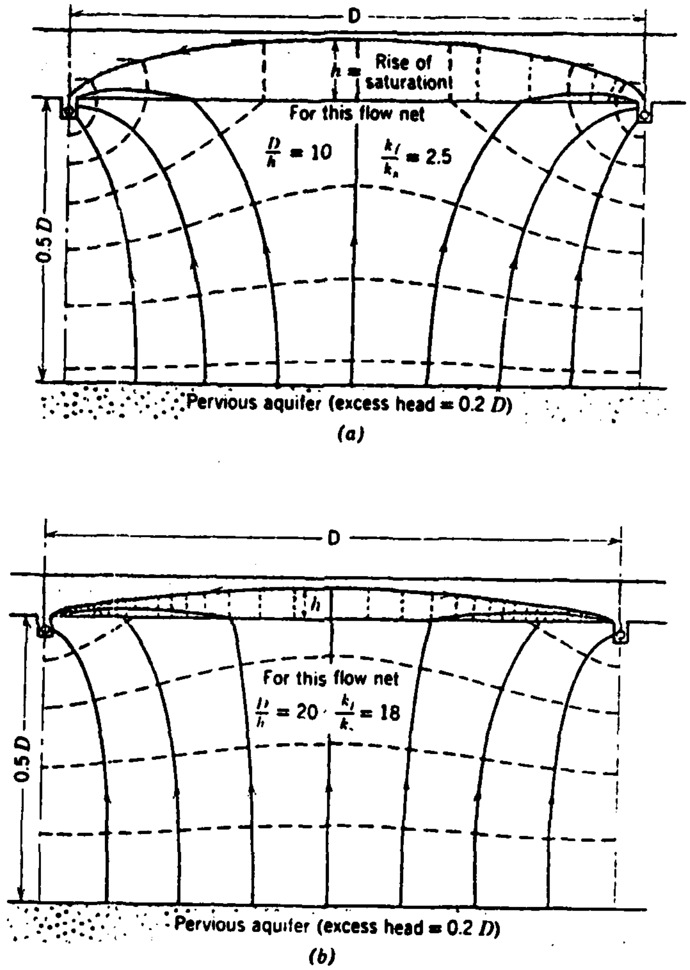


Figure 4.1 Typical Flow Nets for Vertical Seepage into Horizontal Drainage Blankets from Underlying Aquifer (Ref. 12).

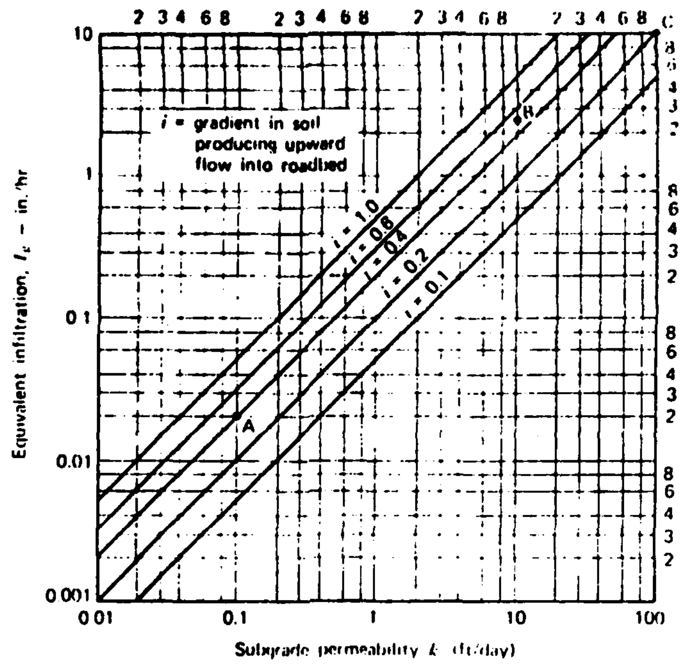


Figure 4.2 Chart for Vertical Groundwater Seepage into Horizontal Drainage Blankets from Underlying Artesian Aquifer (Ref. 10).

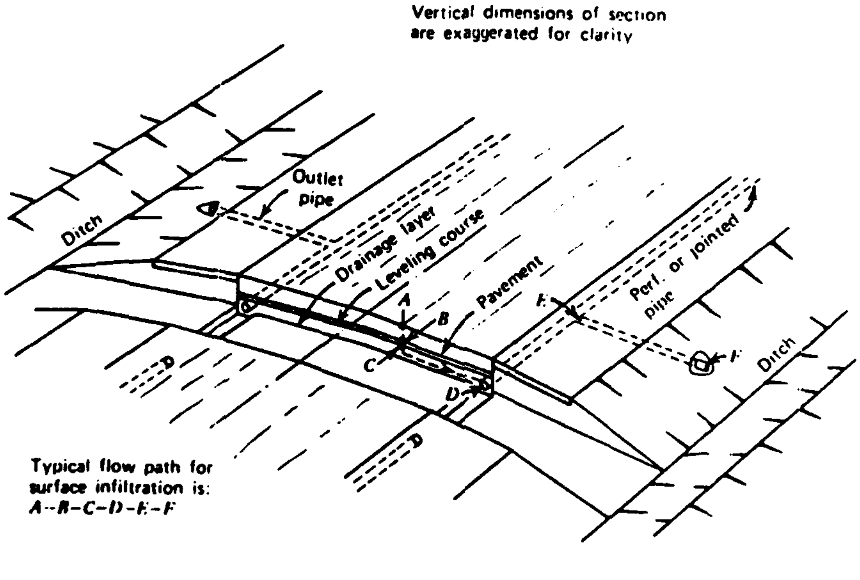


Figure 4.3 Illustration of Flow Path for Condition of Continuity in Pavement Drainage of Surface Infiltration (Ref. 10).

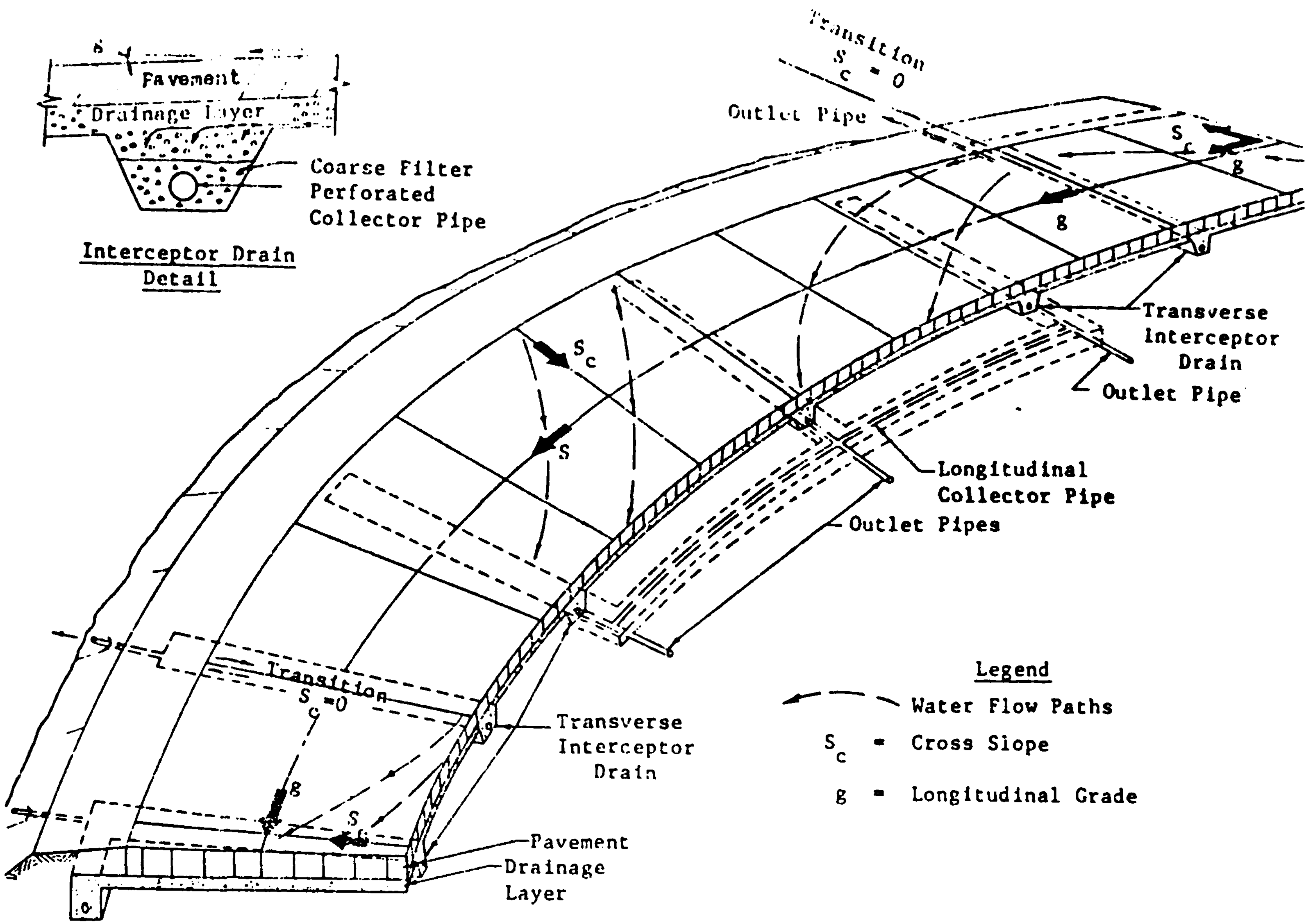


Figure 4.4 Placement of Longitudinal and Transverse Subdrains in a Pavement System (Ref. 11).

