lo. FHWA/RD-81/078

OVING SUBDRAINAGE AND SHOULDERS OF EXISTING PAVEMENTS

January 1982
Final Report

U.S. Department of Transportation

Federal Highway Administration

Offices of Research & Development
Structures and Applied Mechanics Division
Washington, D.C. 20590

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FOREWORD

This report describes methods for improving the design and maintenance of pavement subsurface drainage systems and will be of interest to engineers concerned with the design and maintenance of highway pavements.

The report presents part of the results of University of Illinois research "Improving Subdrainage and Shoulders of Existing Pavements." Other reports in this series are:

FHWA/RD-81/077 "Improving Subdrainage and Shoulders of Existing Pavements - State of the Art"

FHWA/RD-81/079 "A Pavement Moisture Accelerated Distress (MAD) Identification System, Volume I"

FHWA/RD-81/080 "A Pavement Moisture Accelerated Distress (MAD) Identification System, Volume II (User Manual)"

FHWA/RD-81/122 "Structural Analysis and Design of PCC Shoulders"

Sufficient copies of the report are being distributed to provide a minimum of two copies to each regional office, two copies to each division office and three copies to each State highway agency. Direct distribution is being made to the division offices.

Charles F. Scheffey
Director, Office of Research
Federal Highway Administration

NOTICE

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16. Abstract

Laboratory, test track and field results on channeling and pumping are presented. The influence of water on pumping and erosion of fine material from beneath the pavement surface is clearly shown.

Methods for pavement drainage design in expansive soils and to resist the effects of frost action are presented. It is shown that the depth and location of subdrain systems will influence the water flux.

Equipment and procedures for the maintenance and cleaning of subdrain pipes are discussed. A hydraulic jet unit for cleaning pipe subdrains has been discussed in detail.

The influence of curb and gutter on pavement drainage has been briefly discussed. Curb and gutter construction is normally used for roadway erosion control or in urban areas.

A drainage design philosophy is presented which explains the importance of predicting and controlling water contents in pavement drainage design. The methods for controlling water contents in design consisted of protecting the pavement structural section, rendering pavement materials insensitive to water or evacuating the water from the pavement section.

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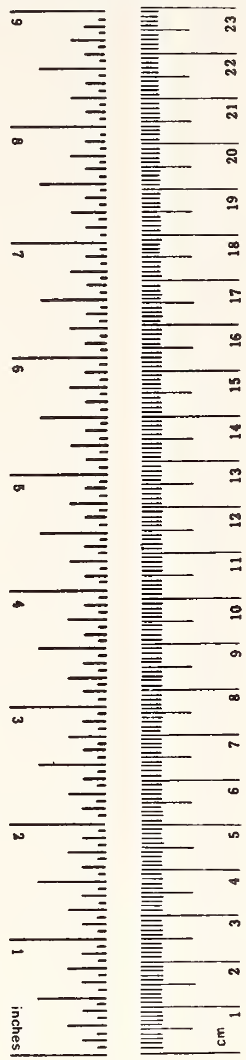
METRIC CONVERSION FACTORS

Approximate Conversions to Metric Measures

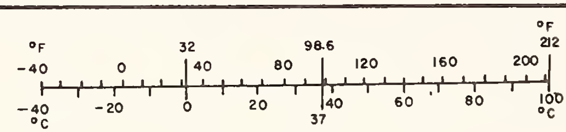
Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
in	inches	*2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
		AREA		
in^2	square inches	6.5	square centimeters	cm^2
ft^2	square feet	0.09	square meters	m^2
yd^2	square yards	0.8	square meters	m^2
mi^2	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 NBS Misc. Publ. 286, Units of Weights and Measures, Price $2.25, SD Catalog No. C13.10:286.



Approximate Conversions from 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



Chapter 1

INTRODUCTION

In the past, the structural design of pavement systems have been mainly concerned with limiting the deflections and stresses caused by wheel loads. While the influence of climatic factors such as water and temperature have been acknowledged, pavement engineers have seldom regarded their basic effects on pavement design and performance.

Although improvements in structural design have been made, investigations of rigid and flexible pavement systems show that water is still a major factor causing distress and loss of serviceability. Numerous cases can be found where water in the pavement structural section has reduced the strength of the material layers through saturation and subsequent reduction in effective stress or as a result of excess pore water pressures created by wheel loads. In the process of relieving pore water pressure, free water will often times erode and eject pavement material at cracks and joints.

In flexible pavements the primary water problem is associated with weakening of the subgrade and pavement structure, while in rigid pavements water causes erosion and ejection of subgrade or subbase materials through pumping action of the slabs.

As a result of renewed emphasis on the need for improved pavement drainage,research entitled "Improving Subdrainage and Shoulders of Existing Pavements" was completed to develop methods for minimizing the infiltration of surface water into the structure of existing pavements and to provide methods for draining water that does infiltrate and become trapped in the pavement system.

As part of this research project the following publications have been prepared to satisfy task requirements:

1. Improving Subdrainage and Shoulders of Existing Pavements State of the Art (Task A) (FHWA/RD-81/077)
2. A Pavement Moisture Accelerated Distress (MAD) Identification System, Volume I (Task B) (FHWA/RD-81/079)
3. A Pavement Moisture Accelerated Distress (MAD) Identification System Users Manual, Volume II (Task B) (FHWA/RD-81/080)
4. Structural Analysis and Design of PCC Shoulders (Task D) (FHWA/RD-81/122)

Chapter 2

OBJECTIVES

The general objective of this final report is to describe study results for the research tasks not covered by the publications listed previously. In this report the results of each remaining study task are presented in separate chapters as follows:

1. Chapter 3, Channeling and Pumping (Task C - Determination of Channeling Conditions, Task I - Determination of Saturated Hydraulic Conductivities of Open Graded Bases, Task J - Analysis of Task C Data and Channeling Studies in Illinois Test Track).
2. Chapter 4, Pavement Drainage Design in Expansive Soils (Task G - Swelling Soils, Task F - New Concepts for Providing Improved Drainage and Pavement Structural Sections, Task D - Alternative Shoulder/Drainage Systems).
3. Chapter 5, Climatic Design Considerations in Subsurface Drainage (Task G - Differential Frost Heave, Task F - New Concepts for Providing Improved Drainage and Pavement Structural Sections).
4. Chapter 6, Effectiveness of Curb and Gutter (Task H - Effectiveness of Curb and Gutter).
5. Chapter 7, Relieving Blocked Subdrainage (Task E - Determination of Methods for Relieving Blocked Subdrainage).
6. Chapter 8, Design Philosophy for Water in Pavement Systems.

Chapter 3

CHANNELING AND PUMPING

3.1 INTRODUCTION

Channeling is defined as the localized erosion of base, subbase, or subgrade materials caused by flowing water. To occur, it is necessary to have a concentration of water great enough to produce the necessary flow, an erodible material, and sufficient velocity of flow to cause erosion. In contrast pumping occurs when the pore water pressure buildup induced by heavy wheel loads is high enough to cause the ejection of material and water through cracks and joints in a pavement slab.

The presence of erodible material and water in the base, subbase, or subgrade hastens the pumping process and leads to loss of foundation support. Recent research work by Phu and Ray (1) indicates that the velocity of ejected water through channels, cracks, and joints can be high enough to cause disintegration, erosion, and subsequent pumping of stabilized bases in addition to unbound bases in rigid pavements

Large deflections combined with excessive moisture contents in the support materials beneath the slab are major problems affecting rigid pavement performance. All failures in the rigid pavement sections at the AASHO Road Test were preceded by pumping of material from beneath the concrete slab (2). Both Yoder (3) and Cedergren (4) noted damage to airport runways and taxiways caused by pumping. These observations were especially prevalent where pavement overload and channelized traffic had occurred. The major forms of distress modes in rigid pavements associated with pumping have been identified by many investigators as faulting (5,6,7,8), corner breaking (9,10), transverse and diagonal cracking (11,12), and edge punchout (13). A more thorough discussion of pumping and other water related distresses in pavement systems can be found elsewhere (14).

In order to evaluate the mechanisms of channeling and pumping in rigid pavement systems both laboratory and field investigations were conducted. The laboratory investigation was conducted on a model pavement section and on the University of Illinois test track facility. Much of the field investigation was accomplished for pavements surveyed in Illinois.

3.2 LABORATORY MODEL INVESTIGATION OF CHANNELING AND PUMPING

3.2.1 GENERAL

A laboratory study of the mechanisms and results of channeling and pumping in a model pavement system subjected to periodic, short duration loading was completed. Load-induced pore water pressures in the pavement system were examined with respect to both time and position in the pavement. The study was conducted on a pavement model composed of a subgrade, crushed limestone base, and portland cement concrete surface with an asphalt concrete shoulder. Channeling and pumping were investigated for three test series using different gradations of crushed limestone as the base course material.

3.2.2 EQUIPMENT DEVELOPMENT

In order to measure the dynamic pore water pressures in the pavement model it was necessary to develop a piezometer which could indicate the change in pressure at the instant loading was initiated. Conventional piezometers which employed porous stones or similar devices were not adequate because of clogging by the fine-grained materials of the base and slow response time. Piezometers which utilized membranes gave erroneous results because of the effective soil pressures against the membrane at the instant of loading. After several pilot studies a piezometer shown in Figure 3.1 was developed. The piezometer consisted of a small hollow sphere composed of two halves which fit snuggly together. Small holes about 1 mm (0.04 in.) in diameter were placed in the outer half of the sphere. The inner half of the sphere was connected to a small stainless steel tube which was in turn connected to a pressure transducer. A thin rubber membrane was stretched between the two sphere halves before they were sealed together. The inner spherical half and tube to the pressure transducer was then filled with water so as to transmit the pressure impulses to the transducer. When placed in a saturated soil or granular material the effective stress generated by the load is resisted by the sphere while the pore water pressure can be transmitted through the small holes to the membrane.

Preliminary tests showed that the piezometer functioned well for pore water pressures ranging from 3.4 to 137.9 kPa (0.5 to 20 psi) and pressure durations of 0.1 to 1.0 seconds. Pressure tests between an unprotected piezometer and the designed piezometer showed excellent correlations. However, the unprotected piezometer became plugged with fine material after only a few hundred load applications while the designed piezometer remained unplugged and operational after 20,000 load applications.

In order to observe channeling and pumping in the base material during repeated loading, a transparent, plexiglass tank shown in Figure 3.2 was constructed. Holes were drilled through the side of the tank for installation of the piezometers. The piezometers were extended to the center of the tank about 12.7 cm (5 in.) from the inner face so that boundary conditions could be kept to a minimum, Figure 3.3.

3.2.3 PAVEMENT MODEL

Figure 3.4 shows a profile view of the pavement model as placed in the plexiglass tank. An AASHTO A-6 soil was used as the subgrade material for the pavement model. The subgrade was compacted into the tank in 10.2 cm (4 in.) lifts at an average dry density of 1882 kg/m^3 (117.5 pcf) and water content of 14.8 per cent based on AASHTO T-99. The maximum dry density of the soil was 1698 kg/m^3 (106 pcf) and optimum moisture content was 14 per cent. Piezometers were placed in the subgrade at the locations shown in Figure 3.4.

Three pavement model tests were conducted using three different crushed limestone gradations as the base course. In test order, the base courses consisted of Illinois CA-7, CA-6, and CA-9 gradations. Table 3.1 shows the gradation specifications for the different base course materials. Table 3.2 shows the gradations and Figure 3.5 shows the gradation curves for the materials actually used in the model tests. Table 3.3 shows the saturated hydraulic conductivity (coefficient of permeability) for the three aggregate base course gradations. As shown in Figure 3.4 each base course was constructed to a thickness of 12.7 cm (5 in) and instrumented with piezometers at locations P1 through P15. The subgrade was not replaced during the series of base course tests. The base course was completely saturated before placement of the concrete surface.

A 12.7 cm (5 in) plain portland cement concrete slab was cast in place on the base for each of the three test series and allowed to cure at least seven days prior to loading tests. The mix design for the concrete slab is shown in Table 3.4.

A 12.7 cm (5 in) thick asphalt concrete shoulder section was placed next to the concrete stab as shown in Figure 3.4. The joint between the concrete slab and asphalt concrete shoulder was not sealed. The concrete slab was loaded through a 20.3-cm (8-in.) diameter steel plate placed 35.6 cm (14 in.) from the joint between the shoulder and concrete slab.

3.2.4 TESTING PROCEDURE

The testing procedure was developed to determine the pore pressure in the saturated base material and to evaluate channeling and pumping caused by repeated load. Figure 3.6 shows the pavement model prepared for dynamic testing. The piezometer system was connected to a multichannel recorder. The surface of the pavement was covered with water to insure that the granular base course and subgrade remained saturated during the test.

The pavement section was loaded so that each pressure level exerted by the 20.3-cm (8-in.) diameter plate was held constant regardless of the slab deformation. The load was increased from 137.9 kPa (20 psi) to 551.5 kPa (80 psi) in 137.9 kPa (20 psi) increments to provide four pressure levels. Each pressure level was maintained for one hour with 4 seconds between loads and a load duration of 0.1 second. After testing at the 0.1 second duration, the duration was increased to 1.0 second and the slab was loaded for 5 minutes at each of the pressure levels. After initial testing the pavement model was continually loaded at one of the pressure levels for 24 hours with 4 seconds between loads and a load duration of 0.1 second. The piezometer readings were monitored at different time intervals during each test period. A tracer dye was injected into the base course at various times so that flow paths and channels could be traced.

After each pavement model had been tested with dynamic loading, the concrete slab and asphalt shoulder were removed and the base course was examined. Base course samples were obtained at different locations in the pavement model and a gradation analysis was conducted.

3.2.5 ANALYSIS AND DISCUSSION OF MODEL STUDY RESULTS

Several piezometers were placed in the subgrade with the hope of recording the change in pore water pressure during loading. However, because of the high density and low permeability of the A-6 subgrade complete saturation was not achieved and the change in pore pressures could not be recorded.

The first gradation of base material tested was the CA-7. As determined from Tables 3.2 and 3.3 and Figure 3.5 the CA-7 would be considered a coarse graded material with reasonably high permeability.

In the CA-7 base, it was found that the pore pressures generally decreased with distance and depth from the load. It was found that at no time and at no point in the base did the change in pore pressure exceed 2.1 kPa (0.3 psi). No specific paths of water movement were determined except for the fact that the water near the sides and near the concrete slab and asphalt shoulder joint was forced up during loading. It was also found that different load durations of 0.1 second or 1.0 second had little effect on the excess pore pressure in the CA-7.

The CA-6 base course test provided different results than the CA-7 and pore pressures changes in excess of 20.7 kPa (3 psi) were observed during loading. Tables 3.2 and 3.3 and Figure 3.5 show the CA-6 to be a dense graded aggregate with a low permeability. The variations in pore pressure were extremely erratic with position in the base course and no pattern could be discerned. However, separate and distinct flow paths were seen in this base course material as shown in Figure 3.7. It appeared that small air bubbles trapped between the particles during saturation would rise to the surface during loading and create a path which water, carrying soil particles, would follow. Large amounts of fine-grained material, all passing the No. 200 sieve, were pumped along the sides of the slab and through the joint between the slab and the shoulder to the surface of the slab.

The pore water pressure results from the test using the CA-9 base course material were similar to those using the CA-6 material and peaked at about 20.7 kPa (3 psi). As seen in Table 3.2 and Figure 3.5, the CA-9 has an intermediate gradation between the CA-7 and CA-6 material. The CA-9 is not considered to be a highly permeable material as shown in Table 3.3. Figure 3.8 shows some of the fine material from the CA-9 being moved towards the bottom of the joint between the concrete slab and asphalt concrete shoulder. As found with the CA-7 base course, load durations of 0.1 second or 1.0 second had little effect on the excess pore pressures in the CA-6 and CA-9 base courses.

Figure 3.9 shows the excess pore water pressure at several positions in the CA-6 base course as the load is increased from 0 to 551.5 kPa (0 to 80 psi) (see Figure 3.4 for piezometer locations). The general trend was an increase in pore pressure as the load increased and a diminishing rate of increase as the load became larger. The highest pore pressure occurred directly below the loading plate. The somewhat erratic changes in pore pressure recorded at P2, P12 and P14 may be caused by instrumentation error or by changes in the base course material during the loading sequence. It was found during testing that the pore pressures would increase uniformly with load as long as the local aggregate structure remained unchanged.

Figure 3.10 shows more clearly the influence number of load applications at various stress levels has on pore water pressures in CA-6 and CA-9 base courses. Figure 3.10 shows a general trend for the pore pressure to increase as the number of loads increase. Figure 3.11 shows more erratic pore water pressure changes with load applications on the CA-6 and CA-9 materials. The fact that the pore water pressures vary considerably with position in the base may indicate that channels are created which allow passage of water and the transfer of fine materials as noted in Figures 3.7 and 3.8.

It would appear that the magnitude of pore water pressure is an important factor affecting degradation, channeling, and pumping in granular base course materials. No pumping or degradation was observed in the CA-7 base course where the pore water pressures never exceeded 2.1 kPa (0.3 psi). However, the base courses composed of CA-6 and CA-9 experienced pore water pressures greater than 20.9 kPa (3 psi) and displayed pumping and degradation problems.

According to Phu and Ray (1) aggregate degradation and pumping can be expected in unbound base courses when the water velocity exceeds 5 m/s (16.4 ft/s). Defining the water level at the surface of the pavement as the datum and assuming the surface velocity as zero Bernoulli's equation for steady, frictionless, incompressible flow can be presented as follows:

$$Z_p + \frac{V_p^2}{2g} + \frac{P_p}{\gamma_w} = 0 \qquad (3.1)$$

where

Z_p = depth from the datum to the location in the base,

V_p = water velocity,

g = acceleration of gravity,

P_p = pore water pressure, and

γ_w = unit weight of water.

By solving Equation 3.1 for a depth of 30.5 cm (12 in.) below the datum and a pore pressure of 20.7 kPa (3 psi) it is found that the water velocity will be 6.9 m/s (22.6 ft/s). This value exceeds that established by Phu and Ray (1) for degradation and pumping. A similar computation for the CA-7 base course using a pore water pressure of 2.1 kPa (0.3 psi) gives a velocity of 3.2 m/s (10.4 ft/s) which is below the criteria of Phu and Ray (1).

The build-up of pore water pressures in the CA-6 and CA-9 base materials was a major cause and/or result of pumping, degradation of the aggregate, and deformation of the concrete slab. There was no appreciable pore water pressure build-up in the open graded CA-7 aggregate and there was no observed pumping or aggregate degradation. However, there was considerable slab deformation on the CA-7 base course. Inspection of the CA-7 base material after testing showed that substantial amounts of A-6 subgrade material had intruded into the lower portion of the base course. Subgrade intrusion was not found to be a problem in the CA-6 and CA-9 base course materials and permanent deformation of the slabs on these materials was actually less than that on the CA-7. Examination of the changes in gradation of the base course materials before and after testing showed trends which indicated channeling, degradation, and pumping.

Table 3.2 and Table 3.5 show the base course gradations before and after testing respectively. The gradations shown in Table 3.5 were determined directly beneath the center of the load. All aggregate gradations used were within the limits of the Illinois Standard Specifications as listed in Table 3.1 prior to testing.

Figures 3.12, 3.13, and 3.14 show before and after testing results at various locations in the CA-6 base course material used in the pavement model. Figure 3.12 indicates that considerable change in gradation occurred at the concrete base-base course interface (Piezometers P1, P2, P3, P4, and P5, Figure 3.4) during repeated loading. The relative change in gradation indicates that most of the material passing the 10 mm (3/8 in.) sieve size was displaced or pumped from directly beneath the slab. Figure 3.13 shows CA-6 base course gradation changes beneath the load (Piezometers P3, P8, and P13, and Figure 3.4). In general there was some decrease in the percentage of fine material passing the No. 4 sieve. The gradation changes in Figure 3.13 would indicate some displacement and pumping of fine base course material. Figure 3.14 shows relative base course gradation changes directly beneath the joint between the concrete slab and asphalt concrete shoulder. It is noted that the percentage of base course material less than 10 mm (3/8 in.) has decreased substantially. Visual inspection of the base course also indicated that a considerable amount of fine material had been pumped or displaced. Some of the fine material along the top of the base course was found to migrate towards the bottom of the base. Also large accumulations of base course fines were found on the surface of the concrete slab and shoulder at the conclusion of testing.

Figures 3.15, 3.16, and 3.17 show the relative gradations for the CA-9 base course material before and after testing. Figure 3.15 shows that substantial gradation change occurred directly beneath the concrete slab (piezometers P1, P2, P3, P4, and P5, Figure 3.4) similar to that shown in Figure 3.12. It is hypothized that the fine material migrated towards the bottom of the base course or was pumped out. The decrease in percentage of larger material would indicate that some degree of degradation may have occurred.

As with the CA-6 material the CA-9 material passing the 10 mm (3/8 in.) sieve size was displaced or pumped. There was not a large amount of change in the base course gradations directly under the load (piezometer P3, P8, and P13, Figure 3.4) or under the pavement edge-shoulder joint shown in Figure 3.16 and Figure 3.17 respectively. However, there was a substantial increase in the material smaller than about 2 mm (0.08 in.) which would indicate that some degradation had occurred. Substantial fine material was also observed on the surface of the pavement slab and shoulder at the completion of the test.

Inspection of the CA-7 base course material showed that there was no major gradation changes resulting from repeated loadings. No gradation sampling was accomplished in the lower portion of the base where subgrade intrusion had occurred however.

In the testing program, the CA-9 base course was tested under 15,000 load applications while the CA-6 base course was tested under 35,000 load applications. The difference in the number of loads resulted from mechanical difficulties during testing of the CA-9 material. Also there is no doubt that some of the differences in the base course gradations before and after testing were caused by testing errors and material variability.

Although there are probably some gradation differences resulting from the testing methods there are some notable trends in the data. In both the CA-6 and CA-9, the materials larger than the 10 mm (3/8 in.) sieve size did not degradate and pump whereas those smaller experienced degradation and pumping. This is substantiated in the test with the CA-7 base material where approximately 95 percent of the material was retained above the 10 mm (3/8 in.) sieve size and no pumping or appreciable gradation changes occurred. The largest gradation change occurred directly beneath the slab on the base course surface followed by change in gradation below the shoulder. A smaller gradation change is noted as samples are taken deeper into the base course beneath the load, or below the shoulder. In the pumping observations water and soil directly beneath the slab were forcefully pumped up along the sides of the slab and up through the joint between the slab and the shoulder. The major flow path in the model appeared to be towards the shoulder and up through the joint with less material being moved as distance from the shoulder increased. Visual inspections of the model during loading revealed that as the number of loads increased water turbulence became more evident under the slab and near the shoulder. Even though a clear and concise distribution of excess pore pressures could not always be recorded, it was found that the maximum pressures were developed near the base of the slab and near the shoulder (along the path of pumping).

The development of flow channels were observed in the granular base materials. Although these channels provided for water movement, and therefore the dissipation of pore water pressures during loading, they also provided a pathway for the movement of fine materials. Since the channels contribute to the displacement and pumping of fine base and subgrade materials they are probably not an asset to pavement performance.

Examination of the permeabilities of the base materials helps to explain whey the CA-6 and CA-9 developed higher excess pore pressures than the CA-7. Table 3.3 shows that the CA-7 is much more permeable than the CA-6 and the CA-9. Consequently the dissipation of excessive pore pressures was extremely rapid in the CA-7 and the base course did not experience water channeling or pumping.

3.3 TEST TRACK INVESTIGATION

3.3.1 GENERAL

As a bridge between laboratory testing and full scale road tests, the testing of prototype pavement sections in a traffic simulating testing operation can sometimes provide much information. Because the prototype sections are nearly full scale, tests performed on the test pavement provide information indicative of field behavior. Since close control of construction, climate and traffic loading is possible, the performance of the pavement and the effect of the variables can be determined.

Ahlberg and Barenberg (15) have reported on the use of the University of Illinois Test Track to model pavement systems and evaluate pavement design and performance. The test track has been found to provide a flexible method for evaluating the influence of water and drainage on pavement performance.

In an agreement between the University of Illinois and the New Jersey Department of Transportation the test track is being used to conduct a study entitled "Evaluation of Drainage Systems for Pavements". This study, under the supervision of Dr. Barenberg, is concerned with the evaluation of open-graded bases under flexible and rigid pavements.

As part of the study to evaluate channeling and pumping, it was requested to analyze a number of New Jersey pavement sections being tested at the Illinois Test Track facility.

The specific objectives of the study were:

1. Measure and evaluate the saturated hydraulic conductivity of open-graded bases.
2. Investigate channeling in the pavement system.

3.3.2 TEST TRACK

A schematic diagram of the University of Illinois Test Track is shown in Figure 3.18. The test track has a 3.81 m (12.5 ft) radius at the outer wall and 1.37 m (4.5 ft) radius at the inner wall with the wheel path ranging from 0.99 m to 1.75 m (3.25 ft to 5.75 ft) from the outer wall.

The depth of the pit is 1.22 m (4 ft) and the width of the pavement perpendicular to the direction of travel is 2.44 m (8 ft). The circular track can be divided into sections with six rectangular sections 1.22 m (4 ft) in length parallel to the direction of travel being the practical limit.

Construction of the test sections and the transition zones between them is done at scale similar to that of actual pavements. This procedure allows for more uniform control and better simulation of actual pavement construction. Mixing equipment prepares batches of material 136 kg (300 lb) or greater. The material is then distributed in the appropriate section and leveling frames trim the material to the desired thickness in each section. Compaction is done with small maintenance equipment such as vibratory plate compactors, vibratory rollers, or mechanical impact compactors.

To maintain close control of the construction process a level reference datum is part of the pit walls. Bolt holes in the walls provide a method to anchor reaction beams for CBR and plate loading tests. Along with laboratory tests on materials and pavement cores these data can be correlated to the several performance indicators measured. Such indicators include: deflection of the pavement due to dynamic wheel loads, roughness, rutting depth,and load applications to failure.

3.3.3 PAVEMENT TEST SECTIONS

It was proposed to test six asphalt pavement sections in the first phase and four portland cement concrete pavement sections during the second phase. However, only the six asphalt pavement sections have been completed at this time and the concrete pavement sections will not be completed until a later date.

The six asphalt concrete pavement sections are described in Table 3.6. The original design thicknesses are scaled down so that the 1451 kg (3200 lb) single wheel load (551.5 kPa (80) psi tire pressure) used in the test track induces a similar stress-strain response as would occur with a 4082 kg (9000 lb) dual wheel load.

A special feature of the drainage layer study was the need to test the hydraulic capabilities of the test sections. To facilitate these measurements, plastic sheeting was placed on each side of the cross section from the silty-clay (AASHTO A-6) subgrade to the bituminous base course material (BSBC) and extended from the inner to outer pit wall. Next to the pit walls each section was provided with an inflow trench so that water could be input uniformly into the layer immediately beneath the BSBC material. For the sections with open-graded drainage layers water was confined by impermeable surfaces so that it flowed into the drainage layer a distance of 30.5 cm (1 ft) after which it could then flow as it would in the field (section had a 1 per cent transverse slope). An outflow trench was constructed next to the outside wall for each of the six pavement sections.

The prototype test sections can be expected to fail at the same number of load applications as the originally designed pavement. If the percentage of time the test sections are exposed to infiltration is similar to field conditions, the response of the sections to the traffic loading can be used in recommendations for actual field design.

3.3.4 SATURATED HYDRAULIC CONDUCTIVITY TEST PROCEDURE

Small piezometric tubes were placed at various locations in the drainage layers in different segments of the test track for the purpose of measuring the hydraulic heads necessary for determining the saturated hydraulic conductivity. It was also proposed to use the piezometric tubes to determine the magnitude of the pore water pressures in the open-graded bases as a result of wheel loads. The numerous piezometric tubes installed in the test track were monitored by use of a rotating fluid switch which was connected to a pressure recording system.

3.3.5 TEST RESULTS AND DISCUSSION

The drainage layers evaluated in the six asphalt concrete pavement sections consisted of a bituminous stabilized open-graded layer (BSOG), non-stabilized open-graded layer (NSOG), and a well-graded crushed stone base (5A), Table 3.6. The BSOG was found to have an average saturated hydraulic conductivity of 1.8 cm/s (5000 ft/day) at the end of the test. The NOSG had a slightly higher conductivity of 1.9 cm/s (5400 ft/day). The well-graded crushed stone base (5A) had an average conductivity of $5.2\text{x}10^{-2}$ cm/s (15 ft/day). (It was noted that in some locations the 5A base had a conductivity less than $3.5\text{x}10^{-4}$ cm/s (1 ft/day).

Evaluation of the pavement sections based on rutting and roughness indicated that the NSOG layer gave the best performance. The BSOG layer performance was evaluated to be slightly below that for the NSOG. As expected the crushed stone base (5A) displayed poor performance.

There was no indication of pumping in any of the test sections and there was no indication of channeling in the drainage layers when the pavement sections were excavated after completion of the test.

A more thorough discussion of the data and findings of this study will be published in a final report to the New Jersey Department of Transportation.

3.4 FIELD INVESTIGATION

3.4.1 GENERAL

Both qualitative and quantitative field investigations of pumping and channeling were conducted in Task C of the project on Improving Subdrainage and Shoulders of Existing Pavements. Pumping was found to be distinguishable and major cause of distress in concrete pavements in numerous states. The evidence of channeling in pavement systems was not easily determined and not as apparent as pumping when evaluating distress mechanisms.

3.4.2 FIELD INVESTIGATION RESULTS

Evidence of pumping and loss of material beneath concrete pavement sections was found in climate ranging from semiarid to wet. It was apparent that both the frequency and magnitude of rainfall were related to pumping.

It was also found that pavements located on long gradients in rolling topography were often most vulnerable to pumping and bleeding of water and fines from the support materials. Figure 3.19 shows water and fine material flowing out from beneath a jointed concrete pavement system located on a long gradient. It is probable that water channeling also exists in this pavement system. In some areas with rolling topography, pavement cuts through the top of a hill will often intercept groundwater sources which will cause water flow along the pavement profile. This condition is easily recognized by the growth of cattails or other water plants in the ditches at the crest of the hill.

Figure 3.20 shows that pumping can be associated with continuously reinforced pavement systems as well as jointed pavement systems when the distress state becomes severe. Furthermore, surveys of pavements in Illinois showed evidence of disintegration and pumping in subbases composed of cement aggregate mixtures (CAM) and bituminous aggregate mixtures (BAM). Figure 3.21 shows disintegrated BAM which has been pumped out from beneath a section of continuously reinforced concrete pavement.

Figure 3.22 displays a cement treated subbase after the concrete pavement slab has been removed (16). The channeling that has occurred at the interface between the slab and cement treated subbase can be noted by the light areas formed by grout injection.

Figure 3.23 shows a section of pavement being prepared for a patch. Even though there was a longitudinal subdrain next to the pavement edge the water did not freely drain from the pavement section. Even after several hours, Figure 3.24, water was still visible next to the pavement edge. Inspection of the pavement section indicated that fine material was responsible for blocking the flow of water to the longitudinal subdrainage trench, Figure 3.25.

Field investigations generally showed that water related distresses in the mainline pavement also extended to the shoulder. Figure 3.26 shows shoulder drop-off resulting from base course or subgrade fines being ejected out from beneath the shoulder. Disintegration and erosion of the shoulder material along the pavement-shoulder joint as shown in Figure 3.27 were evident in numerous pavement systems inspected. These joint openings would often be over 2.5 cm (1 in.) wide and water could frequently be seen in the joint.

Field investigations and drainage studies by Dempsey and Robnett (17) have indicated that the pavement-shoulder joint contributes to most of the water infiltration into a pavement system.

Subdrainage outflow studies on pavement test sections in Georgia and Illinois, Table 3.7 indicated that water infiltration into the pavement section could be substantially reduced by edge sealing. Table 3.8 shows outflows measured at the two test sites and indicates the differences in infiltration volume for unsealed and sealed edge joint conditions. In the Georgia G2 section both the transverse and longitudinal joints were sealed, and no measurable outflow was measured over the duration of the test.

Figures 3.28 and 3.29 show relationships between precipitation and outflow volume for two study periods at the Georgia test site. It is noted that the outflow occurs almost simultaneously with precipitation for the unsealed edge joint condition in Figure 3.28. In Figure 3.29, for the sealed edge joint condition the outflow lags precipitation by several hours and the outflow is considerably less (1.7 percent) than that shown in Figure 3.28 for the unsealed condition (20.4 percent). A more detailed discussion of the pavement test site study can be found elsewhere in the literature (17).

3.4.3 ANALYSIS AND DISCUSSION OF FIELD RESULTS

In a National Cooperative Highway Research Project, Darter and Snyder (18) have studied the distress types and mechanisms of 966 km (600 mi) of jointed reinforced concrete interstate pavement in Illinois. Out of 204 projects surveyed they found 137 which displayed evidence of pumping, Table 3.9. The severity of the pumping ranged from low severity to high severity as shown in Figure 3.30.

Although the jointed reinforced concrete pavement sections surveyed had granular subbases, a survey of continuously reinforced concrete pavements in Illinois showed evidence of disintegration and pumping of the cement aggregate mix (CAM) and bituminous aggregate mix (BAM) subbases (13). In the report summarizing the results of the field survey the following quote was made concerning the BAM subbase which is predominently used in continuously reinforced concrete pavement systems:

> "During the field surveys, numerous edge punchouts were observed as the maintenance crew removed the broken concrete. There was frequently a considerable amount of free moisture within the failure area and the BAM subbase was disintegrated and had been intruded by fine-grained soil."

A major result of pumping is the loss of support under the concrete slab which can cause high stresses and deflections which then lead to an accelerated rate of distress. Darter, LaCoursier, and Smiley (19) computed the critical stresses and vertical deflections of a slab as a function of foundation erodibility. Figure 3.31 shows that as loss of support increases, the maximum stresses and deflections increase, and the point of maximum stress moves in from the edge. For example as the loss of support moves in 0.6 m (2 ft) from the pavement edge the tensile stress in the slab increases

132 percent and edge deflection increases about 75 percent. This loss of support is responsible for much of the faulting, longitudinal cracking, corner cracking, and punchouts observed in concrete pavement systems.

Laboratory studies and field observations indicate that the pumping mechanisms vary depending on the type of base course used. Majidzadeh (20) found a broad range of drainage parameters influencing pumping and channeling in Ohio pavements. From laboratory and field studies, it is observed in unbound granular base courses that fine materials are generally displaced throughout the full depth as a result of load induced pore water pressures. In stabilized bases such as CAM and BAM the fine materials are generally removed or eroded from the interface between the concrete slab and stabilized base. In both the unbound bases and stabilized bases there can be either intrusion or pumping of the subgrade materials.

In a recent study of problems raised by the presence of water in concrete pavements, Phu and Ray (1) developed relationships for determining the critical water velocity which would cause erosion of the subbase course. Table 3.10 shows the relative water velocities causing erosion in unbound and stabilized subbases subjected to 0 and 9 freeze-thaw cycles. Phu and Ray (1) indicated that an unbound gravel does not resist the expulsion of water on the order of 5 m/s (16.4 ft/s) and pumping of the fines will occur rather frequently. They indicated that pumping at this water velocity is more severe in pavements without dowels than those with dowels.

Figure 3.32 shows the influence of compaction on erodibility of a cement treated gravel with and without freeze-thaw cycles. Both freeze-thaw cycles and decreased compaction caused an increase in the amount of material eroded. Figure 3.33 shows more clearly the influence number of freeze-thaw cycles has on the erodibility of cement treated gravel.

Phu and Ray (1) have found that even water velocities ranging from 0.1 m/s to 1 m/s (0.32 ft/s to 3.3 ft/s) were sufficient to displace fines from the top to the bottom of the subbase or from the interior towards the exterior pavement joints. During the study of partial displacement Phu and Ray (21) found that a suction pressure is often produced in the water beneath the concrete slab with passage of load. The development of suction pressures were also found with the passage of load during laboratory testing of the pavement model in this project. Phu and Ray (21) found in their study that the magnitude of water pressure beneath the concrete slab was related to vehicle speed. In general higher water pressures occurred at speeds of 20 to 40 km/h (12 to 24 mi/h) and decreased as speeds approaching 60 km/h (37 mi/h) were reached.

Figure 3.34 shows that ejection of water as a function of pressure is influenced by the joint opening. In general there is a decrease in the water ejection velocity as the size of the joint opening increases. Important factors causing pumping are slab curling and the development of a cavity beneath the concrete pavement slab. Figure 3.35 indicates that a water filled cavity of 0.5 mm (0.02 in.) or greater can create expulsion water velocities great enough to cause pumping under normal loading conditions.

It is for this reason that eroded cavities beneath concrete pavements must be sealed to prevent serious water problems after pavement rehabilitation.

When evaluating pumping problems in jointed concrete pavements it is important to understand the influence of slab thickness. Based on work at the Central Laboratory of the Ponts et Chaussees (22) the following quotation is presented concerning concrete slab thickness:

> "Today, studies based upon the observation of stair-stepping (faulting) and the precise recording of deformations on experimental thick-slab sections have demonstrated qualitatively that the dissymetry of slab-end deformations was not substantially modified and that an increase in thickness could only change the stair-stepping rate. For example, a 10-percent increase in slab thickness can reduce stair-stepping by only about 25 percent when the subbase is treated.
>
> In current design practice, slab thickness is determined so as to avoid slab fatigue cracking, and not with a view to optimum load transfer at joints. As regards pumping and stair-stepping, they are usually approached in terms of factors other than slab thickness. Further, several countries do not take the existence of dowels directly into account in the practical calculation of motorway slab thicknesses. In fact, if joint behavior is to be sufficiently modified, the cost of increasing the thickness of the concrete slab is such that other arrangements are more effective (less erodable foundation, dowels for example). Structural design studies in different countries moreover confirm this observation. An increase in thickness is considered only when it allows other simplifications, which is the case for example with very thick slabs resting directly on a porous subgrade (4/60 mm particle size, for example)."

In order to show the interaction between traffic, climate, subbase materials, and joint load transfer the Ponts et Chaussees (22) developed the three-dimensional space representation shown in Figure 3.36. In Figure 3.36 the background of the technical modifications applied to concrete pavements since 1930 becomes understandable:

1. The significant stair-stepping (faulting) observed between 1920 and 1936 led several countries in Europe and several states of the United States, all having humid climates, to the adoption of dowels (Figure 3.36: line representative of this evolution: A → B).

2. In the past 5 to 15 years, efforts for obtaining good performance even under very intense traffic have led these same countries to prefer treated materials to untreated materials (Figure 3.36: B → C).

3. In this situation, and particularly when waterproofing is properly maintained and/or a drainage system is provided at the slab-subbase interface, performance can be excellent (Figure 3.36: C → D).

4. Several western states of the United States having a dry or medium climate adopted, toward 1945, the use of cement-treated materials with nondowelled slabs (Figure 3.36: E → F).

5. Having observed further stair-stepping in certain cases, between 1968 and 1975, they decided on the use of lean concrete in the sub base (Figure 3.36: F → G), along with drainage and even waterproofing (Figure 3.36: G → E), and this should also lead to excellent performance.

6. Similarly, but in a more humid climate, certain European countries adopted, in the 1960's, the technique of nondowelled slabs over a cement-treated subbase (Figure 3.36: I).

7. It would appear that by acting on the "erodability" factor <u>and</u> on the "water" factor these countries will also be able to achieve good performance if they wish to conserve the simpler technique without dowels (Figure 3.36: I → G → H).

This three-dimensional representation also makes it possible to visualize two important qualitative tendencies:

1. The drier the climate, the smaller the effect of erodability on the variation in the acceptable traffic limit with or without dowelling.

2. The more humid the climate, and the more erodable the subbase, the more valuable will be the use of dowels.

Based on a four year drainage study of new and existing concrete pavements in France, Griselin (23) summarized the findings as follows:

> French concrete pavements, in particular old ones built on erodable subbases, sometimes undergo a rapid modification linked with the presence of water between the concrete slab and the subbase: pumping, fines getting out, stepping. Since 1973 experiments have been performed with lateral drainage trenches in porous materials to evacuate infiltrated water. The earliest on-site experiments on deteriorated pavements showed that when the pavement has become modified beyond a certain limit, the setting in place of lateral drainage can accelerate phenomena of deterioration.

Subsequent experiments, coordinated by a working group comprising laboratories and those responsible for the new construction or the maintenance of pavements, showed the value of lateral drainage.

In the case of new pavements, the provision of lateral drainage prevents almost all deteriorations due to pumping, without risk to the pavement.

In the case of old pavements in good condition (essentially with a non-erodable subbase), four years of experiments showed that lateral drainage very markedly slowed down pumping and may increase the life of the pavement.

On the other hand, when the subbase is very erodable or when deteriorations are already considerable, a drain trench is not a suitable remedy.

The cost of making a drainage trench, which is very low by comparison with that of other maintenance procedures such as overlaying or resurfacing with bituminous concrete, whose efficacity is not always compatible with the cost, justifies the interest now taken in this solution in many countries and the need to continue investigations.

From the literature and research studies it is felt that subbases beneath existing pavement systems must be drainable if subdrainage systems are to be effectively used. It is hypothesized that subdrainage systems will have little influence on pore water pressure build-up and pumping in subbases with $K_s < 0.009$ cm/s (0.0003 ft/s), some influence for $K_s = 0.009$ to 0.09 cm/s (0.0003 ft to 0.003 ft/s), and appreciable influence for $K_s > 0.09$ cm/s (0.003 ft/s).

It is evident that climatic conditions are important to pavement rehabilitation and design. Field studies indicate that load transfer at the pavement joint will improve pavement performance. It is also apparent that the stability, strength, and drainability of the base course materials are very important factors influencing the long term performance of the pavement system.

3.5 SUMMARY AND CONCLUSIONS

3.5.1 SUMMARY

Conditions which cause pumping and channeling in pavement systems were studied in the laboratory and field. Laboratory pavement model tests indicated that dense graded crushed stone base courses would experience channeling and pumping under dynamic loading conditions. An open graded crushed stone base did not pump but subgrade intrusion caused permanent deformation of the pavement slab.

The University of Illinois test track was used to study six pavement drainage layers. It was found that pavements on both the non-stabilized and bituminous stabilized open graded layers performed well under repeated wheel load. The pavement on a well graded crushed stone base displayed the poorest performance.

Field investigations show that pumping continues to be a problem in pavements. However it is indicated that the use of load transfer at pavement joints, nonerodible base materials, good drainage practices, and consideration of climatic conditions can lead to pavements which will perform well during the design life.

3.5.2 CONCLUSIONS

From the research on channeling and pumping the following conclusions are presented:

1. Excess pore pressures, developed as a consequence of loading, will increase as the applied load increases.
2. Excess pore pressures will increase as the number of loads increase.
3. Excess pore pressures will increase as the permeability of the base course decreases.

4. Excess pore pressures beneath a pavement system with a fine-grained base material can reach at least 20.7 kPa (3 psi) which is sufficient pressure to pump fine-grained material to the surface.
5. Materials in the base of a pavement system smaller than the 10 mm (3/8 in.) sieve size are subject to degradation and pumping.
6. Although channeling may aid drainage it is detrimental from the standpoint of displacement of fine base and subgrade materials and pumping.
7. Climatic conditions must be considered in pavement rehabilitation and design.
8. Load transfer devices at pavement joints can decrease faulting and pumping.
9. Stabilized bases and subbases can improve pavement performance.
10. Stabilized bases and subbases must be designed to resist erosion by water.
11. Stabilized open graded bases and non-stabilized open graded bases in combination with a subdrainage system can improve pavement performance.
12. Subgrade intrusion must be considered when using open graded bases as subbases.
13. Pavement layer drainability must be considered in the design of subdrainage systems.

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Table 3.1 Illinois Specifications for Aggregate Gradations.

Gradation Number	Sieve Size Per Cent Passing 1-1/2"	1"	3/4"	1/2"	3/8"	#4	#16	#50	#200
CA6	100	95±5		75+15		45±10	25±15		8±4
CA7	100	95±5		45±15		5±5			6±6
CA9	100	97±3		60±15		30±15	10±10		

Table 3.2. Initial Aggregate Gradation Before Testing.

	Per Cent Passing		
Sieve Size	CA-7	CA-9	CA-6
1-1/2"	100	100	100
1"	98.1	97.5	93.0
3/4"	89.0	94.4	82.5
1/2"	64.0	65.5	73.5
3/8"	29.9	51.5	59.7
#4	4.4	30.0	43.8
#16	0.0	11.5	30.5
#50	0.0	2.1	11.3
#200	0.0	0.6	6.3
Pan	0.0	0.0	0.0

Table 3.3. Saturated Hydraulic Conductivity for Base Course Materials.

Aggregate Gradation Number	Saturated Hydraulic Conductivity	
	cm/sec	ft/day
CA-6	2.1×10^{-4}	0.6
CA-7	202.0×10^{-3}	572.6
CA-9	80.0×10^{-4}	22.7

Table 3.4. Mix Design for Portland Cement Concrete Slab.

Mix Design for Cubic Yard of Concrete:	
Water	301 lbs
Cement (Portland Type I)	674 lbs
River Gravel	2122 lbs
Silica Sand	970 lbs

Table 3.5. Aggregate Gradation at Center of Load After Testing.

Sieve Size	Per Cent Passing CA-7*	CA-9	CA-6
1-1/2"	100	100	100
1"	98.1	95.6	97.3
3/4"	89.0	88.2	86.4
1/2"	64.0	59.7	72.3
3/8"	29.9	48.4	57.4
#4	4.4	31.3	42.4
#16	0.0	13.8	18.2
#50	0.0	3.3	7.0
#200	0.0	0.6	2.4
Pan	0.0	0.0	0.0

* Actual gradation of CA-7 was not determined but judged to be the same as prior to testing.

Table 3.6. Description of Test Sections.

Phase 1

		Section Number*					
		1	2	3	4	5	6
Medium Aggregate Bituminous Concrete	MABC	2	2	2	2	1 1/2	2
Bituminous Base Course	BSBC	3	3	3	3	2	3
Bituminous Stabilized Open Graded Layer	BSOG	-	2.5	2.5[(a)]	-	2.5	2.5
Non Stabilized Open Grade Layer	NSOG	-	-	-	2.5	-	-
Lime-Fly Ash Stabilized 5A		-	-	-	2	2	-
Lime-Fly Ash Stabilized Sand-Gravel Mixture		-	-	-	-	-	3
Well Graded Crushed Stone Base (5A)		4	4	4	2.5	2.5	-
Well Graded Silty-Gravel 1B		8	5	5	5	5	6

Table 3.7. Detailed Description of Drainage Test Sites.

