U.S. Department
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**Federal Highway
Administration**

A Program of Study in Pavement Management

Volume I

Participant Notebook
Lessons 1–31

1

A PROGRAM OF STUDY
IN
PAVEMENT MANAGEMENT

Volume I

Participant Notebook
Lessons 1–31

THE CENTER FOR TRANSPORTATION RESEARCH
BUREAU OF ENGINEERING RESEARCH
THE UNIVERSITY OF TEXAS AT AUSTIN

in Cooperation with the

U. S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
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1984

PREFACE

Our road and street network represents a major area of investment in transportation. The pavement portion of this investment is, in turn, quite substantial. People who are intrusted with the responsibility of expending the funds allocated for these investments require an efficient set of management practices.

The term pavement management has become popular in recent years. In a broad sense, it includes the entire spectrum of interrelated activities that are involved in providing pavements. These range from the planning or programming of investments through to design, construction, maintenance and in-service evaluation.

Any type of management is concerned with information, coordination of activities, making decisions and taking action. This is of course not an easy task, especially in a large and complex area such as pavement management. In addition, few individuals have the opportunity or the responsibility to work in all the activities involved in pavement management. Nevertheless, it is desirable for all people involved in pavement management, no matter what their level of administrative or technical responsibility, to have at least an appreciation for these activities. In this way, their own more in-depth knowledge associated with day to day working activities can contribute more effectively to the overall goal of pavement management --- that is, to achieve the best possible value for available public funds.

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A PROGRAM OF STUDY IN PAVEMENT MANAGEMENT
VOLUME I

TABLE OF CONTENTS

I. BACKGROUND AND INTRODUCTION

II. PAVEMENT DESIGN CONCEPTS

Revised WRH/lg 12/8/83

Lesson 21 Subgrade Moisture Movement and Drainage

Lesson 22 Generating Alternative Pavement Design Strategies

Lesson 23 Life Cycle Costing and Economic Analysis

Lesson 24 Economic Evaluation of Alternative Pavement Strategies

III. PLANNING, MONITORING AND EVALUATION

Lesson 25 Pavement Inventory Data Collection, Equipment and Processing

Lesson 26 Field Measurement of Serviceability and Roughness

Lesson 27 Roughness Measurement Techniques

Lesson 28 Condition Survey Methods and Use

Lesson 29 Skid Measurement Methods

Lesson 30 Developing a Combined Index for "Overall" Performance - Objective Function

Lesson 31 Priority Programming

Revised WRH/lg 6/9/84

A PROGRAM OF STUDY IN PAVEMENT MANAGEMENT
VOLUME II

TABLE OF CONTENTS

Lesson 50 Systematic Pavement Management - Project Level

Lesson 51 Guest Lecturer - To Be Announced

Lesson 52 Review of Existing PMS Practices

Lesson 53 Implementing Pavement Management

VII. ADMINISTRATION AND RESEARCH

Lesson 54 Communication Among Areas of Pavement Management

Lesson 55 Typical Pavement Management Organization Within State Agency

Lesson 56 Collecting and Identifying Research Needs Systematically

Lesson 57 Experimental Design for Realistic Research

Lesson 58 Variability

Lesson 59 Technology Transfer

Lesson 60 Applied Statistical Concepts

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Revised WRH/lg 12/16/83
Lesson 21

LESSON OUTLINE

SUBGRADE MOISTURE MOVEMENT AND DRAINAGE

Instructional Objectives

1. To provide the student with a basic knowledge of the movement of moisture near the pavement structure.
2. To acquaint the student with the various methods of conveying water in the subgrade.

Performance Objectives

1. The student should be able to explain the various types of moisture movement in terms of factors that govern movement.
2. The student should be able to simply sketch the various methods of subgrade drainage and explain the function of each.

Abbreviated Summary	Time Allocations, min.
1. Introduction	5
2. Ground Water	10
3. Gravitational Water	5
4. Held Water	10
5. Soil Suction for Clays	10
6. Subgrade Drains	10
	50 minutes

Reading Assignment

1. Instructional Text

LESSON OUTLINE
SUBGRADE MOISTURE MOVEMENT AND DRAINAGE

1.0 TYPES OF WATER IN SOIL

1.1 Ground Water

Defined as water below the water table.

1.2 Gravitational Water

Defined as water flowing towards the water table under the action of gravity.

1.3 Held Water

Defined as water retained in the soil principally by surface tension forces.

2.0 WATER MOVEMENT

2.1 Ground Water

2.1.1 Darcy's Law. Movement of ground water is governed by Darcy's Law:

$$Q = KiA$$

where

Q is defined as discharge per unit time

K is defined as the coefficient of saturated permeability

i is defined as the hydraulic gradient

A is defined as the cross sectional area of flow

Darcy's Law only applies when flow is through porous material.

2.1.2 Coefficient of Saturated Permeability. K is primarily dependent on the particle size distribution of the soil.

(a) typical values of K (Visual Aid 21.2)
(b) determination of K by permeameter test (Visual Aid 21.3)

2.2 Gravitational Water

2.2.1 Darcy's Law. Due to the soil characteristics Darcy's Law is ineffective.

2.2.2 Water in Transit. Gravitational water falls on the soil under natural conditions and flows through to form a water table on some impermeable structure below. Gravitational water is mainly in transit.

2.3 Held Water

2.3.1 Movement. Held water moves slowly but is not static. Movement is determined by suction and vapor pressure equilibria. Water is generally held in place by:

(a) chemical combination in crystalline structure of soil
(b) surface tension around contact points of particles
(c) capillarities in pores between particles

2.3.2 Effect of Thermal Gradient.

(a) movement of water from warm to cold regions
(b) caused by alternating cycles of vapor condensation and capillary flow
(c) caused by change in water affinity with change in temperature

3.0 SOIL SUCTION

3.1 Definition

Soil suction may be defined as negative pressure in a soil mass that is of sufficient magnitude to create movement of held water.

3.2 Causes

Soil suction is caused by forces causing hydraution, or absorption of water to soil particles together with surface tension at the air water interfaces.

3.3 Numerical Values

(a) value depends on moisture content of soil (Visual Aid 21.4)
(b) values range from zero to several thousand psi

(c) units

(1) psi
(2) cm of H_2O
(3) pF defined as $\log_{10}$ (cm of water)

3.4 Components of Soil Suction

3.4.1 Osmatic (Solute) Potential. Amount of work to transport water reversibly and isothermally.

3.4.2 Gravitational Potential. Amount of work to transport water from one elevation to another.

3.4.3 Capillary Potential. Amount of work to transport water to external gas pressure of a point.

3.4.4 Potential Due to External Gas Pressure. Only considered when external gas pressure differs from atmospheric pressure.

3.4.5 Matrix or Soil Water Suction. Negative gauge pressure to be in equilibrium through a porus permeable wall.

3.4.6 Osmotic Suction. Negative gauge pressure to be in equilbrium through a semi-permeable membrane.

4.0 MEASUREMENT OF SOIL SUCTION

4.1 Types of Apparatus

4.1.1 Suction Plate.

pF range 0 - 3

4.1.2 Pressure Membrane.

pF range 0 - 6.2

4.1.3 Centrifuge.

pF range 3 - 4.5

4.1.4 Vacuum Desiccation and Sorption Balance.

pF range 5 - 7

4.1.5 Calibrated Electrical Absorption Gages.

pF range 3 - 7

5.0 pF HYSTERESIS LOOP

Suction versus moisture depends on soil characteristics.

5.1 Incompressible Soils

Considerable hysteresis between wetting and drying curves. Pores may empty and fill at different suction forces.

5.2 Compressible Soils

Development of high suctions produces a structural condition similar to that present in natural ground.

5.3 Intermediate Clays

Curves fall between those for compressible soils and those for incompressible soils.

6.0 SOIL PROPERTIES AND SOIL SUCTION

6.1 Shear Strength

$$\sigma' = p - \beta'\mu$$

where

σ' = shear strength

p = total normal pressure

β' = a bonding or holding factor

μ = pore pressure

μ can be inferred from the suction-moisture content relationship

6.2 California Bearing Ratio (CBR)

Relationship between suction and bearing ratio

$$CBR = C_1 + C_2S$$

where

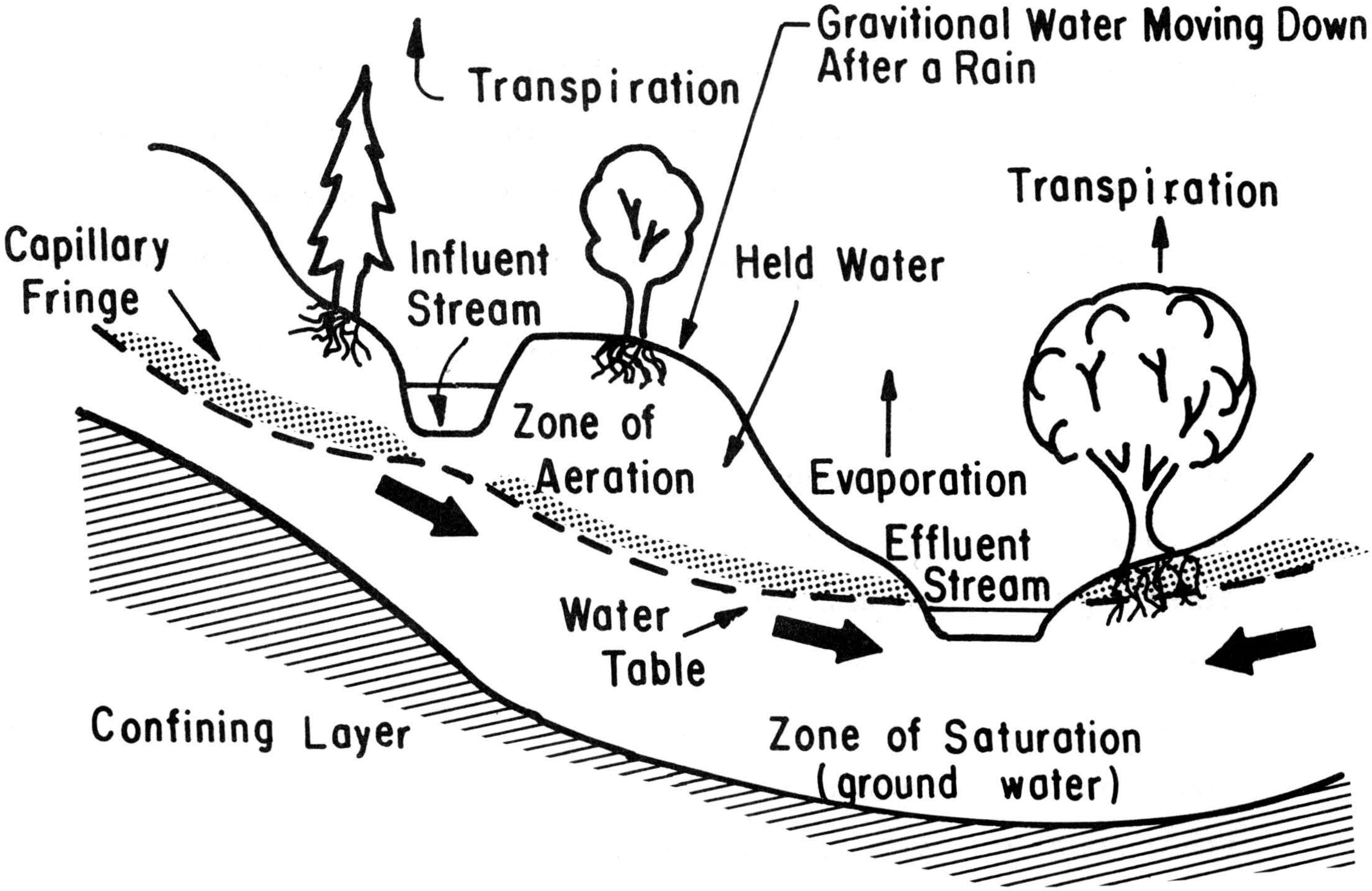
C_1, C_2 = constants

S = soil suction.

LESSON OUTLINE
SUBGRADE MOISTURE MOVEMENT AND DRAINAGE

VISUAL AID	TITLE
Visual Aid 21.1.	Schematic diagram showing occurrence of ground water.
Visual Aid 21.2.	Approximate particle size and permeability of various soils.
Visual Aid 21.3.	Simple constant - head upward - flow permeameter.
Visual Aid 21.4.	Soil suction at various moisture contents and conditions for heavy clay.
Visual Aid 21.5.	Units of measurement for soil suction.
Visual Aid 21.6.	Underdrain.
Visual Aid 21.7.	Interceptor.
Visual Aid 21.8.	Drain trenches.
Visual Aid 21.9.	Draw down water table.
Visual Aid 21.10.	Perched condition.
Visual Aid 21.11.	Interceptor ditches.
Visual Aid 21.12.	Membrane.
Visual Aid 21.13.	Membrane encapsulated.
Visual Aid 21.14.	Lime-treated layer.

Visual Aid 21.1. Schematic diagram showing occurrence of ground water.

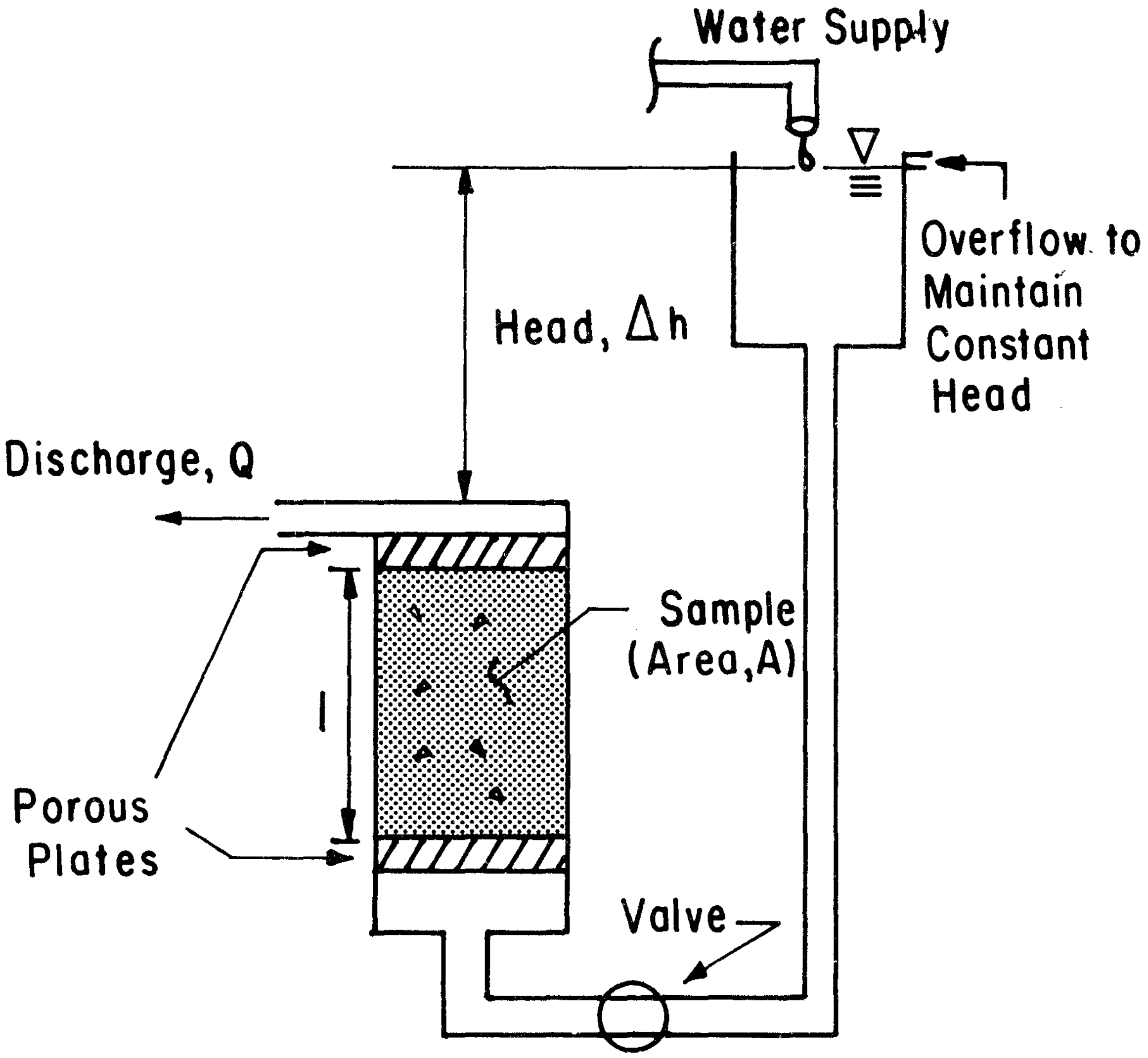


Visual Aid 21.2. Approximate particle size and permeability of various [illegible]