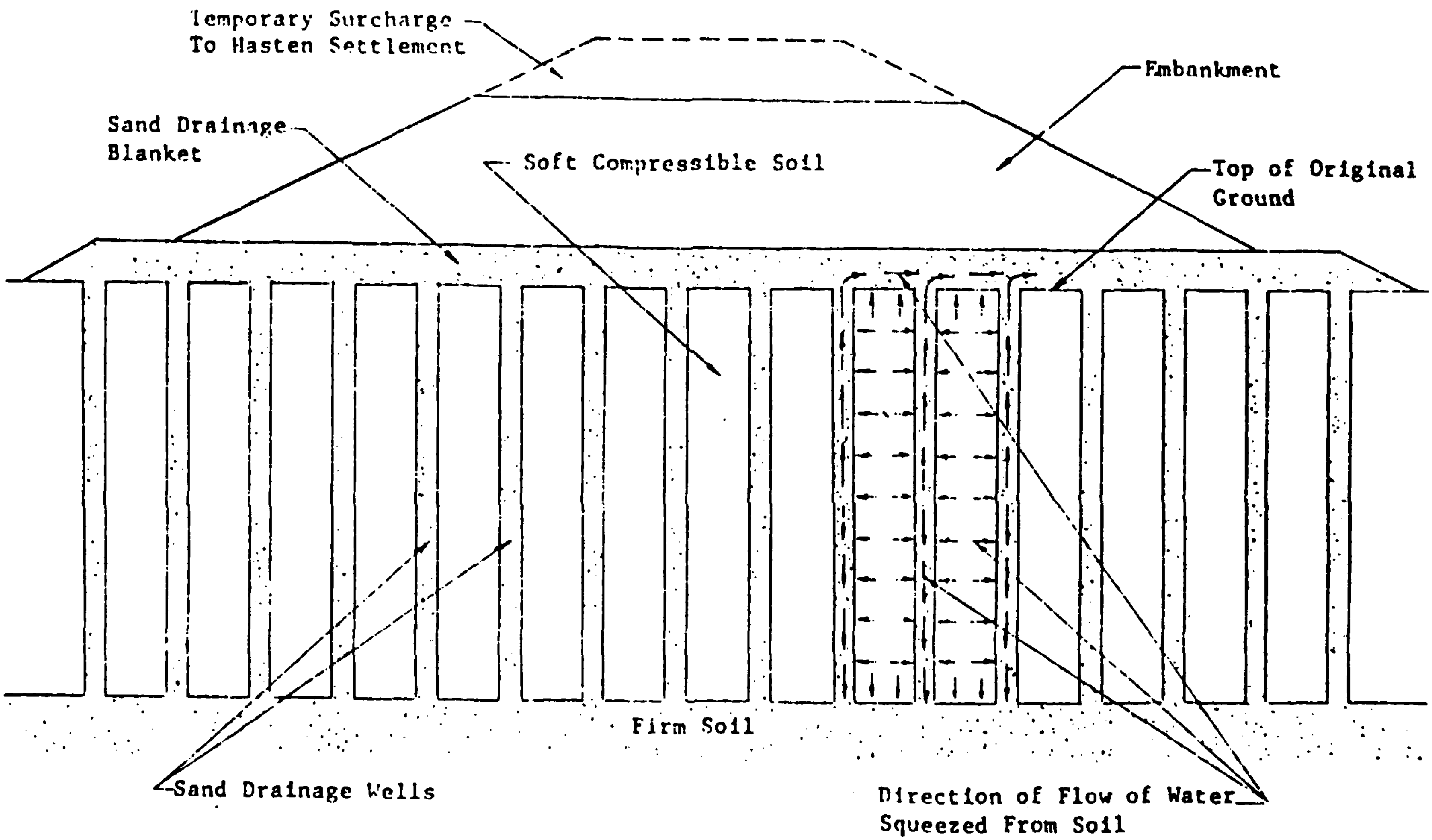


Figure 4.5 Typical Sand Drainage Well Installation (Ref. 11).

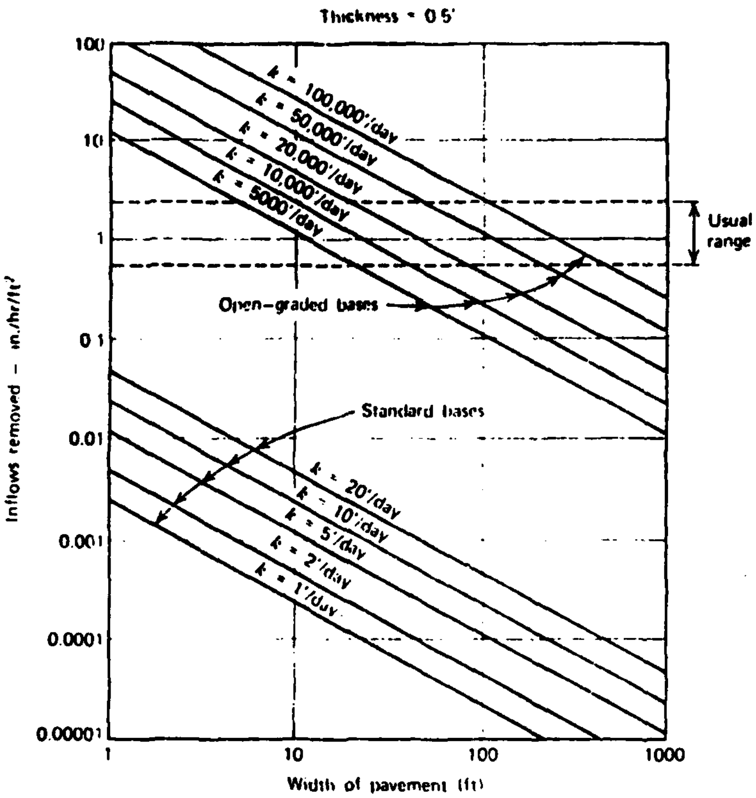


Figure 4.6 Capabilities of Different Bases with Edge Drains to Remove Infiltration (Ref. 10).

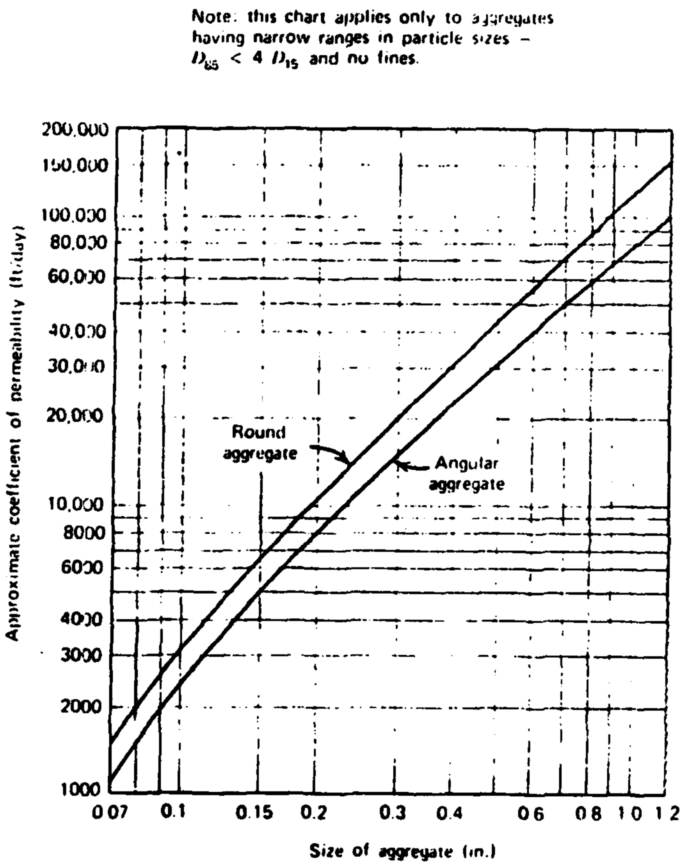


Figure 4.7 Rough Guide for Estimating Coefficient of Permeability of Narrow Size-Ranged Aggregates with no Fines (Ref. 13).

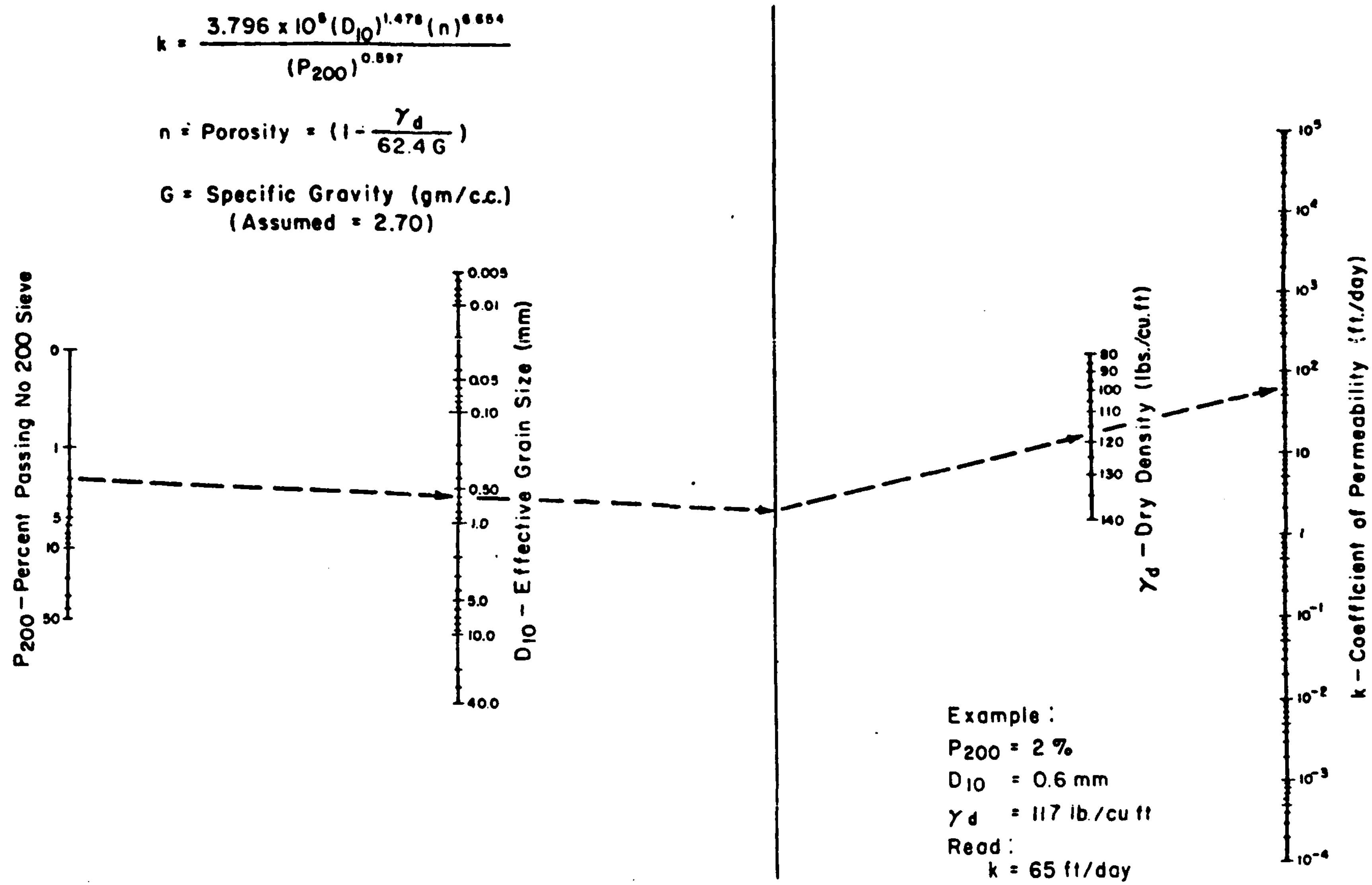


Figure 4.8 Nomographic Procedure to Estimate Permeability of Granular Materials (Ref. 11).

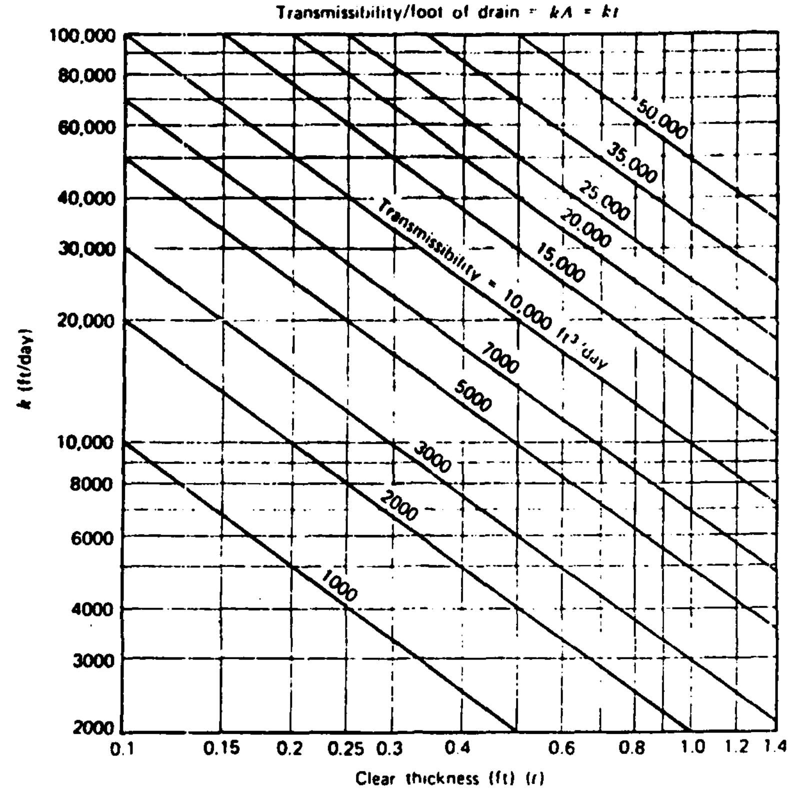


Figure 4.9 Transmissibility of Drainage Layers ft^3/day/foot (Ref. 10).