TEST SITE IDENTIFICATION	G1	G2	I1	I2
LOCATION	Georgia FAI 85N. Fulton County 16.1 km (10 mi) S.W. of Atlanta	Georgia FAI 85N. Fulton County 16.1 km (10 mi) S.W. of Atlanta	Illinois FAI 57.S Champaign County 8.0 km (5 mi) S. of Champaign	Illinois FAI 57N. Champaign County 8.0 km (5 mi) S. of Champaign
SURFACE COURSE				
Type	Plain Jointed Portland Cement Concrete without Dowel Bars	Plain Jointed Portland Cement Concrete without Oowel Bars	Continuously Reinforced Portland Cement Concrete	Reinforced Jointed Portland Cement Concrete with Oowel Bars
Width	7.32m (24 ft)	7.32m (24 ft)	7.32m (24 ft)	7.32m (24 ft)
Thickness	25.40 cm (10 in)	25.40 cm (10 in)	20.32 cm (8 in)	25.40 cm (10 in)
Joint Spacing	9.14 m (30 ft)	9.14m (30 ft)	--	30.48m (100 ft)
Transverse Slope	0.83 % Sloped to outside Shoulder	0.83% Sloped to outside Shoulder	1.56% Crowned at Centerline	1.56% Crowned at Centerline
Longitudinal Slope	1.00 %	1.00%	C.25%	0.25%
Length of Test Section	30.48m (1C0 ft)	30.48m (100 ft)	153.31m (503 ft)	151.79m (498 ft)
Width of Test Section	7.32m (24 ft)	7.32m (24 ft)	3.66m (12 ft)	3.66m (12 ft)
Date Constructed	1964	1964	1965	1965
BASE COURSE				
Type	Crushed Granite, Dense Graded	Crushed Granite, Oense Graded	Crushed Limestone Illinois CA 8	Crushed Limestone Illinois CA 8
Thickness	40.64-60.96 cm (16-24 in)	40.64-60.96 cm (16-24 in)	10.16 cm (4 in)	15.24 cm (6 in)
Saturated Hydraulic Conductivity	4.7 μm/sec (1.3 ft/day)	4.7 μm/sec (1.3ft/day)	3.5 μm/sec (1.0ft/day)*	3.5 μm/sec (1.0 ft/day)*
SUBGRADE				
Type	A-4	A-4	A-6	A-6
Saturated Hydraulic Conductivity	100 nm/sec (2.8×10^{-2} ft/day)*	100 nm/sec (2.8×10^{-2} ft/day)*	10 nm/sec (2.8×10^{-3} ft/day)*	10 nm/sec (2.8×10^{-3} ft/day)*
SHOULDERS				
Type	Asphalt Concrete on Cement Treated Aggregate	Asphalt Concrete on Cement Treated Aggregate	Bituminous Aggregate Mixture, Reconstructed 1976	Bituminous Aggregate Mixture, Reconstructed 1976
Width	0.61m (2 ft) Inside 3.05 m (10 ft) Outside	0.61 m (2 ft) Inside 3.05 m (10 ft) Outside	1.22 m (4 ft) Inside 3.05 m (10 ft) Outside	1.22 m (4 ft) Inside 3.05 m (10 ft) Outside
Thickness	5.08 cm (2 in) Asphalt Concrete 15.24 cm (6 in) Cement Treated Aggregate	5.08 cm (2 in) Asphalt Concrete 15.24 cm (6 in) Cement Treated Aggregate	20.32 cm (8 in)	20.32 cm (8 in)
SUBSURFACE ORAINAGE				
Type	Corrugated Plastic Pipe, Constructed Oct. 1977	Corrugated Plastic Pipe, Constructed Oct. 1977	Concrete Pipe, Constructed 1976	Concrete Pipe, Constructed 1976
Diameter	10.16 cm (4 in) I.D.	10.16 cm (4 in) I.O.	15.24 cm (6 in) I.O.	15.24 cm (6 in) I.O.
Location	Adjacent to Pavement Edge	Adjacent to Pavement Edge	Adjacent to Pavement Edge	Adjacement to Pavement Edge
Oepth	86.36 cm (34 in)	86.36 cm (34 in)	55.88 cm (22 in)	60.96 cm (24 in)
Drainage Envelope	Crushed Granite (open graded)	Crushed Granite (open graded)	Concrete Sand	Concrete Sand
Saturated Hydraulic Conductivity	4 cm/sec (1.1×10^{4} ft/day)*	4 cm/sec (1.1×10^{4} ft/day)*	46 μm/sec (13.0 ft/day)	46 μm/sec (13.0 ft/day)
TEST SITE DRAINAGE AREA	248.1 m^2 (2670 ft^2)	233.6 m^2 (2514 ft^2)	560.8 m^2 (6036 ft^2)	555.2 m^2 (5976 ft^2)

* Estimated Values

Table 3.8. Statistical Data Based on Percentage of Pipe Outflow.

Pavement Test Section	Mean ($\bar{x}$) %	Coefficient of Variation (V) %	Standard Deviation(s) %
Georgia G1 Unsealed	30.6	61.1	18.7
Georgia G1 Sealed	0.7	194.0	1.4
Georgia G2 Sealed	--	--	--
Illinois I1 Unsealed	26.0	76.5	19.9
Illinois I1 Sealed	16.4	69.5	11.4
Illinois I2 Unsealed	52.1	36.7	19.1
Illinois I2 Sealed	11.6	28.4	3.3

$$\text{Percentage of Pipe Outflow} = \frac{\text{Pipe Outflow Volume}}{\text{Precipitation Volume}} \times 100$$

Table 3.9. Pumping of Surveyed Pavements in Illinois (Ref. 18).

Interstate Surveyed	Number of Sections Surveyed	Number of Sections with Evidence of Pumping
I55	18	15
I57	58	29
I70	40	23
I74	25	17
I80	47	37
I270	11	11
I280	5	5
	Total = 204	Total =137

Table 3.10. Critical Water Velocity (Ref. 1).

Curing Conditions	95% Relative Humidity		60% Relative Humidity	
Freeze-Thaw Cycles / Material	none (ft/s)	9 cycles (ft/s)	none (ft/s)	9 cycles (ft/s)
untreated gravel	< 16.4	--	--	--
cement-treated silt	289	203	177	66
cement-treated gravel	292	206	180	82
lean concrete	492	328	328	249

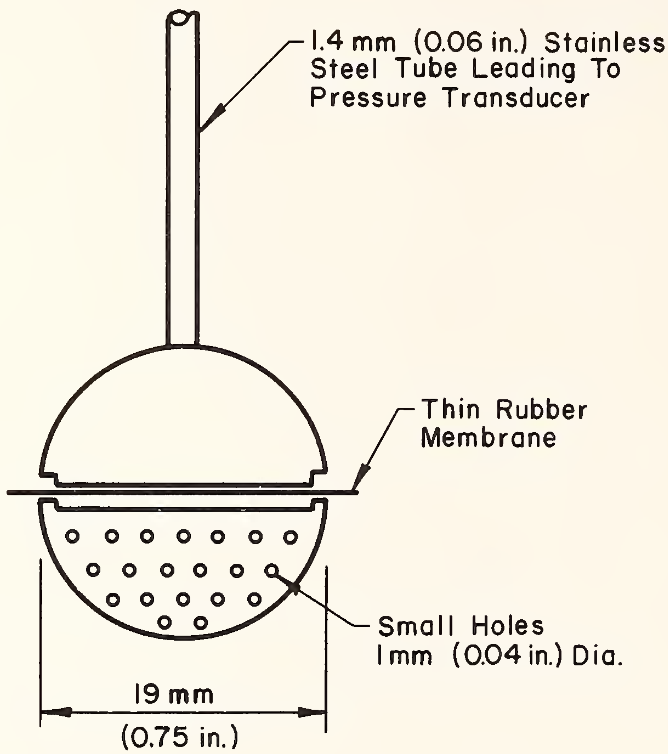

Figure 3.1. Piezometer for Measuring Dynamic Pore Water Pressure.

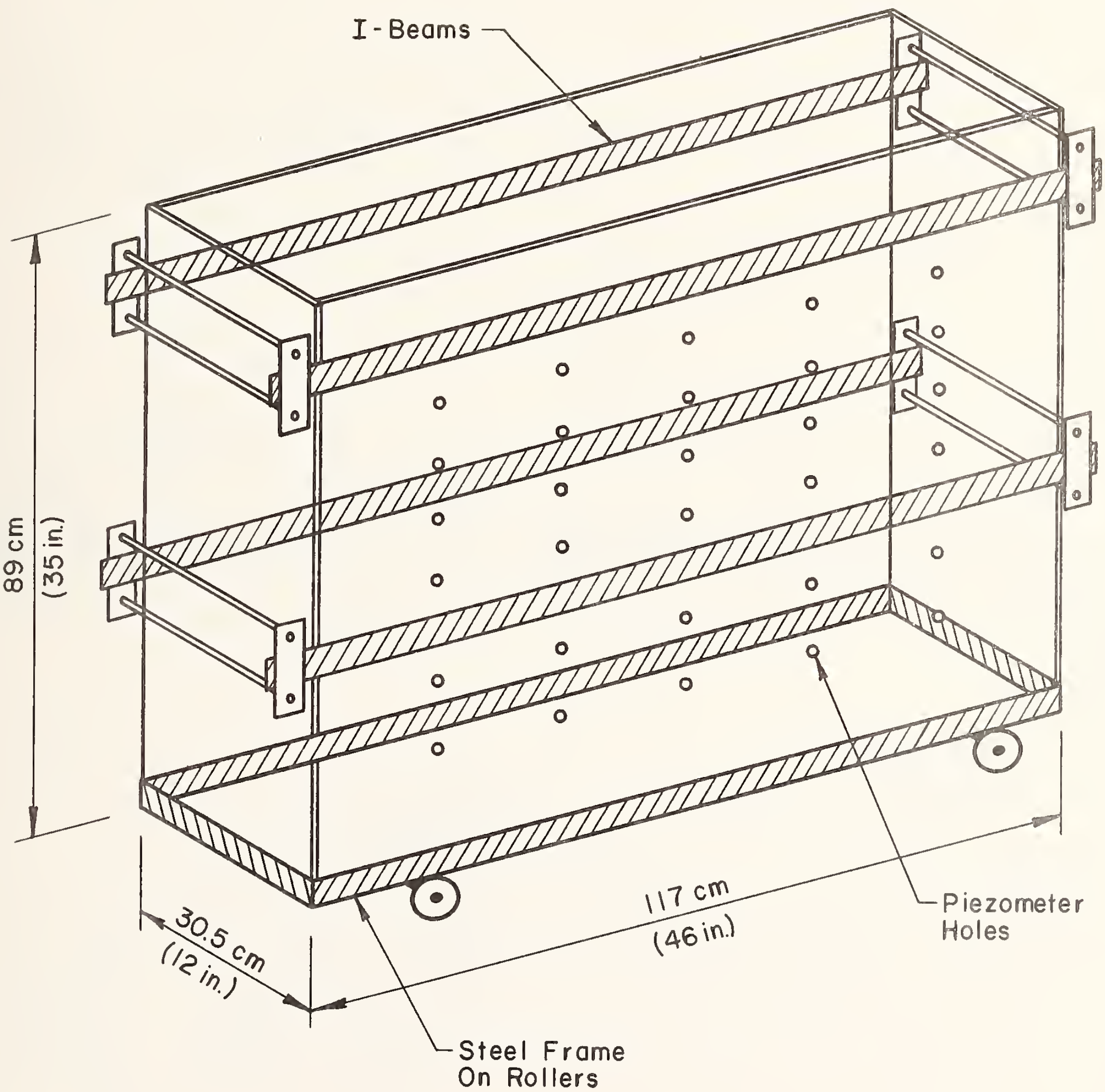

Figure 3.2. Plexiglass Tank for Pavement Test Section.

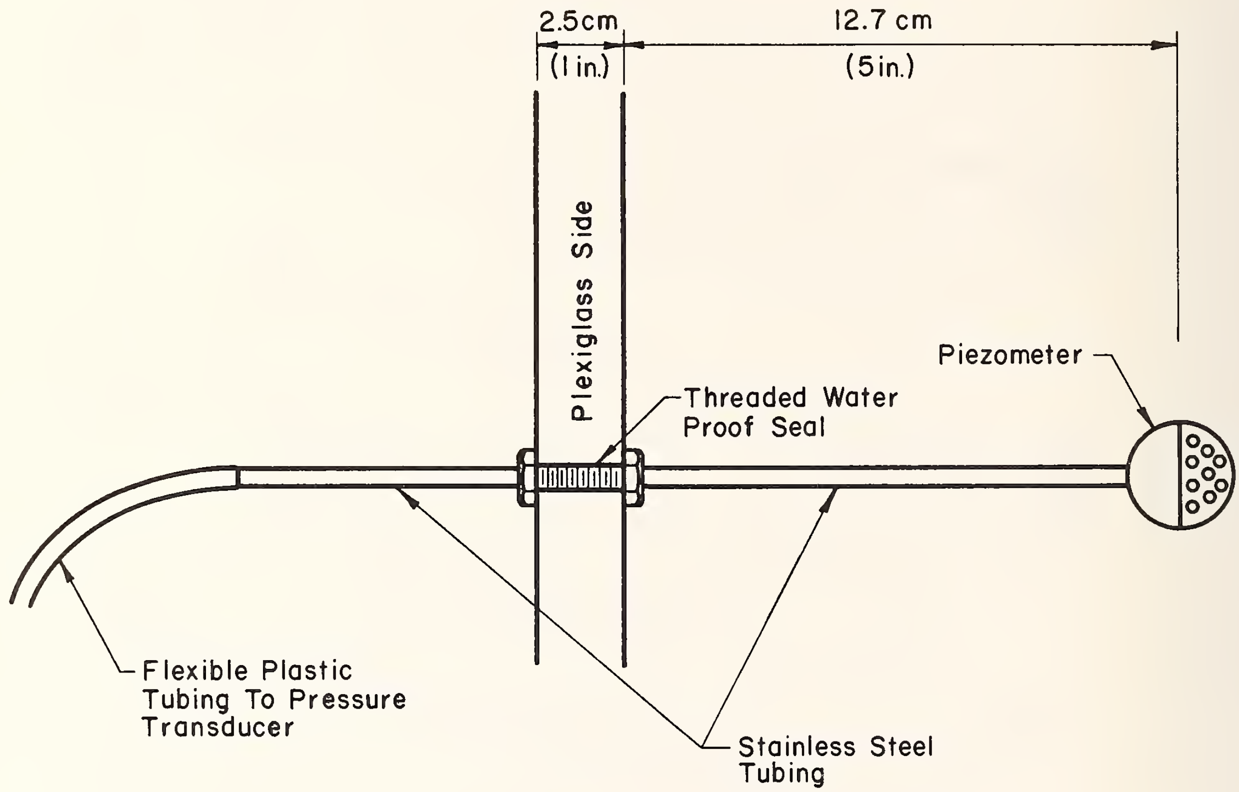

Figure 3.3. Piezometer Installation in Side of Tank.

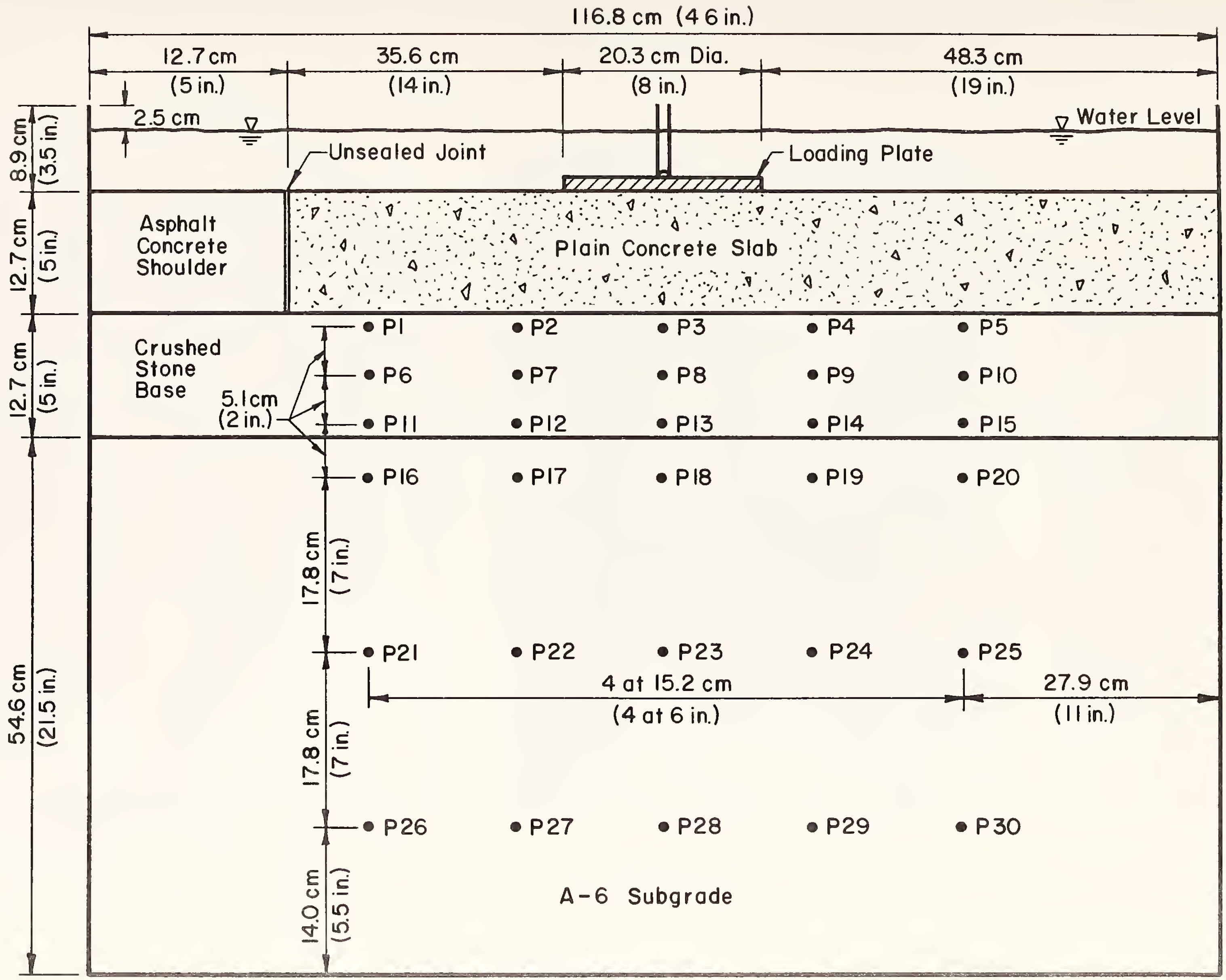

Figure 3.4. Profile of Pavement Model.

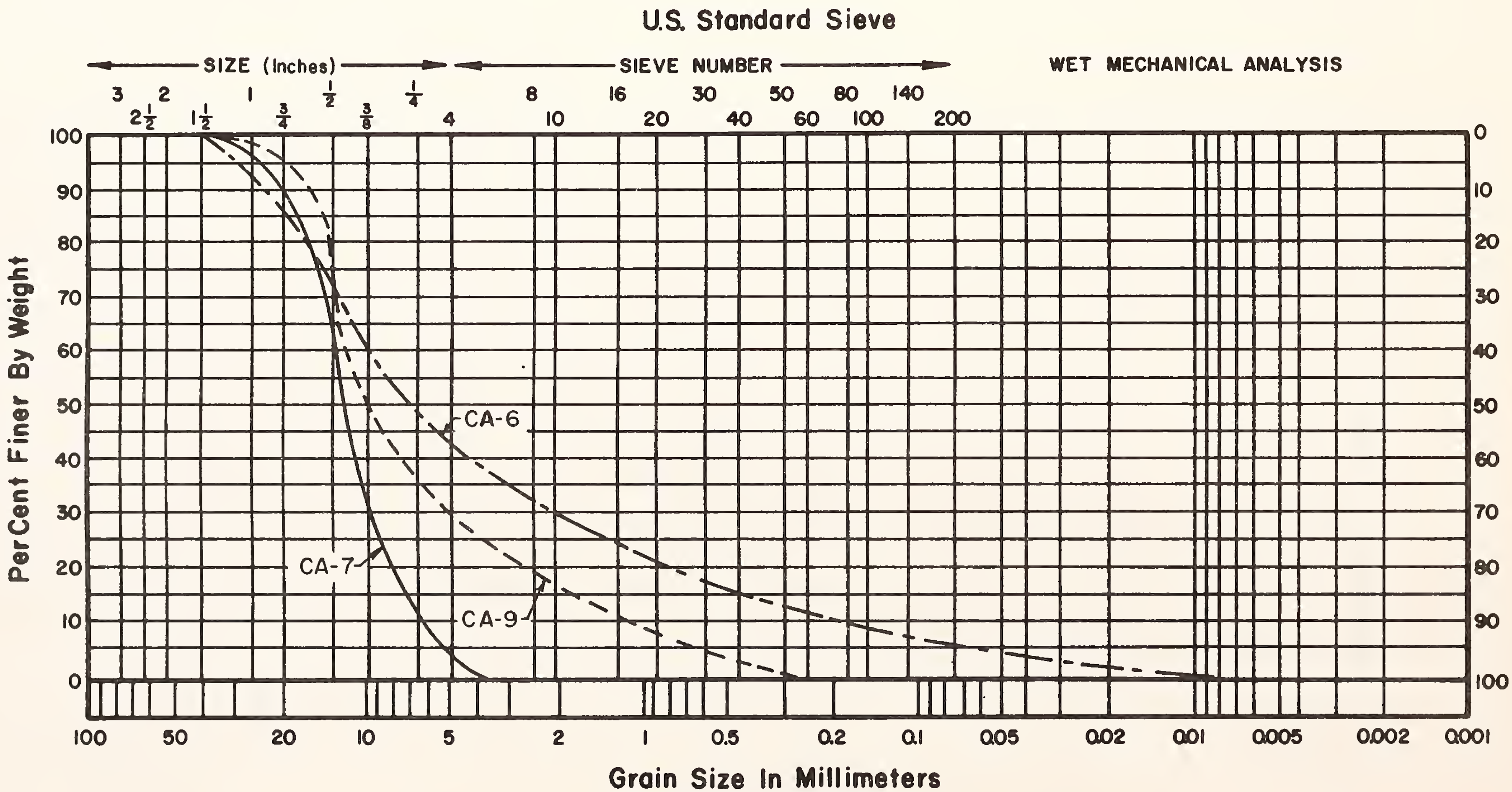

Figure 3.5. Gradation Curves for Base Courses Used in Model Tests.

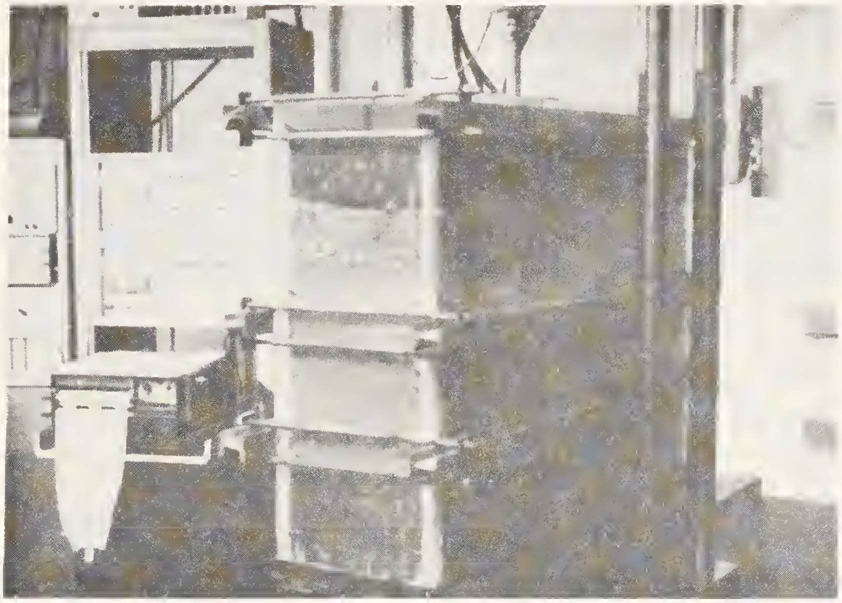

Figure 3.6. Pavement Model Prepared for Testing.

Figure 3.7. Flow Paths of Water in CA-6 Base Course.

Figure 3.8. Fine Material Moving Towards the Concrete Slab and Asphalt Concrete Shoulder Joint in CA-9 Base Course.

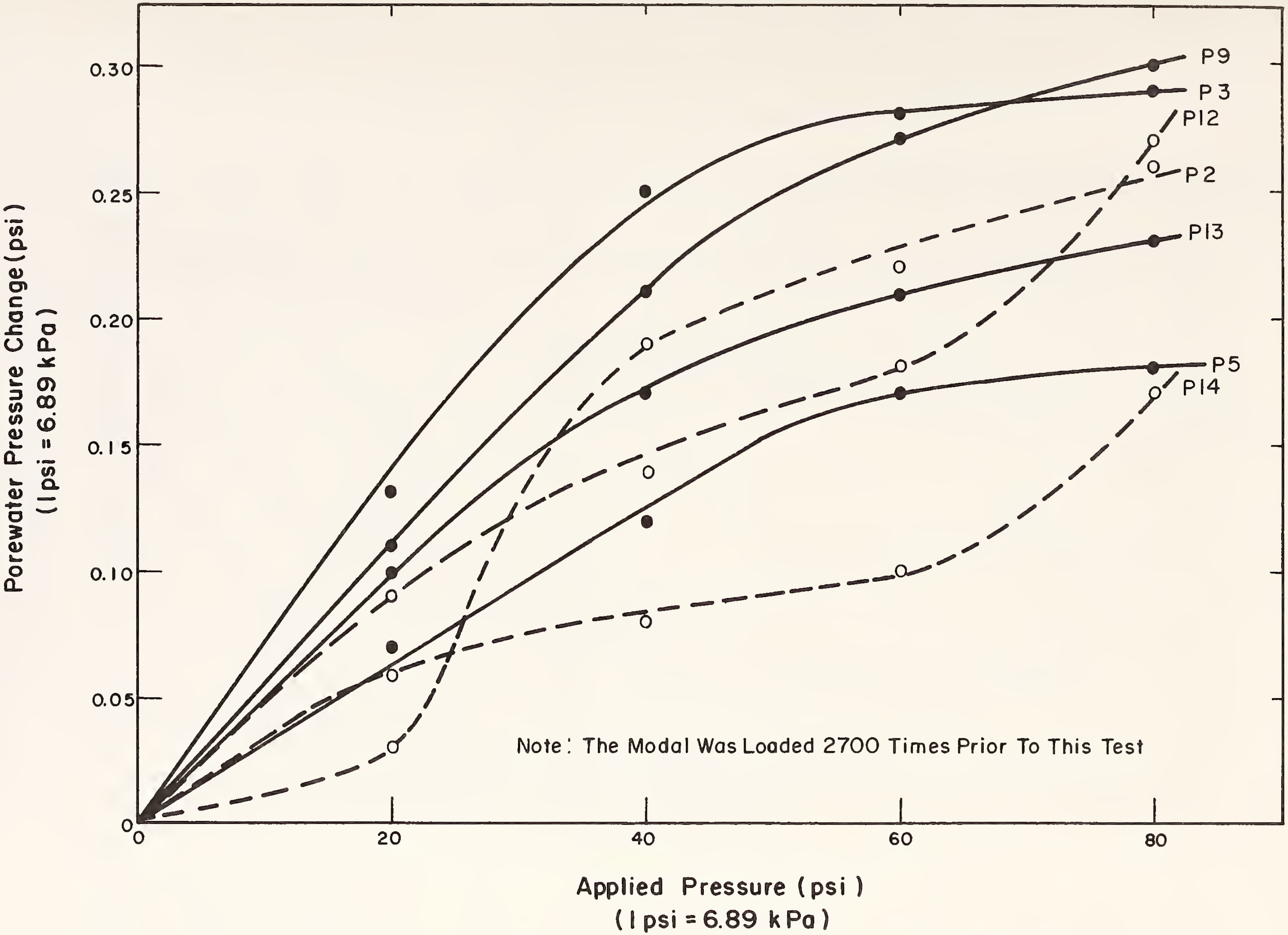

Figure 3.9. Influence of Applied Load on Pore Water Pressure in the CA-6 Base Course.

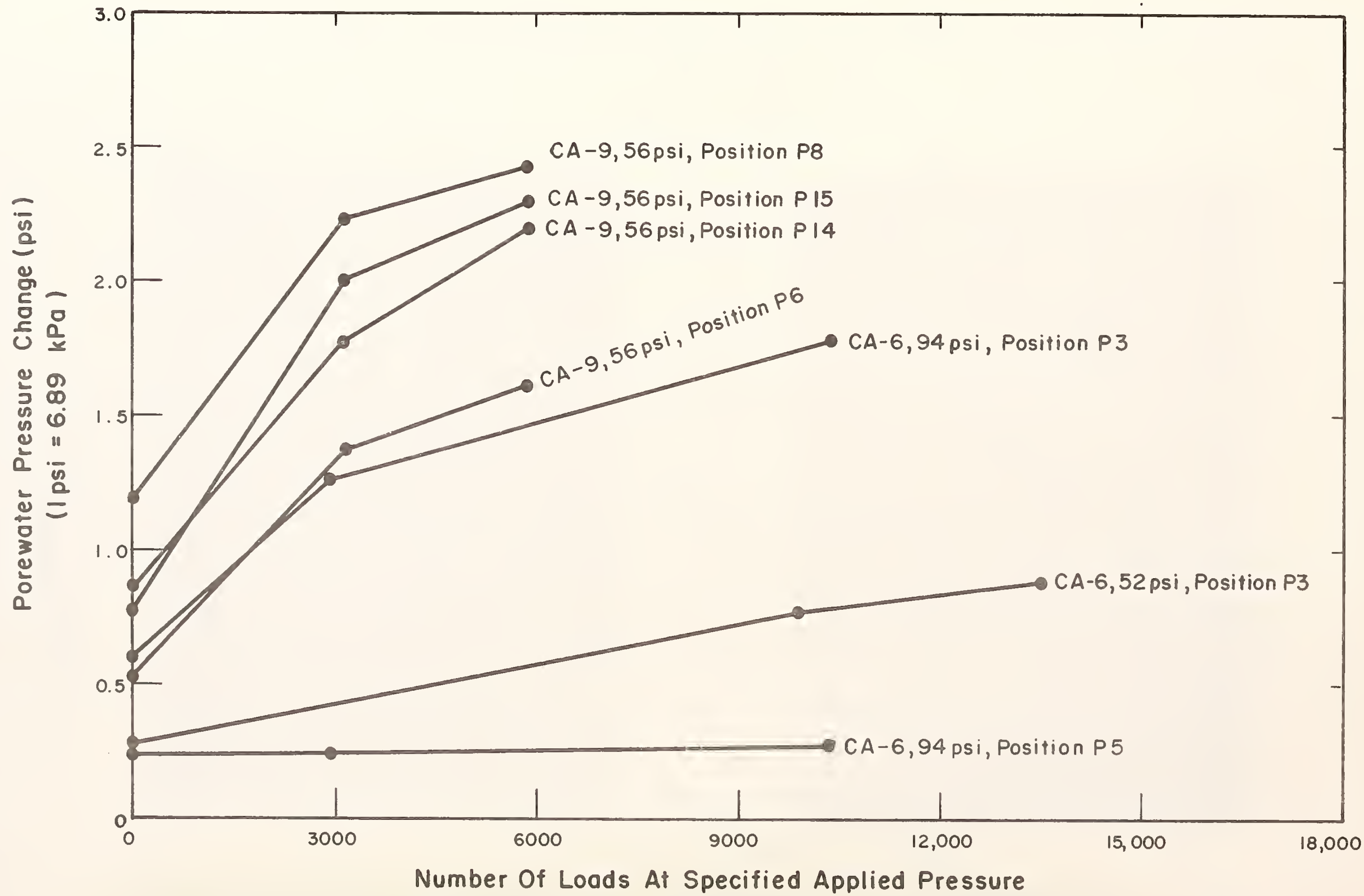

Figure 3.10. Influence Number of Load Applications has on Pore Water Pressure.

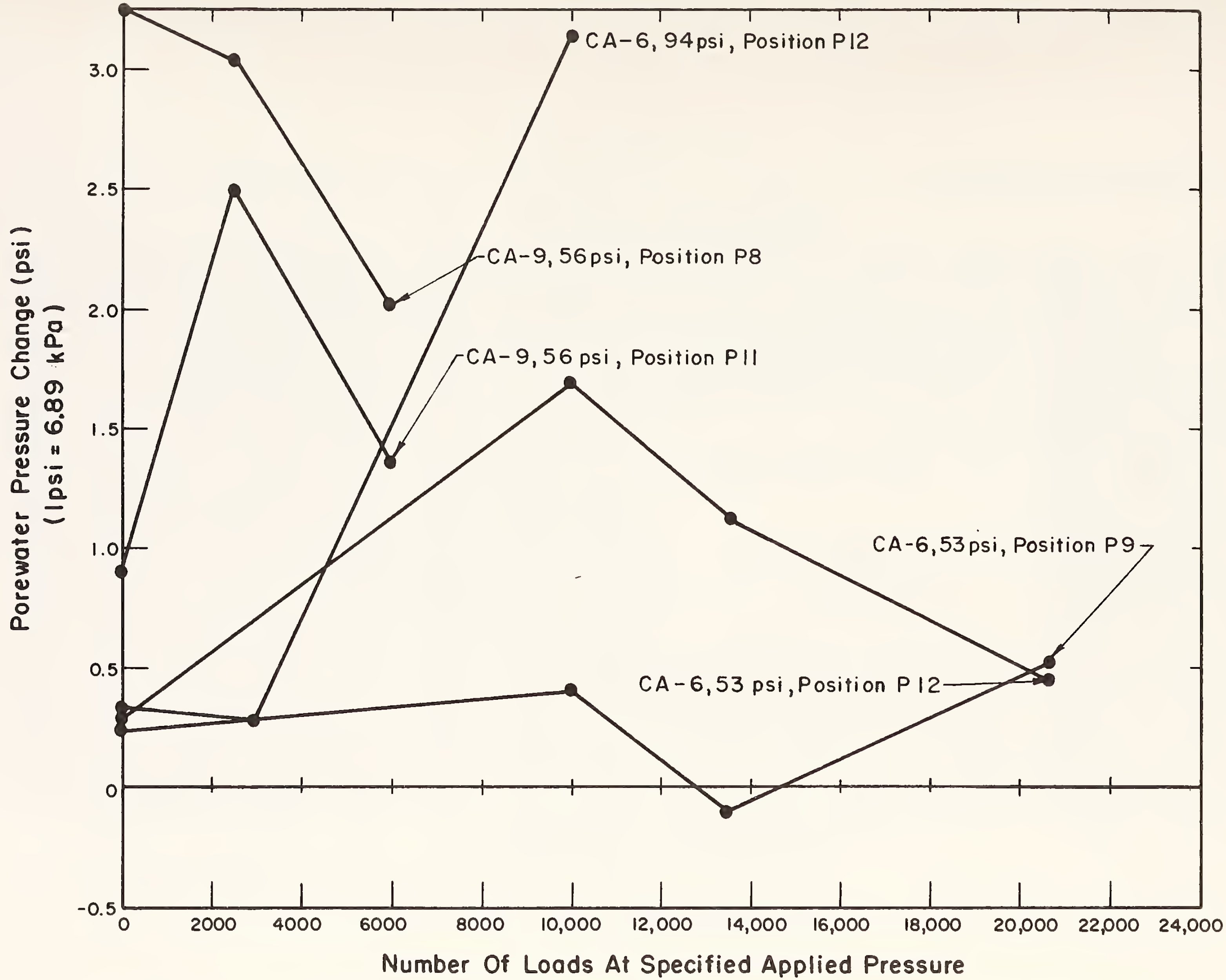

Figure 3.11. Influence Number of Load Applications has on Pore Water Pressure.

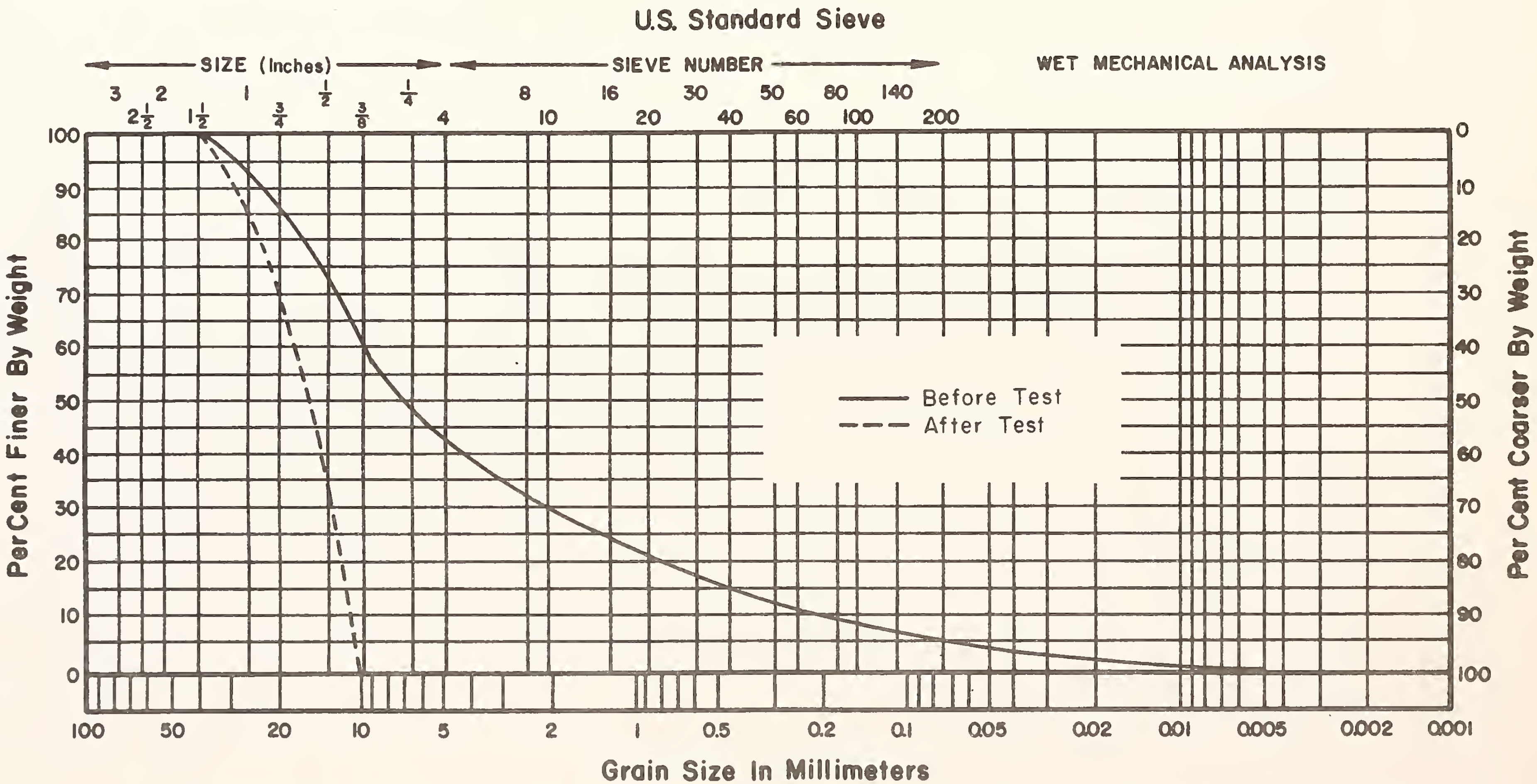

Figure 3.12. CA-6 Base Course Gradation Changes Along the Concrete Slab-Base Course Interface.

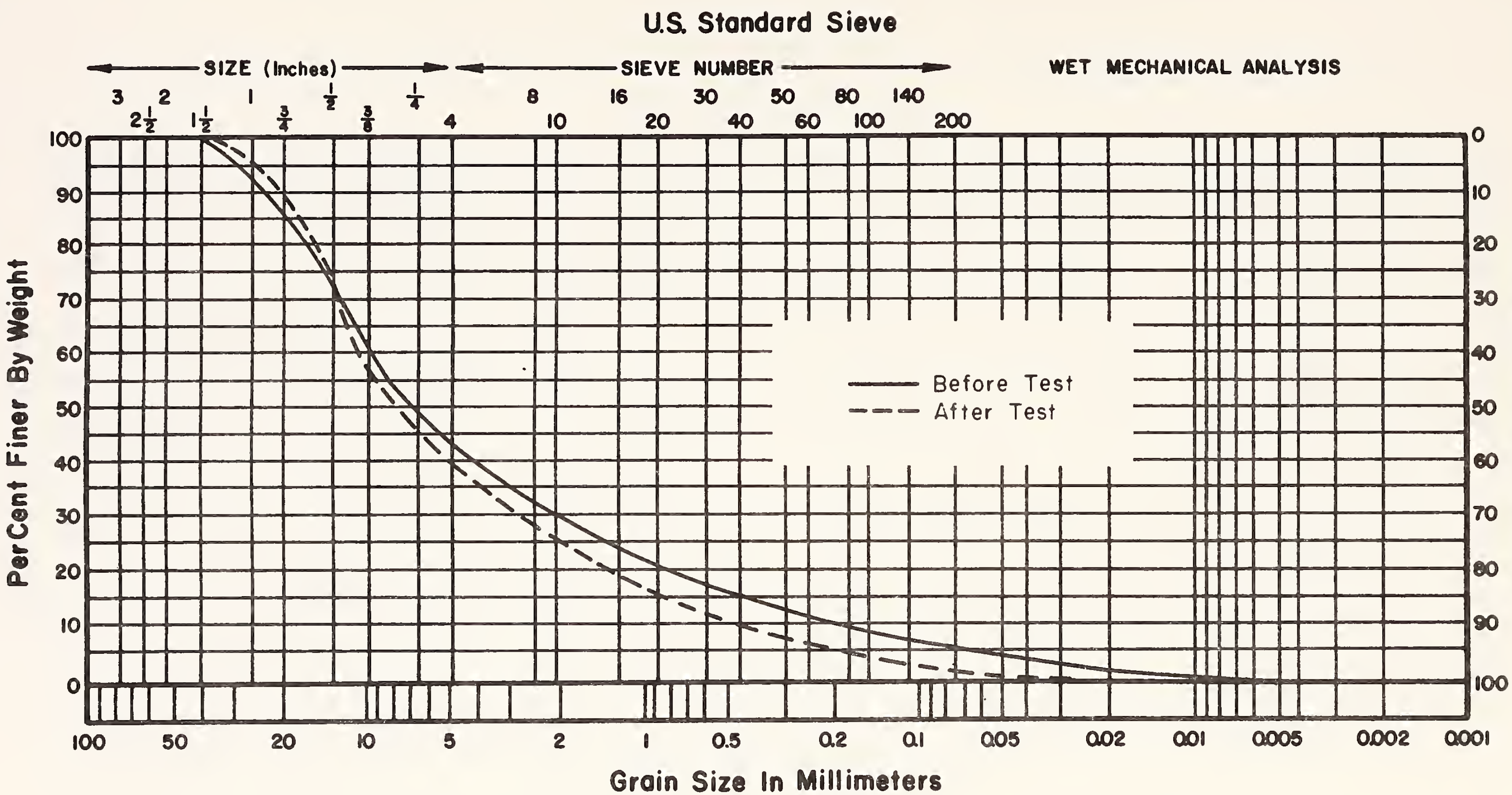

Figure 3.13. CA-6 Gradation Changes Directly Beneath Load.

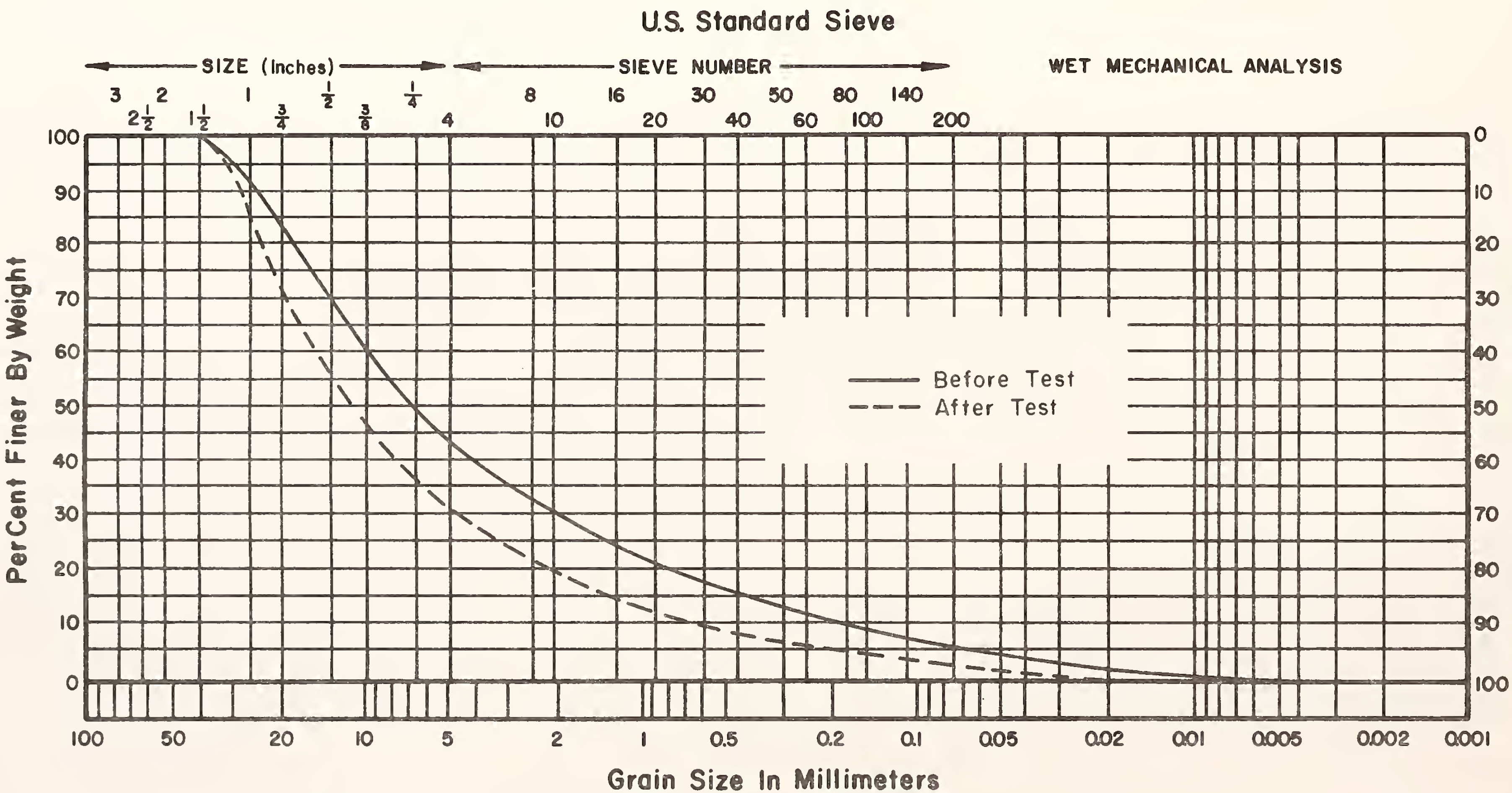

Figure 3.14. CA-6 Base Course Gradation Changes at Joint Between Concrete Slab and Asphalt Concrete Shoulder.

