JOINT HIGHWAY RESEARCH PROJECT

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RIGID PAVEMENT PUMPING:

(1) SUBBASE EROSION

(2) ECONOMIC MODELING

INFORMATIONAL REPORT

Adriaan J. Van Wijk

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Informational Report

RIGID PAVEMENT PUMPING: (1) SUBBASE EROSION AND
(2) ECONOMIC MODELING

TO: H. L. Michael, Director
Joint Highway Research Project

May 16, 1985

FROM: C. W. Lovell, Research Engineer
Joint Highway Research Project

File: 5-10

The attached report is an informational one, for research funded outside the JHRP structure.

The subject is rigid pavement pumping, and significant findings are reported in two research areas, viz., (1) the erosion of subbases, leading to pumping, and (2) the development of an economic model to predict the effectiveness of various rehabilitation techniques for pumping damages.

The information contained should be of significant value to the IDOH, since it is now possible to identify and design nonerodibility subbases. In addition, the economic model, PEARDARP, may be used to predict the costs and effects of various design and maintenance alternatives in the preventive and control of pumping damage.

Respectfully submitted,

C. W. Lovell / 5/27

C. W. Lovell
Research Engineer

CWL/tb

cc: A. G. Altschaeffl	G. K. Hallock	C. F. Scholer
J. M. Bell	J. P. Isenbarger	R. M. Shanteau
W. F. Chen	J. F. McLaughlin	K. C. Sinha
W. L. Dolch	R. D. Miles	J. R. Skinner
R. L. Eskew	P. L. Owens	C. A. Venable
J. D. Fricker	B. K. Partridge	L. E. Wood
W. H. Goetz	G. T. Satterly	S. R. Yoder

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School of Civil Engineering
Purdue University
West Lafayette, Indiana 47907

PART ONE
SUBBASE EROSION AND PUMPING

of the erodibility of materials used in rigid pavements. Also included in this part is a review of pumping literature and design procedures to prevent pumping. The second part, Chapters 9 through 13, addresses the economic evaluation of designs and rehabilitation techniques to prevent pumping.

materials in rigid pavements. Factors which influence the performance of these materials, e.g. the environment, were also incorporated in the testing program. Testing procedures were developed for all three testing devices. The materials types identified were tested using the erodibility testing devices, incorporating the materials properties and environmental factors in the testing procedure. The results of the erodibility testing are contained in Chapters 6 and 7. Chapter 8 summarizes all the aspects involved in the design to prevent pumping.

4. Economic analysis: All the rehabilitation techniques related to pumping were identified, their influence on correcting pumping related distresses discussed, and typical unit costs for these activities obtained. Available economic analysis systems were reviewed and the most appropriate one was selected. An existing pumping prediction model was also modified to include the important pavement and environmental aspects. An economic analysis model was developed including the construction cost, rehabilitation costs, and user's consequences. Chapters 9 to 12 address the these aspects of the economic analysis.

The research results presented in this report can be divided into two parts. The first part, Chapters 2 through 8, addresses the evaluation and characterization

1.3 Scope of Study

The research reported in this report is part of a project on "Design to prevent pumping". The four aspects of rigid pavement pumping were covered in this report.

1. Literature study: A literature search was conducted to obtain all relevant information regarding previous pumping studies, design to prevent pumping, and erodibility testing. A questionnaire was also sent to all highway agencies in the United States to obtain information regarding rigid pavement subbase performances, rigid pavement rehabilitation techniques applied and their results, and designs used to prevent pumping. Results of the literature review and survey are presented in Chapters 2 and 3.
2. Development of a simple erosion testing procedure: Three erosion testing procedures were selected after a thorough literature review. Two of the testing devices were built and calibrated for the study. Chapters 4 and 5 describe the selection and development of the erosion testing devices.
3. Erodibility Testing: First, materials to be tested were identified. The materials first were grouped into unstabilized and stabilized categories. The material types and properties were chosen to cover all the materials usually used as subbase or shoulder

loads. If the pore water pressures can not be dissipated rapidly enough and become sufficiently large, some of the fines from within the material are removed and pumped out. This adversely changes the slab support conditions, which can cause slab cracking.

Another pumping mechanism is the removal of fines from the surface of, especially, stabilized materials. In this process water is accumulated under curled slabs at joints. With the deflection of the approach slab the water is pushed towards the leave slab. The leave slab is then deflected rapidly by the moving wheel load and the water is pushed back under the leave slab at a high velocity. Fines are then removed from under the leave slab and deposited under the approach slab. This causes the formation of voids and produces faulting. Voids change the slab support conditions from uniformly supported to unsupported at some points. Similarly, unstabilized subbases are also subjected to surface erosion, along with pore water pressure build-up that leads to pumping. Material can be removed from shoulders, mainly at the slab-shoulder interface, by surface erosion and either ejected through pavement cracks and joints or deposited under the slabs. This may cause shoulder depressions, faulting, and the intrusion of incompressible materials into the joints.

currently widely used in both the United States and in Europe. However, even the stabilized layers have not eliminated the original pumping problem, since fines are still being eroded by water trapped on the surface of some of these layers. The open-graded layers have been successful in the prevention of pumping, if designed and constructed properly, but they require more controlled material specifications and more stringent construction supervision.

Rigid pavement pumping is defined as (a) the ejection of water and subgrade, subbase or shoulder material through pavement joints, cracks and edges, or (b) the redistribution of material underneath the slab. The major cause of material removal from the layer can be pore water pressure buildup or surface erosion. Fines are removed from stabilized layers and unstabilized shoulders mainly by surface erosion. Three components are necessary for pumping to occur, viz., high slab deflections (heavy wheel loads and/or thin slabs), water in the pavement, and materials that are susceptible to pumping.

In the pumping process, water infiltrates the pavement from the surface, after which the water saturates the subgrade and subbase, if it is not removed rapidly. The deflection of the slab at the slab edges, joints and cracks increase the pore water pressure in the underlying material. These deflections are repeated by passing wheel

The detrimental effects of pumping have been recognized since the 1930's. All the pavements that failed during the AASHO Road Test in the early 1960's showed symptoms of pumping prior to failure. Various experimental road sections and model studies have been used to study the pumping problem since the 1940's. Most of these studies investigated the removal of fines from within the subgrade or granular subbase material, as opposed to removal from the surface of stabilized layers. To date, only researchers in France and California have investigated the surface erosion or abrasion of stabilized and lean concrete subbases. Pervious or open-graded layers have been studied more extensively during the last decade, in keeping with an increased emphasis on drainage.

1.2 Problem Statement

Most of the rigid pavements built in the United States before the 1940's were placed directly on the subgrade. The increase in heavy trucks during World War II caused serious pumping of the subgrade material. These events accelerated the use of granular subbases between concrete and subgrade. Although these layers reduced pumping, subbase material could still be removed in a process termed "blowing". This occurrence led in turn to the use of impervious stabilized layers, and, more recently, pervious or open-graded layers. Such layers are

CHAPTER 1

INTRODUCTION

1.1 Background

Rigid pavement pumping has been a topic of research for many years, but it still remains one of the major contributors to rigid pavement failure. Although pumping is considered a major problem in the performance of rigid pavements, it has not been included explicitly in the analysis and design procedures. Pumping produces voids under the slab, as well as faulting. The designs concentrate on avoiding fatigue due to stresses and deformations of fully supported slabs, and give secondary attention to the effects of environmental factors and pumping. The recently updated PCA design procedure is the only one to include an explicit erodibility criterion. However, this criterion merely distinguishes between stabilized and unstabilized materials, and does not deal with differences in the stabilized layers. Erodibility has also been included as a variable in a few rigid pavement distress prediction models.

modified to include factors such as, drainage, climate,
and subbase type.

subbase materials, and tests used to simulate pumping. Three testing procedures were selected to be used to investigate and characterize the erosion of rigid pavement subbase and shoulder materials, viz., a brush test, a jetting test, and a rotational shear technique. The latter was designed and built especially for testing of subbase materials, and is an improvement on earlier models used by other researchers. Portland cement stabilized, asphalt stabilized, lean concrete, and unstabilized materials were tested in an statistically designed program. The rotational shear device gave the most useful results for stabilized materials. However, this device can not be used to test cohesionless materials. An effort was made to correlate the laboratory erosion results with the performance of the materials in the pavement. Guidelines are provided on the required material properties to minimize pumping due to surface erosion of subbase and shoulder materials.

The feasibility of design and rehabilitation alternatives requires an economic analysis of the alternatives. A computer program (PEARDARP) was developed to evaluate the effect of rehabilitation and design alternatives on rigid pavement pumping and performance. The effects of different rehabilitation techniques on pavement distresses were quantified for inclusion in the program. An existing pumping prediction model was

ABSTRACT

Pumping of rigid pavements is a major contributor to rigid pavement failure. Fines can be removed through pore water pressure buildup in the subbase or through surface erosion of subbase and shoulder materials. A number of studies have been conducted since the 1940s, and a number of remedies have been implemented, viz., granular, stabilized, open-graded, and lean concrete subbases. The effectiveness of these measures varied. Stabilized layers, although reducing pumping, do not eliminate pumping. Pumping in stabilized layers is caused primarily by surface erosion of these layers. The erosion of stabilized layers used in pavements and subject to service conditions in the pavement, have not been studied extensively.

An extensive literature review was conducted to obtain information regarding pumping, with emphasis on designs to prevent pumping, the performance of different

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Informational Report

RIGID PAVEMENT PUMPING:
(1) SUBBASE EROSION AND
(2) ECONOMIC MODELING

By

Adriaan J. Van Wijk
Graduate Assistant in Research

Joint Highway Research Project

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The contents of this report reflect the views of the authors who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views of policies of the Federal Highway Administration. This report does not constitute a standard, specification or regulation.

Purdue University
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CHAPTER 5

DEVELOPMENT OF THE TESTING PROCEDURES

5.1 Introduction

Three tests were used to determine the erodibility of pavement subbase and shoulder materials, viz., rotational shear, jetting, and brushing tests. The rotational shear is the most sophisticated and had to be designed and built for use in the study. In this test, the forces which cause erosion can be controlled and measured accurately. The forces causing erosion in the jetting test could be controlled accurately, but not accurately measured. In the AASHTO brushing test the forces on the sample can neither be applied nor measured accurately. A pressure vessel and a sample container had to be built for the jetting device, while no special equipment was needed for the brushing test.

5.2 Brushing Test

The brush and brushing procedure described in AASHTO T135-76 and T136-76 [AASHTO1970] were used in the simulation of the erosion of subbase and shoulder

materials. The brush consisted of 50 groups of 51 by 1.6 mm (2 by 1/16 in.) flat no 26 bristles placed in 5 rows of 10 groups each on a 191 by 64 mm (7.5 by 2.5 in.) hardwood block. Each group contained 10 bristles. Each sample was brushed 20 times on the sides, covering the sample twice, and 4 strokes on each end. The force of the brush on the sample is defined to be 13.31 N (3 lbf). The force along the sample surface was measured with the same low friction rail system used for the calibration of the jetting device. Assuming a contact area of 0.0045 sq m (6.9 sq in.), the average shear stress per stroke was determined to be 2.75 kPa (0.4 psi).

5.3 Rotational Shear Device

5.3.1 Description and Historical Development

The testing device consists of a transparent cylinder that is rotated around a stationary soil sample, with the annular space filled with water. The water induces a shear stress on the soil sample. If the shear stress is high enough, erosion will take place. The rotational velocity of the cylinder, and thereby the shear stress of the water on the sample, can be varied.

The rotating cylinder test apparatus was developed during the early 1960's by Moore, Masch and Espey [Moore1962, Espey1963] at the University of Texas at Austin. They wished to eliminate variability of the shear

stress in available erodibility tests (like the jetting tests). The new testing device induced a constant shear on the soil sample. It was first used to investigate the erodibility of cohesive materials. Arulanandan et.al. [Arulanandan1973] later also used it to study the relationship between the chemical properties of clays and erosion. Akky and Shen [Akky1973,Akky1974] used the same testing device during the early 1970's to investigate the erodibility of cement-treated materials. They used the above-mentioned apparatus because of its simplicity, the ease with which the properties of the sample can be controlled, and the accuracy with which the hydraulic critical shear stress could be measured. The apparatus can give a measure of the erosion rate and the shear stress independent of such uncertainties as roughness changes and boundary layer growth during testing. Chapuis [Chapuis1983] used the test during the late 1970's and early 1980's to measure the erodibility of different soil types.

5.3.2 Hydraulic Principles

The behavior of the fluid (water) in the annulus is similar to the behavior of water in a viscosimeter. Laminar flow is assumed. The water velocity on the outside is the same as the angular speed of the rotating cylinder and zero on the surface of the sample. The equation relating the shear stress to the geometry is:

$$\tau = 2\mu \left(\frac{R^2}{R^2 - r^2} \right) \omega$$

and

$$T = 2\pi r^2 l \tau$$

where τ = shear stress

μ = dynamic viscosity (0.00002050 lb sec/sq ft or
0.001 N sec/sq m for water at 70 degrees F)

T = torque

R = outer cylinder radius

r = inner cylinder radius

l = length of sample

ω = angular velocity of rotation

The shear stress increases as the speed increases. At the critical rotational speed the flow conditions change from stable Couette flow to unstable flow. The equation relating torque to the geometry is only valid in the stable Couette flow region. The flow changes from laminar (stable) to turbulent (unstable) flow when the critical Reynolds number (or critical rotational speed) is exceeded. These critical values depend on the geometry of the device and the surface roughness of the sample. Rotational speeds at which the flow changes range from 200 rpm [Chapuis] to 800 rpm [Akky1974]. The boundary shear at turbulent flow can be expressed as:

$$\tau_b = \phi N^\alpha$$

where τ_b = boundary shear stress

ϕ = constant of proportionality

N = speed

α = constant (Espey - 2.75 and

Taylor - 3.00)

The constants ϕ and α have to be determined experimentally.

Schlichting [Espey1961] developed a relationship correlating the critical Reynolds number (CRN) with the ratio between the annular space (b) and the radius of the outside cylinder (R) for flow in the annulus. The Reynolds number is defined as [Espey1961]:

$$UR/\nu$$

where

U = peripheral velocity of the outer cylinder

R = outer cylinder radius

ν = kinematic viscosity (= 0.0000106 sq ft/sec

or 0.000001 sq m/sec for water at 20 degrees C)

5.3.3 Apparatus

5.3.3.1 University of Texas at Austin study: The initial test developed by Moore, Masch and Espey had the following features. A cylinder of cohesive soil 76 mm (3 in.) in diameter and 102 mm (4 in.) long was mounted coaxially inside a slightly larger transparent cylinder, which could

slightly larger transparent cylinder, which could be rotated at any desired speed up to 2500 rpm. The annular space between the cylindrical soil sample and the rotating cylinder was filled with a fluid to transmit shear from the rotating cylinder to the surface of the soil sample. Thus the stress was uniform at all points around the surface of the soil sample. The soil sample was stationary, but was mounted on flexure pivots so that the shear stress transmitted to its surface resulted in a slight rotation of the supporting tube. This rotation in turn was calibrated to measure the magnitude of the torque on the sample, and thereby, the shear on the surface area of the soil sample. To minimize the variation in shear stress at the end of the cylinder, end pieces (also 76 mm in diameter) were mounted immediately above and below the soil sample. The end pieces were mounted independently of the sample so that the torque applied to their surfaces would not contribute to the measured torque for the soil sample.

A water-glycerin mixture was first used as a scouring agent, but reacted with the soil sample, and the glycerin was later omitted. The flexure plates in the apparatus were also replaced by bearings during further development. Two outside cylinders were used, with 41 and 53 mm (1.625 and 2.10 in.) radii, respectively.

The authors calculated a critical speed of 344 rpm from the Schlichting relation, and a critical speed of 680 rpm from the shear stress-speed relations for the testing device with an annular gap of 3 mm (0.125 in.). This indicated that a larger range of stable flow occurs in the testing device. This was attributed to better concentricity and the inertial forces operating in the radial direction. Fluid particles near the outer boundary were kept from moving radially inward by centrifugal forces, whereas those particles near the inner boundary do not move outward because of smaller centrifugal forces. It was concluded that for larger annular spaces and this concentric device, the turbulence level at the soil surface should be relatively low, and the instantaneous shear stress fluctuations on the surface relatively small. With the end pieces to eliminate abrupt changes in shear stress, the mean shear should be very stable.

5.3.3.2 University of California at Davis studies: The investigators [Espeyl963] slightly modified the apparatus used at Texas. The sample was mounted on a combination radial and thrust bearing so that the shear stress transmitted to its surface resulted in a slight rotation of the supporting tube. Provisions were also made to test different size samples, viz., 76 mm (3 in.) in diameter and 88 mm (3.45 in.) high and 102 mm (4 in.) in diameter and 117 mm (4.6 in.) high. The annular space was 15 mm

(0.60 in.) in both cases. A 186 W (1/4 hp) Bodine motor with a Bodine variable speed control box was used to drive the outer cylinder at speeds of up to 2400 rpm.

Arulanandan et.al. [Arulanandan1973] used the same device to investigate the erodibility of clay samples.

5.3.3.3 Mon-ter-val study: Chapuis [Chapuis1983] further modified the apparatus used by Shen and Akky. These modifications include the removal of the shaft that held the sample in place, the fixing of the lower end plate, and the measurement of the torque by means of a string-and-pulley system rather than the rotation of a brass rod. The major advantages of these modifications were that the shear stress could be measured more accurately and intact samples could be used. Provisions were also made for the removal of the eroded material by vacuum suction after each test. The device was made of stainless steel and driven by a 560 W (0.75 hp) electric motor to speeds of up to 1750 rpm. The annular space was 0.5 inches (13 mm).

5.3.4 Sample Size and Preparation

5.3.4.1 University of Texas at Austin study: The 76 mm (3 in.) diameter cohesive soil samples used by Espey [1963] were prepared by extruding samples with a Vac-Aire extruding machine under a vacuum. The soil was extruded directly into the outer brass mold which contained the

torque tube. The outer mold was then removed with the sample completely submerged to prevent the sample from adhering to the outer brass mold as it was removed.

5.3.4.2 University of California at Davis studies: The method used by Akky [1974] for cement stabilized materials was less complicated. The soil and cement were mixed and compacted in a standard AASHTO compaction mold of 102 mm (4 in.) in diameter and 117 mm (4.6 in.) high using the procedure described in AASHTO T134-70. A 18 mm (0.75 in.) hole was drilled axially along the length of the sample. All samples were cured in a moisture room for 7 days. The sample was then fastened to the supporting rod between the two end plates and immersed in a beaker of tap water for 2 hours, after which it was drained and the height measured. Some samples were subjected to wet-dry or freeze-thaw cycles before the 2 hour soaking period. The procedures described in the Freezing-Thawing Test (ASTM D560 or AASHTO T136) and Wetting-Drying Test (ASTM D559 or AASHTO T135) were used to simulate the environmental effects.

Arulanandan et. al. [1973] used cylindrical specimens with a diameter of 76 mm (3 in.) and a height of 102 mm (4 in.). The clay samples were prepared by consolidation from slurries.

5.3.4.3 Mon-ter-val study: Both remolded and intact samples were tested. The samples were 76 mm (3 in.) in

diameter and 89 mm (3.5 in.) high. The intact samples were kept at field moisture content until testing.

5.3.5 Testing Procedure

5.3.5.1 University of Texas at Austin study: Espey [1963] used two methods to determine the critical shear stress of the cohesive material:

1. The shear stress was kept a constant for a minute, after which the change in weight was determined. A higher shear stress was applied for another minute, and the weight was again determined. This procedure was repeated until the critical shear stress was reached. The critical shear stress was indicated by a sharp change in slope of weight loss per time against shear stress plot.

The researcher suggested that the shear stress increments should be between 7 and 24 Pa (0.15 and 0.5 psf), with the increment becoming progressively smaller as the critical condition was approached.

2. The shear stress was increased over an one to two minute loading period and the critical scour condition visually observed, as indicated by a sudden excessive movement of the pointer.

5.3.5.2 University of California at Davis studies: Akky [1974] conducted two types of erodibility tests on cement-stabilized samples:

1. Erosion without environmental treatment. In this test the shear loading was set at a preselected value and the sample allowed to erode. The time varied from a few minutes to an hour, depending on the erodibility of the material. The sample was then removed, weighed and the weight loss per surface area determined. This procedure was repeated on the same sample at the same shear stress at least three times. The weight loss per surface area against time could then be plotted. The process was repeated on the same sample at higher shear stresses to develop a family of curves. The intercept on the shear stress axis, corresponding to a zero erosion rate, was defined as the critical shear stress necessary to initiate important surface erosion.
2. Erosion with environmental treatments. The samples were subjected to different wet-dry or freeze-thaw cycles and eroded at preselected shear stresses for 2 minutes. The time period of 2 minutes was found to be sufficient to cause the erodible surface material to separate from the rest of the sample.

Arulanandan et al. [1973] used the same method to

determine the shear stress required for a zero erosion rate, which is the intercept on the applied shear axis.

5.3.5.3 Mon-ter-val study: The sample was placed in the device and the annular space filled with distilled water. The outside cylinder was then rotated at a selected speed for 10 minutes. After 10 minutes the eroded material was removed, dried and weighed. The shear stress was measured after 3 and 10 minutes. The process was repeated, each time at a higher rotational speed until the water became very cloudy or the shear stress increased rapidly. The torque was plotted against the rotational speed, to obtain the internal friction of the device. The measured shear stress minus the internal friction was then plotted against the erosion rate. The critical (threshold) shear stress was defined as the point where the slope of the curve changed rapidly. It was found that better results were obtained when the sample was eroded at a few low shear stresses (to remove loose material) before the actual results were recorded [Chapuis1983].

5.3.6 Materials Tested

A wide variety of materials have been tested in the rotational device. The device was initially used to study the erosion of a medium plastic soil called Taylor Marl [Espey1963]. The material had a maximum size of 1 mm (0.04 in.) and a PI of 26. Arulanandan et al. [1973]

tested a soil called Yolo loam, with a maximum size of 2 mm (0.08 in.) and a PI of 9. Chapuis [1983] tested three soils with Unified classifications of CL, CH and CL-ML, respectively. All three soils had more than 80% of the particles passing the 0.074 mm (No.200) sieve and plasticity indices ranging from 7 to 23. Akky and Shen [Akky1973,Akky1974] investigated the erosion of three cement stabilized soils, viz., Castaic, Quail and Yolo loam soils. The soils had AASHTO classifications of A-1-b, A-2-4 and A-4, respectively. The maximum size of the particles ranged from 2 mm (0.08 in.) to 76 mm (3 in.) and the plasticity indices from 0 to 14. The cement content was varied from 0 to 7 percent and the number of freeze-thaw cycles the samples were subjected to ranged from 0 to 15.

5.3.7 Modification and Calibration of the Rotational Device

Since the rotational shear device was built for the study, the dimensions of the device had to be determined before the fabrication started. One of the objectives of the study was to develop a testing procedure that is easy to use. It was therefore beneficial to use samples that can be prepared in standard equipment. Cement (AASHTO T-134), asphalt (AASHTO T-245 and T247) and lime stabilized, granular (AASHTO T-99), and lean concrete (AASHTO T-23) samples are usually compacted in molds with diameters of

102 mm (4 in.). The sample heights vary from 63.5 mm (2.5 in.) for asphalt stabilized, to 117 mm (4.6 in.) for cement stabilized, to 204 mm (8 in.) for lean concrete samples. The 102 mm (4 in.) diameter and 117 mm (4.586 in.) mold is used for the widest range of materials. The rotational shear device was therefore designed to accommodate samples compacted in this size.

With the sample size selected, the annular space had to be determined. As was shown earlier, the shear stress on the sample is a function of the sample and outer cylinder radii, the angular velocity of the outside cylinder and the water viscosity. Annular spacings ranging from 3 mm (0.125 in.) to 15 mm (0.6 in.) have been used, as mentioned. The smaller the annular space, the larger the shear stress on the sample for the same rotational speed. Most commercially available air or electrical motors of less than 750 W (1 hp) have maximum speeds ranging from 2000 to 3000 rpm. The maximum shear stresses measured in the cited studies ranged from 2.3 Pa [Akky1974] to 35 Pa [Chapuis1983] for a wide range of materials. The equations described in Section 5.2 were used to select annular spaces with which such shear stresses could be obtained. Another consideration in the selection of the annular space was that the annular space should be large enough for the eroded material to be able to move freely in the device. The device was designed to

have 3 annular spacings, viz., 9.5 mm (0.375 in.), 13 mm (0.5 in.), and 16 mm (0.625 in.), by using cylinders with different inside diameters. The maximum size of the aggregate was therefore set at 9.5 mm (0.375 in.) to allow for the free movement of the eroded material at all times.

An air motor of 560 W (0.75 hp) was used to rotate the transparent cylinder at rotational speeds of between 300 and 3000 rpm. A strobe was used to measure the rotational speeds.

The shaft through the sample was eliminated. The sample was held in place by 4, 13 mm (0.5 in.) long tubes attached to a thin metal cap and penetrating into the sample. Epoxy was also used to secure samples to a smooth cap where the cap with the tubes could not be used. The sample rested on the bottom cap. Tape was used to protect the ends of the sample by placing it around the top and bottom ends of the sample covering the caps. The tape prevented water from entering the space between the caps and the sample ends. The top cap was connected to a shaft which transferred the rotation due to shear stress on the sample to a lever arm which pressed against a load cell. The load cell replaced the string-and-pulley system used by Chapuis. The load cell was connected to a recording device.

The amount of erosion was measured by recording the weight of the eroded material rather than weighing the sample after each test. This procedure was more cumbersome, but avoided inaccuracies produced by some degree of saturation during the test and prevented the sample from being disturbed. The base plate was modified to provide for easy removal of the eroded material. Figure 5.1 shows a schematic diagram of the rotational shear device, while the pictures in Figures 5.2 and 5.3 illustrate the different elements of the device.

A number of tests were conducted to calibrate the equipment. For laminar flows on smooth surfaced samples the shear stresses can be calculated from the rotational speeds. However, this condition exists only at very low rotational speeds. The flow of the water in the annular space changes to turbulent flow when the critical Reynolds number (CRN) is reached. The speed at which the CRN is reached depends on the surface roughness of the sample and the annular space. Figure 5.4 shows that the rotational speeds at which the flows change from laminar to turbulent are 680 and 820 rpm for annular spaces of 9.52 and 12.5 mm (0.375 and 0.5 in.), respectively. For a large number of readings on different samples the CRN was reached at about 680 rpm for an annular space of 9.52 mm (0.375 in.) (Figure 5.5). The rate of increase in the shear stress with rotational speed during turbulent flow depends also

Figure 5.1 Schematic Diagram of Rotational Shear Device

Figure 5.2 Picture of Elements of the Rotational Shear Device

Figure 5.3 Picture of the Mounted Rotational Shear Device

Figure 5.4 Effect of Annular Space and Rotational Speed on Shear Stress

Figure 5.5 Effect of Rotational Speed on Shear Stress

on the annular space and the sample surface roughness. Figure 5.4 depicts the effect of annular space on the increase in shear stress with rotational speed. The smaller the annular space, the higher the shear stress on the sample surface for the same rotational speed.

The exponential curve in Figure 5.5 shows that the shear stress produced by the internal friction of the rotational shear device was about 1.6 Pa (0.33 psf) for a large number of samples. Figure 5.6 portrays the relationship between the air pressure flowing to the air motor and the rotational speed. This relationship is unique for this combination of air motor and rotational shear device.

5.4 Jetting Test

5.4.1 Usage of Jetting Tests

As mentioned in the previous chapter, jetting tests have been used extensively in the investigation of the erodibility of materials. Jets of different sizes and placed at different angles have been used. Dash [1968] and Bhasin [1969] for example, used a vertical submerged jet with an orifice of 3.2 or 4.8 mm (0.125 or 0.188 in.) to erode stationary samples. Both varied the water velocities by changing the water level in a head tank above the jet. Nussbaum and Colley [Nussbaum1971] used a jet with a diameter of 3.2 mm (0.125 in.) at a water

Figure 5.6 Air Pressure versus Rotational Speed

pressure of 186 kPa (27 psi) to erode cement stabilized samples for use as erosion protection layers in dams. Pnu and Ray [Pnu1979] used a jet of 1 mm (0.02 in.) at an angle of 45 degrees with the horizontal to erode saturated samples in air.

5.4.2 Jetting Device

The jetting device, shown in Figure 5.7, was developed to characterize the erosion of unstabilized stabilized materials. The device consisted of a jet placed at an angle of about 20 degrees with the sample. Pressures of up to 345 kPa (50 psi) were provided by a pressure vessel. The sample was placed in a plexiglas container with water outlets at two different levels. Both the pressure vessel and the plexiglas container were built for the study. The erosion of samples could be measured in the submerged or unsubmerged conditions by changing the water outlet level in the sample container. Samples could also be tested in or out of the molds. Eight spray nozzles with different orifices and spray angles were used. Table 5.1 summarizes some of the properties of these nozzles. Each induced a different water velocity or shear stress on the sample.

The water velocity at the jet was determined by measuring the flow and dividing it by the effective orifice diameter. It was also calculated using Bernoulli's

Figure 5.7 Diagram of the Jetting Device

Table 5.1 Nozzle Properties

Jet ref. no.	Effective diameter d (mm)	Shape	Max. spray angle (degrees)	x/d	U/U _m	F/F _m	K _c
1515	2.38	sq	15	21	0.35	0.47	0.20
2515	2.38	sq	25	21	0.35	0.47	0.20
1560	4.76	sq	15	11	0.65	0.37	0.37
2560	4.76	sq	25	11	0.65	0.37	0.37
0002	0.99	r	0	50	0.15	0.08	0.495
0005	1.55	r	0	33	0.20	0.12	0.49
0010	2.18	r	0	23	0.35	0.18	0.48
0020	3.18	r	0	16	0.40	0.26	0.445

sq = square

r = round

x = 50 mm (2 in.)

U_m = maximum water velocity (at the nozzle)

U = water velocity at x (on the sample)

U/U_m obtained from McGuirk1977.F_m = maximum force (at the nozzle)

F = force at x (on the sample)

F/F_m obtained from Basin1969.

25.4 mm = 1 in.

energy equation. The equation used was [Roberson1965]:

$$V_2 (1 + h_1) = 2 * p / \rho$$

where

V = water velocity at the nozzle

h_1 = energy loss due to 2 elbows, 1 valve and
the friction in the pipe = 1.1

p = tank pressure

ρ = density of water (997 kg/m³ or
1.94 slugs/ft³)

The actual water velocity on the sample is lower than the velocity at the jet. The distance of first contact between the water and the sample was about 51 mm (2 in.). Using centerline velocity decay relationships developed by McQuirk and Rodi [McQuirk1977] the approximate water velocity on the sample could be determined.

Attempts to measure the force of the water on the sample with the use of a low friction rail were unsuccessful. The forces on the sample were calculated from the calculated water velocities and the effective areas of the jets. The following equation was used [Roberson1965]:

$$F = (V^2 / 2 \gamma (1 + K_c)) A_n$$

where

F = water force

V = water velocity at nozzle

K_c = energy loss (for values see Table 5.1)

A_n = nozzle area

ρ = density of water (997 kg/m^3 or
 1.94 slugs/ft^3)

Forces of the water on the sample surface were calculated using force decay relationships developed by Albertson [Bhasin1969]. Table 5.1 also contains the ratios used to calculate the water velocities and forces on the sample.

The shear stresses on the sample surface were approximated by dividing the forces at the sample by the area of contact. A uniform distribution over the area was assumed. The area of contact was visually observed as about 0.005 sq m (7.75 sq in.) and is only a rough approximation. Table 5.2 summarizes the calculated water velocities, forces, and shear stresses for a nozzle with an effective diameter of 2.38 mm (0.094 in.) and a spray angle of 15 degrees. Figures 5.8 and 5.9 display the water velocities and shear stresses on the sample at different tank pressures, respectively.

Table 5.2 Water Velocities and Shear Stresses

p_t	V_{nc}	V_{nm}	V_s	F_n	F_s	τ
25	4.9	6.0	1.7	0.07	0.014	3.0
50	6.9	8.0	2.4	0.14	0.028	5.5
75	8.5	11.0	3.0	0.22	0.042	8.5
100	9.8	13.0	3.4	0.29	0.056	11.0
150	12.0	16.0	4.2	0.43	0.084	17.0
200	13.8	18.0	4.8	0.58	0.112	22.0

p_t = tank pressure (in kPa)

V_{nc} = calculated water velocity at the nozzle (in m/s)

V_{nm} = measured water velocity at the nozzle (in m/s)

V_s = water velocity on the sample surface (in m/s)
(from V_{nc})

F_n = force at the nozzle (in N)
(V_{nc} used)

F_s = force on the sample surface (in N)

τ = shear stress on the sample = $F_n/0.005$ (in Pa)

1 kPa = 6.9 psi

1 m/s = 3.28 ft/s

1 N = 0.2248 lb

Figure 5.8 Water Velocity against Air Pressure

Figure 5.9 Shear Stress against Air Pressure

CHAPTER 6
MATERIALS, EXPERIMENTAL DESIGN, AND
EXPERIMENT PROCEDURES

6.1 Introduction

The purpose of the testing program was to obtain information about the erosion of rigid pavement subbase and shoulder materials. The variables included in the testing program were chosen to include the properties of most of the subbases used in rigid pavements. This information was obtained from the results of the survey and rigid pavement design procedures, viz., PCA [PCA1984] and AASHTO [AASHTO1981]. The survey results (Table 3.1) indicate that Portland cement stabilized, crushed stone, dense graded, asphalt concrete, sand, and asphalt stabilized subbases are the most widely used in the United States.

Two types of aggregate, viz., pit-run gravel and crushed stone, were selected to represent the unstabilized materials. Portland cement and asphalt are the most widely used stabilized layers and were included in the

testing program. A limited number of tests were also conducted on lean concrete materials. Asphalt concrete was not included, since it is basically nonerodible. The n-value (n is the exponent in the equation $p=(d/D)^n$ where p is the percent passing size d and D is the maximum size) was used to characterize the gradations. Yoder [1966] also used the n-value in pumping studies conducted in the 1950's.

The environmental conditions are important factors in determining the performance of all pavement layers. Environmental factors influence the strength, the durability, and the erosion potential of the pavement materials. The most important environmental factors are temperature and moisture content. Changes in temperature can cause freezing and thawing of the pavement materials, while changes in moisture content can cause alternate wet and dry conditions in the materials. Different materials are affected to different extents by these changes, as well as the number of cycles of each. The occurrence of the conditions and the number of cycles depend on the geographic location of the pavement and the position of the material in the pavement section. The effect of compaction effort (energy) is important to the strength of the material. The compaction effort should also have an effect on the erosion and was therefore included in the study.

6.2 Materials Used

6.2.1 Aggregate

Two types of aggregates were used, viz., crushed stone and pit-run gravel. The pit-run gravel had a bulk specific gravity of 2.644, an apparent specific gravity of 2.710, and an absorption of 1.56%. The crushed stone had a bulk specific gravity of 2.696, an apparent specific gravity of 2.741, and an absorption of 1.28%. Samples were prepared with gradations ranging from n-values of 0.3 to 0.7. The maximum size of the aggregate was selected as 9.52 mm (3/8 in.), since the minimum annular space in the rotational shear device was 9.52 mm (3/8 in.). Figure 6.1 displays the two gradations with the minimum and maximum n-values. The PI obtained from the material passing 0.42 mm (No. 40 sieve) was about 1 for the gravel material. In order to obtain samples with higher plasticity indices, some of the material was replaced with New Haven clay. Gravel samples were prepared with PI's of 1 and 15.

The crushed stone materials passing 0.42 mm (No. 40 sieve) also had a PI of about 1. No crushed stone samples with high PI's were prepared. Density-moisture content relationships were developed for the aggregate using the standard and modified Proctor hammers, as described in AASHTO T99 and T180 Methods C [AASHTO1970], respectively. Figure 6.2 gives the moisture-density relationships for a

Figure 6.1 Aggregate Gradation

Figure 6.2 Moisture-Density Curves for Gravel

gravel with a n-value of 0.6. Other properties of some of the gradations are summarized in Table 6.1.

6.2.2 Portland Cement Stabilized Samples

A type I Portland cement was used in the stabilization of the aggregate and in the preparation of the lean concrete samples.

A few samples were compacted at different cement contents to determine if the added Portland cement changed the optimum moisture content dramatically. The optimum moisture contents were slightly higher due to the added fine material (Portland cement). The optimum moisture contents determined for the unstabilized aggregate were therefore slightly increased to take this into account. The amount depended on the cement content. An example is given in Figure 6.3 for an aggregate with a n-value of 0.4 and a Portland cement content of 5 percent.

6.2.3 Asphalt Stabilized Samples

A slow setting, high float emulsion (SS-1h) was used in the stabilization of the aggregate. The emulsion was made of AC-20 in the ratio of 63% residual asphalt and 37% water. The optimum water and residual asphalt contents were determined by the Illinois method as reported by the Asphalt Institute [AI1979]. The results are summarized in Table 6.2. The method basically consists of first

Table 6.1 Aggregate Gradation Properties

n-value	0.3	0.45	0.5	0.7
PI low	1	1	1	1
PI high	15	-	-	1
C_u	393	54	36	12
C_c	3.9	2.5	2.2	1.8
D_{10}	0.0044	0.057	0.095	0.355
p200	23	11	9	3
w_{opt} T99	7.5	7.2	7.1	6.2
w_{opt} T180	6.1	5.75	5.7	5.4

n = exponent in $p=(d/D)^n$, where
 p is the percent of material passing
size d and D is the maximum size.

PI = plasticity index

C_u = coefficient of uniformity

C_c = coefficient of curvature

D_{10} = Effective diameter (in mm)

p200 = percentage of material by dry weight
passing the 0.074 mm (No.200) sieve

w_{opt} T99 = optimum water content (in %)
- standard Proctor

T180 = optimum water content (in %)
- Modified Proctor

Figure 6.3 Moisture-Density Curves for Cement Stabilized Material

Table 6.2 Optimum Asphalt and Water Contents for
Selected Gradations

n-value		0.3		0.375		0.45		0.525		0.6	
w _{opt}		5.25		4.9		4.5		4.1		3.75	
ac _{opt}		5.5		4.9		4.3		3.7		2.65	

n = exponent in $p=(d/D)^n$, where
p is the percent of material passing
size d and D is the maximum size.

w_{opt} = optimum water content (in %)

ac_{opt} = optimum residual asphalt content (in %)

determining the optimum water content with the expected asphalt content added and then determining the optimum asphalt content. The expected residual asphalt content was determined from an equation presented in the method. With the optimum moisture content known, samples were compacted at four different asphalt contents to determine the optimum asphalt content by measuring or calculating the dry and wet Marshall Stability, the dry bulk density, the percent moisture absorbed, and the percentage of voids.

6.3 Sample Preparation

6.3.1 Unstabilized Materials

The samples were compacted in a mold 116 mm (4.586 in.) high and 402 mm (4 in.) in diameter as described in AASHTO T99-81 or T180-74 [AASHTO1970], except that 5 different compaction efforts were used. The compaction effort ranged from the one described in AASHTO T99 to the one described in AASHTO T180. The three intermediate compaction efforts were obtained by using different combinations of the two hammer weights, the two drop heights, and changing the number of blows. The samples were not removed from the mold during the testing.

6.3.2 Portland Cement Stabilized Materials

The Portland cement stabilized samples were prepared in the same mold as used for the unstabilized samples. The method is described in AASHTO T134-76 [AASHTO1970]. Different compaction energies were again used. The samples prepared for testing in the rotational shear device were compacted on a metal cap, which was used to fix the sample to the torque measuring device. The cap was placed at the bottom of the compaction mold and the sample compacted on the cap. The samples for testing with the brush and jetting device were compacted without caps. All the Portland cement stabilized samples were removed from the mold and cured in a moist room at 21 C (70 F) for at least 7 days.

6.3.3 Lean Concrete Cement Samples

The samples was compacted by rodding (3 layers with 25 rods each) in a 102 by 204 mm (4 by 8 in.) cylinder. They were also cured in a moist room at 21 C (70 F).

6.3.4 Asphalt Stabilized Samples

A mechanical Marshall hammer was used to compact the samples in a mold 62.5 mm (2.5 in.) high and 102 mm (4 in.) in diameter as described in AASHTO T245-82 [AASHTO1970]. The AASHTO test allows for compaction with the mechanical Marshall hammer. The number of blows with

the hammer were varied from 15 to 75 on each end. The samples were cured in the mold at 21 C (70 F) for 3 days and then removed from the molds for further curing at room temperature for at least 1 day.