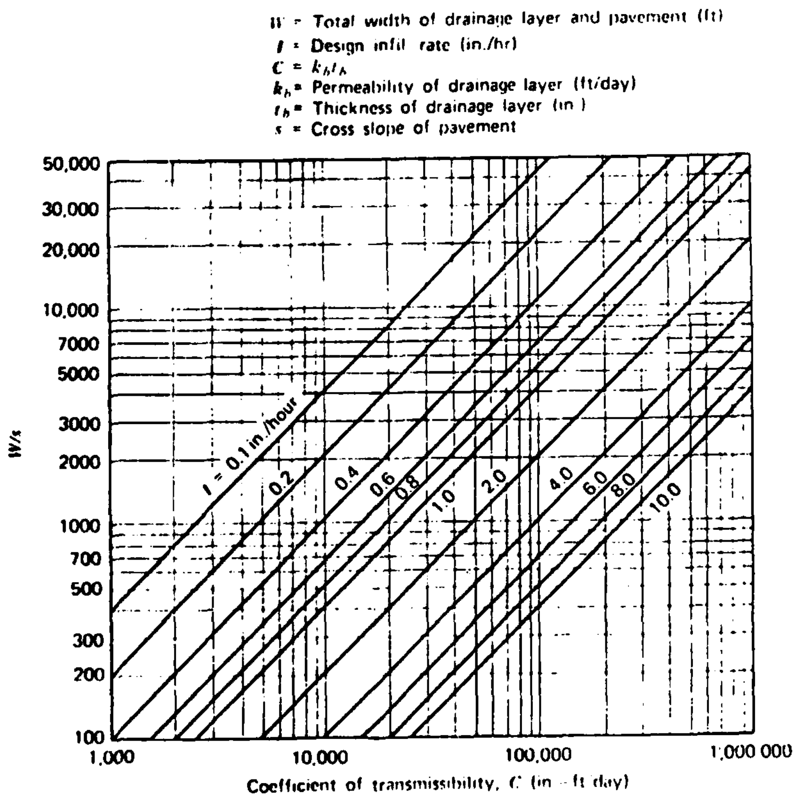


Figure 4.10 Coefficient of Transmissibility Versus W/s Ratio (Ref. 10).

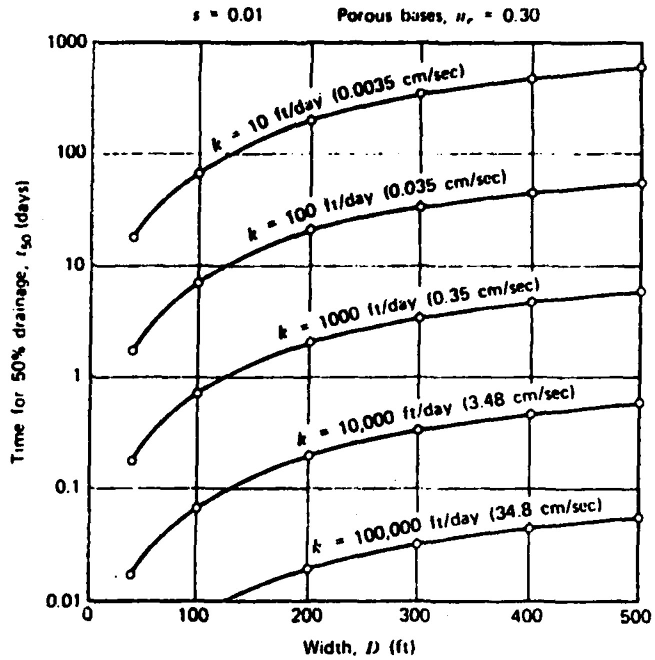


Figure 4.11 Permeability Versus Time for 50% Drainage of Bases with Edge Drains (Ref. 10).

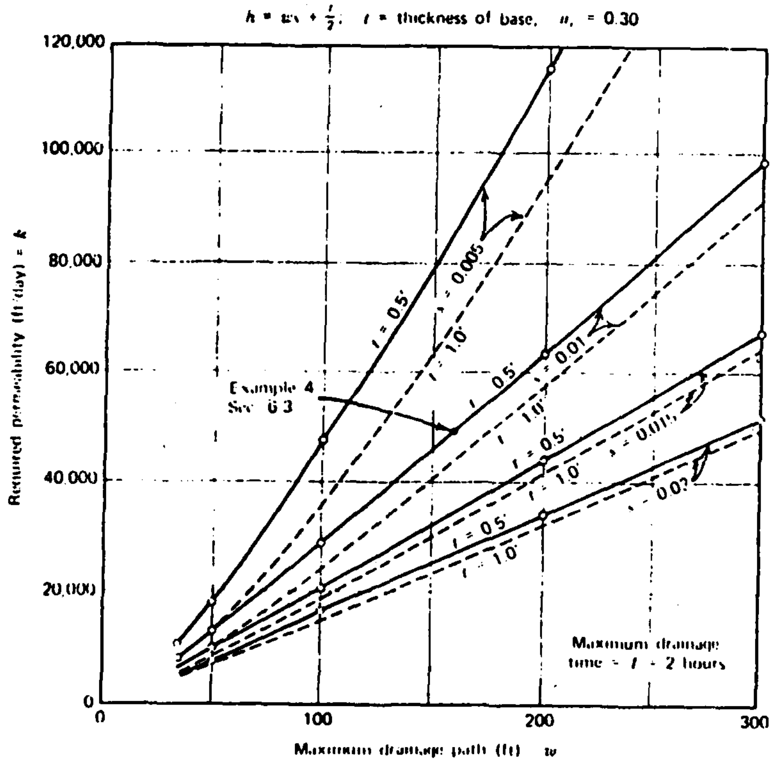
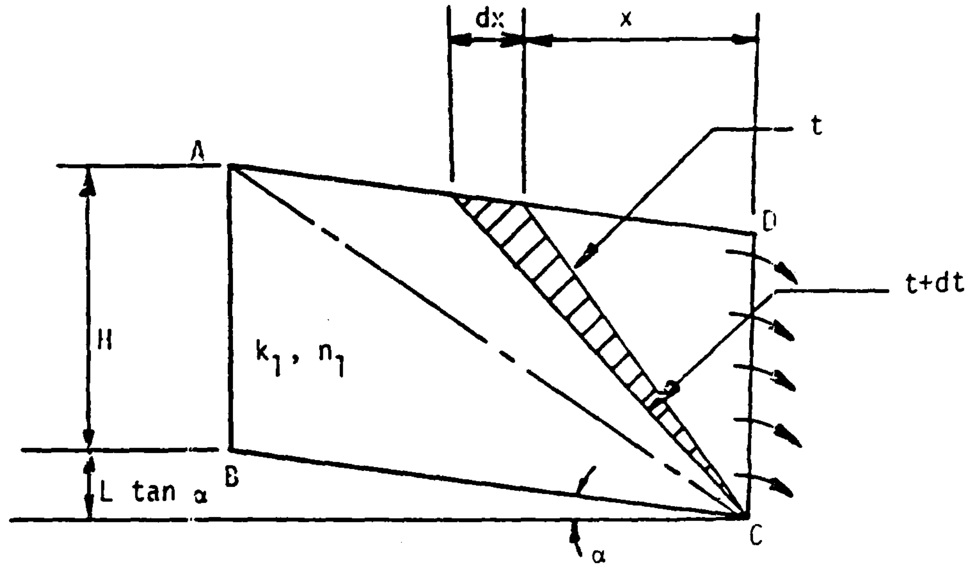
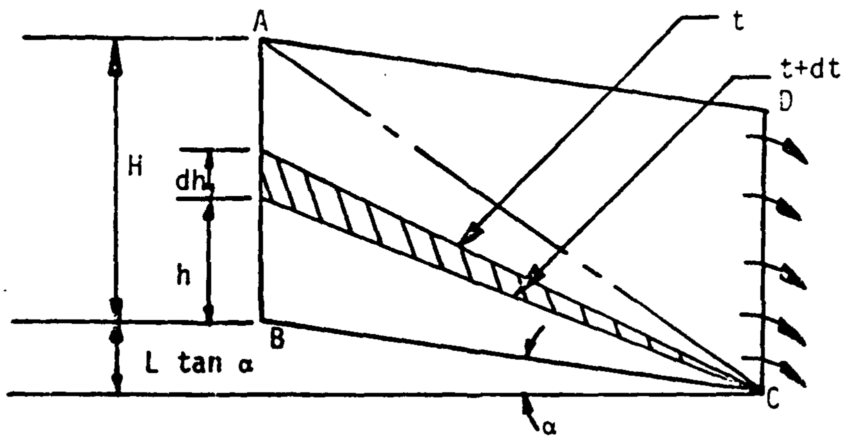


Figure 4.12 Minimum Permeability Required in Order to Drain Base in 2 Hours or Less (Ref. 10).



STAGE 1 U ≤ 50%



STAGE 2 U ≥ 50%

Figure 4.13 Casagrande-Shannon Model for Base Course Drainage (Ref. 14).

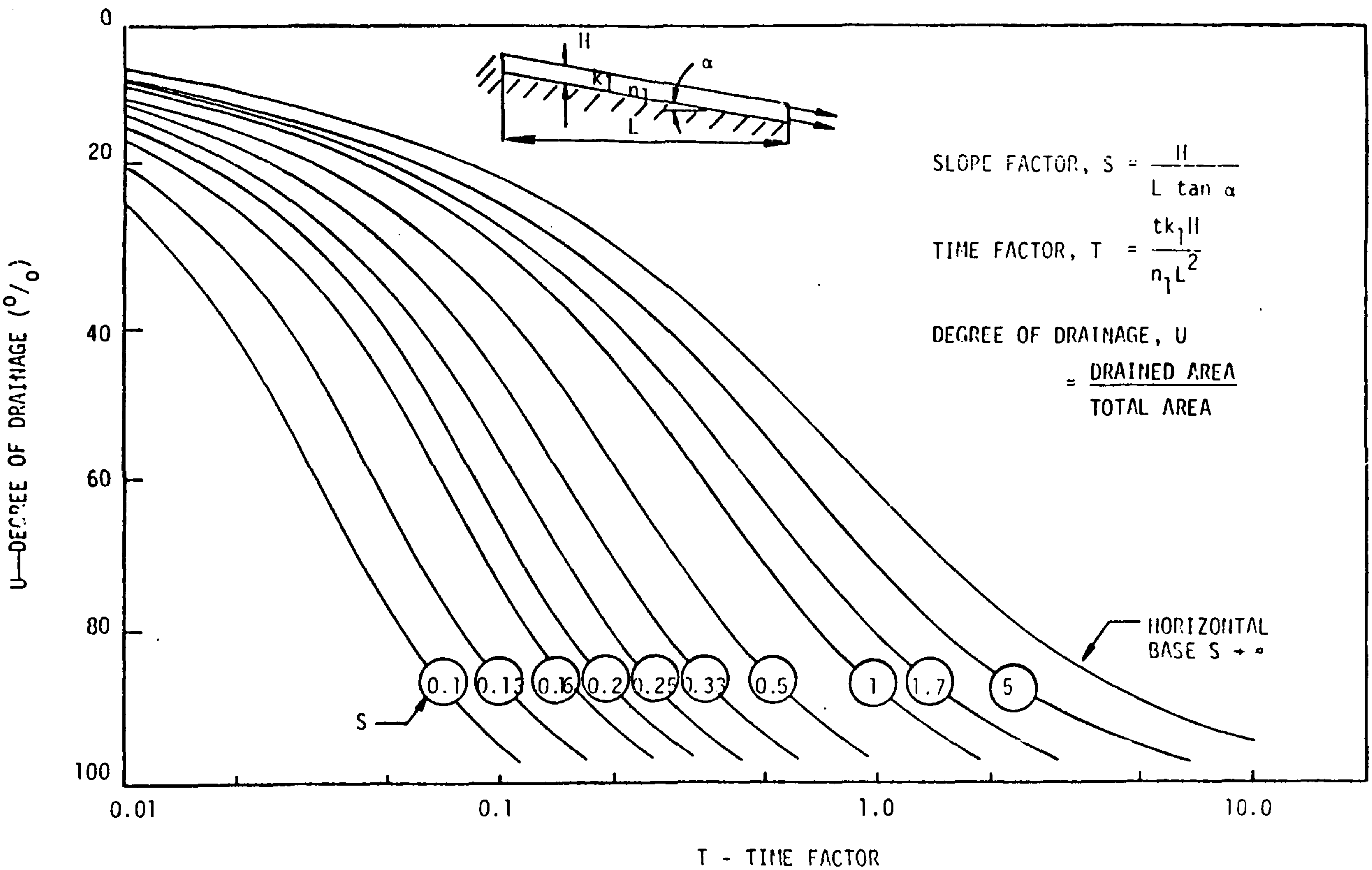


Figure 4.14 Variation of Drainage Area with Slope Factor and Time Factor (Ref. 15).