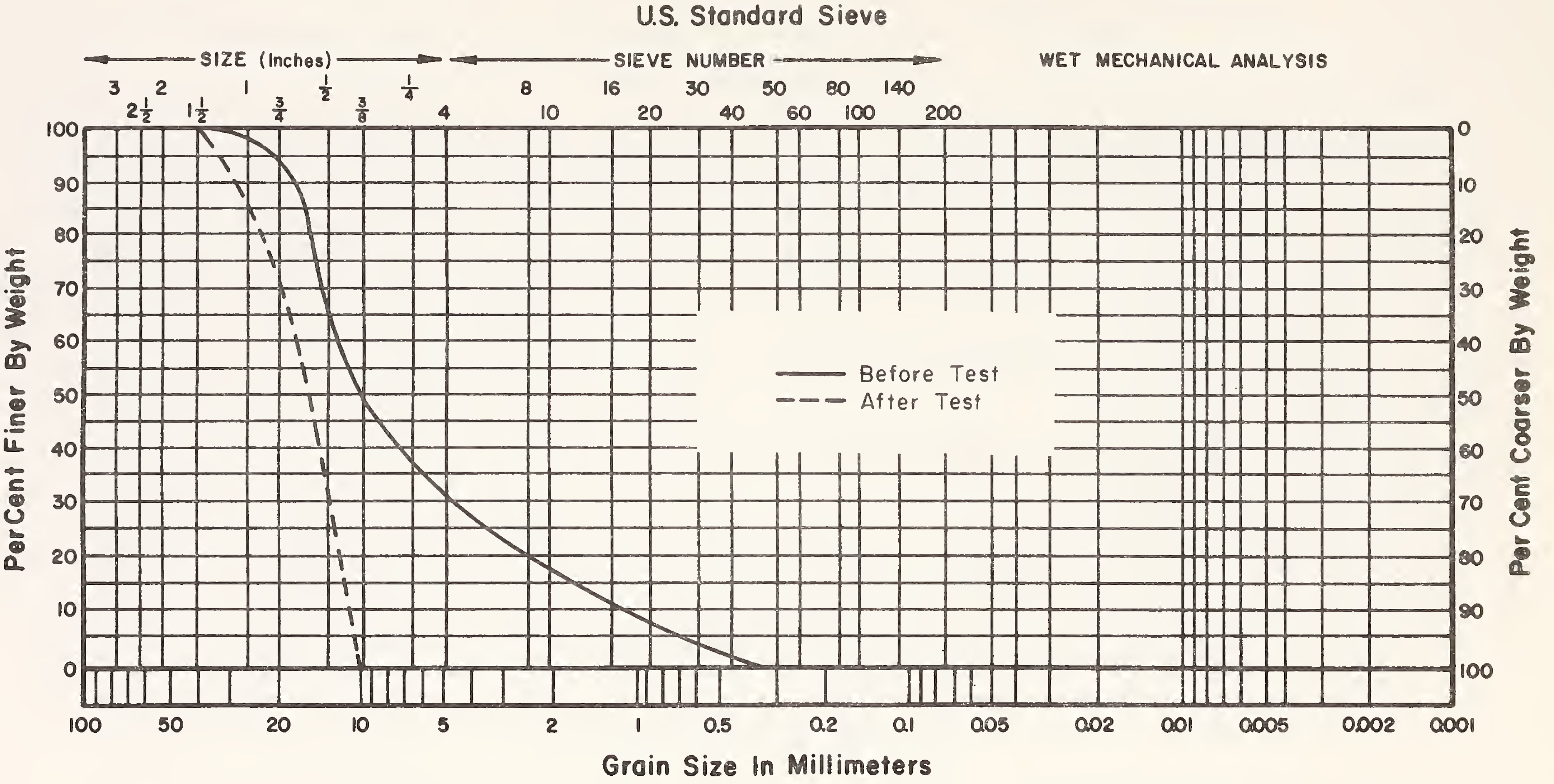

Figure 3.15. CA-9 Base Course Gradation Changes Along the Concrete Slab-Base Course Interface.

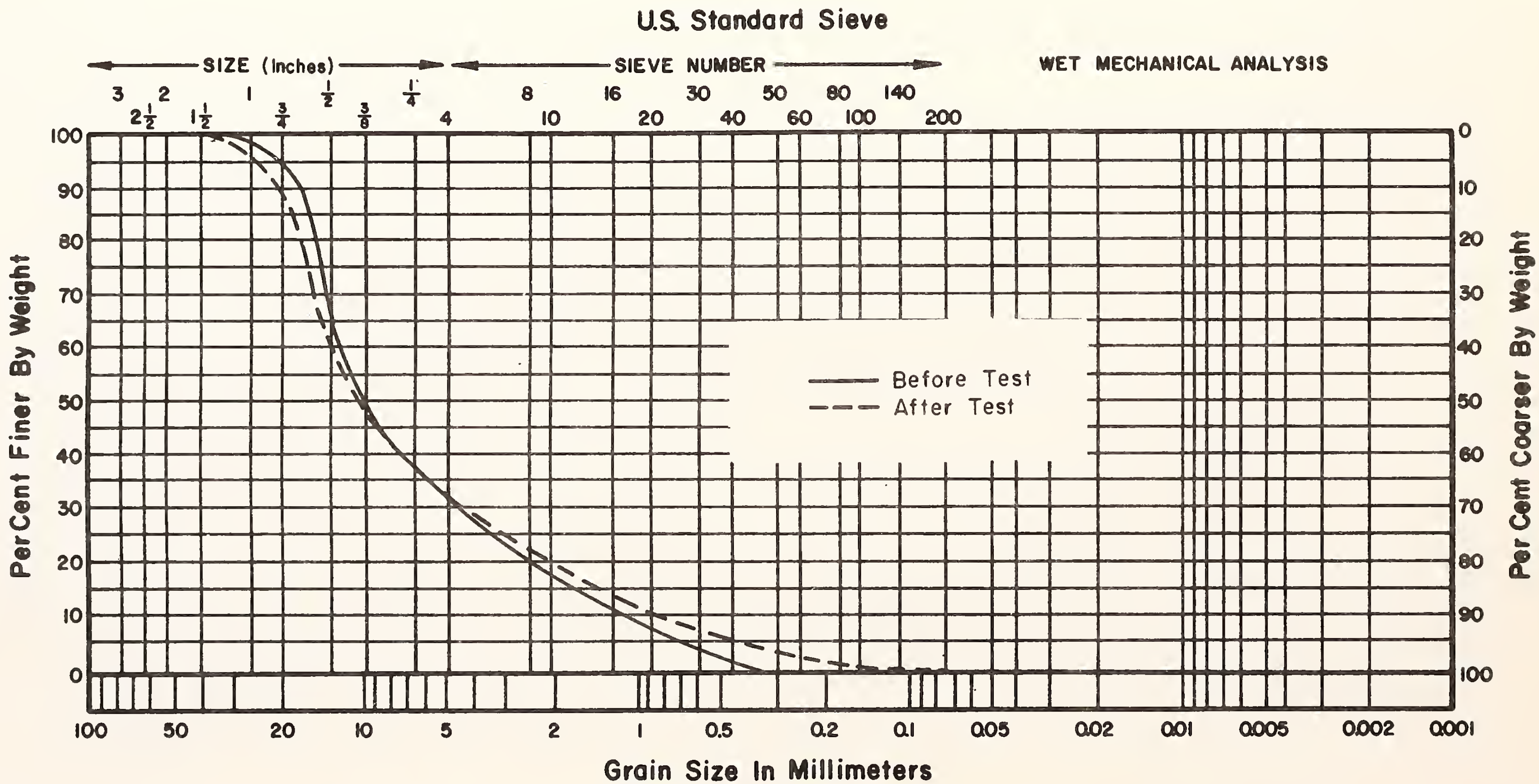

Figure 3.16. CA-9 Base Course Gradation Changes Directly Beneath Load.

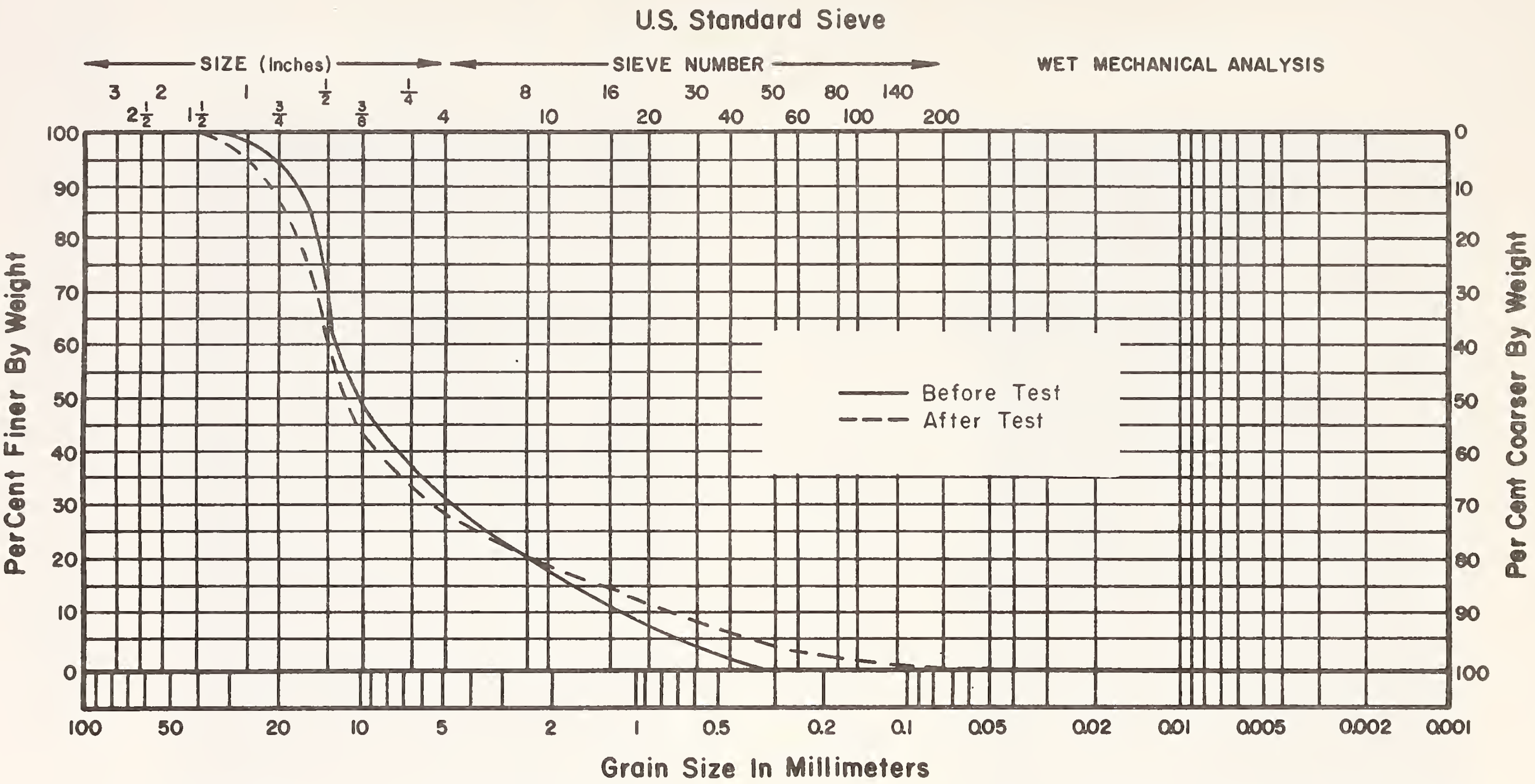

Figure 3.17. CA-9 Base Course Gradation Changes at Joint Between Concrete Slab and Asphalt Concrete Shoulder.

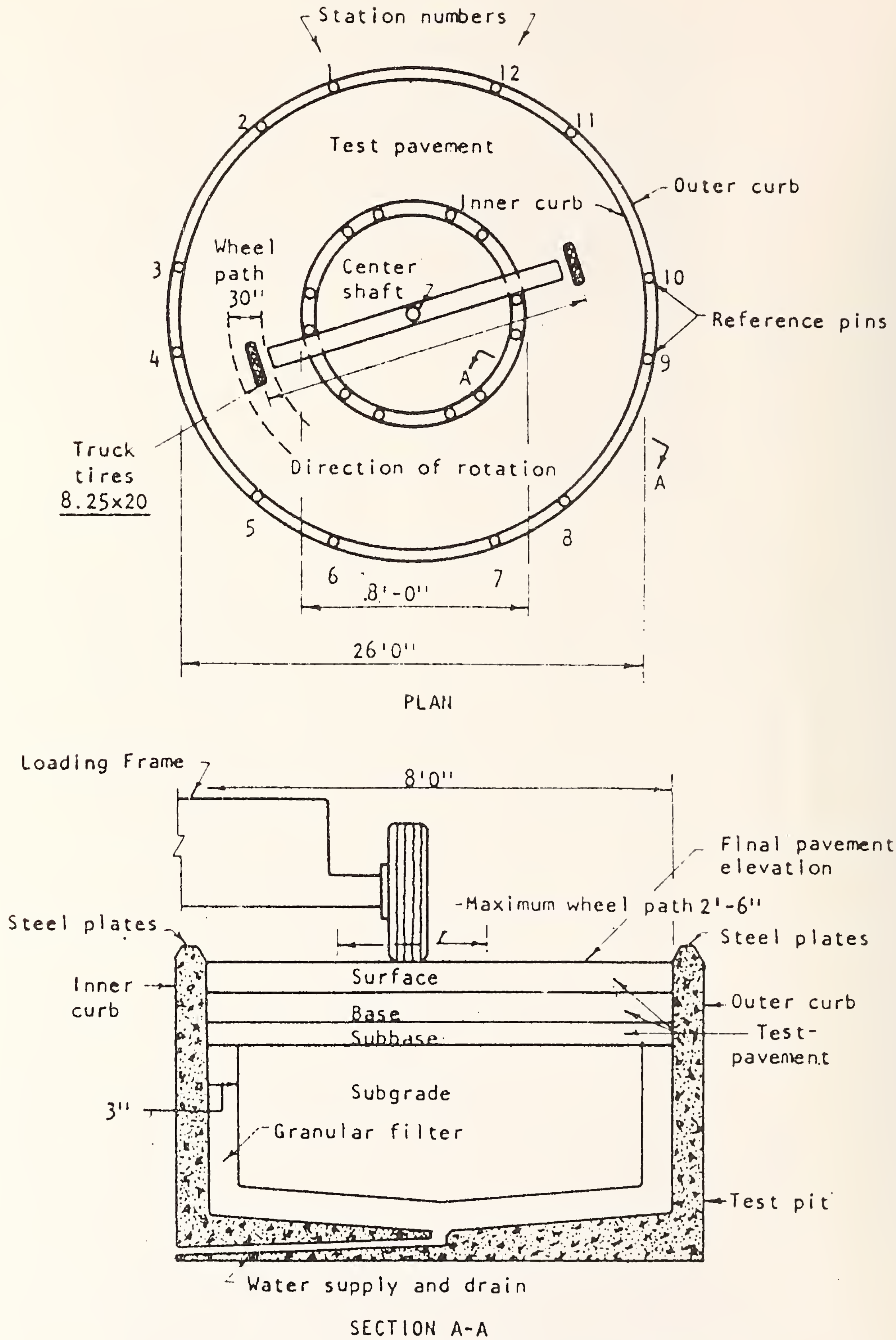

Figure 3.18. Schematic Diagram of the University of Illinois Pavement Test Track.

Figure 3.19. Water and Fine Material Flowing out From Beneath a Jointed Concrete Pavement on a Long Gradient.

Figure 3.20. Pumping Associated with a Continuously Reinforced Concrete Pavement.

Figure 3.21. BAM Which as has been Pumped from Beneath a Continuously Reinforced Concrete Pavement.

Figure 3.22. Channeling Marks at Interface Between a Concrete Slab and Cement Treated Base.

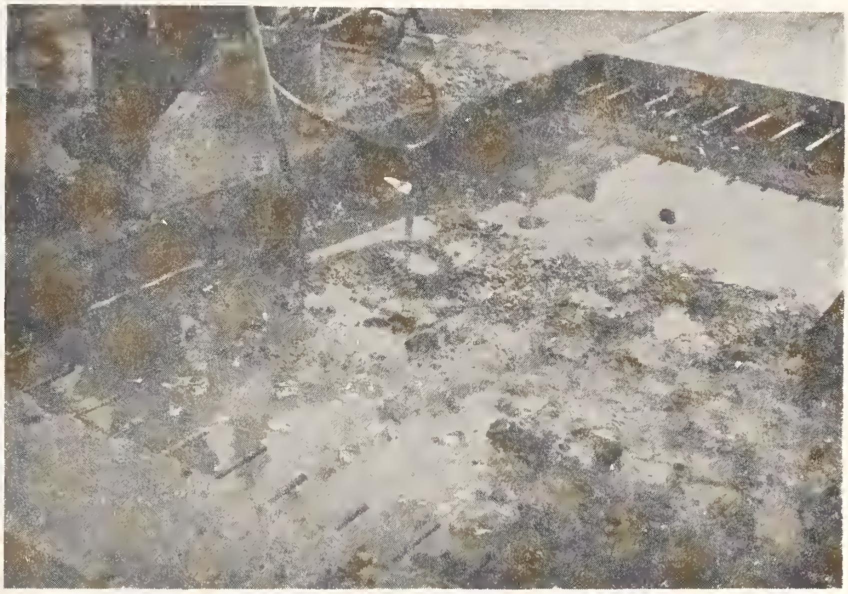

Figure 3.23. Patching Continuously Reinforced Concrete Pavement with an Edge Drain.

Figure 3.24. Visible Water at the Pavement Edge for Patch Shown in Figure 3.23 after Several Hours.

Figure 3.25. Soil Fines Blocking Drainage to a Longitudinal Pavement Edge Drain.

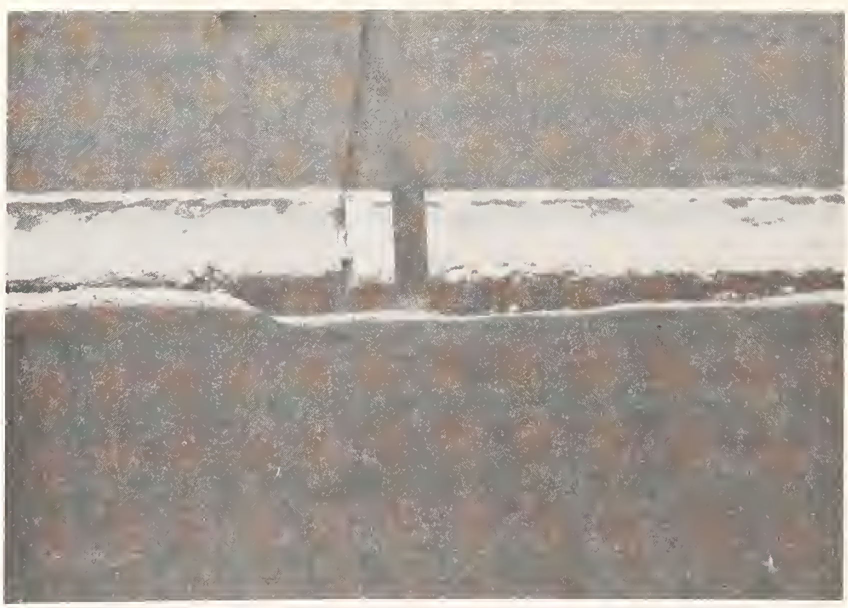

Figure 3.26. Shoulder Drop-off Resulting from Loss of Base Course or Subgrade Material.

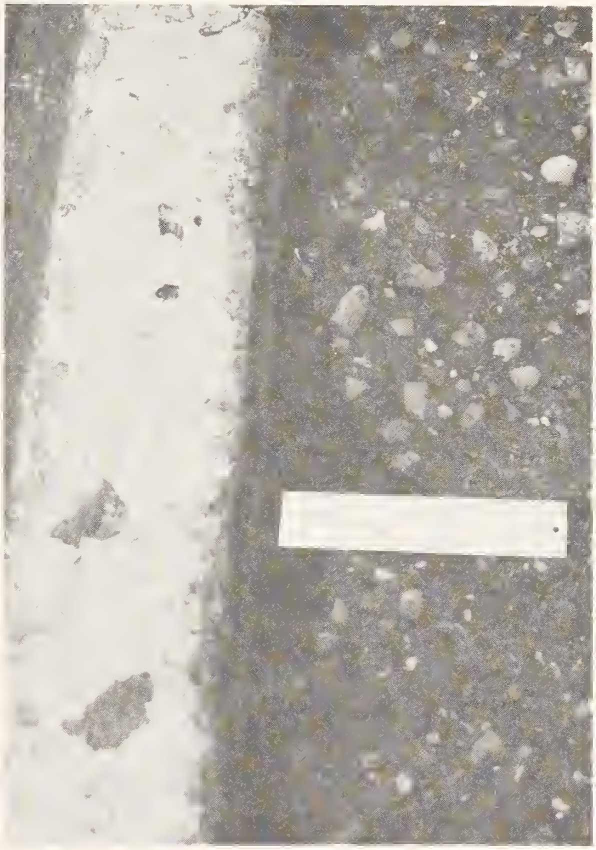

Figure 3.27. Disintegration and Erosion of Pavement-Shoulder Joint.

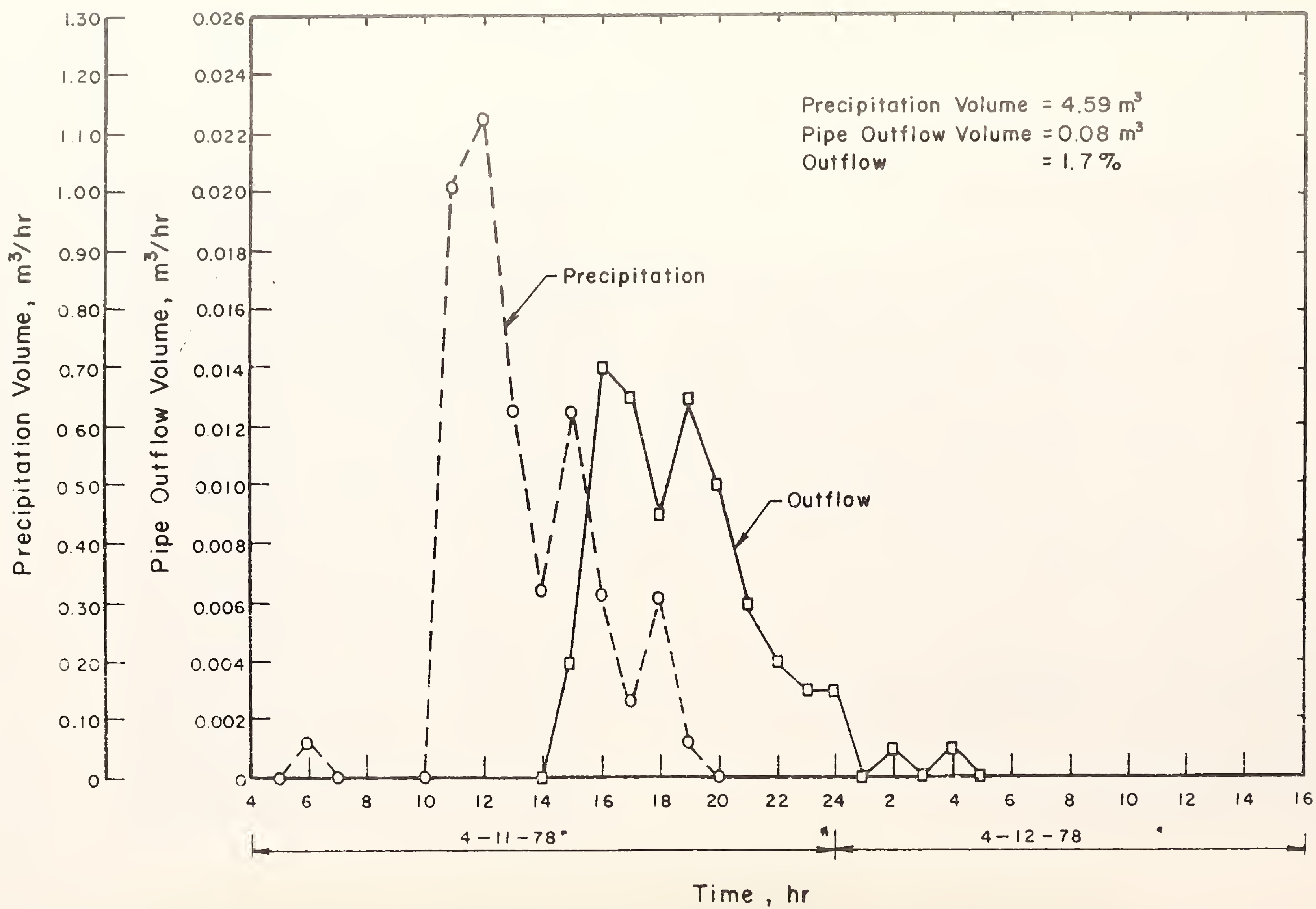

Figure 3.28. Influence of Precipitation on Drainage Outflow in Test Section G1 with Sealed Edge Joint (1 m^3/hr = 264 gal/hr), (Ref. 17).

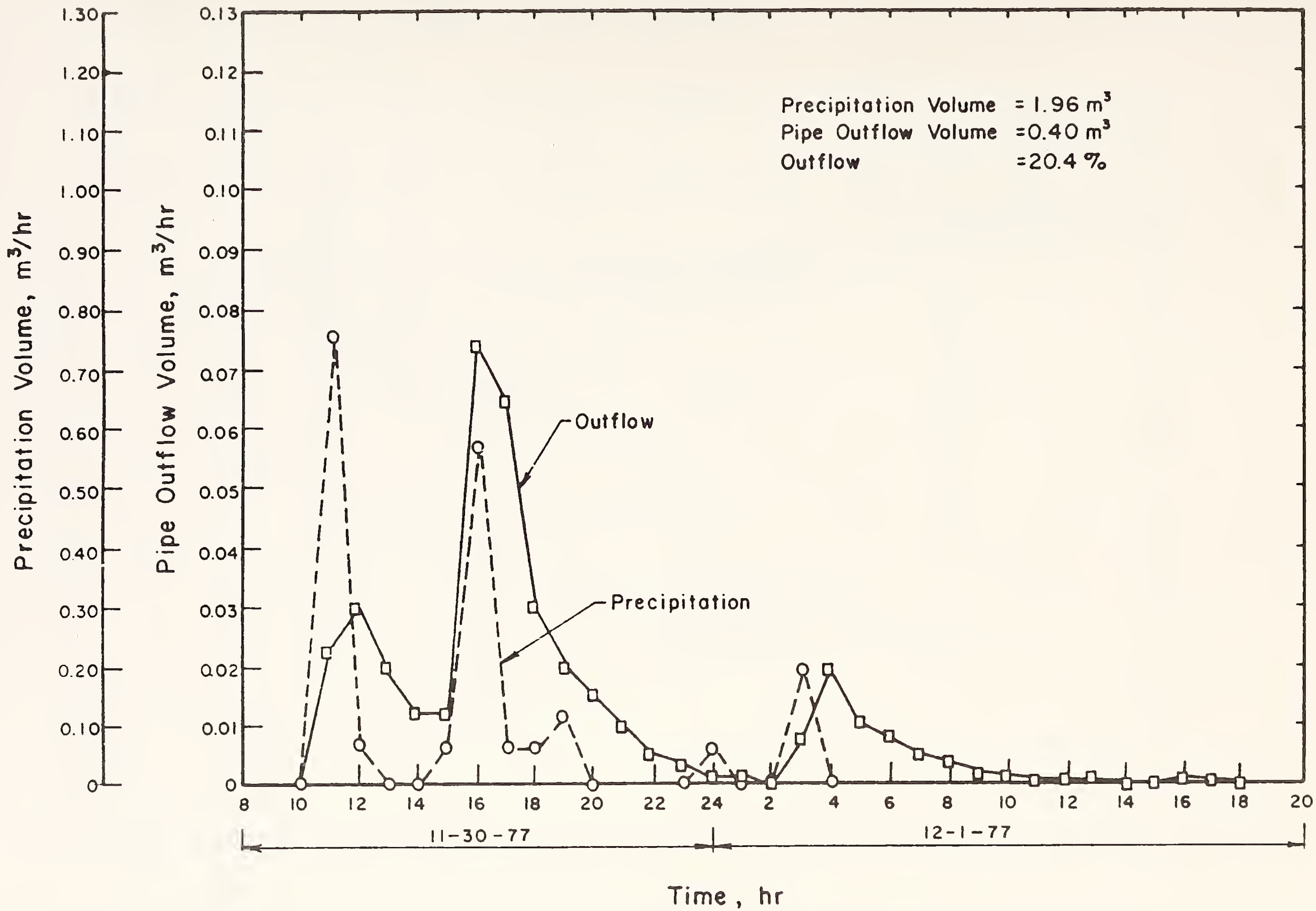

Figure 3.29. Influence of Precipitation on Drainage Outflow in Unsealed Test Section G1 (1 m^3/hr = 264 gal/hr), (Ref. 17).

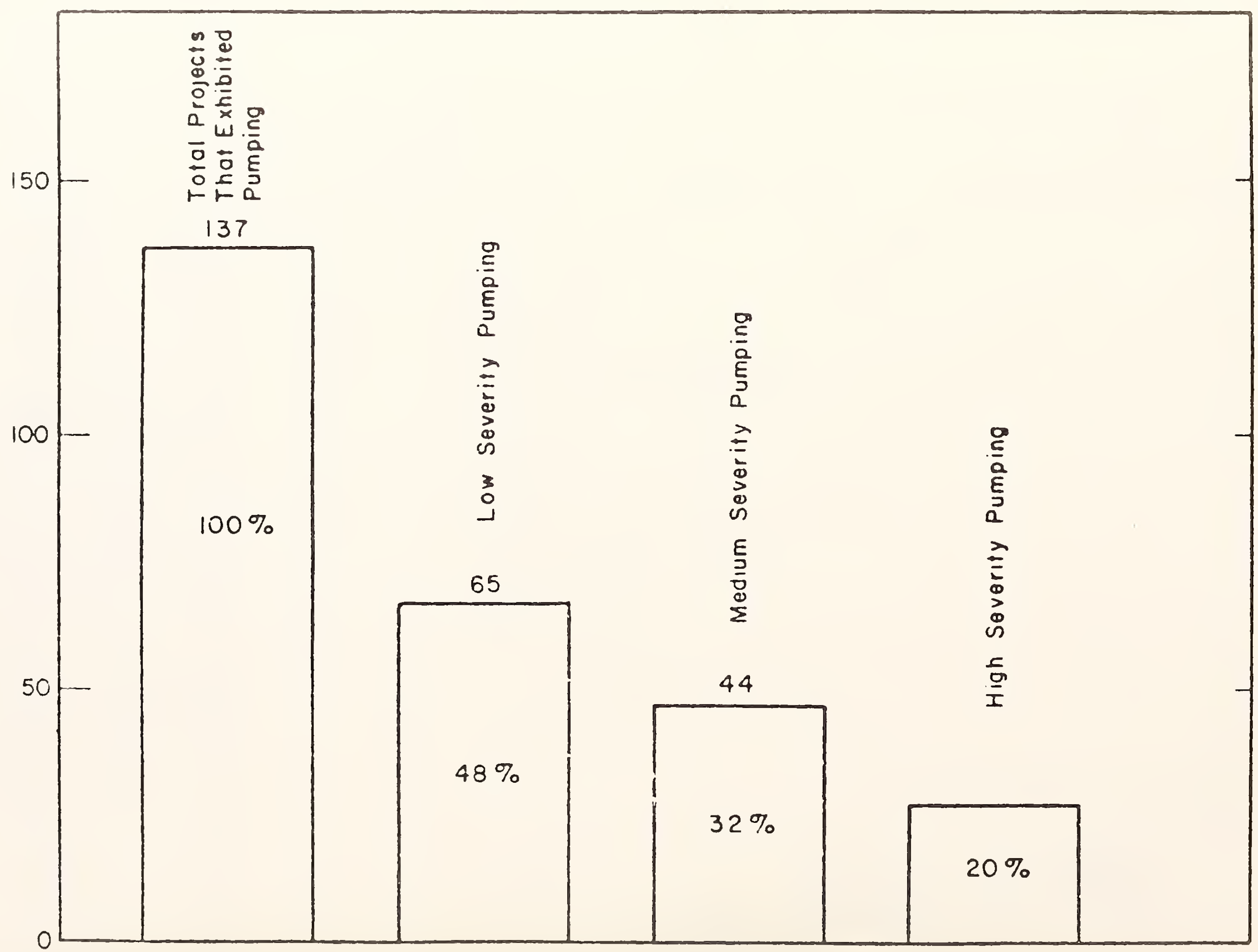

Figure 3.30. Rage of Pumping Severity (Ref. 18).

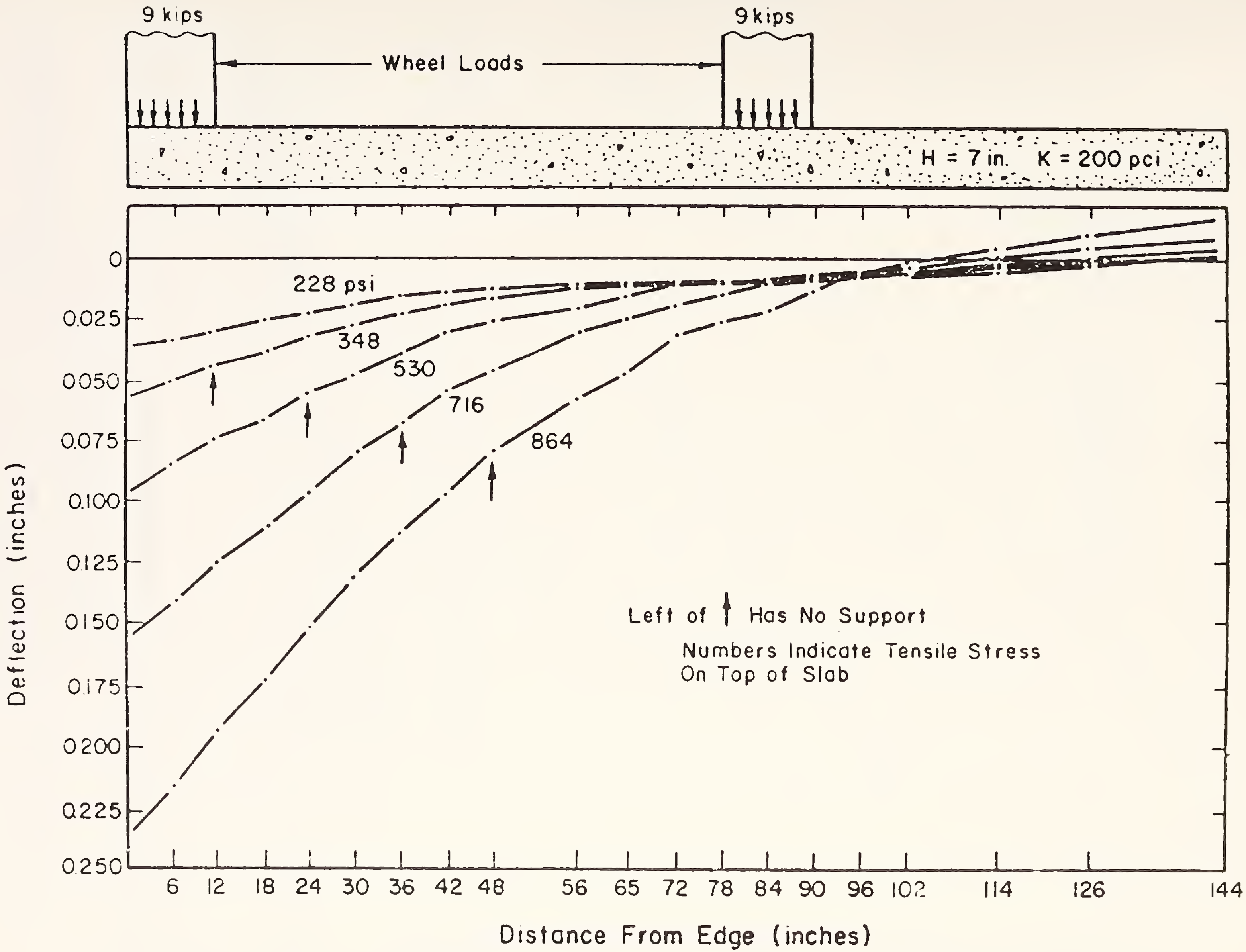

Figure 3.31. Effect of Loss of Support on Critical Stresses and Deflections (Ref. 19).

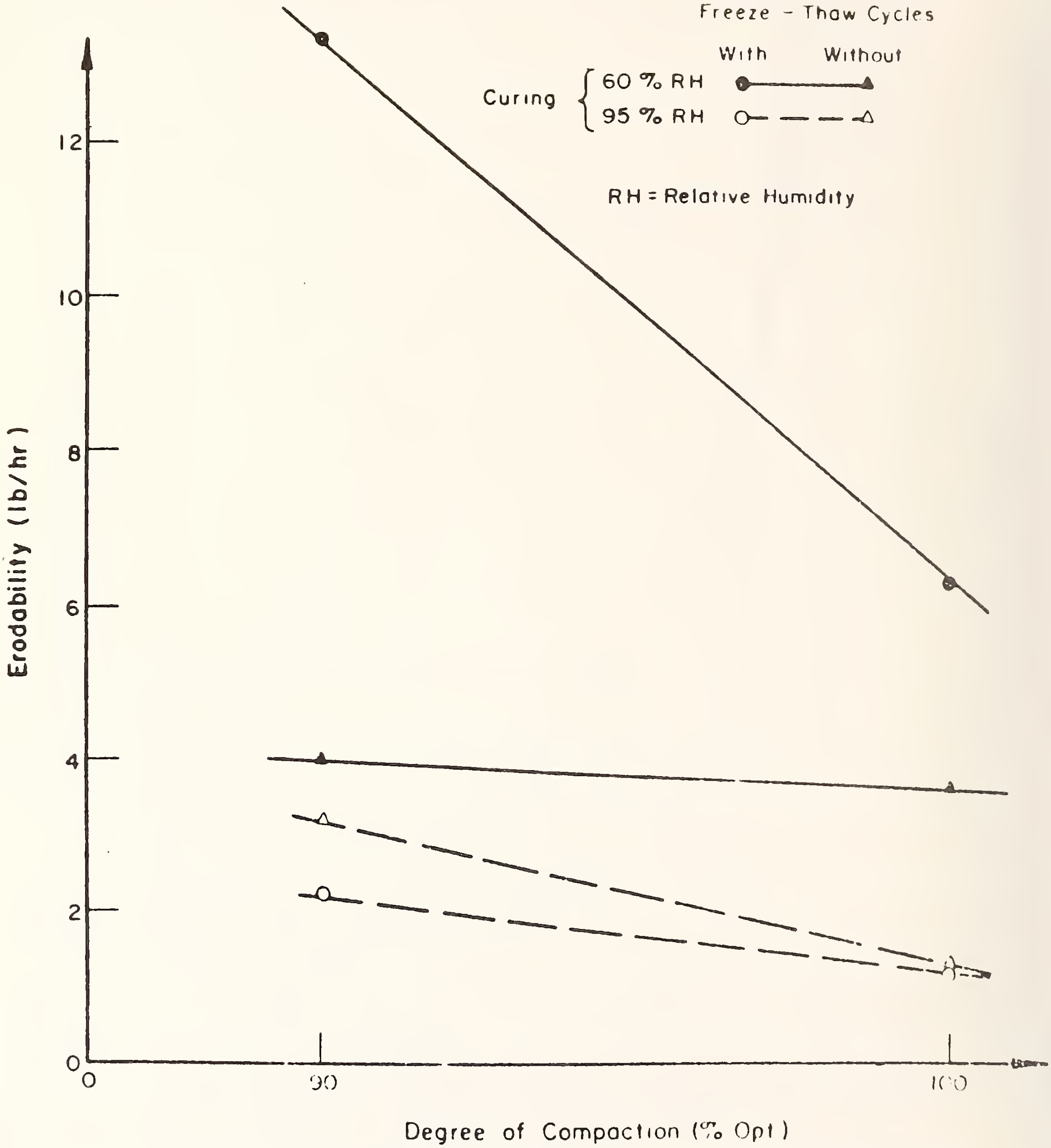

Figure 3.32. Influence of Compaction on the Erodibility of Cement Treated Ground (Ref. 1).

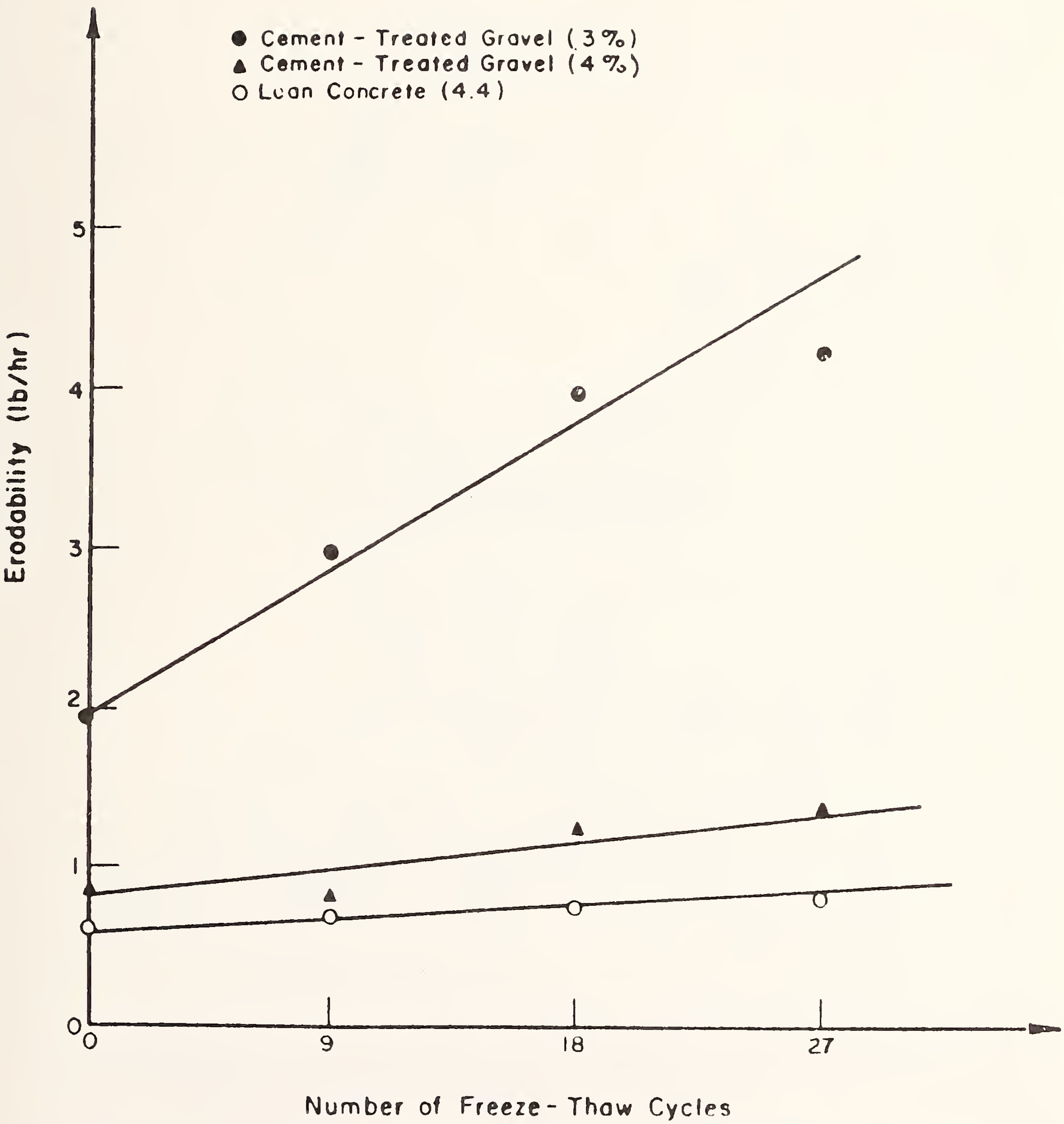

Figure 3.33. Influence of Freeze-Thaw Cycles on Erodibility of Cement Treated Gravel (Ref. 1).

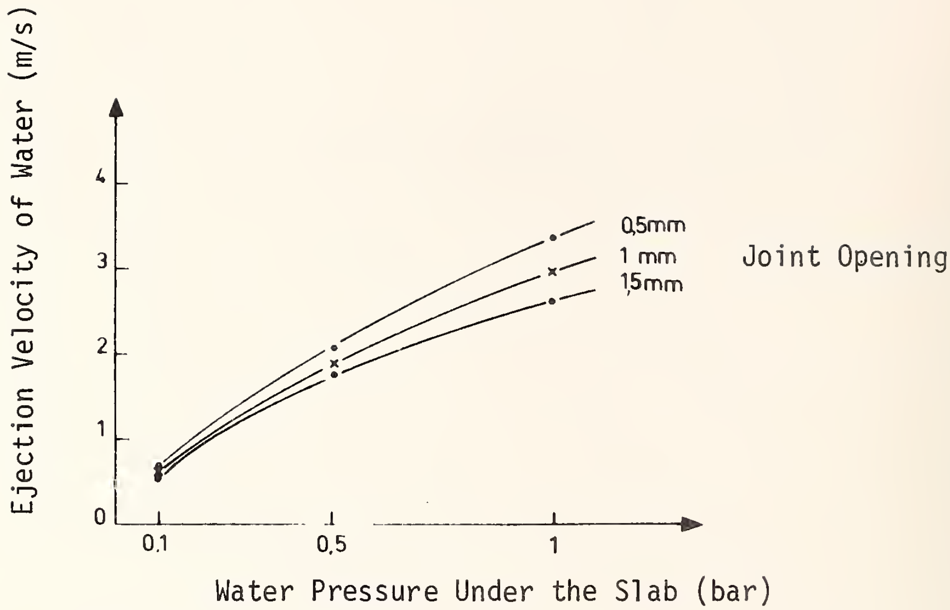

Figure 3.34. Influence of Pressure on Ejection Velocity for Different Joint Openings (Ref. 21).