6.4 Experimental Design

6.4.1 Brushing Test

A composite experimental design was selected as a testing procedure since it requires relatively few samples, e.g., only 32 samples for five main effects (variables) [Cochran1957]. The effects of the linear main effects can be assessed with an ANOVA procedure on the factorial part, and a regression equation can be developed relating the erosion with all main effects [Anderson1974]. The regression model for 5 variables is as follows:

$$\begin{aligned} Er = & \mu + \beta_1 X_1 + \beta_2 X_2 + \beta_3 X_3 + \beta_4 X_4 + \beta_5 \\ & + \beta_{12} X_1 X_2 + \beta_{13} X_1 X_3 + \beta_{14} X_1 X_4 + \beta_{15} X_1 X_5 \\ & + \beta_{23} X_2 X_3 + \beta_{24} X_2 X_4 + \beta_{25} X_2 X_5 \\ & + \beta_{34} X_3 X_4 + \beta_{45} X_4 X_5 + \beta_{11} X_1^2 + \beta_{22} X_2^2 \\ & + \beta_{33} X_3^2 + \beta_{44} X_4^2 + \beta_{55} X_5^2 + \epsilon \end{aligned}$$

where

Er = erosion

μ = constant (mean)

X1 to X5 = 5 variables

X1 = gradation n-value

X2 = compaction energy

X3 = cement or asphalt content

X4 = number of freeze-thaw cycles

X5 = number of wet-dry cycles

β_{ij} = coefficients

ϵ = error

The composite design was used to test the effect of five variables at 5 levels each for the cement and asphalt stabilized materials in 2 separate experiments. The five variables used and the levels used are summarized in Table 6.3. The variables and levels were selected to include the important factors that influence erosion and the ranges of application of these factors.

1. Gradation: The gradation n-values ranged from 0.3 to 0.7. This range includes the gradation specifications given by AASHTO [AASHTO1970] and PCA [PCA1971]. Gradations with higher n-values are more open-graded and less subject to surface erosion.
2. Portland cement and asphalt contents: The amount of Portland cement added ranged from 1 to 16% by weight of aggregate. The amount of asphalt cement added ranged from 1.5% below optimum to 1.5% above optimum. The optimum asphalt contents were predetermined for each gradation.
3. Compaction effort: Required field compaction densities of unstabilized and Portland cement

Table 6.3 Composite Design Levels

		Factor level					unit
		-2	-1	0	+1	+2	
X1	CT	0.3	0.4	0.5	0.6	0.7	n-value
	AT	0.3	0.375	0.45	0.525	0.6	n-value
X2	CT	86	155	234	313	391	lb.in per in ³
	AT	15	30	45	60	75	blows
X3	CT	4	7	10	13	16	% by weight
	AT	-1.5	-0.75	0	-0.75	-1.5	% from optimum
X4	CT	0	2	4	6	8	number of cycles
	AT	0	1	2	3	4	number of cycles
X5	CT	0	2	4	6	8	number of cycles
	AT	0	1	2	3	4	number of cycles

CT = Portland cement stabilized

AT = asphalt stabilized

X1 = gradation n-value

X2 = compaction effort

X3 = cement or asphalt content

X4 = number of freeze-thaw cycles

X5 = number of wet-dry cycles

n = exponent in $p=(d/D)^n$, where
 p is the percent of material passing
size d and D is the maximum size.

stabilized materials are usually specified as a percentage of the standard (AASHTO T99) or modified Proctor (AASHTO T180) density. These two compaction effort and three equally divided levels between them were used. The compaction energy for the Portland cement stabilized and unstabilized samples was calculated by:

$$\text{Compaction energy} = N_B * H * N_L * W / V \text{ (in lb.in per in}^3\text{)}$$

where:

N_B = number of blows per layer

H = drop height (12 or 18 in.)

N_L = number of layers

W = hammer weight (5.5 or 10 lb)

V = mold volume (57.6 in.³)

The laboratory compaction of emulsion mixtures has been specified as 50 (Purdue method) or 75 (Illinois method) blows on each end with the Marshall hammer [All979]. For hot mixed asphalt cements the compaction is 50 or 75 blows on each end (AASHTO T245-82) [AASHTO1970]. The compaction efforts used in the testing program ranged from 15 to 75 blows per end.

4. Effect of freezing and thawing: A standard AASHTO testing procedure to assess the influence of

freezing-and-thawing on the weight loss of Portland cement stabilized materials was developed based on research at the University of Illinois [Dempsey1970, Dempsey1972, Robnett1976]. The researchers concluded from the research that the rate of cooling is important, while the length of freezing is not important as long as complete freezing is accomplished [Dempsey1972]. The length of the freeze-thaw cycle is specified in the AASHTO procedure as 48 hours. The freeze-thaw cycle consists of 24 hours freezing at a temperature of less than -23 C (-10 F) and 23 hours thawing in a moist room. The number of cycles specified is 12, but the researchers indicated that the number should depend on the region in which the pavement is situated. The number of freeze-thaw cycles recorded from October 1959 to March 1960 at the AASHO Road Test was 17 [Dempsey1970]. Although this is more than the 12 specified, there is a strong possibility that if a sample can withstand 12 cycles it can withstand any reasonable number [Cumberledge1976]. A number of accelerated [Kalankamary1963, Packard1963, Dempsey1973] and other [Merrill1968] tests have also been proposed and used, but they were basically developed to determine the effect of freezing and thawing on the strength.

The cooling and thawing rates and temperature specified in the AASHTO procedure seem to simulate the actual field conditions more closely than any other procedure, and were therefore used to investigate the effect of freezing and thawing on erosion. A standard freeze-thaw testing procedure does not exist for asphalt stabilized materials, but the AASHTO procedure also simulate the conditions of this material in the pavement. This procedure was therefore used for both Portland cement and asphalt stabilized materials. The method is not appropriate for unstabilized samples, since they can not be removed from the mold.

5. Effect of wetting and drying: Not as much research background is available on the development of the wetting-and-drying testing procedure specified in AASHTO T136-76 [AASHTO1970] The wetting-drying cycle consists of placing the sample in water at room temperature for 5 hours, followed by 42 hours in an oven at a temperature of 71 C (160 F). The method is basically the only one available for Portland cement stabilized materials, and was used to simulate critical wet and dry durations for these materials.

Basically the same procedure was used for the asphalt stabilized samples, except that the lengths of the cycles were adjusted. A standard procedure

does not exist to simulate the environmental effects on asphalt stabilized materials. Most standard mix design methods include exposure to water, since this influences the strength of the asphalt stabilized sample [All979]. Exposure to high temperatures increases the rate of strength gain during curing of the asphalt stabilized material. Asphalt stabilized materials are subject to stripping in the presence of water. The standard stripping test, AASHTO T182-82 [AASHTO1970], can not be used as a method of including the effect of water on the stabilized sample, since the test is accomplished on the uncompacted material and no cycles are involved. The wetting-and-drying test specified for Portland cement stabilized materials was modified to emphasize the water exposure. The asphalt stabilized samples were exposed to drying for 5 hours at a temperature of 104 C (140 F) and to wetting (submerged in water) for 42 hours.

Thirty two samples were required to complete the composite design, but 68 Portland cement stabilized and 33 asphalt stabilized samples were prepared in total. All the asphalt stabilized samples utilized pit-run gravel, while gravel was used for 61 cement stabilized samples. The remaining 7 were prepared using crushed stone. Thirteen lean concrete samples were prepared to

investigate the effect of the water-cement ratio on the erosion.

6.4.2 Rotational Shear Testing

A full experimental design was not used, instead, results from the brushing test were used to identify important variables and ranges of these variables. The brushing test results indicated that Portland cement content is the most important factor in the erosion. Samples were compacted at 6 different Portland cement contents ranging from 1 to 7%, since this is the range of Portland cement contents at which the largest changes in erosion occur. The shear stresses on the sample are affected by the annular space and the surface roughness of the sample. The surface roughness is a function of the gradation. Samples were therefore also compacted using 3 different gradations, viz., gradations with n-values of 0.3, 0.4, and 0.6. In total 11 different samples were prepared to investigate the effect of Portland cement content and gradation on erosion, as measured by the rotational shear device.

6.4.3 Jetting Test

A number of unstabilized samples were tested to investigate the effect of gradation, PI, and compaction effort on the erosion of these samples.

6.5 Experimental Procedure

6.5.1 Brushing Test

Only the stabilized samples were tested with the brushing test and they were compacted and cured as described in Section 6.3.

After initial curing, some of the Portland cement stabilized samples were exposed to freezing-and-thawing and wetting-and-drying cycles. The lengths of exposure to each conditions as described in AASHTO T135-76, T136-76 [AASHTO1970] were used. Since the samples were exposed to both freezing-and-thawing and wetting-and-drying cycles, the order of the exposures in the wetting-and-drying cycle was changed to ensure water in the sample during the freezing cycle. This meant that the wetting-and-drying cycle consisted of 42 hours of drying and 5 hours of exposure to water.

Each cycle was still 47 hours long. The samples were allowed to dry in air from between 15 and 30 minutes before brushing. The brushing was done in accordance with the brushing procedure described in Chapter 5. The samples were first brushed after initial curing and then after every freeze-thaw cycle, wet-dry cycle, or every 2 days when the samples were not exposed to the environmental conditions. The weight loss during each brushing was recorded in grams. The erosion was then

expressed as grams loss per sq m by dividing the weight loss (in g) by the original brushed area (in sq m).

The strengths of the stabilized samples were also determined. The unconfined compressive strength (UCS) of the Portland cement stabilized samples were obtained at the end of the experiment (31 days after compaction). The Hveem R-values of the asphalt stabilized samples were obtained 4 days after compaction and at the end of the experiment (16 days after compaction). Pulse-velocity tests were also conducted on 10 asphalt stabilized samples before and after exposure to water.

6.5.2 Rotational Shear Testing

The Portland cement stabilized samples were compacted and cured as described in Section 6.3. The sample was placed in water for 1 hour or in the moist room for at least 24 hours before testing. The bottom cap was then attached to the sample with plastic tape. The tape was also placed around the top of the sample covering the top cap and the space between the top cap and the sample. The purpose of the tape was to prevent water from entering the spaces between the sample ends and the caps, causing uncontrolled erosion at the sample ends. The sample was then carefully placed in the rotational shear device, the transparent cylinder fastened in place, and the annular space filled with deionized water.

The cylinder was rotated at different rotational speeds for about 2 minutes each, starting at about 500 rpm and increasing at rates of 500 rpm to about 2500 rpm. The force necessary to keep the sample stationary was recorded at each speed. The shear stresses, calculated from the measured forces, were plotted against the rotational speed. The purpose of this exercise was to remove all the loose material from the sample, to develop a shear stress-rotational speed relationship for each sample, and to determine the internal friction. The water and eroded material were drained through the drains without removing the sample. Care was taken to wash all the eroded material out of the cylinder.

The actual testing consisted of rotating the device at different speeds for 10 minutes each, starting at about 500 rpm and stopping at about 2500 rpm. At least six different rotational speeds were used in each test. Chapuis also found that between 6 and 10 steps were necessary to describe the erosion properly [Chapuis1983]. The force necessary to hold the sample stationary, the air pressure, and the rotational speed were recorded at least three times during each run. The water and the eroded material were drained into a 2000 ml flask after each run at a particular speed. All the eroded material was removed by washing the inside of the cylinder at least three times.

The eroded material was removed by filtering the water through filtering paper and a funnel. The filtering paper and eroded material were dried to a constant weight at 110 C (230 F) and weighed to the nearest 0.001 g. The erosion was recorded as weight loss per eroded surface area per minute (in g per sq m.min). The erosion was then plotted against the shear stress (in Pa). The shear stress used was the calculated shear stress minus the shear stress due to the internal friction of the device. A sharp increase in the erosion, as plotted against the shear stress, indicated the critical shear stress. Two visually fitted straight lines through the plotted points were used to identify this point.

6.5.3 Jetting Device

The unstabilized samples were prepared as described in Section 6.3. The samples were then placed in the container built for the jetting testing. The samples were then subjected to erosion by water from the jetting device at different water velocities. The samples were eroded for 1 minute, after which the weight losses were determined by weighing the eroded material. The recording procedure of the erosion is the same as that used for the rotational shear device (in g per sq m.min). The erosion was plotted against the calculated water velocity and the calculated shear stress. The critical water velocities and shear stresses were determined by visually fitting a

straight line through the plotted values and obtaining the intersection with the water velocity or shear stress axes.

CHAPTER 7

DISCUSSION OF EXPERIMENTAL RESULTS

7.1 Introduction

The primary purpose of the erodibility testing program was to obtain information concerning the erosion of rigid pavement subbase and shoulder materials that can be used to improve the design of rigid pavements. The results were therefore presented and discussed with this in mind. Related research by other researchers is also presented and discussed where appropriate.

Results from the brushing test can not be applied directly to actual pavement conditions, but are useful in the investigation of different factors influencing erosion, and the comparison of the erodibility of materials. On the other hand, results obtained from the rotational shear and jetting devices can be compared more directly with actual pavement conditions.

Erosion resistance can be expressed in terms of a critical shear stress, a critical water velocity, or simply a weight loss. Critical (also called threshold)

shear stress or velocity has been defined as the shear stress or water velocity at which erosion of the particles abruptly accelerates. Therefore, when the shear stress induced under the pavement slab is more than the critical shear stress for the specific material, particles will be removed at a significant rate by the erosion process. The erosion resistance may better be expressed in terms of shear stress than in terms of water velocity, since the determination of water velocity is not always simple in the prototype. The water velocity on the surface of the material is actually zero, and it changes with depth in the flow field, while the governing shear stress is defined as the shear stress on the surface of the material.

7.2 Unstabilized Materials

7.2.1 Related Research

The material properties that affect the erosion rate vary for different types of materials. Erosion of noncohesive materials depends on the particle properties, e.g., their shape, size, and the specific gravity. Relationships have been developed for the prediction of critical shear stress or critical water velocity in terms of certain measures of grain size, and the densities of the collection of solids. One problem with these relationships, e.g., by Hjulstrom, Leviavsky [Graf1971],

Shield and Lane [Vanoni1975, Dash1968], is that fines are often present in the granular materials. Such small particles can act as cementing agents among larger particles, forming particulate aggregations of higher erosion resistance. The erosion of particles smaller than 0.2 mm (0.008 in.) is probably not controlled by their sizes. This fact may not be recognized in some of the relationships [Grafl971].

The erosion of cohesive materials is influenced by a large number of material properties, e.g., degree of plasticity, percentage of clay and silt sizes, dispersion ratio, unconfined compressive strength, and chemical characteristics. Relationships have been developed to relate erosion to one or more of these material indices, e.g., by Smerdan and Beasley. The critical shear stress has been found to increase with an increase in the clay and silt contents [Grafl971, Vanoni1975]. The erosion of sand and clay is linear with the logarithm of time [Dash1969].

Materials used as subbase under rigid pavement slabs usually have a low plasticity and a small amount of material passing the 0.074 mm (No. 200) sieve. The unstabilized material will therefore behave similarly to noncohesive materials. The relationships reported in the literature for noncohesive materials were developed for uncompacted material and the critical shear stresses will

therefore be lower than the critical shear stresses of compacted samples as measured by the jetting device. The cementing effect of fines is also not included in the relationships.

7.2.2 Results of the Jetting Test

The critical water velocities and shear stresses for the unstabilized materials tested are listed in Table 7.1. Figures 7.1 to 7.6 display the results graphically. The critical stresses of noncohesive materials have been shown in sediment studies to increase with the effective diameter of the sediment [Grafl971, Anonil975]. The critical water velocity (bottom or average) also increases for effective diameters greater than 0.5 mm (0.04 in.) for the relationships developed for streambed erosion [Bhasin1969, Vanonil975, Dash1968]. A significant difference in the critical shear stresses for samples with different gradations could not be found. The reason for this might be the effect of the small particles on erosion. The erosion is controlled, not only by the effective particle diameter, but also by the cementing action of the small particles. The samples with lower gradation n-values have more fine material and the cementing action is larger. Figures 7.3 and 7.4 show that the critical water velocity and the shear stress increased with increase in compaction energy, as expected. Both the critical water velocity and shear stress increased with an

Table 7.1 Results of Jetting Test on Unstabilized Materials

Compaction Energy (lb.in/in ³)	Gradation n-value	PI	Water velocity (m/s)	Shear Stress (Pa)
234	0.3	1	1.9	3.5
234	0.3	1	1.25	1.1
234	0.3	15	2.4	5.9
86	0.5	1	0.75	0.8
391	0.5	1	1.9	3.5
234	0.6	2	1.2	1.6
234	0.6	1	1.2	1.6

1 m/s = 2.8 ft/s

1 kPa = 0.145 psi

Figure 7.1 Effect of Gradation N-Value on Critical Shear Stress

Figure 7.2 Effect of Gradation N-Value on Critical Water Velocity

Figure 7.3 Effect of Compaction Effort on Critical Shear Stress

Figure 7.4 Effect of Compaction Effort on Critical Water Velocity

Figure 7.5 Effect of Plasticity Index on Critical Shear Stress

Figure 7.6 Effect of Plasticity Index on Critical Water Velocity

increase in plasticity index (PI), as can be seen in Figures 7.5 and 7.6. A relationship by Smerdan and Beasley [Vanoni1975] for clays show the same trends. The measured shear stress values are higher than those predicted, which can be expected since the critical shear stresses are lower for clays than for granular materials. Although the jetting device could be used to compare different unstabilized materials, the shear stresses and the erosion can not be determined very accurately. Assumptions regarding mainly the area of application had to be made to obtain some measure of the shear stresses. Only a few tests could be run before the mold influenced the erosion by preventing the escape of eroded material. It was thus never possible to perform as many tests on each sample as anticipated.

7.3 Stabilized Materials

7.3.1 Related Research

The erosion of stabilized materials depends mainly on the cementing action of the binder. The surface erosion characteristics of materials have been studied extensively for the design of water channels, and the prevention of agricultural soil loss. Researchers in Iowa conducted a study to investigate the erosion of cement stabilized loess-derived alluvium and sand mixtures for use in channels [Litton1982, Litton1983]. Results showed that

the erosion rate was a function of the logarithm of time, and decreased with increasing sample durability. In a study of the use of Portland cement stabilized materials for dam facings, PCA researchers used jetting and brushing tests to evaluate the erosion of the material in different zones in the dam wall [Nussbaum1971]. They recommended that an AASHTO A-1-b soil (gradation n-value of about 0.4) should have a Portland cement content of at least 2% if used below the water line, 5% if used in the water splash area, and 3% if used above the splash line. The erosion of unstabilized materials is generally a function of the logarithm of time. Litton [1983] found this to be true for Portland cement stabilized loess and sand mixtures. However, Akky [1974] found erosion to increase linearly with time.

Very little research has been conducted to characterize the surface erosion of pavement materials. Only studies in California and France have addressed this aspect of pumping. Results of erosion tests on Portland cement stabilized materials with the California impact test [Woodstrom1983b], the vibrating table, and the rotating brush test [Pnu1979a] are displayed in Figures 7.7 to 7.9. The magnitudes of erosion values for the tests can not be directly compared, since erosion was simulated by applying very different shear stresses on the samples. The results do show, however, that if the

Figure 7.7 Effect of Portland Cement Content and Curing Age on Erosion - California [after Woodstrom 1983]

Figure 7.8 Effect of Portland Cement Content and Material Type on Erosion - Vibrating Table [from Pnu 1979a]

Figure 7.9 Effect of Portland Cement Content and Freeze-Thaw Cycles on Erosion - Brush Test [from Pnu1979a]

Figure 7.10 Effect of Portland Cement Content on Critical Shear Stress [Akky1973]

Portland cement content is high enough, the erosion of the sample is very small. The effect of the environment is important, but only for samples with low Portland cement contents (Figure 7.9). Pnu and Ray [Pnu1979a] found that the compaction effort is important, but the effect diminished with increase in sample strength. They further indicated that erosion reaches a minimum value at an optimum water content. The results were used to calculate an Index of Erodibility (IE), defined as the ratio of average erodibility of a material to the average erodibility of a sample stabilized with 3.5% cement. The effective erosion was determined as the average of the erosion of the sample under four conditions, viz., no freeze-thaw cycles and low humidity curing, no freeze-thaw cycles and high humidity curing, 9 freeze-thaw cycles and low humidity curing, and 9 freeze-thaw cycles and high humidity curing. The IE ranges from about 0.1 for lean concrete mixtures with 8.8% cement to about 150 for unstabilized materials. The researchers recommended that a granular material be stabilized with 5 to 6% Portland cement to prevent erosion. They found that water velocities of less than 3 m/s (9.8 ft/s) were too low to erode Portland cement stabilized layers, but did erode unstabilized layers. Woodstrom [1983] found that Portland cement content has a larger effect on erosion than on the strength for Portland cement stabilized materials.

The results displayed in Figure 7.8 show that lean concrete has a lower erodibility than Portland cement stabilized samples at the same cement content. Another advantage of lean concrete layers is that, since they are placed with a concrete paving machine, no trimming is needed. Trimming can cause the development of a surficial layer which is loose and easily eroded.

Since erosion resistance is merely approximated in these tests, the predictions have limited meaning. Material resistances can be compared for a given test procedure, but do not directly represent resistance to water flow. On the other hand, results from the rotational shear device provide direct values. The determined critical shear stress can be compared to the shear stress produced by loading of the water between the pavement slab and subbase. If the shear stress produced by the water in the pavement is higher than the critical shear stress of the subbase, the subbase will erode. The rate can also be determined from the rotational shear test. Figure 7.10 shows the result of such a test, conducted by Akky on an A-1-b soil [Akky1974].

7.3.2 Results of the Brush Test on Cement Stabilized Samples

Erosion was predicted from the average weight loss of the sample after two complete brushes after the last

Table 7.2 ANOVA - Portland Cement Stabilized
Samples: Erosion

Source	SS	df	MSE	F	F(with pooled error)	
constant	31088.0	1	1089	3047.8	3040.3	*
X1	309.1	1	309.1	30.4	31.2	*
X2	51.5	1	51.5	5.1	5.2	**
X3	2700.4	1	2700.4	265.9	272.8	*
X4	106.4	1	106.4	10.5	10.7	*
X5	958.6	1	958.6	94.4	96.8	*
X1X2	337.7	1	337.7	33.3	34.1	*
X1X3	335.7	1	335.7	33.1	33.9	*
X1X4	47.3	1	47.3	4.7	4.8	**
X1X5	169.3	1	169.3	16.7	17.1	*
X2X3	100.8	1	100.8	9.9	10.2	*
X2X4	517.6	1	517.6	51.0	52.3	*
X2X5	760.6	1	760.6	74.9	76.8	*
X3X4	64.0	1	64.0	6.3	6.5	*
X3X5	237.1	1	237.1	23.3	23.9	*
X4X5	8.9	1	8.9	<1.0	pooled	
Error	40.6	4	10.2			
(Pooled error)	(49.5)	(5)	(9.9)			
F(90,1,4) = 4.54 F(90,1,5) = 4.06						
F(95,1,4) = 7.71 F(95,1,5) = 6.61						
F(75,1,4) = 2.02						

* significant at $\alpha = 5\%$

** significant at $\alpha = 10\%$

where

X1 = gradation n-value

X2 = compaction energy (lb.in. per in.³)

X3 = Portland cement content (percent)

X4 = number of freeze thaw cycles

X5 = number of wet-dry cycles

n = exponent in $p=(d/D)^n$, where

p is the percent of material passing
size d and D is the maximum size.

cycle. The results of the ANOVA analysis is given in Table 7.2. Only the main effects and two-way interactions were tested. An estimation of the within error could be obtained, since replicate samples were tested. The factors with an α -value of more than 25%, if tested by the within error, were pooled with the error for further testing. All the main effects and two-way interactions were significant at $\alpha = 5\%$, except compaction energy and the interaction of gradation n-value and the number of freeze-thaw cycles, which were significant at $\alpha = 10\%$. The variables are therefore all affected by each other. All the variables were expected to influence the erosion significantly. Regression equations were developed to predict the erosion (weight loss) after 7 (Equation 7.2) and 31 days (Equation 7.1). Table 7.3 summarizes the variables used, the coefficients, and other properties of the regression equations. All the coefficients in the composite design model are presented in Table 7.3 to show the effect of all the variables. Table B.9 in Appendix B contains reduced regression models with only the important variables included. Figures 7.11 to 7.15 display how erosion is affected by the five main variables used in the study. The mathematical function itself is not plotted in these Figures, only certain values are shown for illustration purposes. Erosion decreased with compaction effort and Portland cement content. Erosion was a minimum at a gradation n-value of about 0.5. The erosion

Table 7.3 Regression Coefficients - Portland Cement Stabilized Samples

VARIABLE	COEFFICIENTS			
	E_{b31}	E_{b7}	E_{bage} (log)	E_{b31} (log)
constant	1178.1814	1584.2571	2.9552	2.9241
X1	-1459.7314	-1334.6571	-1.3965	
X2 ₂	-0.8221	-1.1912	0.009515	
X1 ₂	973.6656	1589.5526	5.8716	
X2 ²	-0.0011	0.0006	0.0000	
log(X3)	-958.1171	-1455.1376	-1.7628	
log(X4)	88.8042	-	-	
log(X5)	17.2777	-	-	
log(age)	-	-	-0.8727	
X1X2	0.9210	-1.6583	-0.0198	
X1X3	42.9041	36.6005	0.1836	
X1X4	-23.0810	-	-	
X1X5	-0.3667	-	-	
X2X3	0.0939	0.1846	-0.0002	
X2X4	-0.0025	-	-	
X2X5	-0.0161	-	-	
X3X4	-0.2540	-	-	
X3X5	-0.2009	-	-	
X4X5	0.7401	-	-	
UCS ₂	-	-	-	-0.00085
UCS ²	-	-	-	0.000000112
R^2	86%	73%	86%	76%
Adj. R^2	81%	70%	85%	75%
Std.error	67.25	152.88	0.125	0.2052
Coef.var.	63.3%	76.4%	10.9%	12.7%
n	68	84	194	55
Equation	7.1	7.2	7.3	7.4

E_{b7} = brush erosion after 7 days moist curing

E_{b31} = brush erosion 31 days after compaction

E_{bage} = brush erosion with age as a variable

X1 = gradation n-value (0.3 to 0.7)

X2 = compaction energy (86 to 391)(lb-in. per in.³)

X3 = Portland cement content (1 to 16)(percent by weight)

X4 = number of freeze thaw cycles (0 to 8)

X5 = number of wet-dry cycles (0 to 8)

n = exponent in $p=(d/D)^n$, where

p is the percent of material passing
size d and D is the maximum size.

Figure 7.11 Effect of Gradation on Weight Loss
- Portland Cement Stabilized

Figure 7.12 Effect of Compaction Energy on Weight Loss
- Portland Cement Stabilized

Figure 7.13 Effect of Number of Freeze-Thaw Cycles on Weight Loss - Portland Cement Stabilized

Figure 7.14 Effect of Number of Wet-Dry Cycles on Weight Loss - Portland Cement Stabilized

Figure 7.15 Effect of Cement Content on Weight Loss
- Portland Cement Stabilized

Table 7.4 Values of Variables Used in Plots: Portland Cement Stabilized

CURVE	Eq.	X1	X2	X3	X4	X5	age
Figure 7.11							
C1	7.1	—	234	2	0	0	n.a.
C2	7.1	—	234	4	0	0	n.a.
C3	7.1	—	234	6	0	0	n.a.
Figure 7.12							
C4	7.1	0.5	—	2	0	0	n.a.
C5	7.1	0.3	—	6	0	0	n.a.
C6	7.1	0.5	—	6	0	0	n.a.
Figure 7.13							
C7	7.1	0.3	86	1	—	0	n.a.
C8	7.1	0.5	234	1	—	0	n.a.
C9	7.1	0.5	86	2	—	0	n.a.
Figure 7.14							
C10	7.1	0.3	86	2	0	—	n.a.
C11	7.1	0.5	234	2	0	—	n.a.
C12	7.1	0.5	234	6	0	—	n.a.
Figure 7.15							
C13	7.2	0.5	234	—	0	0	7
C14	7.1	0.5	234	—	0	0	31
C15	7.2	0.5	234	—	0	0	7
C16	7.1	0.5	234	—	0	0	31
Figure 7.16							
C17	7.3	0.5	234	2	0	0	—
C18	7.3	0.5	234	6	0	0	—
C19	7.3	0.5	234	8	0	0	—

where

Eq. = number of equation used (Table 7.3)

X1 = gradation n-value

X2 = compaction energy (lb-in. per in.³)

X3 = Portland cement content (percent by weight)

X4 = number of freeze thaw cycles

X5 = number of wet-dry cycles

age = age of sample after preparation (day)

n = exponent in $p = (d/D)^n$, where

p is the percent of material passing
size d and D is the maximum size.

increased with the number of freeze-thaw cycles and the number of wet-dry cycles for low cement contents, low compaction efforts, and small gradation n-values. At high cement contents and high compaction efforts, the freezing-and-thawing and wetting-and-drying had no detrimental effect on the erosion.

A regression equation was also developed to predict the erosion with curing age as one of the variables. Figure 7.16 shows that the erosion decreases with an increase in curing age.

The erosion (weight loss) values predicted by this equation (Equation 7.3 in Table 7.3) for samples after 31 days of curing are not exactly the same as those predicted by Equations 7.1 in Table 7.3, as shown in Figure 7.17. However, the 95% confidence intervals, plotted on Figure 7.17 show that these two equations predict the same values, at $\alpha = 5\%$, for samples with Portland cement contents of less than 13%. A third fewer cases were used to develop Equation 7.1 (curve Y1) than Equation 7.3 (curve Y2), and the latter therefore has a tighter confidence band. The values predicted by Equation 7.3 (curve Y2) are consistently lower than those values predicted by Equation 7.1 (curve Y1). A major reason for the differences in the predicted values is that the curing ages of most of the values used to develop Equation 7.3 were less than 31 days, while Equation 7.1 was developed

Figure 7.16 Effect of Curing Age on Weight Loss
- Portland Cement Stabilized

Figure 7.17 Comparison of Weight Loss Prediction Equations
- Portland Cement Stabilized (31 day)

Figure 7.18 Comparison of Weight Loss Prediction Equations
- Portland Cement Stabilized (7 day)

with results obtained at 31 curing days. The curves plotted in Figure 7.18 show that the weight loss values predicted after 7 curing days by Equations 7.2 and 7.3 are much closer than between the values predicted by Equations 7.1 and 7.3. The weight loss values predicted by Equations 7.2 and 7.3 are the same at $\alpha = 5\%$.

The 32 samples tested as part of the composite design procedure, utilized pit-run gravel as aggregate. A number of samples were also tested to investigate the effect of the use of crushed stone as aggregate on erosion. No difference in weight loss characteristics could be detected between gravel and crushed stone stabilized samples, and the results were pooled in the development of the equations. Figure 7.19 displays the weight loss results for gravel and crushed stone stabilized samples at different Portland cement contents, nine days after compaction.

The unconfined compressive strength (UCS) of the samples was also obtained after 31 days. An analysis of the results is presented in Appendix B. An equation was developed to predict erosion (as weight loss measured from the brush test) from the 31 day UCS. The coefficients are given in Table 7.3 (Equation 7.4). Figure 7.20 shows the plot of erosion against UCS. Erosion was more sensitive than the UCS to Portland cement content and the gradation n-value, while compaction effort had a larger effect on the strength than on erosion.

Figure 7.19 Weight Losses of Gravel and Crushed Stone Stabilized Materials

Figure 7.20 Brush Weight Loss versus Compressive Strength
- Portland Cement Stabilized

7.3.3 Results of the Brush Test on Asphalt Stabilized Samples

The erosion was predicted from a number of different measurements, e.g., average weight loss per cycle, weight loss after all the cycles with the samples in a dry condition, and erosion after all the cycles with the samples in a semi-wet condition. The measure which gave the strongest relationship and was also the most appropriate was the average of two brushes after the last cycle, with the sample in a semi-wet condition (the procedure has been described in the previous Chapter).

The procedure of analysis used for the Portland cement stabilized material was also used for the asphalt stabilized material. The results of the analysis of variance are given in Table 7.5. Of the main effects, compaction effort, asphalt content and number of wet-dry cycles were significant at $\alpha = 5\%$. A number of regression equations to predict the erosion (weight loss) based on the test results were developed. Table 7.6 provides the regression coefficients and ranges of variables used in the development. All the coefficients in the regression model are presented. Table B.10 contains reduced models. Figures 7.21 to 7.25 display the influence of the five variables used in the test. Table 7.6 contains the

Table 7.5 ANOVA - Asphalt Stabilized Samples: Erosion

Source	df	SS	MSE	F	
constant	1	71294.1	71294.1	67.6	*
X1	1	10.9	10.9	<1	
X2	1	9138.6	9138.6	8.7	*
X3	1	9138.6	9138.6	8.7	*
X4	1	271.7	271.7	0.3	
X5	1	4792.1	4792.1	4.6	*
Error	10	10540.2	1540.0		

$$F(95,1,10) = 4.71$$

$$F(90,1,10) = 3.29$$

* significant at = 5%

where

X1 = gradation n-value

X2 = compaction energy (Marshall hammer blows)

X3 = asphalt content (percent from optimum)

X4 = number of freeze thaw cycles

X5 = number of wet-dry cycles

n = exponent in $p=(d/D)^n$, where

p is the percent of material passing
size d and D is the maximum size.

Table 7.6 Regression Coefficients - Asphalt Stabilized Samples

VARIABLE	COEFFICIENTS		
	E_{b16}	E_{b4}	E_{bage}
constant	646.5860	537.0200	511.6115
X1	-1775.8868	-1553.7052	-634.9318
X2	-	-5.9968	-
X3	47.9811	118.2000	69.2456
X4	61.2182	-	-
X5	25.1490	-	-
X1 ²	1807.8486	1650.9968	707.6971
X2 ²	-	0.0290	-
X3 ²	9.2881	8.5796	3.6533
X4 ²	-1.0083	-	-
X5 ²	3.0548	-	-
log(X2)	-167.3990	-	-174.1663
log(age)	-	-	-102.2842
X1X2	4.1211	4.7099	1.9436
X1X3	-219.7609	-293.3122	-192.4926
X1X4	-76.9163	-	-
X1X5	-10.9880	-	-
X2X3	0.6593	-0.2215	0.0635
X2X4	-0.4464	-	-
X2X5	-0.3365	-	-
X3X4	14.2844	-	-
X3X5	-18.6797	-	-
X4X5	-1.6376	-	-
R ²	90%	79%	66%
Adj. R ²	75%	71%	63%
std err	22.7994	22.5976	22.2979
coef var	36.6%	29.5%	48.0%
n	33	33	109
Equation	7.5	7.6	7.7

E_{4b} = brush erosion 4 days after construction

E_{16b} = brush erosion 16 days after construction

E_{bage} = brush erosion with age as a variable

X1 = gradation n-value (0.3 to 0.6)

X2 = compaction energy (15 to 90)(Marshall hammer blows)

X3 = asphalt content (-1.5 to 1.5)(percent from optimum)

X4 = number of freeze thaw cycles (0 to 4)

X5 = number of wet-dry cycles (0 to 4)

n = exponent in $p=(d/D)^n$, where

p is the percent of material passing
size d and D is the maximum size.

Figure 7.21 Effect of Gradation on Weight Loss.
- Asphalt Stabilized

Figure 7.22 Effect of Compaction Energy on Weight Loss
- Asphalt Stabilized

Figure 7.23 Effect of Number of Freeze-Thaw Cycles on Weight Loss - Asphalt Stabilized

Figure 7.24 Effect of number of Wet-Dry Cycles on Weight Loss - Asphalt Stabilized

Figure 7.25 Effect of Asphalt Content on Weight Loss
- Asphalt Stabilized

Table 7.7 Values of Variables Used in Plots: Asphalt Stabilized

CURVE	Eq.	X1	X2	X3	X4	X5	age
Figure 7.21							
A1	7.5	—	45	0	0	0	n.a.
A2	7.5	—	45	-1.5	0	0	n.a.
A3	7.5	—	45	0	0	0	n.a.
Figure 7.22							
A4	7.5	0.45	—	-1.5	0	0	n.a.
A5	7.5	0.45	—	0	0	0	n.a.
A6	7.5	0.45	—	1.5	0	0	n.a.
Figure 7.23							
A7	7.5	0.45	30	0	—	0	n.a.
A8	7.5	0.45	45	0	—	0	n.a.
A9	7.5	0.45	45	-1.5	—	0	n.a.
Figure 7.24							
A10	7.5	0.45	45	-1.5	0	—	n.a.
A11	7.5	0.60	45	0	0	—	n.a.
A12	7.5	0.45	45	0	0	—	n.a.
Figure 7.25							
A13	7.6	0.45	30	—	0	0	4
A14	7.6	0.45	60	—	0	0	4
A15	7.5	0.45	30	—	0	0	16
A16	7.5	0.45	60	—	0	0	16
Figure 7.26							
A17	7.7	0.45	45	-1.5	0	0	—
A18	7.7	0.45	45	0	0	0	—
A19	7.7	0.45	45	1.5	0	0	—

where

Eq. = number of equation used (Table 7.6)

X1 = gradation n-value

X2 = compaction energy (Marshall hammer blows)

X3 = asphalt content (percent from optimum)

X4 = number of freeze thaw cycles

X5 = number of wet-dry cycles

age = age of sample after preparation (day)

n = exponent in $p=(d/D)^n$, where

p is the percent of material passing
size d and D is the maximum size.

regression coefficients. The erosion (weight loss) reached a minimum at a gradation n-value of about 0.5. Low compaction efforts and small gradation n-values increased the erosion at all levels. The effect of the freeze-thaw cycles was in general small and not significant. At low asphalt contents the weight loss decreased slightly with an increase in the number of freeze-thaw cycles, while at high asphalt contents the weight loss increased slightly with an increase in the number of freeze-thaw cycles.

Asphalt stabilized materials are subject to stripping which increases the erosion. Asphalt stripping was simulated in the experimental program by wetting-and-drying cycles, since standard tests are not available to simulate stripping conditions with time. The results do show that the weight loss of the samples is significantly influenced by these cycles, but it is not clear how well the wet-dry cycling simulated stripping conditions. The weight loss increased with number of wet-dry cycles in all cases tested, but the rate of increase was higher at low asphalt contents, smaller size aggregates, and low compaction efforts. The weight loss (stripping) decreased as asphalt content increased. The erosion (weight loss) of the asphalt stabilized samples decreased with age and asphalt content.

A regression equation was also developed to predict the erosion with curing age as one of the variables (Equation 7.7 in Table 7.6). Figure 7.26 shows that the erosion decreases with an increase in curing age. The comparisons of Equations 7.5 and 7.7, and Equations 7.6 and 7.7 are analogous to the discussion of the comparisons of Equations 7.1, 7.2, and 7.3 in Section 7.3.2 (Figures 7.17 and 7.18). Figures 7.27 and 7.28 depict the predicted values and 95% confidence intervals for the weight loss of asphalt stabilized materials after 4 and 16 curing days, respectively. The weight losses of samples after 16 curing days predicted by Equations 7.5 and 7.7, are the same (at $\alpha = 5\%$) for samples with asphalt contents of more than about 1.5 % above optimum. The values predicted by Equations 7.6 and 7.7, at 4 curing days, are the same at all asphalt contents investigated at $\alpha = 5\%$.

As in the case of the Portland cement stabilized materials, the strength of the materials were measured in an attempt to correlate the weight loss with the strength. The results of the analysis of the Hveem R-values are given in Appendix B. A satisfactory correlation between weight loss and the strength (R-value) could not be identified.

Figure 7.26 Effect of Curing Age on Weight Loss
- Asphalt Stabilized

Figure 7.27 Comparison of Weight Loss Prediction Equations
- Asphalt Stabilized (16 day)

Figure 7.28 Comparison of Weight Loss Prediction Equations
- Asphalt Stabilized (4 day)

7.3.4 Results of the Brush Test on Lean Concrete Samples

Only a limited number of lean concrete samples were tested and the results will therefore only be used to show trends. Loss of weight of lean concrete materials depend largely on the Portland cement content and the water-cement (W/C) ratio. Figure 7.29 shows how the weight loss is affected by changes in the W/C ratio. Difference among the erosion of different lean concrete mixtures could be detected, but the brush erosion results can not be compared with that of the Portland cement stabilized materials. The major advantage of lean concrete materials with regards to erosion is that loose particles are not as prevalent on the surface as in the case of Portland cement stabilized materials, due to the type of compaction. The type of erosion caused by the bristles of the brush makes it impossible to detect such differences on the surfaces of the samples.

7.3.5 Results of the Rotational Shear Test

The erosion of Portland cement stabilized samples was evaluated with Portland cement content as the only variable. A number of parameters may be obtained from the results. Figure 7.30 shows typical results and Table 7.8 contains the results of all the tests. A rate of erosion before the critical shear stress (τ_c) is reached, the τ_c ,

Figure 7.29 Effect of Water-Cement Ratio on Weight Loss
- Lean Concrete

Figure 7.30 Typical Rotational Shear Test Results

Table 7.8 Results of Rotational Shear Tests on Cement Stabilized Samples

Cement content (%)	Curing time (days)	Erosion rate 1	Critical	Erosion rate 2	Brush erosion (g/m ² .min)	a	b
			Shear stress (Pa)				
1	7	-	4.5	2.04	817	-	-
2.5	7	-	5.5	0.074	265	-	-
	21	0.183	11.0	0.376	155	0.0508	-0.527
3.0	7	-	10.0	-	552	-	-
	31	0.108	13.0	2.76	331	0.0432	0.185
3.0	7	-	7.0	0.104	839	-	-
	31	0.167	12.0	4.0	486	0.1037	-0.683
4.0	7	-	6.0	0.18	309	-	-
	21	0.004	14.0	0.074	155	0.0174	-0.408
	31	0.019	24.0	0.19	66	0.0260	-0.804
7.0	7	-	6.0	0.18	199	-	-
	21	0.011	26.0	0.16	88	0.0229	-0.703
	31	0.011	33.0	0.16	66	0.0218	-0.627

Erosion rate 1 = erosion rate before τ_c (g/sq m.min per Pa)

Erosion rate 2 = erosion rate after τ_c (g/sq m.min per Pa)

a,b = coefficients in $\log(\text{erosion}) = a \cdot \tau + b$

1 kPa = 0.145 psi

1 m = 3.281 ft

1 kg = 2.205 lb

and the rate after τ_c can be obtained. Two straight lines were visually plotted to obtain the erosion rates and τ_c . Exponential and quadratic regression curves were also calculated. It was found that, although both curves fitted the data well, the shape of the exponential curve was more appropriate, and it was therefore used to describe the data. The exponential curve was not forced through the origin.