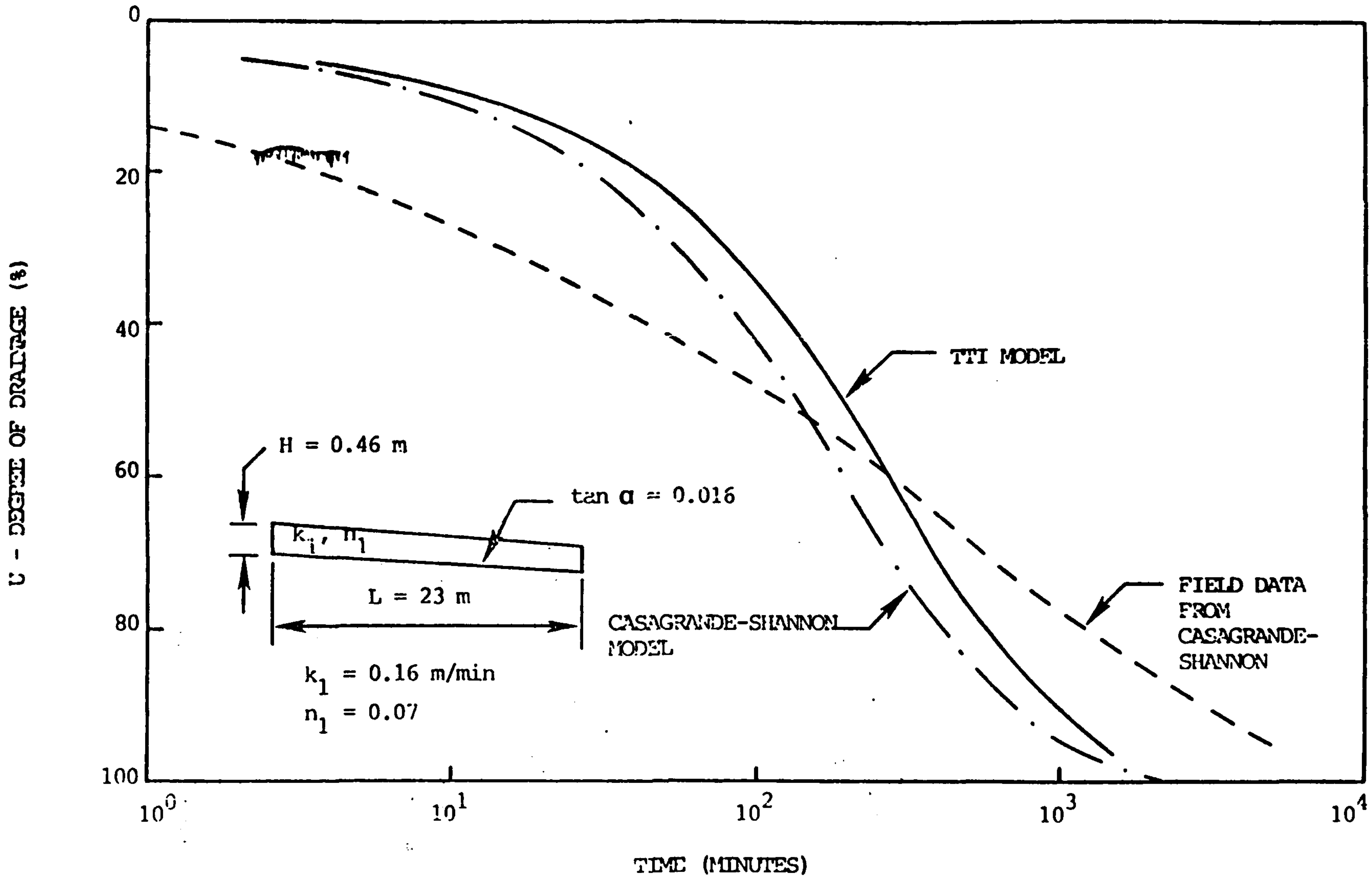


Figure 4.15 Comparison of Model Results for an Impermeable Subgrade (Ref. 16).

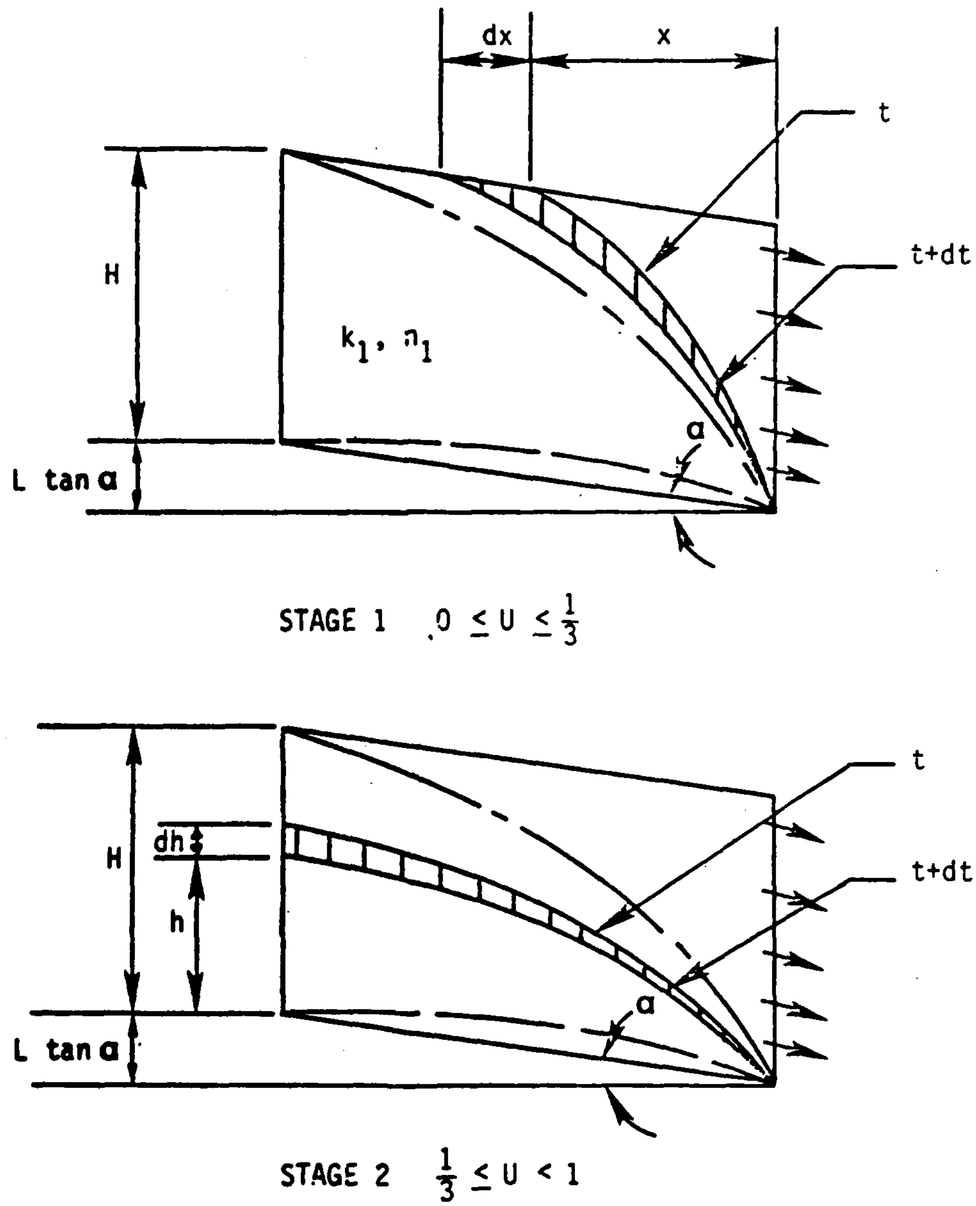
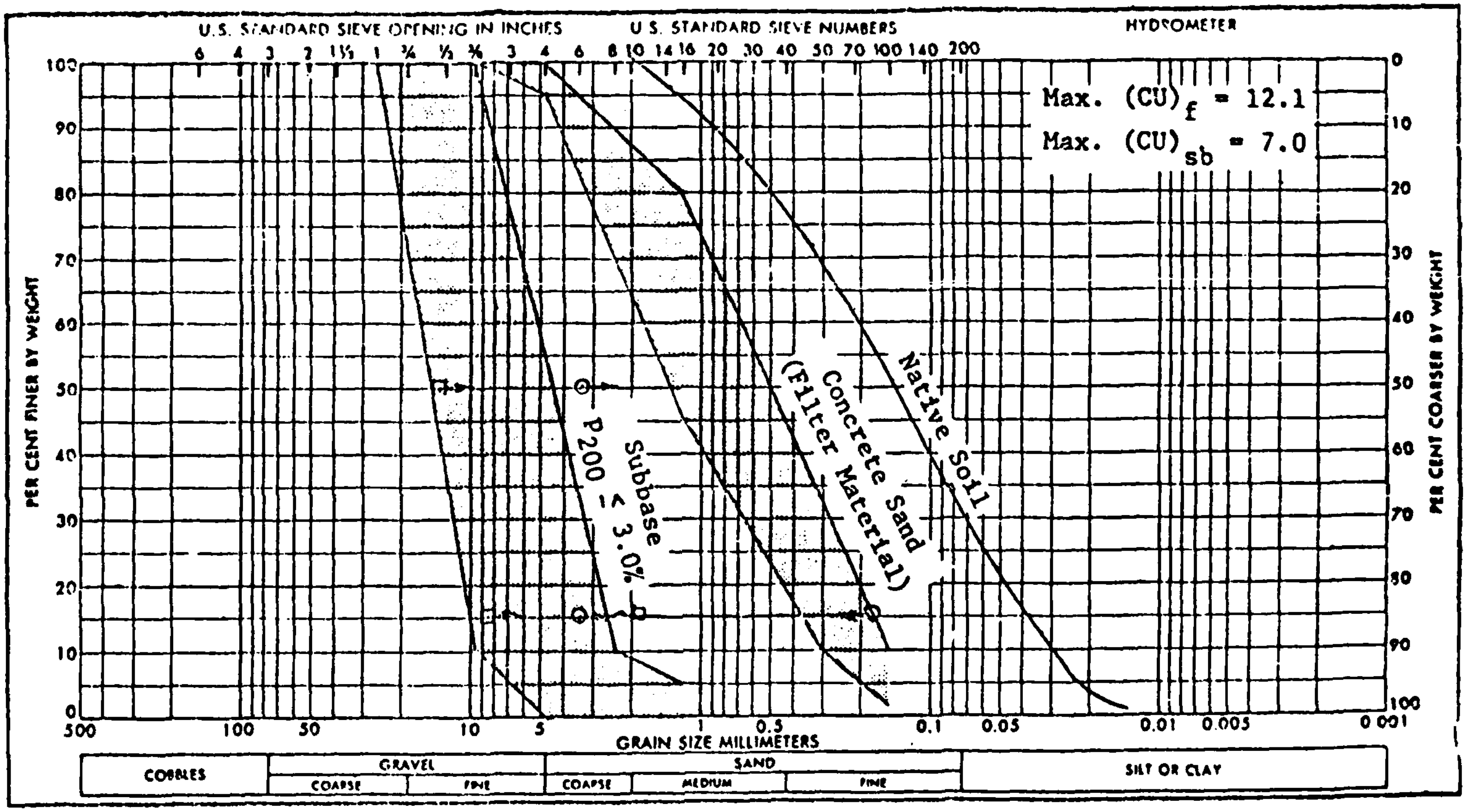


Figure 4.16 TTI Model for Base Course Drainage with an Impermeable Subgrade (Ref. 16).