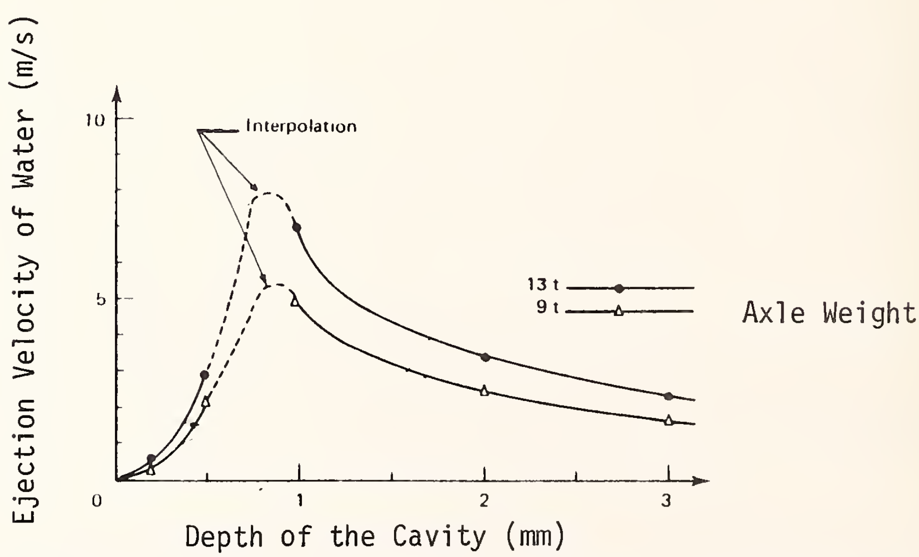

Figure 3.35. Influence of Void Size Under Slab on Ejection Velocity (Ref. 21).

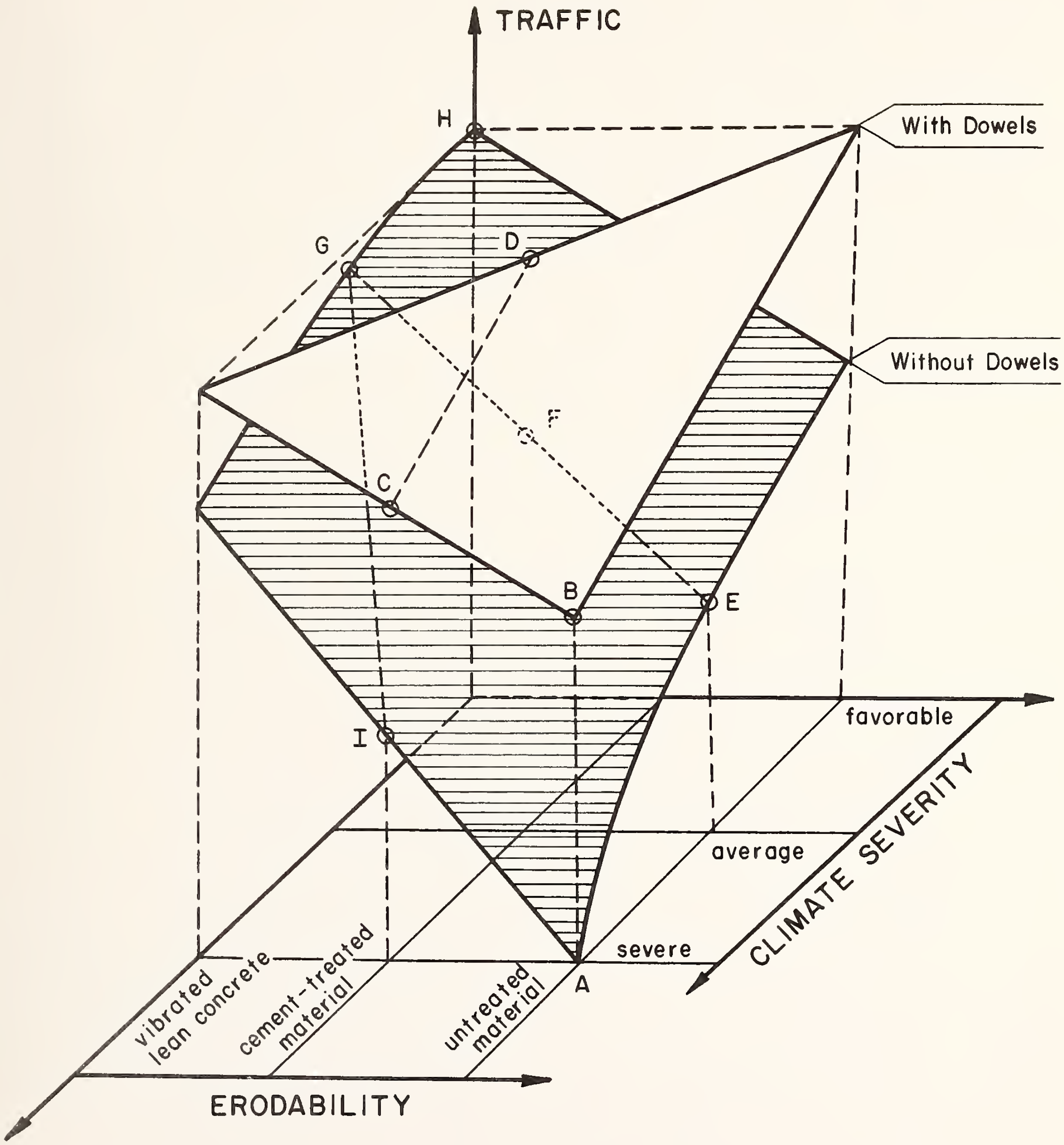

Figure 3.36. Areas Representing Traffic Limits, with and without Dowels, Depending on Erodibility and Climate Conditions (Ref. 22).

Chapter 4

PAVEMENT DRAINAGE DESIGN IN EXPANSIVE SOILS

4.1 GENERAL

The drainage requirements for the shoulders of pavements on expansive clays must account for fourteen major factors, some of which are associated with local climatic and geologic conditions and others which are related to construction details.

The amount of damage that an expansive clay or shale can do to a pavement requires careful attention to most of these factors at any given site.

The climatic factors are as follows:

1. Frequency of precipitation and evaporation forces.
2. The depth to which shrinkage cracks have penetrated or will penetrate the natural ground.
3. The distance from the edge of the shoulder toward the center of the pavement within which moisture (and thus shrinking and swelling activity) will vary with the seasons.
4. The possibility of condensation of moisture vapor beneath the pavement surface due to daily temperature cycles.
5. The likelihood of water from melting snow or rain penetrating into the natural soil along contraction cracks that have occurred upon freezing.

The geologic factors are as follows:

6. The depth of water table, if any.
7. The slope of the terrain and soil type.
8. The depth of each soil layer on site: whether any permeable layer occasionally carries water and whether an inactive

layer is present which limits the depth of vertical and lateral movement.

The construction factors are as follows:

9. Lateral drainage to the roadway ditches.
10. Longitudinal drainage along the shoulder or pavement.
11. The presence of utility trenches or storm sewer pipes beneath the shoulder or pavement.
12. Whether the section is in a cut, fill, or on natural ground.
13. Construction joints between pavement and shoulder which are a natural plane of weakness that can be opened by shrinkage or freeze-contraction and can permit access of water.
14. Future plans for releveling or rehabilitating the pavement and shoulder.

Successful design and construction practice will always strive to work with nature, to keep the soils in as stable a moisture and temperature condition as can be maintained practically. A good practice in expansive clays is to arrange the moisture regime in the natural soil so that it doesn't change once the pavement has been placed. The drainage plan for the shoulders must be integrated with the overall plan of construction, maintenance, and rehabilitation of the pavement.

All thirteen factors should be considered at each site where expansive clay or clay shale occurs. However, in some climatic regions, several of these factors are of only minor importance. For the purpose of this discussion, it is convenient to divide the United States into six broad climatic zones each of which require somewhat different drainage practices.

These six zones are shown on the map of the United States in Figure 4.1. In this figure, the humid and sub-humid climates are divided from the arid and sub-arid climates by a Thornthwaite Index of 0. The Thornthwaite Index is a climatic rating indicating the net annual moisture balance and is explained more in detail elsewhwere (1,2). A positive number indicates that rainfall exceeds the potential evaporation from the ground surface and potential transpiration through plants. The potential evapo-transpiration is calculated from monthly maximum temperatures.

The boundary separating the non-freeze from the freeze-thaw cycling zones is taken from U.S. Weather Bureau data as reported in the Highway Research Board Special Report No. 1 (3). The boundary line represents an extreme frost penetration of 13 cm (5 in.). This contour was selected because the 13 cm (5 in.) depth is a minimum depth of nonexpansive base course or fill material which will be placed on the shoulder. Once frost penetrates this layer it can affect the behavior of the expansive soil below it.

The boundary separating the freeze thaw cycling zones from the hard freeze zones was taken from a Corps of Engineers contour representing a 60-day duration of the normal freeze index (4). Any area in which freezing conditions persist for more than two months is considered to be in the hard freeze zone. The six climatic regions are as follows:

1. Humid, no freeze.
2. Humid, freeze-thaw cycling.
3. Humid, hard freeze.
4. Arid, no freeze.
5. Arid, freeze-thaw cycling.
6. Arid, hard freeze.

Table 4.1 indicates in a general way the major drainage factors to be considered in each zone. It should be noted that all construction factors should be considered in all zones and virtually the same is in the case with geological factors. Thus, the factors which determine the appropriate shoulder drainage practice are primarily climatic.

4.2 DRAINAGE PRACTICES

Drainage practices may be divided into six broad classes:

1. Membranes and moisture seals, Figure 4.2.
2. Interceptor drains, Figure 4.3.
3. Subdrainage of the base course, Figure 4.4.
4. Waterproofing the soil mass.
5. Shoulder moistening.
6. Preventive maintenance.

These drainage practices are placed in conjunction with pre-construction or post-construction techniques which are used to reduce differential movement and cracking. Pre-construction techniques might include: stabilizing the subgrade, ponding, sand wells, removal and replacement, design of the pavement cross-section to prevent ponding or water entrapment, and longitudinal ditch design to promote rapid drainage and discourage infiltration. Post-construction techniques include electro-osmosis, re-leveling with heater-planer or milling machines, overlaying, and reconstruction.

Membranes and Moisture Seals. As seen in Figure 4.2, these include vertical membranes carried below the depth of cracking, horizontal membranes placed on the subgrade and carried to the side of the pavement and shoulder at least one edge distance beyond the traveled way, and sealing the surface of the shoulder. An estimate of the edge moisture variation distance may be taken from Figure 4.5 (5,6). The distance will depend upon the amount of time each year that wetting or drying influences may cause moisture changes beneath the pavement. Consequently, the drying distance is larger than the wetting distance in the dry climates which have a negative Thornthwaite Index. The converse is true in the wet climates which have a positive Thornthwaite Index. The curves in Figure 4.5 are composites from several sources in which moisture changes beneath covered areas were measured. An example of such measurements is in work by Russam and Dagg (7). The curves in Figure 4.5 may be used as estimates of the edge moisture variation distance but should be verified by field moisture measurements when it is possible. The bands indicate expected variations in the mean value of these edge distances since they will depend largely upon the degree of fracturing of the soil and the resulting water conductivity of the soil mass.

Interceptor Drains. Interceptor drains may be placed vertically at the edge of the shoulder and carried down below the depth of cracking. In some cases, interceptor drains are required to cut off the flow from subsurface strata such as occurs when a hard clay underlies a pervious silty material. In this case, water infiltrates down to the top of the clay layer and flows down slope within the pervious material. The filter aggregate placed in these drains should be freely draining and should be protected from the intrusion of fines. As seen in Figure 4.3, a durable, impermeable membrane should be placed between the filter aggregate and the clay subgrade and should be extended beneath the bottom of the drainage pipes. The drainage pipes, in turn, should always be placed in the bottom of the interceptor drains and should be connected to storm sewers or other drainage outlets.

In the first few years after they have been placed, interceptor drains may act as an impermeable membrane as long as only water vapor travels in the filter medium. However, when liquid water becomes available to the filter material it will conduct water very well and begin to function as a drain. The unsaturated permeability of the sand and gravel material used in these drains is generally very low when they are dry or when they have only surface films of water. However, as they approach saturation, their permeability increases significantly. If not functioning properly, interceptor drains can become reservoirs of free water which will be available to cause large differential movements in the adjacent pavement and the shoulder. It is for this reason that the drain should be carefully designed and placed, and an impervious membrane should be placed below the drain and along the pavement side of the interceptor trench.

Sub-drainage of the Base Course. As seen in Figure 4.4, these are horizontal drains placed at the bottom of the base course to carry out infiltration or condensation water to the side of the pavement where it is collected in side ditches or in longitudinal conduits which are connected to storm sewers or other drainage outlets. Permeability requirements for base course drainage can be found in work by Moulton (8).

Waterproofing the Soil Mass. This is a general term for any treatment of the soil that prevents or reduces infiltration. Chemical treatment of the surface of the subgrade with a stabilizing agent or with a hydrophobic surfactant is included in this class of drainage practices. Pressure-injected lime slurry tends to fill the cracks in the soil mass and provides some waterproofing. (9)

Shoulder Moistening. In some areas of the United States where the climate is normally humid, a particularly dry year or number of years produces drying beneath the edge of a paved surface which results in shrinkage cracking. This problem can normally be handled by providing an extra width of paved shoulder which can be resealed by maintenance crews without being exposed to traffic. However, in some urban situations where side-distances are constricted, and there is large average daily traffic, it becomes more feasible to consider a system of moistening the soil below the shoulder to prevent shrinkage cracking. The moistening can be done by using trickling sub-irrigation conduits with frequent small apertures to provide the moistening water. The amount of water provided can be controlled by moisture sensing devices which can provide an electrical signal to a monitoring and controlling device.

Preventive Maintenance. This is one of the more important drainage practices in an expansive soil area. Roadside ditches that have a minor obstruction in them can pond a sufficient amount of water to cause a large heave in the adjacent pavement. Cracks in the pavement or shoulder can provide ready access to the moisture reactive subgrade. In some cases, the construction joint between the pavement and the shoulder is a major cause of moisture-induced roughness and loss of strength. Water pumping out of the construction joint indicates that water has been trapped beneath the pavement and is not draining out through the shoulder. Frequent visual surveys to determine the current state of the drainage condition of the pavement are essential to an effective preventive maintenance program.

Each of these six drainage practices will be discussed more in detail as they apply in each of the six climatic regions of the United States.

4.3 CLIMATIC REGION I - HUMID, NO FREEZE

Distress. This region is characterized by a positive Thornthwaite Index, plentiful rainfall, and wet subgrade soils. Reactive soils in the region will have shrinkage cracks during the dry part of the year which will provide ready access of water beneath a paved area. The soils near the surface are normally at or somewhat drier than the water content they will have when they are covered with a pavement. The characteristic distress suffered by pavements on reactive soils in this climatic region are differential heaving and longitudinal shrinkage cracking. Heaving is particularly bad in areas which are poorly drained or where water can enter under the pavement in poorly compacted backfill or along the interface between drainage structures and the soil. Shrinkage cracking occurs during the dry season within 1 to 1.5 m (3 to 4.5 ft) of the edge of the paved area. As soon as the crack opens, drying air enters the crack and begins to dry out soil that is still farther beneath the pavement. When sufficient energy is built up in the drying soil it will crack itself once more and advance the crack up through the overlying pavement layers. These open cracks provide ready access for water to enter the subgrade. Also, vegetation begins to grow in these cracks.

Pre-Construction Treatment. Successful methods of pre-construction treatment of these soils involves pre-wetting by ponding or by the use of sand wells, and in some cases by lime stabilization.

Climatic Factors. The appropriate drainage practice for this climatic region must take into account the type of pre-construction treatment which has been used so as to prevent the eventual appearance of distress.

If the subgrade has been ponded or pre-swollen with sand wells, longitudinal cracking must be prevented by horizontal or vertical membranes to prevent moisture loss. Horizontal membranes normally consist of sealed shoulders preferably at least two edge distances wide. The edge distance in this climatic region is in the order of 1 meter (3.3 ft). Although it has not been used, shoulder moistening could be employed to keep the soil moisture constant beneath the pavement.

Vertical membranes should be carried below the depth of seasonal cracking. This depth can be inferred from natural water content profiles. The depth at which the ratio of water content to plastic limit becomes a constant can be assumed to be the cracking depth. In this climatic region this depth will range from 1 to 2.5 m (3 to 8 ft). Desiccation cracks will be found at greater depths but their width will be found to be too narrow to transmit free water during the course of a wetting or drying season. Experiments on cracks in concrete have shown this minimum crack width to be about 0.058 cm (0.023 in.) (10).

If the subgrade soil has been lime stabilized, the lime stabilization should be protected from moisture cycling by horizontal or vertical membranes since if it is not protected it will eventually break up into small nodules about the size of gravel which will transmit either additional moisture or the drying influence of low humidity air and will only result in delaying the appearance of the characteristic distress.

Geologic Factors. The geologic factors that are important in this climatic region are high water table, flat slopes and the consequent poor drainage, water bearing strata on high slopes, and highly permeable layers interbedded with the expansive clay.

The level of high water tables can be stabilized by using interceptor drains placed down to the depth of cracking. If the water table is subject to seasonal fluctuations which could cause differential heaving and settling of the clay subgrade, it is best to stabilize the water table level at its lowest expected elevation. If this is impractical because of the required costs, the interceptor drain should be set to the depth of cracking and a certain amount of pavement roughness should be expected.

Highly permeable layers and water-bearing strata on slopes should be intercepted in a similar way.

Poor drainage due to flat slopes is a chronic problem in this climatic region. If water ponds in roadside ditches or in the bottom of drainage structures, it will move quickly into the cracks in the clay and cause differential heaving and tension cracking in the paved surface. Even a small obstruction in a drainage ditch can be a cause of major repair and maintenance work. The cure for this kind of problem requires careful attention to detail in the drainage plan.

The stratigraphy of the road project is important in determining the amount of differential movement which may occur. In areas where dipping strata of clays, shales, and interbedded coarse-grained materials have their contacts, there maybe a considerable variation in the amount of differential movement and pavement cracking. The variation will depend mainly upon the depth at which the inactive layer(s) is encountered. Anchoring membranes into the inactive material can be a good drainage strategy if that material is also impermeable.

Construction Factors. All construction factors should be arranged to keep moisture beneath the pavement and never to allow the subgrade soil to dry out or get any wetter. Lateral and longitudinal drainage should be carefully planned with sufficient slope to prevent ponding against obstructions for more than a day. Utility trenches,storm sewer pipes, or drainage inlet leads beneath a pavement or a shoulder may provide channels of disturbed soil or poorly compacted backfill in which liquid water can find ready access. Positive steps should be taken to seal off each end of such trenches from access to water by vertical membranes down to the depth of cracking or to the bottom of the trench, whichever is deeper. Failure to do so will assure a localized area of continuing maintenance problems.

Construction joints should be kept sealed to prevent both drying and excessive wetting.

Pavement sections in cuts become rougher with age because of the drainage problems normally associated with cuts and with differential rebound due to the removal of overburden. Attention to detail and care in preparing and executing the drainage plan in the cut is essential to maintaining a serviceable pavement. Medians should be used for drainage only rarely for they are chronic sources of excessive wetting and distortion. An exception would be when the drainage water in a median is dropped directly into a culvert which can take the water away rapidly and which can be inspected and have its barrels cleaned out periodically. Generally speaking, drop inlets and inlet leads are usually causes of undesirable roughness.

Pavement sections at natural grade can usually be expected to be rougher than those built on fill because of the variability of the natural soil and its developed network of shrinkage cracks. Horizontal or, more appropriately, vertical membranes are usually good drainage practices to use at natural grade.

Fill sections may tend to lose moisture and crack due to evaporation or transpiration from plants and grasses on the side slopes. To the extent that it is possible, membranes or insulating layers of non-reactive soil should be used to prevent such moisture loss. If this is not done, a certain amount of shrinkage cracking at the edge of the paved surface should be expected.

Future releveling and Rehabilitation. No drainage practice that is adopted during the new construction phase will be completely successful in preventing differential movement or pavement cracking. The cost of applying any drainage method during new construction should be balanced against the present value of future expected costs of releveling, cold-milling, overlaying, or reconstructing the pavement in the future. The strategy which offers the least total cost is the best one to adopt. Generally speaking, this will dictate that great care must be used in drainage and construction details to prevent excessive wetting at those locations where drainage problems exist and less than complete treatment along the rest of the length of the pavement section. It is reasonable to expect more frequent rehabilitation and maintenance activities on these sections than on those pavements which are roughened by traffic loading alone.

4.4. CLIMATIC REGION II - HUMID, FREEZE-THAW

The primary drainage problem in the humid climatic regions is that of keeping the water in the subgrade soils and preventing excessive wetting. All of the problems and solutions mentioned in the discussion of Region I also apply to Region II with the added complications that are brought about by freeze-thaw cycling and thermal cycling that can cause excessive wetting due to condensation.

Distress. Pavements on expansive clays in this region suffer differential movement due to excessive wetting and drying as well as shrinkage cracks due to drying or contraction or expansion on freezing.

Pre-Construction Treatment. The primary means of pre-treating expansive soils involves lime-stabilization and sometimes the removal of the top 0.5 to 1.5 m (1.5 to 4.5 ft) of the subgrade soil and replacement with inert material. Pre-wetting by various means may be tried successfully if the surface soil is initially drier than its eventual expected moisture content.

Climatic Factors. The discussion of Region I climatic factors applies as well to this region with two prominent additions: condensation and freeze-thaw cycling. To prevent the undesirable effects of condensation water collecting beneath the pavement and causing differential swelling, it is useful to place an impermeable membrane on top of the subgrade for the entire width of the pavement and shoulders. Internal drainage should be provided in the base course to conduct the condensed water out of the pavement to collectors.

Freeze-thaw cycling may have several undesirable effects. Freezing always decreases the humidity of soil moisture which tends to shrink the soil. If the shrinkage tendency is not offset by an expansion of the pore water that turns into ice, then the soil will develop tension cracks similar to shrinkage cracks. These cracks, in turn, will provide access to moisture from melted snow which will run down the crack, move horizontally along the interfaces, between pavement layers, re-freeze, and cause large ice-heaves centered on the original crack.

The critical water content below which thermal cracking will occur is usually wet of optimum but the precise water content will vary with the predominant clay mineral in the subgrade soil. The best drainage strategy to avoid thermal cracking is to keep the subgrade more moist than the critical water content by using vertical or horizontal membranes. The critical water content can be determined in the laboratory by running freeze-thaw tests on sealed samples of subgrade at various water contents and measuring whether expansion or contraction occurs.

If the strength of the soil in the moist state is too low to bear traffic stresses, the subgrade should be stabilized or additional thickness of base course should be used or both. The subgrade can be too wet, also, both for the purpose of strength and for preventing expansion cracking due to the formation of ice in the pore spaces. The choice of the proper water content is a careful balance between the strength and contraction cracking requirements. Once the water content has been achieved, membranes should be used to maintain the desired level at optimum to optimum plus 3 percent.

Geological Factors. All of the geological factors mentioned in climatic Region I also apply in this region. The obvious addition is that of the freeze-thaw cycling effect on clay subgrades with high water tables. A water table that is within 10 m (33 ft) of the subgrade elevation can be the indirect cause of expansive cracking of the pavement. Stabilization of the subgrade helps to control this effect.

Construction Factors. All of the construction factors in climatic Region I also apply to this region. All techniques that will prevent drying and excessive wetting, and preserve the required strength should be used.

Future Releveling and Reconstruction. The present value of future expected costs of releveling, crack-sealing, cold-milling, overlaying, or reconstructing should be balanced with initial construction costs (membranes, pre-wetting, stabilization) to determine the optimum combination of the two.

4.5 CLIMATIC REGION III - HUMID, HARD FREEZE

Expansive soils are seldom encountered, but occasionally can cause maintenance and rehabilitation problems which are peculiar to the colder climates. The problems of shrinkage or expansion cracking caused by a deficiency or excess, respectively, of soil moisture become the major problems in this region. Drainage is critical; the correct water content must be established and maintained; both shrinkage and expansion must be prevented.

Distress. Freezing phenomena begin to dominate the distress patterns observed in pavements on expansive clay in this region. Shrinkage cracks (transverse) or expansion cracks (longitudinal) become filled with water which turns to ice and begins to expand the crack. After the spring thaw, the crack becomes a source of water to the subgrade which weakens it, makes it susceptible to rutting, and permits it to swell unevenly about the crack. In Canada, this phenomenon is known as "ridging" and although its causes are not well understood, the drainage practices which will contribute to its reduction can be surmised (11).

Pre-Construction Treatment. Removal of the top 0.5 to 1.5 m (1.5 to 4.5 ft) of the subgrade soil and replacement with an inert material is the safest but not necessarily the most economical pre-construction treatment. However, if both initial cost and rehabilitation costs are considered together, the removal and replacement treatment may become a very cost effective strategy. In this climatic region, the thermal properties of the subgrade and the temperature regime becomes the most important consideration in determining the proper pavement drainage practice to use (12). As in climatic Region II, all drainage practices should be arranged to keep the water content stable at the critical level where neither expansion nor contraction occur if at all possible.

Climatic Factors. The severe temperature excursions above and below freezing account for thermal cracking in asphalt pavements and can cause contraction or expansion cracking in the subgrade as explained previously in the discussion of climatic Region II. The major climatic distinction between this region and Region II is the depth and duration of freezing and the rapidity with which pavements can become unserviceable as a result. Vertical or horizontal membranes can help to moderate a severe condition by providing moisture stability but their cost and the increased effectiveness they provide should be balanced against the present value of maintenance and rehabilitation costs to determine whether it is economical to use them. If vertical membranes are used, they should be carried to the depth of cracking or the depth of freeze penetration, whichever is deeper. If this cannot be done for economic or some other reasons, then pavement cracking, "ridging", and consequent roughness may be expected.

Geological Factors. All of the geological factors that apply in Regions I and II also apply in this region. A high water table that is within 10 m (33 ft) of the subgrade elevation can be the indirect cause of expansive cracking of the pavement. Stabilization or removal and replacement of 0.5 to 1.5 m (1.5 to 4.5 ft) of the subgrade soils with inert materials will aid in controlling this effect.

Construction Factors. All construction factors in Regions I and II also apply here. Emphasis should be placed on maintaining a stable water content near that at which neither expansion nor contraction will occur and sufficient subgrade strength is retained. This water content may be difficult to determine exactly even in the laboratory because the soil behavior may change with a small change of water content from contractive to expansive. Consequently, water contents in the field may be even more difficult to control uniformly.

Future Releveling and Reconstruction. The cost effectiveness of various drainage features in reducing future maintenance and rehabilitation costs in this severe climate becomes the most important consideration in deciding whether to use them or not.

4.6 CLIMATIC REGION IV - ARID, NO FREEZE

This climatic region shares many features in common with the humid, no freeze region (Region I). However, because the soils are normally drier in this region, it becomes more necessary to keep water out of the subgrade rather than to retain it, as was the case in Climatic Region I. In either case, excessive wetting must be excluded by proper drainage. Also in this region, subgrade soil behavior is different depending upon whether it is

1. a fractured, weather soil,
2. a weathered shale, or
3. an intact shale.

In the fractured soil, roughness patterns in the pavement surface are a reflection of the cracking patterns in the subgrade soil and the cracks provide ready access of water beneath the shoulder and the pavement. Weathered shale is capable of developing roughness patterns that are as severe but the rate at which they develop is largely determined by the degree to which an interconnected cracking pattern has developed within them. Intact, unweathered shale that is never exposed to wetting and drying influences will be a stable subgrade material in this climatic region (Region IV). However, in Regions V and VI, this same material can cause trouble due to contraction when it freezes.

Distress. The primary kinds of distress are differential movement and longitudinal cracking. Differential movement is caused partly by swelling and partly by shrinkage. In this region, there is a greater frequency of edge drying than in Region I and drainage problems cause more serious heave problems. Shrinkage cracking patterns are more likely to be interconnected here and will provide ready access of water beneath shoulders and pavements.

Pre-Construction Treatment. Pre-swelling by ponding coupled with vertical or horizontal membranes are used frequently in this region. Stabilization of the subgrade to provide a working table, deep-plowing of lime, careful attention to drainage details, and in some cases, removal and replacement with inert material are also used successfully here. Pressure-injected lime slurry has a better chance of working well here especially if it is applied during dry weather when the shrinkage cracks are open. It may be used in spot locations where its higher cost per unit area may be justified. Trenches for utilities, storm sewers, and inlet leads are sources of special difficulty. Lean concrete backfill has been used to combat trenching problems. In some cases, utility casings have been bored under the completed pavement in order to avoid backfilling a trench. The natural cracking fabric of the soil mass has defeated attempts to prevent heave over the utility by providing access to water along the length of the casing. Interceptor drains have been used in this area but if they are done improperly, they will eventually become sources of water to cause heave. The drainage trenches should be lined with impermeable membranes to keep water out from beneath the shoulder and pavement (Figure 4.3).

Climate Factors. Shrinkage cracks will penetrate 1 to 3 m (3 to 10 ft) with adequate width to transmit liquid water in sufficient quantities to cause vertical heave. In some special cases, these depths may be greater. The edge distance within which moisture will vary seasonally may vary between 1 to 1.5 m (3 to 5 ft) and in the more severely dry years this distance may be 1.5 to 2.5 m (5 to 8 ft). Condensation beneath the pavement can occur in this region wherever daily temperature cycles are large enough and the relative humidity is high enough.

Horizontal membranes or sealed shoulders should be paved 1.5 to 2 edge distances wide and vertical membranes should be at least as deep as the depth of seasonal cracking. Shoulder moistening could be used in this region but the process should be carefully controlled, for too much water in this region will give as much or more trouble than too little. As in Region I, lime stabilization should be protected from excessive wetting and drying cycles.

Geologic Factors. Water table depths are usually deeper than 10 m (33 ft) in this region although there are exceptions to this general rule. Flat slopes and poor drainage, water bearing strata on sufficient slope to provide water beneath the pavements, and highly permeable layers inter-bedded with the clay are involved in most of the pavement and shoulder distress that occurs.

Interceptor drains can successfully cut off flow in permeable layers but the cut off should be made before the pavement and shoulder is placed in order to allow the subgrade soil sufficient time to dry out to a moisture condition that is more nearly what it will be in service.

As in Region I, careful attention must be applied to maintaining a clean lateral and longitudinal drainage system that is free of obstructions.

The depth at which an inactive layer may be found can be of consider-able practical importance, for it will dictate the depth to which vertical membranes may be carried.

Construction Factors. The same comments apply in this region as in Region I, with the exception that all drainage practices in this region are in-tended to keep water out. Greater differential movements may occur here than in Region I if excess moisture penetrates beneath the shoulder. The same drainage practices also recognize that periods of edge drying and shrinkage cracking are more intense in this region and that this type of distress must also be prevented.

Future Releveling and Rehabilitation. Pavements and shoulders that have been roughened by expansive clay may be treated by cold milling, heating and planing, applying a leveling course followed by an overlay or a friction seal course over both pavement and shoulder, or by reconstruction. The present value of the cost per unit area of these techniques should be balanced against the unit cost of pre-construction techniques in choosing the most cost effective method of building and maintaining the pavement in service.

4.7 CLIMATIC REGION V - ARID, FREEZE-THAW

Expansive soils in this region will normally be drier than optimum water content, a condition which promotes freeze contraction in the subgrade soils and can, under the proper conditions, cause crack reflection through the pavement above. A stiff, fissured clay or a shale subgrade in this region will be more likely to cause this kind of distress. It is not uncommon to see transverse shrinkage cracks in the pavement extend beyond the shoulder and into the natural soil beyond.

In all other respects, this region is similar to Climatic Region IV, including the need to plan drainage so as to exclude water from entering the subgrade.

Distress. The primary kinds of distress are differential movement, longitudinal cracking due to moisture variation along the edge of the sealed surface, and transverse cracking due to drying or to freeze contraction. Large heaves will occur where even minor obstructions in drainage patterns are found or where free water gains access beneath the pavement.

Pre-Construction Treatment. All of the same methods of pre-construction treatment discussed under Climatic Region IV are also used in this region. In addition, where it is feasible for other reasons such as strength requirements, greater thicknesses of pavement also act as insulation above the freeze-active subgrade and can help to prevent that kind of distress.

Climatic Factors. Shrinkage cracks will penetrate 1 to 3 m (3 to 10 ft) in depth and their spacing will depend largely upon the degree to which they have been weathered. The spacing in clays may be very close (1 to 8 m) (3 to 25 ft) whereas in shales it can be large (30 to 60 m) (100 to 200 ft). The spacing is an indication of the tensile strength of the intact material. These are typical depths and spacings, and exceptions to these will be observed. The edge distance within which moisture will vary is the same size as in Climatic Region IV. Condensation can collect beneath pavements in this region provided cyclic temperature variations and humidity are high enough. Where this occurs, horizontal membranes should be placed directly on the subgrade for the full width of the pavement and shoulders and lateral drainage should be provided to keep this water from entering the subgrade.

Longitudinal cracking due to edge moisture variation can be reduced by sealing shoulders 1.5 to 2 edge distances wide or by setting vertical membranes at least as deep as the depth of seasonal cracking. Carefully controlled shoulder moistening may be useful in this region.

The same comments made about lime stabilization in Regions I and IV also apply here, i.e., the stabilized materials should be protected from excessive wetting and drying cycles. In addition, the material can be degraded by freeze-thaw cycling and any insulation qualities of base and surface courses will reduce this damage.

Geologic Factors. The geologic factors to be considered are the same in this region as in Region IV, and will not be repeated here.

Construction Factors. The same construction factors apply in this region as in Region IV, with the exception of the insulating qualities of the base and surface layers to reduce transverse cracking due to freeze-contraction of the subgrade soils.

Future Releveling and Rehabilitation. The comments made on these factors in Region IV also apply to this region. Transverse cracking due to freeze-contraction may occur in the asphalt surface course, the base course, or the subgrade. The ideal method of rehabilitating such a pavement is to scarify the entire pavement structure, destroy the existing cracking pattern, and re-construct. Because this technique may not always be economically feasible, as great a depth as possible should be reconstructed. Pre-construction and post-construction treatment methods should be chosen to give the most cost effective combination over the life of the pavement.

4.8 CLIMATIC REGION VI - ARID, HARD FREEZE

There is very little difference in the behavior of expansive clay or shale subgrades in Regions III and VI because freezing phenomena begin to dominate the distress patterns and corrective drainage measures adopted. The major difference to be found is that the subgrade soils in this region are drier than in Region III, with the consequent prevalence of freeze-contraction phenomena. However, if an area is poorly drained, or there is a high water table, expansion cracking and distortion can occur. It will usually be difficult to maintain a water content in the subgrade at which neither expansion nor contraction will occur. As a consequence, careful attention must be paid to the thermal regime and to the insulating properties of all layers above the subgrade to reduce the expected freeze-contraction cracking.

Distress. The distress caused by expansive soils in this region are similar to those in Regions III and V, with the damage due to freezing effects becoming the most important.

Pre-Construction Treatment. Removal of the top 0.5 to 1.5 m (1.5 to 4.5 ft) of the subgrade soil and replacing it with an impervious and thermally inert material is a very effective pre-construction treatment. Lime stabilization of that depth of soil will usually result in a material that still contracts when it freezes. Lab freeze tests should be made on trial samples of the stabilized soil to determine the amount of contraction that should be expected. If the coefficient of thermal contraction of the stabilized material is more than 5 times greater than the layers above it, the stabilized material should not be used and a less active material should be used in its place. All other drainage practices are as discussed in Regions II and III.

Climatic Factors. The same climatic factors apply in this region as in Region III. It is conceivable that carefully controlled shoulder moistening to preserve the critical water content may be employed with some success in this region, although its cost effectiveness may be questionable in all but those areas with high traffic density.

Geologic Factors. Generally, the geologic factors in this region are the same as in Regions IV and V. Low water tables will always suggest that freeze-contraction will occur in the subgrade and, as noted previously, stabilization may not control this problem.

Future Releveling and Rehabilitation. Wherever they occur, transverse contraction cracks will constitute a major weakness in the pavement structure and it is always desirable to re-work them down to the depth to which they penetrate and replace the subgrade soil 3 to 5 m (9 to 15 ft) on each side of the crack with a thermally inert material. Base surface course materials should then be reconstructed over this new material. Even with these precautions, another crack may soon occur at each interface between the subgrade soil and the newly placed material.

When this type of reconstruction is done, vertical and horizontal membranes and drains should be repaired or replaced intact and functioning.

As costly as this kind of rehabilitation work can become over a period of several decades, it may prove cost effective to remove and replace the subgrade soil to the required depth as a pre-construction treatment.

4.9 SUMMARY

The drainage pavement systems on expansive clays can have a significant effect on the performance of those pavements. Because of this, it must be considered an integral part of the design, pre-construction, service, and rehabilitation phases of the life of those pavements. Several factors must be considered in detail in choosing the proper drainage method: distress, climate, geology, pre- and post-construction methods of extending the pavement life, and level of traffic. Of these, the climatic factors (moisture balance and temperature) and local drainage features control the choice of the available means of proper shoulder drainage. There are several broad categories of these drainage practices that are available: membranes and moisture seals, interceptor drains, sub-drainage of the base course, waterproofing the soil mass, shoulder moistening, and preventive maintenance. The cost effectiveness of these techniques should always be considered and added together with the future costs of releveling pavement roughness that is expected to occur because the drainage treatment was less than perfectly effective. The best choice of drainage method will minimize the sum of these costs.

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Table 4.1 Major Drainage Factors in the Climatic Zones of the United States

		FACTORS	HUMID			ARID		
			No Freeze	Freeze-Thaw Cycling	Hard Freeze	No Freeze	Freeze-Thaw Cycling	Hard Freeze
CLIMATE	1	Crack Depth	X	X		X	X	
	2	Edge Distance	X	X		X	X	
	3	Condensation		X		X	X	
	4	Contraction Cracks		X	X		X	X
GEOLOGY	5	Water Table	X	X	X	X	X	X
	6	Slope	X	X	X	X	X	X
	7	Layers	X	X	X	X	X	X
CONSTRUCTION	8	Lateral Drainage	X	X	X	X	X	X
	9	Longitudinal Drainage	X	X	X	X	X	X
	10	Trenches	X	X	X	X	X	X
	11	Cut or Fill	X	X	X	X	X	X
	12	Joints	X	X	X	X	X	X
	13	Rehabilitation	X	X	X	X	X	X

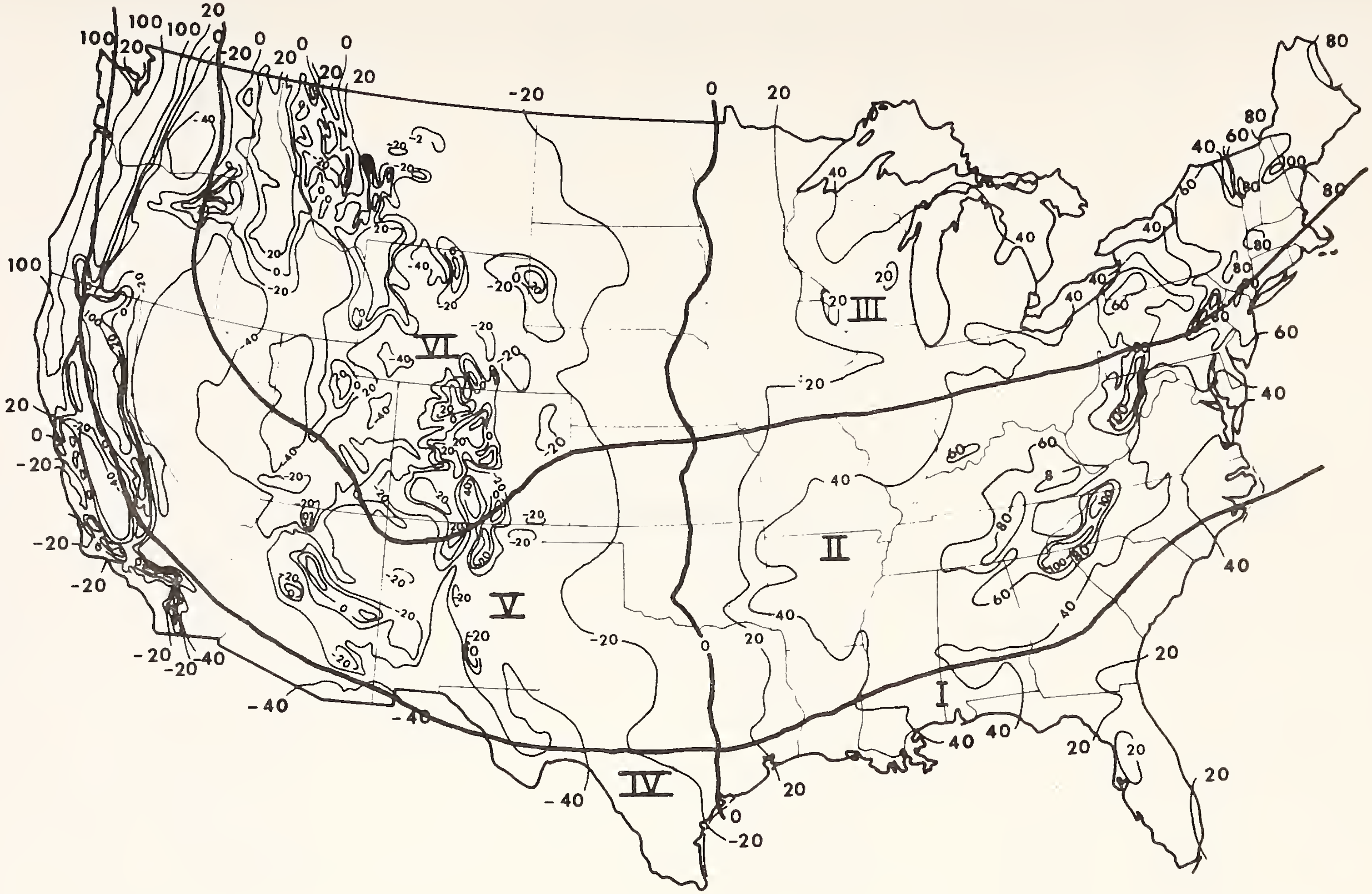


Figure 4.1. Climatic Zones of the United States.

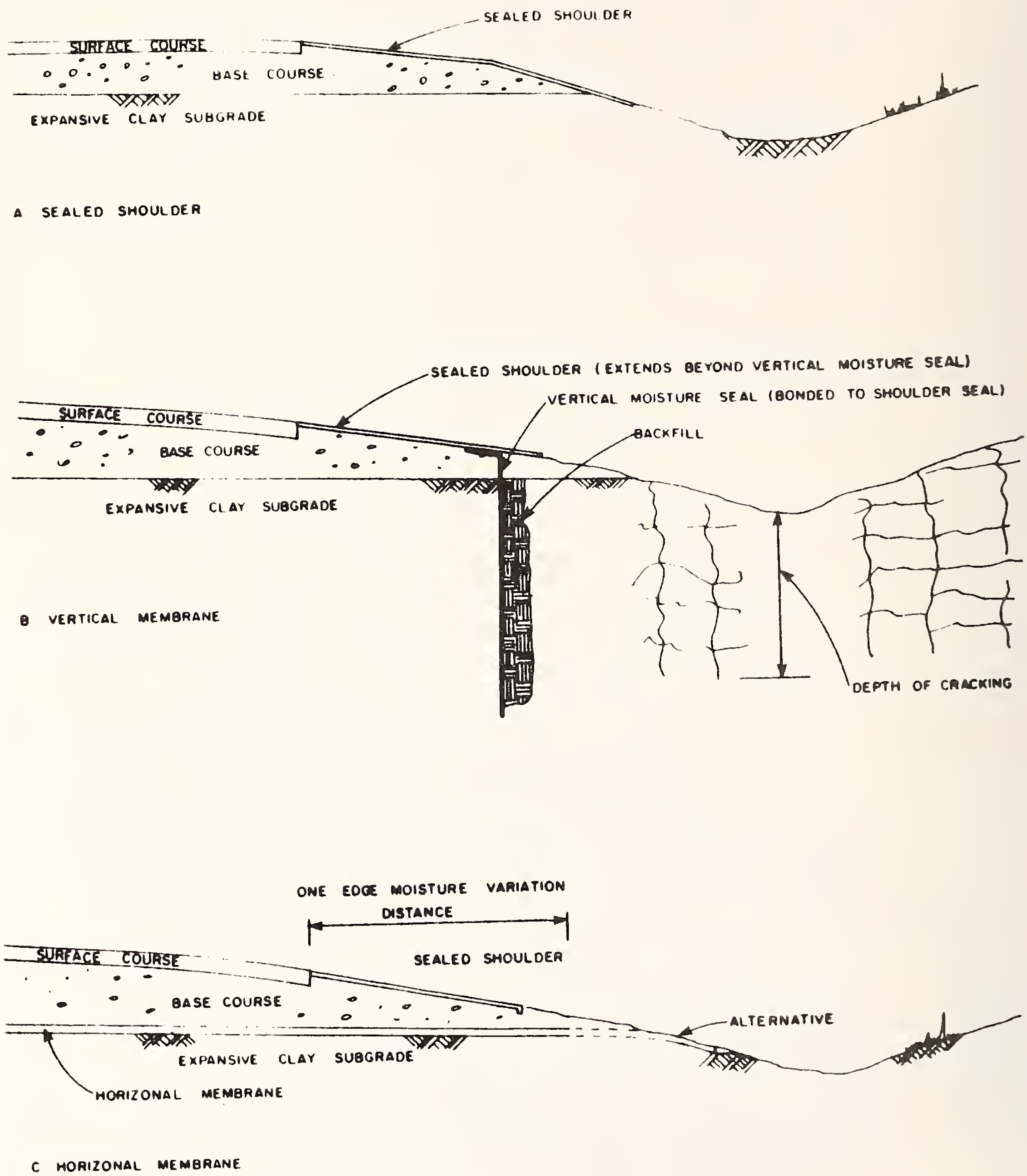


Figure 4.2. Membranes and Moisture Seals.