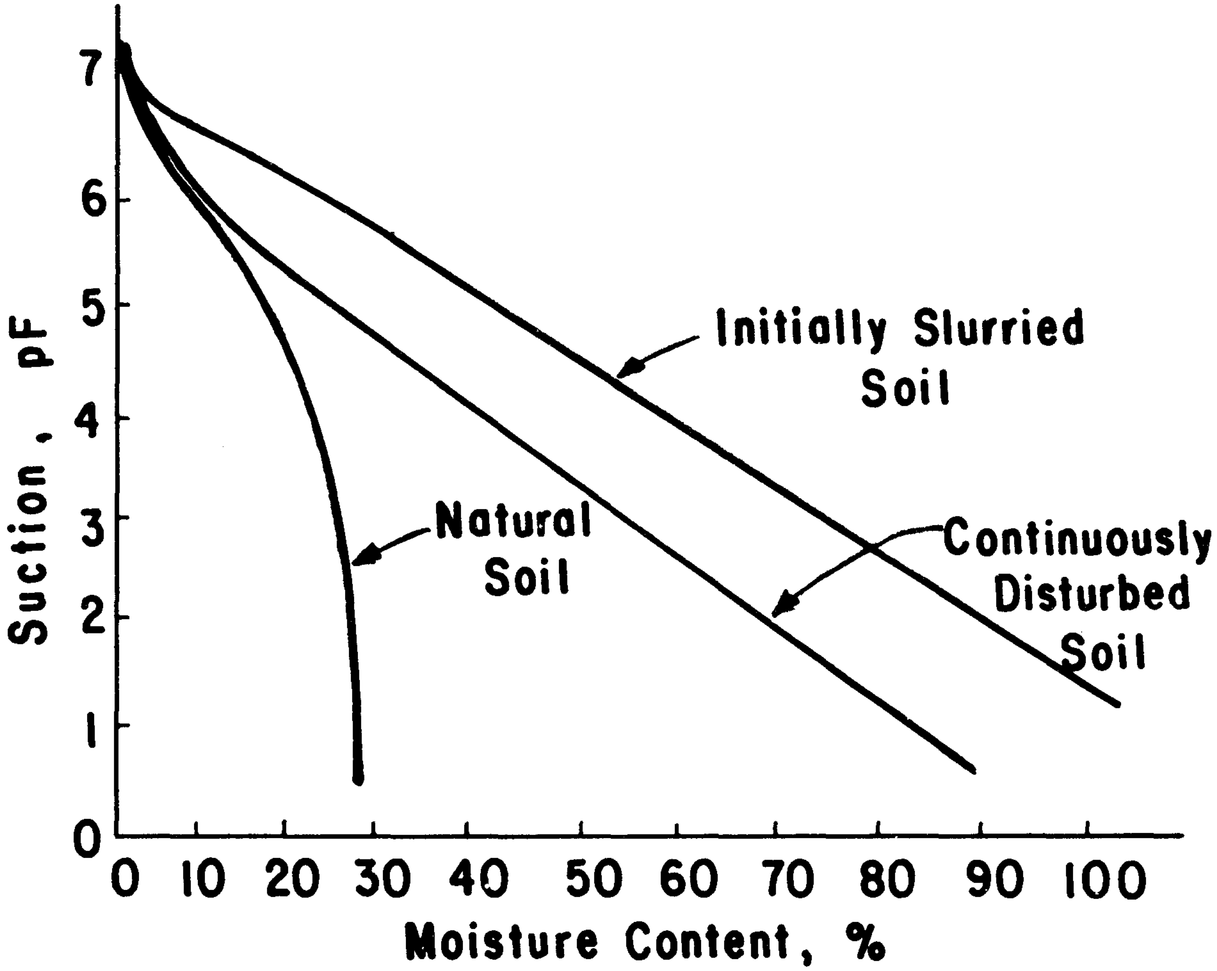
MATERIAL	PARTICLE SIZE,* MM	APPROXIMATE PERMEABILITY, GPD/SQ FT
CLAY	0.0001 - 0.005	10^{-5} TO 10^{-2}
SILT	0.005 - 0.05	10^{-2} TO 10
VERY FINE SAND	0.05 - 0.10	10 TO 50
FINE SAND	0.10 - 0.25	50 TO 250
MEDIUM SAND	0.25 - 0.50	250 - 1,000
COARSE SAND	0.50 - 2.00	1,000 TO 15,000

*1 MM = 0.03937 IN.

Visual Aid 21.3. Simple constant - head upward - flow permeameter.



Visual Aid 21.4. Soil suction at various moisture contents and condition for heavy clay.

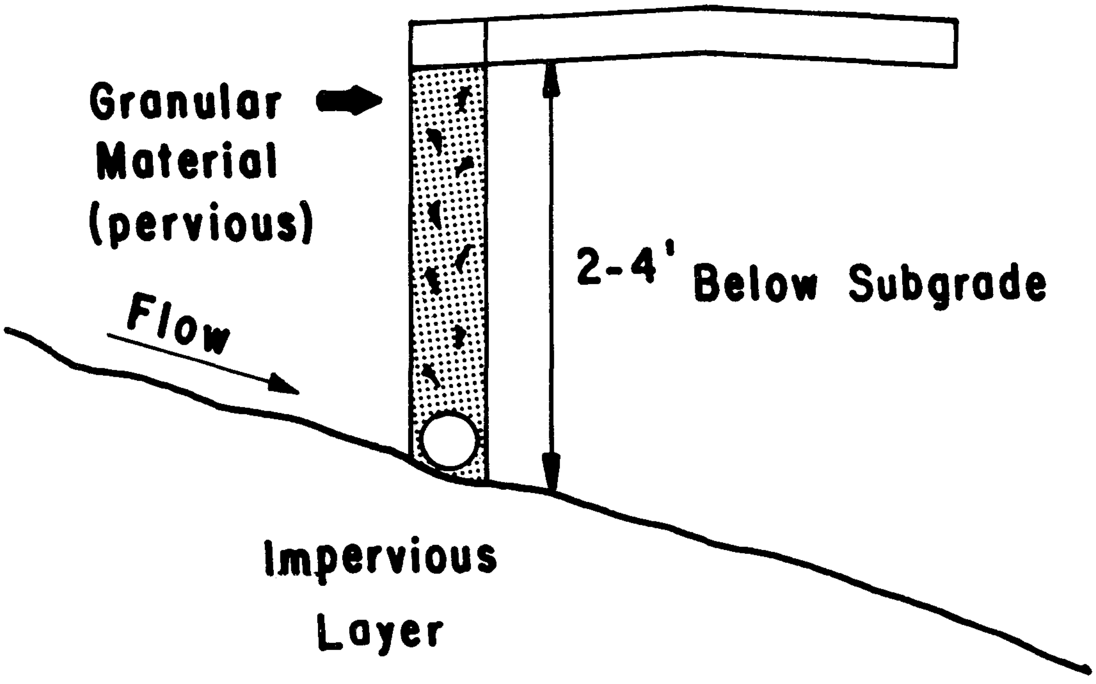


Visual Aid 21.5. Units of measurement for soil suction.

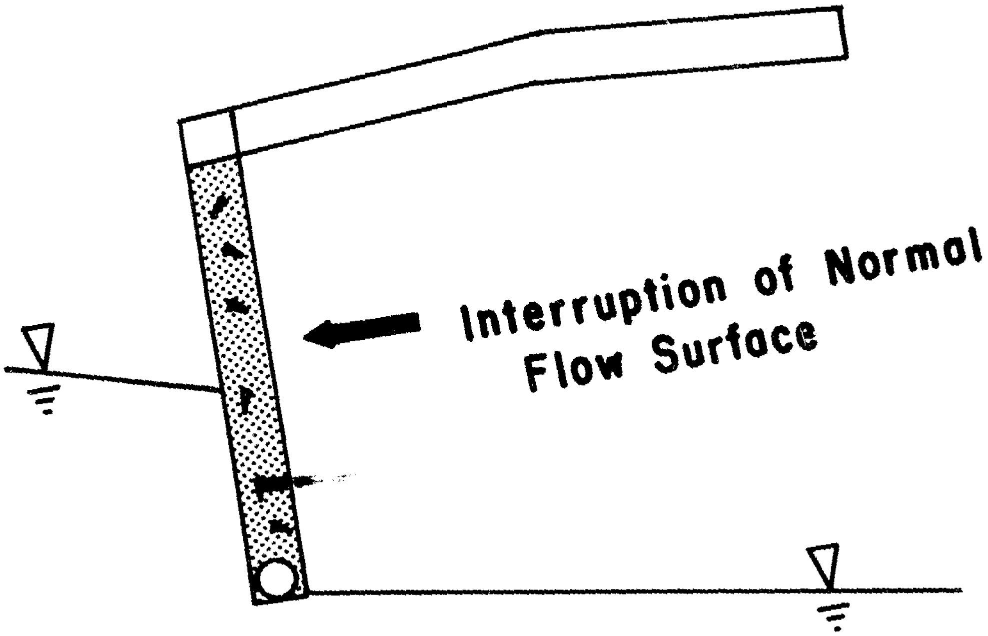
PF*	CM OF H_2O	PSI
0	1	0.0142
1	10	0.142
2	100	1.42
3	1,000	14.2
4	10,000	142.0
5	100,000	1420.0

*PF = $\log_{10}$(CM OF H_2O)

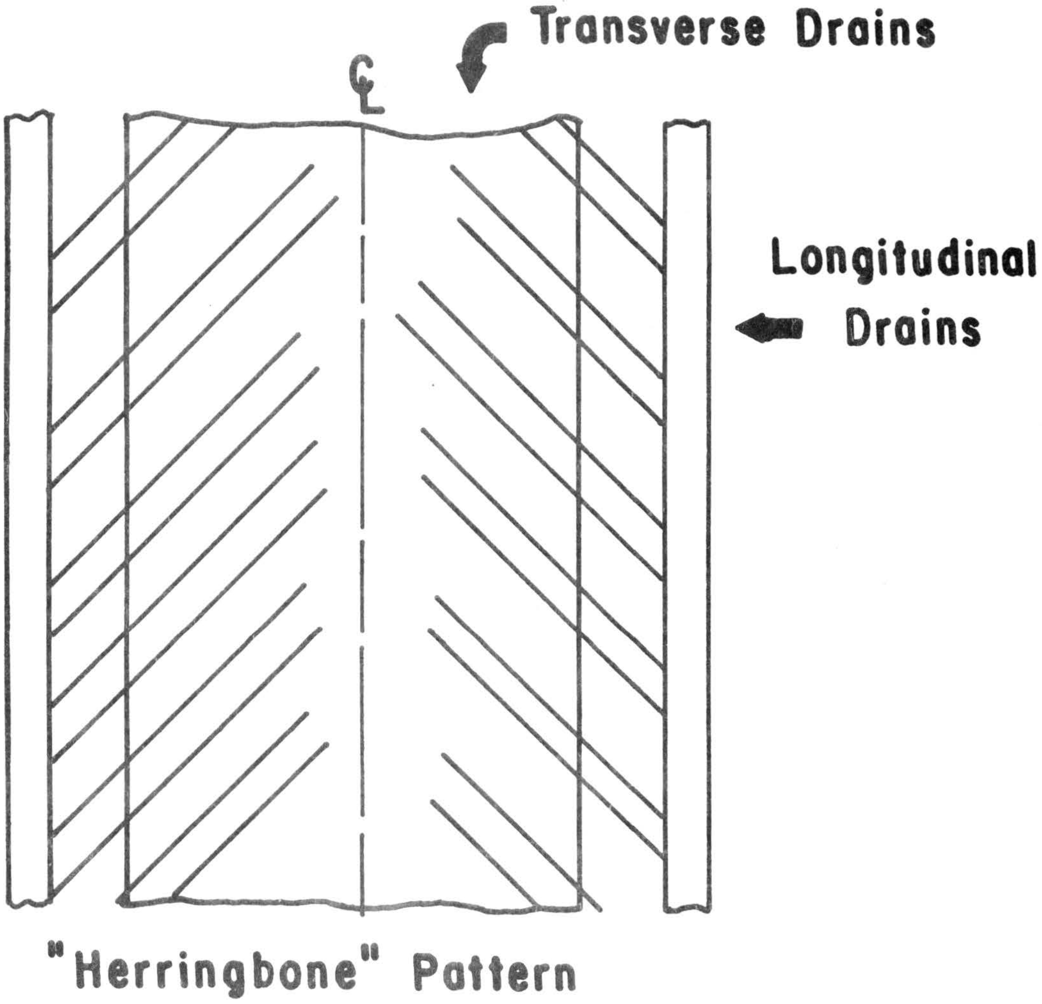
Visual Aid 21.6. Underdrain.



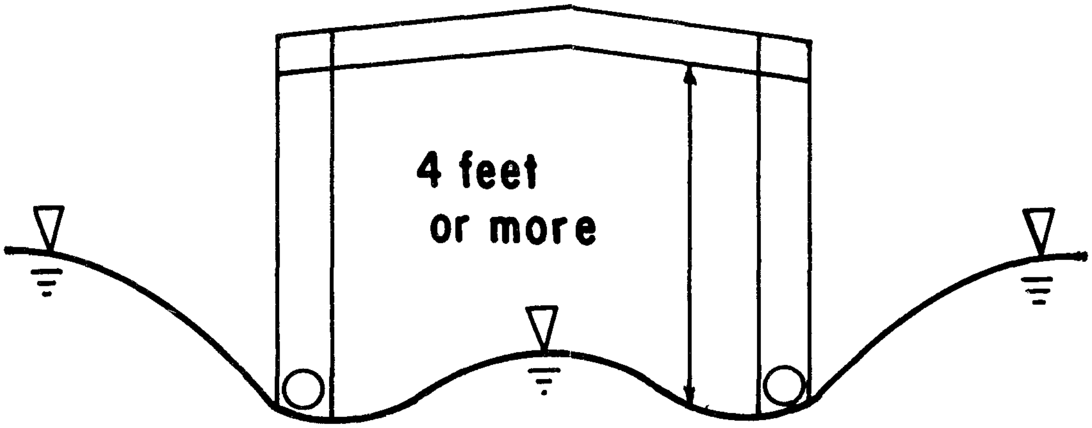
Visual Aids 21.7. Interceptor.