The exponential regression equation may be used to predict the erosion, based on the shear stress, but can not be used to identify τ_c . Table 7.8 contains the results of all the rotational shear tests, with the coefficients of the regression curves. The adjusted R^2 values for these curves were in excess of 90% in all cases. The erosion rates after τ_c has been reached could not be determined accurately in all cases, since some of the sample deteriorated at high shear stresses.

The rotational shear testing has a major limitation in that unstabilized cohesionless materials can not be tested. For example, it was found that samples with cement contents of less than 1% had insufficient cohesion for testing in this device.

The curing age obviously has an influence on the erosion. Samples were tested after different curing ages. Figure 7.31 displays the results of tests conducted at

Figure 7.31 Change in Critical Shear Stress with Curing Time

different curing times on one sample. The critical shear stress increases with curing age. The erosion rates show differences, but a statistical correlation could not be identified. The differences are small, in general, and the erosion rates seem to be almost constant within the curing ages incorporated in the testing program (7 to 31 days).

All of the parameters summarized in Table 7.8 were compared with each other. The most useful and significant comparisons were critical shear stress and Portland cement content. Not enough data points were available to relate erosion rates with cement contents and critical shear stresses. Figure 7.32 presents a comparison of the critical shear stress and the Portland cement content for three different curing times. A regression equation was developed to relate the critical shear stress to the brush erosion (weight loss). Figure 7.33 compares the critical shear stresses with brush weight loss results. The regression equation has an adjusted R^2 value of 61% and is given as follows.

$$\tau_c = 57.517 - 18.4 \cdot \log(E_b)$$

$$(R^2 = 64\%, n = 13)$$

where

τ_c = critical shear stress (Pa)

E_b = brush weight loss (g per sq m)

Figure 7.32 Critical Shear Stress versus Cement Content

Figure 7.33 Critical Shear Stress versus Brush Erosion

The materials tested included three different gradations, but no significant differences in τ_c of the samples could be detected.

The effect of erosion time on erosion rate was also investigated. The erosion rate was found to be a function of a reference erosion rate and the logarithm of time. A erosion time of 10 minutes was selected as the reference time.

$$E_r = -1.095 + 0.88 * E_{10} + 1.003 * \log(t)$$

$$(R^2 = 84\%, \text{ adjusted } R^2 = 80\%, \text{ and } n=11)$$

where

E_r = erosion rate (g per sq m.min)

E_{10} = erosion rate after 10 minutes

t = erosion time (minutes)

Figure 7.34 displays the relationship at three reference erosion rates of 1, 5, and 10 g per sq m.min. Erosion times ranged from 5 to 60 minutes. The equation is only valid for erosion rates of more than about 5 minutes. Erosion behavior of the samples at small erosion times (less than about 5 minute) could not be measured, and can not be predicted, by the equation. During this time, the water temperature rose from 22 C (72 F) to 27 C (80 F). For erosion times of 10 minutes and less, the water temperature remained essentially constant. The shear stresses varied about 2.5 Pa (1.05 psf) over a 10 minute erosion period.

Figure 7.34 Effect of Erosion Time on Erosion

7.4 Correlation with Other Test Results

Results of the California abrasion test, published in two papers [Neall1975, Woodstrom1983b] were used to develop comparisons between the California abrasion test and the brush test. Only the Portland cement content was used as a descriptor of the samples, since the gradations, compaction efforts, etc., were not known for all the samples. Results of abrasion tests on 7 and 28 day cured Portland cement stabilized samples were used to develop the following equation.

$$\log(E_{Cal}) = 0.55 \cdot \log(E_b) - 0.023 \cdot C^2 - 0.07 \cdot C + 2.7$$

$$(R^2 = 60\%, \text{ adj. } R^2 = 58\%, n = 66)$$

where

E_{Cal} = weight loss measured with California
abrasion device (g/sq m)

E_b = brush weight loss (g/sq m.min)

7.5 Correlation with Pavement Conditions

The laboratory results need to be related to the actual behavior of the pavement to be useful in improving the design of subbases and shoulders. Water between the slab and subbase generates the surface erosion of the subbase or shoulder, when the movement of the slab forces the water out of the void at high velocities which induce

high shear stresses. Little is known about the flow characteristics of water between the slab and an essentially impervious subbase. The flow of the water under the slab is complex and influenced by a number of factors, e.g., slab deflection velocity, the magnitude of the deflections, and void dimensions. At small void thicknesses, the water will resist the slab deflection. The major problem in the analysis is not the mathematical description of the water velocity and induced shear stresses, so much as the identification of representative void dimensions and slab deflection velocities.

7.5.1 Related Research

During the late 1970's, French researchers [Pnu1979a,Pnu1979b,Ray1981] investigated the flow of water on impervious subbases. In their theoretical calculations of the velocity of the water under the slab, they identified three void thickness zones in which water behaves differently. These zones were selected based on theoretical, laboratory, and in-situ observations.

1. Voids less than 0.5 mm (0.02 in.) in thickness: The researchers found that small void based on negative temperature gradients in the slab exist 20 to 30 percent of the time under rigid pavements in France. In these very thin layers the water behaves as a viscous fluid. They developed the following equations.

$$V_a = \frac{PH^2}{2\mu L l^2}$$

$$p_a = \frac{P}{lL}$$

$$T = \frac{\mu L l^3}{2F} \left(\frac{1}{h^2} - \frac{1}{H^2} \right)$$

$$V_z = \frac{dh}{dt} = - \frac{F \delta^3}{\mu l L^3}$$

where

V_a = average water velocity

P = axle weight

p_a = average pressure

H = initial thickness of void

Z = slab deflection

μ = dynamic viscosity of water

L = length of the void

l = width of the void

V_z = speed of slab deflection

F = resultant of the axle load force and the
slab resistance

T = time needed to produce deflection

$h = H - Z$ = final void thickness

δ = void thickness at any time

2. Voids larger than 1 mm (0.04 in.) in thickness: In these voids water behaves as an ideal fluid. Pnu and Ray [Pnu1979b] used fluid mechanic principles to develop equations to predict the water velocity.

At the transverse joint (1 direction):-

$$V_a = \frac{LV_z}{2(H - Z)}$$

At the transverse joint and shoulder (2 directions):-

$$V_a = \frac{LlV_z}{2(\frac{L}{Z} + 1)(H - Z)}$$

where

V_a = average water velocity (m/s)

H = void thickness (m)

L = length of void (m)

l = width of void (m)

V_z = speed of slab deflection (m/s)

Z = slab deflection (m)

3. Voids between 0.5 and 1 mm (0.02 to 0.04 in.) in thickness: This was identified as the transition zone, where the fluid is neither ideal nor viscous. Water velocities need to be interpolated from the previous two cases.

7.5.2 Analysis of Water Underneath the PCC Slab

The procedure described above can be used to calculate the expected water velocity under the slab. However, the shear stresses can not be calculated. A set of equations were therefore developed to calculate the water induced shear stresses on the subbase. The slab movement was simulated by a flat stiff plate rotating

around an axis. This is similar to the approach used by Pnu [1979b]. It is a fair representation of the movement of the leave slab at the joint when the wheel load moves from the approach slab onto the leave slab (Figure 7.35). The water velocity was expressed as a parabolic distribution.

$$u(x,y) = A\left(\frac{y}{\delta}\right)^2 + B\left(\frac{y}{\delta}\right) + C$$

$$\text{at } y = 0, u = 0$$

$$\text{thus } C = 0$$

$$\text{at } y = \delta, u = V_z \sin \theta$$

$$\text{thus } B = V_z \sin \theta - A$$

Further,

$$\frac{\partial u}{\partial x} + \frac{\partial v}{\partial y} = 0$$

$$\text{thus, } v = - \int \frac{\partial u}{\partial x} dy$$

$$\text{but, } v = - V_z \cos \theta$$

Solving for A,

$$A = - 6V_z \left(\frac{\sin \theta}{2} + \frac{\cos^2 \theta}{\sin \theta} \right)$$

For small angles,

$$A = - 6V_z \left(\theta/2 + 1/\theta \right)$$

The shear stress is defined as:

$$\tau = \mu \frac{du}{dy}$$

$$\text{thus, } \tau = \mu \left(\frac{2Ay}{\delta^2} + \frac{B}{\delta} \right)$$

where

Figure 7.35 Movement of Water Under the Slab

u = water velocity in the x -direction

v = water velocity in the y -direction

δ = void thickness

θ = angle between the slab and the subbase

μ = kinematic viscosity of water

V_z = speed of slab deflection = $x \frac{\partial \theta}{\partial t}$

t = time

A , B , and C = coefficients

(Also see Figure 7.35)

The maximum shear stress ($\tau_{\max}$) occurs at the contacts with the slab and with the subbase (at $y = \delta$ or $y = 0$). Therefore, the average velocity of the (V_a) water between the slab and the subbase can be determined by.

$$V_a = \frac{1}{3} \left[A \left(\frac{y}{\delta} \right)^2 + B \left(\frac{y}{\delta} \right) \right]$$

At $y = \delta/2$:

$$V_a = 1/3 (A/4 + B/2)$$

The maximum shear stress ($\tau_{\max}$) on the subbase can be obtained from.

$$\tau_{\max} = \mu \frac{B}{\delta}$$

These equations hold only when the water behaves as an ideal fluid. At very small void sizes the resistance of the water on the downward movement of the slab becomes important. Pnu [1979b] indicated that the minimum void thickness (after the slab deflection) for which the equations given above can be used is about 1 mm (0.04

in.). As mentioned earlier, representative slab deflection velocities and void dimensions have not been established. The void dimensions and slab deflections depend on factors such as, the wheel load, the temperature gradient, slab properties, and load transfer characteristics. The French researchers [Pnu1979b] speculated that the slab deflection velocities range from 1 to 100 mm/s (0.04 to 4 in./s). They found that the water pressure under the slab is influenced by the slab deflection and the vehicle speed. Water pressures between 3 and 10 kPa (0.43 and 1.45 psi) were measured, with the pressures usually higher under the leave slab. The water pressure under the slab increased rapidly with an increase in vehicle speed to about 20 km/h (11 mph), then remained fairly constant for vehicle speeds up to about 40 km/h (22 mph), after which the pressure decreased.

The slab is not deflected at a constant velocity, since it is accelerated from an initial stationary position by the wheel load until the reaction of the water becomes high enough to decelerate the slab to an equilibrium position. At the equilibrium, the downward force of the wheel load is equal to the upward force of the water and the slab. This occurs at small void thicknesses where the viscous effect of the water becomes significant. The water pressure, and therefore the force of the water on the slab, can theoretically be calculated by the following equations.

$$\frac{\partial P}{\partial x} = \mu \left(\frac{\partial^2 u}{\partial y^2} \right) - \rho \left(u \frac{du}{dy} + v \frac{du}{dy} + \frac{du}{dt} \right)$$

$$F = \int w \cdot P \, dx$$

where

P = water pressure

u = water velocity in the x-direction
(previously determined)

v = water velocity in the y-direction

μ = kinematic viscosity of water

ρ = density of water

t = time

F = water force on the slab

w = width of the opening (void)

l = length of the opening (void)

(See Figure 7.35)

These equations predict the water pressure and force as functions of: x , y , the angle (θ), the first and second derivative of the angle, the length and width of the void, and the water properties (ρ and μ). These equation can be solved with the void dimensions, and velocity and acceleration of slab deflection (derivatives of θ) known. In lieu of results from the solution of these equations, the French research results [Pnu1979a, Pnu1979b, Ray1981] were used as a guideline in quantifying water velocities and shear stresses under the slab.

The French researchers [Pnu1979b] predicted a maximum water velocity under the approach slab, and between the approach slab and the shoulder of 2.8 m/s (9.2 ft/s). The maximum water velocity under the leave slab was predicted to be 4.4 m/s (14.4 ft/s). They classified materials erodible at a water velocity of 5 m/s (16.4 ft/s) as very erodible, and materials not erodible at a water velocity of 50 m/s (164 ft/s) as non-erodible (see Section 2.3.2). Figures 7.36 and 7.37 show the influence of void size and slab deflection velocity on the water velocity and shear stress, respectively. These values are plotted from the equations for the average water velocity and shear stress derived earlier. Values at four slab deflections velocities are plotted, viz., 5, 10, 20, and 50 mm/s (0.2, 0.39, 0.79, and 1.97 in./s). The void length was taken as 0.75 m (2.46 ft), which can be considered a typical length of void at the joint. Both the water velocities and shear stresses increase rapidly at small void thicknesses. Infinitely high water velocities or shear stresses will not be reached since the reaction of the water at small void thicknesses will reduce the slab deflection and velocity of movement, as discussed. Also plotted on Figure 7.36 is the curve presented by Pnu [1979b] for axle loads of 9000 kg (19800 lb), with a velocity of slab deflection of about 10 mm/s (0.39 in./s). This curve is based on the French research [Pnu1979a, Pnu1979b, Ray1981] described in Section 7.5.1. The researchers hypothesized

Figure 7.36 Effect of Void Size and Velocity of Slab Deflection on the Average Water Velocity

Figure 7.37 Effect of Void Size and Velocity of Slab Deflection on the Shear Stress

that the maximum water velocity will occur in the "transition zone" at a void thickness of about 0.9 mm (0.04 in.) This corresponds to the water velocity obtained by a velocity of slab deflection (V_z) of 10 mm/s (0.39 ft/s). The same shape of the curve was used to identify the maximum shear stress. The shear stress obtained from Figure 7.37 is about 50 Pa (1.04 psf). The shear stresses were assumed to behave similarly to the water velocity in the transition zone and to have a similarly shaped curve, since τ is also related to V_z and the void thickness.

Using a velocity of slab deflection of 10 mm/s (0.4 in.) the maximum shear stress of 50 Pa (1.04 psf) and water velocity of 5 m/s (16.4 ft/s) will occur at a void thickness of about 0.9 mm (0.035 in.). These maximum values are obtained assuming a constant V_z of 10 mm/s (0.4 in./s) for void thicknesses of more than 1 mm (0.04 in.), and provide an upper limit of the water velocities and shear stresses in the pavement. These maximum conditions may only be reached a few times during the life of the pavement. A material that can resist erosion at a shear stress of 50 Pa (1.04 psf) should not show any signs of erosion during the life of the pavement. Examples of such materials are: asphalt concrete and types of lean concretes. A lower value of shear stress may be more realistic to use in the design of rigid pavement subbases and shoulders. A shear stress of 25 Pa (0.52 psf) is

proposed. The shape of the shear stress weight loss curve (Figure 7.33) indicates that the weight loss drops off significantly at shear stresses of less than 20 to 25 Pa (0.42 to 0.52 psf). A shear stress of 25 Pa (0.52 psf) corresponds with a void thickness of 1.4 mm (0.055 in.) and V_z of 10 mm/s (0.4 in./s), or a void thickness of 0.9 mm (0.035 in.) and V_z of 1 mm/s (0.04 in./s). It must be emphasized that these values have not been verified and are provided only as an guideline.

7.6 Results

7.6.1 Unstabilized Materials

The erosion of unstabilized materials occurs at low shear stresses. The shear stresses induced by the water (Figure 7.37) will likely be higher than the τ_c of the unstabilized material. Therefore, any impervious unstabilized material used in rigid pavements will erode. The critical shear stress can be increased by increasing the compaction effort and the PI of the material, but it will probably not be sufficient to prevent pumping.

Unstabilized materials are subject to more than surface erosion. In most unstabilized materials, pumping is a combination of pore water pressure buildup and surface erosion. The more permeable the material, the lesser the impact of surface erosion on pumping. The permeability at which the pore water pressure buildup becomes more

important was not addressed in this study.

7.6.2 Stabilized Materials

Stabilized materials are usually relatively impervious and subject to, primarily, surface erosion. Stabilized materials can be strong enough to withstand surface erosion forces under the slab, depending on the composition of the material and environmental conditions.

The results of erosion testing will be most useful if the large number of variables and combinations of variables included in the testing program can be condensed to a few representative cases. This was done by identifying four climatic regions and four typical gradation-compaction effort combinations. The United States has been divided into nine climatic regions to identify areas in which similar pavements should perform similarly [Carpenter1981b]. These climatic zones have been condensed to six regions by Basma [1984] as shown in Figure 7.38. The erosion test results can be used to predict the erosion of stabilized materials in four climatic regions, viz., a warm dry region, a warm wet region (with wet-dry cycles), a cold dry region (with freeze-thaw cycles), and a cold wet region (with freeze-thaw and wet-dry cycles). Figure 7.38 can be used as an indication of where these conditions will be prevalent. A cold (freeze-thaw) region was represented in the test

Figure 7.38 Climatic Regions in the United States
[from Basmal984]

sequence by 8 freeze-thaw cycles and a wet region by 8 wet-dry cycles, since these were the maximum number of cycles included in the testing program for Portland cement stabilized materials.

Four typical gradation-compaction effort combinations were identified, viz., standard Proctor compaction and gradation n-value of 0.3, standard compaction and n-value of 0.6, modified Proctor compaction and n-value of 0.3, and modified compaction and n-value of 0.6 for Portland cement stabilized materials. Marshall hammer blows of 15 and 75 were used to represent low and high compaction for asphalt stabilized materials.

The weight loss values 31 days after compaction (Equation 7.1 in Table 7.3) were used to characterize the erosion of the Portland cement stabilized materials, since most of the erosion tests were conducted at 31 days after preparation, and the strength after 28 days is specified in the AASHTO design procedure [AASHTO1981]. Figure 7.16 also shows that the change in weight loss is small for samples at any Portland cement content after 31 days. The weight loss values of the asphalt stabilized samples 16 days after compaction (Equation 7.5 in Table 7.6) were used to characterize the erosion of these samples. Most of the erosion tests were conducted at 16 days after compaction and the change in weight loss is small after 16 days (Figure 7.25). The weight loss or brush "erosion"

was normalized to the erosion of a granular material stabilized with 3.5 % cement, a gradation n-value of 0.5, and compacted with an energy of 234 lb-in. per cu in.. The erosion of this sample was chosen after Pnu and Ray [Pnu1979b]. Figures 7.39 to 7.42 display the results for the Portland cement stabilized materials, and Figures 7.43 to 7.46 the results for asphalt stabilized materials.

From these relationships a Portland cement or asphalt content can be selected to ensure low erosion for each one of the four typical gradation-compaction effort combinations for each of the four climatic regions. The selection of a limiting shear stress or erosion level is still an open question. The value of 25 Pa (0.52 psf), recommended in the preceding section, can be used as a guideline in design of subbases and shoulders. A critical shear stress of 25 Pa (0.52 psf) corresponds with a brush "erosion" of about 60 g per sq m.min and a normalized brush "erosion" of 0.33. A shear stress of 50 Pa (1.04 psf) corresponds to a brush "erosion" of about 3 g per sq m.min and a normalized brush "erosion" of 0.02.

In a warm, dry climate a Portland cement content of about 4.0% is needed to ensure low erosion when a high compaction effort is used, while up to 8.5% Portland cement is needed for low erosion when a layer is compacted at a low compaction (Figure 7.39). Each compaction effort and gradation combination is affected differently by the environmental conditions.

Figure 7.39 Erodibility of Portland Cement Stabilized Material - Warm, Dry Climate

Figure 7.40 Erodibility of Portland Cement Stabilized Material - Cold, Dry Climate

Figure 7.41 Erodibility of Portland Cement Stabilized Material - Warm, Wet Climate

Figure 7.42 Erodibility of Portland Cement Stabilized Material - Cold, Wet Climate

Figure 7.43 Erodibility of Asphalt Stabilized Material
- Warm, Dry Climate

Figure 7.44 Erodibility of Asphalt Stabilized Material
- Cold, Dry Climate

Figure 7.45 Erodibility of Asphalt Stabilized Material
- Warm, Wet Climate

Figure 7.46 Erodibility of Asphalt Stabilized Material
- Cold, Wet Climate

The densely compacted Portland cement stabilized materials are only slightly affected by the climatic conditions. A Portland cement content of 4% will be sufficient to prevent erosion at shear stresses of less than 25 Pa (0.52 psf) in all climatic conditions for materials with a small percentage of fines and compacted to an AASHTO T180 density. A Portland cement content of at least 4.5% is required to minimize erosion for materials with a high percentage of fines under similar climatic conditions. Materials compacted to AASHTO T99 density are more susceptible to climatic conditions. For example, the Portland cement content required to prevent erosion at 25 Pa (0.52 psf) for a material with low compaction (AASHTO T99) and a high percentage of fines is about 8.5% in a warm, dry climate (Figure 3.39). In a wet, cold (freeze-thaw) region a similar materials would require 10% Portland cement to resist erosion (Figure 7.42). A material with a low percentage of fines and a low compaction must be stabilized with at least 6.5% Portland cement to ensure low erosion in all four climatic regions.

The effects of compaction effort and gradation on the erosion of asphalt stabilized materials differ in each of the four climatic regions. Therefore, general guidelines can not easily be given. The erosion of each compaction-

gradation combination has to be investigated in each region to select a suitable material and compaction effort. A asphalt material with a large percentage of fines and a low compaction is likely to erode in any of the four climatic regions. The strength and stability of the materials were not considered in the development of these curves. The layers must still be designed to have the required strength and stability.

One of the characteristics of a stabilized layer is the existence of loose material on the surface after construction. These loose particles will erode regardless of the Portland cement or asphalt content. The effect of construction on the erosion could not be included in the testing program and is not included in the erosion values presented in this Chapter.

CHAPTER 8

DESIGN TO PREVENT PUMPING

8.1 Introduction

Present pavement design procedures are based on limiting stresses in the concrete to a level which will provide adequate resistance to fatigue failure in the concrete. At these stress levels, deflections can be sufficient to cause pumping and erosion [Ring1984]. Slabs are analyzed in the fully supported condition, and drainage is not included in the design equations. Until recently none of the widely used rigid pavement design methods specifically accommodated the effect of subbase erosion on the pavement performance. Material strengths are specified in the soaked or saturated condition. Designs are based on the stresses, strains and deflections of the pavement slab, while surveys have shown that most pavements failed due to environmental factors, e.g., water in the pavement.

Pumping depends primarily on the number of heavy vehicle axles, slab deflections, amount of water in the

pavement, and the material used as subgrade, subbase and shoulder. If one of these factors can be made more favorable, pumping can be prevented. The traffic volumes and characteristics are usually fixed, while the other three factors can be somewhat controlled. Therefore, a design to minimize pumping must address the controllable factors.

8.2 Review of Rigid Pavement Design Procedures

Although pumping is considered a major problem in the performance of rigid pavements, it has not been explicitly included in the analysis and design procedures. Designs conventionally concentrate on avoiding fatigue due to stresses and deformations of fully supported slabs, and give secondary or no attention to the effects of environmental factors and pumping.

The AASHTO and Portland Cement Association (PCA) design methods are currently the most widely used in the United States. The AASHTO method is used in 21 states and the PCA method in 17 states [Nussbaum1977].

The AASHTO design method is based on semi-empirical relationships concerning serviceability derived from the AASHO Road Test [AASHTO1981]. No provision is made for the inclusion of drainage or pumping, although it is recommended that permeable layers with side drains be used to reduce pore water pressures. A number of gradation

specifications have been given for unstabilized and stabilized subbase layers for rigid pavements. These gradations have been reproduced in Tables 8.1 and 8.2. It is recommended that these layers be extended into the shoulder. It is further recommended that side drains be used with Type A (open-graded) subbase materials. Precautions must be taken to prevent subgrade intrusion into open-graded subbase layers. Lime and asphalt treated material gradations and binder contents should be determined by laboratory analysis, taking into consideration the ability of the stabilized mixture to resist erosion. However, there is no specification on how to measure erosion or what criterion to use. Econocrete (lean concrete) can be used to prevent erosion. The AASHTO specification M155-63(1980) [AASHTO1982] states that a granular material may be used as a subbase under a rigid pavement if the plasticity index (PI) is less than 6, the liquid limit is less than 25, and the amount of material finer than 0.074 mm (No.200 sieve) is less than 15%. The PCA recommends this practice [PCA1971].

The PCA design procedure has recently been modified to address erosion as a cause of failure of PCC pavements [Packard1983]. The initial PCA design procedure was based on flexural stress and flexural strength relationships of concrete, with the intent of preventing fatigue cracking in the slab. The developers of the revised design

Table 8.1 Selected Gradation Specifications for Unstabilized Subbases

SIEVE SIZE (in mm)	1	2	3	4	5	6	7	8
	PERCENT PASSING							
38.1 (1.5 in.)	100	100	100	100	100	100	100	100
25.1 (1 in.)						95-100	70-90	
19.1 (0.75 in.)	60-90	85-100	52-100	60-100		45-65	70-90	44-62
12.7 (0.5 in.)					60-80	10-30	55-80	
9.52 (0.38 in.)			36-65	40-76		0-15		19-38
4.76 (#4)	35-60	50-80	8-40	28-55	40-55	0-5	35-60	8-23
2.38 (#8)					5-25		25-50	4-14
1.19 (#16)			0-12		0-8			2-9
0.59 (#30)			0-8				12-30	1-5
0.42 (#40)	10-25	20-35		8-23				
0.297 (#50)					0-5			0-3
0.149 (#100)				4-15				0-2
0.074 (#200)	0-7	5-12	0-5	0-10			5-10	0-1
Max. PI	NP	6						
Max. LL	25	25						
C _u			min 4					
Thickness(min)			100 mm					
Permeability (cm/sec.)	<0.1 ¹	<0.007 ¹	6.4	0.0004	0.71	>35	0.1	>0.18*

1. AASHTO open-graded (Type A)
2. AASHTO dense-graded (Type B)
3. Pennsylvania open-graded (HP)
4. Pennsylvania dense-graded (2A)
5. New Jersey open-graded
6. Indiana open-graded (#4)
7. Indiana dense-graded (#53)
8. Yoder, recommended gradations to prevent pumping.

* from Moulton 1980 p. 51

C_u = coefficient of uniformity

1 mm = 0.0394 in.

1 cm/sec = 2835 ft/day

Table 8.2 Selected Gradation Specifications for Stabilized Materials

SIEVE SIZE (in mm)	1	2	3	4	5	6
	PERCENT PASSING					
38.1 (1.5 in.)	100	100	100	100		100
25.1 (1 in.)				100	100	70-95
19.1 (0.75 in.)		85-98	54-100	95-100	90-100	55-85
12.7 (0.5 in.)				85-100		
9.52 (0.38 in.)		20-44	35-70	80-90		
4.76 (#4)	65-100	5-12	23-47	15-25	52-88	30-60
2.38 (#8)		0-5	13-26	2-10		
1.19 (#16)				2-5	29-52	
0.42 (#40)	25-50		5-13		13-42	
0.074 (#200)	0-15		0-10	2% filler	0-30	0-15
Compressive strength (MPa)	2.76-5.17					5.17-10.35
Binder content				3% asph		
Permeability (cm/sec.)		2.4	10e-8	0.71		

1. AASHTO cement treated
2. Pennsylvania open-graded asphalt treated
3. Pennsylvania cement treated
4. New Jersey bituminous stabilized open-graded
5. PCA cement treated
6. PCA lean concrete (Type B)

1 mm = 0.0394 in.

1 cm/sec = 2835 ft/sec

1 kPa = 6.9 psi

procedure found that the performance of the pavement can better be described by a "power term" than by the slab deflections [Packard1983]. The power term is defined as a function of the pressure at the slab-subbase interface, the radius of relative stiffness of the slab, the slab corner deflection, the truck speed, and the subbase stiffness. Adjustments can be made for tied-PCC shoulders and dowels. The effect of type of subbase is considered only through its stiffness. Although it is acknowledged that the erodibility of normal subbases is different from that of high strength subbases, the difference is not included in the design.

Lean concrete subbases are considered to be highly erosion resistant. However, erosion may still occur below the lean concrete layer. It is recommended that subbases be extended as least 305 mm (12 in.) into the shoulder. The erosion criterion is suggested for use in conjunction with the conventional fatigue criterion in the design of rigid pavements. It is further suggested that the erosion criterion be modified by local experience, since factors like climate and drainage have not been included. The PCA [PCA1971] recommends the placement of the slab directly on the subgrade if it has less than 45% finer than 0.074 mm (No.200 sieve) and a PI of less than 6 for moderate traffic volumes. No subbase is necessary when the volume of trucks is less than 100 to 200 vehicles per day in both

directions. The AASHTO specification M155 is also recommended for unstabilized subbases. Cement treated layers can be used to prevent subgrade pumping and infiltration. A filter layer (based on the U.S. Army Corps of Engineers criterion) should be used under open-graded layers to prevent subgrade intrusion. The thickness of any type of subbase should be between 102 and 152 mm (4 to 6 in.). Lean concrete subbases can also be used with success [PCA1980]. Table 8.2 gives the recommended gradations for stabilized and lean concrete mixtures.

A number of knowledgeable investigators have made attempts to incorporate pumping into the design procedure. Darter and Barenberg [Darter1977] included the potential erosion of subbase materials in the design of zero-maintenance pavements. They expressed the amount of erosion of the subbase as the width, in inches, of a rectangular strip parallel to the slab edge that has no contact with the pavement. The erodibility depends on many factors, among them, subbase type, drainage, shoulder type, and the environment. Estimated values for the eroded width vary from 0.915 m (36 in.) in cold, wet areas to 0.305 m (12 in.) in dry, warm areas, for dense-graded unstabilized subbases. Values for stabilized subbases in similar areas would be 0.305 m (12 in.) and 0.15 m (6 in.).

The design procedures have also been modified in some states, e.g., California, to include the experience with the effect of pumping on the performance of rigid pavements. Stabilized and unstabilized subbases have been used in states like, Pennsylvania and New Jersey, to improve drainage and prevent pumping. The gradation specifications for these layers are given in Tables 8.1 and 8.2. Table 8.1 also includes typical Indiana specifications.

Finite element techniques have been used to calculate stresses, strains, and deflections in rigid pavements. Most of these techniques, e.g. ILLISLAB and JSLAB, simulate the rigid pavement system as a thin slab on a set of springs (Winkler foundation). Rescourse International Inc. recently (1984) developed a mechanistic design procedure for PCC pavements (RISC model). The slab foundation is characterized as an elastic layered solid of up to 3 layers [Majidzadeh1984a]. Effects of factors like shoulder type and drainage provision, can be included and evaluated. The design includes fatigue cracking and faulting models. A pumping model, although known to be important, was not included, since no models existed.

8.3 Design to Prevent Pumping

8.3.1 Traffic

PCC pavements can be built directly on the subgrade only in cases where the traffic volume is low. The traffic volumes for which the slab can be placed directly on any subgrade varies from 50 design vehicles per day [Yoder1966] to 200 trucks per day [PCA1971]. For higher traffic volumes different suggestions have advanced. Allen [1948] concluded from a survey of rigid pavement that pumping occurred only on subgrade soil with more than 55% finer than 0.05 mm (No.270 sieve) and with a plasticity index (PI) of less than 7. Pumping can be delayed by appropriate compaction of the subgrade. The PCA [1971] recommends the placement of the slab directly on the subgrade if the subgrade has less than 45% finer than 0.074 mm (No.200 sieve) and a PI of less than 6, for moderate traffic volumes. Yoder [1966] indicated that subbases are necessary over subgrade soils classified as USCORPS Group F3 and F4 soils, as well as F1 soils with more than 10% finer than 0.074 mm (No.200 sieve). Group F2 and relatively clean F1 soils are not susceptible to pumping.

8.3.2 Slab Deflections

Pumping can be reduced by eliminating high slab deflections. Low deflections can be obtained by thick

pavement slabs. However, this can be very expensive. Tied shoulders, thickened slab edges, edge beams, and dowels can also be used to reduce deflections. The reduction in deflection due to these measures may be calculated with one of the available rigid pavement analysis programs (JSLAB, ILLISLAB, RISC-model). The inclusion of tied PCC shoulders is equivalent to an increase of 27 to 38 mm (1.1 to 1.5 in.) in pavement thickness [Tayabji1984]. Dowels also reduce slab deflections. The amount depends on the slab characteristics and the dowel efficiency. The new PCA design is the only design method which adjusts the slab thickness based on the erosion potential of the subbase.

8.3.3 Water in the Pavement

The sources of water or moisture in the pavement are: moisture permeating from the sides, a rise in the water table, water from vertical movement in capillaries or interconnected water flows, moisture in the form of vapor, and through cracks and joints in the slab [Temple1984]. Water entering the pavement structure through the ground usually has a small influence on pumping. Ground water should be handled differently from surface water [Kozlov1983].

Most of the water that causes pumping enters through pavement cracks and joints. The permeability of a PCC

slab is only about 10^{-9} cm/sec (10^{-5} ft/day) and therefore any appreciable amount of water must infiltrate through cracks and joints [Ridgeway1982]. Table 8.3 gives a summary of proposed infiltration rates. The pavement edge-shoulder joint is particularly susceptible to the entry of water, especially when the shoulder is higher than the slab [Dempsey1979]. Without water in the pavement, pumping can not occur. Water in the pavement can be controlled either by denying access or by rapid drainage of any water which gains access. Free water can be drained vertically through the subgrade or laterally through a drainage layer. The first is usually not feasible. The second option, although more feasible, requires a very permeable layer since the energy gradient is very low.

8.3.3.1 Sealing of joints and cracks: As mentioned, most of the water that causes pumping enters the pavement through cracks and joints in the slab or at the slab edges. By keeping the slab joints and edges sealed, water can be prevented from entering the pavement. The sealing of joints and cracks has reduced the amount of water entering the drainage system after precipitation on pavements in Georgia and Illinois. No measurable flow could be measured in a section when all the joints and cracks were sealed [Dempsey1979], but this condition can probably not be sustained for a long time. Sealants

Table 8.3 Water Infiltration Rates

AMOUNT OF WATER ENTERING THE PAVEMENT	REFERENCE
0.01 (N + 1 + W/S) (in m ³ /hour/m) N = number of lanes : W = pavement width S = Slab length	Ridgeway 1982
0.05 to 0.67 of runoff (in mm/hour)	Cedergren1974
if crack opening > 0.9 mm 0.70 of runoff (in mm/hour) if crack opening > 3.2 mm more than 0.95 of runoff (in mm/hour)	Markow1983
0.003 (in m ³ /hour/m)	Lui1983
more than 0.30 of runoff (in mm/hour)	Barksdale1977

1 m³/hr/m = 10 ft³/hr/ft
 1 mm = 0.0394 in.

reduce the infiltration of incompressibles into the joints and prolong the life of the joint. Joints with neoprene seals have lower maintenance than unsealed or liquid-sealed joints [Brown1972]. Although new low modulus silicone sealers have performed well for periods in excess of 6 to 8 years (as opposed to an average of 2 years for the widely used rubberized asphalt sealants), it is very difficult to keep all the seals maintained all the time. Evidence is that even with excellent maintenance sealing programs, not all the water can be prevented from entering the pavement [Marks1981, Gulden1983, Temple1984, Majidzadeh1984b]. The water entering the pavement increases with pavement age, since the joint openings do not return to their original size [McGheel1984]. Long slab lengths add to the problem.

Very little quantitative information is available relating sealant conditions to the performance of the pavement [Thornton1977, Ray1980, Minkarah1980, Dempsey1982]. Most of the recent papers discuss the performance of the sealant [Bugler1984, Zimmer1984], and not the effect on the pavement performance. Darter et.al. could not find a correlation between sealant extrusion or stripping and structural maintenance [Darter1977]. A study in Georgia indicated that the sealing of only transverse joints did not reduce faulting [Thornton1977]. Results of a 10-year study in Wisconsin showed that

sealing joints did not improve the performance of the pavement. Both joint spalling and slab cracking were more severe in the sealed than in the unsealed sections. However, the 229 mm (9 in.) PCC doweled pavement showed no signs of faulting after 10 years. Joints in slabs with lengths of 18.3 and 24.4 m (60 and 80 ft) could not be kept sealed. On the other hand, a survey of highways under similar conditions in California showed that the use of sealants reduce faulting. The unsealed section had an average fault displacement of 5 mm (0.2 in.) after 13 years, while the sealed sections had an average fault displacement of only 3.8 mm (0.15 in.) after 17 years [Dempsey1982].

There are many factors that control the effectiveness of sealants in the improvement of pavement performance. Widely accepted general guidelines have not been established, but those presented in a report by the Permanent International Association of Road Congresses (PIARC) in 1979 may be the best available. This report concluded that joints in pavements with joint spacings of 4 to 6 m (13 to 29 ft) can be left unsealed, when: (a) the traffic is light; (b) the traffic is heavy, but the climate is dry; and (c) the traffic is heavy and the climate is wet, but the pavement is doweled [Ray1980]. All cracks and joints need to be sealed and this might not be possible [FHWA1984]. Therefore, provision must be made

to prevent the water from damaging the subbase and shoulder. This can be achieved by proper drainage. Temple [1984] recommended that pavements in areas with precipitation higher than 150 to 200 cm/year (60 to 80 in/year) should have positive drainage in addition to sealing. In areas with rainfall less than 30 cm/year (12 in/year), drainage need only be provided in problem areas.

Tied PCC-shoulders have the advantage of being nonerrodible and reducing deflections, as well as providing an edge joint that can be sealed more effectively than other edge joints. Research on shoulder cracking in Illinois showed that edge joint sealing reduced shoulder cracking [Dempsey1982a].

8.3.3.2 Drainable layers: The most widely used and effective way to drain the water from underneath a rigid pavement slab is with a drainage layer. This drainage layer, placed directly under the slab, can either daylight, or run into a longitudinal edge drain. A 1-hour duration, 1-year frequency storm is ordinarily selected to design the drainage layer [Ridgeway1982]. Darcy's equation is used to predict the flow rate in the drainage layer. Darcy's law is valid only for laminar flow. Kozlov [1983] indicated that turbulent flow probably exists to some degree in open graded layers, but that the Darcy equation gives reasonable estimates of the time needed for a saturated open-graded layer to drain,

assuming peak flow rates hold throughout the drainage period. The design procedures are given in various references. [Cedergren1974, Barksdale1977, Moulton1980, Ridgeway1982] Researchers at the Texas Transportation Institute (TTI) developed a computer program to analyze the drainage in pavements [Liu1984].

All the elements of the drainage system have to be designed properly to be effective. Permeable layers should have the following characteristics:

1. They should be open-graded enough to drain water in a reasonable length of time with flow rates low enough to prevent internal erosion. Various guidelines have been suggested as a reasonable drainage time. Barksdale [1977] suggested that 50% of the free water should be drained in 1 to 5 hours to substantially lower the water level in the vicinity of the pavement-shoulder joint. He concluded that a layer with a minimum permeability of 0.07 to 0.3 cm/sec (200 to 800 ft/day) is required. Kozlov [1983] recommended that 50% of the water should be drained within 24 hours and that the permeability should be larger than 0.35 cm/sec (1000 ft/day). Both stabilized and unstabilized materials can be used. Asphalt and cement stabilized open-graded layers are used in California to provide a nonerrodible, free draining roadbed [Marks1981]. Permeable stabilized

layers usually consist of layers with 1.5 to 3 % asphalt (with or without a stripping agent) [Kozlov1984,Barksdale1977] or porous lean concrete. Fly-ash (2 to 6%) has also been used added to stiffen the mixture [Ridgeway1982]. Stripping can be a problem in asphalt stabilized layers particularly if such stripping causes the layer to lose permeability [Barksdale1977]. Hoffman [1982] recommended that the material smaller than 2 mm (0.079 in.) should be kept to a minimum, since these materials do not add much to the stability, but reduce the permeability and clog the drains.

2. The layer should be dense enough to support traffic loads, both during construction and over the life of the pavement. Barksdale [1977] reported that asphalt stabilized layers with permeabilities between 0.3 to 0.7 cm/sec (800 to 2000 ft/day) have sufficiently high stabilities to support traffic loads. Kozlov [1983] found that open-graded layers with permeabilities less than 1.1 cm/sec (3000 ft/day) have adequate stabilities. Unstabilized open-graded layers with permeabilities up to 6.5 cm/sec (20000 ft/day) were stable during construction. A stable asphalt treated open-graded layer had a permeability of 2 cm/sec (5000 ft/day) [Hoffman1982].

Open-graded layers used in New Jersey and Pennsylvania were assigned a structural coefficient of 0.14. The open-graded layers in New Jersey performed better than bank-run base layers and equal to crushed stone bases (with an overburden of 150 mm (6 in.)) [Kozlov1983]. The present serviceability index (PSI) after construction of a pavement in Pennsylvania with an asphalt treated open-graded layer was the same as that of a pavement with a dense graded cement treated subbase and 0.2 to 0.3 higher than pavements with unstabilized open-graded layers [Hoffman1982].