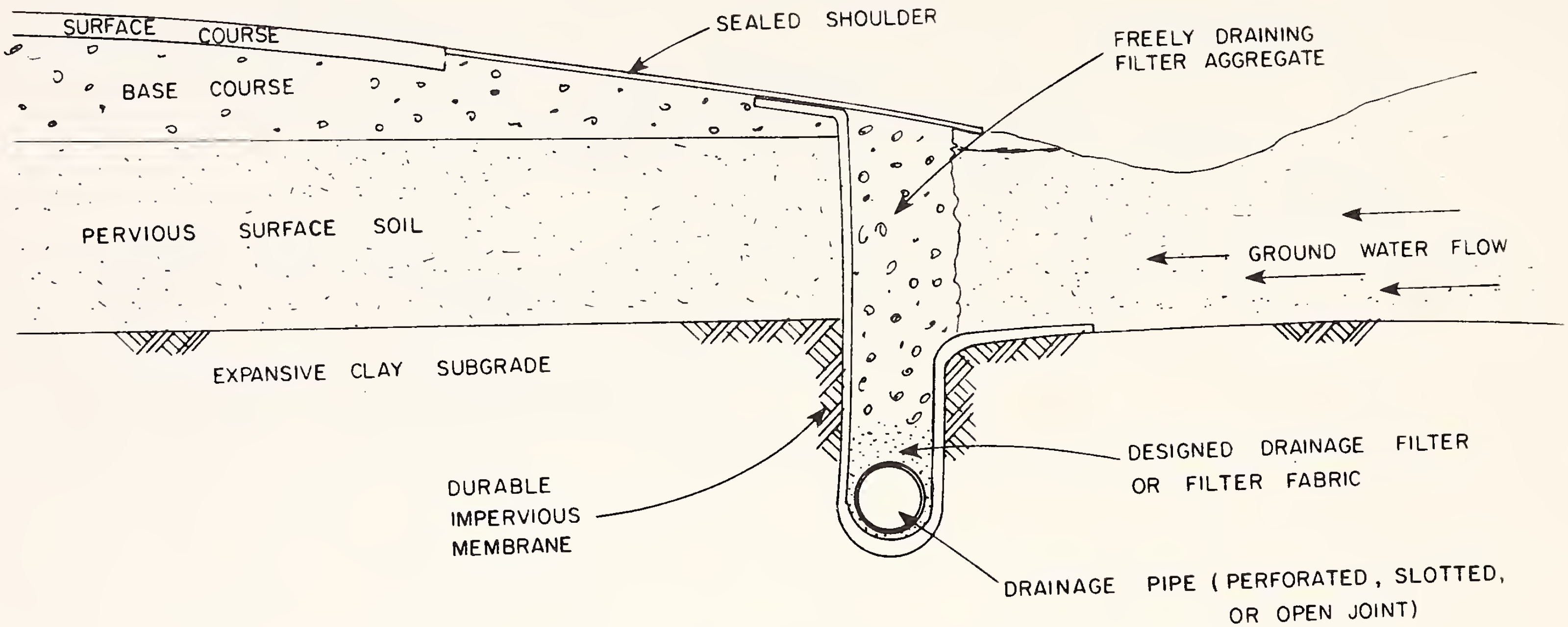


Figure 4.3. Interceptor Drains.

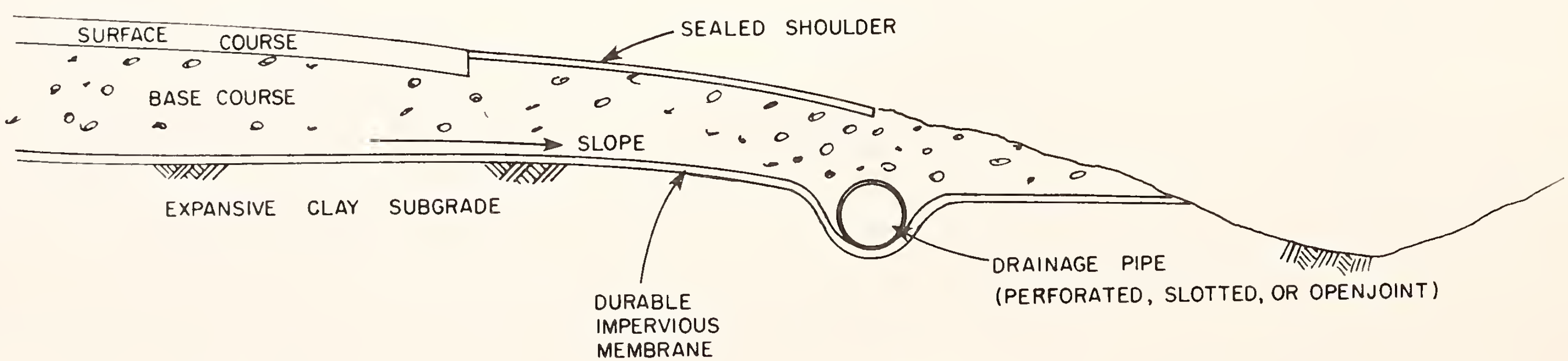
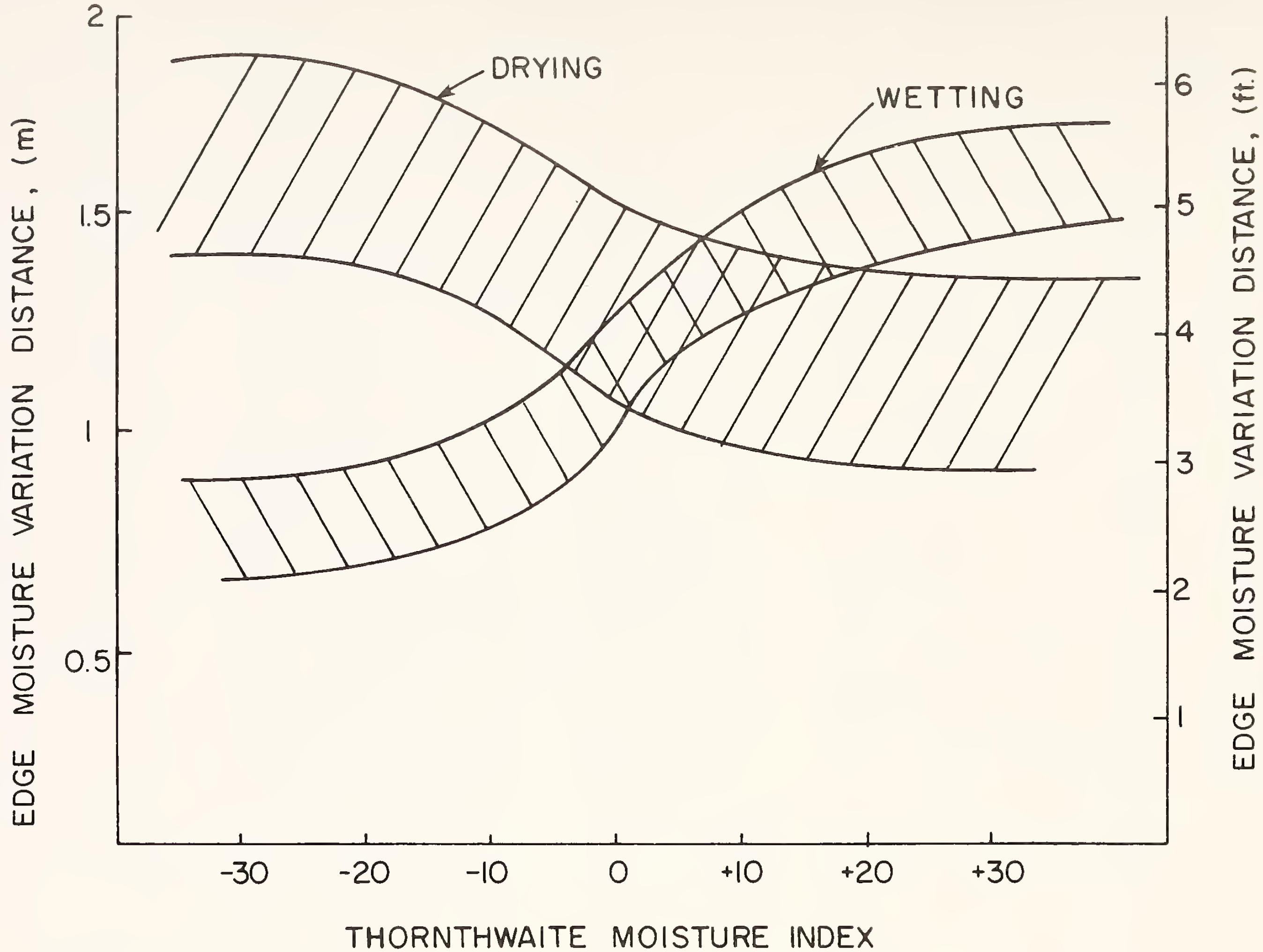


Figure 4.4. Subdrainage of the Base Course.



1 ft = 0.305 m

Figure 4.5. Relationship Between Thornthwaite Index and Edge Moisture Variation Distance.

Chapter 5

CLIMATIC DESIGN CONSIDERATIONS IN SUBSURFACE DRAINAGE

5.1 GENERAL

Because the nature of water movement in soils is complex and involves many variables, a systematic approach to increased knowledge of the subject first requires separating the major variables and determining their relative influence. Recognizing this, the factors influencing the water movement, and thereby the climatic design considerations in subsurface drainage, can be divided into extrinsic (external) and intrinsic (internal) categories.

The extrinsic factors, Figure 5.1, are those which are outside but which act directly on the soil. These factors specify the nature of the climate and loading conditions which exists on and in the pavement system itself.

The intrinsic factors, Figure 5.2, are those which are related to the properties of the soil and its cover. They include the state of the soil mass, the physical properties of the soil mass, the existence of a drainage pipe, the composition of the soil, and the thermal and moisture properties of the soil as well as the envelope material around the drainage pipe.

When designing a drainage system the engineer thus has to take the extrinsic climatic conditions, such as temperature and rainfall, into account together with the intrinsic thermal and moisture properties of the materials.

5.2 EXTRINSIC FACTORS

As indicated in Figure 5.1 two major factors have to be considered as extrinsic factors, namely load and climate. Of these two the climate is the element which is the most variable, complex, and furthermore uncontrollable. Generally, climate is often thought of as the weather and is influenced by combinations of air temperature, wind, precipitation, evaporation, condensation, and solar radiation including both long- and short-wave radiation. The geographical location also influences the climate of an area.

The factors influencing the climatic design can therefore be further divided into the following subcategories:

1. Temperature factors, which directly affect the transfer of heat to or from the ground surface, including air temperature, short- and long-wave radiation received and emitted to or from the ground surface respectively, and wind.
2. Hydrologic factors which exert and indirect influence on the temperature of the ground, because of their effect on the properties of the soil or cover, including precipitation, evaporation, and condensation.
3. Geographical location factors, which exert a direct influence on weather, including elevation, latitude, degree of exposure, and nearness to a water source.

In category 1 Aldrich (1) has described the relationship between air and pavement temperatures as one involving heat transfer at the air-ground interface. The radiation and convection heat transfer are defined in Figure 5.3 for a sunny day, and the principal variables affecting the magnitude of these factors are as follows (Aldrich (1)):

A. Solar and long-wave radiation:

1. Latitude and elevation of pavement site.
2. Atmospheric vapor pressure.
3. The type and amount of cloud cover.
4. Atmospheric conditions (clear or hazy).
5. Type of surface (i.e. color and texture).

B. Convection:

1. Wind velocity.
2. Type of surface (roughness).
3. Type of topography and vegetation near the pavement site.

In category 2 Low and Lowell (2) have stated the following sources of pavement moisture, Figure 5.4:

1. Moisture may permeate the sides, particularly where coarse-grained layers are present or where surface drainage facilities within the vicinity are inadequate.
2. The water table may rise (can take place in the wet seasons of the year).
3. Surface water may enter joints and cracks in the pavement, penetrate at the edges of the surfacing, or percolate through the surfacing and shoulders.
4. Water may move vertically in capillaries or interconnected water films.
5. Moisture may move in vapor form, depending upon adequate temperature gradients and air void space.

The geographical location factors influencing heat transfer are generally related to the solar radiation intensity and therefore have an indirect effect on pavement temperatures. The information concerning the influence of the geographic location on frost action shows that air temperature can be expected to decrease with increasing latitude and elevation, and because moisture flow is governed not only by a moisture gradient but also by a temperature gradient, this observation is of importance for the climatic design. It is generally agreed that the soil temperature beneath pavements with northerly exposure are lower than those beneath pavements with southerly exposure. It is also established that nearness to bodies of water has a moderating effect on the air temperature and tends to reduce the heat gain from solar radiation. This is probably caused by the large specific heat of the mass of water and the relatively large amount of water vapor in the atmosphere. The topography has also an influence on the moisture content of the soil and is partly responsible for the drainability of the soil. This can be observed by examining two soil samples, one from a hill site and one from a depression in the terrain. Assuming the soils belonging to the same soil series, the sample taken in the depression will show a higher moisture content and slower internal drainage.

5.3 INTRINSIC FACTORS

The intrinsic factors required in climatic design are the thermal properties of the pavement materials which include thermal conductivity, heat capacity, latent heat of fusion, and the hydraulic properties.

The thermal conductivity, k, is the quantity of heat which flows normally across a surface of unit area per unit time under a unit thermal gradient. Experimental measurements of thermal conductivity can be accomplished by several methods, all of which are based on the observation of the temperature gradient across a given area of the material conducting heat at a known rate.

The heat capacity, c, is the amount of thermal energy necessary to cause a one-degree temperature change in a unit mass or unit volume of substance. Generally the heat capacity is computed from a heat balance between the heat gained by water in a calorimeter and the heat gained by the specimen.

The latent heat of fusion, L, is the change in the thermal energy in a unit volume of material when the moisture in that material freezes or thaws at a constant temperature. It depends upon the percentage of water which can be frozen in a given volume of material at any given temperature.

In recent years, the thermal properties of soils have been the subject of numerous laboratory and field studies. However, very little effort has been directed at developing thermal values for paving materials such as portland cement concrete and bituminous concrete.

In climatic design it is necessary to consider not only the thermal properties, but also the hydraulic properties. The existence of a drainage pipe, the moisture conductivity of the pavement materials, the subgrade, and envelope material together with the trench geometrics are all important factors to be evaluated thermally.

The moisture conductivity, K, is the quantity of water which flows normally across a surface of unit area per unit time under a unit hydraulic gradient. It can be found by laboratory experiment using measured values of applied moisture gradient and velocity of the water in a soil column provided the soil is saturated. If the soil is unsaturated, measurements of the moisture content have to be made also, because of the dependence of the hydraulic conductivity on the moisture content.

The most common and widely used envelope materials are naturally graded coarse sands and gravels, and the general procedure for designing a drainage envelope for a given soil is to make a mechanical analysis of both the soil and the proposed envelope material, compare the two particle size distribution curves, and decide by some criteria whether the envelope material is satisfactory. Based upon the chosen envelope material the trench geometrics are specified, if not already standardized as shown on Figure 5.5.

5.4 DRAINAGE AND HEAT TRANSFER MODELS

5.4.1 GENERAL

As indicated in the previous section, it is not sufficient to use a moisture model that predicts nonisothermal moisture conditions only, in order to analyze the feasibility of a subsurface drainage pipe installation when upgrading an existing pavement system. A model that predicts the water flow is also needed, and a simple two dimensional finite element computer model has therefore been developed to predict the hydraulic potentials under a pavement given certain boundary conditions (3). From Darcy's law:

$$q = -K \Delta H \tag{5.1}$$

(symbols used in equations are defined in Appendix B). The flux, q, to the collector pipe can be found assuming that the flow condition is saturated steady state. This condition prevails in the wet seasons of the year, and correlates with increased pavement damages during these periods. The assumption of saturated steady state condition in the model seems thus to be justified.

By varying the hydraulic conductivity of the different pavement, subgrade, and trench materials together with the trench geometrics and water table depth, and computing the flux to a subsurface drainage pipe, a comparison can be made of the influence each variable has on the overall drainability of the pavement. The optimum trench geometrics can be found which satisfy the hydraulic requirements for good subdrainage.

For the study of the frost action parameters a heat-transfer model developed by Dempsey (5) has been used and is described in more detail elsewhere.

By studying the influence climate has on frost depth penetration and frost action parameters, the selected trench geometrics and subdrainage design can be evaluated which consider the extrinsic factors related to temperature.

5.4.2 MOISTURE FLUX MODEL

The computer model used to calculate the hydraulic potentials for determining moisture flux uses a finite element program that solves for the hydraulic potential, H in Laplace's equation:

$$K_x \frac{\partial^2 H}{\partial x^2} + K_y \frac{\partial^2 H}{\partial y^2} = 0 \qquad (5.2)$$

for each given nodal point. The method used is a variational formulation of Equation (5.2) for steady state fully saturated water flow in a given region, R, and media, with the values of H specified on the boundary curve C (Figure 5.6). The variational formulation requires the minimization of the functional:

$$I[H] = \iint_R \{1/2[K_x(\partial H/\partial x)^2 + K_y(\partial H/\partial y)^2]\}dxdy \qquad (5.3)$$

with the same boundary conditions to be satisfied.

The region, R, is divided into elements as in Figure 5.6. Triangular elements were chosen in this study although other shapes may be used.

The linear approximating function in each element has the form:

$$H = Ax + By + C \qquad (5.4)$$

where H is the nodal values which are used in the minimization. This function is representing a plane in R and must necessarily pass through the nodal values in order to be a solution. The following three equations can thus be set up to find the three unknown constants A, B and C in Equation 5.4:

$$\begin{aligned} H &= Ax_i + By_i + C_i \\ H &\doteq Ax_j + By_j + C_j \\ H &= Ax_k + By_k + C_k \end{aligned} \qquad (5.5)$$

Solving these three linear equations for A, B and C using Cramer's Rule:

$$A = [H_i(y_j - y_k) + H_j(y_k - y_i) + H_k(y_i - y_j)]/2\Delta$$

$$B = [H_i(x_k - x_j) + H_j(x_i - x_k) + H_k(x_j - x_i)]/2\Delta \quad (5.6)$$

$$C = [H_i(x_j y_k - x_k y_j) + H_j(x_k y_i - x_i y_k) + H_k(x_i y_j - x_j y_i)]/2\Delta$$

which inserted in Equation 5.4 gives:

$$H = (N_i H_i + N_j H_j + N_k H_k) \quad (5.7)$$

where
$$N_i = [(x_j y_k - x_k y_j) + (y_j - y_k)x + (x_k - x_j)y]/2\Delta$$
$$N_j = [(x_k y_i - x_i y_k) + (y_k - y_i)x + (x_i - x_k)y]/2\Delta$$
$$N_k = [(x_i y_j - x_j y_i) + (y_i - y_j)x + (x_j - x_i)y]/2\Delta$$

$$2\Delta = \begin{vmatrix} 1 & x_i & y_i \\ 1 & x_j & y_j \\ 1 & x_k & y_k \end{vmatrix} = 2\times\text{area of triangle } i,j,k$$

Each element contributes to a portion of the total value of the integral (Equation 5.3) and the set of difference equations is obtained by the conditions for minimization:

$$\partial I/\partial H_m = 0,\ m = 1,2,3,\ldots\ldots,n-1,n \quad (5.8)$$

where n is the total number of nodes.

The nodal value m can contribute to the overall value of I[H] due to the circumstance that m can be a part of many elements so that Equation 5.8 may be rewritten as:

$$\sum_e \partial I_e/\partial H_m = 0, \; m = 1,2,3,\ldots\ldots,n-1,n \tag{5.9}$$

where I_e denotes the contribution of the e'th element to Equation 5.3. To obtain an expression for I_e for element i,j,k Equation 5.7 is substituted into Equation 5.3 to get:

$$\begin{aligned} \partial I_e = 1/2\iint_e[K_x(\partial N_i/\partial x\; H_i + \partial N_j/\partial x\; H_j + \partial N_k/\partial x\; H_k)^2 \\ + K_y(\partial N_i/\partial y\; H_i + \partial N_j/\partial y\; H_j + \partial N_k/\partial y\; H_k)^2]dxdy \end{aligned} \tag{5.10}$$

which by differentiating with respect to the value of H for node i gives:

$$\begin{aligned} \partial I_e/\partial H_i = \iint_e[K_x(\partial N_i/\partial x\; H_i + \partial N_j/\partial x\; H_j + \partial N_k/\partial x\; H_k)(\partial N_i/\partial x) \\ + K_y(\partial N_i/\partial y\; H_i + \partial N_j/\partial y\; H_j + \partial N_k/\partial y\; H_k)(\partial N_i/\partial y)]dxdy \end{aligned} \tag{5.11}$$

which can be rewritten, using the definitions of N_i, N_j, and N_k,:

$$\begin{aligned} \partial I_e/\partial H_i = \{[(y_j-y_k)H_i + (y_k-y_i)H_j + (y_i-y_j)H_k]((y_j-y_k)/4\Delta^2)K_x \\ + [(x_k-x_j)H_i + (x_i-x_k)H_j + (x_j-x_i)H_k]((x_k-x_j)/4\Delta^2)K_y\}\iint_e dxdy \end{aligned} \tag{5.12}$$

Finally $\iint dxdy = \Delta$, so:

$$\begin{aligned} \partial I_e/\partial H_i = \{[(y_i-y_k)H_i + (y_k-y_i)H_j + (y_i-y_j)H_k]((y_j-y_k)/4\Delta)K_x \\ + [(x_k-x_j)H_i + (x_i-x_k)H_j + (x_j-x_i)H_k]((x_k-x_j)/4\Delta)K_y \end{aligned} \tag{5.13}$$

In evaluation $\partial I_e/\partial H_j$ and $\partial I_e/\partial H_k$ the same method is used as above and rest of the elements are obtained in the stiffness matrix for the element i,j,k, $\bar{\bar{S}}^e_{ijk}$, which becomes

$$\bar{\bar{S}}^e_{ijk} = \begin{vmatrix} (y_j-y_k)^2K_x + (x_k-x_j)^2K_y & (y_k-y_i)(y_j-y_k)K_x + (x_i-x_k)(x_k-x_j)K_y & (y_i-y_j)(y_j-y_k)K_x + (x_j-x_i)(x_k-x_j)K_y \\ & (y_k-y_i)^2K_x + (x_i-x_k)^2K_y & (y_i-y_j)(y_k-y_i)K_x + (x_j-x_i)(x_i-x_k)K_y \\ \text{(symmetrical)} & & (y_i-y_j)^2K_x + (x_j-x_i)^2K_y \end{vmatrix} \tag{5.14}$$

The following matrix equation for element i,j,k has then been developed:

$$\{\partial I_e/\partial H^e\} = \begin{Bmatrix} \partial I_e/\partial H_i \\ \partial I_e/\partial H_j \\ \partial I_e/\partial H_k \end{Bmatrix} = \bar{\bar{S}}^e_{ijk} \begin{Bmatrix} H_i \\ H_j \\ H_k \end{Bmatrix} = \bar{\bar{S}}^e_{ijk}\overline{H}_e \tag{5.15}$$

To assemble a typical equation for the differential of I given by Equation 5.3 with respect to any nodal value for the whole region, the contribution of elements adjacent to the node is nonzero and the assembly will be clearly according to the following:

$$\overline{I_e/\ H^e} = \bar{\bar{S}}_{total}\overline{H} = \overline{0} \tag{5.16}$$

where $\bar{\bar{S}}_{total}$ is the total stiffness matrix of the system, $\bar{H}$ a column vector containing nodal values of H and $\bar{0}$ a zero column vector.

The total stiffness matrix of the system, $\bar{\bar{S}}_{total}$, is obtained from the individual stiffness matrices of the elements, $\bar{\bar{S}}^{e}_{ijk}$, simply by adding the values of M_{xx} from $\bar{\bar{S}}^{e}_{ijk}$ to the elements S_{xx} in $\bar{\bar{S}}_{total}$.

Due to the boundary conditions for certain nodes placed on the boundary curve C of the region R (Figure 5.6) Equation 5.16 has to be modified because in such cases the differential of I (Equation 5.3) with respect to these nodes is meaningless since there is no variation.

The total stiffness matrix, $\bar{\bar{S}}_{total}$, and the right hand side of the equation system, $\bar{0}$, which is equal to a zero column vector before the modification, is then reformed by simply eliminating equations associated with the boundary condition nodes.

In solving the equation system given by Equation 5.16 the Gauss' Elimination Method is used to obtain the nodal values of H.

5.4.3 HEAT TRANSFER FLUX MODEL

The development, evaluation, and utilization of the heat-transfer model is well documented by Dempsey (4,5) and will not be discussed further in this report.

5.5 DATA INPUT FOR MODELS

In Table 5.1 typical saturated hydraulic conductivity values for different subgrade, base, and filter materials are given. The conductivity values were used as input parameters for the hydraulic model and have been chosen to represent the most common pavement materials used in construction. The coarse gravels with permeability coefficients of

5,000 and 6,000 cm/hr have been used to compare the drainability of the system with the solution obtained using Cedergren's method described in reference (6). In Appendix A the calculations using Cedergren's method can be seen.

The geometrics of the pavement cross sections used are shown in Figures 5.7 and 5.8 (one without and one with a subsurface drainage pipe respectively).

In summary the following variables were used:

1. The water table depth between 1.2 m (4 ft.) and 3.6 m (12 ft.).
2. The depth, d, of the trench 45.7 cm (18 in.) or 66.0 cm (26 in.) and the width fixed at 25.4 cm (10 in.).

In the heat-transfer model typical thermal and physical properties for the shoulder sections given in Figures 5.9 and 5.10 were used together with the climatic and radiation data for Cairo, Springfield, and Chicago in Illinois and Minneapolis-St. Paul in Minnesota. In all cases the winter months from October through March were studied over a period of 10 years. For Cairo, Springfield and Chicago the years were 1937-47 and for Minneapolis-St. Paul 1966-76.

For the Chicago location the influence of different pavement materials and freezing temperatures on the frost depth penetration were also studied using the two shoulder sections shown in Figures 5.9 and 5.10. The freezing temperatures were -1.1 C (30 F) and 0 C (32 F) and were used for the Chicago location with the pavement shown in Figure 5.9. In all other locations 0 C (32 F) was used as the freezing temperature.

In the computation of the frozen and unfrozen hydraulic conductivities, equations developed by Kersten (7) were used:

1. For fine-grained soils:

Unfrozen: $$K_u = \frac{(0.9 \log w - 0.2)10^{0.01\gamma_d}}{12} \quad (5.17)$$

Frozen: $$K_i = \frac{0.01(10)^{0.022\gamma_d} + 0.085(10)^{0.008\gamma_d}(w)}{12} \quad (5.18)$$

2. For coarse-grained soils:

Unfrozen: $$K_u = \frac{(0.7 \log w + 0.4)10^{0.01\gamma_d}}{12} \quad (5.19)$$

Frozen: $$K_i = \frac{0.076(10)^{0.013\gamma_d} + 0.032(10)^{0.0146\gamma_d}(w)}{12} \quad (5.20)$$

where γ_d denotes dry density in pcf and w water content in percent.

When computing the frozen and unfrozen heat capacities of the soils, two other equations developed by Kersten (7) were used:

$$c_u = \frac{100\ c_m + 1.0\ w}{100 + w} \quad (5.21)$$

$$c_i = \frac{100\ c_m + 0.5\ w}{100 + w} \quad (5.22)$$

with a value of 837 J/kg-K (0.20 BTU/lb-F) for c_m, as suggested by Johnson (8) and Jumikis (9).

An average value of the frozen and unfrozen hydraulic conductivities was used for the freezing soil according to Dempsey (5):

$$K_f = \frac{K_u + K_i}{2} \tag{5.23}$$

and for the freezing heat capacity (Dempsey (5)):

$$c_f = \frac{144w\gamma_d}{200\gamma} \tag{5.24}$$

where γ is the total unit weight.

5.6 RESULTS

The calculated amounts of water (flux) that flow to a subdrainage pipe as a function of the water table depth for different filter materials, sizes of pipe section, and subgrade soils (AASHTO classification A-1, A-3, A-4, and A-7) are shown in Table 5.2. The data indicate that there is a strong tendency for the flux values to be independent of the water table depth for the A-4 and A-7 subgrade soils. There is only a slight decrease in the flux for the A-3 soil with an increase in water table depth. It can be seen that there is a difference in amount of water carried to the pipe when using AASHTO A-1 and A-3 materials as filter material. For constant water table depth, the flux values for the A-1 filter material are the highest. A similar less pronounced trend is found for the different pipe section geometrics in which the 45.7 cm (18 in.) pipe section shows the highest flux value.

In Table 5.3 the flux for different base course hydraulic conductivities are shown for the two pipe sections. As indicated by the flux values, the 45.7 cm (18 in.) pipe section causes the flux to increase compared with the 66.0 cm (26 in.) section. Also there is a tendency for the flux value to become constant for a conductivity value of the base of more than 500 cm/hr.

In Tables 5.4 and 5.5 the flux to the water table is given for the different trench sections analyzed. For comparison a section without a trench and a pipe has been included to illustrate the effect installment of a trench and pipe has on the drainability of a given pavement section. Generally lower flux values are observed for a 45.7 cm (18 in.) pipe section as compared with a 66.0 (26 in.) section when the subgrade material is coarse grained (A-1 and A-3). A more constant value, however, is obtained with the finer subgrade materials (A-4 and A-7). The effect of changing the filter material from an A-3 to an A-1 material is also shown in Table 5.4. The flux to the water table increases when the A-1 material is used as filter material. From Table 5.5 it is seen that no change in the flux takes place with a change in hydraulic conductivity of the base material.

In Tables 5.6 and 5.7 the heat-transfer model results for the frost line depth for different locations in Illinois and Minnesota and for different base materials and freezing temperatures in Chicago, Illinois respectively are shown. As expected the frost line depth varies with the climate of the location. A maximum average frost line depth of 111.5 cm (43.9 in.) is observed in March in Minneapolis-St. Paul, Minnesota and a minimum value of 5.1 cm (2 in.) in Cairo, Illinois in January. A change in the frost line depth with change in pavement material and freezing temperature can be seen in Table 5.7. The highest frost line depth is for the pavement section including 17.8 cm (7 in.) of stabilized base (Chicago 2). A pronounced drop in frost line depth is however seen when -1.1 C (30 F) is used as freezing temperature.

In Tables 5.8 through 5.10 the statistics, based on 10 observations, for the mean values of the different parts of a freeze-thaw cycle are presented. The analysis is for nodes 10, 12 and 14, which are located 45.7, 55.9, and 66.0 cm (18, 22, and 26 in.) beneath the surface of the pavement.

The warming and cooling rates for each month for the different locations seem to be fairly constant within stations in Illinois whereas the data for Minneapolis-St. Paul tend to be higher in January and February and lower in November, December, and March.

The below freezing temperatures are fluctuating between -1.1-0 C (31-32 F) for the Illinois stations whereas the same temperatures are around -2.2 C (28 F) for Minneapolis-St. Paul. As for the above freezing temperatures, the data clearly reveals the basic north-south trends in temperature with the lowest temperatures being in Minneapolis-St. Paul and the highest in Cairo. Also the duration of freezing shows this trend in a similar manner, the duration of freezing ranging from a total of 89 days in Minneapolis-St. Paul to 0 days in Cairo.

For the number of freeze-thaw cycles the value did not exceed 3 which happened one time in 10 years at a depth of 45.7 cm (18 in.) in Chicago (pavement 1). As indicated in Table 5.8, an overall conservative average value of 2 is to be expected at 45.7 cm (18 in.) depth. At 55.9 and 66.0 cm (22 and 26 in.) depth the value is 1.

5.7 ANALYSIS AND DISCUSSION

Table 5.2 indicates that the flux to the drainage pipe in all cases is negative for the road section shown in Figure 5.8 with A-1 soil as the subgrade type. This does not mean that no water in reality flows to the pipe, but it is simply a result of a simplification of the boundary conditions around the pipe.

The two boundary conditions in question are:

1. $H = z$ $(h = 0)$
2. $\partial H/\partial x = \partial H/\partial y = 0$, except at slots.

Where condition 2 specifies that no flow occurs across the pipe boundary. In the modeling only boundary condition 1 has been used because of the difficulties in checking, that condition 2 is fulfilled after calculation of H, and because comparative flux values rather than precise values are of interest.

If the results for the A-1 subgrade soil are compared with the other subgrade types for the case where the base type is held constant (type A-3), installment of a pipe, if the subgrade is an A-1 soil, does not improve the drainability of the pavement section because the A-1 subgrade soil in itself provides sufficient conductivity to move all the intruded water to the water table within a reasonable time period.

In Figures 5.11 and 5.12 the effect of using a granular A-1 material as a filter instead of the usual chosen A-3 sand is shown. This improves the drainage from 5 to 8 times with respect to the flux carried to the pipe, however the flow to the water table does not change more than 2 to 3 percent (Table 5.4). The improvement is caused by increase of the conductivity around the pipe.

Moving the pipe closer to the source does have an effect on the flux to the pipe which increases 1.5 to 2 times the original value for the 66.0 cm (26 in.) standard pipe section. This can be observed from Figures 5.13 and 5.14 where the flux to the pipe is shown for a 66.0 and 45.7 cm (26 and 18 in.) pipe section. Also the difference in variation of the flux as a function of the water table depth can be seen in these figures as well as in Figures 5.11 and 5.12, for the sections with A-3, A-4, or A-7 subgrade soils. A slight decrease is observed in the flux with an increase in water table depth for the A-3 soil whereas for the A-4 and A-7 soils there is hardly any variation. This is caused by the higher permeability of the A-3 soil as compared to the A-4 and A-7 soils. The A-4 and the A-7 soils especially have such low permeabilities compared to the base material that most of the water flow is carried to the pie and only a fraction is carried to the water table. This is in agreement with Table 5.4 where the flux values to the water table are shown.

Figure 5.15 shows that changing the conductivity of the base material beyond a K-value of 100 cm/hr does not change the flux to the pipe. The same is true for the amount of water that flows to the water table as seen in Table 5.5. However, the flux change is about 35 percent when the K-value is changed from 2 cm/hr to 100 cm/hr. The two points for K = 5,000 cm/hr and K = 6,000 cm/hr have been chosen to show what changes in the flow to the pipe will occur if a course base material recommended by edergren (6) is used (for the specific calculations see Appendix A). As shown in Figure 5.15, a 66.0 cm (26 in.) pipe trench section with a coarse gravel as base material will give the same flux to the pie as if a standard concrete sand was used as base material and the pipe moved closer to the source. Thus, if the base material cannot be removed as is the case for many rehabilitation projects of concrete pavements, an alternative solution would be to move the pipe closer to the source as possible without causing any frost or load problems.

In this analysis the trench material was held constant. The observed assymptotic increase in the amount of water that flows to the pipe (Figure 5.15) is caused by the influence and control that the slowest drainable material always has over the more drainable. Therefore, if the trench material is the same as the base material, an even better improvement in flux than shown in Figure 5.15 should be expected.

Concerning the frost depth data for the 4 stations, the results using the heat-transfer model show that there is a variation in frost line depth between stations as well as for different pavement sections. In Illinois this variation is from 5.1 cm (2 in.) in Cairo to 33.3 to 41.1 cm (13.1 to 16.2 in.) in Chicago dependent on the pavement material used. The station in Menneapolis-St. Paul, Minnesota represents an extreme with a frost line depth of 111.5 cm (43.9 in.). A careful study of the data for Minneapolis-St. Paul shown in Tables 5.8 through 5.10 indicates that it does not make a difference whether the drainage pipe is placed in a 66.0 or 45.7 cm (26 or 18 in.) trench as far as freezing is concerned. The pipe trench section will remain in frozen state throughout the winter. Placing the pipe at a depth of 111.8 cm (44 in.) or more is unrealistic and will be expensive. Because of the relative low number of freeze-thaw cycles, however, and the fact that the thawing process is anticipated to be faster from the surface rather than from the soil beneath the frozen zone, it is recommended that the pipe be placed at a depth of about 45.7 cm (18 in.) or as close to the surface as possible. The pipe can then start working as soon as possible in the beginning of the thawing season.

A similar conclusion applies to the Cairo station in Illinois. The pipe can easily, and should, be placed as close to the source as possible. With a frost line depth not exceeding 45.7 cm (18 in.) (no below freezing temperatures encountered at this depth), an appropriate depth would be 45.7 cm (18 in.). This is on the safe side. To see if the pipe could be placed even closer to the source an analysis of the number of freeze-thaw cycles for nodes 6, 7 and 8, which are at a depth of 25.4, 30.5, and 35.6 cm (10, 12, and 14 in.) respectively, was completed. It was found that for a period of 10 years the yearly mean number of freeze-thaw cycles would be 0.20 with a standard deviation equal to 0.42 for node 8 at the 35.6 cm (14 in.) depth. This is comparable with the number of freeze-thaw cycles for the Chicago station at the 66.0 cm (26 in.) depth. A pipe depth of 35.6 cm (14 in.) is therefore recommended for the Cairo station.

For the Springfield location a pipe depth of 55.9 cm (22 in.) is recommended because of the similarities in the frost data for the depths of 66.0 and 55.9 cm (26 and 22 in.), whereas for the Chicago station the existing standard section of 76.2 cm (30 in.) is reasonable.

In environments with none or hardly any frost at all (frost depth less than surface coarse) it is recommended that the pipe be placed at least 2.5 to 5.1 cm (1 to 2 in.) below the bottom of the base-subgrade interface. This will allow the water to flow to the pipe from all directions and thus make the pipe work as efficiently as possible. The use of drains with holes or slots located all around the pipe are recommended because the simultaneous gravimetric flow from the source at the edges of the pavement and the horizontal drainage produces a more or less symmetric flow path around the pipe.

For the two different pavement sections in Chicago, Illinois (Table 5.7) it was found that the frost line depth is influenced to a considerable degree by the pavement material used. The lower value for the pavement section composed of 20.3 cm (8 in.) asphalt concrete over 45.7 cm (18 in.) of A-3 filter material and A-6 subgrade soil clearly shows the combination of the insulating effect of the asphalt concrete and the influence of the water content which has a relative high value for the latent heat of fusion. An even more pronounced difference is to be found between the frost line depth based on a -1.1 and 0 C (30 and 32 F) freezing temperature. In Table 5.7 it is shown that with -1.1 C (30 F) a mean frost line depth of 9.6 cm (3.8 in.) is obtained, and with 0 C (32 F) a value of 33.3 cm (13.1 in.) is obtained. When choosing the freezing temperature it is important to make sure that the temperature used in the model represents the actual freezing temperature of the soil in question.

In designing a pipe trench section the engineer has to consider and take the following into account:

1. Hydraulic properties of the materials.
2. Expected number of freeze-thaw cycles.
3. Pavement materials used.
4. Frost line depth.

As often happens, the engineer is tempted to specify a highly drainable material (AASHTO A-1 for example) without realizing that such a material with its relatively low water content as compared with the base and subbase material will tend to increase the frost line depth in the subdrainage trench. This is not a negative factor if it is taken into account by a corresponding lowering of the pipe. If, however, the anticipated number of freeze-thaw cycles is low and the frost depth large, such a lowering of the pipe might be unnecessary as was the case for the pavement in Minneapolis-St. Paul. Another factor influencing the design is that the pipe should be put below the top of the subgrade if possible for maximum efficiency. If a deeper subdrainage pipe is needed, in a frost environment then a thicker base or subbase may be helpful in promoting water flow to the subdrain during thaw periods. This, of course, assumes that the subgrade has a low permeability.

5.8 SUMMARY AND CONCLUSIONS

5.8.1 SUMMARY

An investigation was conducted to show how various factors such as subgrade and base material type, drainage envelope material, trench geometry, water table depth, and climate influence subsurface drainage. Hydraulic properties together with climatic data were the input information required for the two computer programs used. A hydraulic model was used to predict the hydraulic potentials and water flux values, and the heat-transfer model was used to determine the frost line depth and freeze-thaw cycles.

By a combination of the results obtained from the programs using a typical highway cross section as a model, a qualitative judgment could be made of different kinds of improvement for a highway subsurface drainage system in rehabilitation projects where water is shown to cause the major distress. Correspondingly the results apply to the design considerations of the collector system in a new pavement as well.

5.8.2 CONCLUSIONS

From the results of this investigation, the following conclusions were made:

1. Installment of a pipe when the subgrade is AASHTO classification A-1 does not improve the drainability of the pavement system.
2. Using a coarse filter material such as coarse gravel improves the drainage from 5 to 8 times with respect to the flux carried to the pipe when compared to a fine grained filter material.
3. Moving the pipe closer to the source improves the drainage 1.5 to 2 times with respect to the flux carried to the pipe.
4. About 2.5 to 5.1 cm (1 to 2 in.) should be the minimum vertical distance from the top of an impermeable subgrade to the bottom of the subdrainage pipe in environments with little or no frost (frost depth less than thickness of surface course).
5. For the locations with extreme winter climate, i.e. either very warm or very cold like in Cairo, Illinois and Minneapolis-St. Paul, Minnesota, the drainage pipe should be placed closer to the source than current specifications prescribe.
6. The 76.2 m (30 in.) standard drainage section used in Illinois is adequate for the Chicago location but could be shortened to 55.9 cm (22 in.) in Springfield.
7. The exact location of the drainage pipe depends on the type of pavement materials used. For the pavement sections chosen in this study (see Figures 5.7 and 5.8) trench sections 35.6, 55.9, and 76.2 cm (14, 22, and 30 in.), are realistic for Illinois, depending on the climate.

8. Changing the base material by making it a coarser material with a K-value greater than 100 cm/hr without also changing the trench filter material does not change the flux to the pipe more than a few percent whereas a 35 percent change can be expected if the conductivity of the trench filter material is also changed from 2 cm/hr to 100 cm/hr.
9. The water table depth does not influence the flux to the pipe sufficiently to be taken into account in fine-grained subgrade soils.
10. Drains should have slots or holes located throughout the circumference.

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Table 5.1. Typical Saturated Hydraulic Conductivities Used in Drainage Model.

Subgrade (S)/ Base (B)/Filter (F) Material	K-Value (cm/hr)
S A-1	82.0
S A-3	10.3
S A-4	0.22
S A-7	0.025
B CA 6 Concrete Sand	2.0
B Coarse Sand	25
B Coarse Sand	50
B Coarse Sand	100
B Gravel	5,000
B Gravel	6,000
F A-3	10.3
F A-1	82.0

Table 5.2. Flux to Subdrain Pipe as Function of Water Table Depth, Filter Material, Pipe Section Size, and Subgrade Soil.

Subgrade Soil Type	Water Table Depth (ft)	FLUX TO PIPE, q, (cm/hr)		
		d = 26 in. A-3 Filter	d = 18 in. A-3 Filter	d = 26 in. A-1 Filter
A-1	4	* - 813.99	- 842.73	- 981.50
	5	-1041.86		
	6	-1241.39	-1259.74	-1604.55
	9	-1724.98	-1214.62	-2296.73
	12	-2087.64	-1484.97	-2798.50
A-3	4	43.98	113.28	799.25
	6	25.59	91.82	779.61
	9	8.60	75.39	759.05
	10	4.58	71.56	
	11	- 1.23	68.26	
	12	- 1.96	65.40	745.58
A-4	4	222.56	332.21	1036.30
	6	223.44	332.94	1037.18
	9	224.05	333.40	1037.81
	12	224.39	333.76	1038.16
A-7	4	223.80	329.79	1037.84
	5	223.86		1037.89
	6	223.89	329.88	1037.94
	7	223.93	329.92	1037.97
	8	223.95	329.93	1038.00
	9	223.97	329.95	1038.01
	12	224.01	329.99	1038.06

* Negative values indicate flow is to the water table.

d = trench depth, Figure 5.8

Table 5.3. Flux as a Function of Base Course Hydraulic Conductivity.

K-Value of base (cm/hr)	FLUX TO PIPE, q, (cm/hr) d = 26 in. A-3 Filter	d= 18 in.depth A-3 Filter
2.0	223.80	329.79
25.0	272.84	406.81
50.0	284.93	421.30
100.0	294.48	430.36
5000.0	317.79	440.87
6000.0	317.99	440.91

For subgrade A-7 soil, water table depth WT = 4 ft.

d = trench depth, Figure 5.8

Table 5.4. Flux to Water Table for Various Soil Types and Drainage Sections.

Subgrade Soil Type	Water Table Depth (ft)	FLUX TO WATER TABLE, q, (cm/hr)			
		d= 26 in. A-3 Filter	d = 18 in. A-3 Filter	without pipe	d = 26 in. A-1 Filter
A-1	4	214.78	200.81	149.95	288.24
	5	250.30		174.62	
	6	281.46	262.16	197.31	360.02
	9	355.69	332.31	255.58	436.30
	12	409.93	384.87	302.48	489.05
A-3	4	76.26	75.21	74.59	78.13
	6	80.83	79.91	77.67	82.24
	9	84.39	83.61	81.81	85.45
	10	85.15	84.41	82.73	
	11	85.79	85.09	83.50	
	12	86.33	85.66	84.16	87.19
A-4	4	2.47	2.52	2.60	2.51
	6	2.31	2.32	2.38	2.31
	9	2.19	2.20	2.24	2.19
	12	2.14	2.14	2.17	2.14
A-7	4	0.289	0.290	0.300	0.289
	5	0.274		0.283	0.274
	6	0.265	0.266	0.272	0.265
	7	0.258	0.259	0.265	0.258
	8	0.254	0.255	0.260	0.254
	9	0.250	0.251	0.256	0.250
	12	0.244	0.244	0.248	0.243

d = Trench depth, Figure 5.8

Table 5.5. Flux to Water Table as a Function of Base Course Hydraulic Conductivity.

K-value of base (cm/hr)	FLUX TO WATER TABLE, q, (cm/hr)		
	Without pipe	d= 26 in.	d= 18 in.
2.0	0.300	0.290	0.290
25.0	0.300	0.290	0.290
50.0	0.300	0.290	0.290
100.0	0.300	0.290	0.290
5000	0.300	0.290	0.290
6000	0.300	0.290	0.290

For subgrade A-7 soil, water table depth WT = 4 ft.

d = Trench depth, Figure 5.8

Table 5.6. Maximum Frost Line Depth (in.) Using Freezing Temperature = 32F.

Station	October			November			December			January			February			March			Year		
	Avg	σ[1]	V,%[2]	Avg	σ	V,%	Avg	σ	V,%	Avg	σ	V,%	Avg	σ	V,%	Avg	σ	V,%	Avg	σ	V,%
Cairo	0.00	0.00	0.00	0.08	0.33	426.16	0.66	1.33	246.01	1.96	2.22	201.77	0.78	1.70	288.97	0.10	0.34	383.48	0.60	1.70	312.90
Springfield	0.00	0.01	78.74	0.27	0.75	383.79	3.84	3.73	143.31	6.99	4.66	100.17	5.01	4.14	140.30	0.45	0.99	324.22	2.74	4.72	188.77
Chicago 1	0.01	0.04	43.99	0.45	1.06	298.86	6.48	4.88	111.71	13.14	5.33	53.40	13.09	4.76	65.03	2.36	3.96	223.82	5.83	7.58	138.92
Chicago 2	0.00	0.02	49.86	0.35	0.86	322.35	7.51	5.25	110.55	16.19	5.37	44.71	15.62	5.12	62.78	3.21	4.94	229.83	7.05	8.95	136.78
Minneapolis-St. Paul	0.06	0.23	457.07	2.19	3.07	144.17	16.07	5.81	37.57	33.58	4.75	14.25	43.35	1.96	4.58	43.86	3.85	10.36	22.97	18.77	81.75

1) Standard deviation

2) Coefficient of variation = $\frac{(\sigma)\ (100)}{\text{Avg}}$

Table 5.7. Maximum Frost Line Depth (in.) for Chicago.

Station	October			November			December			January			February			March			Year		
	Avg	σ[1]	V,%[2]	Avg	σ	V,%	Avg	σ	V,%	Avg	σ	V,%	Avg	σ	V,%	Avg	σ	V,%	Avg	σ	V,%
Chicago 1	0.01	0.04	43.99	0.45	1.06	298.86	6.48	4.88	111.71	13.14	5.33	53.40	13.09	4.76	65.03	2.36	3.96	223.82	5.38	7.58	138.92
Chicago 2	0.00	0.02	49.86	0.35	0.86	322.35	7.51	5.25	110.55	16.19	5.37	44.71	15.62	5.12	62.78	3.21	4.94	229.83	7.05	8.95	136.78
Chicago 3 (FT=30°F)	0.00	0.01	316.23	0.22	0.17	76.80	2.40	1.72	71.66	3.81	2.89	75.95	2.06	1.15	56.00	0.51	0.43	84.07	1.50	0.65	43.12

1) Standard deviation

2) Coefficient of variation = $\frac{(\sigma)\ (100)}{\text{Avg}}$

Table 5.8. Frost Action Parameter Data at 45.7 cm (18 in.) Depth.