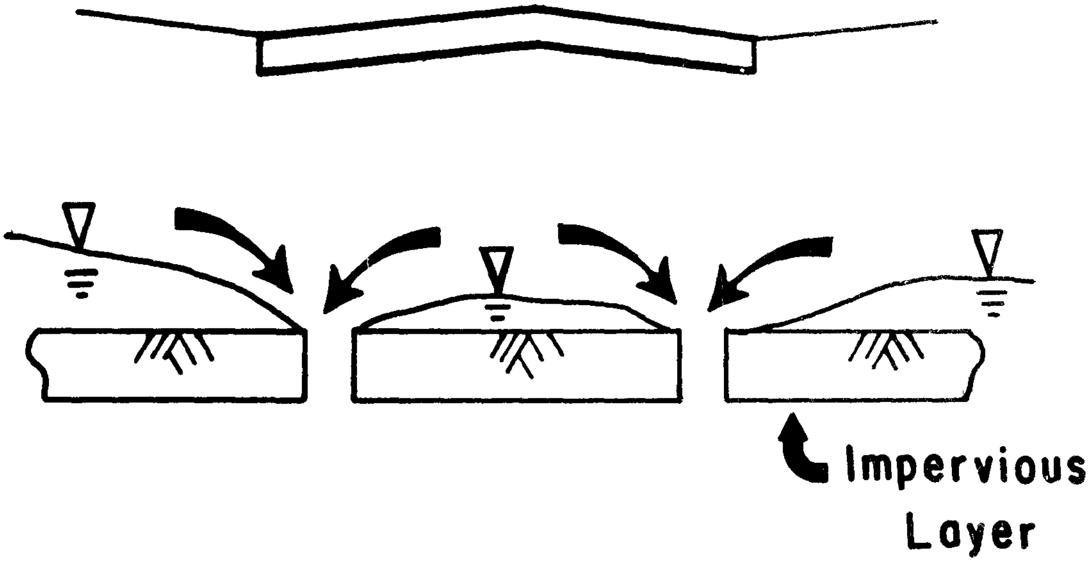
Visual Aid 21.8. Drain trenches.



Visual Aid 21.9. Draw down water table.

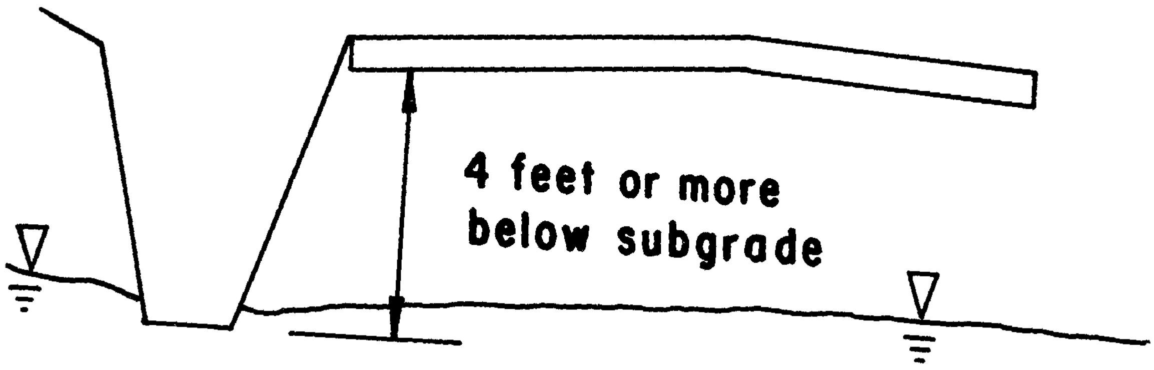


Visual Aid 21.10. Perched condition.

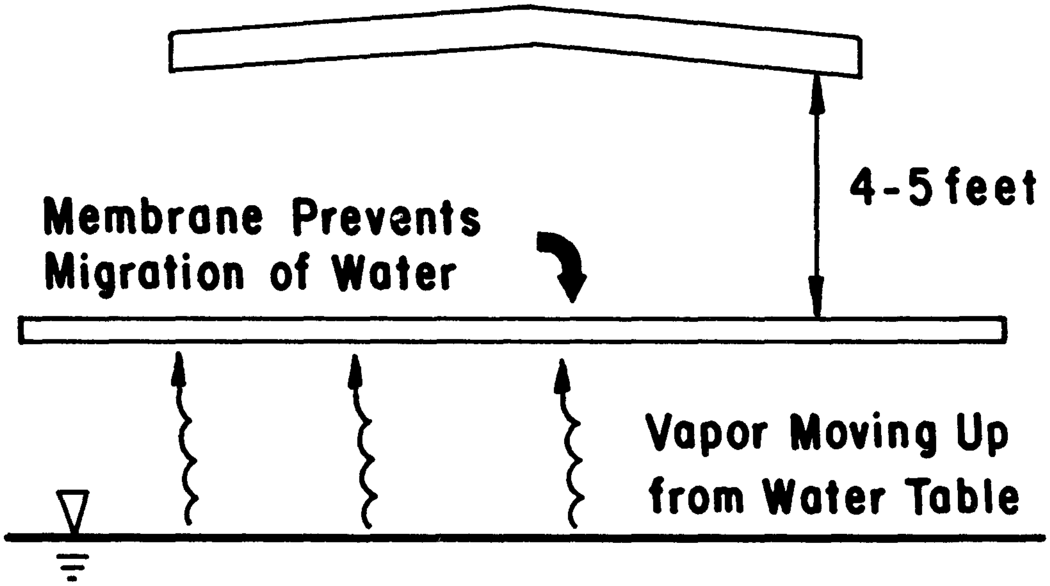


**Drain Water Through Holes
in Impervious Layer**

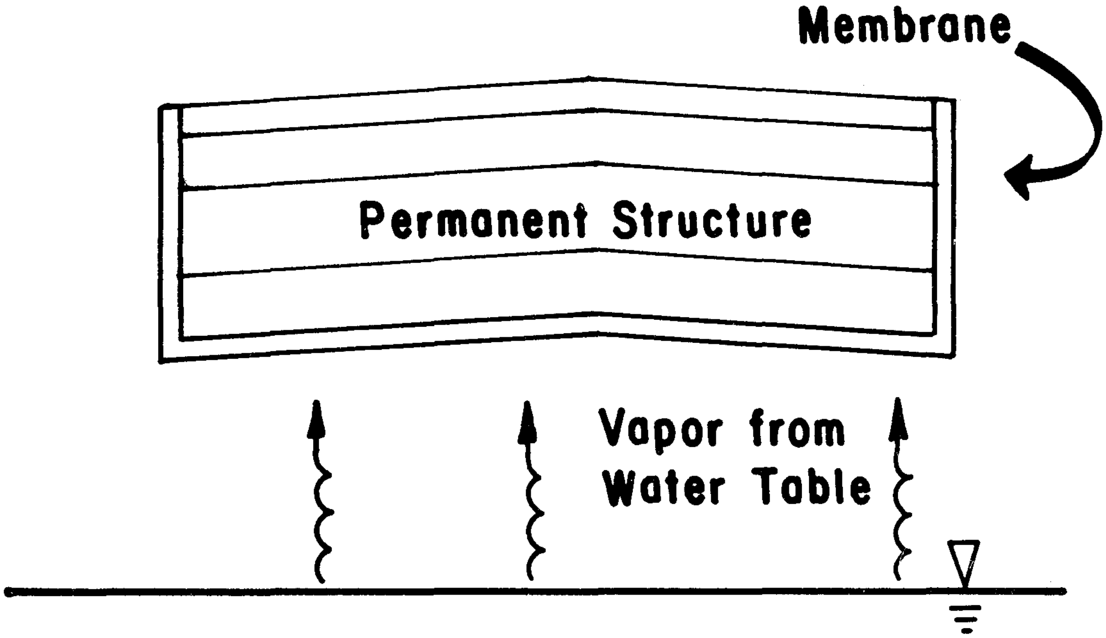
Visual Aid 21.11. Interceptor ditches.



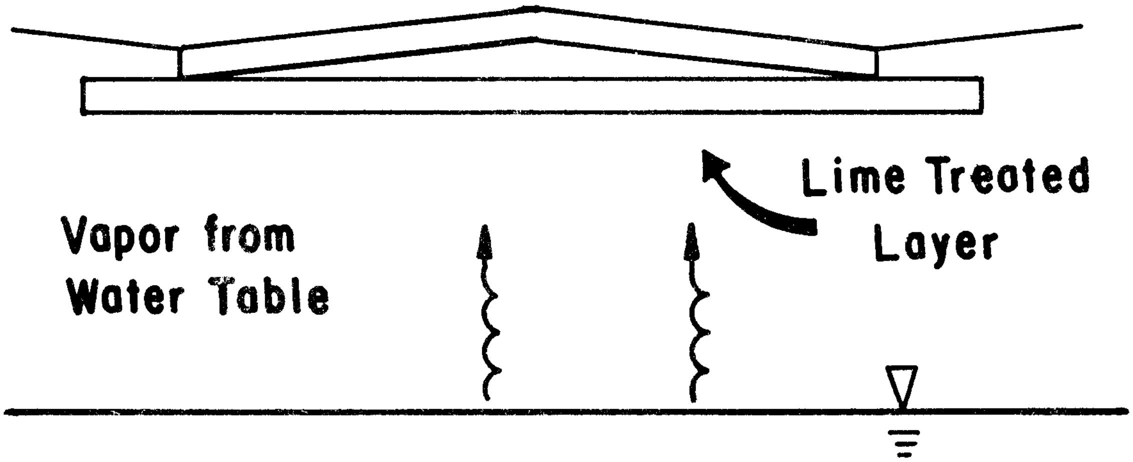
Visual Aid 21.12. Membrane.



Visual Aid 21.13. Membrane encapsulated.



Visual Aid 21.14. Lime-treated layer.



INSTRUCTIONAL TEXT

Unpublished Lecture Notes

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CHAPTER 7. MOISTURE MOVEMENT

Types of Water Movement

The water in soil can be broadly divided into three categories: ground water beneath the water table, gravitational water flowing towards the water table under the action of gravity, and held water retained in the soil principally by surface tension forces.

Of the water which falls on the soil under natural conditions, some passes through to form a water table on some impermeable stratum below. The water which passes through the soil in this form is generally referred to as gravitational water. The water below the water table is the ground water.

When the supply of surface water and the flow of gravitational water cease, some moisture is retained in the smaller pores and channels and on the surface of the particles by surface tension and adsorptive forces. This water which cannot be drained directly may be conveniently termed "held water." The water vapor which fills the soil interstices not occupied by water in the liquid phase and which, under special circumstances may play an important part in determining the distribution of moisture in the soil, may be regarded as constituting part of the held water.

Ground Water. For clays and other fine soils, the effects of ground water can best be studied by assuming that Darcy's Law of saturated flow applies.

$$Q = KiA$$

where

Q = discharge per unit time,

K = coefficient of saturated permeability,

i = hydraulic gradient,

A = cross sectional area of flow.

For a fine grained soil such as clay, K can be obtained by running a variable head permeameter test and by using the following equation:

$$K = \frac{aL}{At} \log_{10} \frac{h_o}{h_i}$$

where

a = cross sectional area of standpipe

A = cross sectional area of sample

t = time

h_o, h_i = original and final hydraulic leads

The coefficient of permeability is highly dependent on the particle size distribution of the soil. It may well vary from 20,000 ft/day for coarse gravels to .0002 ft/day for heavy clays. Due to the fact that horizontal and vertical structureof soils differ, the soil sample must be oriented in the direction in which flow is most liable to occur.

Gravitational Water. Gravitational water is water flowing through the soil to form a water table on some impermeable structure below. Since, in this case, the structural characteristics of the soil are such i.e. porosity, air voids, etc. that Darcy's Law is ineffective. Since gravitational water is mainly in transit, it is of little real importance to soil mechanics. However, in some cases, intercepting drains are used to avoid problems that may arise.

Held Water. Held water is not static but does move slowly. It can be classified as follows:

(1) water chemically combined in the crystalline structure of a soil,
(2) adsorbed water,
(3) water held by surface tension around the points of contact of the particles, and
(4) water held by capillarity in the pores between the particles.

The water that is chemically combined in soil minerals cannot be dried out but at a temperature of 110° C because it is an internal part of the soil solid.

The quantity of water held by surface adsorption depends on the surface area of the particles. This water can be reduced by oven drying the soil but never entirely removed.

The greater part of the held water in granular soils is retained by surface tension either around the points of contact of the particles or in the soil pores and capillaries. When a soil is exposed to moisture, first the water layer around the soil particles gets thicker. Next, the spaces between the soil particles are filled and held by surface tension. At this point, any additional water would cause gravitation flow.

Other Soil-Water Terms

In soils engineering and particularly in agricultural engineering aspects of soils, the terms wilting point, field capacity, and available water are widely used.

Field capacity is defined as the moisture content of soil after gravity drainage is complete. It has been proven that the field capacity is essentially the water retained in soil subjected to a suction of 1/3 atm.

The wilting point represents the soil moisture at the time that plants cannot extract water from the soil. It has been found to be the moisture held at a suction equivalent to the osmatic pressure exerted by the plant roots. Recent tests have indicated that it is represented by the moisture content at a suction of 15 atm.

The difference between the moisture contents at field capacity and at the wilting point is called the available moisture.

In the swelling clay problem, the water movement with which we are basically interested is that of held water. It is in this range of moisture holding capacity that the complexity of the problem of swelling clays finds its apex of complexity.

Movements of Held Water

Even though held water cannot be drained directly in this manner, it should not be regarded as static. Its movements are determined by suction and vapor pressure equilibria. The surface tension and absorptive forces by which it is retained reduce the vapor pressure of the held water and, at the same time, impart to the water itself a state of reduced pressure or suction which is found to increase from zero at saturation to values exceeding 100 lbs/in^2 in dry soils. If equilibrium moisture conditions in a soil suffer a local disturbance, the suction and vapor pressure gradients created cause a movement of moisture in the liquid and vapor phases tending to reestablish equilibrium. Since the particle size distribution is an important factor in determining both the soil-suction-moisture content and the vapor pressure-moisture content relationships, equilibrium will not, in the case of non-uniform soils, correspond to a state of uniform moisture content.

Since changes of temperature affect the vapor pressure of soil to a much greater extent than the suction, temperature gradients are important and should be included before we try to delve to moisture movement in the vapor phase.

Effects of Thermal Gradients

Observations and investigations of the influence of temperature gradients and variations in temperature on the retention and movement of water in the soil have been reported periodically since the early 1900's.

One of the earliest of the many reports noted evidence of diurnal fluctuations in the rate of discharge of water from tile drains and in the level of ground waters in shallow wells. In the course of varied field experiments it was also noted that substantial upward movements of water in unsaturated soils during the winter months under frost free conditions was observed. A considerable change in the rate of vertical capillary flow in soil columns under constant moisture-tension gradients was observed when the columns were subjected to a change in ambient temperature conditions.

After concentrated experimentation and independent study by many researchers, the following conclusions were reached:

(1) A marked transfer of water from warm to cold regions of soil specimens was found. After attempting to segregate liquid and vapor flow characteristics, the conclusion was reached that thermal effects were largely related to flow in the liquid phase.