3. The layer should possess filtration characteristics compatible with the surrounding layers. Filter criteria recommended by the US Corps of Engineers are usually used for aggregate filter materials [Barksdale1977, Ridgeway1982, Kozlov1983]. An example of such a layer that can be placed below the open-graded layer is the NJDOT 1C. Yoder [1966] proposed that subgrade intrusion can be precluded by using n-values for the gradation of less than 1.6 for material with $D=2$ mm (0.078 in.), less than 1.5 for sands, less than 1.0 for material with $D=19$ mm (0.75 in.), and less than 0.8 for material with $D=25$ mm (1.0 in.).

Geofabric can also be used beneath or around the open-graded material. Bell and Hicks [Bell1980] summarized the design criteria for geofabrics as filters by various agencies, viz., USCORPS, Delft Hydraulic Laboratories, Celanese Fibers Marketing Co., and the Ontario Ministry of Transportation. Ridgeway [1982] suggested that the USCORPS criteria be used. Geofabrics have been used mainly on an experimental basis by states like Georgia [Gulden1983], Pennsylvania [Hoffman1982], and Illinois [Dempsey1982]. The performance of the geofabric layers have been marginal. The pulsating nature of the water flow with passing wheel loads seems to be the major problem in the performance of these filters. A leveling coarse is used on top of the open-graded layer in some instances. Yoder [1966] suggested that the leveling coarse should not contain any material passing 0.074 mm (No.200 sieve).

Stabilized layers can also be used underneath the permeable layer. Researchers at the University of Illinois studied the performance of open-graded layers directly on a dense-graded layer, a geofabric, and a lime-fly ash stabilized layer. They found that the dense-graded layer performed better than the geofabric or stabilized layer as a filter [Kozlov1984b].

4. Drainage layers should provide adequate frost protection. A study of the open-graded materials used in New Jersey indicated that they will probably have a very small effect on the maximum frost penetration, and that the modified Bergren equation can be used to calculate the frost penetration [Kozlov1983].
5. Construction of the open-graded layers requires detailed attention. After some experience with the construction of open-graded layers in New Jersey, Kozlov [1984a] recommended the following procedures. The top of the layer underlying the open-graded layer should be stabilized or covered by a geofabric to prevent the intrusion of fines into the open-graded layer. This should be followed by the constructing of the outlet trenches including, the encapsulating geofabric, drainage pipe, and open-graded backfill material. A stationary or portable pugmill can be used to blend aggregate, if necessary. Asphalt stabilized materials should be mixed at temperatures between 130 and 149 C (265 to 300 F). The laydown temperature of the asphalt stabilized drainage layer should be about 121 degrees C (250 F). The drainage layer should be placed with a paving machine or automatic grade controlled stone spreading equipment in order to minimize further surface grading and

material segregation. Lifts should not exceed 100 mm (4 in.). Compact the unstabilized layer with vibratory rollers and the asphalt stabilized layer with 3-wheel and tandem rollers. A control strip should be used to assure adequate compaction. The surface of the unstabilized drainage layer should be protected with a prime coat of 0.15 to 0.35 gal/sy). Traffic should not be allowed directly on the open-graded layer, and care should be taken not to contaminate the layer during construction.

8.3.4 Pumping Resistant Materials

The use of subbase and shoulder materials that are not pump susceptible will also prevent pumping. Fines are removed through surface erosion and/or the pore water pressure build up within the layer. Surface erosion is the more important mechanism for dense stabilized materials, since free water does not readily penetrate such materials. Permeabilities of these stabilized layers are in the order of 10^{-7} cm/sec (0.0003 ft/day).

Fines are removed from unstabilized materials through surface erosion and/or pore water pressure build up, depending on the permeability of the layer. If the material is dense, some free water will penetrate the material, while some will be moved on the surface of the layer. On the other hand, if the material is more

permeable, most of the water may enter the material and pore water pressure build up will cause pumping.

Raad [1982] showed that the pore water pressure build up is related to the subbase permeability and compressibility. Liquifaction of saturated granular materials occurs when the residual pore water pressure becomes equal to the vertical effective stress. Additional load repetitions could then result in the ejection of subbase material. The pore water pressure is increased by an increase in the compressibility of the subbase and a decrease in the permeability of that layer. When the permeability is high enough, the pore water pressure will be too low to cause pumping.

Other research has also shown that subbase layers with a large enough permeability do not pump. Yoder [1966] concluded from a laboratory study that gravel with a n -value of between 0.7 and 1.2 will not pump. (The n -value is the exponent in the equation $P=100 (d/D)^n$, with P the percentage of particles finer than d and D is the maximum gravel size.) Using a prediction equation given by Moulton [1980], the permeability of these gradations will be more than 0.3 cm/sec. (800 ft/day). Dempsey [1982] tested subbase materials with permeabilities of 0.0002, 0.008 and 0.2 cm/sec. (0.6, 23 and 570 ft/day). Only in the subbase with a permeability of 0.2 cm/sec. (570 ft/day) did the pore water pressure dissipate fast enough

to prevent pumping. The pore water pressure was less than 2.1 kPa (0.3 psi), while the pressures in the denser subbases were more than 21 kPa (3 psi). Dynamic pore water pressures measured in the model study of materials used in New Jersey were about 0.5 kPa (0.07 psi), and there was no residual pore water pressure. The residual pore water pressure in the dense graded gravel was 2.2 kPa (0.32 psi), which is high enough to cause pumping at low confining pressures [Kozlov1984a].

The AASHTO specification M155-63(1980) [AASHTO1982] specifies that a granular material may be used as a subbase under a rigid pavement if the PI is less than 6, the liquid limit is less than 25, and the amount of material finer than 0.074 mm (No.200 sieve) is less than 15%. The PCA recommends this practice [PCA1971], but the permeability seems to be too low. Surveys have shown that subbases with more than 10% fines are subject to pumping [Moore1981]. Roads built in Virginia on subbases with 7 to 8% fines performed better than those built on subbases with 12% fines [McGheel1984]. Tables 8.1 and 8.2 contain a selected number of proposed and applied gradations for rigid pavement subbases.

Stabilized subbases are more subject to surface erosion than to removal of fines from within the layers, as indicated earlier. Both asphalt and cement stabilized subbases are subject to erosion. The erodibility is

increased by severe environmental conditions, e.g., freezing and thawing cycles. The environmental effects diminish with increase in curing time, cement content, and compaction effort. Asphalt stabilized subbases are further subjected to stripping by water over time. Erodibility of stabilized materials can be reduced by increasing the binder content. Chapter 7 contains a detailed discussion of the erosion of stabilized materials. Relationships are presented which can be used to select a cement or asphalt content high enough to prevent surface erosion. This may not help in all cases, since during construction the surface of the cement stabilized subbase is usually trimmed to the correct level and slope. This can produce loose particles on the surface of the cement stabilized layer. These loose particles are easily eroded, even when the cement content is sufficiently high. This condition can be eliminated by leaving the subbase surface untrimmed.

Asphalt cement and lean concrete materials are basically nonerodible. Asphalt cement can be used as a subbase or to cap an erodible cement treated layer. The thickness of the cap is usually 25 mm (1 in.) [Gulden1975, Woodstrom1983]. Lean concrete layers are mixed and placed like regular concrete layers. They usually have higher cement contents than cement stabilized layers, which make them less erodible.

8.3.5 Shoulders

The erodibility of materials used in the shoulders are important since pumping of the shoulder frequently takes place, mainly through surface erosion of particles. The same criteria discussed above for the subbase are applicable.

Tied PCC, normal lean concrete, porous lean concrete, asphalt cement, and permeable stabilized shoulders will virtually eliminate erosion, while regularly stabilized shoulders should keep it to a minimum. PCC shoulders perform as well or better than full depth asphalt cement shoulders, in addition, they reduce slab deflections and benefit joint sealing. Asphalt cement shoulders perform better than cement or pozzolan stabilized shoulders [Majidzadeh1984].

8.3.6 Water Collecting System

Another very important aspect of the drainage of pavements is the design of the water collecting systems. As mentioned, the drainage layer can either be allowed to daylight or drain into a longitudinal drain. The design of these longitudinal drains includes the angle of the outlets, the location, spacing and arrangement of the collectors, the geofabric used, the pipe and slot sizes, and the material to be used as backfill. Guidelines on these designs are given in detail elsewhere

[Barksdale1977, Kozlov1984a]. Kozlov [1984a] recommended that a longitudinal drain be used (instead of the daylighting of the drainage layer) due to problems with the clogging of the daylighted zone.

Longitudinal (edge) drains are used to drain the water away from the pavement. Edge drains are often used with stabilized subbases to remove the water from between the slab and the subbase. They should be provided when the average annual precipitation is more than 250 mm (10 in.) or the equivalent single axle load (ESAL) value is more than 250 per day during the design life of the pavement [Majidzadeh1984a]. Edge drains will not be effective if the subbase is erodible or has a low permeability. (See the Section on rehabilitation and retrofit drains in Chapter 10).

CHAPTER 10
PUMPING RELATED DISTRESSES AND
REHABILITATION TECHNIQUES

10.1 Introduction

Pumping causes fines to be removed or redistributed underneath the slab. The removal of the material changes the support conditions of the slab by the creation of voids. Slab deflections and stresses are increased, which lead to cracking and slab breakup. The redistribution of fines, usually from the leave slab to the approach slab, causes faulting and uneven slab support conditions. Another consequence of the pumping of fines is the infiltration of incompressible material into cracks and joints. This restricts the movement of the slab and causes joint spalling and blowups. Pumping of the shoulder material can cause shoulder depressions, opening of edge joints, and additional buildup of joint material under the slab and in the joints.

The classical progression of jointed rigid pavement failure follows five stages.

1. Stage A: Water infiltration and small shoulder depressions.
2. Stage B: More water infiltration, causing larger shoulder depressions and some faulting. Fines appear on the shoulder.
3. Stage C: Voids are formed. Faulting increases and more fines appear on the shoulder. Joint spalling and small cracks become apparent.
4. Stage D: Faulting becomes serious, extensive amounts of fines appear on the shoulder, and joint spalling becomes severe. Blowups, extensive cracking, and slab crack heave occur.
5. Stage E: Complete pavement failure.

10.2 Distress Types

Detailed descriptions of rigid pavement distresses can be found in many other publications (see Appendix B). The distress types as they relate to pumping can basically be divided into two categories, viz., the distresses which are caused primarily by pumping, and those in which pumping is a contributing factor.

10.2.1 Caused Primarily by Pumping

10.2.1.1 Structural cracking: Corner, diagonal, longitudinal, and transverse cracking may occur due to the loss in slab support. The existence of voids induces excessive deformations of the slab, with resulting stresses in excess of the flexural strength of the concrete. All these cracks can extend vertically through the entire slab thickness.

10.2.1.2 Faulting: Faulting is defined as the difference of elevation across a crack or joint. Faulting occurs due to the build-up of fines underneath the approach slab and the existence of voids. The fines come from the erosion of subbase and shoulder material in the presence of water. The amount of fines produced by the joint abrasion and the movement of the slab on the subbase in the dry condition is negligible [Ray1977].

10.2.1.3 Shoulder depressions: Shoulder depression (lane-shoulder dropoff) is defined as the difference in elevation between the traffic lane and the shoulder. Shoulder depressions are caused by the removal of the underlaying material through pumping and/or settlement of shoulder materials. Pumping related shoulder depressions typically appear next to transverse joints and cracks. These depressions can extend up to 0.5 m (19 in.) from the outer edge of the pavement.

10.2.1.4 Slab rocking: Voids on both ends of the slab can cause the slab to move around a supported area (fulcrum) in the middle of the slab.

10.2.2 Pumping as a Contributing Factor

10.2.2.1 Joint spalling: The accumulation of incompressible material in the joints, and excessive slab movements, can cause joint spalling due to stress intensification at the joint. Spalling does not extend vertically through the slab. The incompressible materials are produced by the pumping action.

10.2.2.2 Sealant failure: Incompressible material in the joints and excessive joint movement can also cause sealant failure in the form of stripping or extrusion of sealants.

10.2.2.3 Durability cracking: Durability cracking (D-cracking) is caused by freeze-thaw expansive pressures and other moisture related damage of certain aggregates. It typically begins at the bottom of the slab. The accumulation of water due to pumping and voids accelerates this process of aggregate deterioration.

10.2.2.4 Blow-ups: Slab blow-ups can also be caused by incompressible material in the joints. Blow-ups typically occur during the warm spring and summer months.

10.2.2.5 Load transfer damage: Excessive joint movements can damage load transfer devices or reduce the aggregate interlock.

10.3 Rehabilitation Techniques

Rehabilitation techniques can be used to correct pavement distresses and thereby improve the condition (PSI) of the pavement. These activities may or may not have an influence on the rate of pavement deterioration. Rehabilitation techniques can be used to prevent or reduce the occurrence of pavement distresses and thereby reduce the rate of pavement condition deterioration.

The pavement condition is often measured in terms of present serviceability index (PSI). For rigid pavements, the traditional PSI relationship is a function of slope variance (due to roughness), the amount of patching, and the length of cracking. The types of distresses which influence the PSI are faulting, blowups, slab rocking, and cracking. Joint spalling usually requires patching and can also increase the roughness. Therefore, if faulting, blowups, slab rocking, or cracking is removed, the PSI is improved. The succeeding section will briefly discuss the rehabilitation techniques related to the correction and prevention of pumping related distresses, emphasizing their effect on the condition and performance of the pavement. PCC rehabilitation techniques have been

described and discussed thoroughly in other publications (see Appendix B) and will not be repeated in detail here. Only the rehabilitation techniques related to pumping and pumping distresses will be described. Some techniques improve the condition of the pavement, others reduce the rate of condition deterioration of the pavement, and a few will do both. The sequence in which these rehabilitation activities is applied is important. A typical sequence may exist of slab replacement, subsealing, grinding, crack and joint sealing, and the installation of drainage systems.

Pumping related rehabilitation techniques can loosely be divided into three groups, viz., mainly correct pumping related distresses, primarily prevent or reduce further pumping, or achieve both. The classification is subjective, since relatively little quantitative information is available on the effects of different rehabilitation techniques on pumping progression. Table 10.1 summarizes the influence of the different rehabilitation techniques on the distresses.

10.3.1 Corrective Techniques

10.3.1.1 Grinding: Diamond grinding or surface milling improves the skid resistance and rideability of the pavement by removing faulting and other roughnesses. Diamond grinding can not be used economically when the

Table 10.1 Effect of Rehabilitation Techniques on Pavement Distresses

ACTIVITY	PUMPING	FAULTING	CRACKING	PSI
Grinding	N	C	N	C
Shoulder repair (2)	R(3)	R(3)	R(3)	R(3)
Recementation	N	N	C	C
Underseal	C,R(4)	R(4)	R	C,R
Retrofit drains	R	R(6)	R(6)	R(6)
Crack sealing	R(6)	R(6)	R(6)	R(6)
Edge support	R	R	R	R
Load transfer improvement	R	R	R	R
Relief joints	N	N	R	R
Full depth without LTD	C	C	C	C
with LTD	C,R	C,R	C,R	C,R
Partial depth without LTD	N	C	C(1)	C
with LTD	R	C,R	C(1),R	C,R
Resurfacing	R	C,R	C,R	C,R
Mudjacking	C,R(4)	C,R(4)	R(5)	C,R

LTD = load transfer device

C = correct distress (at least partially)

R = change rate of distress propagation

N = no influence on distress

- (1) Only partially repair cracks.
- (2) Shoulder repair improves the safety.
- (3) Shoulder repair might reduce the amount of water entering the pavement.
- (4) Grout is erosion resistant, but erosion of the underlying subbase still takes place. The combined effect is unknown.
- (5) Mudjacking can cause cracking during application.
- (6) The effect on the rate of deterioration depends on the pavement condition and environmental effects.

faults are larger than 6 mm (0.25 in.) [ACPA1983]. Surface milling has also been used with satisfactory results.

The reported performance ranges from fair to good. These methods reduce the roughness and do not change the rate of pumping. The Mays roughness can be reduced to 793 mm per km (50 in. per mile) on the average [NCHRP1983].

10.3.1.2 Shoulder repair: Shoulder depressions can be corrected by levelling the depressions with bituminous materials. This will not improve the PSI, but will improve the safety and may reduce the amount of water entering the pavement edge-shoulder joint.

10.3.1.3 Recementation of cracks: Recementation of cracks has been used to restore the structural integrity of cracked slabs. Recementation is accomplished by injecting a liquid epoxy under pressure into the cracks [Barenberg1981]. The pavement condition will be changed since the cracks are removed, but pumping and faulting rates may not be greatly changed. The egress of water into the cracks should be greatly reduced. The performance of recemented cracks has been very poor to good.

10.3.2 Preventive Techniques

10.3.2.1 Undersealing: The loss of subbase support is corrected by the filling of the voids with liquid asphalt or cement grout mixture in a process called undersealing, subsealing or stabilization. Voids under joints, cracks, or at the slab edges can be filled. Holes are often drilled through the subbase to fill voids between the subbase and subgrade as well. Different numbers of holes and hole patterns per joint or crack have been used. In Indiana, one hole 0.9 m (3 ft) from the joint or crack in the leave slab is used, while in Illinois on an experimental section, three holes in the leave slab and two in the approach slab (when necessary) were used. The holes in the latter case were 51 mm (2 in.) in diameter and drilled 102 mm (4 in.) into the subbase. The cement grouting was stopped when the upward slab deflection was more than 1.3 mm (0.05 in.) or when grout ejected [Slifer1985]. In Indiana, asphalt cement is used to fill the voids and an upward deflection criterion of 6 mm (0.25 in.) is used.

Undersealing will be successful only if care is taken that the void is not overfilled and new voids thereby created. Undersealing alone does not stop pumping. It should be supplemented with sealing or retrofit drains [Thornton1980]. The filling of voids will restore the slab support conditions, but may not reduce subsequent

pumping or erosion. Subsealing does not increase the initial design structural capacity or eliminate faulting. The PSI is not changed by undersealing, but the rate of cracking will be changed due to the restoration of the slab support conditions. It is conceivable that undersealing will effect the rate of pumping, at least initially, since the slab deflections are reduced. However, Gulden [1983] reported that, although the cement-limestone dust used as grout is durable and erosion resistant, erosion still took place under the grout. It is fairly well established that undersealing does not stop erosion, but how it influences the rate of erosion is still unresolved.

The grout used to fill the voids does not clog the underdrains [Temple1984]. Voids to be undersealed are usually detected by means of slab deflection measurements. The Indiana method [Muttil1985] and the method developed by Crovetti and Darter [Crovetti1984] are probably the best available.

10.3.2.2 Retrofit drains: Retrofit (edge, longitudinal, or trench) drains have been used to reduce pumping and faulting. Retrofit drains increase the drainage of water from the pavement and can improve the shoulder stability. These drains should be used in conjunction with other rehabilitation activities, e.g., undersealing, grinding, resurfacing. The installation of retrofit drains is a

preventive measure, since the condition of the pavement is not changed. However, the drains can reduce the rate of deterioration and prolong the life of the pavement.

The effects of retrofit drains on the pavement performance vary. In New York, New Jersey, and Georgia, retrofit drains have not been successful in reducing pumping and faulting.

Underdrains used since 1981 in the state of New York had no influence on faulting. The subbases were all dense graded aggregate [Moore1981]. Edge drains were not successful in reducing faulting and pumping after 10 years of use in New Jersey, due mainly to the clogging of the open graded material with fines from the pumping pavement slab [Kozlov1983]. Edge drains have not been used since 1978 (280 lane miles) in Georgia due to problems with existing drains at that time. The drains removed the visible signs of pumping, but did not reduce the rates of faulting and cracking. The drains actually increased the removal of fines from the subbase in some cases. The geofabric used to protect the edge drain kept the fines out of the drain, but became clogged, which reduced the permeability. The permeability of the geofabrics was inadequate for the amount of water that had to pass during a wheel load, but adequate for drainage at other times [Gulden1983].

Retrofit drains have been successful in California, Iowa, Louisiana, and in Illinois.

Properly designed and constructed edge drains reduced the rate of faulting and slab breakup in California. Attempts to envelope permeable material with a geofabric and to use unstabilized permeable material failed. Drains became plugged on old highways with moderate faulting on flat grades (less than 0.2%) within one year [Woodstrom1983]. Edge drains were the most effective when used on pavements which were generally in good condition, with very little or no faulting, but starting to show pumping. Drains extended the life of the pavement from 20-25 years to 30-35 years. The rate of faulting after the installation of the drains was found to be about an eighth of the rate before installation. A typical example would be a rate of 0.05 mm/yr (0.002 in./yr) for an initial rate of 0.43 mm/yr (0.017 in./yr) [Ames1985].

Faulting on I-84 in Iowa was less on the sections with edge drains than on the sections without edge drains. Drains worked well on pavements with waterbound McAdam and cement stabilized subbases. No significant change in rate of deterioration or faulting was observed after the installation of edge drains due to improper drainage trench depths and the state of existing faulting (average fault was 4.6 mm or 0.18 in.). The best results were obtained with the top of the drainage pipe at least 125 mm

(5 in.) below the bottom of the slab [Marks1981,Ridgeway1982].

Seventy five percent of the edge drains installed in Louisiana performed well on pavements with cement stabilized subbases and no seals after 5 years. An asphalt stabilized drainage layer used in the shoulder performed well after 7 years in use. The effect of drains on PSI could not be measured, but they were not effective when an abundance of pumpable fines was available. The concrete drainage cap used to cover the drain tended to shrink and crack, and increased the intrusion of water into the drain [Temple1984].

Darter reported that pavements with retrofit drains performed better than pavements without these drains in Illinois [Moore1981].

Ray [1983] reported that the experience in France has also been mixed. Retrofit drains can increase the removal of fines, due to the loosening of material during construction of the drains. Retrofit drains can reduce faulting and cracking if the subbase is not highly erodible. Drains should not be used on pavements with erodible subbases, in high rainfall areas under heavy traffic, or when the subbase is broken and rests on an unstabilized subgrade under heavy traffic. The particle removal rate from the subbases can be increased by drains

along pavements with subbases with high pumping and erosion rates. These particles will clog both filters and drains. Slab deflections can be increased soon after the placement of retrofit drains due to the removal of blocked fines. Ray [1983] recommended that trenches should not cut through the subbase, and thereby reduce the edge support. A cut depth of 10 to 35 mm (0.4 to 1.2 in) into the subbase is recommended.

Prefabricated systems have also been used. These systems are easy to install and are in general not more expensive than conventional systems.

The difference in performance of retrofit drains seems to depend on the condition of the pavement and the properties of the subgrade. Unstabilized subbases were mainly used in the states where the retrofit drains did not perform well, while cement stabilized subbases were mainly used in those states where the drains performed better. The following factors seem to be important in the design and performance of retrofit drains.

1. The type and condition of the subbase: Retrofit drains used in pavements with pump and erosion susceptible subbases will probably not be effective. Drains will increase the rate of water flow over or through the subbase which can increase the erosion. This can even be more pronounced in high rainfall

areas. Eroded material clogs the drains. This can cause an accumulation of water under the slab. The same criteria used in the design of nonerrodible stabilized layers should be used to determine if retrofit drains will perform properly.

Dempsey [1982a] gave guidelines for use of retrofit drains for rigid pavements with unstabilized subbases. He stated that retrofit drains will have little influence if the permeability of the existing subbase is less than 0.009 cm/sec. (26 ft/day), but will have appreciable influence on the pore water pressure and pumping when the permeability is more than 0.09 cm/sec. (260 ft/day).

Retrofit drains are not recommended for in pavements with high pumping and faulting without other remedial measures, since the drains will increase the removal of fines. Retrofit drains are recommended for pavements in good condition in areas with high rainfall and heavy traffic.

2. Pavement condition: Badly deteriorated pavements with high deflections and broken slabs will probably not benefit from subdrains.
3. Design of the drains: The design of the filter layers, the backfill material, the collector pipes, and outlets is important. Drainage of water from all

the relevant drainage paths should be accommodated. Improper design is one of the major causes of the ineffectiveness of retrofit drains. The design aspects have been discussed in detail in a number of reports [Temple1984,Kozlov1983]. The performance of geofabrics as filters has not been verified. Care should therefore be exercised in the use and selection of a geofabric. Indications are that stabilized backfill material in the drains improve their effectiveness [Woodstrom1983].

4. Construction and maintenance: Care should be taken during construction not to disturb the slab and weaken the support conditions, especially along the sides. Maintenance of the drainage outlets is important to the performance of the retrofits drains. Edge support can be retained by not cutting drains through the subbase.

10.3.2.3 Surface sealing: The sealing of cracks and joints will only be effective when the slabs are stable. As discussed in Chapter 8, very little quantitative information is available relating sealant conditions to the performance of the pavement [Thornton1977, Ray1980, Minkarah1980, Dempsey1982a]. Most recent papers discuss the performance of the sealant [Bugler1984,Zimmer1984] and not the effect on the pavement performance. Although it is theoretically possible to keep the water out by

sealing all the cracks and joints, this does not seem to be practical. However, sealing of joints and cracks, does limit the entry of water into the pavement and prevents the infiltration of incompressible material into the joints. Sealing will not be successful if the slab is badly deteriorated or when the sealant is of poor quality. Pavements in areas with little water or with good subdrainage will benefit little from sealing. Sealing usually leads to an extension of the pavement life, at least for a few years, if scheduled and constructed properly [NCHRP1983].

Sealants can be divided into three groups, viz., hot poured elastomeric (rubberized asphalt, liquid asphalt), low modulus silicone, and preformed compression seals. Their performance depends on the shape factor, the joint spacing, the physical properties of the sealant, the condition of the joint, and the installation. The life expectancies of the sealants range from 1 to 5 years for the hot poured sealants to about 10 years for the other two types [Majidzadeh1984,Gulden1983,Darter1977]. The sealing of cracks and joints does not change the condition or roughness of the pavement. The performance of sealing has been very poor to excellent.

10.3.2.4 Provision of edge support: Slab deflections can be reduced by providing edge support. Edge support can be in the form of a tied PCC-shoulder or the installation of

an edge beam. Tied PCC-shoulders with 100% load transfer reduce the deflection and stresses of the slab corners by 50%. A 600 mm (24 in.) wide edge beam can also reduce slab deflections by at least 50%. A wider beam will reduce the deflections even more [NCHRP1983]. An analysis [Tayabji1984] has shown that tied PCC-shoulders have the same effect on the deflections as an increase of 25 to 37 mm (1 to 1.5 in) in slab thickness. The same criteria are appropriate with regard to design of shoulders on new pavements as for the design of PCC-shoulders as a rehabilitation measure.

The provision of edge support does not change the condition (PSI) of the pavement, but it does change the rate of pumping, faulting, cracking and thus pavement deterioration. The general performance of this type of rehabilitation has been fair to good.

10.3.2.5 Load transfer improvement: Perfect load transfer can reduce stresses and deflections to half that of a pavement with no load transfer. All faulted joints and cracks with load transfer of less than 50 to 60%, when measured in the early morning, should be provided with load transfer devices [NCHRP1983]. A number of different load transfer devices are available, e.g., Vee, Double Vee, Figure Eight, Georgia split pipe device, and dowels. Gulden [1985] found that dowels were the most successful in Georgia.

The short term experience with load transfer restoration has been satisfactory. However, the long term performance has not been established. Load transfer devices will not change the PSI, but should reduce slab deflections and therefore reduce the pavement deterioration rate.

10.3.2.6 Pressure relief joints: Pressure relief joints relieve the stresses in the slab due to restriction of the slab movement. A relief joint usually consists of replacing a piece of the slab with a strip of asphalt material. Neither the PSI nor the rate of pumping, faulting, or cracking will be changed significantly.

10.3.3 Corrective and Preventive Techniques

10.3.3.1 Full depth slab repair: Full depth patching is used to correct badly deteriorated joints and cracked slabs. Joint deterioration that extends deeper than one half the slab thickness, e.g., D-cracking at the bottom of the slab, should be corrected with a full depth patch [NCHRP1983]. Full depth slab repairs with Portland cement can be made with or without the installation of new load transfer devices. Full depth patches may consist of rectangular patches or inverted T-sections. Care should be taken that the patch does not trap water underneath the pavement. This will increase pumping if the subbase is erodible. A porous bedding over a geofabric should be

placed beneath the patches to improve drainage [Marks1981]. Full depth patches with dowels will reduce the rate of pumping. The general performance of full depth patches ranges from poor to excellent.

Full depth bituminous patches have been used as a temporary repair or as part of an overlay project. These patches have been reported to last up to six months in Virginia [FHWA1983].

10.3.3.2 Partial depth slab repair: Partial depth repairs are used to repair joint deterioration, usually spalling. They will extend the life of the pavement by improving the roughness, reducing blow-ups, and improve the performance of the sealants. A durable patch when properly placed should endure for the remaining life of the pavement [NCHRP1983]. As in the case of full depth slab repair, partial repair can be accomplished with and without the installation of load transfer devices. Repairs with the installation of load transfer will likely reduce the rate of pumping by reducing deflections. The performance of partial depth repairs ranges from poor to very good.

10.3.3.3 Resurfacing: Asphalt or PCC overlays may be used. Asphalt overlays are often placed on crack relief layers or geofabrics to prevent reflection cracking. PCC overlays can be unbounded, partially bonded, or fully

bonded. Overlays reduce the roughness of the pavement and increase the thickness of the slab. Overlays should change the rate of pumping due to lower slab deflections.

10.3.3.4 Mudjacking (Slab jacking): Mudjacking can be used to, besides filling the void, also improve the rideability by reducing the faulting. Slabs can be lifted up to 9 mm (0.35 in.) [Martin1981]. Problems with slab cracking have been experienced during the mudjacking operation, but the effect on rideability has been satisfactory in Pennsylvania. Some slab movement and pumping still existed after mudjacking [FHWA1983]. This method has not been successful in California [Ames1981]. The general performance is very poor to good. Slab jacking will in general not reduce the pumping rate significantly, but will reduce the rate of cracking by restoring the slab support conditions.

10.4 Recycling and Reconstruction

Methods like breaking and seating and recycling have been used to restore the initial condition of the pavement. These methods are applied when complete failure is reached (Stage E), and rehabilitation measures are not sufficient to improve the pavement condition.

CHAPTER 11

PUMPING PREDICTION MODELS

11.1 Introduction

Pumping prediction models are essential in the analysis and design of PCC pavements. These models allow the determination of void development and indicate what maintenance is required and when it should be applied.

The only models that are currently available to quantify the volume of pumped material are two based on the AASHO Road Test data [Markow1984, Larralde1984]. Other researchers [Rauhut1982, Darter1983] have used pavement performance data to develop a regression equation to predict a severity level for pumping.

Majidzadeh et al.[1984a] reviewed all available distress models in their recent study on the "Mechanistic Design of Rigid Pavements" and also concluded that an adequate pumping model is not available. Therefore, a pumping model was not included in their analysis.

A large number of factors influence pumping and should be included in any useful pumping prediction model. These are listed as follows.

1. Slab properties: Slab deflection and the velocity of deflection influence the velocity of the water moving between the slab and subbase, and the magnitude and rate of pore water pressure development. The slab thickness, slab length, amount of load transfer, modulus of the concrete in the slab, and magnitude of reinforcement will all influence the amount and velocity of slab deflection. The slab properties, along with the subbase support values, will also determine the area of the slab that is not in contact with the subbase.
2. Subbase properties: The modulus properties of the subbase, in combination with the subgrade modulus, will influence the slab deflections. Unstabilized subbases are subjected to pumping by the build up of pore water pressure and surface erosion, depending on their permeability. Stabilized subbases are mainly susceptible to pumping through surface erosion. Therefore, the permeability, surface erosion, and strength characteristics of the subbases are critically important in the development of a pumping model.

3. Pavement drainage properties: Pumping is generally a lesser problem in pavements with good drainage. Water is necessary for pumping to occur, and minimizing the quantity of water and the amount of time it resides in the pavement will reduce pumping. The size of cracks and joints, and the condition of the sealants strongly influence the amount of water entering the pavement.
4. Environmental factors: Environmental factors are critically important elements of a pumping model. The amount and distribution of precipitation determine the availability of water. The temperature changes influence the slab deflection. The number of freeze-thaw and wet-dry cycles affect the erosion properties of the pavement materials.
5. Traffic properties: The frequency and number of axle loads influence the slab deflection and the velocity of deflection. The distribution of axle loads with time of day can also be important. Load applied early in the morning, with the slab in the curled position, can be more severe than loads applied during the middle of the day.

The ideal pumping model should include all these factors. Unfortunately, information regarding all these elements are not readily available. Three different procedures

were applied in an attempt to develop an adequate pumping model, viz., (a) applying theoretical relationships, (b) using highway agency underseal quantity data, and (c) using the AASHO Road Test data.

11.2 Existing Pumping Models

11.2.1 Based on AASHO Road Test Data

The AASHO Road Test provides the sole source of data on the actual amount of pumped material under controlled conditions. This is by no means the ideal data, but the best available. Values for factors, like the effect of the sealant and the origin of the pumped fines (subbase or shoulder), might have affected the volume of pumped material, but could not be identified. The volume of pumped material was measured during the AASHTO Road Test and normalized as pumping index values. A distinction was made between nonreinforced (4.6 m or 15 ft slabs) and reinforced (12.2 m or 40 ft slabs) pavements. The slab thickness, slab length, subbase thickness, and traffic were varied in the Road Test.

A model, using the AASHO Road Test data, was developed for use in the EAROMAR Version 2 program [Markow1984]. In this model, the pumping index (P_i) is related to slab thickness and cumulative equivalent 80 kN (18000 lb) single axle loads (ESAL). Arbitrarily selected drainage adjustment factors were included in the model.

Table 11.1 Existing Pumping Prediction Models

Model 1: [Reference: Markow1984]

$$P_i = m * \Sigma \text{ESAL} * f_d$$

$$\log m = 1.07 - 0.34 * D$$

where

P_i = pumping index

D = slab thickness (in.)

ΣESAL = cumulative equivalent 80 kN (18000 lb) single axle loads

f_d = drainage adjustment factor

= 0.2 for good drainage ($k = 10000$ ft/day)

= 0.6 for fair drainage ($k = 100$ ft/day)

= 1.0 for poor drainage ($k = 0.1$ ft/day)

k = subbase permeability

$$1 \text{ cm/sec} = 2835 \text{ ft/day}$$

Model 2: [Reference: Larralde1984]

$$\text{NPI} = \exp [-2.884 + 1.652 \log(\Sigma \text{ESAL} * \text{DE} / 10,000)]$$

where

NPI = normalized pumping index (in.³)

DE = deformation energy per application (in.-lb)

ΣESAL = cumulative 80 kN (18000 lb) equivalent single axle loads

Table 11.1, continued

Model 3: [Reference: Rauhut1984]

For nonreinforced jointed PCC pavements (JPCP):

$$\ln p = 1.39 * \text{DRAIN} + 4.13$$

$$\beta = \frac{0.772(D-2.3)^{1.61}}{\text{PPTN}} + 0.0157 * \text{JLTS} * D + 0.104 * \text{STAB} \\ + 0.17 * \text{DRAIN} + 0.137 * \text{SOILTYP} - 0.247$$

For reinforced jointed PCC pavements (JRCP):

$$\ln p = 1.028 * \text{STAB} + 0.0004966 * D^{3.47} - 0.01248 * \text{FRINDEX} \\ + 1.667 * \text{CBR} + 5.476$$

$$\beta = -0.01363 * \text{DMOIST} + 0.02527 * D - 0.423$$

$$g = \left(\frac{\text{ESAL}}{\rho} \right)^\beta$$

where

g = amount of distress (damage) as a fraction of a
pumping level of 3 (severe)

$\text{DRAIN} = 0$ no underdrains
1 underdrains

PPTN = average annual precipitation (cm)

$\text{JLTS} = 0$ undowelled
1 dowelled

$\text{STAB} = 0$ unstabilized subbase
1 stabilized subbase

$\text{SOILTYP} = 0$ granular foundation soil
1 coarse foundation soil

DMOIST = Thornthwaite moisture index

FRINDEX = freezing index

CBR = California bearing ratio of foundation soil

D = slab thickness (in.)

ESAL = cumulative equivalent 80 kN (18000 lb) single
axle loads

Table 11.1 contains this model (Model 1).

A second model has been developed by Larralde [1984]. He correlated the amounts of deformation energy imposed by one application of a 80 kN (18,000 lb) single axle load with the normalized pumping indices to obtain a pumping potential model. The pumping indices were normalized to eliminate the effect of slab length and reinforcement. The amounts of deformation energy were computed using a finite element technique. The model is presented as Model 2 in Table 11.1.

These pumping prediction equations do not include all factors that might significantly influence pumping. Both authors realized this and suggested that correction factors be developed.

11.2.2 Regression Analyses

Darter [1983] and Rauhut [1982] developed pumping prediction models from in-service distress observations. Pumping is predicted as a severity or damage level. The Rauhut model is the more comprehensive of the two and used data from six states contained in the Concrete Pavement Evaluation System (COPES) data bank. Two models, one for plain (JPCP) and one for nonreinforced (JRCP) rigid pavements, developed by Rauhut [1982], are presented in Table 11.1 as Model 3. The JPCP equation included precipitation, drainage, subbase type, subgrade type, load

transfer, slab thickness, and traffic. The JRCP equation included, subgrade modulus, freezing index, Thornthwaite moisture index, subbase type, slab thickness, and traffic.

11.3 Theoretical Considerations

A theoretical model provides a means to include all the important factors in a pumping model. The rotational shear device provides information that can be useful in the development of a model to predict the volume of eroded material. A complete theoretical model is not presented here, since not enough information is currently available to include all the required parameters. However, elements of such a model and the use of the rotational shear device results in such a model will be discussed. A model based on results of the rotational shear device will be valid only for the prediction of the erosion of essentially impermeable material (usually stabilized layers), where erosion is the major mechanism of pumping. In more permeable materials, pore water pressure buildup is the major mechanism of pumping. A different theoretical model will be needed in such a case.

A characterization of the magnitude of slab deflections, velocity of slab deflections, and the movement of water under the slab is required in the theoretical analysis. Water accumulates between the slab and impermeable subbase when the slab is in the curled

position or when a void exists. The water enters the pavement mainly through cracks, slab joints, and shoulder joints. Very little of the water that enters the pavement structure through the shoulders and subgrade accumulates between the slab and subbase. Water will therefore only be present during and shortly after a rainfall or during snow and ice melt periods. Some of these factors have been described in Chapter 7. Erosion takes place under the slab when the shear stress of the water is larger than the critical shear stress of the subbase or shoulder material. This may occur every time a heavy vehicle crosses the joint. A theoretical pumping model could take the following form.

$$V_v = \Sigma \text{ESAL} * t * E_r * \text{Area} * 1/\rho * F$$

where

V_v = volume of eroded material (can also be the
volume of the void)

ΣESAL = cumulative equivalent 80 kN (18 kip) single
axle loads

ρ = density of eroded material

t = erosion time

E_r = erosion rate per unit area

Area = subbase and shoulder area of erosion

F = adjustment factor for rainfall, time of day,
sealant condition, subdrainage, etc.

The critical shear stress and the erosion rate can be

obtained with the rotational shear device. The area of the subbase and shoulder subjected to erosion can be determined by the structural analysis of the slabs in the curled position. The area most likely to erode is that which is not continuously in contact with the slab, due to slab curling and other voids. The erosion time can be determined from an analysis of the slab deflection. Erosion rates are measured in the rotational shear device at erosion times much higher than those existing in the field, and the prediction of erosion rates for very short erosion times has not yet been addressed. Erosion will only take place when water is available and when an opening between the slab and subbase exists. These factors can be included in the model by means of a set of adjustment factors.

11.4 Analysis of Volume of Undersealed Material

Another source of information on pumping volumes is the amount of undersealing required during pavement rehabilitation. Undersealing records of the Indiana Department of Highways were analyzed to develop predictions of volume of undersealed material required for different pavement configurations and traffic volumes.

A number of jointed PCC sections on interstates in Indiana were analyzed for maintenance requirements during 1980 to 1983, in an effort to develop regression equations

PART TWO
ECONOMIC MODELING

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for prediction of the volume of required undersealed material. If it is assumed that the volume of undersealed material is equal to the void volume, this equation can also serve as a void prediction model. This may not be a totally valid assumption, since research has shown that joints can accommodate grout even without underslab voids [Crovettil1984]. However, such equations do provide information about the rehabilitation requirements and costs for use in an economic analysis.

Only nine highway sections could be identified with sufficient information to use in a regression analysis. These sections were on I-69, I-70, and I-74, and all were 254 mm (10 in.) jointed reinforced PCC pavements. The pavement sections were undersealed between 15 and 19 years after construction. The 1978 traffic volumes ranged from 10,000 to 25,000 vehicles in both directions per day. A number of dependent variables were evaluated, viz., volume of underseal material per hole, number of holes, volume of underseal material per mile, and percent of joints undersealed. The Indiana Department of Highways used a deflection procedure, called the "Indiana Method" [Muttil1985], to determine the joints and cracks to be undersealed for each section. Only one hole was drilled per joint or crack. The actual volume of undersealed material used and the number of holes drilled were obtained from construction records. Only traffic and age

could be used as independent variables, since these were the only variables that had different levels.