Station	October			November			December			January			February			March			Year		
	Avg.	σ 1)	V,% 2)	Avg.	σ	V,%	Avg.	σ	V,%	Avg.	σ	V,%	Avg.	σ	V,%	Avg.	σ	V,%	Avg.	σ	V,%
Cooling Rates, °F/hr																					
Cairo	0.10	0.05	50.68	0.10	0.05	55.94	0.07	0.04	64.08	0.06	0.04	70.86	0.08	0.05	60.78	0.09	0.05	54.32	0.08	0.05	62.05
Springfield	0.10	0.05	54.56	0.08	0.05	60.73	0.05	0.04	81.38	0.04	0.03	90.71	0.05	0.04	79.42	0.08	0.05	60.23	0.07	0.05	77.32
Chicago 1	0.09	0.05	52.23	0.08	0.05	63.39	0.03	0.03	95.95	0.02	0.02	117.26	0.02	0.03	104.00	0.07	0.04	69.08	0.05	0.05	93.90
Chicago 2	0.11	0.06	53.07	0.07	0.06	66.35	0.04	0.03	90.33	0.02	0.02	116.08	0.03	0.03	106.41	0.08	0.06	71.97	0.06	0.06	96.41
Minn. - St. Paul	0.09	0.05	56.02	0.06	0.04	67.54	0.01	0.01	110.32	0.08	0.06	93.17	0.10	0.08	80.04	0.04	0.04	117.52	0.06	0.06	105.05
Below Freezing Temperatures, °F																					
Cairo	None	--	--	None	--	--	None	--	--	None	--	--	None	--	--	None	--	--	None	--	--
Springfield	None	--	--	None	--	--	None	--	--	31.58	0.44	1.39	31.13	0.46	1.47	None	--	--	31.54	0.37	1.17
Chicago 1	None	--	--	None	--	--	31.73	0.14	0.44	31.45	0.25	0.81	31.45	0.24	0.75	31.80	0.06	0.20	31.55	0.24	0.77
Chicago 2	None	--	--	None	--	--	31.39	0.28	0.89	31.15	0.52	1.70	31.15	0.39	1.28	31.61	0.18	0.57	31.19	0.50	1.64
Minn. - St. Paul	None	--	--	None	--	--	31.17	0.54	1.75	26.54	3.20	12.41	25.80	3.31	13.09	30.16	1.94	3.05	27.90	9.89	12.75
Duration of Freezing, Days																					
Cairo	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Springfield	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	3.23	0.00	0.00	0.80	1.24	15.50	0.00	0.00	0.00	2.54	2.54	16.38
Chicago 1	0.00	0.00	0.00	0.00	0.00	0.00	2.21	0.00	0.00	8.33	0.51	2.58	8.33	1.09	15.35	1.23	0.00	0.00	16.93	1.79	12.71
Chicago 2	0.00	0.00	0.00	0.00	0.00	0.00	4.10	0.00	0.00	13.12	1.83	17.11	12.53	0.00	0.00	1.89	0.00	0.00	27.26	4.35	30.60
Minn. - St. Paul	0.00	0.00	0.00	0.00	0.00	0.00	12.09	0.00	0.00	31.00	0.00	0.00	28.00	0.00	0.00	25.63	1.99	19.84	89.40	12.33	27.22
Warming Rates, °F/hr																					
Cairo	0.11	0.07	64.11	0.06	0.04	67.45	0.05	0.04	74.60	0.07	0.04	68.05	0.07	0.05	71.37	0.10	0.06	58.99	0.08	0.06	72.33
Springfield	0.10	0.07	70.11	0.07	0.04	68.83	0.04	0.04	89.57	0.04	0.04	106.04	0.06	0.05	95.26	0.10	0.06	63.19	0.07	0.06	85.92
Chicago 1	0.09	0.07	70.32	0.06	0.04	71.37	0.03	0.03	98.14	0.02	0.02	117.04	0.03	0.03	115.09	0.09	0.06	75.13	0.06	0.06	109.00
Chicago 2	0.11	0.08	70.12	0.08	0.05	70.86	0.04	0.03	107.78	0.02	0.03	170.20	0.03	0.04	143.80	0.10	0.08	82.23	0.06	0.07	117.16
Minn. - St. Paul	0.08	0.06	81.87	0.05	0.04	87.12	0.02	0.02	120.90	0.05	0.05	98.94	0.06	0.06	114.83	0.02	0.03	169.01	0.05	0.06	123.47
Above Freezing Temperatures, °F																					
Cairo	61.19	4.40	7.17	55.45	4.76	8.60	45.35	3.45	7.66	41.97	3.05	7.29	44.64	3.14	7.03	53.08	4.84	9.12	50.35	8.41	16.72
Springfield	59.11	4.22	7.12	50.99	4.97	9.76	39.89	3.55	8.90	36.19	2.53	6.86	38.00	2.64	6.87	46.94	5.04	10.66	45.72	9.33	20.41
Chicago 1	56.36	3.70	6.56	47.98	4.64	9.65	37.62	3.03	7.96	34.07	1.70	4.92	34.74	1.85	5.26	42.02	4.86	11.35	43.52	8.98	20.66
Chicago 2	56.95	4.15	7.28	47.32	5.10	10.76	37.24	3.01	7.95	33.98	1.75	5.06	34.54	1.57	4.46	42.37	5.37	12.44	44.22	9.20	20.87
Minn. - St. Paul	52.05	3.31	6.38	41.52	4.37	10.43	33.41	1.12	3.33	None	--	--	None	--	--	36.52	2.74	7.35	43.50	8.14	18.68
Duration of Thawing, Days																					
Cairo	30.38	0.00	0.00	30.00	0.00	0.00	31.00	0.00	0.00	31.00	0.00	0.00	28.00	0.00	0.00	31.00	0.00	0.00	181.38	0.00	0.00
Springfield	30.38	0.00	0.00	30.00	0.00	0.00	31.00	0.00	0.00	26.71	0.10	0.99	25.83	0.15	11.14	31.00	0.00	0.00	157.05	6.87	16.45
Chicago 1	30.38	0.00	0.00	30.00	0.00	0.00	27.62	1.24	10.60	19.69	0.31	2.35	16.87	0.48	8.35	29.77	0.00	0.00	98.03	28.19	50.15
Chicago 2	30.38	0.00	0.00	30.00	0.00	0.00	26.90	0.00	0.00	12.08	1.06	9.47	14.16	1.36	10.42	29.11	0.00	0.00	82.11	30.80	56.07
Minn. - St. Paul	30.38	0.00	0.00	30.00	0.00	0.00	18.91	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	2.27	1.01	13.50	62.74	18.30	52.08
Number of Freeze Thaw Cycles																					
Cairo	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Springfield	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.20	0.42	210.82	0.20	0.63	316.23	0.00	0.00	0.00	0.40	0.97	241.52
Chicago 1	0.00	0.00	0.00	0.00	0.00	0.00	0.30	0.48	161.02	0.50	0.53	105.41	0.30	0.67	224.98	0.00	0.00	0.00	1.10	0.99	90.40
Chicago 2	0.00	0.00	0.00	0.00	0.00	0.00	0.40	0.52	129.10	0.60	0.52	86.07	0.20	0.42	210.82	0.00	0.00	0.00	1.20	0.79	65.73
Minn. - St. Paul	0.00	0.00	0.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.20	0.42	310.82	1.20	0.42	35.14

Table 5.9. Frost Action Parameter Data at 55.9 cm (22 in.) Depth.

Station	October			November			December			January			February			March			Year		
	Avg.	σ[1]	V,%[2]	Avg.	σ	V,%	Avg.	σ	V,%	Avg.	σ	V,%	Avg.	σ	V,%	Avg.	σ	V,%	Avg.	σ	V,%
Cooling Rates, °F/hr																					
Cairo	0.06	0.03	58.34	0.06	0.04	62.02	0.05	0.03	66.95	0.04	0.03	69.93	0.05	0.03	65.90	0.05	0.03	66.11	0.05	0.03	66.94
Springfield	0.06	0.04	63.18	0.06	0.03	60.42	0.03	0.03	78.37	0.02	0.02	83.42	0.03	0.02	68.28	0.05	0.03	68.53	0.04	0.03	79.12
Chicago 1	0.06	0.03	60.23	0.05	0.03	63.23	0.03	0.02	88.17	0.01	0.01	109.55	0.01	0.01	108.21	0.04	0.03	74.86	0.03	0.03	96.24
Chicago 2	0.08	0.05	56.79	0.07	0.04	63.46	0.03	0.02	88.54	0.01	0.02	118.26	0.02	0.02	116.83	0.05	0.04	76.84	0.04	0.04	100.04
Minn. - St. Paul	0.05	0.03	61.41	0.04	0.03	64.29	0.01	0.01	95.28	0.05	0.05	109.51	0.08	0.07	80.04	0.03	0.03	112.46	0.04	0.05	113.71
Below Freezing Temperatures, °F																					
Cairo	None	--	--	None	--	--	None	--	--	None	--	--	None	--	--	None	--	--	None	--	--
Springfield	None	--	--	None	--	--	None	--	--	31.47	0.34	1.08	31.35	0.25	0.79	None	--	--	31.38	0.28	0.89
Chicago 1	None	--	--	None	--	--	None	--	--	31.47	0.26	0.83	31.50	0.14	0.46	31.77	0.06	0.18	31.58	0.19	0.56
Chicago 2	None	--	--	None	--	--	31.79	0.11	0.34	31.17	0.44	1.43	31.26	0.28	0.92	31.42	0.17	0.54	31.35	0.34	1.11
Minn. - St. Paul	None	--	--	None	--	--	31.58	0.23	0.73	27.85	2.76	10.18	26.52	2.77	10.66	30.12	0.73	2.46	28.36	2.93	10.45
Duration of Freezing, Days																					
Cairo	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Springfield	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.87	0.00	0.00	2.43	0.00	0.00	0.00	0.00	0.00	3.30	0.00	0.00
Chicago 1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	3.52	0.00	0.00	6.39	0.00	0.00	0.54	0.00	0.00	10.44	0.00	0.00
Chicago 2	0.00	0.00	0.00	0.00	0.00	0.00	2.03	0.00	0.00	9.91	0.00	0.00	11.45	0.00	0.00	1.16	0.00	0.00	22.50	1.71	8.32
Minn. - St. Paul	0.00	0.00	0 00	0.00	0.00	0.00	5.81	0.00	0.00	29.98	0.00	0.00	28.00	0.00	0.00	28.59	0.00	0.00	92.38	0.00	0.00
Warming Rates, °F/hr																					
Cairo	0.07	0.05	75.44	0.04	0.03	69.86	0.04	0.03	75.61	0.04	0.03	70.46	0.05	0.04	75.28	0.07	0.04	67.45	0.05	0.04	78.66
Springfield	0.07	0.05	75.11	0.04	0.03	72.53	0.03	0.02	85.52	0.03	0.03	95.19	0.04	0.04	96.87	0.07	0.04	68.49	0.05	0.04	88.18
Chicago 1	0.06	0.05	76.87	0.04	0.03	81.24	0.03	0.02	89.92	0.02	0.02	89.27	0.02	0.02	103.85	0.06	0.05	84.16	0.04	0.04	108.23
Chicago 2	0.08	0.06	75.44	0.05	0.04	78.69	0.03	0.03	93.82	0.02	0.03	155.62	0.02	0.03	137.80	0.08	0.06	84.47	0.05	0.06	113.89
Minn. - St. Paul	0.05	0.05	84.47	0.03	0.03	80.69	0.01	0.01	79.28	0.04	0.04	96.21	0.05	0.05	115.28	0.01	0.02	174.94	0.03	0.04	127.81
Above Freezing Temperatures, °F																					
Cairo	60.63	4.22	6.94	55.80	4.37	7.84	45.90	3.18	6.97	42.34	2.72	6.43	44.77	2.82	6.30	52.72	4.51	8.59	50.42	7.98	15.86
Springfield	58.69	3.96	6.72	51.51	4.62	8.98	40.58	3.33	8.22	36.58	2.32	6.22	38.24	2.39	6.17	46.66	4.72	10.03	45.70	9.03	19.76
Chicago 1	56.11	3.39	6.03	48.55	4.32	8.88	38.03	2.94	7.69	34.02	1.48	4.27	34.77	1.67	4.74	41.72	4.62	10.87	42.93	8.84	20.61
Chicago 2	56.75	3.78	6.65	47.98	4.75	9.88	37.69	2.94	7.72	34.10	1.61	4.66	34.72	1.48	4.20	42.02	5.16	12.07	43.81	9.03	20.67
Minn. - St. Paul	52.08	2.92	5.62	42.26	4.14	9.74	33.71	1.26	3.71	32.24	0.15	0.46	None	--	--	37.36	3.10	8.20	43.09	8.08	18.73
Duration of Thawing, Days																					
Cairo	30.38	0.00	0.00	30.00	0.00	0.00	31.00	0.00	0.00	31.00	0.00	0.00	28.00	0.00	0.00	31.00	0.00	0.00	181.38	0.00	0.00
Springfield	30.38	0.00	0.00	30.00	0.00	0.00	31.00	0.00	0.00	30.13	0.00	0.00	25.57	0.00	0.00	31.00	0.00	0.00	170.66	5.58	7.52
Chicago 1	30.28	0.00	0.00	30.00	0.00	0.00	31.00	0.00	0.00	27.48	0.00	0.00	20.60	1.40	13.97	30.46	0.00	0.00	148.95	15.87	22.02
Chicago 2	30.28	0.00	0.00	30.00	0.00	0.00	27.84	1.56	13.87	20.31	0.26	3.37	15.73	0.40	4.87	29.84	0.00	0.00	112.35	26.43	40.36
Minn. - St. Paul	30.38	0.00	0.00	30.00	0.00	0.00	25.19	0.00	0.00	1.02	0.00	0.00	0.00	0.00	0.00	2.41	0.00	0.00	75.23	16.07	34.78
Number of Freeze Thaw Cycles																					
Cairo	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Springfield	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.10	0.32	316.23	0.00	0.00	0.00	0.00	0.00	0.00	0.10	0.32	316.23
Chicago 1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.20	0.42	210.82	0.10	0.32	316.23	0.00	0.00	0.00	0.30	0.48	161.02
Chicago 2	0.00	0.00	0.00	0.00	0.00	0.00	0.30	0.48	161.02	0.30	0.48	161.02	0.10	0.32	316.23	0.00	0.00	0.00	0.70	0.67	96.42
Minn. - St. Paul	0.00	0.00	0.00	0.00	0.00	0.00	0.80	0.42	52.70	0.20	0.42	210.82	0.00	0.00	0.00	0.00	0.00	0.00	1.00	0.00	0.00

Table 5.10. Frost Action Parameter Data at 66.0 cm (26 in.) Depth.

Station	October			November			December			January			February			March			Year		
	Avg.	σ[1]	V,%[2]	Avg.	σ	V,%	Avg.	σ	V,%	Avg.	σ	V,%	Avg.	σ	V,%	Avg.	σ	V,%	Avg.	σ	V,%
Cooling Rates, °F/hr																					
Cairo	0.03	0.02	68.66	0.04	0.03	61.88	0.03	0.02	68.85	0.03	0.02	69.60	0.03	0.02	70.95	0.03	0.02	72.05	0.03	0.02	69.76
Springfield	0.04	0.02	65.69	0.04	0.03	62.99	0.03	0.02	76.10	0.02	0.01	78.09	0.02	0.02	76.81	0.03	0.02	67.11	0.03	0.02	79.48
Chicago 1	0.03	0.02	69.20	0.04	0.02	62.91	0.02	0.02	83.77	0.01	0.01	98.72	0.01	0.01	109.48	0.03	0.02	79.58	0.02	0.02	97.68
Chicago 2	0.04	0.03	65.76	0.04	0.03	63.25	0.02	0.02	83.88	0.01	0.01	92.70	0.01	0.01	102.36	0.03	0.02	79.94	0.03	0.03	100.38
Minn. - St. Paul	0.04	0.02	66.96	0.03	0.02	59.64	0.01	0.01	84.40	0.03	0.04	110.07	0.06	0.05	86.08	0.02	0.02	74.57	0.03	0.03	115.70
Below Freezing Temperatures, °F																					
Cairo	None	--	--	None	--	--	None	--	--	None	--	--	None	--	--	None	--	--	None	--	--
Springfield	None	--	--	None	--	--	None	--	--	31.85	0.07	0.21	31.71	0.12	0.39	None	--	--	31.72	0.13	0.40
Chicago 1	None	--	--	None	--	--	None	--	--	31.83	0.08	0.25	31.83	0.08	0.25	31.76	0.06	0.20	31.69	0.14	0.44
Chicago 2	None	--	--	None	--	--	None	--	--	31.52	0.25	0.81	31.00	0.23	0.73	31.57	0.16	0.50	31.45	0.23	0.73
Minn. - St. Paul	None	--	--	None	--	--	31.38	0.23	0.75	28.71	2.27	8.11	27.21	2.28	8.58	30.11	0.58	1.95	28.70	2.37	8.39
Duration of Freezing, Days																					
Cairo	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Springfield	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.29	0.00	0.00	2.30	0.00	0.00	0.00	0.00	0.00	2.59	0.00	0.00
Chicago 1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	2.09	0.00	0.00	5.40	0.00	0.00	0.30	0.00	0.00	7.80	0.00	0.00
Chicago 2	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	3.68	0.00	0.00	5.60	0.00	0.00	1.34	0.00	0.00	10.62	0.00	0.00
Minn. - St. Paul	0.00	0.00	0.00	0.00	0.00	0.00	1.83	0.00	0.00	26.72	0.00	0.00	28.00	0.00	0.00	29.06	0.00	0.00	85.60	0.00	0.00
Warming Rates, °F/hr																					
Cairo	0.05	0.04	81.23	0.03	0.02	75.88	0.02	0.02	70.52	0.03	0.02	71.86	0.03	0.02	73.38	0.04	0.03	70.79	0.04	0.03	82.46
Springfield	0.05	0.04	77.47	0.03	0.02	74.79	0.02	0.02	79.23	0.02	0.02	81.83	0.03	0.02	84.32	0.04	0.03	72.38	0.03	0.03	86.00
Chicago 1	0.04	0.03	75.56	0.03	0.02	73.92	0.02	0.01	86.50	0.02	0.01	72.80	0.02	0.02	93.38	0.04	0.03	77.44	0.03	0.03	97.62
Chicago 2	0.05	0.04	76.54	0.03	0.02	78.74	0.02	0.02	74.14	0.02	0.02	81.01	0.02	0.02	102.35	0.05	0.04	86.87	0.03	0.04	107.39
Minn. - St. Paul	0.04	0.03	83.26	0.02	0.02	74.11	0.01	0.01	58.36	0.03	0.03	95.49	0.04	0.04	116.14	0.01	0.02	177.27	0.02	0.03	131.82
Above Freezing Temperatures, °F																					
Cairo	60.01	4.05	6.73	56.00	4.03	7.19	46.35	2.96	6.43	42.65	2.43	5.73	44.83	2.55	5.70	52.30	4.23	8.08	50.42	7.59	15.08
Springfield	58.20	3.72	6.38	51.89	4.31	8.31	41.19	3.14	7.65	36.99	2.13	5.67	38.41	2.17	5.58	46.34	4.44	9.48	45.77	8.68	18.97
Chicago 1	55.80	3.13	5.58	48.99	4.03	8.21	38.68	2.82	7.27	34.46	1.40	4.01	34.74	1.34	3.77	41.51	4.32	10.22	42.94	8.58	19.98
Chicago 2	56.37	3.39	5.99	48.60	4.36	8.95	38.24	2.85	7.41	34.02	1.28	3.67	34.63	1.34	3.81	41.81	4.73	11.10	43.02	8.89	20.70
Minn. - St. Paul	52.02	2.59	5.00	42.90	3.94	9.11	34.08	1.36	3.99	32.23	0.13	0.39	None	--	--	36.70	2.81	7.66	42.63	8.00	18.75
Duration of Thawing, Days																					
Cairo	30.38	0.00	0.00	30.00	0.00	0.00	31.00	0.00	0.00	31.00	0.00	0.00	28.00	0.00	0.00	31.00	0.00	0.00	181.38	0.00	0.00
Springfield	30.38	0.00	0.00	30.00	0.00	0.00	31.00	0.00	0.00	30.71	0.00	0.00	25.70	0.00	0.00	31.00	0.00	0.00	171.01	5.90	7.60
Chicago 1	30.38	0.00	0.00	30.00	0.00	0.00	31.00	0.00	0.00	38.91	0.00	0.00	22.60	0.00	0.00	30.70	0.00	0.00	159.34	11.52	16.19
Chicago 2	30.38	0.00	0.00	30.00	0.00	0.00	31.00	0.00	0.00	25.91	0.91	6.47	22.40	0.00	0.00	29.66	0.00	0.00	148.86	12.84	19.15
Minn. - St. Paul	30.38	0.00	0.00	30.00	0.00	0.00	29.17	0.00	0.00	4.28	0.00	0.00	0.00	0.00	0.00	1.94	0.00	0.00	81.40	17.57	36.45
Number of Freeze Thaw Cycles																					
Cairo	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Springfield	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.10	0.32	316.23	0.00	0.00	0.00	0.00	0.00	0.00	0.10	0.32	316.23
Chicago 1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.20	0.42	210.82	0.00	0.00	0.00	0.00	0.00	0.00	0.20	0.42	210.82
Chicago 2	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.30	0.48	161.02	0.00	0.00	0.00	0.00	0.00	0.00	0.30	0.48	161.02
Minn. - St. Paul	0.00	0.00	0.00	0.00	0.00	0.00	0.20	0.42	210.82	0.80	0.42	52.70	0.00	0.00	0.00	0.00	0.00	0.00	1.00	0.00	0.00

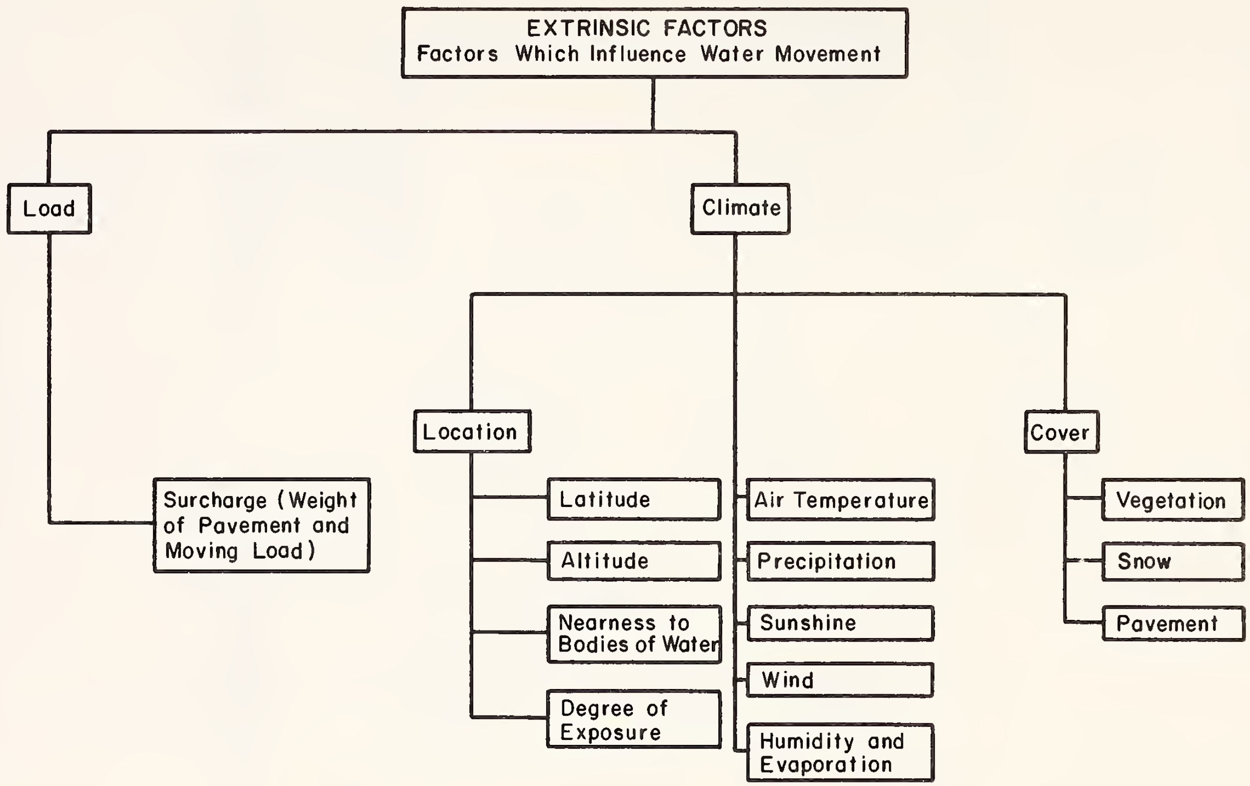

Figure 5.1. Extrinsic Factors Influencing Water Movement.

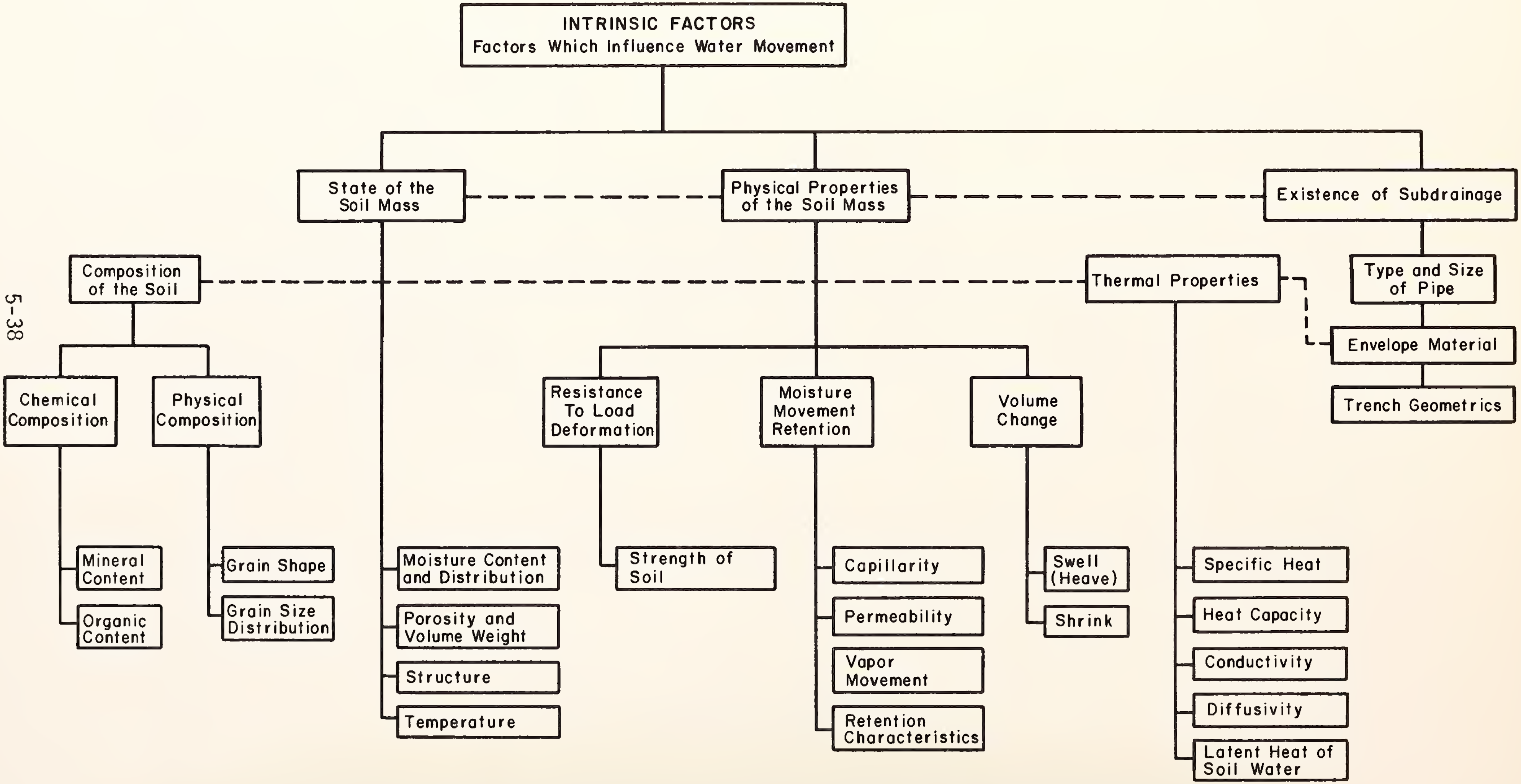

Figure 5.2. Intrinsic Factors Influencing Water Movement.

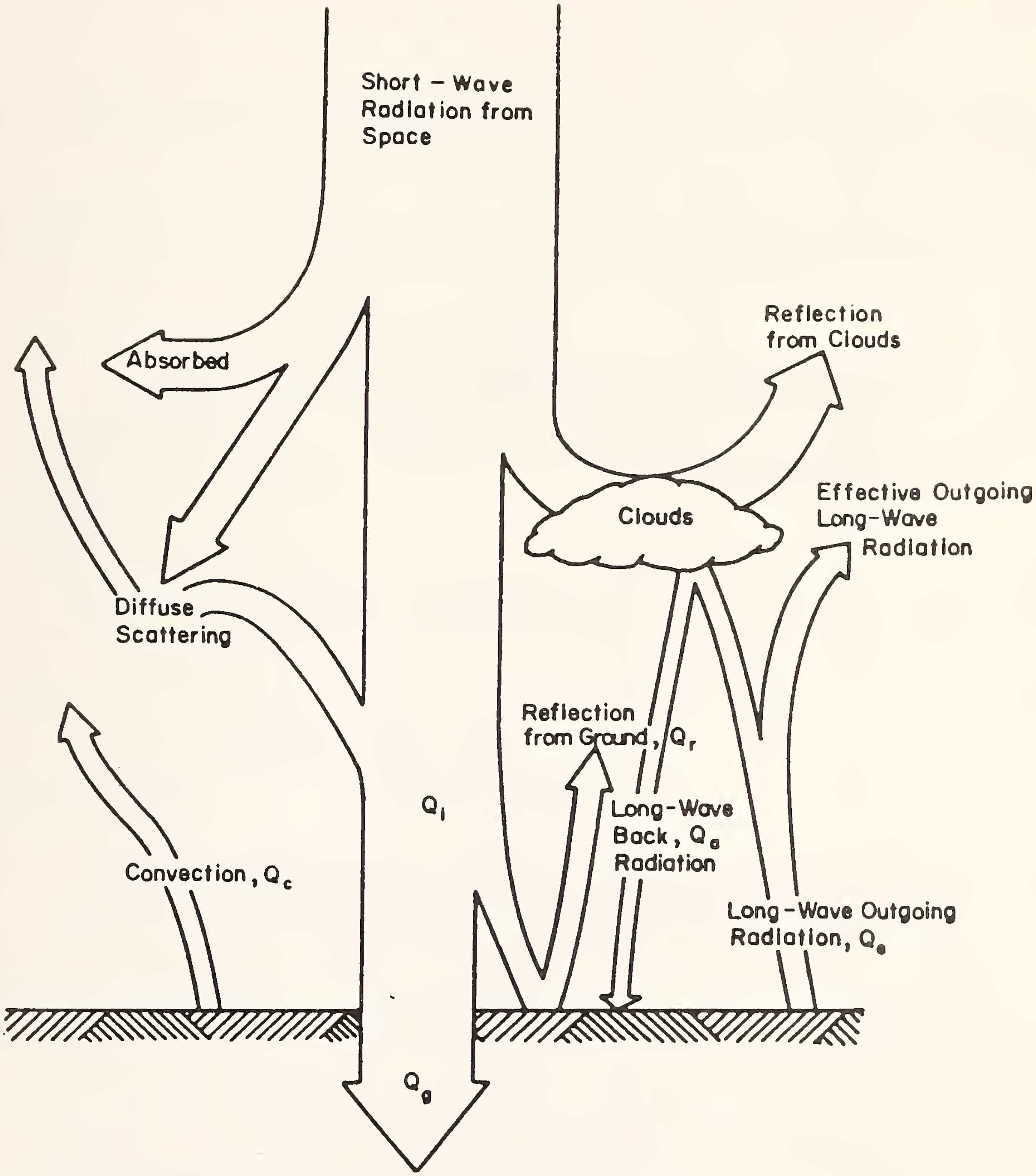

Figure 5.3. Heat Transfer between Pavement Surface and Air on a Sunny Day (Ref. 1).

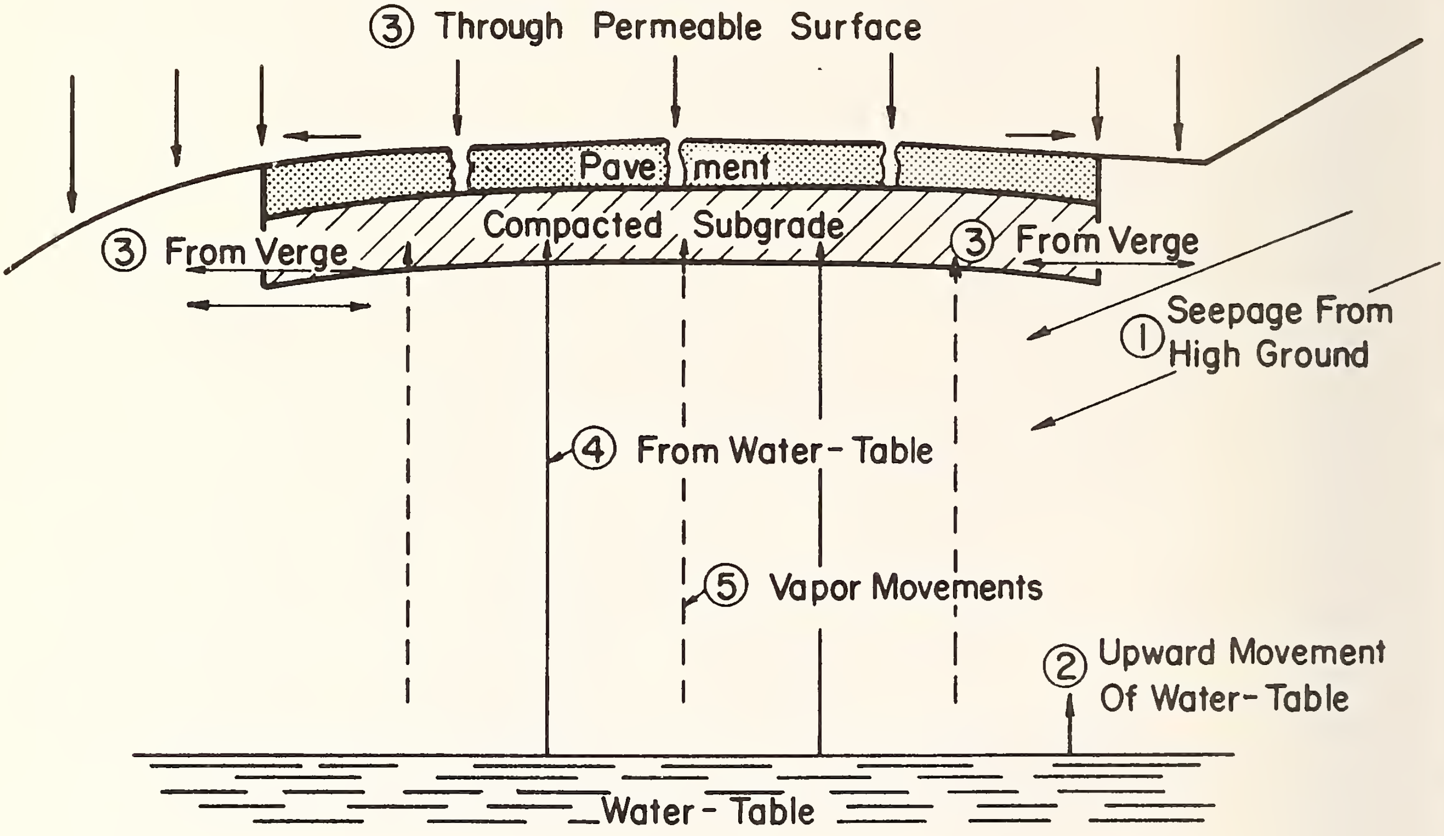

Figure 5.4. Sources of Moisture in Pavement Systems (Ref. 2).

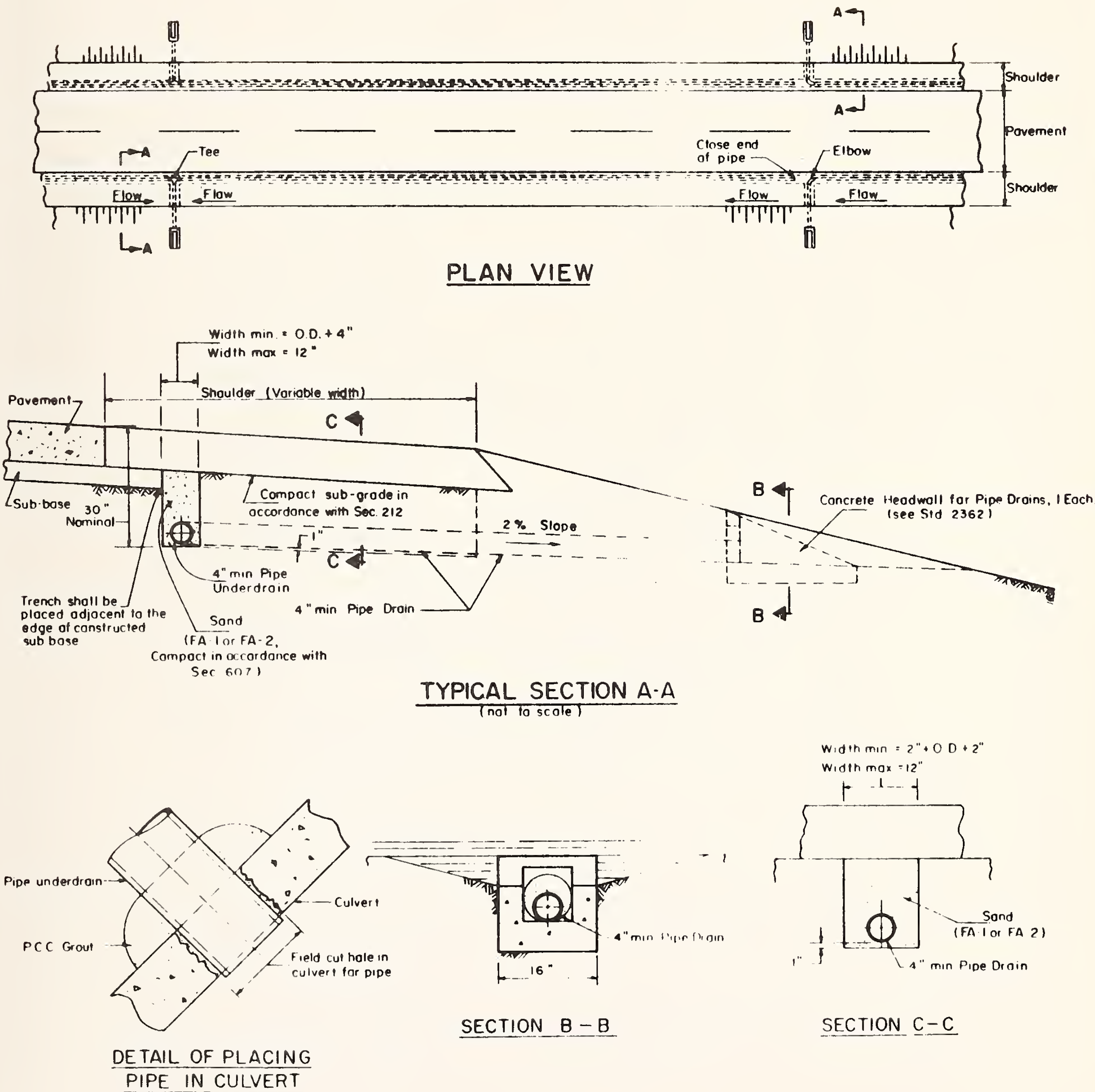

Figure 5.5. Design Standards for Illinois Subdrainage Systems.

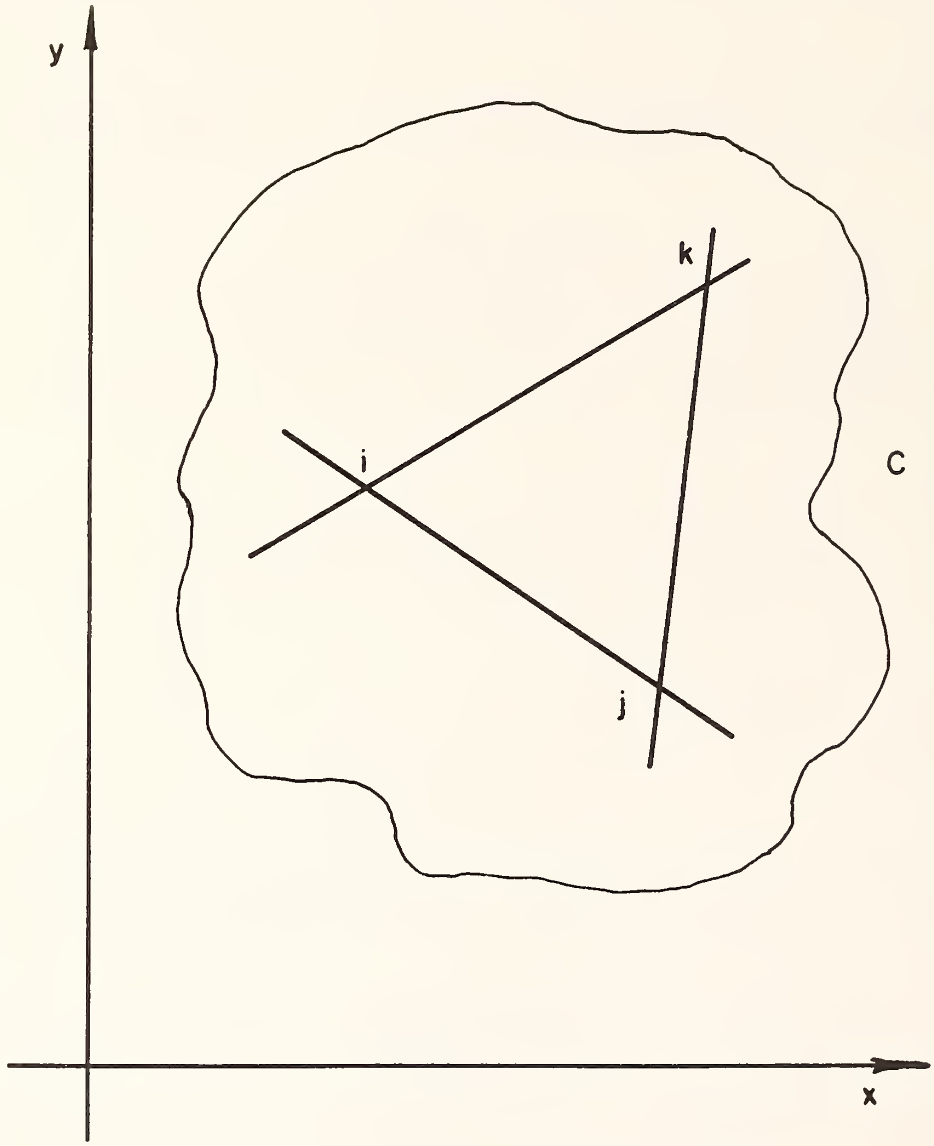

Figure 5.6. Finite Element for Drainage Model.

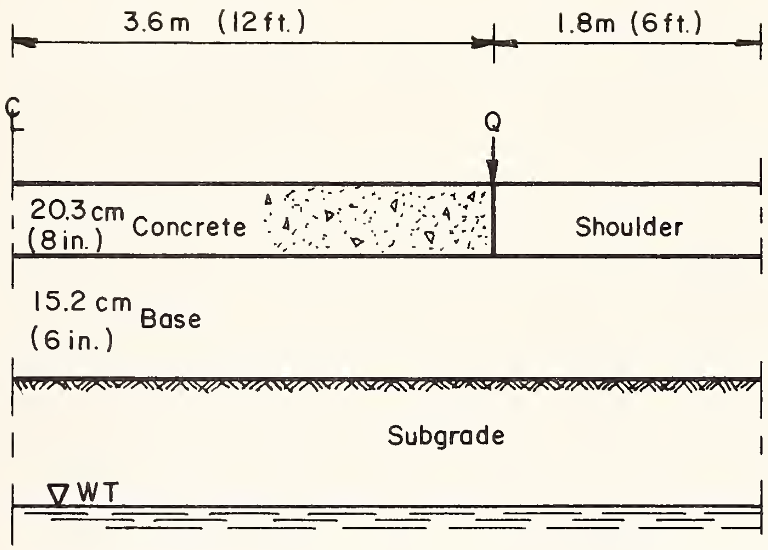

Figure 5.7. Pavement Section Without a Subdrain Pipe used in Drainage Model Study.

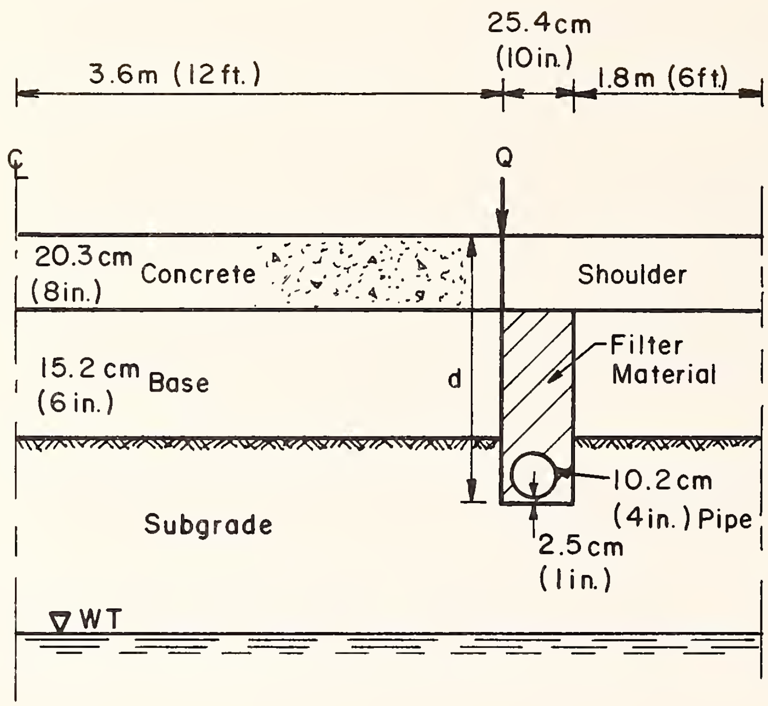

Figure 5.8. Pavement Section with Subdrain Pipe Used in Drainage Model Study.