(2) Moisture movement under the influence of a thermal gradient is the result of a combined cyclical process of vapor condensation and local capillary flow.

(3) There is flow of moisture in the film phase along the internal surface of the porous system due to a change in water affinity with change in temperature. "The physical explanation of the phenomenon is that the exchangeable cations possess a greater activity (are more dissociated) at the cold than at the warm side; they cannot move to the warm side because they are held by negative charges of the mineral surfaces. The only way to decrease the concentration potential is by movement of water from the warm to the cold side."

It is interesting to note that by different studies and techniques the investigations have arrived independently at similar conclusions. These conclusions are not always reinforcing, however. As observed in the first two conclusions, one names liquid water flow as the basic mechanics of water moisture due to thermal gradients, while the other promotes vapor condensation and local capillary flow as the causes for this motion. The generalized conclusions of the above mentioned investigation may be summarized as follows:

When a column of soil is subjected to a temperature gradient, the flow of moisture from warm to cool regions occurs largely in the vapor phase. As the vapor condenses in the cooler regions, a flow of moisture, in the liquid phase from cold to warm regions is created once a favorable pressure gradient within the liquid has been established. When the soil is sufficiently moist to permit active liquid flow, a state of equilibrium cannot be reached and a continuous circulation of water takes place within the enclosed column.

Water Movement in the Vapor Phase

At moisture contents well below saturation the air spaces inside the soil can provide continuous passages through which water may move in the form of vapor, and consequently, it is possible for changes in the moisture content of the soil to occur owing to the movement of water vapor from one region of the soil to another through these air channels. This movement is due to differences in relative humidity of the void space water vapor in the different parts of the soil. (The relative humidity of the water vapor is defined as the pressure of the water vapor in soil expressed as a percentage of the saturated vapor pressure of water at the same temperature.)

Differences in relative humidity are associated with variations in soil type, soil moisture content, and temperature. Under actual and practical road conditions, the temperature is the only one of these factors likely to be of importance, since local variations in moisture content can only cause appreciable differences in relative humidity when the soil has a comparatively low moisture content (<4% for sands and below 10% for clays). Temperature gradients such as those created in the soil by the daily and annual temperature cycles may cause considerable differences in vapor pressure within the top few feet of the soil, and, if free channels are available in which water vapor can move, a transfer of moisture will occur.

When the soil is normally close to saturation, free movement of water vapor is largely prevented and, thus, appreciable transfer of moisture is also prevented. In arid areas where very low moisture contents and large temperature variations are expected in the soil, the movement of water in the vapor phase is of great importance. This movement is one explanation for the high moisture contents which are found under some roads constructed in arid and semiarid climates. These impermeable surfacings are able to prevent the evaporation by which the moisture accumulated in the surface layers of the soil is normally removed.

As a conclusion we can say that the transport of water vapor in the soil is controlled by temperature differences. Vapor movement is from high temperature (high vapor pressure) to low temperature. Vapor transport is an important factor in moisture movement when the moisture content is lowered to the point where capillary moisture is discontinuous. Under this condition, however, moisture content and temperature gradients are usually so small that the quantity of water moved is negligible. When the surface soil is frozen, the vapor pressure gradient is upward and is accentuated by the lower vapor pressure of ice relative to water at that low temperature. Thus, when frozen soil thaws, its moisture content may be greater than at the time of freezing conversely, during summer, vapor pressure gradients would be downward were it not for the evaporation and transpiration. These conditions lower the surface temperature and significantly shift the gradient patterns.

It is apparent and the general consensus that soil suction, negative pressures, by dry soils are the primary source of movement potential in swelling clays. For this reason this subject is given a fuller treatment than was given to the previous types of soil-water movement.

Soil Suction in Swelling Clays

Soil suction is perhaps the backbone of the moisture movement phenomena in swelling clays. The soil suction which a soil experiences is caused by forces causing hydration, or adsorption of water to soil particles together with the surface tension at the air water interfaces. These forces combine to produce a state of reduced pressure, or suction, in the soil. The soil suction is experienced mostly in the held water fraction of the moisture range. It has been found experimentally that the increase in soil suction with decreasing moisture content is continuous over the entire moisture range.

The value of the suction is dependent on the moisture content of the soil. The suction-moisture relationship for clays is found experimentally to be continuous for all soils. Figure 7.1 shows that the soil suction increases rapidly with decreasing moisture content.

> It follows from the previous discussion that in clays which are normally saturated at moisture contents above 15%, the suction in the water is due mainly to particle hydration and adsorption whereas in granular soils, surface tension plays the most important role.

Values of suction range from zero at complete saturation to several thousands of psi in oven dry soils.

The suction pressure can be expressed as negative pressure in psi or in other more convenient units. It will be shown later that the most common units for soil suction are those which express it in terms of cm of water. The term pF which has also been employed is the $\log_{10}$ (cm of water) and is used for ease in plotting. The water moisture is almost always plotted as a percent.

The following chart relates the three basic units of pressure used:

pF	h (cm of H_2O)	psi
0	1	.0142
1	10	.142
2	100	1.42
3	1000	14.2
4	10000	142

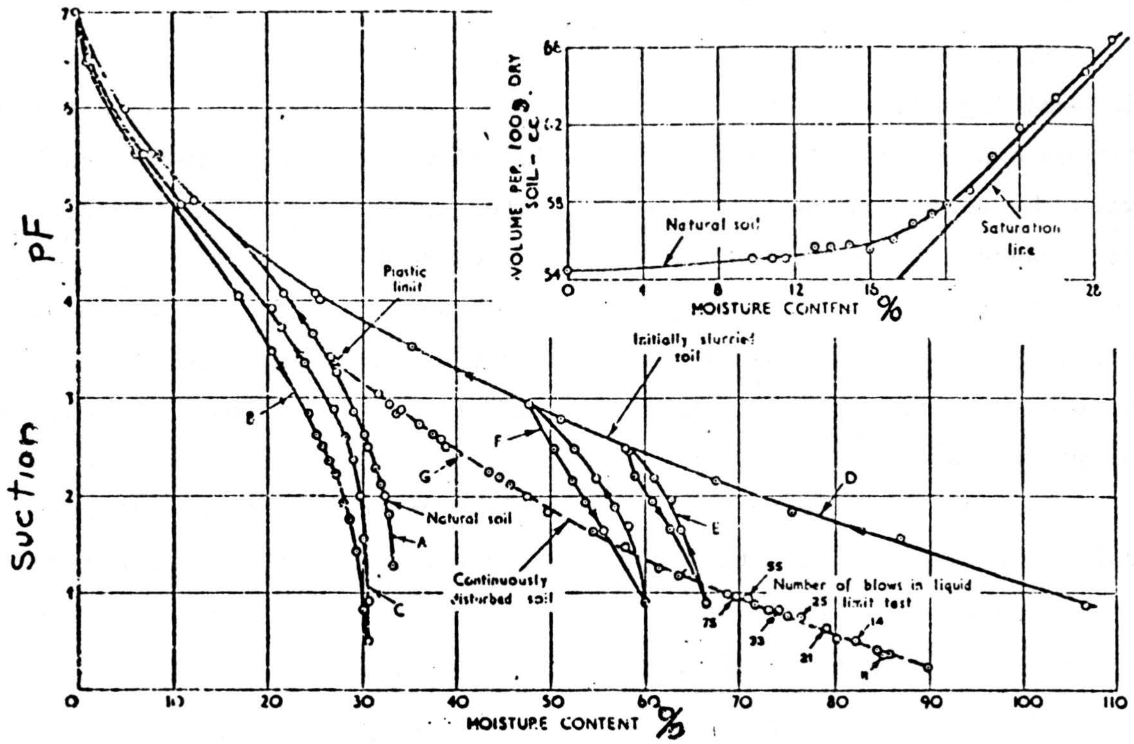


Fig. 7.1. Suction/Moisture Content and shrinkage relationships for a heavy clay soil

Components of Soil Suction

The total potential of soil water, φ , is the amount of work that must be done per unit quantity of pure water in order to transport reversibly and isothermally on infinitesimal quantity of water from a pool of pure water at a specified elevation at atm. pressure to the soil water at the point under consideration. It may be convenient to shorten the term to total potential or soil water potential and to divide it into parts, the division such that the sum of the parts equals the total potential.

Osmotic (Solute) Potential. Amount of work per unit quantity of pure water to transport reversibly and isothermally an infinitesimal quantity of water from a pool of pure water at a specified elevation at atm. pressure to a pool containing a solution identical in composition with the soil water but in all other respects identical to the reference pool.

Gravitational Potential. Amount of work from a pool containing a solution identical in composition to the soil water at a specified elevation at atm. pressure to a similar pool at the elevation of the point under consideration.

Capillary Potential. Amount of work from a pool containing a solution identical in composition to the soil water at the elevation and the external gas pressure of the point under consideration in the soil water.

Potential Due to External Gas Pressure. This potential component is to be considered only when the external gas pressure differs from atm. pressure.

Matrix or Soil Water Suction. Negative gauge pressure relative to the external gas pressure on the soil water to which a solution identical in composition with the soil water must be subjected in order to be in equilibrium through a porous permeable wall with the soil water.

Osmotic Suction. Negative gauge pressure to which a pool of pure water must be subjected in order to be in equilibrium through a semipermeable (permeable only to water molecules) membrane with a pool containing a solution identical in composition with the soil water.

Total Suction. Total suction is equal to the sum of the soil water suction and the osmotic suction.

pF - Hysteresis Loop

The nature of the suction vs moisture relationship depends on the characteristics of the soil. For a better more lucid understanding of this relationship it is convenient to consider three categories. They are the following:

(1) incompressible soils and materials of rigid structure,

(2) compressible soils which may remain saturated at high suctions where the void space remains filled with water due to volume changes accompanying volume changes, and

(3) sandy clays or intermediate clays.

Incompressible Soils. The characteristic suction curves for two incompressible materials, in this case two grades of chalk (soft limestone), are shown in Fig 4.9 There is considerable hysteresis between the wetting and the drying curves. This arises from the fact that pores may empty at a suction different from that at which they will fill. The vertical part of the drying curves indicates that considerable suctions can be applied to the pore water without change of moisture content, the effect being limited to a change in the radii of the water menisci at the surface pores. When the air-entry suction of those pores is reached drainage commences and is indicated by the change in slope of the curve. For both types of chalk the wetting curve is below and to the left of the drying curve. This indicates a more gradual decrease in suction as the moisture is increased. The differences between the two sets of curves in Fig 4.9 indicates different densities and hence different saturation moisture contents for the two materials.

Compressible Soil. The nature of the relationship for compressible clay soil is shown in Fig 7.1. Curve A represents the drying condition for an undisturbed sample taken from the ground and initially wetted to a very low suction. Curve B refers to the same soil wetted from an oven dry condition. Curve C is the second drying curve. Curve D indicates the same type of soil in an initially slurried condition. As seen in the figure, the curve is identical with the drying curve or the natural soil Curve A at suction of 4.5 pF, which suggests that the development of high suctions produces a structural condition similar to that present in the natural ground. The intermediate suction loops E and F show the effect of the suction-moisture relationship of slurrying at a lower moisture content than that used for the soil used to obtain Curve D.

Intermediate Clays. The suction curves for intermediate soils partake of the characteristics of both the compressible and the incompressible soils. The curves for undisturbed natural samples of such soils lie between those of the sands and the heavy clays to form a continuous spectrum. In this spectrum, the vertical characteristics of the sands at low suctions seem to become less and less pronounced as one passes through the sandy and silty clays towards the heavy soils. When heavy clays in the natural moisture condition are remolded and compacted to different degrees, the compacted material consists of aggregates of saturated or nearly saturated clay with comparatively large air voids. Although it is the magnitude of these air voids which determine the difference in dry density between the samples, their presence has little effect on the suction-moisture relationship as a whole.

Even though by carrying out a series of tests on sandy or silty clay soils at different initial densities a family of curves is obtained; these curves are not strictly comparable with those for an incompressible soil. The reason for this lies in the fact that these curves do not represent isodensity conditions because of the natural swelling and shrinkage of the soils. By carrying out swelling/shrinkage test simultaneously with the suction determinations, a family of suction curves corresponding to constant density conditions can be prepared. These density curves show the same crossover characteristics as the curves for incompressible soils. A detailed study of the suction/moisture/density relationship of this kind is likely to be of importance in soil swelling studies. Figure 7.2 shows typical suction-moisture-density curves for drying of a silty soil in three degrees of compaction. Figure 7.3 shows the same relationship for various densities of a fine sand. The same crossover characteristic is also observed here.

It is apparent that the suction of a given soil is a difficult parameter to determine. Since the factors governing soil suction are infinitesimal in nature the tests which exist follow an empirical basis. Nevertheless, the measurement of soil suction is an important factor to consider.

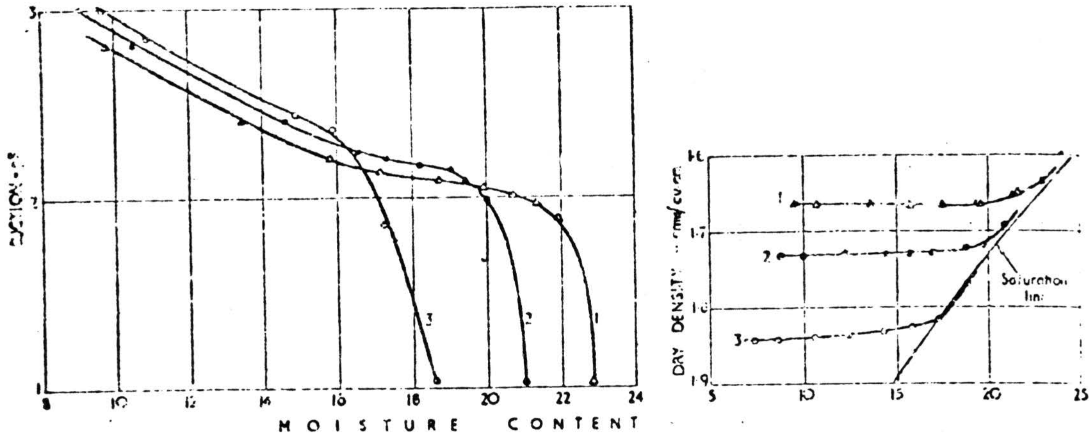


Fig. 7.2. Relationship between suction and moisture content of a silty soil at three

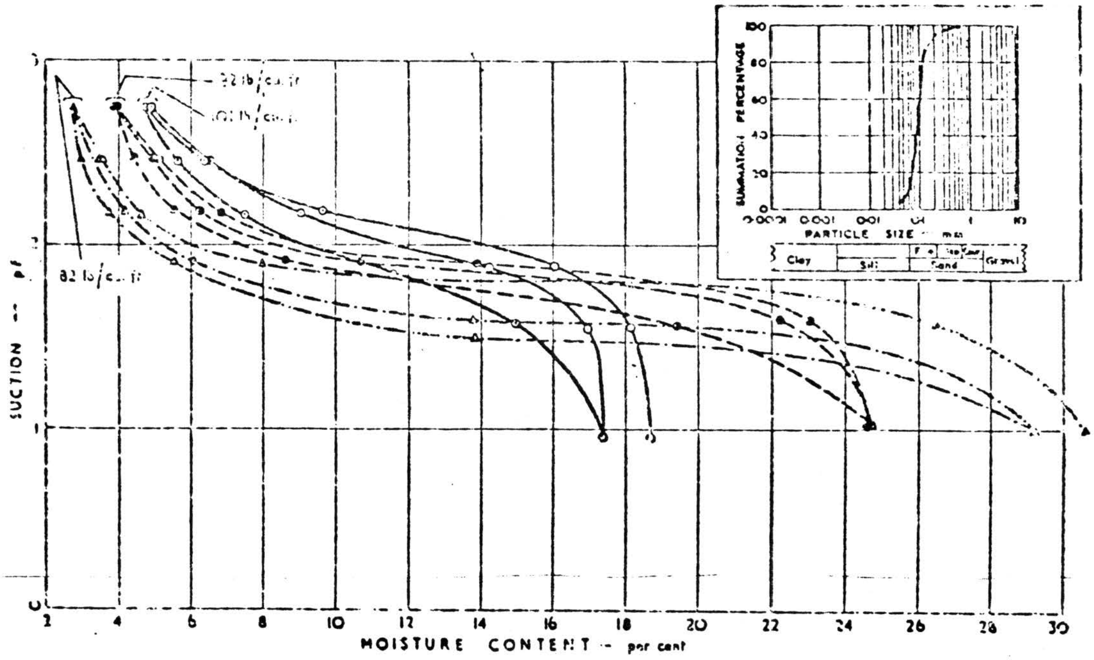


Fig. 7.3. Relationship between suction and moisture content at various dry densities for a fine sand.