⊙ Filter Criteria to Protect Natural Soil
□ Filter Criteria to Protect Filter Material Against Intrusion of Subbase

Figure 4.17 Gradation Bands for Subbase, Filter Layer, and Subgrade Material (Ref. 17).

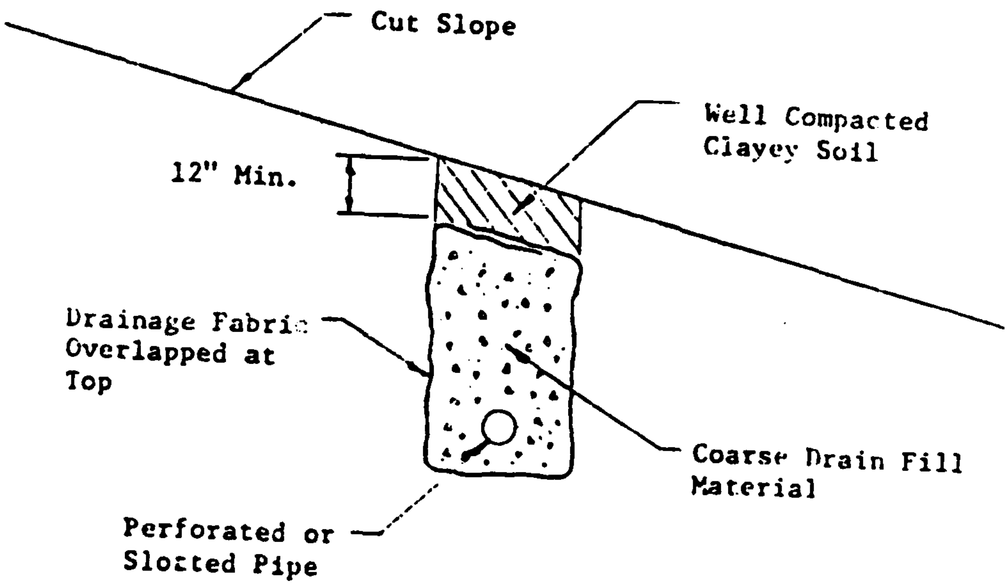


Figure 4.18 Typical Filter System for Interceptor Drain Using Coarse Filter Aggregate and Drainage Fabric (Ref. 11).

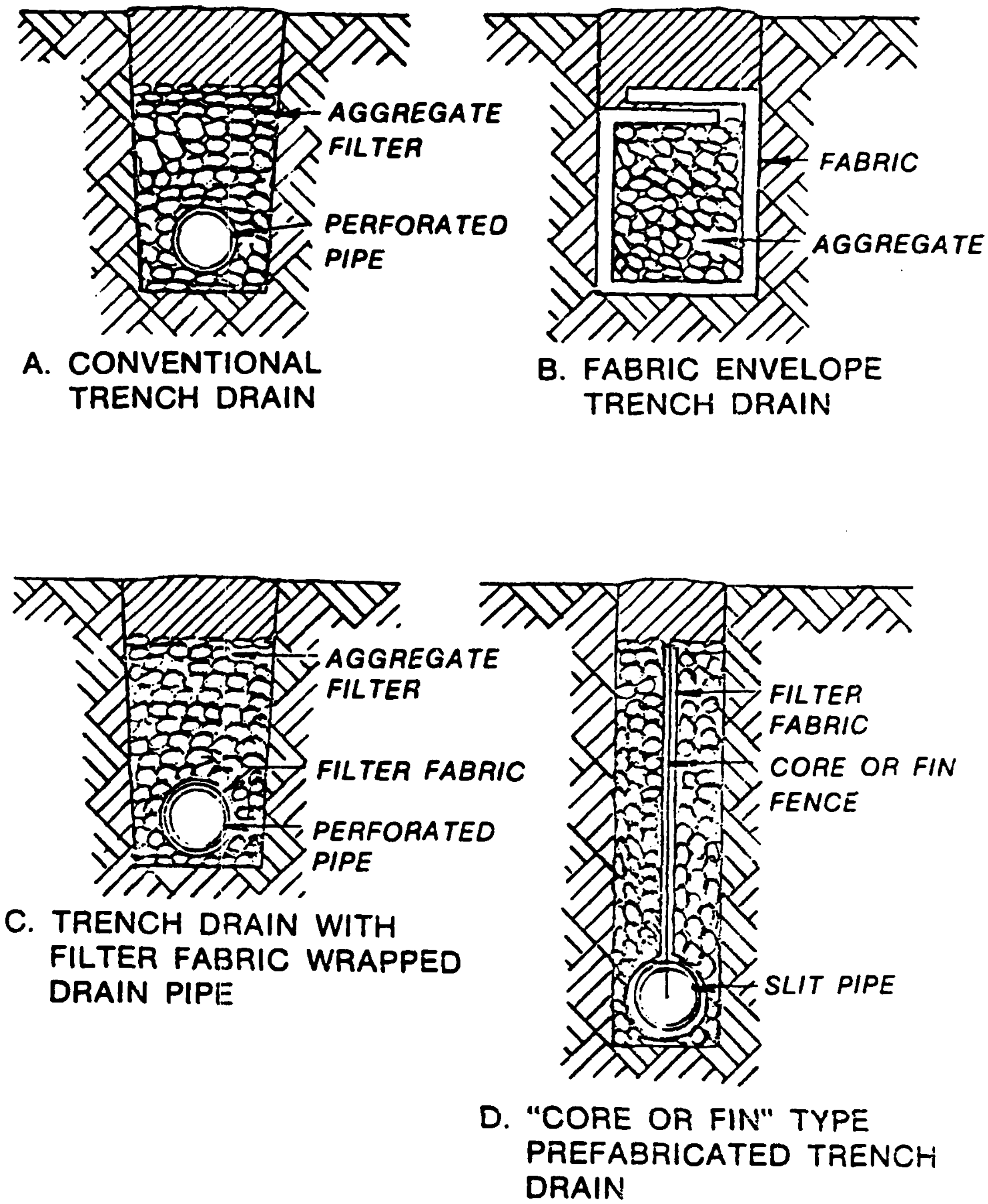
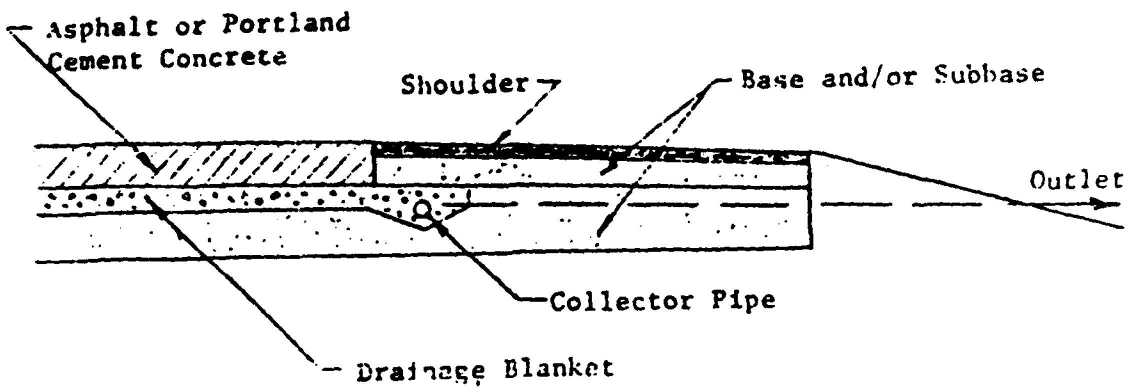
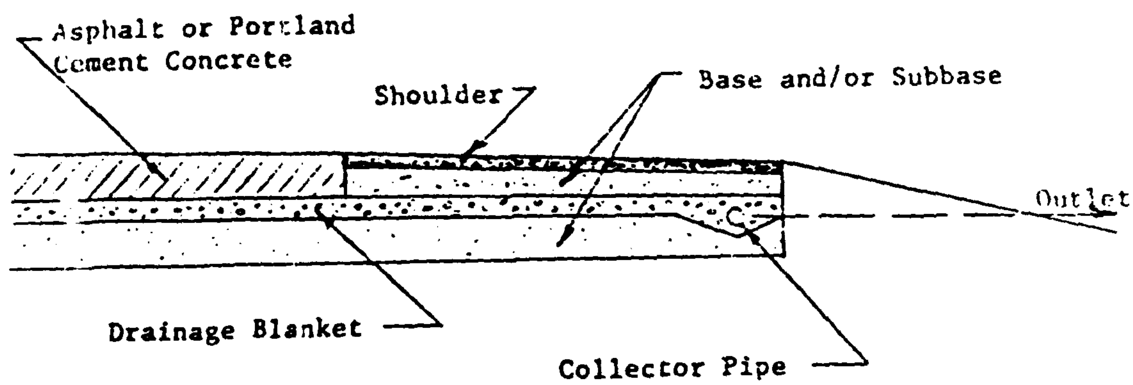


Figure 4.19 Examples of Types of Trench Subdrains (Ref. 23).



(a)



(b)

Figure 4.20 Typical Location of Shallow Longitudinal Subdrainage Pipes (Ref. 11).