The volume of underseal material injected per joint did correlate with pavement age and traffic volume. The following regression equation was obtained.

$$\log (UM) = 0.023 * \Sigma ESAL + 0.043 * A + 0.07$$

$$(R^2 = 55\%, \text{ adjusted } R^2 = 36\%, n = 9)$$

where

UM = volume of liquid asphalt per hole (gallons)

Σ ESAL = total number of equivalent axle loads

A = pavement age (from construction to first undersealing in years)

This equation predicts the volume of liquid asphalt per joint for a new pavement (Σ ESAL = 0 and AGE = 0) as 4.4 liters (1.17 gallons) or 0.0045 m^3 (0.16 cu ft). Figure 11.1 shows the effect of age and traffic on the volume of underseal material used.

11.5 Proposed Pumping Prediction Model

The only two models currently available to predict the volume of pumped material were both developed from AASHO Road Test data. The model developed by Larralde [1984] fits the AASHO Road Test data better than the EAROMAR2 model [Markow1984] and was selected to be improved for use in economic analysis procedures. Models

Figure 11.1 Effect of Traffic and Age on Volume of Undersealing

based on AASHO Road Test data have the following major deficiencies.

1. The volume of ejected material is used as an indication of the size of the void underneath the slab. The effect of the redistributed material is not considered.
2. All the material is assumed to have originated from the subbase. The pumped material from the shoulders and cavities in the shoulder is not considered.
3. Joint and crack sealing conditions are not considered. The joints were sealed after construction, but the seals were not maintained during the Road Test.
4. The model is limited to pavements with one type of subbase, one type of drainage, in one climatic region, and with dowels.

The first three deficiencies are inherent in the data and can not readily be corrected. The following section describes the effort to expand the model to include other important variables.

11.5.1 Development of Adjustment Factors

The Rauhut [1982] models currently provide the only information to develop adjustment factors for the pumping

models based on the AASHO Road Test data. The variables selected to be added to the pumping model were: subbase type, type of load transfer, subdrainage, sealant condition, and environmental factors, since they have been incorporated in the Rauhut models and are important elements in a pumping model. Slab thickness and traffic had already been included in the pumping model.

Attempts were made to correlate the damage predicted by the models to the pumping index. With a strong correlation, the damage models could be used directly to adjust the pumping model. The Rauhut models were used to predict the damage for the pavements properties and climatic conditions (Ottawa, Illinois) at which the pumping indices were measured. These properties and conditions are summarized in Table 11.2. More than two hundred cases were compared. Satisfactory regression equations could not be obtained, although the correlation between the pumping index and predicted damage was significant at $\alpha = 5\%$. Another alternative was to separately analyze the effects of: subbase type, drainage, load transfer adequacy, and the environment on the predicted damage, and calculate an adjustment factor for each. The adjustment factors were determined based on the ratio of the damage caused by variables at levels different from those at the AASHO Road Test to the damage caused by a pavement at the Road Test conditions

Table 11.2 Variable Levels Used in Development of
Adjustment Factors

AASHO Test Road Conditions:

Unstabilized subbases	STAB = 0	DMOIST = 25
No drainage	DRAIN = 0	FRINDEX = 625
Dowels	JLTS = 1	PPTN = 81 cm
Granular subgrade	SOILTYP = 0	CBR = 3
ΣESAL = 1 to 30 million		

Plain PCC pavements:

H = 8 to 12 in.

1. With stabilized subbases:

Stabilized subbases	STAB = 1
No drainage	DRAIN = 0
Dowels	JLTS = 1
Granular subgrade	SOILTYP = 0
	PPTN = 81 cm

2. Without dowels:

Unstabilized subbases	STAB = 0
Drainage	DRAIN = 0
No dowels	JLTS = 0
Granular subgrade	SOILTYP = 0
	PPTN = 81 cm

3. With drainage:

Unstabilized subbases	STAB = 0
Drainage	DRAIN = 0.3, 0.7, 1.0
Dowels	JLTS = 1
Granular subgrade	SOILTYP = 0
	PPTN = 81 cm

4. With coarse subgrade:

Unstabilized subbases	STAB = 0
Drainage	DRAIN = 0
Dowels	JLTS = 1
Coarse subgrade	SOILTYP = 1
	PPTN = 81 cm

5. In dry climates:

Unstabilized subbases	STAB = 0
No drainage	DRAIN = 0
Dowels	JLTS = 1
Granular subgrade	SOILTYP = 0
	PPTN = 30 cm

Table 11.2, continued

6. In wet climates:

Unstabilized subbases	STAB = 0
No drainage	DRAIN = 0
Dowels	JLTS = 1
Granular subgrade	SOILTYP = 0
	PPTN = 90 cm

Reinforced PCC pavements:

H = 8 to 12 in.

1. With stabilized subbases:

Stabilized subbases	STAB = 1
	DMOIST = 25
	FRINDEX = 625
	CBR = 3

2. Warm, dry climate:

Stabilized subbases	STAB = 0
	DMOIST = 0
	FRINDEX = 0
	CBR = 3

3. Warm, wet climate:

Stabilized subbases	STAB = 0
	DMOIST = 40
	FRINDEX = 0
	CBR = 3

4. Cold, dry climate:

Stabilized subbases	STAB = 0
	DMOIST = 0
	FRINDEX = 625
	CBR = 3

5. Cold, wet climate:

Stabilized subbases	STAB = 0
	DMOIST = 40
	FRINDEX = 625
	CBR = 3

Note: see Table 11.1 for explanation of the variables

1 in. = 25.4 mm

(reference case). The damage for each of the factors analyzed is not constant, but changes with the ESAL and the slab thickness. ESAL levels ranged from 1 to 30 million and slab thicknesses ranged from 203 to 305 mm (8 to 12 in.).

The JPCP model was used to obtain comparisons for subdrainage, subbase type, precipitation, and load transfer adequacy for plain pavements. The levels of the variables are given in Table 11.2. Slab thicknesses between 203 and 305 mm (8 to 12 in.) were again used. The high rainfall condition was represented by an average annual rainfall of 90 cm (35 in.), since the model also gave erroneous results at higher precipitation values for the variables used in the study. The low rainfall area was represented by an average annual rainfall of 30 cm (12 in.). Regions IA, IB, and IC in Figure 7.38 indicate wet climates in the United States, while dry climates are indicated by regions IIA, IIB, IIC. Subdrainage in the model is defined as a value of 0 or 1. For the purposes of this study the subdrainage was categorized into four levels, with values still ranging from 0 to 1. Four combinations of permeable layers and edge drains were defined which should similarly influence the pavement pumping, and therefore faulting. Selection was based on the information on permeable layers and edge drains presented in Chapters 8 and 10. Table 11.3 contains a

Table 11.3 Subdrainage Levels

- excellent : - Stabilized or unstabilized subbases with
 $k > 0.35$ cm/sec (with edge drains)
 - Non erodible stabilized subbbases (with edge drains)
- good : - Stabilized or unstabilized subbases with
 $k > 0.35$ cm/sec (no egde drains)
 - Non erodible stabilized layer (no edge drains)
 - Unstabilized subbases with
 k between 0.09 and 0.35 cm/sec (with edge drains)
- fair : - Unstabilized subbases with
 k between 0.09 and 0.35 cm/sec (no edge drains) ,or
 k between 0.009 and 0.09 cm/sec (with edge drains)
- poor : - Unstabilized subbases with
 $k < 0.009$ cm/sec (with or without edge drains)
 - Very erodible stabilized subbases
 (with or without edge drains)
 - Unstabilized subbases with
 k between 0.009 and 0.09 cm/sec (no edge drains)

Erosion classification:

Non erodible: $\tau_c > 50$ Pa

(e.g. asphalt concrete)

Erodible: τ_c between 25 and 50 PaVery erodible: $\tau_c < 25$ Pa(See Figures 7.39 to 7.45 for τ_c -values) k = subbase permeability τ_c = critical shear stress

1 cm/sec = 2835 ft/day

description of these four subdrainage levels. Three subdrainage levels ("DRAIN") were arbitrarily given values of 1.0, 0.5, and 0.0 good, fair, and poor subdrainage, respectively. A drainage level of excellent was added to accommodate pavements with a low pumping potential. An arbitrary adjustment factor of 0.01 was used.

The JRCP model was used to compare subbase types, and environmental conditions for reinforced pavements. The values of the variables in the model, used to represent the four climatic zones, were selected to fall within the ranges of the Rauhut model and to represent average warm, cold, wet, and dry conditions. The levels of the variables for each case considered are summarized in Table 11.2.

Another important factor that should be included in a pumping model is the effect of the sealant condition. Sealants reduce the amount of water entering the pavement structure as discussed in Section 8.3.3.1. Limited information is available on the effect of joint and crack sealing on pumping. The only study with some quantitative information on the water penetration rates for different sealant conditions was conducted by Dempsey and Robnett [Dempsey1979] on pavements in Georgia and Illinois. They found that sealing of joints and cracks of a plain PCC without dowels reduced the amount of water entering the drainage system with more than 95%. The amount of water

entering the drainage system of a reinforced doweled PCC pavement was reduced to about 20% by the sealing of cracks and joints. These two values give an indication of the effect of sealants on water penetration, but they are not directly related to the performance in regards to pumping. Many factors, such as, edge drain type, slab length, effectiveness of the sealant, etc., must be included in the prediction of the effect of the sealing of cracks and joints on pumping. Therefore, an adjustment factor for the effect of sealant condition was not developed. An adjustment factor was also not included for subgrade CBR in the JRCP model, since the damage factor decreases rapidly with an increase in CBR to become very small at CBR values of only 10 for the AASHO Road Test conditions.

The proposed pumping model and adjustment factors are summarized in Table 11.4. This model predicts the volume of ejected material. Indications are that the volume of the void under the slab is larger than the volume of the ejected material. The pumping prediction model provides only a prediction of the minimum volume of undersealed material required. Zero void joints can be filled with up to 0.05 m^3 (1.8 ft^3) of grout without noticeable upward deflection [Crovetti1984]. The model based on the IDOH data of the volume of underseal material used indicates that a volume of about 0.005 m^3 (0.2 ft^3) of underseal material can be injected under newly constructed joints

Table 11.4 Proposed Pumping Model

$$NPI = F * \exp [-2.884 + 1.652 \log(\Sigma ESAL * DE / 10,000)]$$

$$\log(DE) = 3.5754 - 0.3323 * D$$

$$P = 36.67 * NPI$$

$$nP = P / v_{void}$$

$$PU = P + (1 * nP)$$

where

NPI = normalized pumping index (in.³)

DE = deformation energy per application (in.-lb)

P = volume of pumped material (ft³ per mile)

nP = number of pumping joints (per mile)

vvoid = average void volume per joint (ft³)

PU = volume of underseal material required (ft³ per mile)

D = slab thickness (in.)

$\Sigma ESAL$ = cumulative 18000 lb equivalent

single axle loads

F = f_{JPCP} if nonreinforced PCC

= f_{JRCP} if reinforced PCC

$$f_{JPCP} = f_{sbl} * f_d * f_{lt} * f_{prec} * f_{sg}$$

f_{sbl} = subbase adjustment factor

= 1.0, for unstabilized

= $0.65 + 0.18 * \log(\Sigma ESAL)$, for stabilized

f_g = drainage adjustment factor

= 1.0, for poor drainage

= $0.91 + 0.12 * \log(\Sigma ESAL) - 0.03 * D$, for fair drainage

= $0.68 + 0.15 * \log(\Sigma ESAL) - 0.04 * D$, for good drainage

= 0.01, for excellent drainage

f_{lt} = load transfer adequacy adjustment factor

= 1.0, with dowels

= $1.17 - 0.68 * \log(\Sigma ESAL) - 0.078 * D$, without dowels

f_{prec} = rainfall adjustment factor

= $0.89 + 0.26 * \log(\Sigma ESAL) - 0.07 * D$,

for dry climates

= $0.96 - 0.06 * \log(\Sigma ESAL) + 0.02 * D$,

for wet climates

Table 11.4, continued

f_{sg} = subgrade adjustment factor
 = 1.0, for granular subgrades
 = $0.57 + 0.21 \cdot \log(\Sigma ESAL)$, for coarse subgrades

$f_{JRCP} = f_{sb2} \cdot f_e$

f_{sb2} = subbase adjustment factor
 = 1.0, for unstabilized
 = $0.91 - 0.02 \cdot D$, for stabilized

f_e = adjustment for climate
 = $0.011 + 0.003 \cdot \log(\Sigma ESAL) - 0.001 \cdot D$,
 for a dry, warm climate
 = $1.44 - 0.03 \cdot \log(\Sigma ESAL) - 0.06 \cdot D$,
 for a wet, warm climate
 = $1.04 - 0.32 \cdot \log(\Sigma ESAL) - 0.08 \cdot D$,
 for a dry, cold climate
 = $0.54 - 0.85 \cdot \log(\Sigma ESAL) + 0.19 \cdot D$,
 for a wet, cold climate

where

D = slab thickness (in.)

$\Sigma ESAL$ = cumulative equivalent 80 kN (18000 lb) single
 axle loads (in millions)

(age=0 and Σ ESAL=0). Therefore, a void size of 0.03 m^3 (1 ft^3) was added at each joint for the calculation of the volume of underseal material.

The effect of drainage, subbase type, and climatic conditions on the pumping of plain PCC pavements are depicted in Figures 11.2 to 11.5. Figures 11.6 and 11.7 display the effects of these variables on the pumping of reinforced PCC pavements.

Figure 11.2 Effect of Subdrainage on Volume of Pumped Material - Plain PCC Pavements

Figure 11.3 Effect of Dowels on Volume of Pumped Material - Plain PCC Pavements

Figure 11.4 Effect of Subbase Type on Volume of Pumped Material - Plain PCC Pavements

Figure 11.5 Effect of Climate on Volume of Pumped Material - Plain PCC Pavements

Figure 11.6 Effect of Climate on Volume of Pumped Material - Reinforced PCC Pavements

Figure 11.7 Effect of Subbase Type on Volume of Pumped Material - Reinforced PCC Pavements

CHAPTER 12
SELECTION AND DEVELOPMENT OF AN
ECONOMIC ANALYSIS PROCEDURE

12.1 Introduction

The purpose of the economic analysis was to develop a model or program that can be used to evaluate the effect of different design and rehabilitation alternatives on rigid pavement pumping. Since pumping is a major contributor to PCC pavement failure, the effect of certain measures on pumping will influence the general performance of the pavement. An economic analysis program used to evaluate design and rehabilitation techniques to prevent pumping will be similar to any other economic analysis procedure. The difference will be in the design and rehabilitation alternatives evaluated. A considerable amount of research has been focussed on certain aspects of the economic evaluation of pavements. It was therefore felt that it would be an unwarranted duplication of effort to develop a completely new economic analysis system for the evaluation of the effect of rigid pavement design, correction, and prevention procedures on pumping. It would

be more beneficial to develop a program that can be used independently, but also can be incorporated in one of the existing systems.

Considerable emphasis was placed on the evaluation of the effect of design and rehabilitation on pumping and the pavement distresses. Chapter 8 contains a discussion of designs to prevent pumping, while pumping related distress types and rehabilitation techniques have been discussed in Chapter 10.

12.2 Selection of Economic Analysis Program

The EAROMAR Version 2 (EAROMAR2) was selected to serve as a basis for the development of an economic analysis program to evaluate different aspects of rigid pavement pumping for the following reasons:

1. EAROMAR2 is the most recent program, and seems to be the most comprehensive of the available economic analysis programs.
2. The system includes all the components which are necessary for an economic analysis of the effects of pumping and related maintenance activities. Although the EAROMAR2 system is not yet widely used, it has excellent potential and is available from the FHWA.
3. The pavement maintenance costs are based on predicted pavement distresses and not on historical maintenance

data. Pavement distresses can, at least currently, be predicted more accurately than the cost of different maintenance activities over time. To analyze the effect of different maintenance measures on pumping, the costs of the individual maintenance activities were required. No models are currently available to directly obtain these costs.

4. The analysis is related to a pavement section and not a highway network.
5. The systems uses the net present value or annual costs to compare different design and maintenance strategies.

The EAROMAR2 program is lengthy (about 45,000 lines of code) and contains features that were not considered necessary in the economic analysis required in the pumping study, e.g., all of the traffic behavior features.

Where general conclusions were needed regarding pavement design, maintenance, and prevention methods related to pumping in this study, inclusion of all traffic or environmental options was not necessary. Therefore, a simplified economic analysis program was developed based on the EAROMAR2 concepts. Some of the EAROMAR2 models and equations were improved. Since the program was written in a modular format to facilitate improvements, these improvements can be readily accomplished. The program was

not written to replace systems like EAROMAR2, but to be used as a more simple procedure to specifically evaluate the aspects of rigid pavement pumping. However, if more features than those contained in the simple program are necessary, or if the total EAROMAR2 is readily available, the EAROMAR2 system, with the recommended adjustments, should be used in the economic analysis.

12.3 Overview of EAROMAR2

The EAROMAR2 system has been well documented and it is not the intent of this overview to describe all aspects in detail [Markow1984]. The components of EAROMAR2 will be discussed briefly, with emphasis on the distress prediction models, user consequences, and maintenance activities.

The highway geometry information used in the program include, number and width of lanes, widths of shoulders, and horizontal and vertical curvatures. Three types of pavements can be identified, viz., flexible, rigid, and composite. Information used for PCC pavements includes: surface conditions, surface course thickness, elastic moduli, modulus of rupture, thermal coefficients, subgrade modulus, and drainage conditions. Any of these values can be varied for different seasons. The environment can be incorporated by either specifying the seasonal changes or using the AASHTO regional factor. Information needed to

accommodate seasonal changes include, length of season, average temperature and moisture level.

Trip purposes and variations over time can be accommodated. The annual average daily traffic (AADT) is transformed to hourly volumes and the Highway Capacity Manual speed-flow relationships are used for congested flows. Analysis is accomplished on an hourly basis. Models are included to compare vehicle operating costs, travel time and costs, accident costs, and pollution levels as a function of speed, speed changes, congestion and pavement condition on a season-by-season basis. The effect of the season is incorporated by adjusting the pavement material properties.

The program can handle projects ranging from new roadway construction, to the extension of existing roads, to alignment changes. Inputs into the program include a description of the project, timing of the project, roadway closure, and project costs.

The results (costs and pavement conditions) are summarized for each season and then for each year. At the end of the analysis period, total discounted costs are presented.

The accuracy of the EAROMAR2 program depends on the reliability of basically three areas, viz., the pavement damage models, the maintenance activities, and the

evaluation of the user consequences. These three aspects will be discussed in more detail.

12.3.1 Maintenance Costs

Maintenance costs depend on the time of day and day of the week the activity is performed. The input variables are labor, equipment and material costs, production rates, adjustment in wages for time of day, and configuration of the work zone.

The types of maintenance activities incorporated into the program for rigid pavements are: crack filling, patching, joint filler replacement, slab replacement and mudjacking (slabjacking). Overlays are considered construction projects, but can also be used as a maintenance activity.

The program is flexible in the handling of maintenance strategies. These strategies can either be provided by the user, or developed in the program from default values. The user can specify "quality standards" (e.g., maximum damage allowed) and/or frequency of maintenance. This is incorporated through Boolean expressions in the program. If nothing is specified, all the damage present will be corrected with the appropriate maintenance activity.

Resources involved in maintenance activities are divided into labor, equipment, and materials. Each of these resources can have various items, which must be specified by the user. Wage rates can be adjusted to accommodate overtime activity. Equipment and material costs are assumed to be constant by hour and by day.

The simulation of the maintenance activities is accomplished within the EAROMAR2 program on a seasonal basis, considering each maintenance activity in turn, within each roadway section. The basis for predicting maintenance costs is the maintenance workload, which is expressed in terms of activity work units by season and by roadway section. The maintenance costs are predicted from resource requirements estimated by the EAROMAR2 program and unit costs provided by the user.

12.3.2 Pavement Damage Prediction

Maintenance was considered a demand-responsive activity (as opposed to being based on historical trends). The rate of damage accumulation was used rather than the cumulative damage. To determine the required maintenance it was necessary to predict the type and amount of damage expected to occur. What was needed in the EAROMAR2 program was the prediction of field distress over time as a function of several independent variables, as well as the effect of rehabilitation on pavement performance.

Unfortunately, very little is known (quantitatively) about these predictions.

The researchers reviewed existing empirical and mechanistic models. They concluded that none of the models reviewed accounts for the effect of maintenance, rehabilitation or overlays. No models were available to predict more localized forms of pavement distress, which are nevertheless important in predicting future requirements for maintenance, e.g., joint filler deterioration or pumping.

The effect of the environment is incorporated by seasonal adjustments in pavement material properties. Therefore, the damage prediction models had to be converted to predict the rate of additional damage. The effect of traffic loading was incorporated using the lane concept (a factor ranging from 0 to 1). Since models were not developed for all distress types, the user has the option of specifying estimated rates of damage accumulations over time. The damage models developed for rigid pavements are:

1. Linear cracking: Since transverse cracking is more important than longitudinal cracking, only transverse cracking was considered. Transverse cracking can be induced by fatigue (excessive traffic loads, inadequate slab thickness, loss of sublayer support)

and/or the environment (temperature induced curling and joint lockup). A relationship developed by Darter for zero-maintenance plain jointed concrete pavements was used. Maximum tensile stresses at the edge of the slab were computed, using a finite element program to analyze the effect of slab thickness, load configurations and location, sublayer support and temperature gradient. The total stress at the slab edge is given as a function of slab thickness, load, sublayer support, erodibility along the edge, thermal gradient, slab length, and thermal coefficient of contraction. The amount of cracking is predicted from this calculated stress, the load applications, the pavement age and the concrete modulus of rupture. The erodibility along the edge is calculated from a pumping model developed from the AASHO Road Test data. The properties of the pavement materials can be adjusted, based on the time of year and their permeabilities from models developed in the study.

2. Faulting: The researchers examined a relationship developed by Brokaw, but ultimately chose one developed by Packard. He related the average fault to age, slab length and thickness, drainage, type of base, and traffic. This relationship was modified to compute the average number of joints with faults of more than 6.4 mm (0.25 in.) per lane mile. The

drainage of the subgrade was classified as: poor, fair, or good, depending on the permeability value. The type of subbase can be either stabilized or granular.

The EAROMAR2 researchers calculated the number of faulting joints greater than 6.3 mm (0.25 in.) by multiplying the average fault value by the number of joints and dividing by 6.3 mm (0.25 in.). Doweled pavements can be expected to exhibit 0.25 to 0.33 of the faulting of undoweled pavements, with the ratio decreasing with increasing age. The faulting of doweled pavements was therefore predicted as a decimal part of that for undoweled pavements, but dependent on age.

3. Joint seal deterioration: This factor evaluates the deterioration, stripping, or other non-performance of joint sealants. The useful lifetimes and performances are product specific. An analytical model was not developed and the deterioration rate must be provided by the user. Water infiltration through cracks and joints can cause pumping, faulting, spalling, blowups, midslab cracking, joint movement, and transfer device failures. No correlation could be found between sealer damage and structural maintenance, however.

4. Spalling: The infiltration of fines from the subgrade or subbase into the joints can cause spalling. Darter developed a model from data of the Michigan Test Road, relating spalling to joint spacing and pavement age.
5. Pumping: A model was developed from pumping data observed at the AASHO Road Test. The pumped joints per lane mile were related to slab length, pumping index and subbase drainage. Pumping index is in turn related to slab thickness and load applications, and is a measure of the volume of fines observed at the edge of the pavement. The pumping index is divided by the depth of the void (assumed in this model to be 51 mm or 2 in.) to be used as a measure of the erodibility along the edge. An average void size of 2 by 4 by 0.02 m (72 by 144 by 2 in.) and joint spacings of 4.6 m (15 ft) were used in the development of the model.
6. Blowups: A model was developed relating the number of blow-ups per lane per year to the susceptibility of the aggregate to blowups, pavement age and joint spacing. The effect of the infiltration of incompressibles into the joint could not be included due to the lack of data.
7. Roughness (Serviceability): A model developed by

Brokaw was examined, but a modified serviceability equation developed by Darter from AASHO data was finally used. Darter related serviceability to faulting and roughness between joints, which is related to loads, slab length, slab thickness, modulus of rupture and elasticity, radius of applied edge load and foundation support.

12.3.3 Traffic Characteristics and User Consequences

The treatment of traffic in the EAROMAR2 program is very detailed. All the relevant factors are included. Many of the roadway operational characteristics developed for the original EAROMAR program were retained. Since the treatment of the roadway characteristics and user consequences in the EAROMAR2 system is very complete, and most of the concepts and relationships were retained in the development of a simplified economic analysis model, they warrant a detailed discussion.

1. Traffic considerations: Traffic volume, which includes variations along the route, lane distributions and time variations (traffic growth) were incorporated in EAROMAR2. Traffic composition, which includes trip purpose (with daily and seasonal variations), vehicle type, fuel type, axle weight equivalencies, passenger car equivalents and emission factors. The two most important factors are trip purpose and vehicle type.

The diversion of traffic to different routes due to lane closures is not included in the EAROMAR2 program, mainly because it operates with links rather than with the entire network.

2. Free flow conditions: Highway Capacity Manual procedures were used to simulate free-flow operating speeds. A set of equations developed by Butler for the original EAROMAR program (to approximate the Highway Capacity Manual curves for average speeds) was modified in EAROMAR Version 2. A relationship developed by Karan and Haas was used to determine the limiting speed due to roughness.
3. Conditions during maintenance: In the determination of the user consequences it is necessary to simulate the traffic flow and changes therein during the application of maintenance. Maintenance activities usually require lane closures, which affect the traffic flow. The types of lane closures and the required signalization practices are described in NCHRP Syntheses 1 [NCHRP1969] and 25 [NCHRP1974]. The type of closure is provided by the user in the EAROMAR2 system as one of three, viz., lane restrictions, crossovers, and detours. The length of the closure zone and the time it will be closed must also be specified.

The simulation of congestion and queuing were done in EAROMAR2 by adapting existing relationships. The simulation of road operations is performed within each road section for each hour of the day, consistent with other aspects of the EAROMAR design. Average characteristics in terms of demand, capacity, speed, and average length of queue within an hour are computed for each section. The simulation also accounts for the limiting effects of bottlenecks, capacities on all affected upstream sections, as well as for continuation of queues through contiguous sections.

The speed delays and congestion increase vehicle operating costs and travel time, change accident potential, and increase pollution levels.

4. User costs: In the analysis of the effect of different factors, the authors used basically four sources: Winfrey's textbook [Winfrey1969], a FHWA report [Graham1977], and NCHRP Reports 111 [Winfrey1971], 122 [Claffey1971] and 133 [Curry1972]. The following conclusions were reached:

1. Fuel consumption is related to vehicle speed, pavement serviceability, speed change, curvature, and idle consumption rate.

- ii. Oil consumption rate was related to fuel consumption rate, vehicle speed, and fuel and oil prices.
- iii. Tire wear is related to speed changes, tire price, and serviceability index.
- iv. Maintenance parts and labor costs are considered to have a constant value, and are therefore not counted in the economic analysis.
- v. Vehicle depreciation is also taken as a constant value and is not considered in the analysis.
- vi. Value of travel time savings. The researchers opted to have the user specify the value of time by trip purpose and by vehicle, since the value of time is influenced by a large number of factors.
- vii. Accident rate. The authors studied a FHWA report [Graham1977] on accidents due to construction. They concluded that the changes in accident rates due to maintenance are more meaningful than mere accident numbers. The change in accident rate was related to the ratio of the number of lanes before rehabilitation to the number of lanes during rehabilitation. The severity of the accidents tends to be lower

during the construction period. Input values required were: the base year accident rate, percentage distribution by severity class, and the average accident cost by severity class.

viii. Air pollution. The NCHRP Report 133 was used as a basis for the determination of hydrocarbon and carbon monoxide emissions. Relationships were obtained for emission levels at uniform freeway speeds, speed changes, and queuing, as a function of the speed and the volume-capacity (V/C) ratio.

12.4 Economic Analysis of Rehabilitation and Design Alternatives

The Purdue Economic Analysis of Rehabilitation and Design Alternatives for Rigid Pavements (PEARDARP) was written to evaluate the effects of different pavement designs and rehabilitation techniques on pumping and pavement performance, using EAROMAR2 concepts as bases. The analysis program has all the elements required for an economic analysis. PEARDARP is not a pavement management program, since pavement conditions can not be specified. However, information provided in the program may be useful in pavement management systems. The program can be used to evaluate alternatives on the basis of current or constant dollars, depending on the specification of

rehabilitation costs, and the selection of the interest rates. Evaluation based on constant dollars is more widely used, as described in Section 9.2.6. Table 12.1 contains a description of the information needed as input. A listing of the program, an example input, and output are given in Appendix D.

12.4.1 PEARDARP Analysis Procedure

The analysis is conducted on a yearly basis, using the average conditions existing during the year. Adjustments are not made for seasonal changes, in either the pavement conditions or the traffic characteristics. Methods proposed for the adjustment in pavement support conditions due to water saturation by Markow [1983] and Lui [1983] were considered during the developments of the program, but were not included in the program. The effect of water saturation on the support conditions of rigid pavement performance is small compared to the other uncertainties included in an economic analysis. The user costs are calculated for an average day, and multiplied by 365 to obtain a yearly cost. The analysis is conducted for highway travel in one direction. All of the pavement and traffic characteristics must be specified in one direction. The costs in both directions can easily be obtained by multiplying the PEARDARP results by two, if the travel and pavement characteristics in both directions are the same. The pavement distresses are predicted at

Table 12.1 PEARDARP Input Information

1. Related to the pavement
 - slab thickness (in.)
 - slab length (ft)
 - slab elastic modulus (psi)
 - rupture strength (psi)
 - subgrade reaction (pci)
2. Related to drainage, subbase type, load transfer, and climate
 - subgrade type and drainage (if coarse=poor, if granular=good)
 - subdrainage (excellent, good, fair, poor)
 - subbase type (stabilized or unstabilized)
 - sealant type (liquid asphalt or low modulus)
 - load transfer device (undoweled or doweled)
 - reinforcement (with or without)
 - climatic region (dry-warm, dry-cold, wet-warm, or wet-cold)
3. Related to the maintenance
 - critical fault 1 (in.)
 - critical fault 2 (in.)
 - standard deviation of faulting (in.)
 - average void volume (cu ft)
4. Related to the traffic and users cost calculations
 - AADT (vpd)
 - truck factor (decimal)
 - proportion in design lane (decimal)
 - traffic growth rate (%)
 - design speed (mph)
 - speed limit (mph)
 - number of lanes
 - width factor (decimal)
 - passenger car equivalent
 - max. service flow (vpd)
 - factor to obtain speed from speed limit
5. Related to volume distributions during 24 hours
 - traffic volume distribution in each hour
6. Related to traffic and accident cost data
 - proportion of passenger cars (%)
 - proportion of pick-ups (%)
 - proportion of S-U trucks (%)
 - proportion of combination trucks (%)
 - proportion of diesel trucks (%)
 - section length (mi)
 - accident rate (no. of accident per MVM)
 - cost per accident (\$)

Table 12.1, continued

- 7. Related to vehicle operation costs
 - Fuel, oil, tire and time costs for each vehicle class
- 8. Analysis time and output information
 - last year of analysis
 - interval of years to be printed
- 9. Construction cost information
 - construction cost (\$)
- 10. Maintenance activity information
 - rehabilitation activity type
 - rehabilitation activity cost (\$/unit)
 - total number of units required
 - traffic diversion cost (\$)
 - section length of rehabilitation operation (mi)
 - no. of lanes open during rehabilitation
 - duration of rehabilitation (day)
 - speed limit during construction (mph)
 - year of rehabilitation
 - pumping factor (change in pumping due to rehabilitation)
 - faulting factor (change in faulting due to rehabilitation)
 - cracking factor (change in cracking due to rehabilitation)
 - roughness factor (change in roughness due to rehabilitation)
 - additional value (e.g. overlay thickness)

All values in one direction of travel

the end of each year. These pavement distresses are used in turn to predict the required rehabilitation needs and the rehabilitation costs. The average equivalent 80 kN (18000 lb) single axle load (ESAL) values in each year were used in the calculation of pavement distresses. The average pavement condition during the year is used to calculate the user's costs. The rehabilitation activities are assumed to be applied at the beginning of each year.

The program consists of the following elements. (Appendix D contains a listing of the PEARDARP program, the input, and output).

1. Main program: All input information provided by the user is read into the main program. A listing of most of the input data is printed at the beginning of the program as TABLE I. The pavement distresses are also predicted by the main program. The construction costs, rehabilitation types, and rehabilitation costs are produced in TABLE II of the output.
2. Subroutine for the rehabilitation aspects: The costs of different rehabilitation activities applied in each year are read, added to provide a yearly cost, and printed. Adjustments are also made, as required, to the pavement properties or conditions due to the applied rehabilitation method.

3. Subroutine to calculate user costs during rehabilitation: The added user costs due to the delay and disruption of normal traffic flow during rehabilitation are calculated.
4. Subroutine to determine weighted vehicle speeds: As mentioned, a weighted vehicle speed for a typical day of the year is calculated by weighting the calculated speeds in each hour by the traffic volume.
5. Subroutine to calculate the vehicle running costs: The running costs are calculated for each of the five vehicular classes. The consumption rates are first calculated and these are multiplied by the unit costs.
6. Subroutine to calculate discounted costs: Construction and rehabilitation costs, vehicle operating costs, time costs, and accident costs are discounted separately and combined at interest rates ranging from 0 to 20%. A single discount rate is not used, since it is important to evaluate the alternatives at different discount rates. The cost elements are presented separately, since it is often useful to evaluate the costs separately.
7. Print subroutines: Five subroutines are used to print tables containing the pavement distresses (TABLE III), the consumption rates of each of five vehicular

classes (TABLE IV), the user costs (TABLE V), the annual costs (TABLE VI), and the discounted costs (TABLE VII). All these values are calculated every year, but can be presented at any equally spaced yearly interval.

TABLE III presents the following information: traffic volume, ESAL-values, number of spalled joints, volume of pumped material, number of pumping joints, average fault, number of faulted joints, damaged area, length of cracks, patched area, roughness, and present serviceability index (PSI). These values can be used by the user to identify: years at which rehabilitation is necessary, which distresses need to be corrected, and the quantity of rehabilitation necessary. TABLE IV in the output provides the average traffic volumes, the average vehicle speeds, and the average PSI values, in addition to the fuel, oil, and tire consumption rates for each of the five vehicle classes each year. The information provided in TABLE IV is used to calculate the road user costs for each vehicle class in TABLE V. TABLE VI in the output contains all the annual costs. These costs are discounted at interest rates ranging from 0 to 20% annually and presented in TABLE VII as present and annual costs. Vehicle operating, time, accident, and construction and rehabilitation

costs are presented separately. Any combination of these may be obtained by adding the appropriate values.

12.4.2 Construction Costs

The initial construction cost is required as input in a lump sum. The factors in the design of the rigid pavement which can affect the performance related to pumping are: the pavement type, slab thickness, slab length, subbase type, shoulder type, existence of edge drains, and the existence of load transfer.

A salvage value can not directly be included in the program. However, the differences in rehabilitation costs to obtain a certain PSI at the end of the analysis period can be used as a measure of the salvage value if necessary. As discussed in Section 9.2.4, the salvage value is uncertain and its effect is often small.

12.4.3 Rehabilitation Costs

The rehabilitation costs were calculated from the predicted pavement distresses.

12.4.3.1 Pavement distress prediction models: The reliability of the distress prediction models is an important question. One of the purposes of the economic analysis was to verify or improve the prediction models. The distress types that are important in a study involving

pumping are pumping, faulting, cracking, joint spalling, roughness, and PSI. Pumping, faulting, and cracking are directly affected by pumping. The amounts of faulting and cracking affect the PSI. Joint spalling is indirectly affected by pumping, but provides an indication of when partial slab repair is required and how much repair will be needed. Roughness between the joints is also not directly affected by pumping, but influences the PSI. The PSI is predicted based on the lengths of cracks, the patched area, and the roughness due to faulting and irregularities between the joints. The six pavement distress prediction models previously discussed were included in PEARDARP. They are summarized in Table 12.2 and are discussed below.

1. Faulting model: Six faulting models, viz., Brokaw [1974], Gulden (1974), Packard [1977], Darter (1982), and Rauhut (1983), were evaluated. None of these models include the effects of edge drains or subbase permeability. The model developed by Packard was found to be the most appropriate for this study, since it includes factors such as subbase type and subgrade drainage, in addition to traffic, age, slab thickness and length. The EAROMAR2 used the same model and Majidzadeh [1984] also found it to be the best one available. It is conceivable that the existence of subdrainage (permeable layers and edge

Table 12.2 Distress Prediction Models

1. Faulting:-

$$F_{n-avg} = (1.29 + (K_1 * (T * A^2))) * f_{SD} / 32.0$$

$$K_1 = \frac{48.95 * S^{0.610} (J-13.5)^b}{D^{3.9}}$$

$$T = \frac{\Sigma Vol * pt}{n}$$

$$F_{d-avg} = f_d * F_{n-avg}$$

$$f_d = \frac{1}{(1+A)^{0.5}}$$

$$\Delta F_{n-avg} = \frac{0.465 * K_1 ((i+i)^n * vol_0 + \Sigma Vol * pt)}{32 * (\Sigma Vol * pt * n)^{0.535}} * f_{SD}$$

nF = determined from a normal distribution of fault values

where

F_{n-avg} = average fault in in. (non doweled)

F_{d-avg} = average fault in in. (doweled)

ΔF_{n-avg} = change in average fault with time (in. per year)

nF = number of faulted joints

b = 0.241 for granular subbase

0.037 for stabilized subbase

D = slab thickness (in.)

J = slab length (ft)

S = subgrade drainage: 1 = good

2 = poor

i = growth rate (decimal)

pt = proportion of trucks in the design lane

Vol₀ = traffic volume in year 0

n = year

A = age (years) = n

ΣVol = cumulative traffic volume (in one direction)

f_{SD} = subdrainage : 0.1 = excellent

0.6 = good

1.0 = fair

1.4 = poor

Table 12.2, continued

2. Pumping:-

See Table 11.1 for the pumping model

3. Cracking:-

$$DA = e^{(\text{atan}(a_1 + a_2 + \log(\Sigma\text{ESAL}) + a_3 * D + a_4 * k_R) * 6}$$

$$CR = (DA/4000) * 2 * 5280/1 * 1/63.36$$

$$a_1 = 39.006 \quad a_3 = -4.387$$

$$a_2 = 3.941 \quad a_4 = -0.036$$

For stabilized materials:

$$\log(k_c) = 0.7405 * \log(D) + 0.7256 * \log(k) + 0.5559$$

$$k_R = k_c$$

For unstabilized materials:

$$\log(k_c) = 0.3483 * \log(D) + 0.8163 * \log(k) + 0.8163$$

$$k_R = 1.7 * k_c$$

where

DA = damage area per joint (in.²)

CR = length of crack (lf per 1000 ft²)

ΣESAL = cumulative equivalent 80 kN (18 kip) single axle loads

D = slab thickness (in.)

k = modulus of subgrade reaction (pci)

k_c = composite modulus of slab support (pci)

k_{cd} = dynamic composite modulus of slab support (pci)

Table 12.2, continued

4. Roughness:-

$$R = 360 - 216 \left(1.5 - \frac{1}{1+e^{-\beta/\rho X}} + \frac{1}{1+e^{(\Sigma \text{ESAL} - \beta/\rho X)}} \right)$$

$$\beta = -50.088 - 3.775D - 30.644D^{0.5}$$

$$\rho = -6.697 + 0.139D^2$$

$$X = 10^{1.774Y}$$

$$Y = \log \left(\left(\frac{M_r}{690} \right) \frac{4 \log(8.789 \frac{D^{0.75}}{F}) + 0.359}{4 \log(Z^{0.25} (0.54 \frac{D^{0.75}}{F})) + 0.359} \right)$$

$$F = (30.56 + D^2)^{0.5} - 0.675D$$

$$Z = E/k$$

$$\Delta R = - \frac{216 \text{ ESAL}_0 (1+i)^n e^{(\Sigma \text{ESAL} - \beta/\rho X)}}{(1+e^{(\Sigma \text{ESAL} - \beta/\rho X)})^2}$$

where

R = roughness (in. per mile)

ΔR = change in roughness with time (in. per mile per year)

D = slab thickness (in.)

E = modulus of the slab (psi)

k = modulus of subgrade reaction (pci)

M_R = 28-day modulus of rupture (psi)

ΣESAL = cumulative 80 kN (18 kip) equivalent single axle loads

ESAL_0 = initial 80 kN (18 kip) equivalent single axle loads

i = traffic growth rate (decimal)

Table 12.2, continued

5. Joint Spalling:-

$$F_s = 1 - e^{-\alpha(J-8)}$$

$$\alpha = 0.0000162 A^{3.0806}$$

$$\Delta F_s = 0.00005 (J - 8) n^{2.0806} * e^{-\alpha(J-8)}$$

where

F_s = fraction of joints spalled

ΔF_s = change in fraction of joints spalled with time
(fraction per year)

A = age (years)

J = joint spacing (ft)

6. Present Serviceability Index:-

$$PSI = 5.41 - 1.80 \log (SV + 1) - 0.09 (C+P)^{0.5}$$

$$SV = SVR + SVF$$

$$SVR = 0.000145 R^{2.255}$$

$$SVF = \frac{0.00159}{J} * F^{1.7229}$$

where

PSI = present serviceability index

SV = slope variance (radians² * 10⁶)

SVR = slope variance due to roughness

SVF = slope variance due to faulting

J = slab length (ft)

C = linear cracks (lf per 1000 sq ft)

P = patches area (sq ft per 1000 sq ft)

F = average fault (in.)