Measurement of Soil Suction

Many different measurement devices have been designed to determine the soil suction and moisture content relationships of soils. This is of great importance since the thermodynamic approach to soil moisture movement analysis can be used to evaluate moisture migration in terms of moisture content on a weight basis if the appropriate relationship between suction and the moisture content are known. Table 7.1 lists the different methods which have been developed to determine this relationship

TABLE 7,1. TYPES OF APPARATUS FOR SUCTION MEASUREMENT

Method	pF Range
Suction Plate	0-3
Pressure Membrane	0-6.2
Centrifuge	3-4.5
Vacuum Desiccation and Sorption Balance	5-7
Calibrated Electrical Absorption Gages	3-7

The two most common tests used by research and laboratory work on the suction plate and the pressure membrane methods. As can be seen from Table 7.1, different methods have different ranges and their usage depends on the precision required by the experimenters.

Pressure Membrane Apparatus. In the pressure membrane apparatus of Fig 7.4, the sample is placed in contact with a cellulose membrane which is itself in contact with water at atmospheric pressure. The air pressure surrounding the sample is increased to produce a pressure differential between the water in the soil and the water in the membrane. Moisture leaves the sample until the suction (in psi) in the soil is numerically equal to the applied pressure. The suction-moisture content relationship can then be examined by carrying out tests at increasing suctions. For research purposes the range of this method, previously regarded as pF 0 to pF 5.0 has been extended by the Road Research Laboratory to suctions as high as pF 6.2, the latter value involves pressures of 1500 atmospheres.

Suction Plate. The form of the suction plate equipment in use at the Rand Research Laboratory is shown in Fig 5 Demountable chemical glassware is used, the No. 5 sintered glass porous plate being fused into the narrow end of a No. B.40 cone. This is fused at its other end into the wide end of a No. B.24 cone to the narrow end of which is also fused a glass tube of 5 mm internal diameter. The No. B.24 cone fits into a female joint at the mouth of a standard filter flask, the length of the glass tube on the filter unit being adjusted so that it reaches almost to the bottom of the flask when the apparatus is set up. The filter unit is filled with air-free water and the flask itself contains air-free water above the level of the tube connected to the filter unit. A reduced pressure of between a few centimeters and one atm. can be applied to the sintered plate by evacuating the space in the filter flask, the minimum being determined by the difference in level between the plate and the water in the flask. A glass cap is fitted over the No. B.40 joint carrying the sintered plate and this is connected by a tube that would otherwise be the open end of the monometer recording the pressure in the filter flask. This minimizes fluctuations of pressure in the apparatus due to variations of atmospheric pressure and temperature.

The suction-moisture content relationship is explored by allowing samples to reach moisture equilibrium successively with plates operating at a range of suctions. This equilibrium wet weight for each suction is measured and the moisture contents on a dry weight basis are calculated from the oven dry weight obtained at the conclusion of the tests.

So far we have looked at soil suction as the basic parameter to be considered when studying moisture movement in swelling clays. There exist also physical correlations and design properties which can be related to suction moisture properties of soils. Not much work has been done in this area, however, but what has been done indicates promise.

Soil Properties and Soil Suction

Shear Strength. It is probable that the shear strength of unsaturated soils (pF 70) can be determined by an expression similar to that for determining the volume.

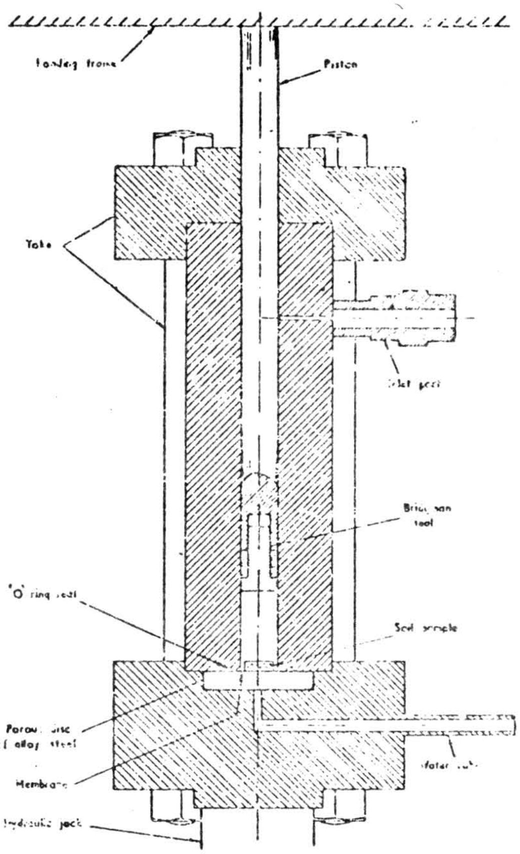


Figure 4.4 Membrane apparatus for high pressure.

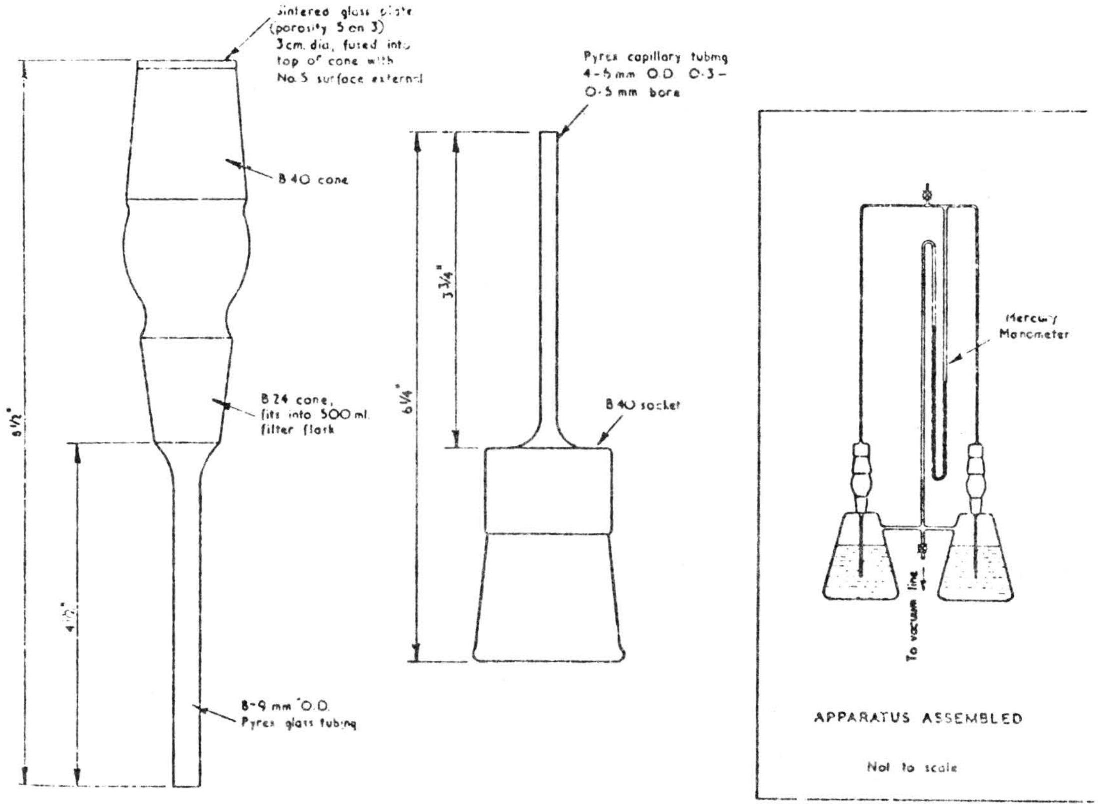


Figure 4.5 Suction plate apparatus.

$$\sigma' = p - \beta' \cdot \mu$$

where p is the total normal pressure on the shear plane and μ is the pore water pressure on the shear plane. The pore pressure μ will in general be below atmospheric pressure and in principle can be measured by some suitable energy method. In practice it may be inferred from a knowledge of the suction-moisture content relationship for the soil and from the known effect of applied pressure on pore water pressure.

The coefficient β' appears to be a holding or bonding factor and is a measure of the number of bonds of water under tension effective in contributing to the shear strength of the soil. The coefficient s is a measure of the theoretical or microscopic shearing strength per bond. Considering the variation of the shear strength of soil with moisture content, as the soil dries the suction increases and the strength of the bonds per unit area of water contact also increases. On the other hand, the number of bonds and the area of each individual bond both decrease as the soil becomes more unsaturated. In this way the product $\beta \times s$ may reach a maximum as the soil dries, giving also a maximum of shear strength. At such a maximum

$$\beta' \cdot ds + s \cdot d\beta' = 0$$

The maximum shear strength will occur at suctions less than one atmosphere for coarse sands, β being zero for such soils when air dry. For silts the maximum will occur at higher suctions than for sands; for clays the maximum shear strength will occur at very high suctions (Fig 7 .7).

Some experimental data has been plotted on Fig 7 .7 which shows the increase in shear strength within an increase in soil suction. The suctions were determined before shearing and when the condition of suction was represented by a point on a unique line,

California Bearing Ratio (CBR). Studies which have been made of the CBR of soils at various moisture contents and dry densities also suggest a close relationship between suction and the bearing ratio of the soil. Figure 7.7 snows a family of curves relating CBR on a log scale with moisture content for various dry densities. These curves indicate an optimum moisture content at which the highest CBR can be obtained. As was expected, the optimum moisture content decreased as the density increases.

Figure .8 relates suction and CBR both on arithmetic scales. It is obvious that there exists a notable increase in CBR as the suction of the soil is increased. A change in density of 15 pct shows a glaring increase in CBR as shown in the figure. However, at higher suctions the paucity of points makes a generalization impossible.

The results suggest a linear relationship of the type

$$\text{CBR} = C_1 + C_2 \cdot S$$

where

C_1 , C_2 = constants

S = soil suction

In the CBR test a bearing capacity failure may occur in some cases. Restraint due to the pot and incomplete mobilization of the full bearing capacity at 0.1 inch deflection will in other cases make a bearing capacity analysis inapplicable to the test. The relationship between the bearing capacity of soil, the apparent angle of internal friction, ϕ , and the apparent cohesion, C , is of the form

$$\text{Bearing capacity} = f_1(\phi) + C \times f(\phi)$$

If the apparent cohesion is proportional to the product $\beta' \cdot s$, the equation becomes

$$\text{Bearing capacity} = f_1(\phi) + \beta' \cdot s f_3(\phi)$$

The approximate linearity of the variation of CBR values with suction at suctions below one atmosphere for unsaturated silty sand (LL24, PL22) is shown in Fig. 7.8.

As a resume on the water movement in clays it must be kept in mind that motion of moisture is a complex thermodynamic and physical phenomena. Many mathematical derivations have been offered to solve this problem. Basically, it can be said that few have succeeded in solving it even with the most basic assumptions.

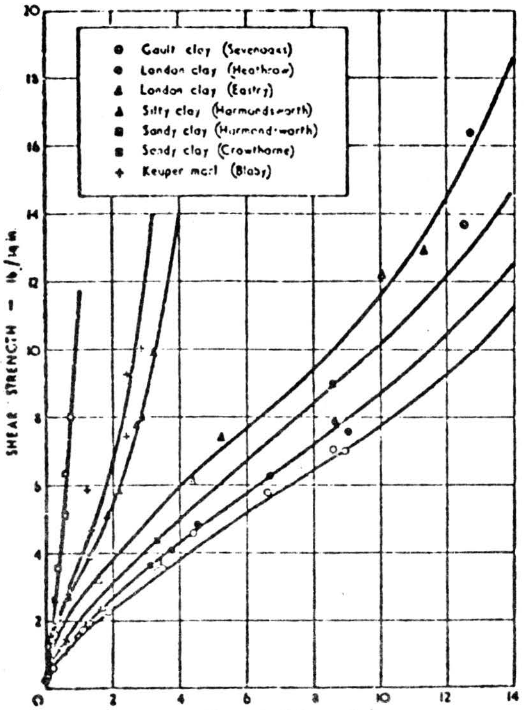


Figure 9.6 Relationship between shear strength and soil moisture suction for continuously disturbed soil.

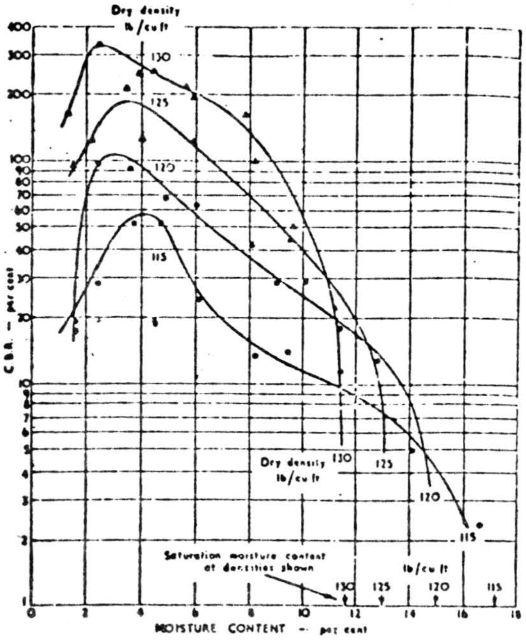


Figure 9.7 Variation of C.B.R. with moisture content for a sand at different densities.

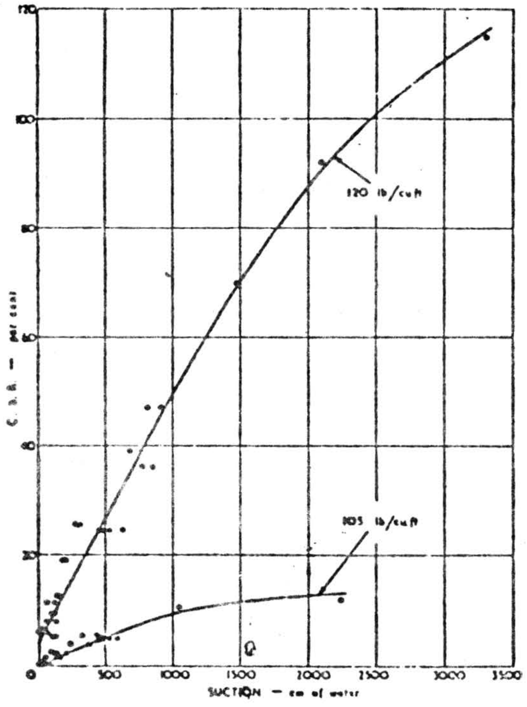


Figure 9.8 Variation of C.B.R. with suction of a silty sand at densities of 105 and 120 pcf.