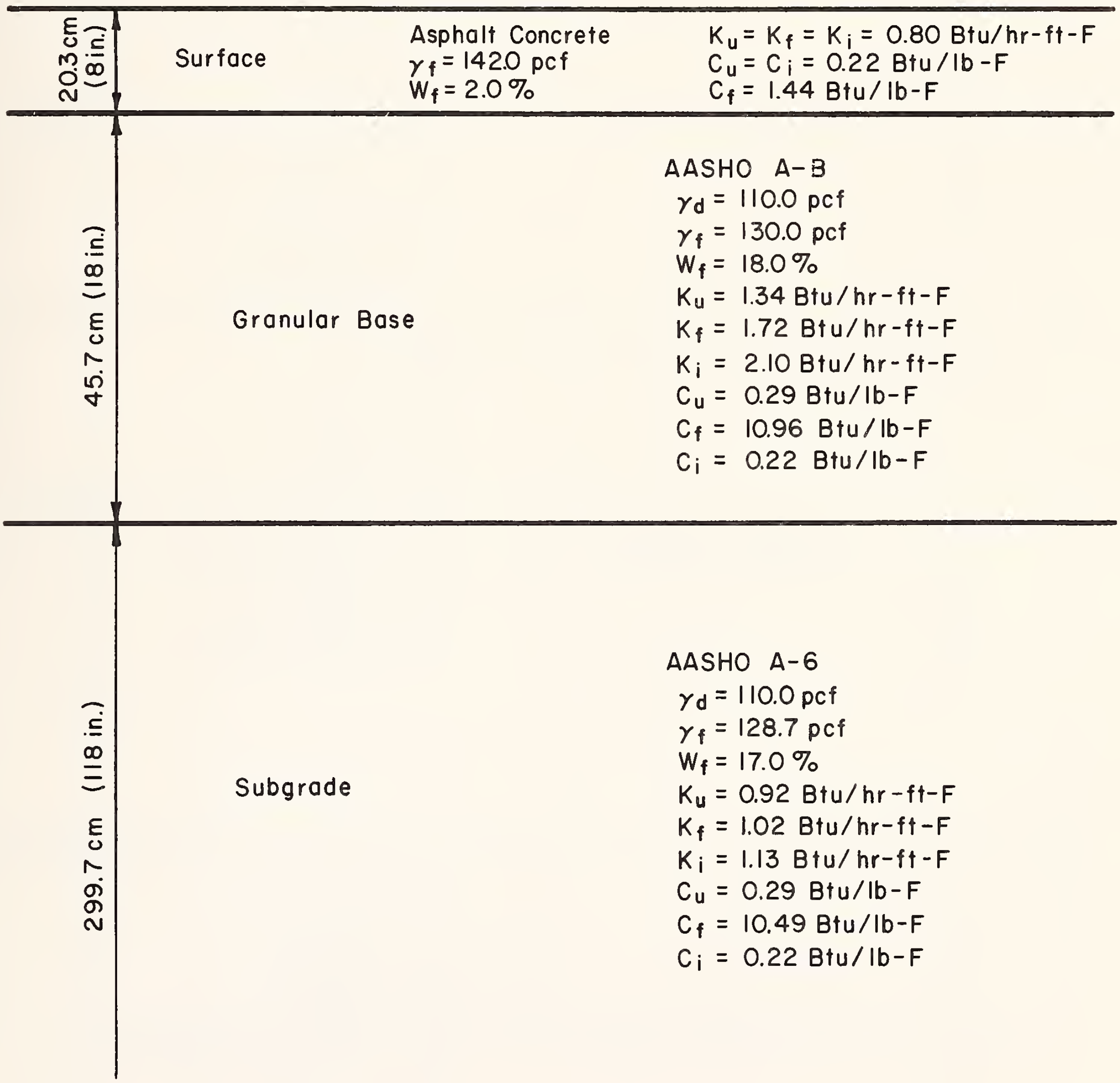

Figure 5.9. Thermal and Physical Properties of Typical Shoulder Section used in Model.

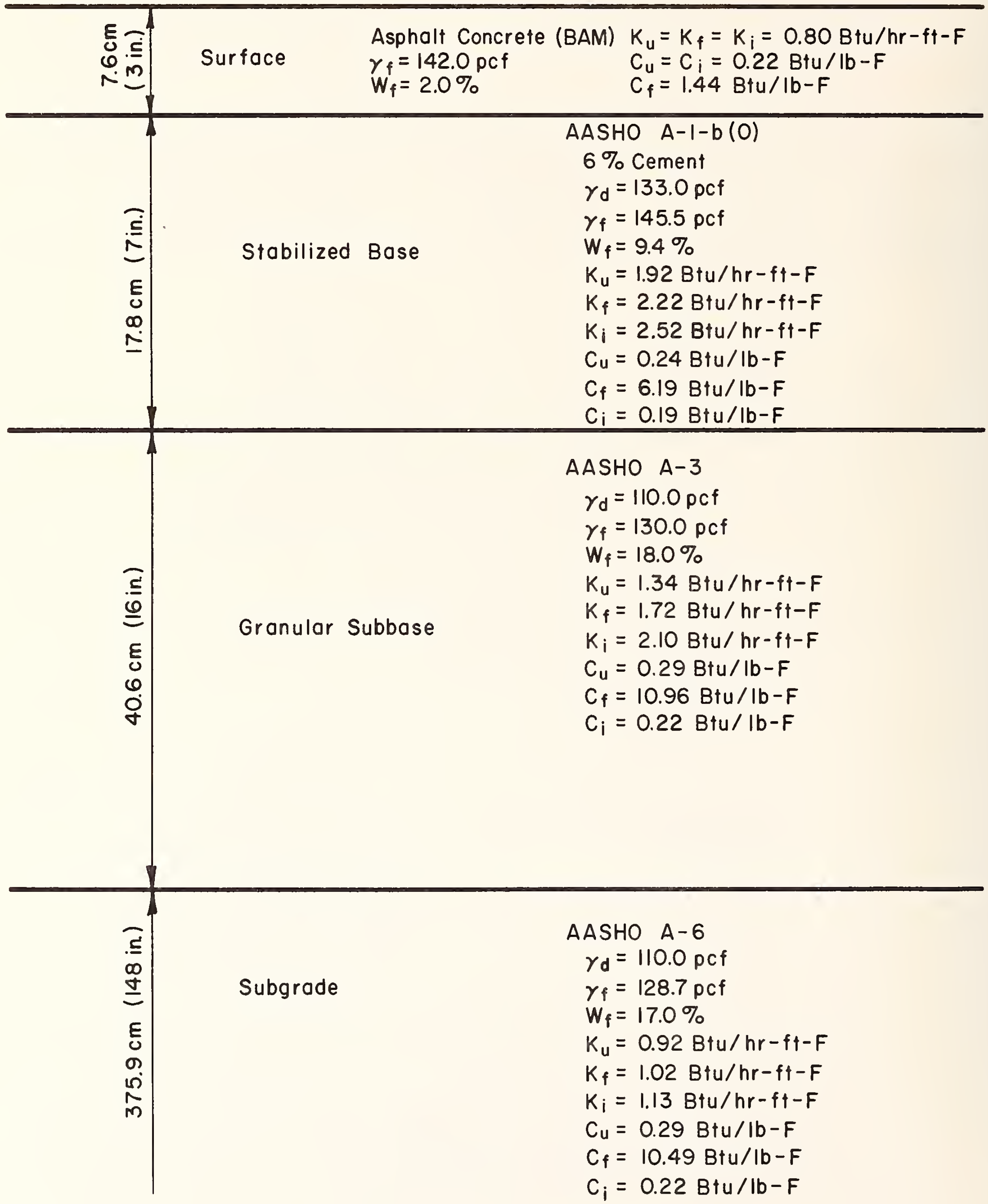

Figure 5.10. Thermal and Physical Properties of Typical Shoulder Section Modeled.

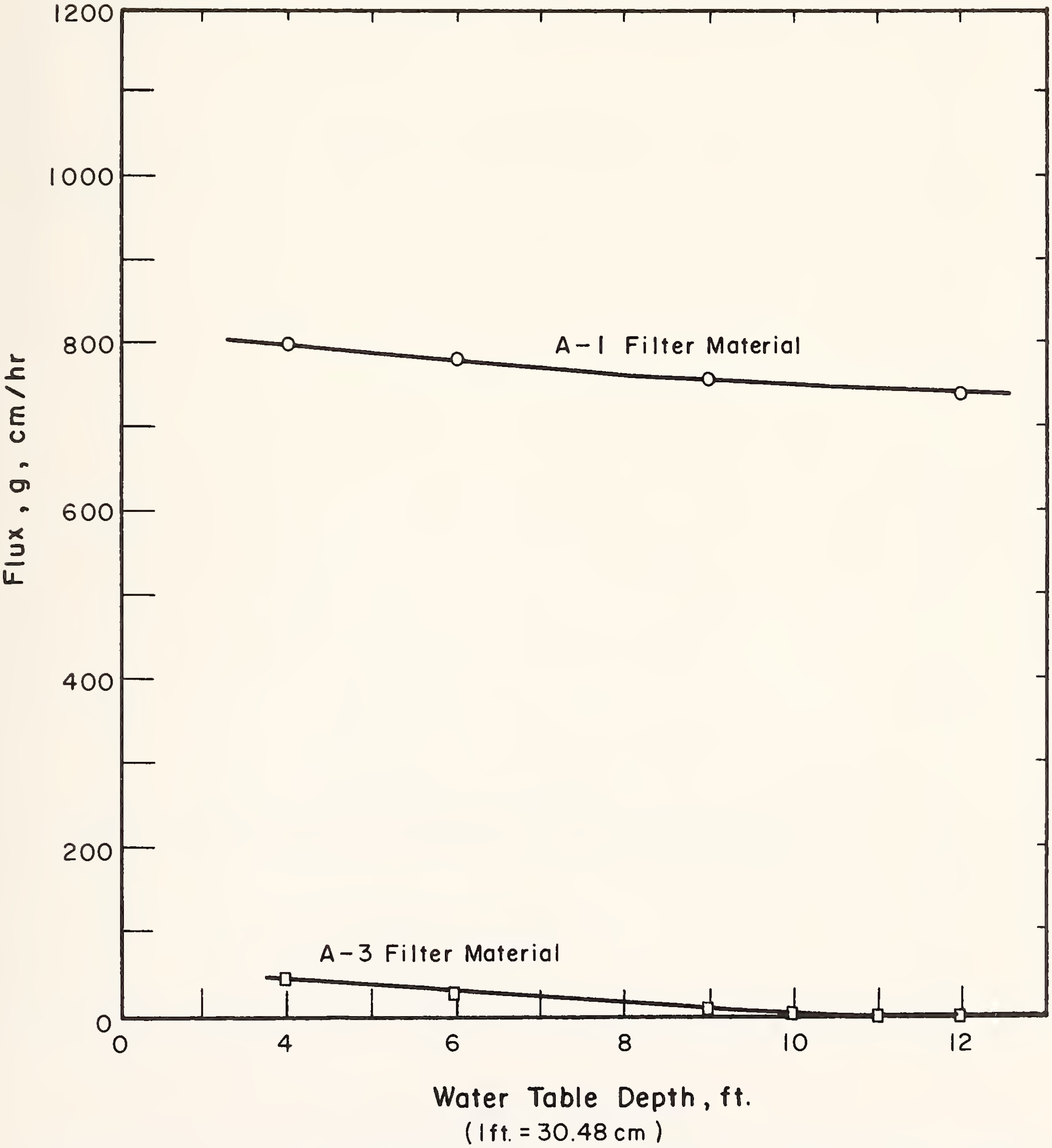

Figure 5.11. Flux to Subdrain Pipe as a Function of Water Table Depth for A-1 and A-3 Filter Materials and AASHTO A-3 Subgrade Soil.

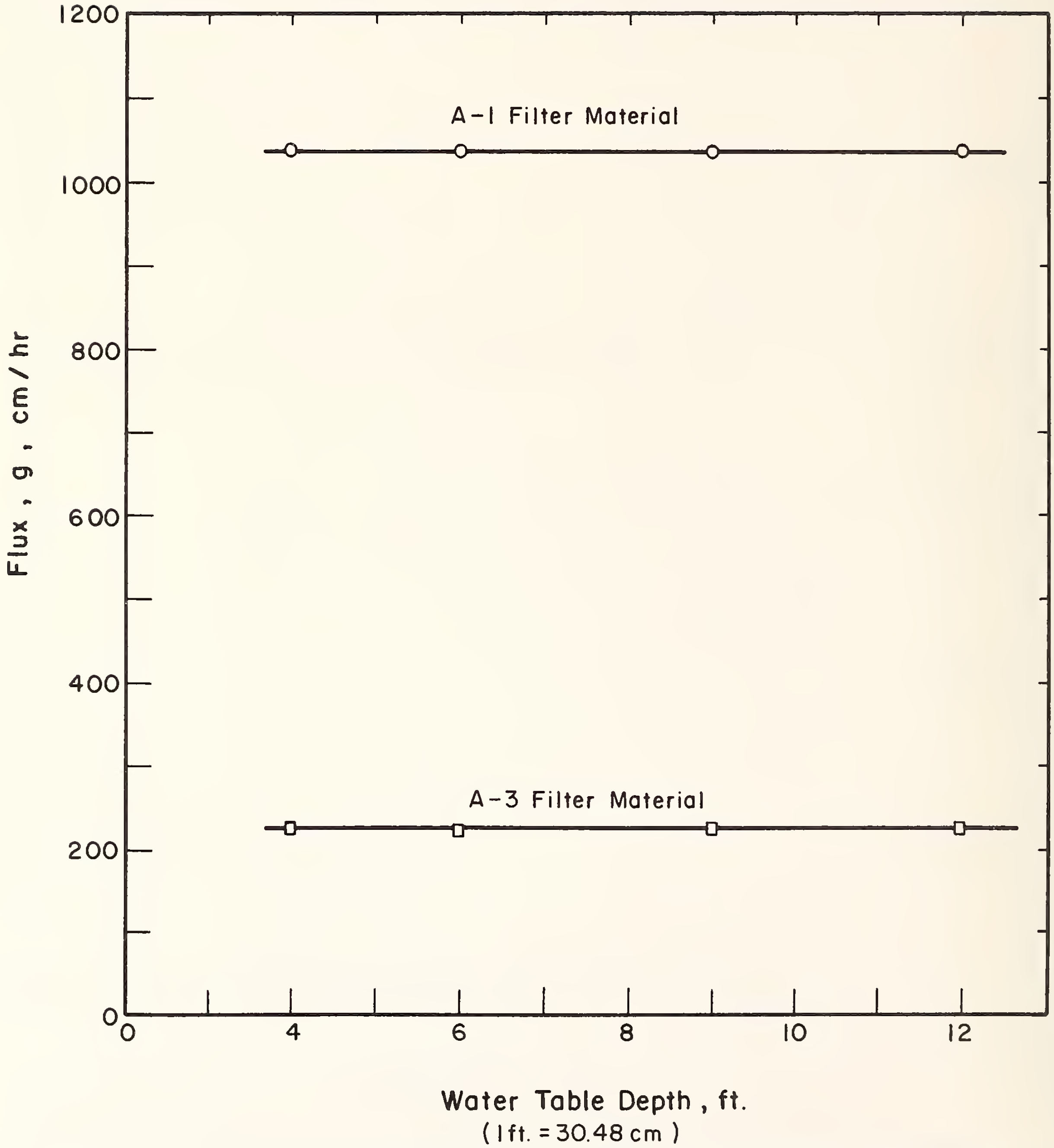

Figure 5.12. Flux to Subdrain Pipe as a Function of Water Table Depth for A-1 and A-3 Filter Materials and AASHTO A-4 or A-7 Subgrade Soils.

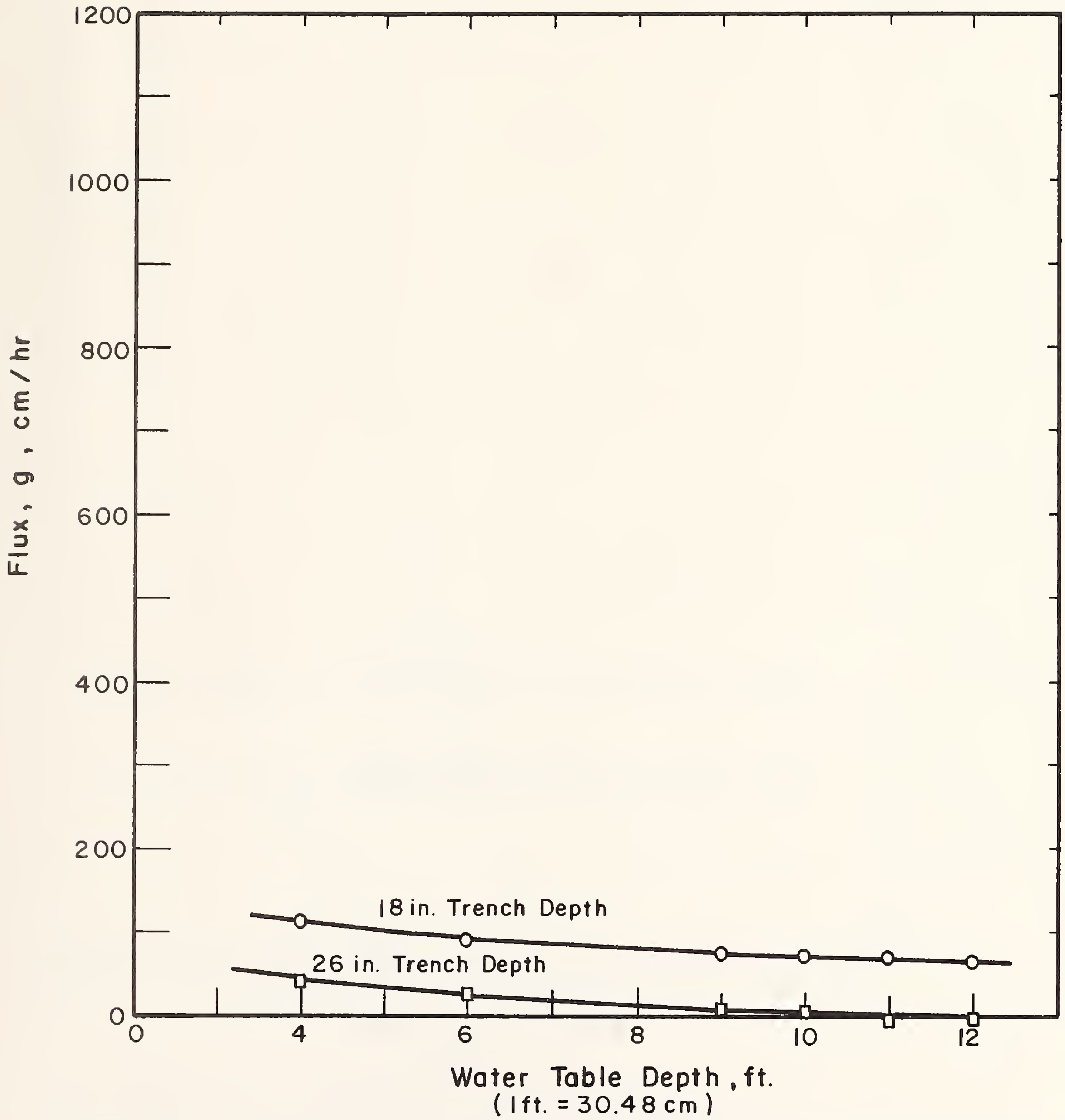

Figure 5.13. Flux to Subdrain Pipe as a Function of Water Table Depth for Various Trench Sections in Pavement with AASHTO A-3 Subgrade Soil.

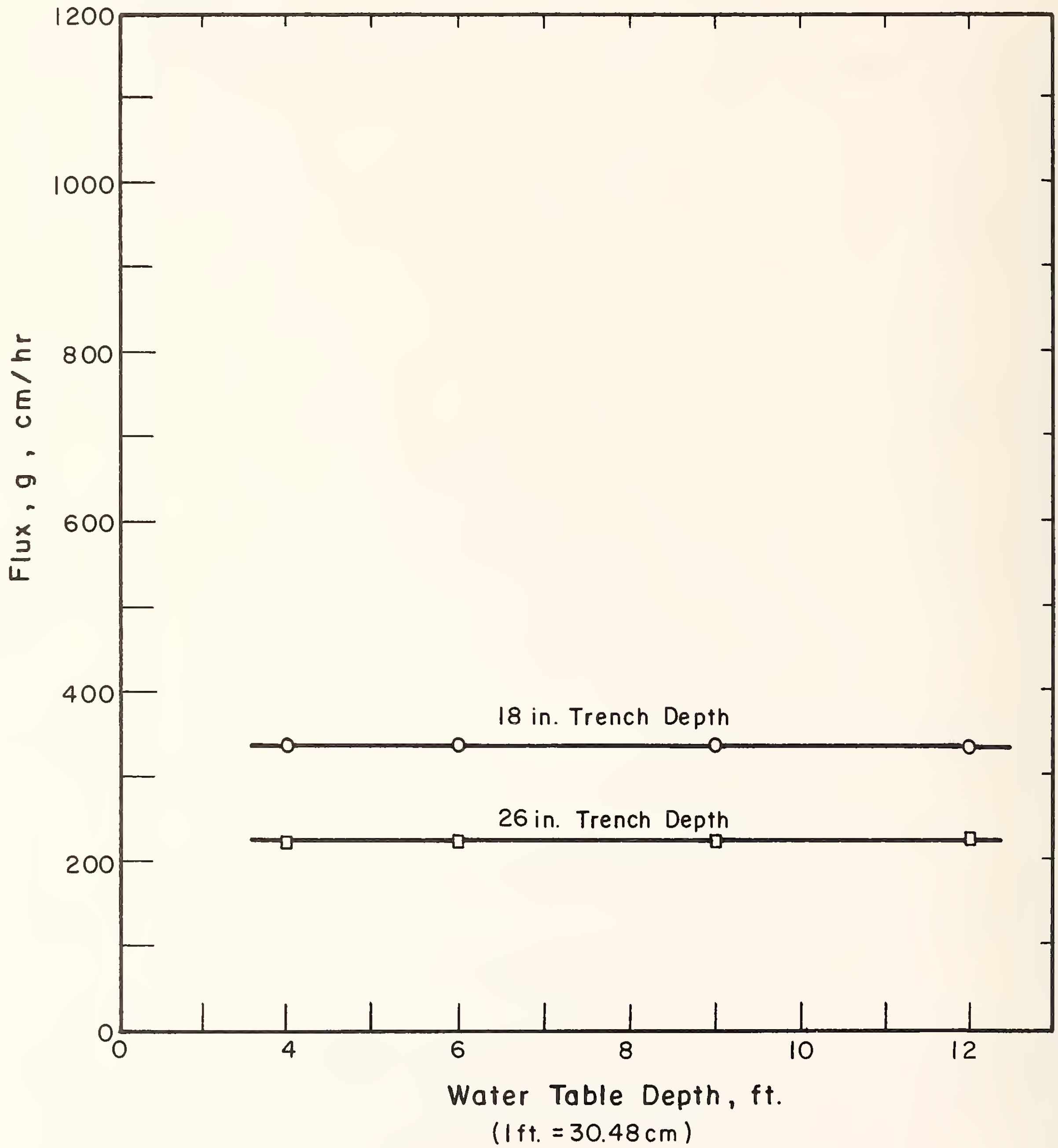

Figure 5.14. Flux to Subdrain Pipe as a Function of Water Table Depth for Various Trench Sections in Pavement with AASHTO A-4 or A-7 Subgrade Soils.

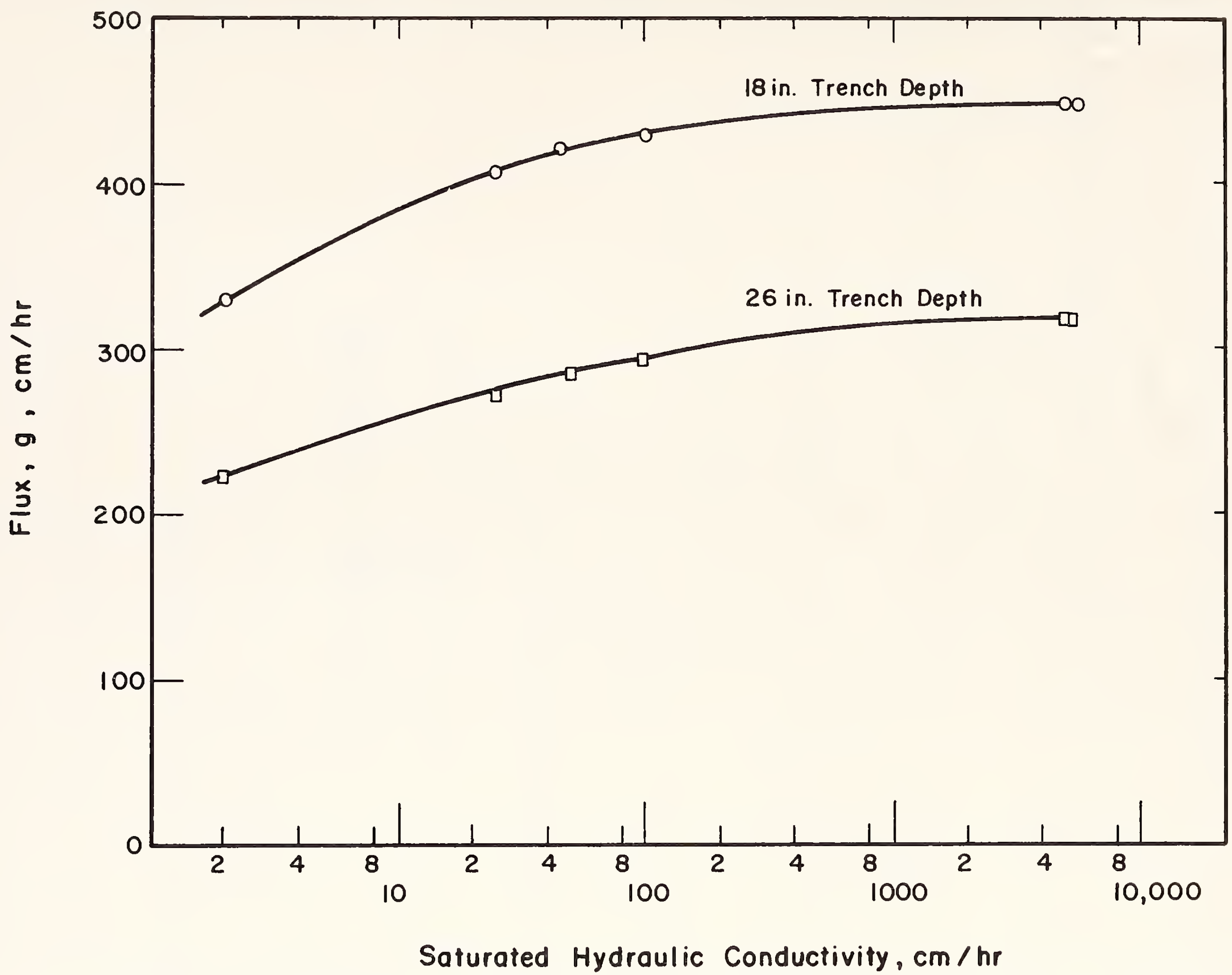

Figure 5.15. Influence of Saturated Hydraulic Conductivity on Flux to Subdrain Pipes in Two Different Trench Sections.

Appendix A

Back calculations with Cedergren's method and data for base material.

Place: Illinois

Mean annual max. rainfall in 24 hrs: 2.35 - 3.10 in

$\rightarrow$ 5.97 - 7.87 cm

Mean annual number of days with precipitation 0.01 in or more: 109 - 120 days

Mean annual maximum rainfall in 1 hr: 1.15 - 1.38 in

$\rightarrow$ 2.92 - 3.51 cm

With mean annual max. rainfall in one hour we get for the following pavement section data:

W = 28 ft, cross slope = 2%, longitudinal grade = 0%

Design infiltration rate = 2/3 x 1.15 = 0.77 in/hr.

2/3 x 1.38 = 0.92 in/hr.

Length of flow path: L = 28 ft

$$\text{gradient}: \quad i = \frac{40 \times 0.02}{28} = \underline{0.029}$$

Required transmissibility KA = Q/i = 0.77 in/hr

or 0.92 in/hr.

$$\text{Infiltration pr. } ft^3 = q_1 = 0.77 \left(\frac{24}{12}\right) = 1.54 \text{ ft/day}$$

$$q_2 = 0.92 \left(\frac{24}{12}\right) = \underline{1.84 \text{ ft/day}}$$

So:

$$Q_1 = 28 \times 1.54 = 43.12 \sim 44 \text{ ft}^3/\text{day}$$

$$Q_2 = 28 \times 1.84 = 51.52 \sim 52 \text{ ft}^3/\text{day}$$

Thus:

$$K_1A = Q_1/i = 44/0.029 = 1517 \text{ ft}^3/\text{day} \sim 1600 \text{ ft}^3/\text{day}$$

$$K_2A = Q_2/i = 52/0.029 = 1793 \text{ ft}^3/\text{day} \sim \underline{\underline{1800 \text{ ft}^3/\text{day}}}$$

With 6" of base drainage layer with effective thickness 5", then

$A = 0.42 \times 1{,}0 = 0.42\ ft^2$ and

$K_1 = 1600/0.42 = 3810$ ft/day ~ 4,000 ft/day

$K_2 = 1800/0.42 = 4286$ ft/day ~ 4,500 ft/day

$$K_1 = 4000 \text{ ft/day} = \frac{4000 \times 30.48}{24} = \underline{5080 \text{ cm/hr}}$$

$$K_2 = 4500 \text{ ft/day} = \frac{4500 \times 30.48}{24} = \underline{5715 \text{ cm/hr}}$$

Thus the K-value range for the base should be 5000 - 6000 cm/hr

With pipe diameter 4", pipe gradient ~ 0.001, 2 lane road infiltration 0.77 and 0.92 we get:

150 ft distance between outlets.

Appendix B

Identification of symbols:

c	= Volumetric heat capacity, cal/cm^3/^{o}C
H	= Hydraulic potential, cm
i,j,k	= Corner points of triangular element
k	= Thermal conductivity, cal/cm/sec/^{o}C
K	= Hydraulic conductivity, cm/sec
L	= Latent heat of fusion, cal/cm^3
q	= Water flux, cm/sec
$\bar{\bar{S}}^{e}_{i,j,k}$	= Stiffness matrix for element i,j,k
$\bar{\bar{S}}_{total}$	= Total stiffness matrix for whole area
w	= Volumetric water content, %
x,y	= Horizontal and vertical co-ordinates, cm
z	= Distance to the water table, cm
γ	= Total unit weight of soil, psi
γ_d	= Dry density of soil, psi

Chapter 6

EFFECTIVENESS OF CURB AND GUTTER

6.1 GENERAL

The utilization of curb and gutter at the shoulder edge has not been widely discussed from the standpoint of drainage. In a National Cooperative Highway Research Program Synthesis it was reported that only six states normally used curbs at the shoulder edge for improved drainage (1). It was reported in the Synthesis that twenty states used curbs under certain circumstances such as for erosion control on high fills or in urban areas (1).

6.2 STUDY FINDINGS

During project field surveys it was found that most curb and gutter construction was utilized in urban areas to provide drainage to a collection system or catch basin. There was also use of curb and gutter for directing water to suitable outlets for the purpose of erosion control on high fills and in pavement cut sections.

Figures 6.1 and 6.2 show concrete curb and gutter along FAI5 in Portland, Oregon. The shoulder joints in this pavement system were very tight and the pavement was performing very well. Figure 6.3 shows a gutter system on FAI75 in Atlanta, Georgia. This high volume pavement system is also performing extremely well.

Figure 6.4 shows a concrete valley gutter design used on FAI70 in Denver, Colorado. This concrete gutter system is placed between the shoulder and the mainline pavement. Field inspection has shown that this gutter system is performing well and appears to be beneficial to the performance of the mainline pavement and shoulder.

California uses considerable curb and gutter for erosion control. Figure 6.5 shows an asphalt concrete curb and gutter on a fill section of FAI5. Figure 6.6 shows a similar curb and gutter construction in a cut section on FAI5. The water is normally carried by the curb and gutter to a drainage outlet as shown in Figure 6.7.

6.3 DISCUSSION OF FINDINGS

From the field investigations it would appear that well designed curb and gutter systems can be beneficial in directing water away from the mainline pavement and shoulder. To perform well the joints in the curb and gutter must be tight so that water cannot infiltrate into the underlying layers. It is also evident that curb and gutter systems must be adequately designed to withstand stresses induced by loads and temperature and remain durable.

It was generally found that curb and gutter systems which did not perform well were associated with pavement systems which were also performing poorly. Both the pavement and curb and gutter system would show faulting, cracking, and disintegration in areas where water was infiltrating into the underlying layers. Often times severe distress in these pavement systems would also be found near the collection inlet or catch basin.

The use of curb and gutter for erosion control in cuts and fills such as those in California appeared to be quite effective and justified. The asphalt concrete curb seemed to be durable and quite compatible with the asphalt concrete shoulder. The drainage outlet shown in Figure 6.7 was well designed and handled outflows effectively.

6.4 SUMMARY AND CONCLUSIONS

Although curb gutter are used by numerous states, few formal studies have been made concerning their performance and benefit to pavement drainage.

In this study curb and gutter were normally found in urban areas to control pavement surface water or in areas where erosion control was necessary.

Based on the field investigation it is concluded that curb and gutter systems can be very effective in erosion control. In relation to pavement drainage it is concluded that curb and gutter systems can be beneficial to roadway performance provided they are designed with adequate strength and durability, and that all joints are tight and sealed to prevent water infiltration into the underlying layers.

REFERENCES

1. "Design and Use of Highway Shoulders", NCHRP Synthesis 63, National Cooperative Highway Research Program, Washington, D. C., 1979.

Figure 6.1. Concrete Curb and Gutter on FAI5 in Oregon.

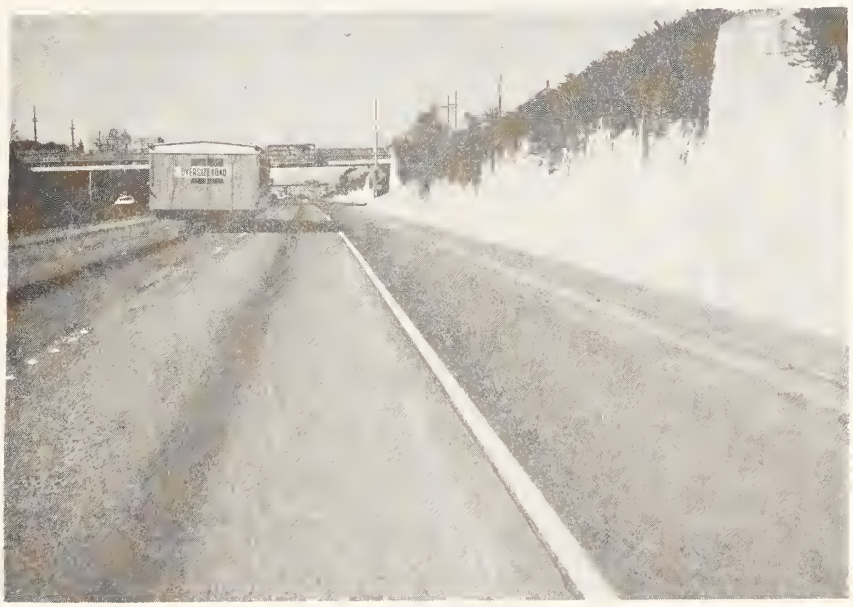

Figure 6.2. Concrete Curb and Gutter on Depressed Section of FAI5 in Oregon.

Figure 6.3. Curb and Gutter Section on FAI75 in Atlanta, Georgia.

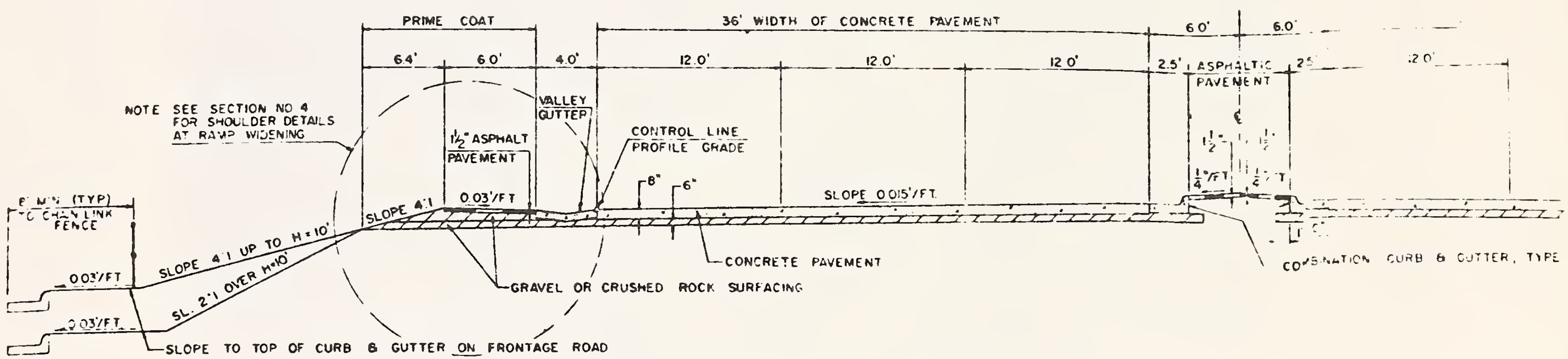

Figure 6.4. Plan Showing Concrete Valley Gutter for FAI70 in Denver, Colorado.

Figure 6.5. Asphalt Concrete Curb and Gutter on FAI5 Fill Section in California.

Figure 6.6. Asphalt Concrete Curb and Gutter on FAI5 at Cut Section in California.

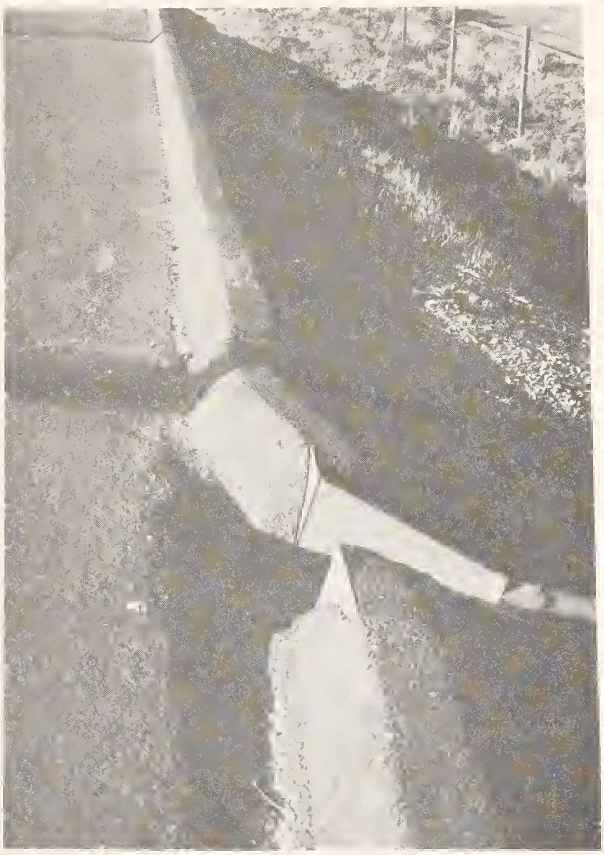

Figure 6.7. Curb Drainage Outlet Used in California.

Chapter 7

RELIEVING BLOCKED SUBDRAINAGE

7.1 GENERAL

Blocked or poorly maintained subdrainage can have detrimental effects on pavement performance. A blocked subdrain may provide a source of water to the pavement system. For this reason it is important to schedule maintenance of pavement subdrainage systems and determine if they are working properly.

Inefficiency in pavement subdrainage is normally caused by poor design or construction practices and clogging with soil, plant roots, or chemical deposits.

To insure that a subdrain is operating properly the following observations are recommended at periodic intervals:

1. Outflow observations to determine discharge rate.
2. Water table observations to determine whether the water table over the drain is lowered to drain shortly after rain stops.
3. Chemical observations to determine if chemical precipitates are present which will clog the drain.

In an effort to relieve blocked subdrainage systems and restore them to full efficiency cleaning and maintenance procedures were investigated. Based on this investigation it became evident that high pressure water jetting could be effectively used to clean soil and plant roots from subdrain pipe. It was found that a 2 percent solution of sulfur dioxide gas and water could be used to remove manganese and iron deposits from subdrains.

7.2 SUBDRAIN BLOCKAGE AND MAINTENANCE

Although field investigations indicated that many miles of subdrainage are performing satisfactorily there was considerable evidence of poor maintenance and blocked drainage. Figures 7.1 and 7.2 show a sample of subdrainage outlets which have been blocked with soil. It was also common to find the outlets blocked with grass clippings from roadside mowing.

Figure 7.3 shows a drainage outlet which has been covered with coarse limestone instead of connected to a headwall or positive outlet. Figure 7.4 shows water flowing from the subdrainage systems after the coarse limestone had been removed from the outlet. There had not been any rainfall on the pavement for a week prior to removing the coarse limestone. Examination of the coarse limestone cover over the pipe outlet indicated that it was filled with both limestone and soil fines along with grass roots; and therefore it had a very low saturated hydraulic conductivity.

The problems shown in Figures 7.1 through 7.4 indicate the need for good subdrainage design practices and maintenance. The drainage outlets especially should be checked several times each year for blockage.

7.3 DEVELOPMENT OF SUBDRAINAGE CLEANING UNIT

Figure 7.5 shows the unit that was developed for cleaning subdrainage pipe. The unit consists of a high pressure pump capable of producing 6894 kPa (1000 psi) of pressure, Figure 7.6, which is driven by a four cylinder engine, Figure 7.7. The nose reel shown in Figure 7.5 contains over 183 m (600 ft) of high pressure 25.4 mm (1 in.) hose. The hose is fed off the reel by the propelling action of the high pressure nozzle shown in Figures 7.8 and 7.9. The hose is rewound by an electric drive motor controlled at the side of the hose reel, Figure 7.10.

In Figure 7.9 the angled jets at the rear of the nozzle propel the nozzle through the pipe and wash the sediments back towards the pipe opening. In some cases a jet may be installed at the nose of the nozzle to help clear a blocked pipe.

Figure 7.11 shows the cleaning action of the nozzle in a pipe and Figure 7.12 shows more clearly the spraying action of the nozzle.

7.4 FIELD TESTING OF SUBDRAINAGE CLEANING UNIT

The jet cleaning unit was field tested on both 10.2 cm (4 in.) circular bituminous fiber pipe and half-round corrugated metal pipe. Figures 7.13, 7.14, and 7.15 show a sequence of the cleaning operation. It was necessary to excavate the shoulder and insert the jet cleaning nozzle directly into the longitudinal drain since the outlet was at a right angle. Approximately 180 m (590 ft) of subdrain was cleaned with the jet cleaning unit.

Figure 7.16 shows some of the sand which was cleaned from an outlet by use of the jet cleaning unit. It was estimated that the nozzle moved at about 15.2 m/min (50 ft/min).

Figure 7.17 shows the jet cleaning unit mounted on a trailer which can be pulled by a water truck. Tests to date indicate that the unit is mobile and very effective for cleaning clogged drainage pipes.

7.5 EVALUATION AND DISCUSSION

Based on laboratory and field evaluationit is felt that subdrainage pipe sections 152 m to 183 m (500 ft to 600 ft) can be cleaned and restored with the jet cleaning unit. This distance is compatible with the spacings normally used between outlets on longitudinal subdrainage systems. Pump pressures of 2758 kPa to 5515 kPa (400 psi to 800 psi) are more than adequate for cleaning sand, silt, and fine materials from a subdrain pipe. These pressures were not found to be detrimental to the subdrain pipes. These high pressures do dictate that safety precautions be taken during cleaning operations to prevent personal injury.

In order to facilitate access to longitudinal subdrainage systems it is recommended that the lateral be connected with a radius as shown in Figure 7.18. It is felt that a radius of 30.5 cm (1 ft) would be adequate for the nozzle and hose to enter the longitudinal subdrain.

It is important to observe subdrain outflows after construction and periodically thereafter to determine if blockage has occurred. It is felt that outflow checks should be made about every 5 years after construction. If outflow decreases by more than 30 percent then cleaning should be considered. It is estimated that a well designed longitudinal subdrainage system should function with very little problems for 10 to 15 years.

The subdrain outlets require more frequent maintenance. It is felt that the outlets should be checked at least two times each year for blockage and possibly each time after the roadway is mowed since grass clippings can easily clog the drain outlet. A headwall or splash block is recommended at the pipe outlet.

In pavement systems where the base course has been extended to the pavement shoulder, (daylighted) programs are necessary to keep fine material from clogging the base course and to keep vegetation growth to a minimum. Based on field observations daylighted base courses are not recommended in areas where fine soil and material can be blown in by the wind. It is felt that daylighted bases can provide good pavement drainage provided they are designed with adequate saturated hydraulic conductivity and stability, and the exposed portion can be maintained free of vegetation and fine materials.

In subdrains where chemicals such as mangenese or iron are a problem it is recommended that a 2 percent solution of sulfur dioxide gas and water be injected into the pipe. This procedure can be quite successful in keeping the drain pipe holes or slots open.

7.6 SUMMARY AND CONCLUSIONS

Procedures for relieving blocked subdrainage systems were presented and demonstrated. It was shown that a jet cleaning unit could be effectively used to clean long sections of subdrain pipe.

Based on study findings the following conclusions are supported:

1. Drainage outlets require considerable maintenance and should be checked at least two times each year.
2. Pipe outflows should be checked after construction and at periodic intervals of about 5 years to determine if blockage is occurring.
3. Subdrainage pipes up to 183 m (600 ft) can be effectively cleaned using a jet cleaning unit.
4. Manganese and iron can be chemically removed from subdrain pipe by using a solution of sulfur dioxide gas and water.

Figure 7.1. Blocked Subdrainage Outlet Pipe.

Figure 7.2. Subdrainage Outlet Headwall Blocked With Soil.

Figure 7.3. Subdrainage Outlet Pipe Covered With Crushed Stone.

Figure 7.4. Crushed Stone Cover Being Removed From Pipe Outlet in Figure 7.3.