R = roughness (in. per mile)

Figure 12.1 Effect of Subdrainage on Faulting

drains) will affect faulting. Since subdrainage is not included in the Packard faulting model, a subdrainage factor was added to the model. The only quantitative information on the effect of the improvement of subdrainage (due to the installation of retrofit drains) was reported by Ames [1985]. He indicated that the installation of retrofit drains at highways in California reduced the faulting rates with typically an eighth. The same classifications of subdrainage developed for use in the pumping model were used. The Packard faulting model was assumed to have been developed for pavements with "fair" subdrainage, which is probably the average drainage conditions of existing PCC pavements. The California experience was used as a guideline to develop values to quantify the effect of subdrainage on faulting. The subdrainage definitions, PEARDARP abbreviations, and values are given in Table 12.3. The subdrainage ("good1", "fair2", etc.) is specified by the user from information provided in Table 12.3. Figure 12.1 displays the effect of subdrainage on faulting.

A normal distribution of joint faults was used to calculate the number of faulted joints with faults larger than any two specified values. This enables the user to determine the number of joints with faults larger than these two values. This is

Table 12.3 Treatment of the Effect of Rehabilitation Techniques on Pavement Distresses in PEARDARP

ACTIVITY	PUMPING	FAULTING	CRACKING	ROUGHNESS	PATCHING	PEARDAPP ABBREVIATION
Underseal	value	--	--	--	--	"under"
Full depth(1)	value	value	value	value	value	"fuldd"
	rate	rate	rate	rate		
Full depth(2)	value	value	value	value	value	"fulld"
Surface seal	--	--	--	--	--	"shmod" (3)
						"sasph" (4)
Retrofit	rate(5)	rate(5)	--	--	--	"subdr"
Asphalt shoulder	--	--	--	--	--	"shasp"
PCC shoulder	rate(6)	rate(6)	rate(6)	rate(6)	--	"shpcc"
Edge support	rate(6)	rate(6)	rate(6)	rate(6)	--	"esupp"
Grinding	--	value		value(7)	--	"grind"
Load transfer	rate	rate	rate	rate	--	"dowel"
Reurfacing	rate(8)	rate(8)	value	value	value	"overl"
	(10)	(10)	(10)	(10)		

value = the distress prediction model is set to the initial condition

rate = the distress progression rate is changed by changing the variables in the model without setting the distress to the initial condition if "value" is not changed

- (1) patching with the restoration of the load transfer
- (2) patching without the restoration of the load transfer
- (3) high modulus seal (life = 10 years)
- (4) liquid asphalt seal (life = 2 years)
- (5) rates are changed because the drainage factors are change in the prediction models
- (6) the same as an increase in slab thickness of 25 mm (1 in.)
- (7) the roughness is set to a maximum value of 50 in. per mile
- (8) the effective thickness changes by adding the overlay thickness to the slab thickness
- (9) if an asphalt overlay is used, the roughness prediction model
- (10) an adjustment factor included which can be used to influence the distress prediction models

important in the specification of rehabilitation procedures, since the effectiveness of grinding is affected by the fault sizes. Grinding is not successful for faults larger than 6.3 mm (0.25 in.), while it is commonly used to correct faults larger than 2.3 mm (0.09 in.) [Gulden1983]. Gulden [1974] described the distribution of faults based on the Faulting Index. This description was used to obtain a standard deviation of 1.3 mm (0.05 in.). This value has not been verified and the standard deviation was therefore left to be specified by the user. The number of faulted joints can be used to specify the amount of grinding necessary by multiplying the number of faulted joints with the area to be ground at each joint.

Packard [1977] developed the faulting model for undoweled pavements. The EAROMAR2 researchers developed an adjustment for doweled pavements based on data from Florida. They found that the faulting in doweled pavements is about a third to a fourth of that of undoweled pavements. The damage functions developed by Rauhut [1983] indicate the same order of magnitude reduction. Therefore, the EAROMAR2 adjustment was used in the analysis. The rate of faulting was obtained by differentiating the faulting prediction equation.

2. Pumping model: The pumping model has been discussed thoroughly in Chapter 11. The model proposed in Chapter 11 was used. The number of pumping joints is sometimes used as an criterion for the application of maintenance. The pumping model predicts the volume of pumped material and the number of pumping joints is obtained in the program by dividing the calculated volume of pumped material by the average void volume. The average void volume is provided by the user. Crovetti [1984] reported that average void volumes range from 0.06 to 0.23 m³ (2 to 8 cu ft). A value of 0.34 m³ (12 cu ft) is used in EAROMAR2, while Majidzadeh [1984] used an average void volume of 1640 cm³ (100 in.³) per inch of slab length. The quantities of grout depends on: the amount of slab lift during grouting, the amount of slab curling, subbase type and condition, subgrade type, extent of "holes" or "discontinuities", shoulder type, and availability of channels for grout flow [NCHRP1984].
3. Joint deterioration: Joint deterioration models have been developed by Darter [1982] and Rauhut [1983], but the joint spalling prediction model used in EAROMAR2 was found to be more appropriate and was used. The number of spalled joints is calculated by the program. This number can be applied by the user to specify the amount of partial slab repair needed.

An average spall per joint of 0.37 sq m (2 sq ft) was recommended in EAROMAR2. The spalling prediction model was included only to be used to identify partial depth repair needs.

4. Cracking: A regression model developed by Larralde [1984] from a mechanistic analysis of the pavement slab was used to predict the cracking in the slabs. This cracking model includes the effects of pumping and voids. The model was developed for a slab on any type of subbase, but the void prediction model used included only unstabilized subbases. The cracking is predicted as a damaged area per joint by the model. The damage area was defined by Larralde [1984] as the product of the number of nodes where strains in the slab would induce cracks and the area of influence of each node. The area of influence of each node was constant at 0.258 m^2 (4000 in.^2). An average crack length of 610 mm (24 in.) in each influence area was assumed, to obtain the linear length of cracks. The linear length of cracks is further converted in PEARDARP to linear cracks per 1000 ft^2 .

The cracking prediction model was developed with the dynamic composite slab support (k_R) as a variable. The composite support (k_c) is determined in the program from the subgrade support (k) by regression equations based on data presented in the

PCA rigid pavement design method [PCA1984]. The composite slab support is converted to a dynamic composite slab support by multiplying k_c by 1.7 [Fischer1984, Ioannides1984, Larralde1984]. Only the composite support of unstabilized layers is converted to a dynamic composite support, since research has not yet been reported on the dynamic behavior of stabilized materials.

5. Patching: A model was not used to predict the patched area. The patched area was obtained from the area of full or partial depth patching applied during a rehabilitation operation.
6. Roughness: The roughness (between joints) prediction equation developed by Darter and presented in EAROMAR2 was used.
7. Present serviceability index: Several models have been developed to predict PSI, e.g., Brokaw [1974], Darter [1977], Darter [1983], and Rauhut [1982]. A model developed from the AASHO Road Test data was used in the EAROMAR2 program and also used in PEARDARP.

The effect of pumping on blowups is small compared to the other factors, e.g. type of aggregate used in the concrete, and was therefore not included in the analysis.

12.4.3.2 Rehabilitation techniques: The rehabilitation techniques related to pumping related distresses were discussed in Chapter 10. Most of the distresses can be corrected or prevented by a combination of rehabilitation techniques. Each of the rehabilitation techniques has a different effect on the distress type and the rate of distress progression. The effects of these techniques can usually be observed, but to quantify them is more difficult. This is unfortunately necessary in the economic analysis procedure. An attempt was made to quantify the effects of all the appropriate rehabilitation techniques. Only the techniques which influenced pumping related distresses were included. Each of the pavement distress prediction models will be affected differently by different rehabilitation techniques. The premise that one of four things can happen to the distresses, with the application of a certain rehabilitation technique, was used to characterize the influence on each distress (Figure 12.2). The four possibilities are.

1. The distress can be corrected to the initial condition and the rate of distress progression can be changed (Y1 in Figure 12.2). The correction of a distress is achieved in the program by setting the distress to the initial condition, which means setting the ESAL value to zero. The deterioration rate will start from the initial condition, with

Figure 12.2 Effect of Different Design and Rehabilitation Alternatives on the Pavement Condition

different parameter values in the model. Full depth patching with the installation of dowels is an example of this. The pavement condition is restored and the slab deflections are reduced, which means a reduction in distress progression rates. This is accomplished in the program by changing the parameters in the model, e.g., including the effect of dowels in the model.

2. The distress can be corrected to the initial condition, but the rate of distress progression will be the same as it was at the time of rehabilitation (Y2 in Figure 12.2). In this case the deterioration rate will start from the initial condition. Although the parameter values will be the same, the distress progression rates will be different from the progression rates immediately after construction, since the ESAL values are different (if the growth rate is not zero). For example, diamond grinding or patching without the installation of dowels. This is achieved in the program by setting the ESAL value to zero without changing the parameters in the model.
3. The distress will not be corrected, but the rate of progression will change (Y3 in Figure 12.2). Examples of this are the installation of retrofit drains and tied-PCC shoulders. Faulting, cracking, or roughness is not improved or corrected, but their

rates of progression will change due to lower slab deflections or better drainage. The parameters in the pavement distress models are changed to accomplish these changes in distress progression rates in PEARDARP.

4. The distress will not be corrected and the distress progression rate will not change (Y4 in Figure 12.2). For example, the correction of shoulder depressions has no influence on faulting or cracking.

The distress prediction procedure in PEARDARP can be refined by the use of derivatives of the prediction models instead of the cumulative predictions. However, this is essentially achieved through the use of the derivative of Σ ESAL to calculate distress values and, with limited knowledge on the exact influence of rehabilitation techniques on the distress progression rates, it is an unnecessary complication. The derivatives of the progression models for which closed form solutions are available are presented in Table 12.2. With the availability of more information on the effect of rehabilitation techniques on distress progression, this refinement may be considered. The disadvantage of the current PEARDARP procedure is that the partial rehabilitation of distresses can not be evaluated. The user needs to specify rehabilitation techniques and quantities to fully correct a particular distress.

However, not all distresses have to be corrected. This is not a serious shortcoming in the program, since a particular distress is usually fully corrected, once the effort is made to improve a certain section of pavement. The rehabilitation techniques considered in PEARDARP, their abbreviations in the program, and their effects on the different distress types are presented in Table 12.4. The effect of some of the maintenance techniques, e.g. crack and joint sealing, was not included in the program, since quantitative predictors of their effect on the pavement distresses could not be found. To accommodate the inclusion of their effects, four adjustment factors have been included in the program. These factors can be used to adjust the distress progression rates, when the adjustments included in the program are not adequate. The adjustment factors pertain to pumping, faulting, cracking, and roughness.

The effect of the sealing of cracks and joints is important and may influence the distress progression rates. Not enough quantitative information was available to develop adjustment factors, as discussed in Sections 10.3.2.3 and 11.5.1. Adjustments have to be specified by the user through the adjustment factors mentioned above. The discussion in Section 11.5.1 may help in the selection of adjustment factor values.

Table 12.4 Description of Subdrainage in PEARDARP

Definition of subdrainage levels:

- excellent : - Stabilized or unstabilized subbases with
 $k > 0.35$ cm/sec (with edge drains)
 ('exell')
- Non erodible stabilized subbases (with edge drains)
 ('exell')
- good : - Stabilized or unstabilized subbases with
 $k > 0.35$ cm/sec (no edge drains)
 ('good2')
- Non erodible stabilized layer (no edge drains)
 ('good2')
- Unstabilized subbases with
 k between 0.09 and 0.35 cm/sec (with edge drains)
 ('good1')
- Slightly erodible stabilized layer (with edge drains)
 ('good1')
- fair : - Unstabilized subbases with
 k between 0.09 and 0.35 cm/sec (no edge drains)
 ('fair2')
- or k between 0.009 and 0.09 cm/sec (with edge drains)
 ('fair1')
- Slightly erodible stabilized layer (no edge drains)
 ('fair2')
- poor : - Unstabilized subbases with
 $k < 0.009$ cm/sec (with or without edge drains)
 ('poor1')
- Erodible stabilized subbases
 (with or without edge drains)
 ('poor1')
- Unstabilized subbases with
 k between 0.009 and 0.09 cm/sec (no edge drains)
 ('poor2')

Erosion classification:

- Non erodible: $\tau_c > 50$ Pa
 (e.g. asphalt concrete)
- Slightly erodible: τ_c between 25 and 50 Pa
- Erodible: $\tau_c < 25$ Pa
- (See Figures 7.39 to 7.45 for τ_c -values)

Note: The word and number in parentheses indicate
 the abbreviation used in PEARDARP

Table 12.4, continued

Effect of the installation of retrofit drains:

'poor1' remains 'poor1'
'poor2' changes to 'fair1'
'fair1' remains 'fair1'
'fair2' changes to 'good1'
'good1' remains 'good1'
'good2' changes to 'exell'

k = subbase permeability

τ_c = critical shear stress

1 cm/sec = 2835 ft/day

Another case in which the user should use the adjustment factors is when retrofit drains are installed. The effect of retrofit drains on the progression of distresses depends on the pavement condition, and the subbase drainage and erosion characteristics, as discussed in Section 10.3.2.2. The only pavement distress prediction models affected by retrofit drains are the faulting and pumping models. In PEARDARP the effect of retrofit drains is incorporated by changing the subdrainage conditions as summarized in Table 12.3. The information in the table indicates that the installation of retrofit drains will not improve the performance of the pavement if the subbase has a low permeability or is highly erodible. The user can specify additional amounts of reduction in faulting, cracking, and pumping progression due to the installation of retrofit drains by using the adjustment factors.

The program can handle only PCC overlays, since the distress prediction models are only applicable to rigid pavements. An asphalt overlay essentially changes the pavement to a flexible pavement. Different pavement distress models need to be included to predict the behavior of such a pavement. The discussion and inclusion of flexible pavement distress models were outside the scope of this study and therefore are not included. The EAROMAR2 program contains flexible pavement distress

models. The pavement distress prediction models used for the rigid pavement were also used for the PCC overlay, since distress prediction models are not available for PCC overlays.

Rehabilitation techniques, such as grinding, can reduce the accident rates on wet pavements. The effect of these rehabilitation techniques is not incorporated into PEARDARP, since quantitative models were not available.

12.4.3.3 Rehabilitation costs: With the distresses known at a certain time, the user can select appropriate rehabilitation techniques to correct them. The program requires as input the rehabilitation technique, when it will be applied, the quantity, the unit costs, the traffic control costs, the number of lanes closed, the length of the rehabilitated section, and the length of time this situation will exist (Table 12.1). Any number of rehabilitation activities can be applied simultaneously. The type and amount of rehabilitation need to be specified by the user. The program output (TABLE III) provides the information needed. For example, if a pavement is to be rehabilitated when the PSI is less than 2.5, the level of each distress at the end of that year can be obtained from TABLE III. The user then needs to specify the type and amount of rehabilitation to be used to correct the distresses. This information is then included in the input and the program run again. PEARDARP will provide

the pavement distresses, user costs, and discounted costs for this new situation.

If the economic analysis is to be conducted in constant dollars, the rehabilitation unit costs have to be provided in constant dollars, i.e., excluding the effect of inflation, unless they change at a rate significantly different from the inflation rate. In instances where the unit costs are expected to increase at rates different than the inflation rate, the rehabilitation costs should reflect these differences. If the analysis is to be conducted in current dollars, the future rehabilitation unit costs have to be provided as actual costs in the future year.

During a rehabilitation project, traffic through the section must be safely maintained. Traffic signs, cones and flag persons have to be provided. NCHRP 1 [NCHRP1969] and 25 [NCHRP1974] provide guidelines on traffic control during rehabilitation. The traffic control cost is specified by the user as a lump sum for each rehabilitation activity.

12.4.4 User Consequences

The highway geometry was limited to a level, tangent section for the analysis of designs and rehabilitation activities related to pumping. Five vehicle classes were identified, viz., passenger cars, pick-up trucks, single

unit trucks, combination trucks (gasoline), and combination trucks (diesel). These vehicle groups include most of the vehicles currently using the highways. The vehicles were classified in these five categories, since the vehicle running costs are different for each group. The EAROMAR2 running cost equations were used. The equations used to calculate fuel, oil, and tire consumption rates are given in Table 12.5. Table 12.6 contains the coefficients used in these equations. The unit costs of fuel, oil, and tires are specified by the user.

Maintenance, depreciation, and other fixed costs were not included in the analysis. The value of time is different for drivers in each of the vehicle classes and was left for the user to specify.

The accident cost has two elements, viz., accident rate and average accident cost. Both have to be specified by the user. The accident rates were not adjusted for changes in pavement condition, since the effect of roughness on the accident rates is small. The accident rates were increased during periods of lane closures based on the EAROMAR2 adjustment.

$$\Delta AR = -15.97 + 63.18 (N_n/N_c)$$

where

$$\Delta AR = \text{increase in accident rate (\%)}$$

Table 12.5 Vehicle Running Cost Models

Fuel Consumption:

$$FL_g = A + IC/S + Cs^2 + FL_{ch}$$

$$FL_{ch} = a_{ch} (S \Delta S^{1.44})^{b_{ch}}$$

$$C = \frac{B}{2b_f^3}$$

$$A = a_f - 3B/(2b_f)$$

FL_g = fuel consumption rate (gallons per 1000 hr)

FL_{ch} = fuel consumption due to speed changes (gallons per 1000 hr)

IC = idle consumption rate (gallons per 1000 hr)

S = vehicle speed (mph)

ΔS = speed change

For diesel powered trucks:

$$F = FL_g * 0.65 * F_c$$

Oil Consumption:

$$OIL = FL (a_o + b_o * S + C_o S^2)$$

where

OIL = oil consumption (quarts)

Tire consumption:

$$TC_s = a_t S^b + 0.03022(\Delta S)^{1.27}$$

where

TC_s = tire consumption (tire)

For the values of the coefficients see Table 12.6

Table 12.6 Running Cost Model Coefficients

Coef- ficient	Passenger car	Pickup truck	Single Unit (12 kip)	Semi- trailer (40 kip)	Semi- trailer (diesel)
IC	580	450	650	840	840
a_f	30	25	20	35	35
b_f	44	47	59	163	163
a_o	0.074	0.0672	0.051	0.011	0.048
b_o	-0.00452	-0.00056	-0.0005	0.001125	0.004
c_o	0	0	0	-0.000016	-0.00008
a_t	0.0185	0.0269	0.0621	0.1098	0.1490
b	1.29	1.22	1.20	1.26	1.25
a_{ch}	0.0000778	0.000096	0.000279	0.000615	0.000615
b_{ch}	0.56	0.51	0.50	0.62	0.62

N_n = number of lanes under normal operations (2-way)
 N_c = number of lanes under rehabilitation
 conditions (2-way)

The vehicle speeds were determined from the procedure outlined in the Highway Capacity Manual [HRB1965] (also used in EAROMAR2), and the proposed 1985 Highway Capacity Manual [TRB1984]. This involves the determination of the volume capacity (V/C) ratio. The EAROMAR2 equations developed to describe average travel speed - V/C ratio relationship were adjusted slightly for this study. Equations relating vehicle speed with pavement roughness and the speed limit presented in EAROMAR2 were used. One coefficient of the equation relating vehicle speed to the speed limit has to be specified by the user. A different value than that presented by Butler [1974] and used in EAROMAR2 is suggested. The equations used in the analysis are presented in Tables 12.7 and 12.8. The initial (base year) traffic volume is defined as the traffic volume (in vehicles per day) immediately after initial construction. This volume is specified by the user. All further traffic volumes are calculated using this value. The PSI values are predicted at the end of each year, but the average PSI values during the year are used in the vehicle speed calculations. The average speed and average traffic volumes are used in the user cost calculations.

Table 12.7 Volume-Capacity Ratio Equations

$$SF = MSF * N * f_w * f_{hv} * f_p$$

$$f_{hv} = 1/[1 - P_t (E_T - 1)]$$

where

SF = service flow rate

MSF = maximum service flow rate = 2000 vehicles per hour

N = number of lanes

f_w = adjustment factor for lane widths

f_p = adjustment factor for driver population

f_{hv} = adjustment factor for heavy traffic

P_T = proportion of heavy vehicles in traffic stream

E_T = passenger car equivalent = 2 for level sections

V/C = volume/SF = volume-capacity ratio

Table 12.8 Vehicle Speed Models

$$S_r = 21.4 + 0.04 * D_1 * PSI + 0.007 * S^2$$

$$S_1 = f_1 * D_1 - 3.6 * V/C$$

$$S_d = S1 - S4 - S6$$

$$S6 = (V/C) \left(\frac{0.6 * D_s - 27.5}{S} \right) - S5$$

$$S5 = S1 - 30 - S4$$

$$S4 = (0.4 * D_s - 15) * V/C$$

$$S1 = 0.58 * D_s + 20$$

where

S_r = speed based on roughness (mph)

S_1 = speed based on the speed limit (mph)

S_d = speed based on the V/C ratio (mph)

D_s = design speed (mph)

d_1 = speed limit (mph)

$f_1 = 0.9$ (EAROMAR2)

$f_1 = 1.1$ (PEARDAPP)

V/C = volume-capacity ratio

For explanation of S1, S2, S3, S4, S5, and S6 see Figure 12.3

Figure 12.3 Average Travel Speed versus V/C Ratio
[from Markow1984]

The running costs are calculated for a typical day. The V/C ratio and travel speed are calculated for every hour of the day. The hourly travel speeds are then weighted by volume to give a weighted travel speed which is used in the rest of the analysis. This is one of the major simplifications from the EAROMAR2 program. The typical traffic distribution for 24 hours is provided by the user.

Queuing is not explicitly considered in the traffic flow calculations. The effect of a change in highway capacity on vehicle speeds and travel times is considered through changes in the V/C ratio.

The effect on air pollution was not included in this analysis.

CHAPTER 13

THE USE OF PEARDARP

13.1 Introduction

The economic analysis procedure developed to analyze different aspects of rigid pavement pumping design (PEARDARP) and described in the previous chapter can be used to evaluate a large number of design and rehabilitation alternatives. An evaluation of all the different options is not possible in this chapter. The economic analysis program can be used for that. However, a set of design and rehabilitation alternatives will be analyzed to demonstrate the use of the program. Moreover, representative construction and rehabilitation activity unit costs are presented, which can be used in the analysis of alternatives.

The economic analysis program was not developed to replace total economic analysis systems, like EAROMAR Version 2. EAROMAR2 is much more flexible in accommodating traffic characteristics, roadway geometry and daily seasonal variations in maintenance unit costs.

EAROMAR2 further automatically applies maintenance when a certain specified distress level is reached. However, PEARDARP provides an improvement in the incorporation of the effect of design and rehabilitation techniques on the performance. Since EAROMAR2 was written in a modular format, the improvements contained in PEARDARP can be incorporated into EAROMAR2 to provide a program with the benefits of both. PEARDARP is more than adequate to analyze design and rehabilitation alternatives. The total sophistication provided by EAROMAR2 is ordinarily unnecessary, considering all the uncertainties involved in an economic analysis.

13.2 Cost Elements

The unit costs of the components of the economic analysis program are important and can have a large effect on the results. Typical construction costs are summarized in Table 13.1. A large number of sources were consulted to obtain average unit costs for the rehabilitation techniques. The rehabilitation costs vary considerably among states. Table 13.2 provides unit and production costs for rigid pavement rehabilitation techniques. Table 13.3 gives typical costs for fuel, oil, tire, accident, and time costs. All the unit costs are in 1984 dollars. Indices, like the Consumer Price Index or Producer Price Index, can be used to update unit costs, as necessary.

Table 13.1 Typical Construction Costs

Pavement slab:-

Plain Jointed PCC (no dowels):	\$65 per cy
Reinforced Jointed PCC (no dowels):	\$70 per cy
Dowels at 6.4 m (20 ft) spacing:	add \$2 per sy

Subbases:-

Asphalt stabilized (open-graded):	\$45 per cy
(dense-graded):	\$35 per cy
Cement stabilized (dense-graded):	\$35 per cy
Lean concrete (impervious):	\$50 per cy
Unstabilized material (dense-graded):	\$18 per cy
(open-graded):	\$20 per cy

Shoulders:-

Asphalt concrete:	\$50 per cy
Tied PCC-shoulder:	\$60 per cy
Asphalt stabilized (open-graded):	\$45 per cy
Unstabilized (dense-graded):	\$18 per cy

Drainage:-

Edge drains:	\$8 per lf
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Filter layers:-

Geofabrics:	\$1.50 per sy
Dense-graded subbase:	\$18 per cy

1 yard = 0.914 m

Table 13.2 Rehabilitation Activity Costs

ACTIVITY	UNIT	UNIT COST RANGE (\$)	AVERAGE UNIT COST (\$)	PRODUCTION
Grinding				
Soft aggr.	sy	2.30 - 3.40	3.00	2500-3500
Medium aggr.	sy	3.40 - 5.70	4.50	ft/day
Hard aggr.	sy	5.70 - 9.10	7.50	
Shoulder				200-300
repair	cf		4.50	cf/day
Underseal				
Asphalt (1)	cf		3.50	200-300
Cement (2)	cf	9.40 - 11.00	10.00	cf/day
Retrofit				2000-3000
drains	lf	2.50 - 18.0	8.00	lf/day
Crack sealing(3)				
Liquid asphalt	lf	1.20 - 5.00	1.40	2000-3000
Low modulus	lf		1.75	lf/day
Edge support				
PCC shoulder	cy		150	
Edge beam	cy		150	
Load transfer				
improvement	joint	200 - 250	225	
Relief joints	lf	56 - 180	110	100 lf/day
Full depth				
Asphalt	cy		150	15-25
PCC w/o LTD	cy	180 - 380	270	cy/day
PCC w LTD	cy	234 - 530	360	
Partial depth				
Asphalt	cy		150	20 cy/day
PCC w/o LTD	cy		200	
PCC w LTD	cy		235	
Resurfacing				
PCC : thin	sy		2 in.= 4.20	
	sy		3 in.= 6.30	
thick	sy		6.80+1.70*D	
Mudjacking	cf		10	150-250
Removal	sy		3.50	cf/day
Cracking/Seating	sy		0.75	

(1) one hole per joint

(2) three to five holes per joint

(3) includes cutting and cleaning

D = thickness (in.) LTD = load transfer device

1 in. = 25.4 mm

Table 13.3 Critical Pavement Distress Levels

DISTRESS TYPE	PLAIN JOINTED	REINFORCED	REFERENCE
Joint deterioration	140 joints/mi	53 joints/mi	Rauhut1982
Spalling	65 - 120 sq ft/1000 sq ft		Darter
PSI	2.5	2.5	Rauhut1982
Cracking	818 ft/mi	1500 ft/mi	Rauhut1982
	10 -25 lf/1000 sq ft		Darter1977
Faulting	0.25 in.	0.4 in.	Rauhut1982
	0.125 in.		NCHRP1979
	0.12 - 0.25 in.		Spellman1972
	0.19 in.		Brokaw1974
	0.15 - 0.19 in.		Packard1977
	0.16 in.		Gulden1974
	7.5 - 15 in./1000 ft		Darter

1 in. = 25 mm

13.3 Specification of Rehabilitation

PEARDARP was written to provide information regarding the changes in pavement distresses, user's consequences, and discounted costs with time for one alternative. PEARDARP does not have the capability to automatically apply a rehabilitation technique, when a limiting distress criterion is reached. The user must select a rehabilitation strategy based on the results of a specified design and/or rehabilitation alternative. Different criteria can be used to decide on a maintenance strategy. Typical distress values used as rehabilitation need criteria are presented in Table 13.3.

A large number of rehabilitation techniques and technique combinations can be used to correct or prevent a certain distress. The techniques were discussed in Chapter 10. Appendix C contains a list of useful publications on PCC pavement rehabilitation.

13.4 Possible Analyses Using PEARDARP

The PEARDARP program can be used to reach a number of conclusions, viz.,

1. The changes in pavement distress types can be predicted with time for different design and rehabilitation alternatives, since the program includes the effect of all the elements on the pavement performance.

2. Changes in user costs can be predicted. However, a program like EAROMAR2 allows for more flexibility in the specification of traffic characteristics.
3. Construction, rehabilitation, and user costs for different alternatives can be compared for a reference year or on an annual basis.

The PEARDARP program can consider only one alternative at a time. It also does not contain default maintenance strategies. The rehabilitation alternatives need to be specified by the user. The output provides the means to consider all the distress types and select an appropriate rehabilitation time and activity. The amount of rehabilitation must also be specified. The PEARDARP output provides enough information for this to be accomplished fairly easily. For example, the number of faulted joints greater than a certain fault value are given. If grinding is used to correct the faulting distress, the quantity of grinding needed can be obtained by multiplying the area to be ground at each joint with the number of faulted joints.

13.5 Examples of Analyses

The use of PEARDARP can best be described with an example. Tables 13.4 and 13.5 contain typical road user

Table 13.4 Typical User Costs

VEHICLE TYPE	FUEL (\$/gal)	OIL (\$/qrt)	TIRES (#/tire)	TIME (\$/hr)
Passenger car	1.30	1.30	65	5.75 per person
Pickup truck	1.30	1.30	65	5.75 per person
Single-unit	1.12	0.90	220	13.45 per vehicle
Combination	1.12	0.90	220	15.35 per vehicle
Diesel truck	1.12	0.90	220	15.35 per vehicle

Average automobile occupancy = 1.56 adults/vehicle

Accident rate = 0.915 per million vehicle miles (in 1 direction)

Accident cost = \$7300 per accident

All rates costs were updated to 1984, where necessary

[References: Wong1983, AASTHO1977, Zaniewski1982]

Table 13.5 Typical Traffic Characteristics

Traffic volume distribution on a rural road [Ref. Wong1984]:

HOUR ENDING	PROPORTION	HOUR ENDING	PROPORTION
0100	0.017	1300	0.056
0200	0.013	1400	0.060
0300	0.009	1500	0.066
0400	0.009	1600	0.046
0500	0.008	1700	0.083
0600	0.013	1800	0.074
0700	0.025	1900	0.065
0800	0.042	2000	0.055
0900	0.051	2100	0.047
1000	0.053	2200	0.038
1100	0.054	2300	0.033
1200	0.055	2400	0.027

Vehicle type distribution on a rural interstate [Ref. Baerwald1976]:

Passenger cars: 76%
 Single unit trucks: 9%
 Combination trucks: 15%

Design load characteristics [Ref. AI1981]

Truck factor: 0.42
 Trucks in design lane: 0.9

costs and traffic characteristics. Table 13.6 provides the pavement, traffic, geometric, and climatic properties used in the examples. A detailed description of construction cost, rehabilitation techniques, and rehabilitation costs are presented in Appendix E.

13.5.1 Example 1: Effect of Subdrainage

The effects of subdrainage on pavement condition and user costs are investigated in the first example. The pavement and traffic characteristics are described in Table 13.6. The one alternative consists of a pavement with a granular subbase without edge drains, while the other alternative utilizes the same granular subbase with edge drains. The subdrainage can be classified as "fair" and "good", according to the information in Table 12.4. Figure 13.1 portrays the changes in present serviceability index (PSI) with time for a pavement with edge drains ("good" subdrainage) and for a pavement without edge drains ("fair" subdrainage condition). The pavement with the edge drains reach a PSI value of 2.5 after 18 years, while a PSI value of 2.5 is reached after only 13 years for the pavement without an edge drain. The user costs are also influenced by subdrainage, and the effect is shown in Figure 13.2.

In order to compare the alternatives in an economic analysis, the method of comparison has to be established.

Table 13.6 Pavement and Traffic Characteristics

PCC slab :	Length = 20 ft.	Thickness = 10 in.
	Mr=650.0 psi	E = 3000000 psi
	Dowels: no	Reinforcement: no
Subbase :	Type= unstabilized	Thickness = 6 in.
	Drainage= fair (varied in Example 1)	
Subgrade :	k=150.0 pci	Drainage = poor
Traffic :	Volume = 10000 vpd (varied in Example 3)	
	Truck factor = 0.42	Trucks in design lane = 90%
	Growth rate = 4.%	
	Passenger cars = 70.%	Pick-up trucks = 6%
	S-U trucks = 9%	Combination trucks = 10%
	Diesel trucks = 5%	
	Traffic volume distribution from Table 13.5	
Geometric:	Design speed = 60 mph	Speed limit = 55 mph
	No. lanes = 2.0	Length = 1 mi
Climate:	cold, wet

Figure 13.1 Effect of Subdrainage on PSI

Figure 13.2 Effect of Subdrainage on User Cost

The PEARDARP program allows the user to compare costs on an annual basis or at a reference year for any analysis period. The analysis period is specified by the user.

The present values or equivalent annual cost of alternatives with equal analysis periods can be compared. When the analysis periods are not equal, but the cash flow cycles are identically repeated, the equivalent annual cost of the alternatives can be compared. Repeating identical cash flows can generally not be assumed in the analysis of pavements, since the traffic volume usually increases nonlinearly with time. Thus, user costs will not be constant in each cycle. The length of time between rehabilitation applications for a particular alternative will also not be the same, since the pavement distresses are affected nonlinearly by traffic volume and age. However, this approach can be used when only the construction costs of alternatives are compared. The results using this procedure on the pavement design alternatives in Example 1, are presented in Table 13.7 (Method 1).

The analysis period for pavements are often taken as the time between construction and the time when a certain distress level is reached. The analysis periods for pavements are seldomly the same since pavements reach the selected distress level at different times. If a constant analysis period is used, the differences in pavement

Table 13.7 Example 1: Results

	GOOD		FAIR				
	C&R	User	C&R	User	ΔC&R	ΔUser	ΔALL
Construction	440	--	400	--	40	--	--
Rehab. at 26 years	52	6	54	11	+2	+5	--
Method 1 (Annual Cost)	47	2650	51	2526	+4	-124	-120
Method 2 (Annual Cost)	41	2942	37	3026	-4	+84	+80
Method 3 (Annual Cost)	41	2937	38	3020	-3	+83	+80

All costs in \$1000

(+) indicates a savings compared to the reference (FAIR) case

C&R : Construction and Rehabilitation cost

GOOD : Pavement with edge drains

(analysis period use in Method 1 = 18 yr)

FAIR : Pavement without edge drains

(analysis period use in Method 1 = 13 yr)

Method 1 : Analysis over the period until PSI reaches 2.5

Method 2 : Analysis over 26 years with no rehabilitation
after 25 years

Method 3 : Analysis over 26 years with rehabilitation (grinding,
undersealing, and partial patching) after 25 years

Discount rate = 8% per year

conditions at the end of the analysis period have to be considered. For example, one design and alternative may produce a pavement with a PSI of 1.6 after 25 years, and another alternative, a pavement with a PSI of 2.0 after 25 years (Example 1). The pavement with the PSI of 2.0 is worth more than the pavement with a PSI of 1.6. The effect of the pavement condition is to some extent included in the user costs, since the PSI effects the user costs through changes in vehicle speeds. The worth of differences in distress levels is difficult to define and determine, and may be small compared to the present value of the construction and rehabilitation costs. Therefore, the effect of the differences in pavement condition at the end of the analysis period is often neglected in the economic analysis of pavement design and rehabilitation alternatives [Shandler1984, Kulkarni1984, Wong1984, Darter1985]. The results of such an analysis over 26 years, with the pavement designs in Example 1 as alternatives are presented in Table 13.7 (Method 2).

An effort was made to determine the difference in worth of the pavements at the end of the analysis period. A salvage value can usually not be used to determine the worth of pavements with different distress levels, since the removal and recycling costs are not affected by pavement condition. However, the differences in rehabilitation costs to bring the pavements to the same

distress level can be used as a measure of the worth of the pavements. Grinding, undersealing, and partial patching were used to upgrade both pavements to approximately the same PSI at the end of 25 years. Since the rehabilitation is applied in PEARDARP at the beginning of the twenty sixth year, the analysis was conducted over 26 years. The results are presented as Method 3 in Table 13.7. A discount rate of 8% was used in all cases.

Methods 2 and 3 give basically the same results. The inclusion of the worth of the pavement at the end of the analysis period is not important when the analysis period is long, the interest rates are high, or the differences in rehabilitation costs of the alternatives at the end of the analysis period are small. The effect of the difference in pavement condition at the end of the analysis period will not need to be included in most analyses. The use of the equivalent annual cost over the life of each alternative can be used only to compare initial construction costs. The effect of user costs can not be analyzed using this method.

13.5.2 Example 2: Effect of Rehabilitation Techniques

Using the pavement without the edge drains in Example 1, the effect of different rehabilitation techniques was evaluated. Five rehabilitation techniques were evaluated:

1. Alternative 1 (ALTER1): Grinding, undersealing, and partial depth patching.
2. Alternative 2 (ALTER2): The installation of retrofit drains.
3. Alternative 3 (ALTER3): Grinding, undersealing, partial depth patching, and installation of retrofit drains.
4. Alternative 4 (ALTER4): Full depth patching.
5. Alternative 5 (BASE): No rehabilitation.

The rehabilitation is applied at the beginning of year 14. The effects of the different rehabilitation applications are shown in Figure 13.3. Alternative 3 will increase the pavement life more than any of the other alternatives. The installation of retrofit drains (alternative 2) does not improve the condition of the pavement, it only reduces the rate of distress progression. The savings in user costs, as compared to the no rehabilitation (BASE) case, are displayed in Figure 13.4. Table 13.8 summarizes the results of the analysis.

13.5.3 Example 3: Feasibility of Edge Drains

The use of edge drains on pavements results in savings to the road user, as shown in Example 1. However, at low traffic volumes, the cost of the edge drains may be more than the user cost benefits. PEARDARP can be used to generate results that can be used to determine the feasibility of using edge drains. Figure 13.5 displays

Figure 13.3 Effect of Different Rehabilitation Techniques on PSI

Figure 13.4 Effect of Different Rehabilitation Techniques on Savings in User Costs

Table 13.8 Example 2: Results

	BASE	ALTER1	ALTER2	ALTER3	ALTER4
Construction cost	400	400	400	400	400
Rehab. cost at 14 years	0	48	52	90	118
User costs during rehab.	0	8	5	11	15
Equivalent annual C&R	37	39	39	40	41
Equivalent annual user cost	3026	2918	3014	2902	2960
Δ C&R	—	-2	-2	-3	-4
Δ User costs	—	+108	+12	+124	+66
Δ All costs	—	+106	+10	+121	+62

All costs in \$1000

(+) indicates a savings compared to the reference (BASE) case

C&R : Construction and Rehabilitation cost

BASE : no rehabilitation

ALTER1 : Grinding, undersealing, and partial depth patching

ALTER2 : Installation of retrofit drains

ALTER3 : Grinding, undersealing, partial depth patching,
and the installation of retrofit drains

ALTER4 : Full depth patching

Discount rate = 8% per year

the present value of the user cost savings at different Σ ESAL levels for the two pavements used in Example 1, for 3 slab thicknesses. The analysis period in the example was 20 years and the road section was 1.6 km (1 mi) long. The discount rate was again 8%. Figure 13.5 can be used to determine at what traffic levels edge drains are feasible for the pavement and conditions used in the example. For example, if the Σ ESAL over 20 years is 1 million, edge drains will be feasible if they cost less than \$53000 (\$10 per lf on one side) for a 200 mm (8 in.) pavement slab. An edge drain will be feasible for a pavement with a slab of 300 mm (12 in.) if the Σ ESAL is more than about 2.6 million.

13.6 Concluding Remarks

Only three examples were discussed in this Chapter to illustrate the use of PEARDARP. The examples were selected to demonstrate the use of the program and not to present a complete economic evaluation of all rigid pavement design and rehabilitation alternatives. The evaluation method (present value or equivalent annual cost) and the costs and benefits used (construction and rehabilitation costs, vehicle operating costs, time costs, or any combination of these) were not fully explained. This is nevertheless an important consideration and the PEARDARP user is advised to recognize the effect of the possible evaluation methods.

Figure 13.5 User Benefits against Cumulative Axle Loads

CHAPTER 14

CONCLUSIONS AND RECOMMENDATIONS

14.1 Conclusions

1. The literature review showed that pumping has been a problem for many years and still is one of the major contributors to rigid pavement distress. Surface erosion is recognized as the cause of the removal and redistribution of fines from stabilized materials. Various methods have been applied over the years to minimize pumping. Most of these methods involved the subbase layer. Subbases that have been used include, dense-graded, open-graded, stabilized, open-graded stabilized, lean concrete, and asphalt cement materials.
2. A survey of highway agency practices and experiences indicated that open-graded, lean concrete, and asphalt cement (used as a cap on the subbase) performed well. The experiences with impervious stabilized layers are mixed, while dense-graded unstabilized materials do not perform well, in general.

3. Three testing techniques were used to rate the erodibility of pavement materials, viz., a jetting device, a brush test, and a rotational shear device. The brush test is a very simple test and was used to characterize the erosion of a large number of stabilized samples. The brush test was successful in comparing the erosion of different lean concrete samples, but was not successful in comparing the erosions of lean concrete materials and cement stabilized materials.

The use of the jetting test to characterize the erosion of unstabilized materials was the least successful of the tests. Although differences in erosion among unstabilized samples could be detected, the accuracy of the calculated and measured shear stresses are suspect.

The rotational shear device was successful in determining the critical shear stresses and erosion rates of cement stabilized materials. A relationship was developed between the brush erosion and the critical shear stress determined from the rotational shear test.