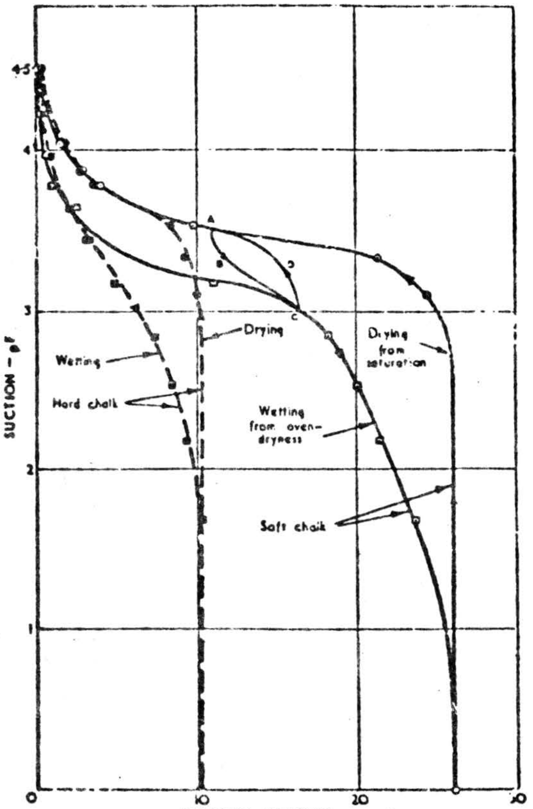


Figure 9.9 Relationships between suction and moisture content for samples of hard

Thermodynamics and Soil Water

Following the thermodynamic methods of Gibbs it is possible to show that ϕ, the total potential of a constituent, can be divided into μ, the chemical potential and θ, the positional potential; thus

$$\phi = \mu + \theta$$

The chemical potential is the chemical free energy per mole and depends on the nature and state of the constituent. The positional potential is the potential free energy per mole and depends on the position of the constituent in an external force field or fields. The external force field will be any field which extends over a region which is large compared to the size of a molecule.

Since ϕ is a measure of the escaping tendency of a constituent, assume that the gradient of ϕ is the force tending to cause movement. If it is further assumed that frictional forces are proportional to V, the steady state velocity, the following can be written as

$$V = -K \frac{d\phi}{dx} \qquad (1)$$

where

K = transmission constant

By substitution

$$V = -K \left[\frac{d\mu}{dx} + \frac{d\theta}{dx} \right] \qquad (2)$$

In an ideal solution,

$$d\mu = \bar{V}dP - \bar{S}dT + \frac{RT}{N} dN \qquad (3)$$

where $\bar{V}$, $\bar{S}$, N are the partial molar volume, partial molar entropy, and mole fraction of the constituent, respectively. P is the pressure, T is the absolute temperature, R is the gas constant. Therefore

$$V = -K\left[\bar{V}\frac{dP}{dx} - S\frac{dT}{dx} + \frac{RT}{N}\frac{dN}{dx} + \frac{d\theta}{dx}\right] \qquad (4)$$

Each of the terms can be regarded as a force tending to cause water movement with respect to a particular frame of reference. If the frame of reference is not the same for the different forces they will not have the same transmission constant.

From Eq 4, several equations applicable to the movement of water in soil may be obtained. If temperature is uniform, free salts are absent or uniformly distributed, and the only force field present is the gravitational field,

$$dT = 0$$

$$dN = 0$$

$$d\phi = Mgdh$$

from Eq 2

$$d\mu = \bar{V}dP$$

$$d\theta = \rho g\bar{V}dh$$

where

$$V = -K\bar{V}\left[\frac{dP}{dx} + \rho g\frac{dh}{dx}\right] \qquad (5)$$

where

ρ = density of water

h = height above datum

Since

$$Q = VA$$

then

$$Q = -K\bar{V}A\left[\frac{dP}{dx} + \rho g\frac{dh}{dx}\right] \qquad (6)$$

where

A = cross sectional area of pores

In this form the equation is a derivation of Darcy's equation.

Water Equilibria. When equilibrium is attained, the velocity of the water is zero and Eq 4 becomes

$$V = 0 = \bar{V}dP - \bar{S}dT + \frac{RT}{N} dN + d\theta$$

This is integrated between the reference phase, indicated by the zero superscript and any other phase at the same temperature to give

$$\bar{V}(P - P^{o}) = RT\ln \frac{N^{o}}{N} - (\theta - \theta^{o})$$

To apply this equation to the swelling of clays, let the symbols without superscripts refer to the median plane between two parallel clay plates and the symbols with superscripts to the external solution. Then the pressure difference $(P - P^{o})$ is equal to the hydrostatic repulsive force which tends to separate the plate and cause swelling.

CHARACTERIZATION OF PAVEMENT MATERIALS FOR FUNDAMENTAL STRUCTURAL ANALYSES

INTRODUCTION

Elastic and viscoelastic structural analyses of pavements as layered systems are increasingly becoming a part of working design practice. This is largely due to the ease with which such analyses can be done be readily available computer programs and the easy understandability of the results. Moreover, there is growing evidence that the results of these analyses can be directly related to observed field performance.

The inputs to these fundamental structural analyses must come from laboratory and field evaluations. Computers unfortunately cannot test materials. Consequently, the pavement designer must have realistic values for materials properties, traffic loads and temperature before he can conduct the analysis.

Materials testing technology in the pavement field has, for good reason, been largely built on a comparative basis, using index-type tests. Such index testing is useful for within comparison of materials but it is often inadequate for comparison between materials, especially when nonconventional materials are being considered. In addition, index-type tests do not provide the fundamental materials properties needed for structural analysis.

These fundamental properties may be evaluated in a number of different ways, both in the field and in the laboratory. Because field testing is usually time consuming and not always practical, laboratory methods have received considerable emphasis. However, even though a fundamental property is being evaluated, different types of tests can give widely different results. It follows then that the predicted structural response of the pavement can similarly vary widely, depending upon what test results are used.

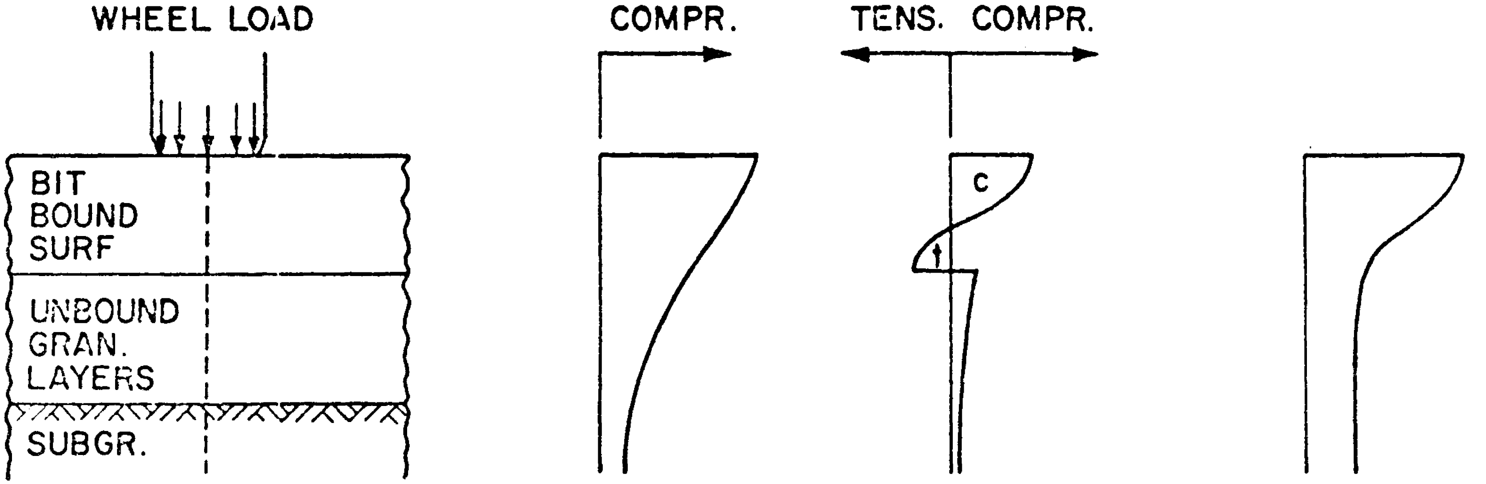
LAYER ANALYSIS OF PAVEMENTS AND BASIC MATERIALS INPUT REQUIREMENTS

STRUCTURAL RESPONSE OF A PAVEMENT TO TRAFFIC LOAD

A pavement that carries a traffic load will be stressed in the general manner shown in Figure 1. Maximum stresses occur under the center of the load. Figures 1(b) and 1(c) show these in terms of a vertical stress and a horizontal stress. When the load and pavement thickness are within certain ranges, the horizontal stress will be tensile in the bottom part of the bound layer. The distribution of temperature, as schematically shown in Figure 1(d), will affect the magnitude of these stresses.

Layered methods of structural analysis are used to calculate these stresses, given certain input information concerning traffic loads, materials properties and temperature distribution. As well, the strains or deflections in the directions of the stresses, or in any other desired directions, can be calculated. Stresses, strains and deflections can also be calculated at points away from under the center of the load, in any desired direction.

Figure 1(a) shows the traffic load in a single position. In reality, of course, the load is moving. Consequently, the stresses shown in Figures 1(b) and 1(c) can be considered as peak stresses which occur when the load is directly over the vertical dotted line shown in 1(a). When the load is approaching, or leaving, smaller vertical and horizontal stresses will occur along that line.



(a) PAVEMENT LAYERS

(b) DISTRIBUTION OF VERTICAL STRESS UNDER CENTER LINE OF WHEEL LOAD

(c) DISTRIBUTION OF HORIZ. STRESS UNDER CENTER LINE OF WHEEL LOAD

(d) TEMPERATURE DISTRIBUTION

FIGURE 1 - TYPICAL STRESS AND TEMPERATURE DISTRIBUTIONS UNDER A WHEEL LOAD

STRUCTURAL RESPONSE CALCULATIONS OF MAJOR INTEREST AND THEIR USES

The stresses of usual interest to the pavement designer - i.e., those which he can relate to observed pavement behavior or performance - are the vertical compressive stress at the top of the subgrade and the horizontal tensile stress at the bottom of the bound layer. Similarly, the strains of usual interest are the vertical compressive strain at the top of the subgrade and the horizontal tensile strain at the bottom of the bound layer. The deflection of usual interest is that at the surface of the pavement, which of course can be compared to actual field measurements.

The major use of horizontal stress or strain calculations at the bottom of the bound layer is for fatigue analyses. Vertical strain calculations at the top of the subgrade are mainly used in permanent deformation or rut depth analyses. Vertical compressive stresses on the subgrade, and deflection at the surface of the pavement, have been used by a number of investigators to relate to pavement performance (1).

BASIC MATERIALS PROPERTIES REQUIRED

The basic materials properties required as inputs for elastic or viscoelastic layer analysis of a pavement structure are as follows:

1. Modulus of each layer material, and the subgrade. For bituminous bound layers, the variation of modulus with temperature and rate of loading should be known.
2. Poisson's ratio of each layer material (i.e., the ratio of lateral displacement to vertical displacement of the material, under the particular test conditions).
3. Creep compliance and related properties; compliance characterizes the stress - strain - time relationships for materials at various temperatures.

The determination of these values for the various materials can be accomplished by a wide variety of testing methods, as subsequently discussed in this paper.

OPERATIONAL PROGRAMS FOR LAYER ANALYSIS

There are a number of operational programs available for layer analysis of pavements. They include the following:

1. BISTRO or BISAR (elastic layered program developed by Shell Oil Co.)
2. CHEVRON (elastic layered program developed by Chevron Co.)
3. FEPAVE II or FEPAVE IV (elastic layered program developed at the University of California, Berkeley).
4. VESYS IIM (viscoelastic and elastic layered program developed for the Federal Highway Administration).

CRITERIA FOR MATERIALS TESTS

Satisfactory design of pavements requires an understanding of the load-deformation-time relationship and the strength properties of the materials to be used. Strength represents a limiting condition. As such, it is not directly applicable to design because pavements are not expected to fail under a single application of load. The load-deformation-time characteristics can, however, relate to a single application or to many applications of load.

Materials which are actually used in pavements behave in a very complex manner and do not display completely elastic or viscoelastic properties. The load-deformation-time properties depend on the magnitude of the load, the rate of loading, temperature, and moisture content. However, because of the wide variety of materials, the complexity of behavior, and the difficulty of characterizing materials service behavior it is necessary to treat materials as though they have simple linear elastic or viscoelastic properties.

The type and extent of the testing program used to determine these characteristics relates to the following general criteria:

1. Ease of testing,
2. Reproducibility of test results,
3. Size of project and variability, and
4. Ability to estimate fundamental properties.

EASE OF TESTING

In contrast to research, ease of testing is one of the more important criteria to be applied to any proposed test method. Often an "imperfect" test method should be favoured because of its simplicity and the ability to conduct the test without costly equipment, extensive test time, or extensive training of personnel. Thus, a test which can be readily implemented and used in the field and by design agencies is desirable.

Simplicity and low cost should not, however, be the primary basis for selecting a given test or testing program. In comparison to the total cost of designing, constructing, and maintaining a pavement, the cost of the testing program usually is insignificant.

REPRODUCIBLE TEST RESULTS

A second criterion related to the choice of test is a small error associated with testing. A test method ideally should be able to reproduce test results for essentially identical specimens. One measure of this reproducibility is the coefficient of variation obtained from laboratory prepared and tested specimens of a given mixture. The variation obtained represents inherent variation of the mixture and specimen and testing error. Variation associated with testing and the specimen should be minimized.