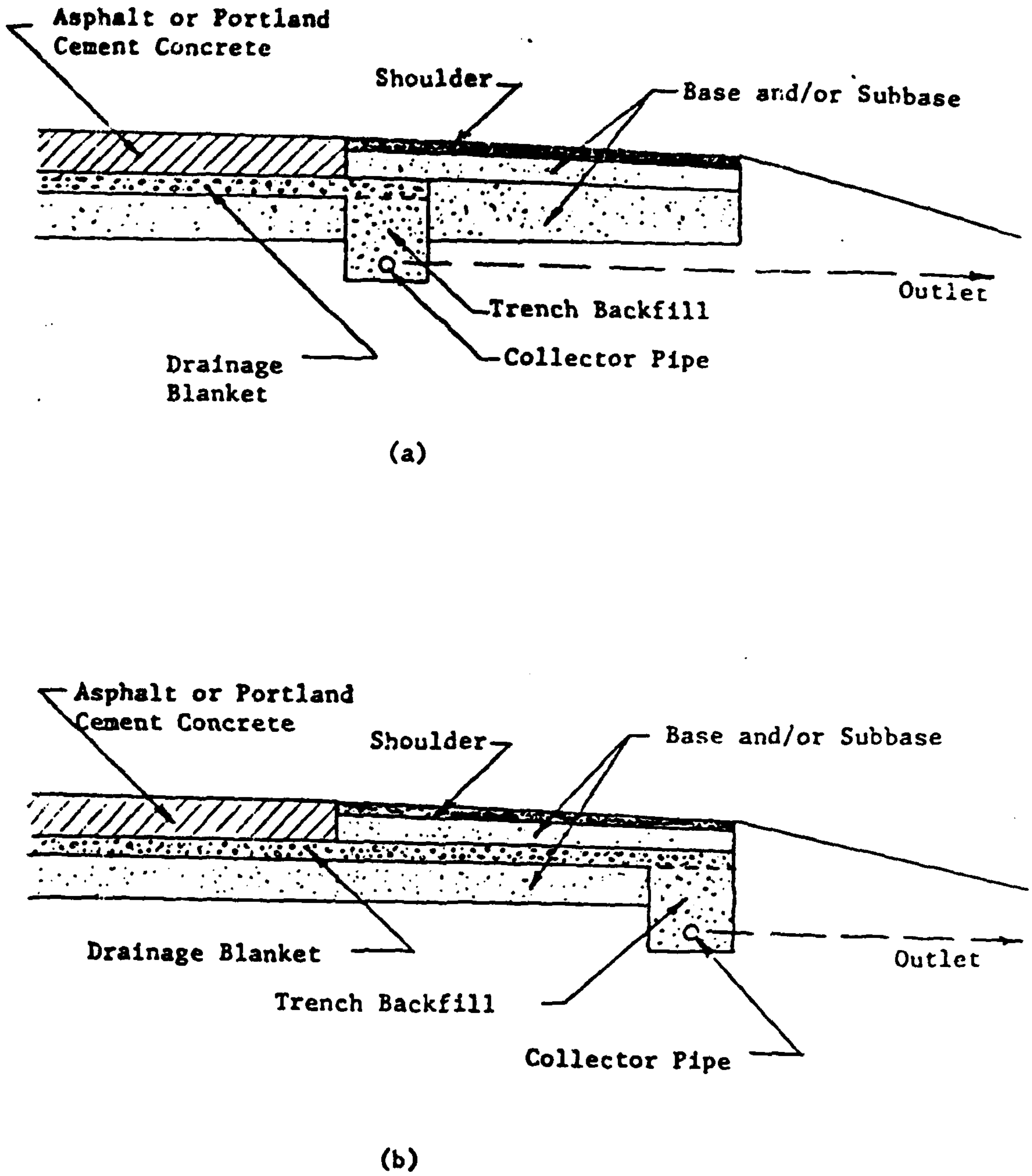


Figure 4.21 Typical Locations of Deep Longitudinal Subdrainage Pipes (Ref. 11).

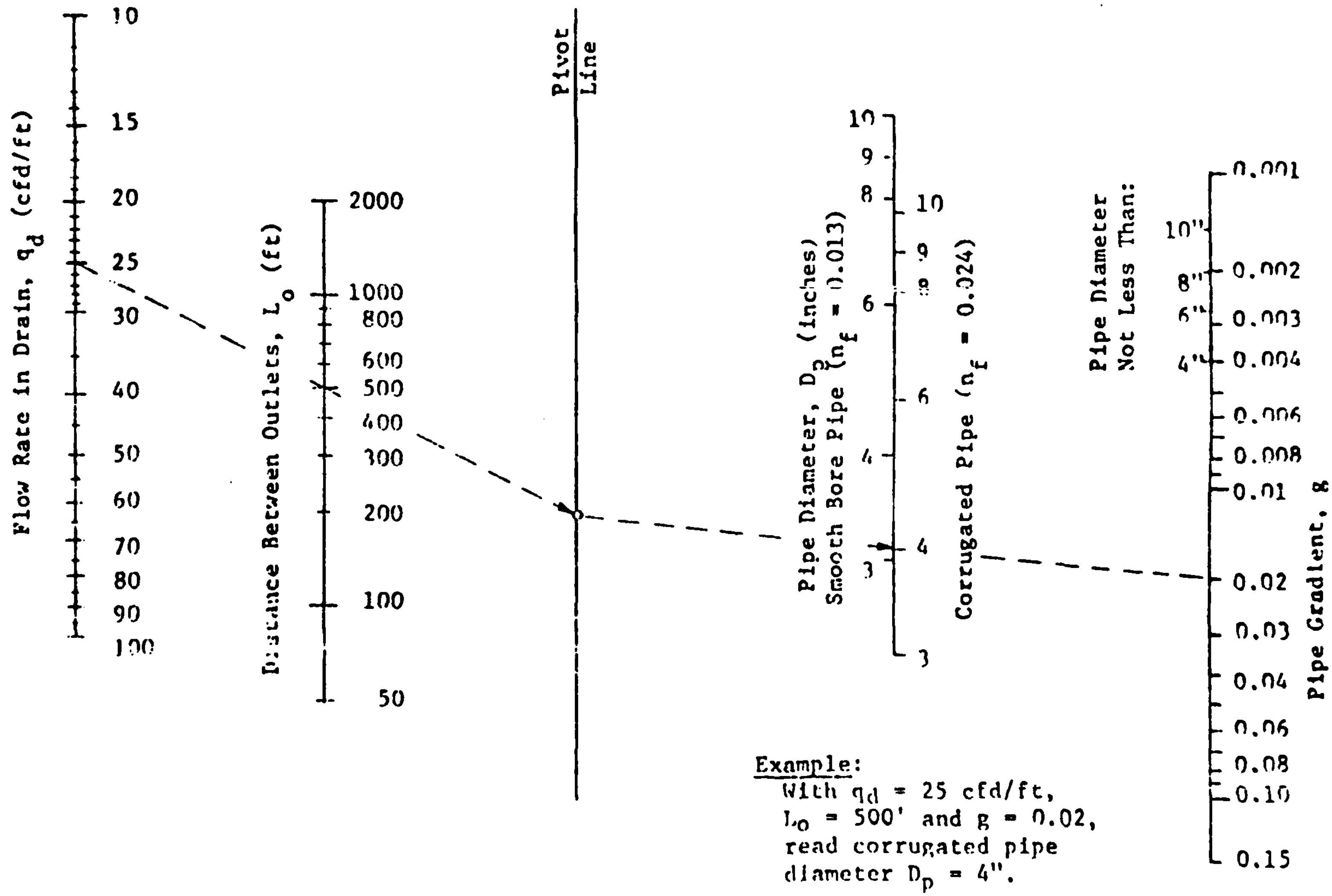


Figure 4.22 Nomogram Relating Subdrainage Pipe Size with Flow Rate, Outlet Spacing, and Pipe Gradient (Ref. 10).

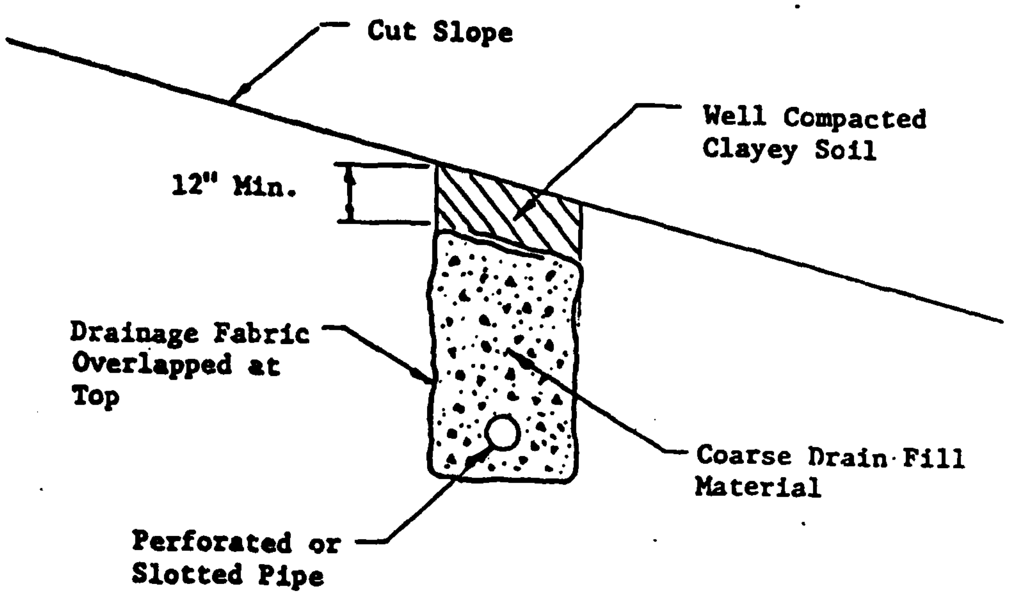


Figure 4.23 Typical Filter System for Interceptor Drain Using Coarse Filter Aggregate and Drainage Fabric (Ref. 11).

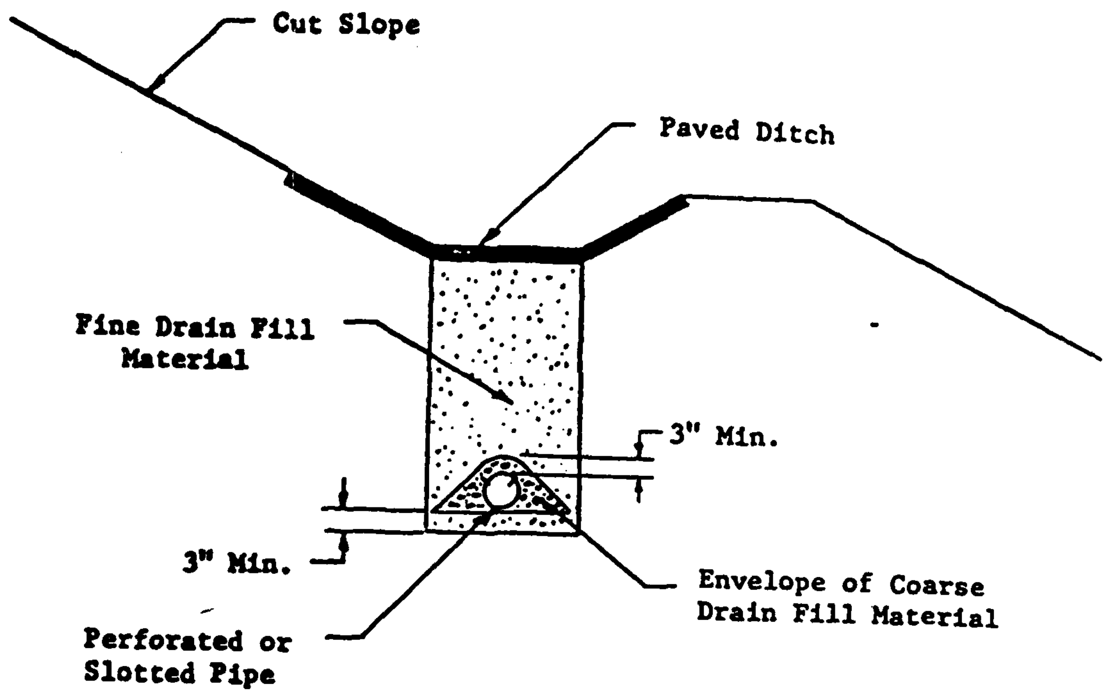


Figure 4.24 Typical Filter System for Interceptor Drain Using Only Filter Aggregates (Ref. 11).