4. Cement content is the most important factor in the erodibility of cement stabilized materials. The

compaction effort and gradation are also important, but to a lesser extent. Environmental factors, e.g., freeze-thaw and wet-dry cycles, are only important when: the cement content is low, the compaction effort is low, and the material contains a large percentage of fines.

5. The erosion of asphalt stabilized materials is affected by the asphalt content, the compaction effort, and environmental factors. Wetting-and-drying has a larger influence on the erosion of asphalt stabilized materials than freezing-and-thawing.
6. Relationships were developed relating the brush erosion of asphalt and cement stabilized materials to three material properties and two environmental factors. The asphalt stabilized material exhibited, in general, lower erosion than the cement stabilized materials. However, asphalt stabilized material is subject to stripping and this was not explicitly included in the testing program.
7. Indications are that the shear stresses induced by the water under the slab are higher than the critical shear stress for unstabilized samples. Therefore, impervious unstabilized materials will always be affected by pumping due to surface erosion. In less

impervious unstabilized layers, the pore water pressure buildup is the controlling pumping mechanism.

8. Stabilized materials may be eroded in a pavement, depending on their properties, mainly the asphalt and cement contents. A family of curves were developed for four gradation-compaction⁴-effort combinations in each of four climatic regions, relating the normalized erosion to cement or asphalt content. These relations can be beneficial in the selection of rigid pavement subbase or shoulder materials to prevent pumping.
9. Pumping damage models were used to develop adjustment factors for subbase type, drainage, dowels, and climatic conditions for a pumping prediction model based on AASHO Road Test data.
10. The effect of rehabilitation techniques on pumping related distresses is discussed and typical rehabilitation cost values are presented.
11. An economic analysis program, PEARDARP, was developed to evaluate rehabilitation and design alternatives to prevent pumping. The effects of different rehabilitation techniques on pavement performance are included in the program. The program was written to supplement, and not replace, programs such as

EAROMAR2. PEARDARP can be used to compare design or rehabilitation alternatives and/or to improve existing programs, e.g. EAROMAR2.

14.2 Recommendations for Further Research

1. The method to test the erosion of unstabilized subbase and shoulder materials must be refined through further research. The effect of the de-icing salts on erosion should also be evaluated.
2. The behavior of the water under the slab, including the shear stresses induced by the water on the subbase, needs to be studied further.
3. The adjusted pumping prediction model requires verification and subsequent improvement. The development of the pumping prediction model based on theoretical considerations should be pursued.
4. More research is necessary to quantify the effects of different rehabilitation methods on pavement distress progression.

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APPENDICES

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Appendix DPEARDDARP Listing and Example

1. PEARDARP LISTING

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C *****
C *
C *      PURDUE ECONOMIC ANALYSIS OF REHABILITATION AND DESIGN      *
C *      ALTERNATIVES TO PREVENT PUMPING (PEARDAPP)                  *
C *
C *      by A. J. van Wijk                                           *
C *      April 1985                                                 *
C *      Purdue University, West Lafayette, Indiana                 *
C *
C *      Written in Fortran F77 on an UNIX system                    *
C *
C *****
C
common tesal(0:50),pi(0:50),npump(0:50),PU(0:50),TOTAL4(0:50)
common cr(0:50),r(0:50),avgflt(0:50),nspall(0:50),ATOTAL4(0:20)
common ps(0:50),resal(0:50),fSl,mk,ffm,pesal(0:50),PVTOTAL4(0:20)
common FL(5,50),OIL(5,50),TC(5,50),psi(0:50),S(0:50),X(5,15)
common YCOST(5,50),ZCOST(5,50),TCOST(5,50),ACOST(0:50),Y(5,5)
common TOTAL1(0:50),TOTAL2(0:50),TOTAL3(0:50),Z(15),DA(0:50)
common psiavg(0:50),Savg(0:50),vavg(0:50),v(0:50),nyr,vol,h,pfm
common PVTOTAL1(0:20),ATOTAL1(0:20),pwf(0:50),crf(0:20),cfm,rfm
common PVTOTAL2(0:20),ATOTAL2(0:20),PVTOTAL3(0:20),ATOTAL3(0:20)
common dis(24),pt,et,nl,msf,Ds,S1m,vh(0:50),fw,Sp(0:50),nper
common EXZCOST(5,50),EXTCOST(5,50),EXACOST(0:50),TMCOST(0:50)
common CONST,TOTMAC(0:50),PVTOTMAC(0:50),ATOTMAC(0:50),nsp,add
common nf,ctraffic,cls,cnl,ctime,cS1m,myr,nfin,n,ni,nter,icor
common Fn(50,2),F(0:50),P(0:50),cesal(0:50),fv(0:50),seal,l
common dowel,sbdrain,sgdrain,spa(0:50),pa(0:50),tv(0:50)
common ncorp,ncorf,ncorc,ncorr
dimension esal(0:50),NPI(0:50)
character*4,sbdrain,sgdrain,sbtype,sealtype,dowel
character*4,seal
character*5,climate,reinforce
character*50,title
character*3,dow,rforce
character*1,option
character*9,clim
integer,i,n,j,myr,nfin,n,ni,nter,icor,nf,mk,nsp,eff,nper
integer,ncorp,ncorf,ncorc,ncorr
real,esal,ig,FL,OIL,TC,S,psi,X,esal0,K1,K2,fcc,fc
real,fw,et,msf,fSl,vol,cpsi,hs,K5,K6,K7,K8,fcf,fcp,P1,F1,D1,R1
real,h,l,mr,E,k,fpt,tf,lf,f2,b,fcrit1,fcrit2,vol,ps,S1m,Ds
real,npump,avgflt,P,nl,nspall,c1,f3,Fn,per,F,fv,vavg
real,fdow,fpr,fdr,fsbl,fsb2,fz,NPI,Fp,K3,K4,traf,critflt

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c
c Coefficients used in vehicle running cost equations
c
  data X(1,1),X(1,2),X(1,3),X(1,4),X(1,5),X(1,6),X(1,7),X(1,8),
  +X(1,9),X(1,10),X(1,11),X(1,12),X(1,13)/30,44,0.000031,2.5,0.074,
  +-0.000452,0,0.00084,1.29,580,1,0.00007784,0.56/
  data X(2,1),X(2,2),X(2,3),X(2,4),X(2,5),X(2,6),X(2,7),X(2,8),
  +X(2,9),X(2,10),X(2,11),X(2,12),X(2,13)/25,47,2.6e-10,5.08,0.0672,
  +-0.00056,0,0.00084,1.22,450,1,0.000096,0.51/
  data X(3,1),X(3,2),X(3,3),X(3,4),X(3,5),X(3,6),X(3,7),X(3,8),
  +X(3,9),X(3,10),X(3,11),X(3,12),X(3,13)/20,59,0.003,1,0.05,-0.0005,
  +0,0.00094,1.2,650,1,0.000279,0.50/
  data X(4,1),X(4,2),X(4,3),X(4,4),X(4,5),X(4,6),X(4,7),X(4,8),
  +X(4,9),X(4,10),X(4,11),X(4,12),X(4,13)/35,163,0.0029,1,0.011,
  +0.001125,-0.0000165,0.000915,1.26,840,1,0.000615,0.62/
  data X(5,1),X(5,2),X(5,3),X(5,4),X(5,5),X(5,6),X(5,7),X(5,8),
  +X(5,9),X(5,10),X(5,11),X(5,12),X(5,13)/35,163,0.003,1,0.048,0.004,
  +-0.00008,0.001242,1.25,840,0.65,0.000615,0.62/
c
c ***** READ INPUT VALUES *****
c
c Related to title
  read^(a50)^,title
c
c Related to the pavement
  read*,h,l,E,mr,k,hs
c
c Related to drainage and subbase type
  read l,sbdrain,eff,sgdrain,sbtype,sealtype,dowel,reinforce,climate
  1 format(a4,i,4(x,a4),x,a5,x,a5)
c
c Related to the maintenance
  read*,fcrit1,fcrit2,sdflt,vvoid
c
c Related to traffic and users' cost calculations
  read*,vol,tf,lf,ig,Ds,Slm,nl,fw,et,msf,fSl
c
c Related to volume distributions during 24 hours
  read^(24f3.3)^,(dis(i),i=1,24)
c
c Related to traffic and accident cost data
  read*,(Z(i),i=1,8)
c
c Fuel, oil, tire and time costs
  do 20 i=1,5
    20 read*,(Y(i,jj),jj=1,4)
c
c Read analysis period
  read*,nper,nfin,ni

```

```

c Read present value and equivalent annual cost option
  read^(a)^,option
c
c Read construction costs
  read*,CONST
c
c Read year of first maintenance
  read*,myr
c
c ***** PRINT INPUT INFORMATION *****
c
  print^("*****")^
  print^("t3,a50,t54")^,title
  print^("*****")^
  if (dowel.eq.^dowl^ ) then
    dow=^yes^
  else
    dow=^ no^
  endif
  if (reinforce.eq.^reinf^ ) then
    rforce=^yes^
  else
    rforce=^ no^
  endif
  print^(/,t5,"TABLE I : INITIAL PAVEMENT AND TRAFFIC INFORMATION")^
  print^ (t5,"-----",/)^
  print^("PCC SLAB : Length (ft) = ",f3.0," ; Thickness (in.) = ",
+ f4.1)^,l,h
  print^ (t12,"Mr (psi) = ",f5.1," ; E (psi) = ",f9.1)^,mr,E
  print^ (t12,"Dowels: ",a3,2x," ; Reinforcement: ",a3)^,dow,rforce
  print^ (/,"SUBBASE : Type: ",a4," ; Thickness (in.) = ",f4.1)^,
+ sbtype,hs
  print^ (/,"SUBDRAINAGE : ",a4,i)^,sbdrain,eff
  print^ (/,"SUBGRADE : k (pci) = ",f5.1," ; Drainage: ",a4)^,k,
+ sgdrain
  print^ (/,"TRAFFIC : Volume (vpd) = ",f7.0," ; Growth rate (%) = ",
+ f3.0)^,vol,ig
  print^ (t12,"Pass. cars (%) = ",f3.0," ; Pick-ups (%) = ",f3.0)^,
+ Z(1),Z(2)
  print^ (t12,"S-U trucks (%) = ",f3.0," ; Comb. Trucks (%) = ",f3.0)^,
+ Z(3),Z(4)
  print^ (t12,"Diesel trucks (%) = ",f3.0)^,Z(5)
  print^ (t12,"Truck factor = ",f5.3)^,tf
  print^ (t12,"Proportion of trucks in design lane = ",f4.2)^,lf
  print^ (/,"GEOMETRY : Design speed (mph) = ",f4.1," ; Speed limit
+ (mph) = ",f4.1)^,Ds,Slm
  print^ (t12,"No. lanes = ",f3.1," ; Length (mi) = ",f3.1)^,nl,Z(6)
  print^ (/,"USERS COSTS:")^
  print^ (t14,"Gas($/gal)",t26,"Oil($/qrt)",t38,"Tires($/tire)",
+ t54,"Time($/hr)")^

```

```

    print("Pass. cars:",t17,f4.2,t28,f4.2,t42,f5.1,t55,f5.1)^,
+Y(1,1),Y(1,2),Y(1,3),Y(1,4)
    print("Pick-ups   :",t17,f4.2,t28,f4.2,t42,f5.1,t55,f5.1)^,
+Y(2,1),Y(2,2),Y(2,3),Y(2,4)
    print("S-U trucks :",t17,f4.2,t28,f4.2,t42,f5.1,t55,f5.1)^,
+Y(3,1),Y(3,2),Y(3,3),Y(3,4)
    print("Comb.trucks:",t17,f4.2,t28,f4.2,t42,f5.1,t55,f5.1)^,
+Y(4,1),Y(4,2),Y(4,3),Y(4,4)
    print("Diesel     :",t17,f4.2,t28,f4.2,t42,f5.1,t55,f5.1)^,
+Y(5,1),Y(5,2),Y(5,3),Y(5,4)
    if (climate.eq.^wmdry^) then
        clim=^warm, dry^
    elseif (climate.eq.^cddry^) then
        clim=^cold, dry^
    elseif (climate.eq.^wmwet^) then
        clim=^warm, wet^
    else
        clim=^cold, wet^
    endif
    print(/,"CLIMATE: ",a9)^,clim
    print(/,"CRITICAL FAULT1 (in.) =",f6.3)^,fcrit1
    print(t10,"FAULT2 (in.) =",f6.3)^,fcrit2
    print(t10,"S.D. (in.) =",f6.3)^,sdflt
    print(/,"*****")^
c
c ***** PRINT CONSTRUCTION AND MAINTENANCE COSTS *****
c
    print(/,t15,"TABLE II : CONSTRUCTION AND MAINTENANCE COSTS ($)")^
    print(t16,"-----")^
    print(/,t15,"CONSTRUCTION AND MAINTENANCE",t53,
+"ADDITIONAL USERS COSTS ")^
    print("YEAR",t12,"ACTIVITY",t30,"COSTS ",t40,"TOTAL ",t50,
+"VEH. OPER.   TIME   ACCIDENT")^
    print(" 0 ",t7,"Construction",t36,f10.0)^,CONST
    mk=0
c
c ***** VOLUME CALCULATIONS *****
c pt=% trucks : fpt=proportion of trucks in design lane
c esal=18 kip single axle load per year : tesal=cumulative esal
c -----
c
    pt=(Z(3)+Z(4)+Z(5))/100
    fpt=pt*tf*lf
    esal0=365*fpt*(vol+vol*(1+ig/100.0))/2000000.0
    res=100.0
    pa(0)=0.0
    icor=0

```

```

ncorp=0
ncorf=0
ncorc=0
ncorr=0
fcp=0.0
fcf=0.0
fcc=0.0
fcr=0.0
c Start the yearly calculations
  nsp=-1
  nf=-1
  do 10 n=0,nfin
    v(n)=vol*((1+ig/100.0)**n)
    if (n.eq.myr) then
      call maintenance
      call mainconsump
    else
      go to 30
    endif
30  esal(n)=esal0*(1+ig/100.0)**n
    tesal(n)=tesal(n-1)+esal(n)
    tesal(0)=0.0
    if (tesal(n).eq.0.0) then
      traf=0.0
    else
      traf=alog10(tesal(n))
    endif
c
c ***** DRAINAGE AND SUBBASE TYPE *****
c
  if (dowel.eq.'dowl') then
    fdow=1.0
  else
    fdow=1.17 - 0.68*traf + 0.078*h
  endif
  if (sbdrain.eq.'exel') then
    fSD=0.1
    fdr=0.01
  elseif (sbdrain.eq.'good') then
    fSD=0.6
    fdr=0.68 + 0.15*traf-0.04*h
  elseif (sbdrain.eq.'fair') then
    fSD=1.0
    fdr=0.91 + 0.12*traf - 0.03*h
  else
    fSD=1.4
    fdr=1.0
  endif
  if (sbtype.eq.'stab') then

```



```

b=0.037
fsb1=0.18*traf+0.65
fsb2=-0.02*h+0.91
kcd=10**(0.7405*log10(hs)+0.7256*log10(k)+0.5559)
else
b=0.241
fsb1=1.00
fsb2=1.00
kc=10**(0.3483*log10(hs)+0.8163*log10(k)+0.2394)
kcd=1.7*kc
endif
if (sgdrain.eq.'good') then
f2=1.0
fsg=0.57 + 0.21*traf
else
f2=2.0
fsg=1.0
endif
if (sealtype.eq.'lmod') then
nyr=10
else
nyr=2
endif
if (climate.eq.'wmdry') then
fpr=0.26*traf-0.07*h+0.89
fz=0.011+0.0026*traf-0.001*h
elseif (climate.eq.'wmwet') then
fpr=-0.06*traf+0.018*h+0.96
fz =-0.3*traf-0.057*h+1.44
elseif (climate.eq.'cddry') then
fpr=0.26*traf-0.07*h+0.89
fz=0.32*traf-0.08*h+1.04
else
fpr=-0.06*traf+0.018*h+0.96
fz=-0.85*traf+0.19*h+0.54
endif
c
c ***** PUMPING *****
c DA in sq yd
c P(n) in cf per section length
c
if (reinforce.eq.'reinf') then
Fp=fsb2*fz
else
Fp=fsb1*fdr*fdow*fpr *fsg
endif
pesal(n)=pesal(n-1)+esal(n)
pesal(0)=0.0

```

```

if (n.eq.ncorp) then
  if (icor.eq.1 .and. pesal(ncorp-1).gt.0.0) then
    DE1=10**(3.5754-0.3323*h)
    K5=pesal(n-1)*DE1*100.0
    K6=(-2.884+1.652*alog10(K5))
    P1 = Fp*exp(K6)
    fcp=NPI(n-1)-P1
  else
    fcp=0.0
  endif
else
  fcp=fcp
endif
if (pesal(n).eq.0) then
  P(n)=0.0
else
  DE=10**(3.5754-0.3323*h)
  K3=pesal(n)*DE*100.0
  K4=(-2.884+1.652*alog10(K3))
  NPI(n) = Fp*exp(K4)
endif
P(n)=(NPI(n)+fcp)*36.67*Z(6)
npump(n)=P(n)/vvoid
PU(n)=P(n)+npump(n)*1.0
c
c ***** FAULTING *****
c avgflt in in.
c
  nf=nf+1
  fv(n)=fv(n-1)+v(n)
  fv(0)=0.0
  if (n.eq.ncorf) then
    if (icor.eq.1 .and. fv(ncorf-1).gt.0.0) then
      K7=48.95*(f2**0.61)*((1-13.5)**b)/(h**3.9)*fSD
      K8=(fv(n-1)*1f*pt/n)*nf**2
      F1=(1.29+(K8**0.465*K7))/32.0
      fcf=avgflt(n-1)-F1
    else
      fcf=0.0
    endif
  else
    fcf=fcf
  endif
  K1=48.95*(f2**0.61)*((1-13.5)**b)/(h**3.9)*fSD
  if (n.eq.0) then
    K2=0.0
  else
    K2=(fv(n)*1f*pt/n)*nf**2

```

```

endif
avgflt(n)=((1.29+(K2**0.465*K1))/32.0)+fcf
if (dowel.eq.'dowl') then
  F(n)=avgflt(n)/(1+nf**0.5)
else
  F(n)=avgflt(n)
endif
c Assume a normal distribution to calculate the number of joints
c
do 900 j=1,2
  if (j.eq.1) then
    critflt=fcrit1
  else
    critflt=fcrit2
  endif
  zflt=(critflt-F(n))/sdflt
  if (zflt.ge.-0.5 .and. zflt.lt.0.0) then
    per=100-(30.85+38.3*(zflt+0.5))
  elseif (zflt.ge.-1.0 .and. zflt.lt.-0.5) then
    per=100-(15.87+30*(zflt+1.0))
  elseif (zflt.ge.-1.5 .and. zflt.lt.-1.0) then
    per=100-(7.68+18.38*(zflt+1.5))
  elseif (zflt.ge.-2.0 .and. zflt.lt.-1.5) then
    per=100-(2.28+8.8*(zflt+2.0))
  elseif (zflt.ge.-2.5 .and. zflt.lt.-2.0) then
    per=100-(0.13+4.30*(zflt+2.5))
  elseif (zflt.ge.-3.0 .and. zflt.lt.-2.5) then
    per=100-(0.0+0.26*(zflt+3.0))
  elseif (zflt.ge.0.0 .and. zflt.lt.0.5) then
    per=100-(50+38.3*zflt)
  elseif (zflt.ge.0.5 .and. zflt.lt.1.0) then
    per=100-(69.15+30*(zflt-0.5))
  elseif (zflt.ge.1.0 .and. zflt.lt.1.5) then
    per=100-(84.38+18.38*(zflt-1.0))
  elseif (zflt.ge.1.5 .and. zflt.lt.2.0) then
    per=100-(93.32+8.8*(zflt-1.5))
  elseif (zflt.ge.2.0 .and. zflt.lt.2.5) then
    per=100-(97.72+4.30*(zflt-2.0))
  elseif (zflt.ge.2.5 .and. zflt.lt.3.0) then
    per=100-(99.87+0.26*(zflt-2.5))
  elseif (zflt.gt.3.0) then
    per=0
  else
    per=100.0
  endif
  Fn(n,j)=per*(5280/(100*1))*Z(6)
900 continue

```

```

c
c ***** CRACKING *****
c DA in sq in.
c cr in lf per 1000 sf
c
  cesal(n)=cesal(n-1)+esal(n)
  cesal(0)=0.0
  if (n.eq.ncorc) then
    if (icor.eq.1 .and. pesal(ncorc-1).gt.0.0) then
      D1=(exp(atan(39.006+3.941*alog10(cesal(n-1)*1000000)
+4.387*h-0.036*kcd)*6))*5280*Z(6)/(144*1*9)
      fcc=DA(n-1)-D1
    else
      fcc=0.0
    endif
  else
    fcc=fcc
  endif
  if (cesal(n).eq.0) then
    DA(n)=0.0
  else
    DA(n)=((exp(atan(39.006+3.941*alog10(cesal(n)*1000000)
+4.387*h-0.036*kcd)*6))*5280*Z(6)/(144*1*9))+fcc
  endif
  cr(n)=(DA(n)/27.78)*2.0*9
  cpsi=cr(n)/(Z(6)*63.36)
c
c ***** SPALLING *****
c
  nsp=nsp+1
  c1=0.0000162*(nsp**3.0806)
  f3=1-exp(-c1*(1-8))
  nspall(n)=(5280/1)*f3*Z(6)
c
c ***** PATCHING *****
c spa in sf per 1000 sf
c
  spa(n)=spa(n-1)+pa(n)
  spa(0)=0.0
  pat=spa(n)/(63.36*Z(6))
c
c ***** PSI *****
c
  if (n.eq.ncorr) then
    if (icor.eq.1 .and. pesal(ncorr-1).gt.0.0) then
      cc2=-50.088-3.775*h+30.644*(h**0.5)
      cc3=-6.697+0.139*(h**2)
      cc4=((30.56+(h**2))**0.5)-0.675*h
      cc5=E/k
      cc6=alog10((mr/690)*((4*(alog10(8.789*(h**0.75)/cc4))+0.359)/(4*
+(alog10((cc5**0.25)*0.54*(h**0.75)/cc4))+0.359)))

```

```

cc7=10**(1.774*cc6)
R1=360
+-216*((1.5-1.0/(1+exp(-cc2/(cc3*cc7))))+(1.0/(1+exp((resal(n-1)
+-cc2)/(cc3*cc7)))))
    fcr=r(n-1)-R1
    else
        fcr=0.0
    endif
else
    fcr=fcr
endif
c2=-50.088-3.775*h+30.644*(h**0.5)
c3=-6.697+0.139*(h**2)
c4=((30.56+(h**2))**0.5)-0.675*h
c5=E/k
c6=alog10((mr/690)*((4*(alog10(8.789*(h**0.75)/c4))+0.359)/(4*
+(alog10((c5**0.25)*0.54*(h**0.75)/c4))+0.359)))
c7=10**(1.774*cc6)
resal(n)=resal(n-1)+esal(n)
resal(0)=0.0
r(n)=360-216*((1.5-1.0/(1+exp(-c2/(c3*c7))))+(1.0/(1+exp((resal(n)
+-c2)/(c3*c7))))) + fcr
svr=0.000145*(r(n)**2.255)
svf=(F(n)/(0.008213*(1**0.5804)))*1.7229
sv=svr+svf
ps(n)=5.41-1.80*alog10(1+sv)-0.09*(cps1+pat)**0.5
psi(n)=amin1(4.5,ps(n))
10 continue
c call vehicle speed subroutine
    call speed
c call pavement distress print subroutine
    call prntdistr
c call the consumption subroutine
    call consump
c call consumption print subroutine
    call prntcons
c call users' cost print subroutine
    call prntuser
c call PV and Annual cost subroutine
    call pwvalue
c call yearly costs print subroutine
    call prntcosts
    if (option.eq.'n') go to 40
c call PV and Annual cost print subroutine
    call prntpv
40 stop
end

```

```

C *****
C ***** Subroutine to determine maintenance costs *****
C *****
C
      subroutine maintenance
      common tesal(0:50),pi(0:50),npump(0:50),PU(0:50),TOTAL4(0:50)
      common cr(0:50),r(0:50),avgflt(0:50),nspall(0:50),ATOTAL4(0:20)
      common ps(0:50),resal(0:50),fSl,mk,ffm,pesal(0:50),PVTOTAL4(0:20)
      common FL(5,50),OIL(5,50),TC(5,50),psi(0:50),S(0:50),X(5,15)
      common YCOST(5,50),ZCOST(5,50),TCOST(5,50),ACOST(0:50),Y(5,5)
      common TOTAL1(0:50),TOTAL2(0:50),TOTAL3(0:50),Z(15),DA(0:50)
      common psiavg(0:50),Savg(0:50),vavg(0:50),v(0:50),nyr,vol,h,pfm
      common PVTOTAL1(0:20),ATOTAL1(0:20),pwf(0:50),crf(0:20),cfm,rfm
      common PVTOTAL2(0:20),ATOTAL2(0:20),PVTOTAL3(0:20),ATOTAL3(0:20)
      common dis(24),pt,et,nl,msf,Ds,SIm,vh(0:50),fw,Sp(0:50),nper
      common EXZCOST(5,50),EXTCOST(5,50),EXACOST(0:50),TMCOST(0:50)
      common CONST,TOTMAC(0:50),PVTOTMAC(0:50),ATOTMAC(0:50),nsp,add
      common nf,ctraffic,cls,cnl,ctime,cSIm,myr,nfin,n,ni,nter,icor
      common Fn(50,2),F(0:50),P(0:50),cesal(0:50),fv(0:50),seal,l
      common dowel,sbdrain,sgdrain,spa(0:50),pa(0:50),tv(0:50)
      common ncorp,ncorf,ncorc,ncorr
      dimension mcost(15),TMC(0:50)
      character*5,maint
      character*4,sbdrain,sgdrain,dowel
      real maincost,mcost,h
      integer myr,mk,mn,icor
      read*,ctraffic,cls,cnl,ctime,cSIm
600 read 5,maint
      5 format(a5)
      read*,maincost,cunits,pfm,ffm,cfm,rfm,add
      mk=myr
      if (maint.eq.'under') then
        mcost(1)=maincost*cunits
        pesal(n-1)=0.0
        print'(t2,i2,t7,"Undersealing",t28,f7.0)',n,mcost(1)
      elseif (maint.eq.'fulld') then
        mcost(2)=maincost*cunits
        fv(n-1)=0.0
        pesal(n-1)=0.0
        nsp=0
        nf=0
        cesal(n-1)=0.0
        pa(n)=36*cunits/h
        print'(x,i2,t7,"Full depth repair",t28,f7.0)',n,mcost(2)
      elseif (maint.eq.'partd') then
        mcost(3)=maincost*cunits
        pa(n)=36*cunits/4
        fv(n-1)=0.0
        nsp=0
        nf=0

```

```

    print^(x,i2,t7,"Partial depth repair",t28,f7.0)^,n,mcost(3)
elseif (maint.eq.^slmod^) then
    mcost(4)=maincost*cunits
    nyr=nyr+10
    print^(x,i2,t7,"Low modulus ",t28,f7.0)^,n,mcost(4)
elseif (maint.eq.^sasph^) then
    mcost(5)=maincost*cunits
    nyr=nyr+2
    print^(x,i2,t7,"Asphalt ",t28,f7.0)^,n,mcost(5)
elseif (maint.eq.^subdr^) then
    mcost(6)=maincost*cunits
    if (eff.eq.1) go to 500
    if (sbdrain.eq.^poor^) then
        sbdrain=^fair^
    elseif (sbdrain.eq.^fair^) then
        sbdrain=^good^
    else
        sbdrain=^exel^
    endif
    icor=1
    ncorp=myr
    ncorf=myr
500 print^(x,i2,t7,"Subdrain install.",t28,f7.0)^,n,mcost(6)
elseif (maint.eq.^shash^) then
    mcost(7)=maincost*cunits
    print^(x,i2,t7,"AC shoulder repair",t28,f7.0)^,n,mcost(7)
elseif (maint.eq.^shpcc^) then
    mcost(8)=maincost*cunits
    h=h+1
    icor=1
    ncorp=myr
    ncorf=myr
    ncorc=myr
    ncorr=myr
    print^(x,i2,t7,"PCC shoulder",t28,f7.0)^,n,mcost(8)
elseif (maint.eq.^esupp^) then
    mcost(9)=maincost*cunits
    h=h+1
    icor=1
    ncorp=myr
    ncorf=myr
    ncorc=myr
    ncorr=myr
    print^(x,i2,t7,"Edge support",t28,f7.0)^,n,mcost(9)
elseif (maint.eq.^grind^) then
    mcost(10)=maincost*cunits
    fv(n-1)=0.0
    nf=0
    if (resal(n-1).ge.3.0) resal(n-1)=3.0
    print^(x,i2,t7,"Grinding",t28,f7.0)^,n,mcost(10)

```

```

elseif (maint.eq.'dowel') then
  mcost(11)=maincost*cunits
  dowel='dowl'
  icor=1
  ncorp=myr
  ncorf=myr
  print'(x,i2,t7,"Load transfer",t28,f7.0)',n,mcost(11)
elseif (maint.eq.'fl1dd') then
  mcost(12)=maincost*cunits
  fv(n-1)=0.0
  pesal(n-1)=0.0
  nsp=0
  cesal(n-1)=0.0
  pa(n)=36*cunits/h
  nf=0
  dowel='dowl'
  print'(x,i2,t7,"Full depth (dowel)",t28,f7.0)',n,mcost(12)
elseif (maint.eq.'prtdd') then
  mcost(13)=maincost*cunits
  fv(n-1)=0.0
  nsp=0
  pa(n)=36*cunits/4
  dowel='dowl'
  icor=1
  ncorp=myr
  print'(x,i2,t7,"Partial depth (dowel)",t28,f7.0)',n,mcost(13)
elseif (maint.eq.'overl') then
  mcost(14)=maincost*cunits
  npa=0
  if (resal(n-1).ge.2.0) then
    resal(n-1)=2.0
  endif
  fv(n-1)=1.29/32
  nf=0
  spa(n)=0
  h=h+add
  icor=1
  ncorp=myr
  ncorc=myr
  print'(x,i2,t7,"Overlay",t28,f7.0)',n,mcost(14)
else
  mcost(15)=maincost*cunits
  print'(x,i2,t7,"Other methods",t28,f7.0)',n,mcost(15)
endif
read*,myr
if (myr.eq.mk) go to 600
do 700 mn=1,15
700 TMC(n)=mcost(mn)+TMC(n)
   TMCOST(n)=TMC(n)+ctraffic
return
end

```



```

C *****
C ***** Subroutine to calculate users' cost during maintenance *****
C *****
C
      subroutine mainconsump
      common tesal(0:50),pi(0:50),npump(0:50),PU(0:50),TOTAL4(0:50)
      common cr(0:50),r(0:50),avgflt(0:50),nspall(0:50),ATOTAL4(0:20)
      common ps(0:50),resal(0:50),fSl,mk,ffm,pesal(0:50),PVTOTAL4(0:20)
      common FL(5,50),OIL(5,50),TC(5,50),psi(0:50),S(0:50),X(5,15)
      common YCOST(5,50),ZCOST(5,50),TCOST(5,50),ACOST(0:50),Y(5,5)
      common TOTAL1(0:50),TOTAL2(0:50),TOTAL3(0:50),Z(15),DA(0:50)
      common psiavg(0:50),Savg(0:50),vavg(0:50),v(0:50),nyr,vol,h,pfm
      common PVTOTAL1(0:20),ATOTAL1(0:20),pwf(0:50),crf(0:20),cfm,rfm
      common PVTOTAL2(0:20),ATOTAL2(0:20),PVTOTAL3(0:20),ATOTAL3(0:20)
      common dis(24),pt,et,nl,msf,Ds,Slm,vh(0:50),fw,Sp(0:50),nper
      common EXZCOST(5,50),EXTCOST(5,50),EXACOST(0:50),TMCOST(0:50)
      common CONST,TOTMAC(0:50),PVTOTMAC(0:50),ATOTMAC(0:50),nsp,add
      common nf,ctraffic,cls,cnl,ctime,cSlm,myr,nfin,n,ni,nter,icor
      common Fn(50,2),F(0:50),P(0:50),cesal(0:50),fv(0:50),seal,l
      common dowel,sbdrain,sgdrain,spa(0:50),pa(0:50),tv(0:50)
      common ncorp,ncorf,ncorc,ncorr
      dimension CZCOST(5,2),CTCOST(5,2),CYCOST(5,2),Fch(5,50),Tch(5,50)
      real sf,fhv,lane,chacc,rate,EX1,EX2,nl,Sch
      integer myr
      do 300 i=1,5
      do 400 mi=1,2
      if (mi.eq.1) then
        lane=nl
        limit=Slm
      else
        lane=cnl
        limit=cSlm
      endif
      fhv=1.0/(1-pt+et*pt)
      sf=msf*lane*fw*fhv
      SSp=0.0
      do 500 j=1,24
        vh(j)=v(n)*dis(j)
        vcrat=vh(j)/sf
        if (vcrat.gt.1.0) then
          Sd=30*(2-vcrat)**2
        else
          S1=0.58*Ds+20
          S4=(0.4*Ds-15)*vcrat
          S5=S1-30-S4
          S6=((vcrat)**(0.6*Ds-27.5))*S5

```

```

    Sd=S1-S4-S6
endif
S1=(fS1*limit)-(3.6*vcrat)
Sp(j)=amin1(Sd,S1)
SSp=SSp+(Sp(j)*vh(j))
500 continue
WSp=SSp/v(n)
Sr=21.4+0.04*limit*psi(n-1)+0.007*limit**2
S(mi)=amin1(WSp,Sr)
Sch=S(mi)-cS1m
psiavg(mi)=psi(n-1)
Savg(mi)=S(mi)
vavg(mi)=v(n)
So=X(i,1)
Fo=X(i,2)
A=Fo-((3*X(i,10))/(2*So))
C=X(i,10)/(2*So**3)
Flg=A+(X(i,10)/S(mi))+(C*Savg(mi)**2)
Fr=1+((X(i,3)*Savg(mi)**X(i,4))*((4.5-psiavg(mi))/3.0))
Fch(i,mi)=2*X(i,12)*(S(mi)*Sch**1.44)**X(i,13)
FL(i,mi)=Flg*Fr*X(i,11)
OIL(i,mi)=FL(i,mi)*(X(i,5)+X(i,6)*Savg(mi)+X(i,7)*Savg(mi)**2)
TCs=X(i,8)*Savg(mi)**X(i,9)
Frs=0.68+1.3*(4.5-psiavg(mi))
Tch(i,mi)=2*0.03022*Sch**1.27
TC(i,mi)=TCs*Frs
CYCOST(i,mi)=FL(i,mi)*Y(i,1)+OIL(i,mi)*Y(i,2)+TC(i,mi)*Y(i,3)
CZCOST(i,mi)=(CYCOST(i,mi)*vavg(mi)*(Z(i)/100.0)*cls)/1e+3
CTCOST(i,mi)=(1.0/Savg(mi))*vavg(mi)*Z(i)/100.0*cls*Y(i,4)
400 continue
EXZCOST(i,n)=(CZCOST(i,2)-CZCOST(i,1))*ctime+Tch(i,mi)+Fch(i,mi)
EXTCOST(i,n)=(CTCOST(i,2)-CTCOST(i,1))*ctime
300 continue
chacc=-15+63*((2*n1)/(n1+cn1))
rate=Z(7)*(1+chacc/100.0)
EXACOST(n)=(v(n)*ctime*rate*Z(8))/1.0e+6
EX1=EXZCOST(1,n)+EXZCOST(2,n)+EXZCOST(3,n)+EXZCOST(4,n)+
+EXZCOST(5,n)
EX2=EXTCOST(1,n)+EXTCOST(2,n)+EXTCOST(3,n)+EXTCOST(4,n)+
+EXTCOST(5,n)
print('t7,"Traffic control",t28,f7.0,t36,f10.0,t50,3(f8.0,2x))',
+ctraffic,TMCOST(n),EX1,EX2,EXACOST(n)
return
end

```

```

C
C *****
C ***** Subroutine to determine the weighted vehicle speed *****
C *****
C
      subroutine speed
      common tesal(0:50),pi(0:50),npump(0:50),PU(0:50),TOTAL4(0:50)
      common cr(0:50),r(0:50),avgflt(0:50),nspall(0:50),ATOTAL4(0:20)
      common ps(0:50),resal(0:50),fSl,mk,ffm,pesal(0:50),PVTOTAL4(0:20)
      common FL(5,50),OIL(5,50),TC(5,50),psi(0:50),S(0:50),X(5,15)
      common YCOST(5,50),ZCOST(5,50),TCOST(5,50),ACOST(0:50),Y(5,5)
      common TOTAL1(0:50),TOTAL2(0:50),TOTAL3(0:50),Z(15),DA(0:50)
      common psiavg(0:50),Savg(0:50),vavg(0:50),v(0:50),nyr,vol,h,pfm
      common PVTOTAL1(0:20),ATOTAL1(0:20),pwf(0:50),crf(0:20),cfm,rfm
      common PVTOTAL2(0:20),ATOTAL2(0:20),PVTOTAL3(0:20),ATOTAL3(0:20)
      common dis(24),pt,et,nl,msf,Ds,Slm,vh(0:50),fw,Sp(0:50),nper
      common EXZCOST(5,50),EXTCOST(5,50),EXACOST(0:50),TMCOST(0:50)
      common CONST,TOTMAC(0:50),PVTOTMAC(0:50),ATOTMAC(0:50),nsp,add
      common nf,ctraffic,cls,cnl,ctime,cSlm,myr,nfin,n,ni,nter,icor
      common Fn(50,2),F(0:50),P(0:50),cesal(0:50),fv(0:50),seal,l
      common dowel,sbdrain,sgdrain,spa(0:50),pa(0:50),tv(0:50)
      common ncorp,ncorf,ncorc,ncorr
      real sf,fhv,nl
      fhv=1.0/(1-pt+et*pt)
      sf=msf*nl*fw*fhv
      do 400 n=0,nfin
      SSp=0.0
      do 500 i=1,24
      vh(i)=v(n)*dis(i)
      vcrat=vh(i)/sf
      if (vcrat.gt.1.0) then
        Sd=30*(2-vcrat)**2
      else
        S1=0.58*Ds+20
        S4=(0.4*Ds-15)*vcrat
        S5=S1-30-S4
        S6=((vcrat)**(0.6*Ds-27.5))*S5
        Sd=S1-S4-S6
      endif
      Sl=(fSl*Slm)-(3.6*vcrat)
      Sp(i)=amin1(Sd,Sl)
      SSp=SSp+(Sp(i)*vh(i))
500  continue
      WSp=SSp/v(n)
      Sr=21.4+0.04*Slm*psi(n)+0.007*Slm**2
      S(n)=amin1(WSp,Sr)
400  continue
      return
      end

```

```

c
c *****
c *** Subroutine to determine the fuel, oil and tire consumptions *****
c *****
c
subroutine consump
common tesal(0:50),pi(0:50),npump(0:50),PU(0:50),TOTAL4(0:50)
common cr(0:50),r(0:50),avgflt(0:50),nspall(0:50),ATOTAL4(0:20)
common ps(0:50),resal(0:50),fsl,mk,ffm,pesal(0:50),PVTOTAL4(0:20)
common FL(5,50),OIL(5,50),TC(5,50),psi(0:50),S(0:50),X(5,15)
common YCOST(5,50),ZCOST(5,50),TCOST(5,50),ACOST(0:50),Y(5,5)
common TOTAL1(0:50),TOTAL2(0:50),TOTAL3(0:50),Z(15),DA(0:50)
common psiavg(0:50),Savg(0:50),vavg(0:50),v(0:50),nyr,vol,h,pfm
common PVTOTAL1(0:20),ATOTAL1(0:20),pwf(0:50),crf(0:20),cfm,rfm
common PVTOTAL2(0:20),ATOTAL2(0:20),PVTOTAL3(0:20),ATOTAL3(0:20)
common dis(24),pt,et,nl,msf,Ds,Slm,vh(0:50),fw,Sp(0:50),nper
common EXZCOST(5,50),EXTCOST(5,50),EXACOST(0:50),TMCOST(0:50)
common CONST,TOTMAC(0:50),PVTOTMAC(0:50),ATOTMAC(0:50),nsp,add
common nf,ctraffic,cls,cnl,ctime,cslm,myr,nfin,n,ni,nter,icor
common Fn(50,2),F(0:50),P(0:50),cesal(0:50),fv(0:50),seal,l
common dowel,sbdrain,sgdrain,spa(0:50),pa(0:50),tv(0:50)
common ncorp,ncorf,ncorc,ncorr
do 60 n=0,nfin
do 50 i=1,5
psiavg(n)=(psi(n)+psi(n-1))/2
Savg(n)=(S(n)+S(n-1))/2
vavg(n)=(v(n)+v(n-1))/2
So=X(i,1)
Fo=X(i,2)
A=Fo-((3*X(i,10))/(2*So))
C=X(i,10)/(2*So**3)
Flg=A+(X(i,10)/S(n))+(C*Savg(n)**2)
Fr=1+((X(i,3)*Savg(n)**X(i,4))*((4.5-psiavg(n))/3.0))
FL(i,n)=Flg*Fr*X(i,11)
OIL(i,n)=FL(i,n)*(X(i,5)+X(i,6)*Savg(n)+X(i,7)*Savg(n)**2)
TCs=X(i,8)*Savg(n)**X(i,9)
Frs=0.68+1.3*(4.5-psiavg(n))
TC(i,n)=TCs*Frs
YCOST(i,n)=FL(i,n)*Y(i,1)+OIL(i,n)*Y(i,2)+TC(i,n)*Y(i,3)
ZCOST(i,n)=(((YCOST(i,n)*vavg(n)*Z(i)/100.0*Z(6)*365)/1e+3)+
+(EXZCOST(i,n)))/1000.0
TCOST(i,n)=(((1.0/Savg(n))*vavg(n)*Z(i)/100.0*Z(6)*365*Y(i,4))
++(EXTCOST(i,n)))/1000.0
ACOST(n)=(((vavg(n)*365*Z(7)*Z(8))/1.0e+6)+(EXACOST(n)))/1000.0
50 continue
TOTAL1(n)=ZCOST(1,n)+ZCOST(2,n)+ZCOST(3,n)+ZCOST(4,n)+ZCOST(5,n)
TOTAL2(n)=TCOST(1,n)+TCOST(2,n)+TCOST(3,n)+TCOST(4,n)
++TCOST(5,n)
TOTAL3(n)=ACOST(n)
TOTAL4(n)=TOTAL1(n)+TOTAL2(n)+TOTAL3(n)
TOTMAC(n)=TMCOST(n)/1000.0
TOTMAC(0)=CONST/1000.0
60 continue
return
end

```