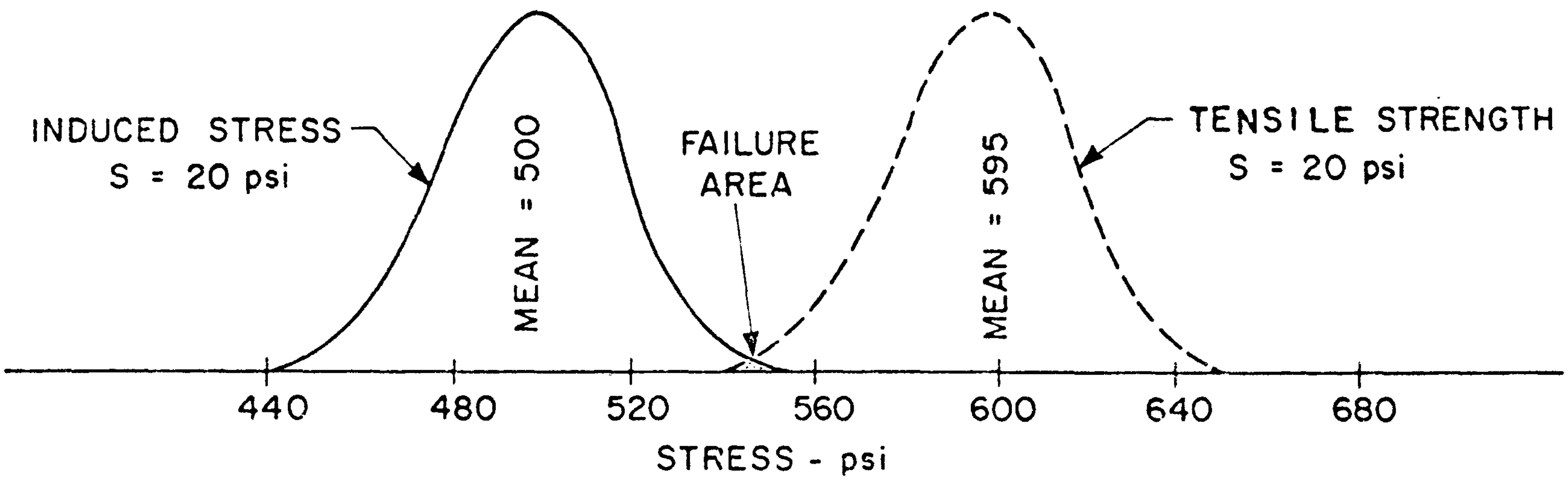
SIZE OF PROJECT AND VARIABILITY

The size and cost of the project and the inherent variability of the materials involved must be considered in establishing the type and extent of the materials testing program.

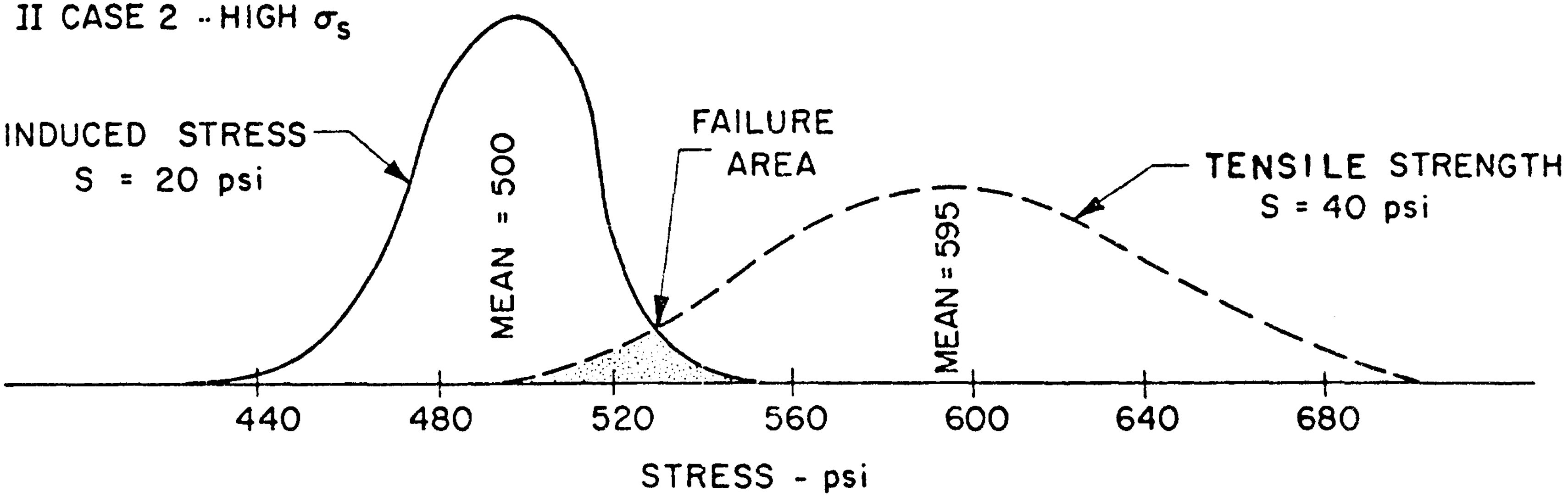
Materials variability must be quantified for meaningful design. It is obvious from even the most cursory evaluation of pavement performance and distress that variation is one of the most significant factors to be considered. If, for example, 10 percent of a pavement fails then the entire pavement has probably failed in terms of performance.

The concept of variability and its relationship to failure is illustrated in Figure 2. It shows the variations of tensile stress and tensile strengths for a hypothetical pavement. The area of overlap represents a failure condition in which stress exceeds strength. If the variation in material characteristics increases, the probability of failure increases. Similar examples could be shown involving other properties or a combination of these properties. Examples of the magnitude of such effects are shown in a subsequent part of this paper.

Closely related is the question of inherent variability and the extent of testing. It is ridiculous to conduct an elaborate and extensive testing program on a small sample of material which is quite variable. Such a program would yield a great deal of information which would be



II CASE 2 - HIGH σ_s



S = STANDARD DEVIATION

FIGURE 2 - GRAPHICAL PRESENTATION OF FAILURE FOR TWO CONDITIONS WITH THE SAME MEAN VALUES BUT DIFFERENT VARIABILITY

meaningful to only a very small portion of the pavement. Likewise, a very limited program would not provide useful information.

A realistic approach would involve the determination of average values, variation, and significant changes in material properties. For example, where a new pavement is to be constructed, or an existing pavement overlayed, significant changes in subgrade soil support should be identified because of their relationship to required design thickness. Of course project size and cost are important. As the size of a project increases, variability will also increase. At the same time as size or cost increases, the justification for a more extensive testing program increases. Thus, the extent and nature of the testing program ultimately relates to the variability expected, the cost of the project, and the consequences of failure.

FUNDAMENTAL PROPERTIES

The final criterion relates to the ability of the tests to measure the fundamental or basic properties previously mentioned. In terms of elastic design this means that modulus values (as derived from the load-deformation-time characteristics of the material) and Poisson's ratio need to be known. In viscoelastic design the basic properties involve creep compliance or a related property. Empirical test results are only of value to an empirical design procedure.

Attempts at using empirical tests to estimate fundamental properties through correlations should be rejected unless better information is not available or cannot be obtained. Such correlations are usually only very approximate.

CLASSIFICATION AND DESCRIPTION OF MATERIAL CHARACTERIZATION TECHNIQUES

Although the problem of materials characterization has been with us for many years and a great deal of work has been done, it would appear that there is very little agreement with regard to type of test and test procedures required. According to Deacon (2) this lack of agreement is explainable for the following reasons:

1. The variety of materials encountered by the designer is unlimited because of their nature and the manner in which they are manufactured.
2. The nature of the pavement structure in which these materials are used depends upon the intended function of the pavement.
3. During the service life of a pavement, material properties are altered by such factors as thixotropy, aging, curing, densification, change of moisture content, etc.
4. The response of a pavement material to load is extremely complex and is characterized by non-linear, inelastic, rate-dependent, anisotropic behaviour which is sensitive to temperature and moisture.
5. Solutions to the pertinent boundary value problems have been essentially non-existent until recently.
6. The approach to the problem, has been piecemeal at best and has involved many different researchers from many different agencies each striving for an optimal solution to a singular problem of limited scope and sometimes prejudiced intent.

It could be added that for the past 50 years or more pavement design agencies have always pressed for an immediate answer to their needs and problems. Long term, well thought out, sequential efforts have usually been rejected because of the time and expense involved.

Nevertheless, a wide variety of test methods and procedures have been developed over the years, some of which have been long forgotten, but many of which are still being used today. These test methods can be classified as field or laboratory tests, empirical or fundamental, or according to the mode of test (e.g., tension, compression, shear, flexural, torsion, or some indirect method relating empirical results to other tests or test parameters).

Empirical tests generally yield index properties related to fundamental materials properties such as strength and stiffness modulus. However, these index properties only have meaning on a comparative basis (i.e., previous tests on similar materials), or in terms of correlations with fundamental properties. An example of a widely used index test is the California Bearing Ratio (CBR) test.

Tests which yield fundamental properties directly are much more useful and a strong emphasis to use them has been apparent in recent years. Examples of such tests include the indirect tensile, triaxial, plate load, Dynaflect, and flexural tests.

For purposes of discussion these tests will be classified either as field or laboratory in nature. They may be listed as follows:

1. Field Tests
    (a) California Bearing Ratio (empirical),
    (b) Plate Load,
    (c) Benkleman beam, and
    (d) Dynaflect or other vibratory tests
2. Laboratory Tests - Elastic
    (a) Dynamic, complex modulus,

    (b) Resilient modulus,
    (c) Flexural stiffness,
    (d) Dynamic or static indirect tension, and
    (e) Stiffness by nomograph means

3. Laboratory Test - Viscoelastic
    (a) Creep compliance
    (b) Relaxation
    (c) Complex modulus

Empirical test methods, with the exception of the CBR test which is so widely used, will not be discussed.

With regard to the test considered, it must be recognised that variations in techniques exist and that there is a tendency based on history to use one test method for one material and another test for another material.

While this is not a testimonial for standardization solely for the sake of standardization, techniques quite often differ simply because previous work has not been adequately considered. As well, a conscious effort to be different or vested interest in the particular techniques and design procedure. may sometimes apply.

The following discussion briefly describes and summarizes the basic characteristics of the previously listed tests.

FIELD TESTS

Field tests basically can only be used to evaluate an existing condition. Thus, they can be used to evaluate the subgrade for a proposed pavement prior to design, an existing pavement to determine its basic structural condition, or an existing pavement to determine its support characteristics for use in the design of an overlay. Results generally must be considered in terms of the conditions which exist at the time that the tests are conducted.

California Bearing Ratio Test

The CBR Test is a load-deformation test (or more accurately a load-penetration test) which can be performed either in the laboratory or in the field. The test is conducted by forcing a small cylindrical piston, (3 in^2) end area, into a soil or other material. Load-penetration data is collected and the CBR value is computed by comparing the load required to produce a given penetration (normally 0.1 in.) to the load required to produce the same penetration for a standard material. Thus, the CBR expresses the quality of the material in terms of what was once considered to be an excellent base material. In essence, there is no way to evaluate the material except in terms of CBR values for previously tested materials which have been used in pavements. Values for materials, such as those which are stabilized (i.e., with high strengths), are meaningless.

Attempts have been made to arrive at more fundamental properties by establishing a correlation between CBR and the modulus of elasticity. An example is a simple correlation, developed by Heukelom and Foster (3), relating field-determined CBR values and stiffness values obtained using the Shell vibratory method. The relationship is

$$E = 1500 \text{ CBR}$$

where E = modulus of elasticity (psi)

CBR = California Bearing Ratio Value (percent)

Shook and Kallas (4) report that there is little evidence as to the accuracy of the method and, as previously discussed, it is doubtful that such a simple correlation can actually relate the two parameters. As well, it is subject to a great deal of scatter and the correlation is only valid for the conditions and range of values for which it was established. The use of any such correlation is discouraged except as a means of establishing a rough estimate.

Plate Load Test

A plate load test normally is conducted in the field and has the positive characteristics of being direct and easily understood. It has been used for many years by some agencies with the result that there are many variations in techniques. Essentially, the test consists of loading a circular plate of a given diameter and measuring the deflection of the surface upon which the plate is supported. Loads can be applied in a static or repeated way.

In the static test loads are applied in increments and held until essentially all deflection has occurred. When the time rate of deflection has reached an acceptable level, an additional increment of load is applied.

Static testing can be used to obtain the ultimate bearing capacity of a material, the modulus of subgrade reaction, or indirectly the modulus of elasticity of the material using a layered system analysis.

In the repeated tests, deflections are measured and after a given number of load applications, the load is increased. A typical load vs accumulated deflection relationship, from Reference 5 is shown in Figure 3.

The test can be conducted on natural soil, compacted subgrade soil, or on any exposed pavement layer; however, it is restricted to an evaluation

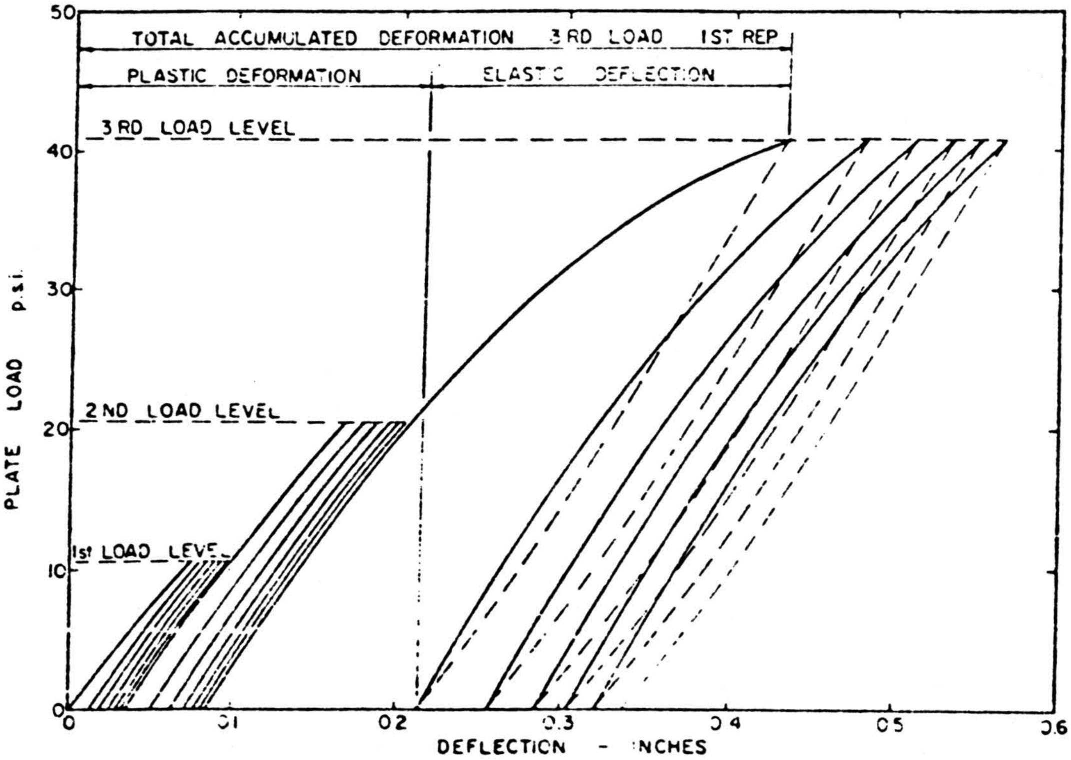


FIGURE 3 - TYPICAL LOAD-DEFLECTION DIAGRAM FOR REPETITIVE PLATE LOAD TESTING (AFTER REF.^{5})

of an existing in place material. Thus, its primary use would be to determine the load-deformation characteristics of a subgrade for use in the design of a pavement or the load-deformation characteristics of a pavement for use in the design of an overlay.