CHAPTER 5

PAVEMENT SURFACE DRAINAGE

5.1 INTRODUCTION

The stopping distance required by a landing aircraft can vary widely depending on the friction between the tire and the runway. When there is a layer of water on the runway, such as during a rainstorm, the friction level can be greatly reduced and there is danger of aircraft hydroplaning. In order to prevent hydroplaning the drainage capacity or rate of runoff on the runway surface must be increased.

The loss of tire friction on wet or flooded pavements is caused by a combination of dynamic and viscous hydroplaning. Hydroplaning occurs when the physical contact between the tires and the runway is lost and the tires are supported by a layer of water. Viscous forces are the result of fluid viscosity effects and are predominant where a thin film of water is present on a smooth runway. This type of hydroplaning can be eliminated by a rough runway microtexture. The dynamic forces increase with increasing water depths. This type of hydroplaning can be eliminated by the quick removal of water from the runway surface. Water removal from the tire/pavement interface can be accomplished by grooving or water accumulation can be reduced by using porous friction courses. Both viscous and dynamic forces are present to some extent when hydroplaning occurs.

An example of a partial hydroplaning condition is shown in Figure 5.1 (1). Zone 1 is the section of the tire footprint supported by bulk water. Zone 2 is the part of the tire supported by a thin film of water. Zone 3 is the only section of the tire in contact with the pavement. The size of zone 1 relates to the time required for the tire to squeeze the bulk water out of

the tire footprint at this speed. The same condition applies to the size of zone 2 for the thin film of water. The size of zone 3 relates to the fraction of the coefficient of friction which can be obtained under these conditions. The area of zone 3 divided by the tire contact area and multiplied by the unit shear force gives the friction coefficient that the tire can develop under these conditions.

Research has been completed to find ways to identify slippery runways on which hydroplaning can more readily occur (1). Several ground vehicle devices, such as the British Mu-meter and the Diagonal Braked Vehicle (DBV), have been developed to give a measurement of friction on a pavement. While attempts have been made to correlate these with actual aircraft stopping distances the comparisons have been fair to good between ground vehicles, but poor between ground vehicle and aircraft, and between different aircraft. At best the devices can be used to measure relative friction between pavement surfaces to decide which runways have the lowest friction and therefore require maintenance. FAA Advisory Circular No. 150/5320-12A issued in July 11, 1986 provides guidelines on the use of these devices to evaluate airport pavement surface friction characteristics (2).

5.2 PAVEMENT GROOVING

Grooving helps to prevent hydroplaning by providing channels for water to escape from beneath the tire at the tire/pavement interface, thus reducing the chances of hydroplaning. Also the drainage rate is increased by the polished groove channels created by diamond saw cutting which greatly reduces water flow resistance when compared to water draining over the comparatively rough pavement surface.

There are several different ways of grooving a runway surface. Plastic grooving is the grooving of a concrete surface when it is still in the plastic state. This type of grooving is not considered as effective as other techniques because the grooves can be interrupted or misaligned at the pavement edge and the groove channel walls have rougher surfaces. Saw cut grooves provide smooth, evenly spaced channels. This is the most common form of grooving to reduce hydroplaning. One disadvantage of saw cut grooving is that a concrete runway surface must be thoroughly cleaned afterwards with high powered water jets to remove all of the concrete dust or the air from the jet engines of the airplanes will cause dust clouds which reduce visibility and can be a safety hazard (1). Reflex percussive grooves are less expensive to construct because of the higher operating speeds of the equipment and longer life of the grooves before they must be replaced. The braking performance on runway surfaces with these v-shaped grooves, shown in Figure 5.2, has been found to be comparable to conventional saw cut grooves (3).

To form a reflex percussive groove the cutting head strikes the surface of the concrete, causing the material under the area of impact to deflect downward. The compressive strain caused is almost immediately given up in generating a rebound which causes the material to go into tension, which is nearly equal to the initial compression. Since the concrete is weak in tension it fractures. This method does not loosen the aggregate particles or create micro fractures in the surrounding concrete so the pavement is not weakened. While this method is good for portland cement concrete it may not be as successful on asphalt concrete because the cut is not clean.

The three identifying groove dimensions are width, depth, and pitch or distance between groove centerlines. An investigation by Agrawal and

Daiutolo (3) concluded that changing the pitch created substantially more savings than changing groove size. The FAA recommends 1/4 in. grooves spaced at 1 1/4 in. for installation on runways where the potential for hydroplaning exists. Experiments by Agrawal and Daiutolo (3) were conducted to measured the coefficient of friction under different conditions for speeds from 70-to 150-knots and pitches up to 4 in. The friction levels available on grooves with a 3-in. pitch under wet operating conditions are not significantly below those obtained on grooves spaced at 1 1/4 in. while the cost of installation is reduced by about 25%. Comparisons also showed that reflex percussive grooves spaced at 4 1/2 in. are comparable to conventional grooves spaced at 2 in. The installation of these grooves could be as low as one half that of conventional grooves with a pitch of 1 1/4 in.

Grooving can cause damage to large, heavy aircraft tires when landing as they first skid on the runway before rotation is started. The damage, known as chevron cuts, was investigated by NASA (4). Their conclusion was that the damage can be reduced by prerotation of the tires. Also, in the early 1970's, the aircraft tire industry developed new tread rubber compounds and tread designs that significantly reduce the amount of chevron cuts from runway grooves. Data from American Airlines reports, show that this increased the number of landings per tire change by 50% while the number of grooved runways increased approximately three times (4).

Reed, Kibler, and Agrawal (5) developed a mathematical model to simulate runoff from grooved runways. A hydraulically equivalent ungrooved surface which has a width equal to the wetted perimeter of a grooved surface is used to preserve the shear area. The model simulates flow depths for different groove spacings. The model parameters used are the transverse slope of the surface, surface texture, groove size and shape, groove spacing, and a

uniform rainfall rate. A computer program executed the model successfully and satisfied mass continuity, but there were several weaknesses detected. The model did not take into account the more polished surface of a saw-cut groove. A weighted average may be put into the model to take this into account. Another weakness with the model is that the lateral inflow is based on the size of the wetted perimeter compared to the top width of the groove. One would think that lateral inflow would be independent of groove shape, but not in this model. Early experimental results indicated that arbitrary allotments based on the wetted perimeter are too conservative. A test to verify the model was written up by Reed, Proctor, Kibler, and Agrawal (6). In this test similar rainfall was applied to a grooved laboratory slab. The water movement was traced with dye to see how much of the water was carried in the grooves. Water depths were measured using pressure transducers. All of the water in the test was carried in the grooves until they filled up and overflowed at a downstream point. The water depths on the upstream surface were negligible. The roughness coefficient for the pavement surface was found to be higher than expected and that of the grooves was lower than expected. After the models were adjusted, water depth reduction at the pavement edge was 28% versus 19% found originally. Figure 5.3 shows the percent reduction in water depth as a function of distance for various groove spacings. Figure 5.4 shows the water depths predicted by the model for various groove spacings at a rainfall rate of 3 in./hr.

5.3 POROUS FRICTION COURSE

5.3.1 General

A considerable amount of discussion concerning open-graded asphalt friction courses or porous friction courses (PFC) can be found in reports by Jones (7), Smith, et. al. (8), Tomita and Forrest (9), Johnson and White (10), and Agrawal (11).

A porous friction course (PFC) is a type of surface treatment, usually 5/8 in.- to 3/4 in.- thick, designed to reduce hydroplaning and increase skid resistance on pavements. This is accomplished by allowing the surface water to drain through the layer, vertically and then laterally. A major reason for the effectiveness of the PFC is the elimination or reduction in thickness of the sheets of water between the tire and the pavement surface.

Since the PFC is considered to be a surface treatment (less than 1 in. thick), it does not add to the structural integrity of the pavement structure. It is, however, processed in a mix plant and laid down in a manner similar to a conventional asphalt concrete surface, as opposed to being sprayed on like some surface treatments.

5.3.2 Design of the Porous Friction Course (PFC) Mix

Two important design parameters for a PFC are the asphalt content and the gradation of the aggregate. A change in either one of the two in the design mix can alter the performance of the PFC greatly.

The gradation of the aggregate is very important since the main purpose of a PFC is to retain enough void content to enable adequate drainage of water through the layer. A minimum void content of about 15% is recommended for design purposes. Thus, the aggregate gradation has to be fairly uniform

to provide a high void content. A typical gradation for an aggregate to be used in a PFC is shown below in Table 5.1 (12). Other aggregate requirements for a PFC include low abrasion loss, high resistance to polishing, and that the aggregate should be completely crushed. As shown in Table 5.1, there is some fine aggregate in the gradation. This small amount of fines is just enough for stabilization of the coarse fraction which constitutes the majority of the aggregate. One property of the coarse fraction of the aggregate that has to be evaluated is the skid resistance. Skid resistance is a function of both macrotexture and microtexture. This means that the coarse aggregate must provide the necessary microtexture without help from the fine aggregate. It can be seen that an aggregate must meet many requirements in order for the PFC to perform as desired.

A second important factor in the design of a PFC is the asphalt content. The PFC does not conform to the usual standards of stability and flow for choosing asphalt content. On the basis of these two properties, the PFC does not yield definitive results. Therefore, a substantial amount of engineering judgment is required in the selection of the asphalt content in the mix. Too little asphalt content can cause premature stripping and ravelling to occur where as too much asphalt content will fill the void space and hinder drainage. Great care must also be taken in selecting an optimum mixing temperature and the grade of asphalt cement used in the mix. Grades of AC-10, AC-20, AC-40, AR-40, and AR-80 have been recommended for use in the mix, depending on the climate. The more viscous the binder, the thicker the film on the aggregate will be. Also a more viscous asphalt can be mixed at a higher temperature without the binder running off of the aggregate.

5.3.3. Performance

Only recently has the PFC been employed on airfields. The PFC can be and has been very effective in reducing hydroplaning on runways. One such runway is runway 14-32 at the Greensboro-High Point-Winston-Salem regional airport in North Carolina (10). Before a PFC layer was placed, six hydroplaning incidents occured over a few years. One resulted in $3.5 million dollars damage to the aircraft. Despite some freezing and many heavy rains during the following winter and spring after a PFC was laid down on the Runway 14-32, no hydroplaning incidents occured.

Another runway corrected by a PFC placed on it was Runway 17-35 at the U.S. Naval Air Station in Dallas, Texas (7). Originally, there was slow surface drainage after rainstorms on the runway due to flat cross slopes and poor surface geometrics. Another problem was that the runway was shorter than usual which decreased the allowable breaking distance. A 5/8 in. thick PFC was placed which increased surface drainage substantially and reduced the chance for hydroplaning to occur. Table 5.2 shows some characteristics of the mix.

Some benefits other than improved skid resistance and decreasing hydroplaning can be attributed to the addition of a PFC layer. The PFC retards the formation of ice on the pavement surface. Also, there is improved surface smoothness, improved visibility of painted markings, and less glare at night during wet weather.

The key to the success of the PFC is its permeability. The permeability has to be maintained at an adequate level at all times to ensure a reduction in hydroplaning. This means that maintenance operations should focus on the removal of silt, sand, rubber, and other foreign matter from the wearing course to maintain its high permeability. Failure to do so will result in a

substantial reduction in the effectiveness of the PFC. Some rates of rainfall that can be removed by a 0.05 ft thick PFC are shown in Table 5.3 (13). These values are dependent on the permeability in the layer.