```

c
c *****
c ***** Subroutine to calculate PV and Annual costs *****
c *****
c
      subroutine pwvalue
      common tesal(0:50),pi(0:50),npump(0:50),PU(0:50),TOTAL4(0:50)
      common cr(0:50),r(0:50),avgflt(0:50),nspall(0:50),ATOTAL4(0:20)
      common ps(0:50),resal(0:50),fsl,mk,ffm,pesal(0:50),PVTOTAL4(0:20)
      common FL(5,50),OIL(5,50),TC(5,50),psi(0:50),S(0:50),X(5,15)
      common YCOST(5,50),ZCOST(5,50),TCOST(5,50),ACOST(0:50),Y(5,5)
      common TOTAL1(0:50),TOTAL2(0:50),TOTAL3(0:50),Z(15),DA(0:50)
      common psiavg(0:50),Savg(0:50),vavg(0:50),v(0:50),nyr,vol,h,pfm
      common PVTOTAL1(0:20),ATOTAL1(0:20),pwf(0:50),crf(0:20),cfm,rfm
      common PVTOTAL2(0:20),ATOTAL2(0:20),PVTOTAL3(0:20),ATOTAL3(0:20)
      common dis(24),pt,et,nl,msf,Ds,S1m,vh(0:50),fw,Sp(0:50),nper
      common EXZCOST(5,50),EXTCOST(5,50),EXACOST(0:50),TMCOST(0:50)
      common CONST,TOTMAC(0:50),PVTOTMAC(0:50),ATOTMAC(0:50),nsp,add
      common nf,ctraffic,cls,cnl,ctime,cslm,myr,nfin,n,ni,nter,icor
      common Fn(50,2),F(0:50),P(0:50),cesal(0:50),fv(0:50),seal,l
      common dowel,sbdrain,sgdrain,spa(0:50),pa(0:50),tv(0:50)
      common ncorp,ncorf,ncorc,ncorr
      integer j,n
      real fj,crf
      do 80 j=0,20
      do 70 n=0,nper
      fj=j/100.0
      pwf(n)=(1+fj)**n
      PVTOTAL1(j)=PVTOTAL1(j)+(TOTAL1(n)/pwf(n))
      PVTOTAL2(j)=PVTOTAL2(j)+(TOTAL2(n)/pwf(n))
      PVTOTAL3(j)=PVTOTAL3(j)+(TOTAL3(n)/pwf(n))
      PVTOTAL4(j)=PVTOTAL4(j)+(TOTAL4(n)/pwf(n))
      PVTOTMAC(j)=PVTOTMAC(j)+(TOTMAC(n)/pwf(n))
70 continue
      if(j.eq.0) then
      crf(0)=1.0/nper
      else
      crf(j)=(fj*((1+fj)**nper))/(((1+fj)**nper)-1)
      endif
      ATOTAL1(j)=PVTOTAL1(j)*crf(j)
      ATOTAL2(j)=PVTOTAL2(j)*crf(j)
      ATOTAL3(j)=PVTOTAL3(j)*crf(j)
      ATOTAL4(j)=PVTOTAL4(j)*crf(j)
      ATOTMAC(j)=PVTOTMAC(j)*crf(j)
80 continue
      return
      end

```



```

C
C *****
C ***** Subroutine to print pavement distresses *****
C *****
C
  subroutine prntdistr
    common tesal(0:50),pi(0:50),npump(0:50),PU(0:50),TOTAL4(0:50)
    common cr(0:50),r(0:50),avgflt(0:50),nspall(0:50),ATOTAL4(0:20)
    common ps(0:50),resal(0:50),fSl,mk,ffm,pesal(0:50),PVTOTAL4(0:20)
    common FL(5,50),OIL(5,50),TC(5,50),psi(0:50),S(0:50),X(5,15)
    common YCOST(5,50),ZCOST(5,50),TCOST(5,50),ACOST(0:50),Y(5,5)
    common TOTAL1(0:50),TOTAL2(0:50),TOTAL3(0:50),Z(15),DA(0:50)
    common psiavg(0:50),Savg(0:50),vavg(0:50),v(0:50),nyr,vol,h,pfm
    common PVTOTAL1(0:20),ATOTAL1(0:20),pwf(0:50),crf(0:20),cfm,rfm
    common PVTOTAL2(0:20),ATOTAL2(0:20),PVTOTAL3(0:20),ATOTAL3(0:20)
    common dis(24),pt,et,nl,msf,Ds,Slm,vh(0:50),fw,Sp(0:50),nper
    common EXZCOST(5,50),EXTCOST(5,50),EXACOST(0:50),TMCOST(0:50)
    common CONST,TOTMAC(0:50),PVTOTMAC(0:50),ATOTMAC(0:50),nsp,add
    common nf,ctraffic,cls,cnl,ctime,cSlm,myr,nfin,n,ni,nter,icor
    common Fn(50,2),F(0:50),P(0:50),cesal(0:50),fv(0:50),seal,1
    common dowel,sbdrain,sgdrain,spa(0:50),pa(0:50),tv(0:50)
    common ncorp,ncorf,ncorc,ncorr
    character*4,seal
    real nspall,npump
    integer n,nfin,ni,nter
    print`(/,"*****")`
    print`(/,t35,"TABLE III : PAVEMENT DISTRESSES")`
    print` (t35,"-----",/)`
    print` (lx,"YEAR",t7,"VOLUME",t16,"ESAL",t22,"SPALLS",t32,
+ "PUMPING",t48,
+ "FAULTING",t65,"CRACKING",t76,"SEALS",t82,"PATCHING",t92,"R",
+ t97,"PSI")`
    print` (t2,"END",t8,"vpd",t15,"e+6/yr",t22,"joints",t30,"vol",t35,
+ "joints",t43,
+ "avg",t48,"joints",t55,"joints",t63,"area",t69,"length",t85,
+ "sy",
+ t91,"in.")`
    print` (t31,"cf",t43,"in.",t48,"crit1",t55,"crit2",
+ t64,"sy",t71,"lf",t91,"/mi")`
    print` (t22,"(----- per section length -----
+ -----)")`
    print` ("-----")`
    do 100 n=0,nfin,ni
      if (nyr.ge.n) then
        seal='good'
      else
        seal='bad'
      endif
      print` (i3,f9.0,x,f6.3,x,f5.0,2x,f6.0,2xf4.0,x,f6.3,x,2f6.0,
+ 2x,f6.0,x,f5.0,3x,a4,2x,f6.0,lx
+ f4.0,x,f5.2)` ,n,v(n),tesal(n),nspall(n),PU(n),npump(n),F(n),
+ Fn(n,1),Fn(n,2),DA(n),cr(n),seal,spa(n),r(n),psi(n)
100 continue
    return
  end

```

```

C
C *****
C ***** Subroutine to print consumption rates *****
C *****
C
      subroutine prntcons
      common tesal(0:50),pi(0:50),npump(0:50),PU(0:50),TOTAL4(0:50)
      common cr(0:50),r(0:50),avgflt(0:50),nspall(0:50),ATOTAL4(0:20)
      common ps(0:50),resal(0:50),fSl,mk,ffm,pesal(0:50),PVTOTAL4(0:20)
      common FL(5,50),OIL(5,50),TC(5,50),psi(0:50),S(0:50),X(5,15)
      common YCOST(5,50),ZCOST(5,50),TCOST(5,50),ACOST(0:50),Y(5,5)
      common TOTAL1(0:50),TOTAL2(0:50),TOTAL3(0:50),Z(15),DA(0:50)
      common psiavg(0:50),Savg(0:50),vavg(0:50),v(0:50),nyr,vol,h,pfm
      common PVTOTAL1(0:20),ATOTAL1(0:20),pwf(0:50),crf(0:20),cfm,rfm
      common PVTOTAL2(0:20),ATOTAL2(0:20),PVTOTAL3(0:20),ATOTAL3(0:20)
      common dis(24),pt,et,nl,msf,Ds,Slm,vh(0:50),fw,Sp(0:50),nper
      common EXZCOST(5,50),EXTCOST(5,50),EXACOST(0:50),TMCOST(0:50)
      common CONST,TOTMAC(0:50),PVTOTMAC(0:50),ATOTMAC(0:50),nsp,add
      common nf,ctraffic,cls,cnl,ctime,cSlm,myr,nfin,n,ni,nter,icor
      common Fn(50,2),F(0:50),P(0:50),cesal(0:50),fv(0:50),seal,l
      common dowel,sbdrain,sgdrain,spa(0:50),pa(0:50),tv(0:50)
      common ncorp,ncorf,ncorc,ncorr
      print`(/,"*****")`
      print`(/,t30,"TABLE IV : VEHICLE CONSUMPTION RATES",t67,
+ "(per 1000 veh. miles)",/)`
      print` (t30,"-----")`
+----)`
      print` ("DURING",t8,"AVG.",t15,"AVG.",t21,"AVG.",t31,"PASS. CARS",
+ t49,"PICK-UPS",
+ t66,"S-U TRUCKS",
+ t83,"COMB. TRUCKS",t100,"DIESEL TRUCKS")`
      print` (t2,"YEAR",t7,"VOLUME",t14,"SPEED",t21,"PSI",
+ t28,"Fuel Oil Tire",
+ t46,"Fuel Oil Tire",t63,"Fuel Oil Tire",
+ t81,"Fuel Oil Tire",t99,"Fuel Oil Tire")`
      print` (t8,"vpd",t15,"mph",t28," gal qrt no.",t46,
+ " gal qrt no.",t63," gal qrt no.",t81,
+ " gal qrt no.",t99," gal qrt no.")`
      print` ("-----")`
+-----)`
      do 200 n=0,nfin,ni
      print` (i3,x,f8.0,2x,f4.1,2x,f4.2,5(x,f6.1,x,f4.1,2x,f4.2))`
+ ,n,vavg(n),Savg(n),psiavg(n),FL(1,n),OIL(1,n),TC(1,n),FL(2,n),
+ OIL(2,n),TC(2,n),FL(3,n),OIL(3,n),TC(3,n),FL(4,n),OIL
+ (4,n),TC(4,n),FL(5,n),OIL(5,n),TC(5,n)
200 continue
      return
      end

```

```

C
C *****
C ***** Subroutine to print users' costs *****
C *****
C
      subroutine prntuser
      common tesal(0:50),pi(0:50),npump(0:50),PU(0:50),TOTAL4(0:50)
      common cr(0:50),r(0:50),avgflt(0:50),nspall(0:50),ATOTAL4(0:20)
      common ps(0:50),resal(0:50),fsl,mk,ffm,pesal(0:50),PVTOTAL4(0:20)
      common FL(5,50),OIL(5,50),TC(5,50),psi(0:50),S(0:50),X(5,15)
      common YCOST(5,50),ZCOST(5,50),TCOST(5,50),ACOST(0:50),Y(5,5)
      common TOTAL1(0:50),TOTAL2(0:50),TOTAL3(0:50),Z(15),DA(0:50)
      common psiavg(0:50),Savg(0:50),vavg(0:50),v(0:50),nyr,vol,h,pfm
      common PVTOTAL1(0:20),ATOTAL1(0:20),pwf(0:50),crf(0:20),cfm,rfm
      common PVTOTAL2(0:20),ATOTAL2(0:20),PVTOTAL3(0:20),ATOTAL3(0:20)
      common dis(24),pt,et,nl,msf,Ds,Slm,vh(0:50),fw,Sp(0:50),nper
      common EXZCOST(5,50),EXTCOST(5,50),EXACOST(0:50),TMCOST(0:50)
      common CONST,TOTMAC(0:50),PVTOTMAC(0:50),ATOTMAC(0:50),nsp,add
      common nf,ctraffic,cls,cnl,ctime,cslm,myr,nfin,n,ni,nter,icor
      common Fn(50,2),F(0:50),P(0:50),cesal(0:50),fv(0:50),seal,l
      common dowel,sbdrain,sgdrain,spa(0:50),pa(0:50),tv(0:50)
      common ncorp,ncorf,ncorc,ncorr
      print`(/,"*****")`
      print`(/,t37,"TABLE V : ROAD USERS COSTS ($1000)")`
      print`(t37,"-----",/)`
      print`("DURING",t12,"PASS. CARS",t31,"PICK-UPS",t48,
+"S-U TRUCKS",t65,"COMB. TRUCKS",t82,"DIESEL TRUCKS",t101,
+"ALL")`
      print`(t2,"YEAR",5(4x,"Vehicle",3x,"Time"),5x,"Acc.")`
      print`("-----",
+t51,"-----")`
      do 300 n=0,nfin,ni
      print`(i3,3x,11(f8.0,x))`n,ZCOST(1,n),TCOST(1,n),ZCOST(2,n),
+TCOST(2,n),ZCOST(3,n),TCOST(3,n),ZCOST(4,n),TCOST(4,n),
+ZCOST(5,n),TCOST(5,n),ACOST(n)
300 continue
      return
      end

```



```

C
C *****
C ***** Subroutine to print annual costs *****
C *****
C
      subroutine prntcosts
      common tesal(0:50),pi(0:50),npump(0:50),PU(0:50),TOTAL4(0:50)
      common cr(0:50),r(0:50),avgflt(0:50),nspall(0:50),ATOTAL4(0:20)
      common ps(0:50),resal(0:50),fS1,mk,ffm,pesal(0:50),PVTOTAL4(0:20)
      common FL(5,50),OIL(5,50),TC(5,50),psi(0:50),S(0:50),X(5,15)
      common YCOST(5,50),ZCOST(5,50),TCOST(5,50),ACOST(0:50),Y(5,5)
      common TOTAL1(0:50),TOTAL2(0:50),TOTAL3(0:50),Z(15),DA(0:50)
      common psiavg(0:50),Savg(0:50),vavg(0:50),v(0:50),nyr,vol,h,pfm
      common PVTOTAL1(0:20),ATOTAL1(0:20),pwf(0:50),crf(0:20),cfm,rfm
      common PVTOTAL2(0:20),ATOTAL2(0:20),PVTOTAL3(0:20),ATOTAL3(0:20)
      common dis(24),pt,et,nl,msf,Ds,Slm,vh(0:50),fw,Sp(0:50),nper
      common EXZCOST(5,50),EXTCOST(5,50),EXACOST(0:50),TMCOST(0:50)
      common CONST,TOTMAC(0:50),PVTOTMAC(0:50),ATOTMAC(0:50),nsp,add
      common nf,ctraffic,cls,cnl,ctime,cSlm,myr,nfin,n,ni,nter,icor
      common Fn(50,2),F(0:50),P(0:50),cesal(0:50),fv(0:50),seal,l
      common dowel,sbdrain,sgdrain,spa(0:50),pa(0:50),tv(0:50)
      common ncorp,ncorf,ncorc,ncorr
      print`(/,"*****")`
      print`(/,t21,"TABLE VI : ANNUAL COSTS ($1000)")`
      print` (t21,"-----",/)`
      print` (t2,"YEAR",t9,"VEH. OPER.",t23,"TIME",t34,"ACCIDENT",t45,
+ "ALL USER",t58,"CONST & MAINT")`
      print` ("-----",
+ t52,"-----")`
      do 110 n=0,nfin,ni
      print` (2x,i2,4(f10.0,2x),3x,f10.0)` ,n,TOTAL1(n),TOTAL2(n),
+ TOTAL3(n),TOTAL4(n),TOTMAC(n)
110 continue
      return
      end

```

```

C *****
C ***** Subroutine to print PV and Annual costs *****
C *****
C
  subroutine prntpv
  common tesal(0:50),pi(0:50),npump(0:50),PU(0:50),TOTAL4(0:50)
  common cr(0:50),r(0:50),avgflt(0:50),nspall(0:50),ATOTAL4(0:20)
  common ps(0:50),resal(0:50),fsl,mk,ffm,pesal(0:50),PVTOTAL4(0:20)
  common FL(5,50),OIL(5,50),TC(5,50),psi(0:50),S(0:50),X(5,15)
  common YCOST(5,50),ZCOST(5,50),TCOST(5,50),ACOST(0:50),Y(5,5)
  common TOTAL1(0:50),TOTAL2(0:50),TOTAL3(0:50),Z(15),DA(0:50)
  common psiavg(0:50),Savg(0:50),vavg(0:50),v(0:50),nyr,vol,h,pfm
  common PVTOTAL1(0:20),ATOTAL1(0:20),pwf(0:50),crf(0:20),cfm,rfm
  common PVTOTAL2(0:20),ATOTAL2(0:20),PVTOTAL3(0:20),ATOTAL3(0:20)
  common dis(24),pt,et,nl,msf,Ds,Slm,vh(0:50),fw,Sp(0:50),nper
  common EXZCOST(5,50),EXTCOST(5,50),EXACOST(0:50),TMCOST(0:50)
  common CONST,TOTMAC(0:50),PVTOTMAC(0:50),ATOTMAC(0:50),nsp,add
  common nf,ctraffic,cls,cnl,ctime,cslm,myr,nfin,n,ni,nter,icor
  common Fn(50,2),F(0:50),P(0:50),cesal(0:50),fv(0:50),seal,l
  common dowel,sbdrain,sgdrain,spa(0:50),pa(0:50),tv(0:50)
  common ncorp,ncorf,ncorc,ncorr
  print`(/,"*****")`
  print`(/,t23,"TABLE VII : PRESENT VALUES AND ANNUAL COSTS ($1000)"
+)`
  print` (t23,"-----",t59,
+"-----",/)`
  print` ("INTEREST",t11,"VEHICLE OPERATING",t38,"TIME",
+t56,"ACCIDENT",t72,"ALL USER COSTS",t90,"CONSTR. & MAINT.")`
  print` (t2,"RATE(%)",5(5x,"PV",6x,"ANNUAL"))`
  print` ("-----",
+t52,"-----")`
  do 500 j=0,20,2
    print` (3x,i2,4x,3(f8.0,x,f8.0,2x),f8.0,x,f8.0,2x,f8.0,x,f8.0)` ,
    +j,PVTOTAL1(j),ATOTAL1(j),
    +PVTOTAL2(j),ATOTAL2(j),PVTOTAL3(j),ATOTAL3(j),PVTOTAL4(j),
    +ATOTAL4(j),PVTOTMAC(j),ATOTMAC(j)
500 continue
  print` (/,"Analysis period = ",i3,/)` ,nper
  return
  end

```

2. PROGRAM INPUT

ALTERNATIVE: EXAMPLE

10 20 3000000. 650 150 6

good1 poor unst lmod ndwl plain wmwet

0.25 0.09 0.05 2.0

15000 0.42 0.9 4 60 55 2 0.99 2 2000 1.1

017013009009008013025042051053054055056060066046083074065056047038033027

70 6 9 10 5 1.0 0.915 7300

1.30 1.30 65 9.0

1.30 1.30 65 5.75

1.12 0.90 220 13.45

1.12 0.90 220 15.35

1.12 0.90 220 15.35

20 30 1

yes

440000

19

10000 1 1 10 45

under

10 644 1 1 1 1 0

19

grind

4.5 7040 1 1 1 1 0

19

partd

200 5.0 1 1 1 1 0

26

10000 1 1 10 45

under

10 937 1 1 1 1 0

26

grind

4.5 4208 1 1 1 1 0

26

partd

200 0.5 1 1 1 1 0

35

 * ALTERNATIVE: EXAMPLE *

TABLE I : INITIAL PAVEMENT AND TRAFFIC INFORMATION

PCC SLAB : Length (ft) = 20. ; Thickness (in.) = 10.0
 Mr (psi) = 650.0 ; E (psi) = 3000000.0
 Dowels: no ; Reinforcement: no

SUBBASE : Type: unst ; Thickness (in.) = 6.0

SUBDRAINAGE : good1

SUBGRADE : k (pci) =150.0 ; Drainage: poor

TRAFFIC : Volume (vpd) = 15000. ; Growth rate (%) = 4.
 Pass. cars (%) =70. ; Pick-ups (%) = 6.
 S-U trucks (%) = 9. ; Comb. Trucks (%) =10.
 Diesel trucks (%) = 5.
 Truck factor =0.420
 Proportion of trucks in design lane =0.90

GEOMETRY : Design speed (mph) = 60.0 ; Speed limit (mph) = 55.0
 No. lanes = 2.0 ; Length (mi) = 1.0

USERS COSTS:

	Gas(\$/gal)	Oil(\$/qrt)	Tires(\$/tire)	Time(\$/hr)
Pass. cars:	1.30	1.30	65.0	9.0
Pick-ups :	1.30	1.30	65.0	5.8
S-U trucks :	1.12	0.90	220.0	13.4
Comb.trucks:	1.12	0.90	220.0	15.4
Diesel :	1.12	0.90	220.0	15.4

CLIMATE: warm, wet

CRITICAL FAULT1 (in.) = 0.250
 FAULT2 (in.) = 0.090
 S.D. (in.) = 0.050

TABLE II : CONSTRUCTION AND MAINTENANCE COSTS (\$)

YEAR	ACTIVITY	CONSTRUCTION AND MAINTENANCE		ADDITIONAL USERS COSTS		
		COSTS	TOTAL	VEH. OPER.	TIME	ACCIDENT
0	Construction		440000.			
19	Undersealing	6440.				
19	Grinding	31680.				
19	Partial depth repair	1000.				
	Traffic control	10000.	49120.	-7824.	13323.	3567.
26	Undersealing	9370.				
26	Grinding	18936.				
26	Partial depth repair	100.				
	Traffic control	10000.	38406.	-8837.	17030.	4694.

TABLE III : PAVEMENT DISTRESSES

YEAR END	VOLUME vpd	ESAL e+6/yr	SPALLS joints	PUMPING vol cf	FAULTING joints crit1	FAULTING joints crit2	CRACKING area sy	CRACKING length if	SEALS PATCHING sy	R in. /mi	PSI
0	15000.	0.	0.	0.	0.	0.040	0.	42.	0.	0.	36.
1	15600.	0.527	0.	47.	16.	0.052	0.	61.	918.	595.	0.
2	16224.	1.075	0.	84.	28.	0.064	0.	79.	1090.	706.	0.
3	16873.	1.645	2.	115.	38.	0.075	0.	101.	1177.	763.	0.
4	17548.	2.237	4.	144.	48.	0.085	0.	123.	1235.	800.	0.
5	18250.	2.854	7.	171.	57.	0.096	0.	145.	1277.	828.	0.
6	18980.	3.495	13.	197.	66.	0.107	0.	167.	1310.	849.	0.
7	19739.	4.162	20.	222.	74.	0.118	0.	188.	1338.	867.	0.
8	20529.	4.855	29.	246.	82.	0.130	1.	206.	1361.	882.	0.
9	21350.	5.576	41.	270.	90.	0.141	4.	220.	1381.	895.	0.
10	22204.	6.326	55.	293.	98.	0.152	7.	231.	1399.	906.	0.
11	23092.	7.106	71.	316.	105.	0.164	13.	243.	1415.	917.	0.
12	24015.	7.917	89.	338.	113.	0.176	18.	251.	1429.	926.	0.
13	24976.	8.761	108.	360.	120.	0.188	29.	257.	1442.	935.	0.
14	25975.	9.638	128.	382.	127.	0.200	41.	260.	1454.	942.	0.
15	27014.	10.550	147.	404.	135.	0.212	61.	263.	1466.	950.	0.
16	28095.	11.499	166.	425.	142.	0.224	81.	264.	1476.	956.	0.
17	29218.	12.486	184.	447.	149.	0.237	106.	264.	1486.	963.	0.
18	30387.	13.512	201.	468.	156.	0.250	132.	264.	1495.	969.	0.
19	31603.	14.580	0.	75.	25.	0.045	0.	49.	1504.	975.	45.
20	32867.	15.690	0.	124.	41.	0.051	0.	60.	1512.	980.	45.
21	34181.	16.844	2.	166.	55.	0.060	0.	73.	1520.	985.	45.
22	35549.	18.045	4.	205.	68.	0.069	0.	89.	1528.	990.	45.
23	36971.	19.294	7.	242.	81.	0.079	0.	109.	1535.	995.	45.
24	38450.	20.592	13.	277.	92.	0.089	0.	130.	1542.	999.	45.
25	39988.	21.943	20.	310.	103.	0.100	0.	153.	1549.	1004.	45.
26	41587.	23.347	0.	85.	28.	0.045	0.	49.	1555.	1008.	50.
27	43250.	24.808	0.	140.	47.	0.051	0.	60.	1562.	1012.	50.
28	44981.	26.327	2.	188.	63.	0.059	0.	73.	1568.	1016.	50.
29	46780.	27.907	4.	231.	77.	0.069	0.	89.	1574.	1020.	50.
30	48651.	29.551	7.	272.	91.	0.079	0.	109.	1579.	1023.	50.

TABLE IV : VEHICLE CONSUMPTION RATES (per 1000 veh. miles)

DURING YEAR	AVG. VOLUME	AVG. SPEED mph	AVG. PSI	PASS. CARS			PICK-UPS			S-U TRUCKS			COMB. TRUCKS			DIESEL TRUCKS		
				Fuel gal	Oil qrt	Tire no.	Fuel gal	Oil qrt	Tire no.	Fuel gal	Oil qrt	Tire no.	Fuel gal	Oil qrt	Tire no.	Fuel gal	Oil qrt	Tire no.
0	7500.	26.2	2.25	36.2	2.2	0.20	38.6	2.0	0.16	53.6	2.0	0.17	158.3	4.6	0.20	103.1	10.1	0.27
1	15300.	52.3	4.42	56.4	2.8	0.11	68.3	2.6	0.08	134.4	3.2	0.09	170.6	4.2	0.11	110.9	4.3	0.14
2	15912.	51.9	4.25	57.9	2.9	0.14	68.3	2.6	0.10	134.1	3.2	0.11	171.8	4.3	0.13	111.7	4.5	0.17
3	16548.	51.6	4.09	59.3	3.0	0.16	68.2	2.6	0.13	133.7	3.2	0.13	172.9	4.3	0.16	112.4	4.7	0.21
4	17210.	51.3	3.95	60.4	3.1	0.19	68.2	2.6	0.14	133.4	3.3	0.15	173.8	4.4	0.18	113.1	4.8	0.24
5	17899.	51.0	3.81	61.5	3.1	0.21	68.1	2.6	0.16	133.1	3.3	0.17	174.7	4.5	0.20	113.7	5.0	0.27
6	18615.	50.7	3.69	62.4	3.2	0.23	68.0	2.6	0.18	132.7	3.3	0.18	175.5	4.5	0.22	114.2	5.2	0.29
7	19359.	50.4	3.57	63.2	3.2	0.25	67.9	2.6	0.19	132.4	3.3	0.20	176.3	4.5	0.24	114.7	5.3	0.32
8	20134.	50.2	3.46	63.9	3.3	0.27	67.8	2.7	0.20	132.1	3.3	0.21	177.0	4.6	0.26	115.2	5.4	0.34
9	20939.	49.9	3.35	64.6	3.3	0.28	67.7	2.7	0.22	131.8	3.3	0.22	177.6	4.6	0.27	115.7	5.6	0.36
10	21777.	49.7	3.25	65.2	3.4	0.30	67.6	2.7	0.23	131.5	3.3	0.24	178.3	4.7	0.29	116.1	5.7	0.38
11	22648.	49.5	3.15	65.8	3.4	0.31	67.5	2.7	0.24	131.2	3.3	0.25	178.9	4.7	0.30	116.5	5.8	0.40
12	23554.	49.3	3.06	66.4	3.4	0.33	67.3	2.7	0.25	130.9	3.3	0.26	179.4	4.7	0.32	116.9	5.9	0.41
13	24496.	49.1	2.97	66.9	3.5	0.34	67.2	2.7	0.26	130.6	3.3	0.27	180.0	4.8	0.33	117.3	6.0	0.43
14	25476.	48.9	2.88	67.3	3.5	0.35	67.1	2.7	0.27	130.3	3.3	0.28	180.5	4.8	0.34	117.6	6.1	0.45
15	26495.	48.7	2.79	67.7	3.5	0.37	66.9	2.7	0.28	130.0	3.3	0.29	181.0	4.8	0.36	118.0	6.3	0.46
16	27554.	48.5	2.71	68.2	3.5	0.38	66.8	2.7	0.29	129.7	3.3	0.30	181.5	4.9	0.37	118.3	6.4	0.48
17	28657.	48.4	2.63	68.5	3.6	0.39	66.7	2.7	0.30	129.5	3.3	0.31	181.9	4.9	0.38	118.6	6.4	0.49
18	29803.	48.2	2.55	68.9	3.6	0.40	66.5	2.7	0.31	129.2	3.3	0.32	182.4	4.9	0.39	118.9	6.5	0.51
19	30995.	49.9	3.31	64.3	3.3	0.29	67.3	2.6	0.22	131.2	3.3	0.23	177.2	4.6	0.28	115.4	5.6	0.37
20	32235.	51.5	4.04	59.7	3.0	0.17	68.2	2.6	0.13	133.6	3.2	0.14	173.2	4.4	0.17	112.7	4.7	0.22
21	33524.	51.1	3.90	60.8	3.1	0.20	68.2	2.6	0.15	133.3	3.3	0.15	174.1	4.4	0.19	113.3	4.9	0.25
22	34865.	50.8	3.75	62.0	3.2	0.22	68.1	2.6	0.17	132.9	3.3	0.17	175.1	4.5	0.21	114.0	5.1	0.28
23	36260.	50.5	3.60	63.0	3.2	0.25	68.0	2.6	0.19	132.5	3.3	0.19	176.1	4.5	0.24	114.6	5.3	0.31
24	37710.	50.2	3.45	64.0	3.3	0.27	67.8	2.7	0.20	132.1	3.3	0.21	177.0	4.6	0.26	115.3	5.5	0.34
25	39219.	49.9	3.31	64.9	3.3	0.29	67.7	2.7	0.22	131.7	3.3	0.23	177.9	4.6	0.28	115.9	5.6	0.37
26	40787.	50.6	3.66	62.3	3.2	0.24	67.8	2.6	0.18	132.4	3.3	0.19	175.3	4.5	0.23	114.1	5.2	0.30
27	42419.	51.4	4.00	60.1	3.0	0.18	68.2	2.6	0.14	133.5	3.2	0.14	173.5	4.4	0.17	112.9	4.8	0.23
28	44115.	51.0	3.82	61.4	3.1	0.21	68.1	2.6	0.16	133.1	3.3	0.16	174.6	4.4	0.20	113.6	5.0	0.26
29	45880.	50.6	3.65	62.7	3.2	0.24	68.0	2.6	0.18	132.6	3.3	0.19	175.8	4.5	0.23	114.4	5.2	0.30
30	47715.	50.2	3.48	63.8	3.3	0.26	67.8	2.7	0.20	132.2	3.3	0.21	176.9	4.6	0.26	115.1	5.4	0.33

TABLE V : ROAD USERS COSTS (\$1000)

DURING YEAR	PASS. CARS		PICK-UPS		S-U TRUCKS		COMB. TRUCKS		DIESEL TRUCKS		ALL Acc.
	Vehicle	Time	Vehicle	Time	Vehicle	Time	Vehicle	Time	Vehicle	Time	
0	121.	657.	10.	36.	24.	126.	62.	160.	25.	80.	18.
1	329.	673.	33.	37.	87.	129.	122.	164.	44.	82.	37.
2	358.	705.	34.	39.	92.	135.	131.	172.	49.	86.	39.
3	388.	738.	36.	40.	98.	142.	140.	180.	53.	90.	40.
4	417.	772.	38.	42.	105.	148.	150.	188.	58.	94.	42.
5	447.	808.	40.	44.	111.	155.	160.	197.	62.	98.	44.
6	477.	844.	42.	46.	117.	162.	170.	206.	67.	103.	45.
7	507.	883.	44.	48.	124.	170.	180.	215.	72.	108.	47.
8	539.	923.	46.	51.	130.	177.	190.	225.	76.	112.	49.
9	571.	964.	48.	53.	137.	185.	201.	235.	82.	117.	51.
10	604.	1007.	51.	55.	145.	193.	213.	245.	87.	123.	53.
11	639.	1052.	53.	58.	152.	202.	224.	256.	92.	128.	55.
12	674.	1099.	55.	60.	160.	211.	237.	268.	98.	134.	57.
13	711.	1147.	58.	63.	168.	220.	249.	280.	104.	140.	60.
14	749.	1198.	60.	66.	176.	230.	262.	292.	110.	146.	62.
15	788.	1251.	63.	68.	185.	240.	276.	305.	116.	152.	65.
16	829.	1305.	66.	71.	194.	251.	290.	318.	122.	159.	67.
17	872.	1363.	69.	75.	203.	262.	305.	332.	129.	166.	70.
18	916.	1422.	72.	78.	213.	273.	320.	347.	136.	173.	73.
19	837.	1438.	71.	79.	201.	276.	297.	350.	120.	175.	79.
20	764.	1440.	71.	79.	193.	277.	276.	351.	105.	175.	79.
21	822.	1507.	75.	83.	205.	290.	295.	367.	114.	184.	82.
22	882.	1578.	79.	86.	218.	303.	315.	384.	123.	192.	85.
23	945.	1651.	83.	90.	231.	317.	336.	402.	133.	201.	88.
24	1011.	1729.	87.	95.	245.	332.	357.	421.	144.	211.	92.
25	1078.	1809.	91.	99.	259.	348.	380.	441.	154.	220.	96.
26	1033.	1863.	91.	102.	252.	358.	368.	454.	145.	227.	104.
27	1016.	1899.	94.	104.	256.	365.	367.	463.	140.	231.	103.
28	1098.	1990.	99.	109.	273.	382.	393.	485.	153.	242.	108.
29	1184.	2085.	104.	114.	290.	401.	421.	508.	166.	254.	112.
30	1273.	2185.	109.	120.	308.	420.	450.	532.	180.	266.	116.

TABLE VI : ANNUAL COSTS (\$1000)

YEAR	VEH. OPER.	TIME	ACCIDENT	ALL USER	CONST & MAINT
0	243.	1060.	18.	1321.	440.
1	614.	1085.	37.	1736.	0.
2	664.	1136.	39.	1839.	0.
3	716.	1190.	40.	1946.	0.
4	767.	1245.	42.	2054.	0.
5	819.	1302.	44.	2165.	0.
6	872.	1362.	45.	2279.	0.
7	927.	1423.	47.	2397.	0.
8	982.	1488.	49.	2519.	0.
9	1040.	1554.	51.	2645.	0.
10	1099.	1624.	53.	2776.	0.
11	1160.	1696.	55.	2912.	0.
12	1224.	1771.	57.	3052.	0.
13	1289.	1850.	60.	3199.	0.
14	1357.	1931.	62.	3351.	0.
15	1428.	2016.	65.	3509.	0.
16	1501.	2105.	67.	3673.	0.
17	1577.	2197.	70.	3844.	0.
18	1656.	2293.	73.	4022.	0.
19	1533.	2318.	79.	3931.	49.
20	1410.	2322.	79.	3810.	0.
21	1510.	2430.	82.	4022.	0.
22	1616.	2544.	85.	4245.	0.
23	1728.	2663.	88.	4479.	0.
24	1843.	2787.	92.	4722.	0.
25	1962.	2917.	96.	4974.	0.
26	1908.	3004.	104.	5016.	38.
27	1873.	3062.	103.	5038.	0.
28	2015.	3208.	108.	5331.	0.
29	2165.	3362.	112.	5639.	0.
30	2321.	3522.	116.	5959.	0.

TABLE VII : PRESENT VALUES AND ANNUAL COSTS (\$1000)

INTEREST RATE(%)	VEHICLE OPERATING		TIME		ACCIDENT		ALL USER COSTS		CONSTR. & MAINT.	
	PV	ANNUAL	PV	ANNUAL	PV	ANNUAL	PV	ANNUAL	PV	ANNUAL
0	2280.	1144.	34968.	1748.	1133.	57.	58981.	2949.	489.	24.
2	18165.	1111.	28031.	1714.	906.	55.	47102.	2881.	474.	29.
4	14666.	1079.	22877.	1683.	737.	54.	38280.	2817.	463.	34.
6	12034.	1049.	18994.	1656.	610.	53.	31638.	2758.	456.	40.
8	10028.	1021.	16027.	1632.	513.	52.	26568.	2706.	451.	46.
10	8478.	996.	13731.	1613.	438.	51.	22647.	2660.	448.	53.
12	7265.	973.	11930.	1597.	379.	51.	19574.	2621.	446.	60.
14	6305.	952.	10499.	1585.	332.	50.	17136.	2587.	444.	67.
16	5535.	934.	9348.	1577.	294.	50.	15214.	2566.	443.	75.
18	4911.	918.	8411.	1571.	264.	49.	13640.	2548.	442.	83.
20	4399.	903.	7641.	1569.	238.	49.	12381.	2543.	442.	91.

Analysis period = 20

Appendix E
Information Used in Examples

Table E.1 Construction Costs Used in the Examples

Note:

All costs per mile for two lanes in one direction

Lanes are 12 ft wide.

Shoulders are 6 ft wide (on each side)

Impermeable subbases extend 1 ft into the shoulder (on each side)

Permeable subbases extend 3 ft into the shoulder (on one side)

Edge drains provided on one side only

Unit costs obtained from Table 13.1

PCC Slab : (without reinforcement or dowels)

8 in. slab = \$204000

10 in. slab = \$254000

12 in. slab = \$305000

Subbase :

6 in. dense-graded = \$46000

8 in. dense-graded = \$61000

4 in. open-graded = \$39000

4 in. dense-graded filter = \$30500

4 in. lean concrete = \$85000

6 in. lean concrete = \$127000

4 in. Portland cement stab. = \$59000

6 in. Portland cement stab. = \$89000

Shoulder :

8 in. asphalt cement = \$78000

10 in. asphalt cement = \$98000

12 in. asphalt cement = \$117000

Edge drains :

\$42000

Table E.2 Rehabilitation Techniques and Costs Used
in the Examples

Grinding:

Purpose: Correct faulting
Joints with faults > 0.09 in.
Area = slab length/2 * design lane width
Cost = \$4.50 per sy
Production rate = 3000 sy per day

Undersealing:

Purpose: Fill all voids
Cost = \$10 per cf
Production rate = 250 cf per day

Partial depth patching (no dowels):

Purpose: correct (replace) spalling
Average spall per joint = 2 sf
Average depth per joint = 4 in.
Volume = $(2/9) * (4/36) * \text{spalled joints (in cy)}$
Cost = \$200 per cy
Production = 10 cy per day

Full depth patching (no dowels):

Purpose: correct (replace) damaged area
Volume = damaged area * slab thickness/36 (in cy)
Cost = \$270 per cy
Production rate = 20 cy per day

Retrofit drains:

Purpose: improve drainage
Cost = \$8 per lf
Production rate = 2500 lf per day

Traffic control:

Cost = \$10000 if rehabilitation lasts less than 10 days
= \$12000 if rehabilitation lasts more than 10 days

Note: Only the distresses in the design lane to be corrected

1 in. = 25.4 mm

1 cy = 0.7646 cu m

Table E.3 Example 2: Pavement Distress Levels

	Values after 25 years					
	1	2	3	4	5	6
BASE	987	0.55	264	259	1549	1.58
ALTER1	685	0.24	264	89	1549	2.46
ALTER2	878	0.44	264	259	1549	1.82
ALTER3	428	0.16	241	89	1549	2.76
ALTER4	685	0.24	264	259	1492	1.91
GOOD	616	0.35	246	259	1549	2.05

Column

- 1 pumped volume (cf)
- 2 average fault (in.)
- 3 number of joints with faults > 0.10 in.
- 4 number of spalled joints
- 5 damaged area (sy)
- 6 PSI

BASE : no rehabilitation (without egde drains)

ALTER1 : Grinding, undersealing, and partial depth patching

ALTER2 : Installation of retrofit drains

ALTER3 : Grinding, undersealing, partial depth patching,
and the installation of retrofit drains

ALTER4 : Full depth patching

GOOD : no rehabilitation (with edge drains)

Rehabilitation applied after 13 years