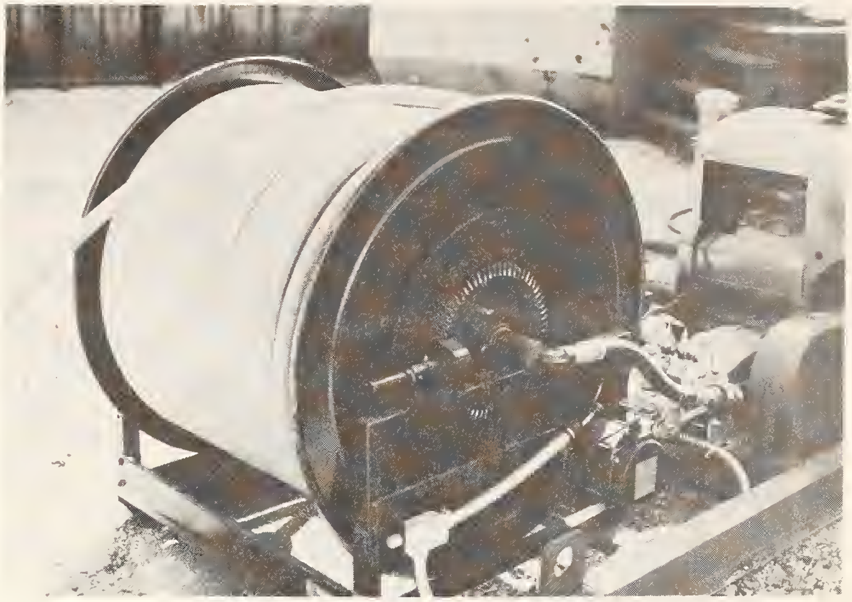

Figure 7.5. High Pressure Jet Cleaning Unit.

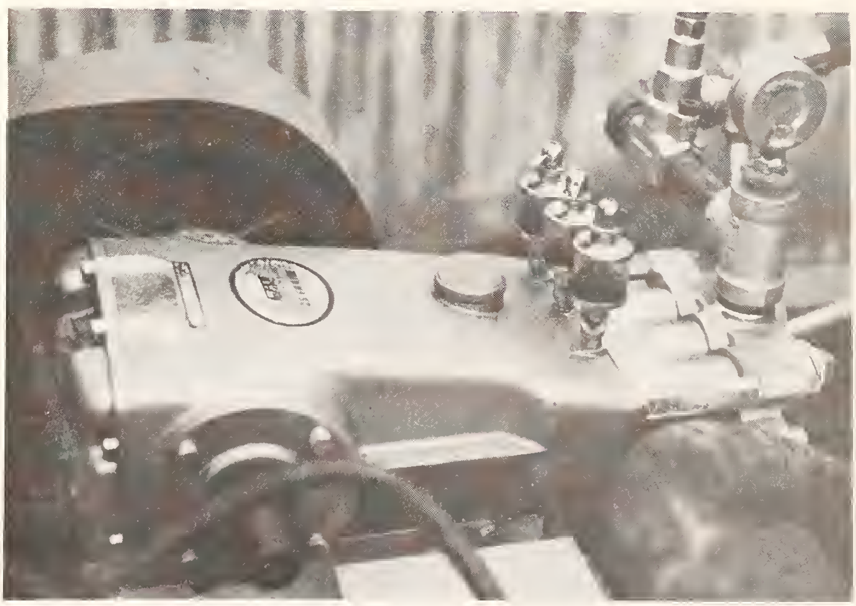

Figure 7.6. High Pressure Pump.

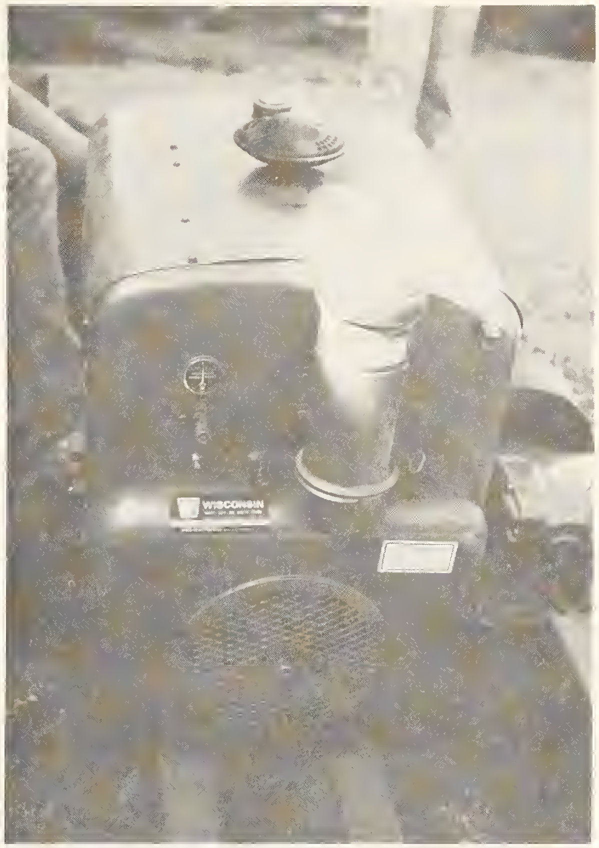

Figure 7.7. Engine for Driving High
Pressure Pump.

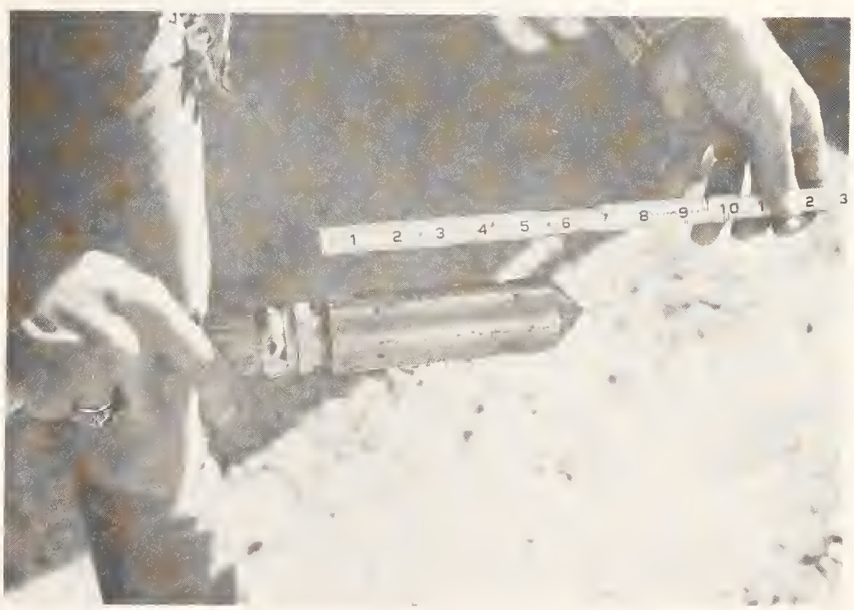

Figure 7.8. High Pressure Nozzle.

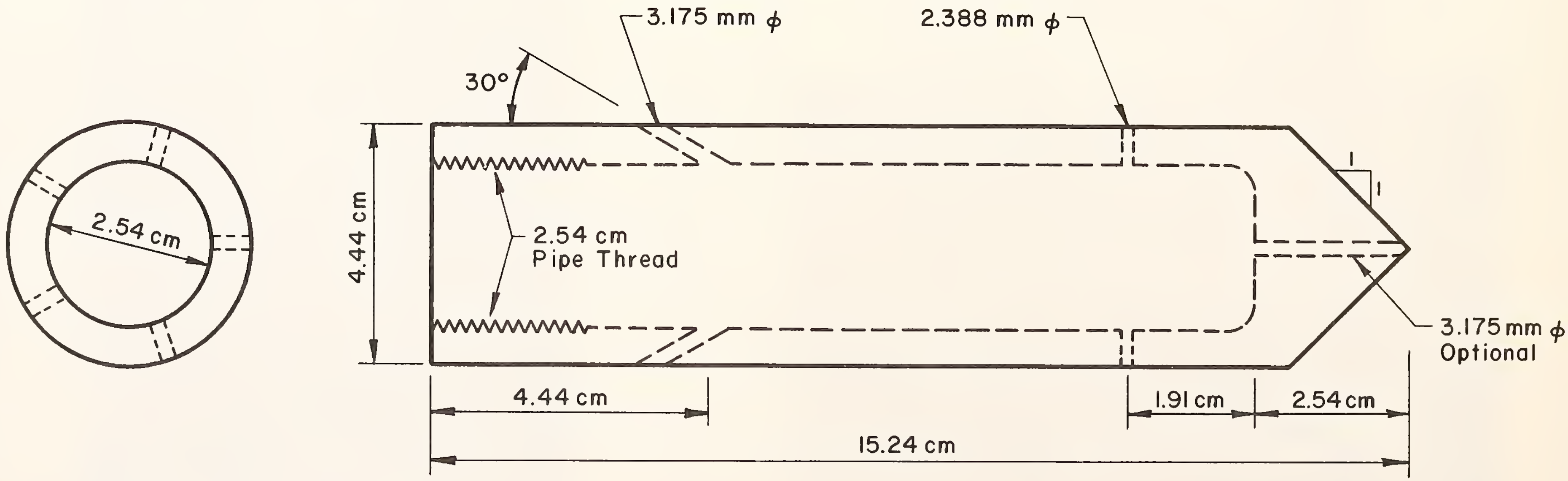

Figure 7.9. Dimensions of the High Pressure Pipe Cleaning Nozzle.

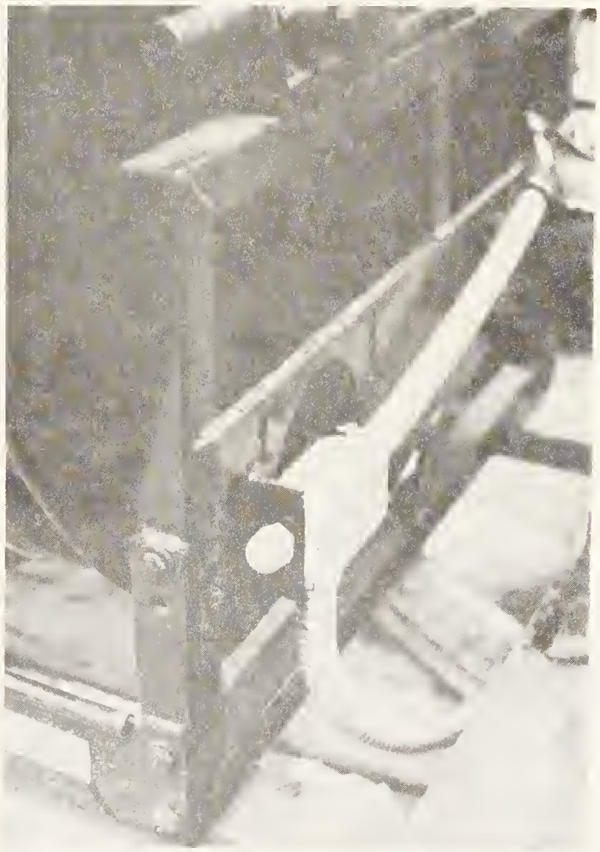

Figure 7.10. Control for Electric Drive Motor which Rewinds Hose on Reel.

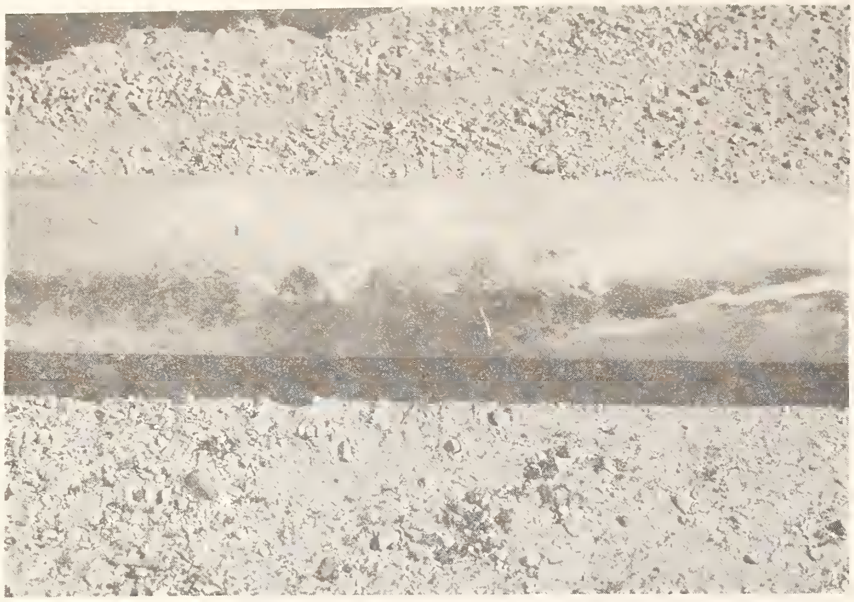

Figure 7.11. Cleaning Action of High Pressure Nozzle.

Figure 7.12. Spraying Action of Nozzle Jets.

Figure 7.13. Excavation to Longitudinal Subdrain Pipe. (Note poor connection between longitudinal pipe and outlet pipe.)

Figure 7.14. Insertion of Cleaning Jet and Hose into Subdrain Pipe.

Figure 7.15. Jet Cleaning a Longitudinal Bituminous Fiber Pipe Section.

Figure 7.16. Sand Removed from a Drainage Outlet by use of the Jet Cleaning Unit.

Figure 7.17. Jet Cleaning Unit Mounted on a Trailer.

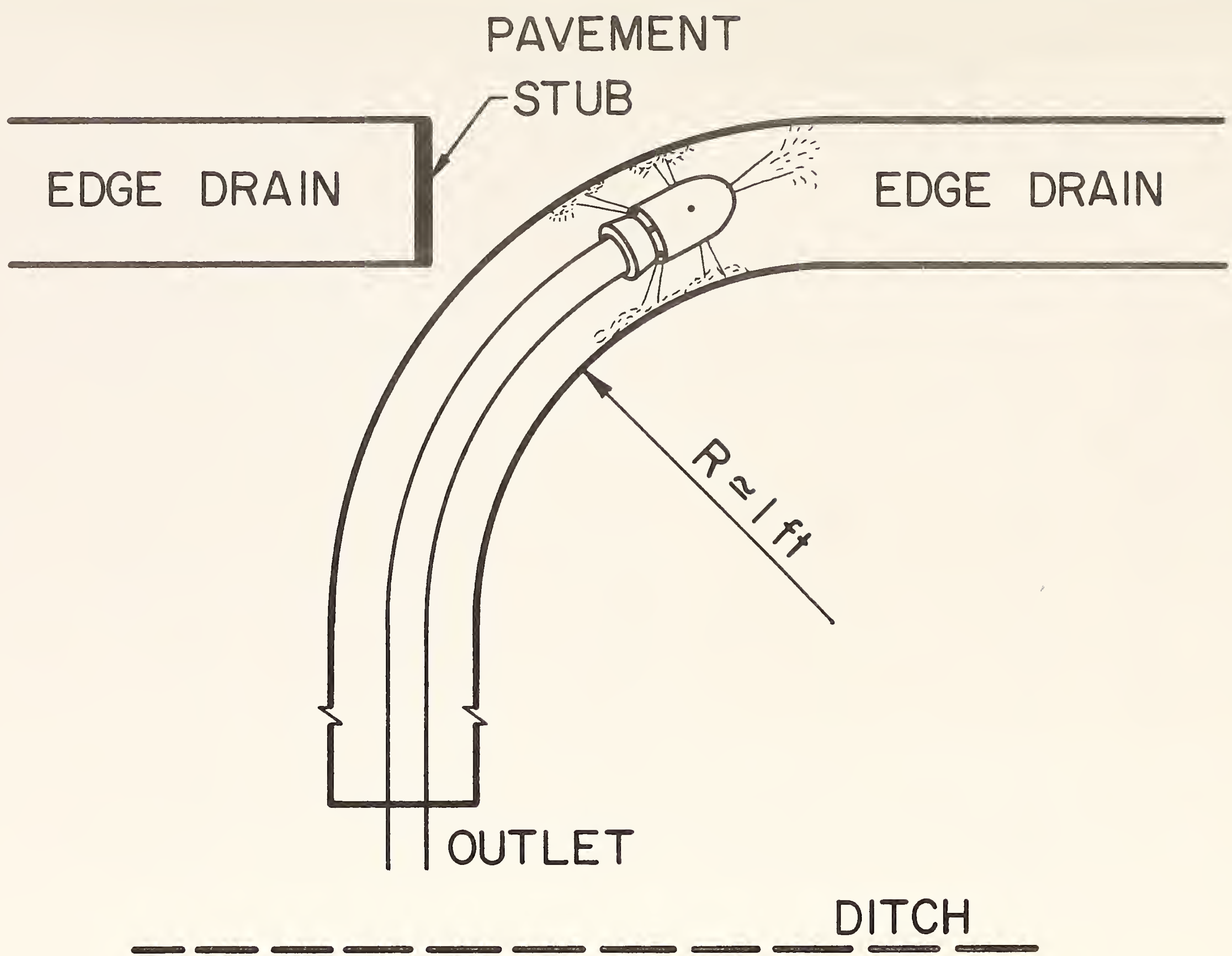

Figure 7.18. Radius Required Between Outlet Pipe and Longitudinal Edge Drain for Jet Cleaning.

Chapter 8

DESIGN PHILOSOPHY FOR WATER IN PAVEMENTS

8.1 GENERAL

From the study of "Improving Subdrainage and Shoulders of Existing Pavements", a design philosophy for water in pavement systems based on concepts for predicting water conditions and methods of controlling water content is recommended. Figures 8.1 through 8.6 show the steps required for designing pavement systems which will resist the influence of water. The approach shown is similar to that proposed by the Organization for Economic Co-operation and Development in a road research report entitled "Water in Roads" (1).

In this chapter each of the drainage design considerations shown in Figure 8.1 through 8.6 will be briefly discussed and pertinent project references where solutions can be found will be listed.

8.2 INFLUENCE OF WATER CONTENT AND FLUCTUATIONS

As indicated in Figure 8.1 it is necessary to understand how water influences pavement materials and performance, and to recognize distresses which are related to water problems. Once a water problem is recognized it is then possible to take comprehensive steps to prepare a design procedure for controlling the water problem.

A comprehensive guide for distress identification and measurement can be found in the following project references:

1. A Pavement Moisture Accelerated Distress (MAD) Identification System Users Manual, Volume II(2).
2. Highway Pavement Distress Identification Manual (3).

8.3 PREDICTION OF WATER CONTENT AND ITS FLUCTUATIONS

Figure 8.2 shows the important steps required for the prediction of water content in pavement systems. In Figure 8.2 it is shown that several variables need to be considered during this design phase.

8.3.1 DETERMINATION OF WATER IN PAVEMENT SYSTEM

The presence of water in a pavement system can be recognized by surface indicators such as pumping, bleeding, faulting, and other water related distresses. In existing pavements the presence of water can be determined during patching and maintenance operations or by means of pavement borings. The procedures for analyzing intrinsic factors that impact the moisture related performance of a pavement are described in the following:

1. Improving Subdrainage and Shoulders of Existing Pavements - State of the Art (4).
2. A Pavement Moisture Accelerated Distress (MAD) Identification System, Volume I (5).
3. Evaluation of Pavement Systems for Moisture-Accelerated Distress (6).
4. Chapter 5 in this Report.

8.3.2 DEVELOPMENT AND USE OF DRAINAGE MODELS

The use of drainage models for determining moisture contents and water flux in a pavement system are highly recommended. The analysis of water flux in the various pavement layers by Darcy's equation or some similar method is mandatory. The procedures outlined in the Highway Subdrainage Manual by Moulton (7) and Module III.G. Subdrainage in Techniques for Pavement Rehabilitation (8) are highly recommended. Project reports important to this step are as follows:

1. Improving Subdrainage and Shoulders of Existing Pavements - State of the Art (4).
2. Final Report, Users Manual, Drainage 02 (9).
3. Chapter 5 in this Report.

8.3.3 FIELD AND LABORATORY OBSERVATIONS

All available geologic and soils data should be evaluated to determine whether pavement water problems will exist. Soil maps published by the United States Department of Agriculture can be most helpful in locating potential drainage problem areas. In the field the topography, soils, and geology should be carefully analyzed for potential water sources. The types of vegetation present can also be helpful in determining if water is present. A careful study of the hydrological boundary conditions associated with new pavement design or an existing pavement system is warranted during this design phase. The following project reports are recommended during this study phase:

1. Improving Subdrainage and Shoulders of Existing Pavements - State of the Art (4).

2. Influence of Precipitation, Joints, and Sealing on Pavement Drainage (10).
3. Chapter 3 in this Report.
4. A Pavement Moisture Accelerated Distress (MAD) Identification System, Volume I (5).
5. A Pavement Moisture Accelerated Distress (MAD) Identification System Users Manual, Volume II (2).

8.3.4 DETERMINE PROPERTIES OF SOIL-WATER SYSTEMS

In order to properly design for water problems in a pavement system, it is imperative that the saturated hydraulic conductivitie (coefficient of permeability) be determined for each affected pavement material. If a subdrainage system is to be part of the pavement, the hydraulic properties of all materials associated with that system should be evaluated also. The FHWA report by Moulton and Seals (11) which describes a procedure for determining the in-situ permeability of base and subbase courses should be utilized during this study phase. Other methods for obtaining saturated and unsaturated hydraulic conductivities of pavement materials are described in Reference 4.

8.3.5 EXTRINSIC FACTORS

The extrinsic factors influencing water in pavements have been described in this project and consists mainly of climatic inputs. Procedures for utilizing extrinsic factors to determine the potential for moisture accelerated distress in pavement systems located in various climatic regions can be found in the following reports:

1. A Pavement Moisture Accelerated Distress (MAD) Identification System, Volume I (5).
2. A Pavement Moisture Accelerated Distress (MAD) Identification System Users Manual, Volume II (2).

8.4 METHODS OF CONTROLLING WATER CONTENT

Through the process of predicting the water content in a pavement system it is expected that insight into methods of controlling water content will evolve. Based on this study methods of controlling water content in design are shown in Figure 8.3 and classified as follows:

1. Protection of roadway.
2. Rendering materials insensitive to water.
3. Evacuation of water from the pavement system.

8.4.1 PROTECTION OF ROADWAY

Figure 8.4 shows some of the methods suggested for protecting the roadway from water. Waterproofing techniques can include the use of surface materials with low permeability, low permeability membranes, and joint and crack sealants.

Lateral protection is gained by the use of shoulders and by the use of curb and gutter.

Anti-capillary and anti-frost courses include the use of granular materials which are free of fines and which do not allow for high capillary water content which may cause frost heaving.

In swelling soils the treatment guidelines proposed by Snethen (12) should be evaluated. Based on pumping studies, the amount of fine material passing the number 4 sieve size should be limited in graded granular base courses. However, granular base courses should be well graded above the number 4 sieve in order to provide stability. Materials which do not degradate should be used and the possibility of subgrade intrusion should be evaluated. Project reports relevant to this method of water controll are:

1. Chapter 3 in this Report.
2. Chapter 4 in this Report.
3. Chapter 6 in this Report.
4. Influence of Precipitation, Joints, and Sealing on Pavement Drainage (10).
5. Structural Analysis and Design of PCC Shoulders (13).

8.4.2 RENDERING INSENSITIVE TO WATER

Figure 8.5 shows that materials and soils may be rendered insensitive to water through compaction and/or stabilization and chemical treatment. In using this method it is important to obtain materials of high quality so that pumping and erosion do not occur as a result of slab action. Compaction control is very important for obtaining quality materials stabilized with lime, lime-flyash, cement, or bituminous materials.

Comprehensive soil stabilization guidelines for mixture design, pavement design, and construction have been developed for the FHWA by others (14, 15).

8.4.3 EVACUATION OF WATER

Figure 8.6 shows methods for removing water from a pavement system. Deep drainage should be considered where water table problems or springs exist. Structural drainage should be considered for water from infiltration, groundwater, and ice lense melt water. Well designed subdrainage systems can be effective in providing deep drainage and structural drainage.

Surface drainage is very important in pavement water evacuation. Through good pavement design, construction, and maintenance considerable surface water can be carried to the ditches with little infiltration into the structural section.

The use of vegetation for drainage purposes has been used mainly on roadway cut and fill slopes. It has been found that trees and plants with deep root systems and high transpiration rates can be helpful in maintaining the stability of roadway slopes.

Drainage systems for evacuating water from pavements are described in the Highway Subdrainage Manual by Moulton (7), Module III.G. Subdrainage in Techniques for Pavement Rehabilitation (8), and the following project reports:

1. Improving Subdrainage and Shoulders of Existing Pavements - State of the Art (4).
2. A Pavement Moisture Accelerated Distress (MAD) Identification System, Volume I (5).
3. Chapter 3 in this Report.
4. Chapter 4 in this Report.
5. Chapter 5 in this Report.
6. Chapter 6 in this Report.

8.5 SUMMARY AND RECOMMENDATIONS

In Figures 8.1 through 8.6 the steps required to design for water in pavements have been presented. It has been shown that the combined processes of prediction and control of water are necessary for the ultimate design of pavement systems which will resist water related distresses.

The control of water in pavements may require one or several of the techniques outlined in Figure 8.3.

It is recommended that the approach outlined in this chapter be utilized in the design of pavements to resist the effects of water. The procedure provides for flexibility and adaptability to varying hydrologic, climatic, and geographic conditions. Many of the research findings from the study on Improving Subdrainage and Shoulders of Existing Pavements can be readily inserted into the procedure.

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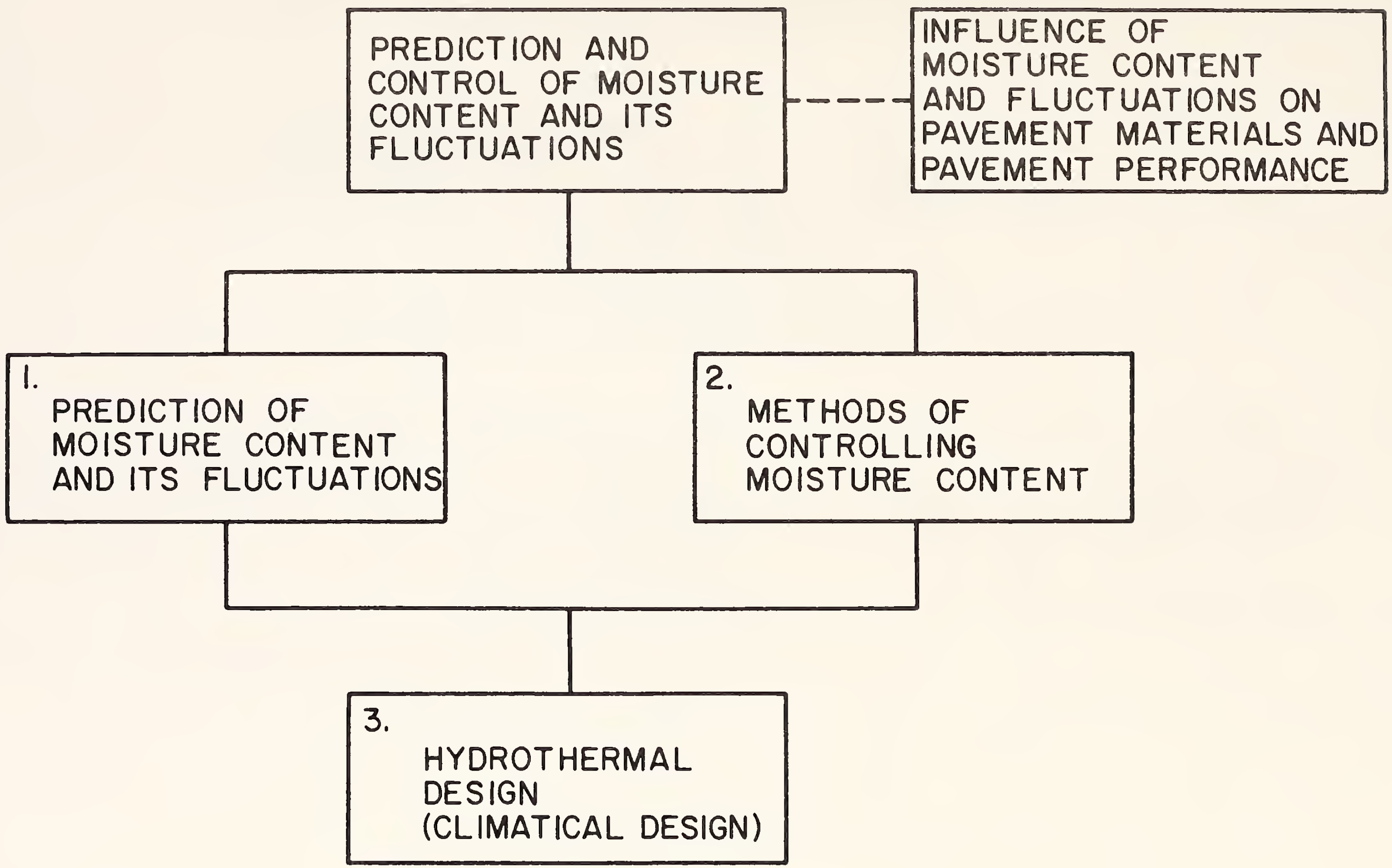

Figure 8.1. Prediction and Control of Moisture Content and Its Fluctuations.

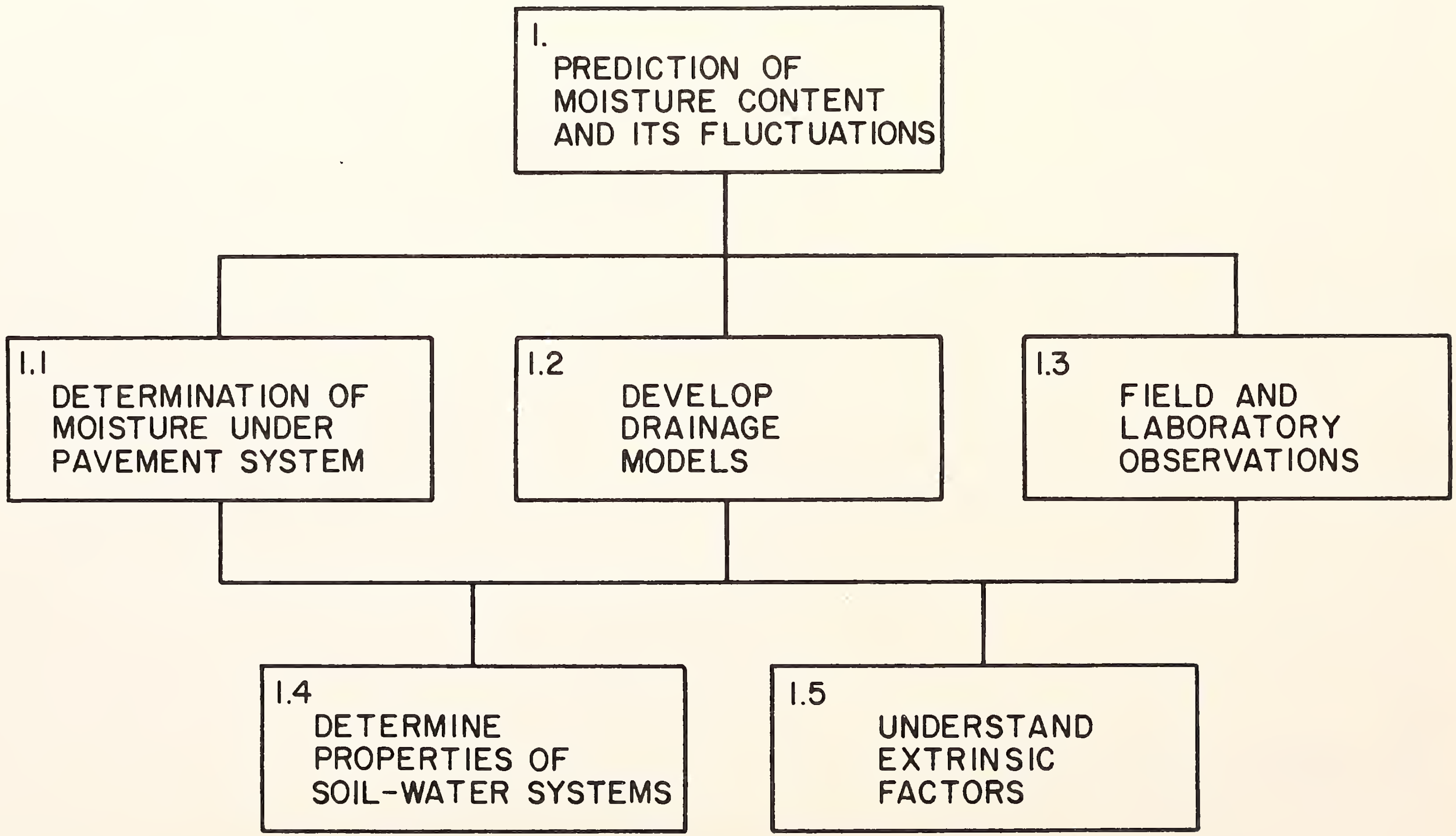

Figure 8.2. Prediction of Moisture Content and Its Fluctuations.

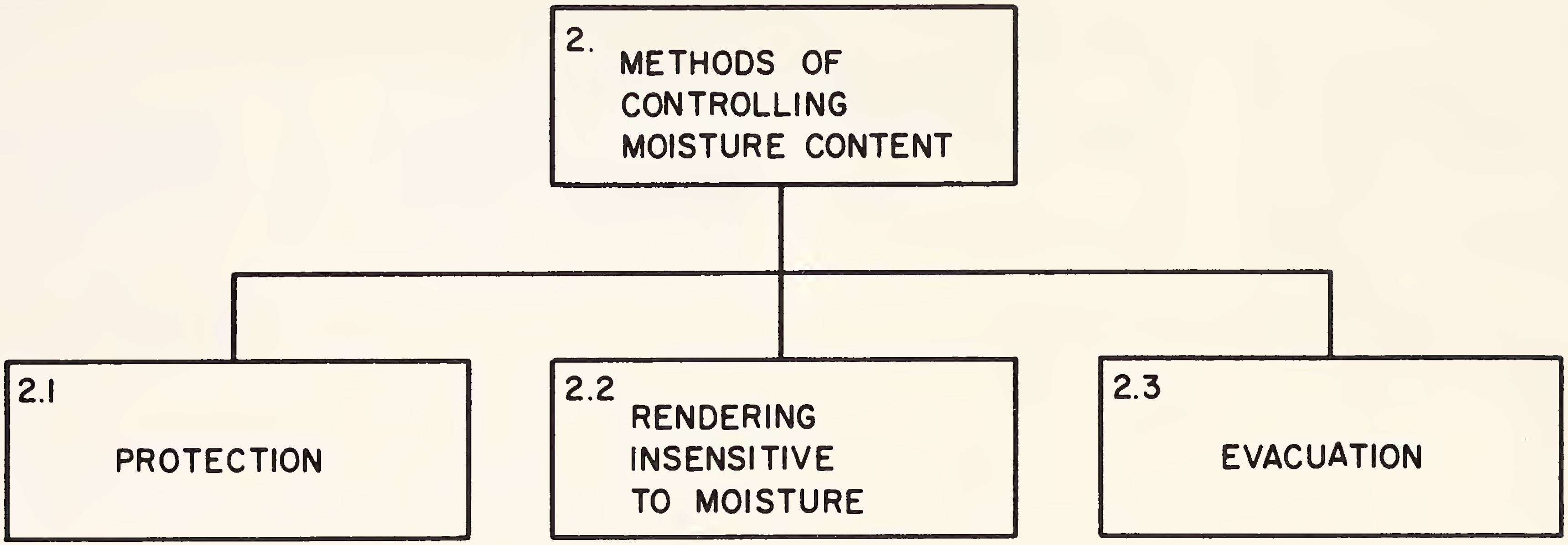

Figure 8.3. Controlling Moisture Content in Pavement Systems.

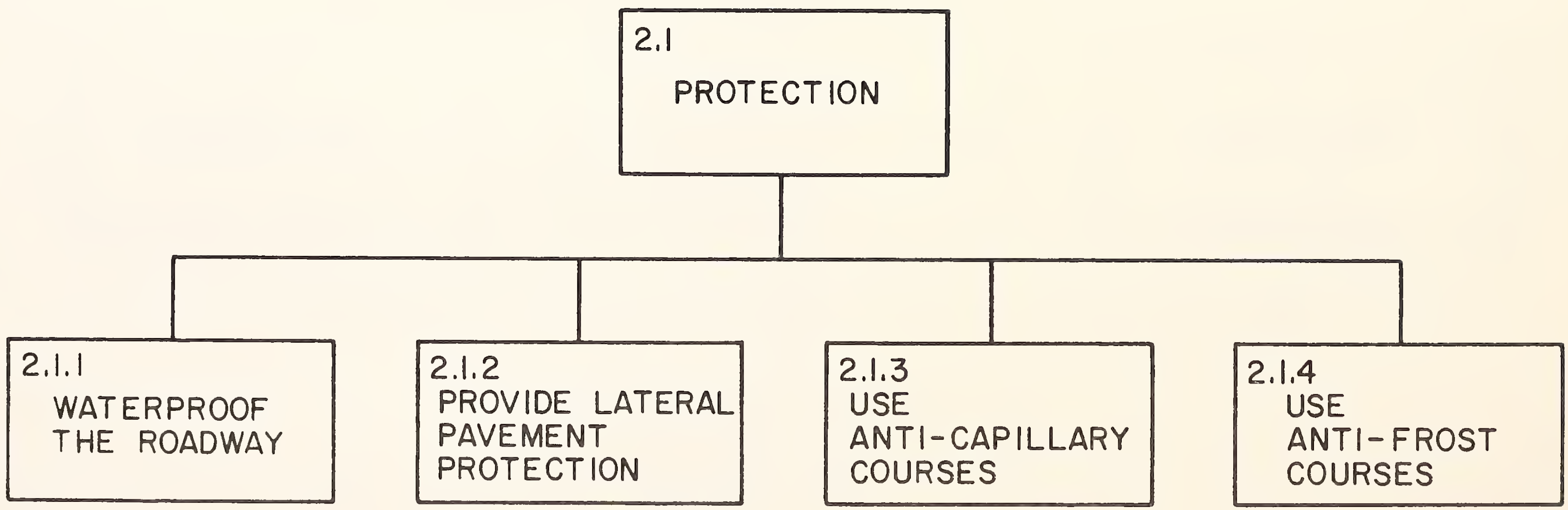

Figure 8.4. Protection of the Roadway.

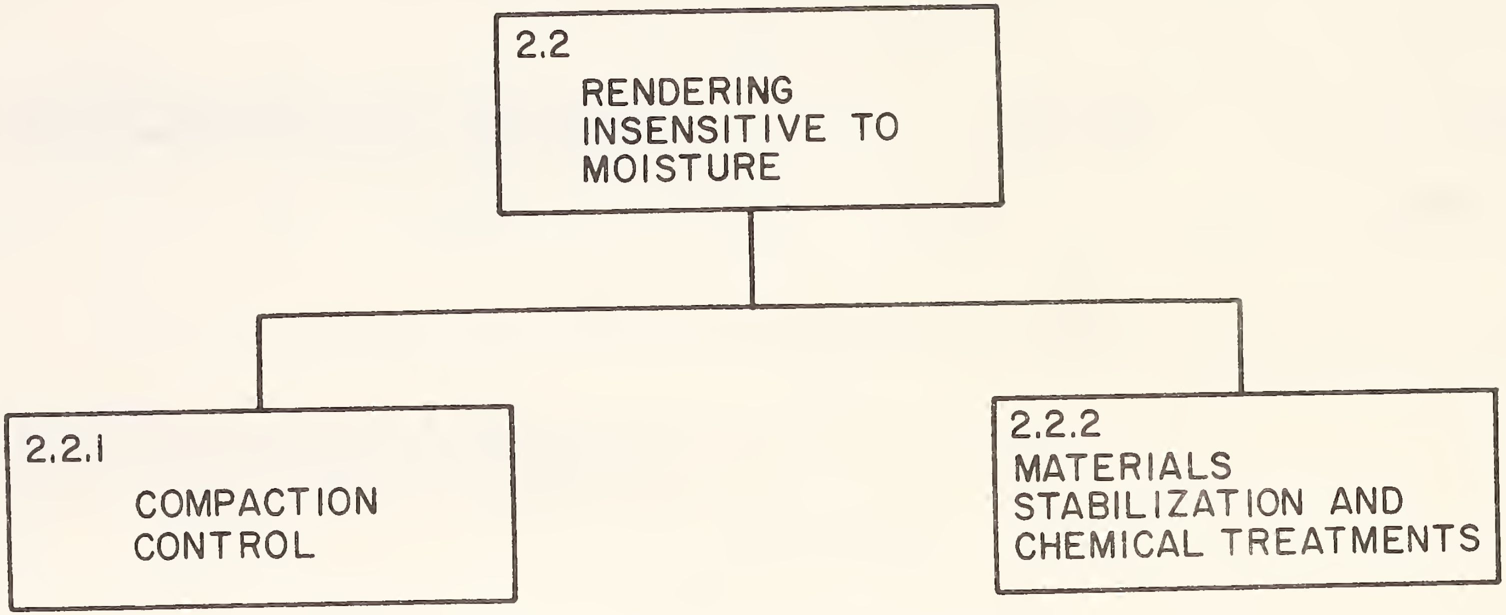

Figure 8.5. Rendering Materials Insenstive to Moisture.

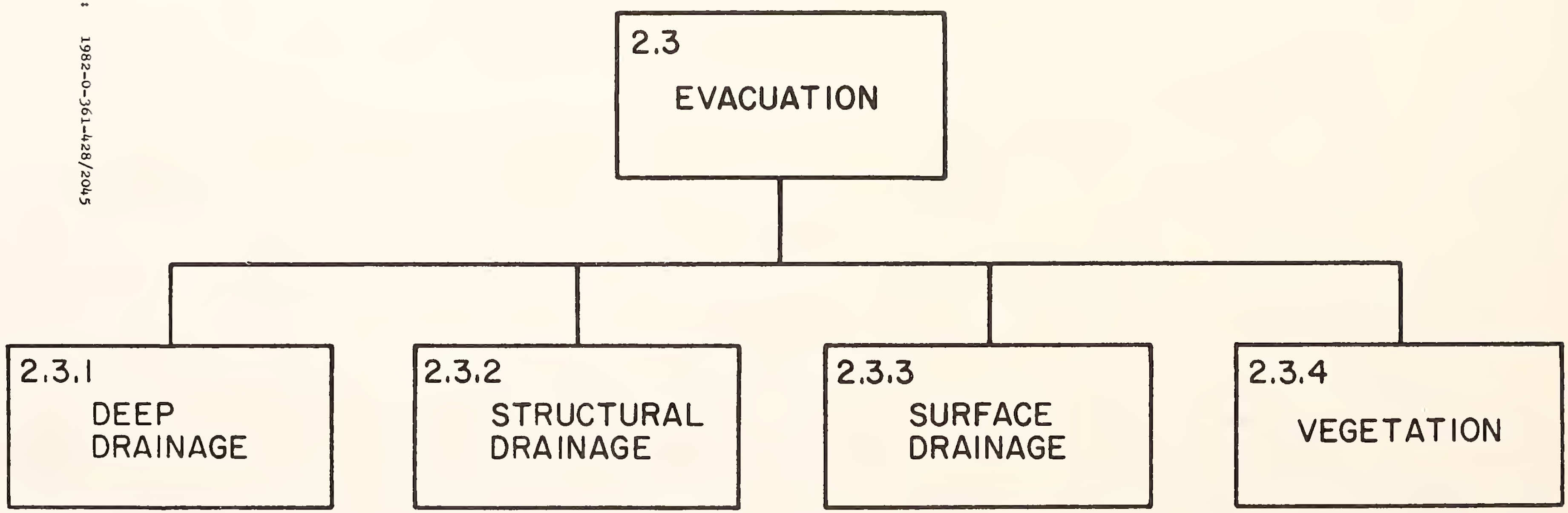

Figure 8.6. Evacuation of Water from Pavement Section.

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FEDERALLY COORDINATED PROGRAM (FCP) OF HIGHWAY RESEARCH AND DEVELOPMENT

The Offices of Research and Development (R&D) of the Federal Highway Administration (FHWA) are responsible for a broad program of staff and contract research and development and a Federal-aid program, conducted by or through the State highway transportation agencies, that includes the Highway Planning and Research (HP&R) program and the National Cooperative Highway Research Program (NCHRP) managed by the Transportation Research Board. The FCP is a carefully selected group of projects that uses research and development resources to obtain timely solutions to urgent national highway engineering problems.*

The diagonal double stripe on the cover of this report represents a highway and is color-coded to identify the FCP category that the report falls under. A red stripe is used for category 1, dark blue for category 2, light blue for category 3, brown for category 4, gray for category 5, green for categories 6 and 7, and an orange stripe identifies category 0.

FCP Category Descriptions

1. Improved Highway Design and Operation for Safety

Safety R&D addresses problems associated with the responsibilities of the FHWA under the Highway Safety Act and includes investigation of appropriate design standards, roadside hardware, signing, and physical and scientific data for the formulation of improved safety regulations.

2. Reduction of Traffic Congestion, and Improved Operational Efficiency

Traffic R&D is concerned with increasing the operational efficiency of existing highways by advancing technology, by improving designs for existing as well as new facilities, and by balancing the demand-capacity relationship through traffic management techniques such as bus and carpool preferential treatment, motorist information, and rerouting of traffic.

3. Environmental Considerations in Highway Design, Location, Construction, and Operation

Environmental R&D is directed toward identifying and evaluating highway elements that affect the quality of the human environment. The goals are reduction of adverse highway and traffic impacts, and protection and enhancement of the environment.

4. Improved Materials Utilization and Durability

Materials R&D is concerned with expanding the knowledge and technology of materials properties, using available natural materials, improving structural foundation materials, recycling highway materials, converting industrial wastes into useful highway products, developing extender or substitute materials for those in short supply, and developing more rapid and reliable testing procedures. The goals are lower highway construction costs and extended maintenance-free operation.

5. Improved Design to Reduce Costs, Extend Life Expectancy, and Insure Structural Safety

Structural R&D is concerned with furthering the latest technological advances in structural and hydraulic designs, fabrication processes, and construction techniques to provide safe, efficient highways at reasonable costs.

6. Improved Technology for Highway Construction

This category is concerned with the research, development, and implementation of highway construction technology to increase productivity, reduce energy consumption, conserve dwindling resources, and reduce costs while improving the quality and methods of construction.

7. Improved Technology for Highway Maintenance

This category addresses problems in preserving the Nation's highways and includes activities in physical maintenance, traffic services, management, and equipment. The goal is to maximize operational efficiency and safety to the traveling public while conserving resources.

0. Other New Studies

This category, not included in the seven-volume official statement of the FCP, is concerned with HP&R and NCHRP studies not specifically related to FCP projects. These studies involve R&D support of other FHWA program office research.

* The complete seven-volume official statement of the FCP is available from the National Technical Information Service, Springfield, Va. 22161. Single copies of the introductory volume are available without charge from Program Analysis (HRD-3), Offices of Research and Development, Federal Highway Administration, Washington, D.C. 20590.