As with most new concepts, the PFC is not without its shortcomings. At the present time, widespread use of the PFC is being slowed somewhat due to a number of problems concerning design, construction, and durability. Some of these problems include rapid formation of reflective cracking, ravelling, stripping, and delamination when placed directly over PCC pavements. Porous friction course surfaces tend to become clogged with rubber deposits when used on high activity runways. Cleaning PFC's of deposited rubber is difficult and this has discouraged their use on high activity runways. These problems are mainly caused by the unique characteristics of this wearing course, that is, the thinness of the layer as well as the high void content. These faults should not discourage future use of the PFC however. More research and experimentation is being performed on the PFC. If shortcomings of the PFC can be improved upon, its use could become prevalent on airport pavements, particularly runways, because of the benefits that it has to offer.

REFERENCES

1. Horne, W.B., "Status of Runway Slipperiness Research," Transportation Research Record No. 624, Transportation Research Board, Washington, D.C., 1976.
2. "Methods for the Design, Construction, and Maintenance of Skid Resistant Airport Pavement Surfaces," AC No. 150/5320-12A, Federal Aviation Administration, Washington, D.C., 1986.
3. Agrawal, S.K. and Daiutolo, H., "The Braking Performance of an Aircraft Tire on Grooved Portland Cement Concrete Surfaces," Interim Report, FAA-RD-80-78, FAA-CT-80-35, Federal Aviation Administration Technical Center, Atlantic City Airport, N.J., 1981.
4. Byrdsong, T.A., McCarty, J.L., and Yager, T.J., "An Investigation of Aircraft Tire Damage Resulting From Touchdown on Grooved Runway Surfaces," NASA-TN-D-6690, 1972.
5. Reed, J.R., Kibler, D.F., and Agrawal, S.K., "Mathematical Model of Runoff From Grooved Runways," Transportation Research Circular No. 274, Transportation Research Board, Washington, D.C., 1984.
6. Reed, J.R., Proctor, M.L., Kibler, D.F., and Agrawal, S.K., "Measurements of Runoff Depths from a Grooved Laboratory Slab," Transportation Research Circular No. 274, Transportation Research Board, Washington, D.C., 1984.
7. Jones, M.P., "Friction Overlay Improves Runway Skid Resistance," Civil Engineering, American Society of Civil Engineering, New York, N.Y., 1973.
8. Smith, R.W., et al., "Design of Open Graded Asphalt Friction Courses," FHWA-RD-74-2, Federal Highway Administration, Washington, D.C., 1974.
9. Tomita, H. and Forrest, J. B., "Porous Friction Surfaces for Airfield Pavements," AFWL-TF-74-177, U.S. Air Force, Albuquerque, N.M., 1974.

10. Johnson, E.A. and White, T.D., "Porous Friction Course Solves Airport Hydroplaning Problem," Civil Engineering, American Society of Civil Engineering, New York, N.Y., 1976.

11. Agrawal, S.K., "Braking of an Aircraft Tire on Grooved and Pourous Asphaltic Concrete," FAA-RD-82-77, FAA-CT-82-147, FAA Technical Center, Atlantic City Airport, N.J., 1983.

12. Bituminous Material and Mix Design," Department of Civil Engineering, University of Illinois, Champaign-Urbana, 1984.

13. Cedergren, H.R.. "Drainage of Highway and Airfield Pavements," Wiley, New York, N.Y., 1974.

Table 5.1 A Typical Aggregate Gradation for PFC (Ref. 12).

Sieve	Percent Passing
3/8"	100
#4	30-50
#8	5-15
#200	2-5

Table 5.2 Aggregate Gradation and Mix Characteristics Used at Dallas Naval Air Station (Ref. 7).

Aggregate Gradation

Sieve	Percent Passing
1/2"	100
3/8"	97
#4	38
#8	15.7
#30	6.1
#200	2.0

Asphalt content, %	6.5
Mixing Temperature, F	280
Mixing Viscosity, Centistokes	450

Table 5.3 Rates of Rainfall that Can Be Removed by 0.05 ft. Thick PFC Overlay (Ref. 13).

k (ft./day)*	Q (ft.^{3}/day)	Rainfall Rate (in./hr)
1,000	0.5	0.012
5,000	2.5	0.060
10,000	5.0	0.120
20,000	10.0	0.240

* Applies to 1-ft. strip of pavement with sloping distance of 40 ft. and slope in direction of flow of 0.01.

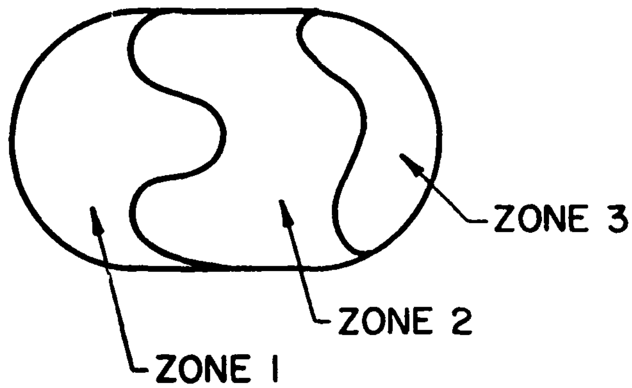


Figure 5.1 Tire Imprint Pattern on a Wet Pavement (Ref. 1).

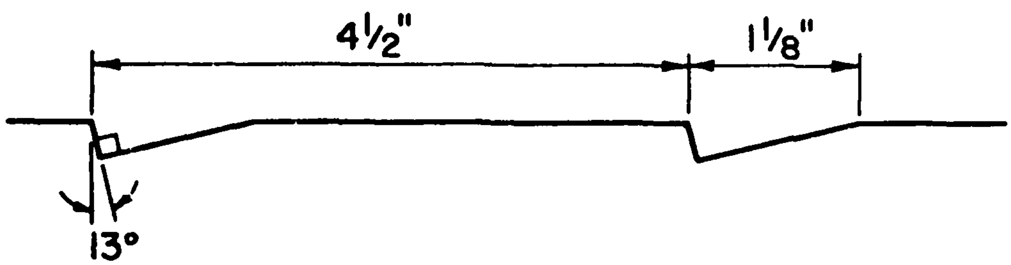


Figure 5.2 Reflex Percussive Grooves (Ref. 3).

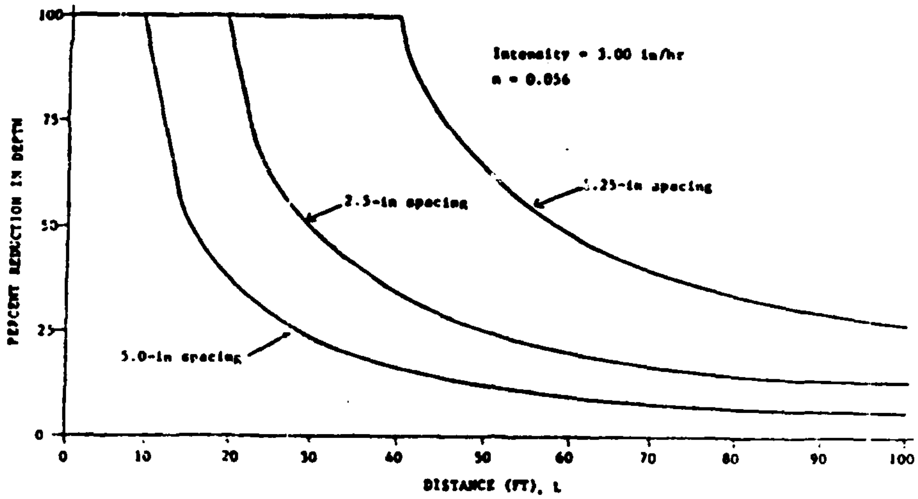


Figure 5.3 Predicted Reduction in Water Depth Versus Distance for Various Groove Spacings (Ref. 5).

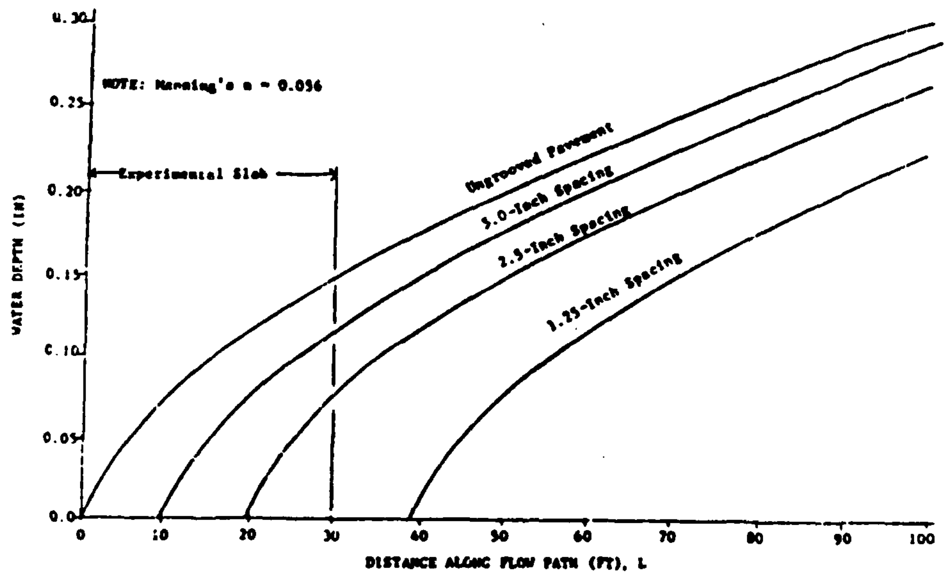


Figure 5.4 Predicted Water Depths for Various Groove Spacings for Rainfall Intensity of 3 in./hr (Ref. 5).

CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS

6.1 CONCLUSIONS

From the literature review to examine the state-of-the-art of airport drainage, it can be concluded that:

1. Although airport surface drainage design is covered reasonably well in the FAA Advisory Circulars on Airport Drainage, there is a substantial need for airport pavement subdrainage design guidelines.
2. Knowledge can be transferred from highway design of drainage systems to the design of airport drainage systems. The main difference between the two areas is that the width of a runway or taxiway is much greater than that of a highway.
3. The drainage time of infiltrated water in the pavement system is more important in airports than in highways because of the longer drainage paths. Water being in the pavement section longer makes the pavement more susceptable to the damaging effects of water (moisture accelerated distress).
4. In order to alleviate the distresses caused by the damaging effects of water, open graded bases or drainage layers (which are more permeable than the standard bases used) should be considered in the drainage of airport pavements. A setback in the use of open graded bases is that they do not provide a very stable working platform during construction. Thus, some kind of trade-off has to be made between the positive drainage effects of the open graded base and the good stable platform that the less permeable standard bases have to offer.

5. The use of filter layers and geotextile filter fabrics should be important considerations in airport pavement subdrainage construction.

6. Hydroplaning is a serious problem on runways and must be reduced or eliminated for safety reasons. At the present time this is accomplished with transverse grooved PCC pavements and porous friction course (PFC) overlays.

6.2 RECOMMENDATIONS

1. Comprehensive design guidelines need to be developed for airport pavement subsurface drainage.

2. Experimental strips of runway should be installed with more permeable, open graded bases that also provide a good working platform. This would require that the gradations of such open graded bases be adjusted so as to obtain the better working platform and still retain the higher permeabilties characteristic of the open graded bases.

3. More airport testing should be completed with plastic pipe to become more familiar with the performance of plastic pipe in the field. It would appear that plastic pipe has a future in airport drainage, but its use in the field is being hindered by lack of experience.

4. More research should be done with the mix design of the porous friction course. The durability of the PFC has to be increased to avoid rapid formation of reflective cracking, ravelling, stripping, and delamination in the field. This wearing course is effective in reducing hydroplaning and efforts should be made to reduce the mix problems that now exists.

4. Experimental intermediate longitudinal and transverse drains should be installed on runways and monitored so as to try to decrease the depth of surface water runoff. These drains should be placed at strategic points so that the landing gear of the aircraft is not affected in any way.