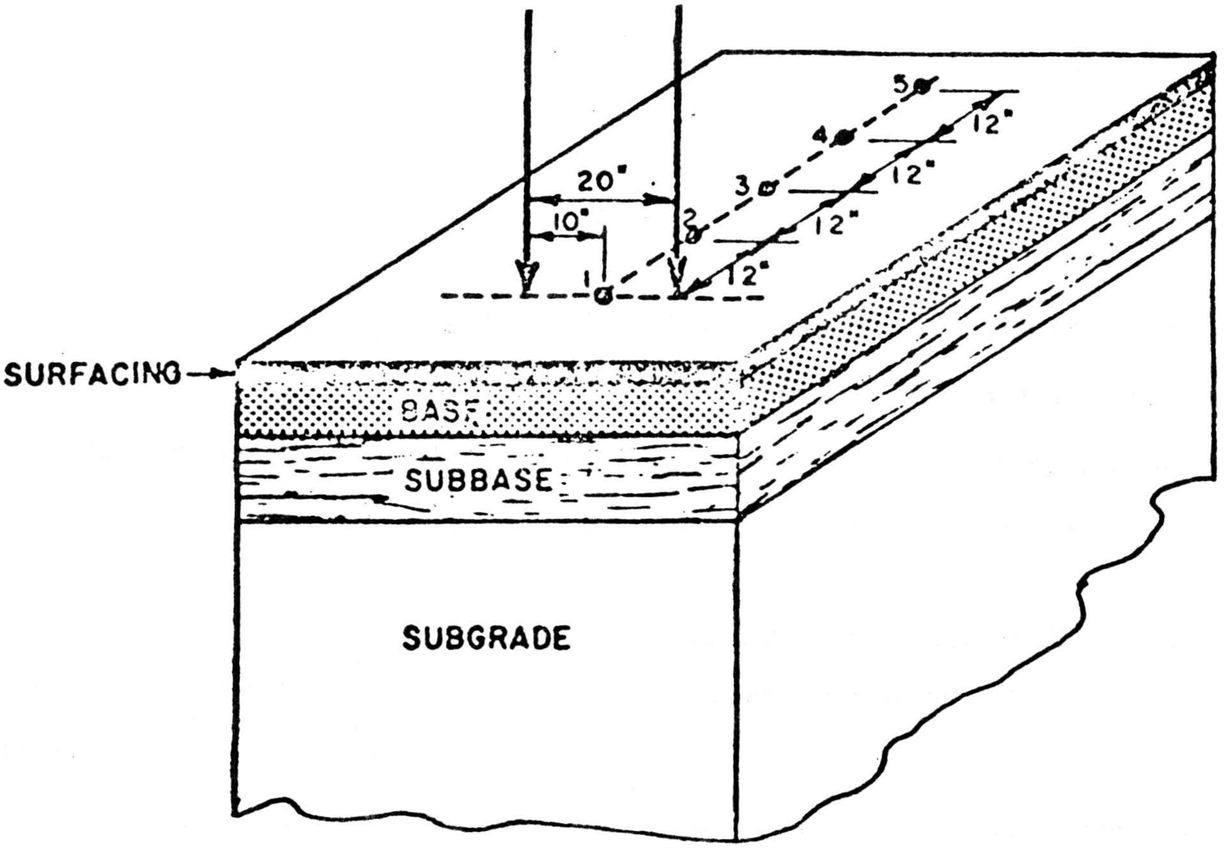
Dynaflect and Vibratory Methods

The Dynaflect System, which is somewhat related to the plate load test, consists of a dynamic force generator mounted on a two-wheel trailer, a control unit, a sensor assembly and a sensor (geophone) calibration unit. This system allows rapid and precise measurements of roadway deflections under a cyclic force of known magnitude.

The cyclic force generator consists of a pair of unbalanced fly wheels which rotate in opposite directions at 480 rpms to produce a cyclic vertical force of 1000 pounds. The resulting deflections are sensed at a series of points on the surface of the pavement as shown in Figure 4.

The deflections have been correlated with Benkelman Beam deflection values by many agencies. As well, the curvature of the deflected surface can be calculated and is used in some design procedures.

Non-destructive Vibratory testing consists of applying sinusoidal vibrations to pavement and analyzing the wave propagation resulting from these vibrations. The shape of the dispersion curve provides information related to the elastic characteristics of the material and the geometrics of the pavement. Many different types of equipment and techniques have been used but perhaps the best known has been developed by Shell and used for such applications as the Brampton Test Road (7).



The vertical arrows represent the load wheels and the points numbered 1 through 5 represent the position where sensors 1 through 5 pick up the motion of the pavement surface

FIGURE 4 - POSITION OF DYNAFLECT SENSOR AND LOAD WHEELS (AFTER REF. 6)

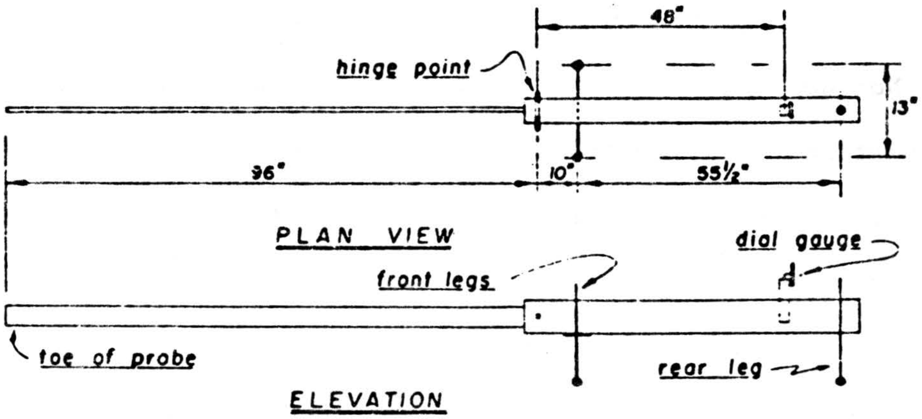


FIGURE 5 - LINE DIAGRAM SHOWING CRITICAL DIMENSIONS OF BENKLEMAN BEAM (AFTER REF. 8)

The modulus value is calculated as the ratio of stress to recoverable (resilient) strain under repeated loading conditions.

The major differences are that:

1. A confining pressure can be applied in the resilient modulus test.
2. Inelastic as well as elastic behavior can be measured in the complex modulus test.

A third stiffness test has been described by Deacon (2) in which a beam is subjected to repeated flexure. A flexural stiffness modulus is calculated from the center point deflections under load (not the recoverable deflection).

In addition, the indirect tensile test has been used, both dynamically and statically, to obtain estimates of modulus and other load-deformation characteristics.

Complex Modulus

Sinusoidal vertical loads are applied to 4-inch diameter by 8-inch high specimens and strains are measured. Typical load-strain-time relationships are shown in Figure 6, from Reference (4). Values of the complex modulus and phase lag are calculated using the following equations:

$$\left|E^*\right| = \frac{\sigma}{\varepsilon}$$

$$\text{and } \phi = \frac{t_i}{t_p} \quad (360^\circ)$$

where: $\left|E^*\right|$ = absolute value of the complex modulus, psi,

ϕ = phase lag, degrees

σ = amplitude of the sinusoidal vertical stress, psi (Figure 6),

ε = amplitude of resulting vertical strain

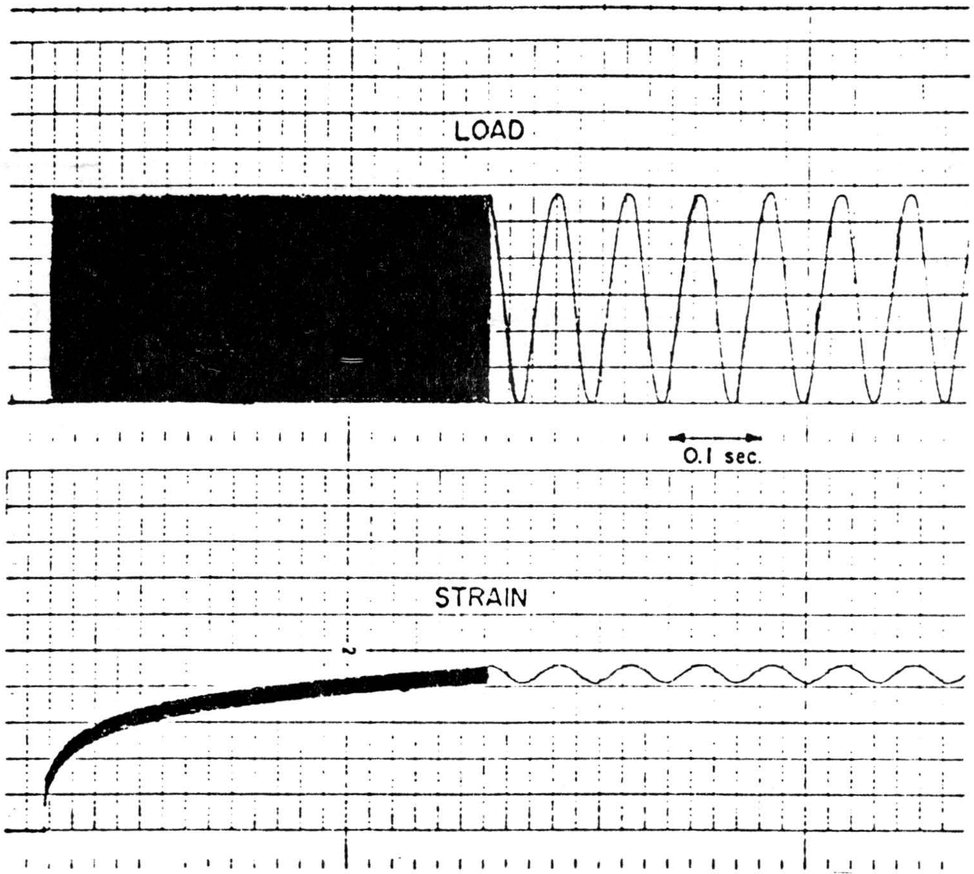


FIGURE 6 - RECOFDED TRACE FROM A DYNAMIC COMPLEX MODULUS TEST (AFTER REF. 4)

t_i = time lag between a cycle of stress and the resulting cycle of stress, seconds,

t_p = time for a cycle of stress, sec.

Typical values of complex modulus are shown in Table 1a. Estimates of Poisson's ratio can also be obtained from the complex modulus test by measuring strains perpendicular to the applied load. Typical values from Reference (2) are shown in Table 1b.

Resilient Modulus

Recommended procedures for the resilient modulus test for subgrade soils, untreated granular bases, and sub-bases are described in References(4, 11, 12).

A haversine wave load is applied for 0.1 sec. and removed for 0.4 sec. at a frequency of 120 loads per minute. The confining pressures vary from 0 to 50 psi, depending on the type of material. The resulting axial deformations are recorded. A typical load-deformation-time relationship for a test is shown in Figure 7. Normally for granular base and sub-base materials specimens are 6-inches in diameter and 12-inches high while for soils a 4-inch diameter and 8-inch high specimen is used.

The modulus of resilience M_R is calculated from the following equation:

$$M_R = \frac{\sigma_d}{\varepsilon_r}$$

where: M_R = modulus of resilient deformation, psi

σ_d = repeated deviator stress (stress difference),

ε_r = repeated recoverable strain

TABLE 1a

Mean Values of $|E^*|^1$ Averaged Over Frequency, After Ref. (4)

Pavement Course	Temperature (F)	Vertical Stress 17.5 psi	 35 psi	 70 psi
Asphalt	40	18.51	19.73	20.04
Concrete	70	6.62	7.09	7.36
Surface	100	1.63	1.68	1.87
Asphalt	40	22.74	22.60	22.56
Concrete	70	7.08	7.65	8.07
Base	100	1.45	1.79	1.45

[1] in psi x 10^5

TABLE 1b

Poisson's Ratio Determined by
Dynamic Complex Modulus Test Procedures, After Ref. (4)

Pavement Course	Temperature (F)	Poisson's Ratio Loading Frequency 1 cps	 4 cps	 14 cps
Asphalt	40		0.282	
Concrete	70	0.492	0.494	0.375
Surface	100		0.374	
Asphalt	40		0.362	
Concrete	70	0.470	0.445	0.366
Base	100		0.433	

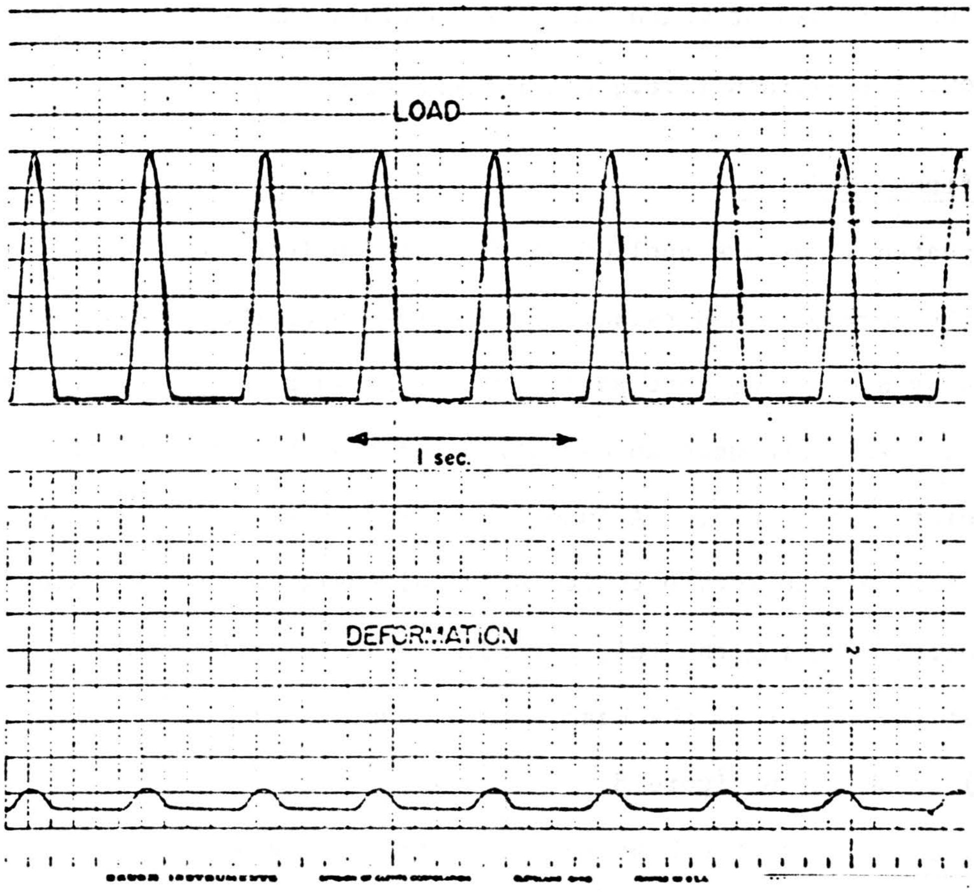


FIGURE 7 - RECORDED TRACE FROM A MODULUS OF RESILIENT DEFORMATION TEST
(AFTER REF. 4)

Values of M_R are determined after some number of repeated applications of the load at which time the specimen exhibits essentially constant recoverable strain (i.e., after "conditioning").

Flexural Stiffness Modulus

Repeated loads are applied at the third points of a beam specimen in the form of a haversine. The duration of the load is 0.1 sec. followed by a 0.4 sec. rest period resulting in a frequency of 120 loads per sec. This produces an essential constant bending moment over the center point of the beam. A load is applied in the opposite direction forcing the beam to return to its original position and maintaining it in that position during the rest period. The deflection under load is measured at the center of the beam. A schematic of the apparatus in Reference (4) is shown in Figure 8.

The stress and strain at the outer fibers and stiffness modulus after about 200 load applications are calculated from the following equations:

$$\sigma = \frac{3aP}{bh^2}$$

$$\epsilon = \frac{12hd}{3\ell^2-4a^2}$$

$$E_s = \frac{Pa(3\ell^2-4a^2)}{48Id}$$

where: σ = stress in the outer fibers, psi,

ϵ = strain in the outer fibers,

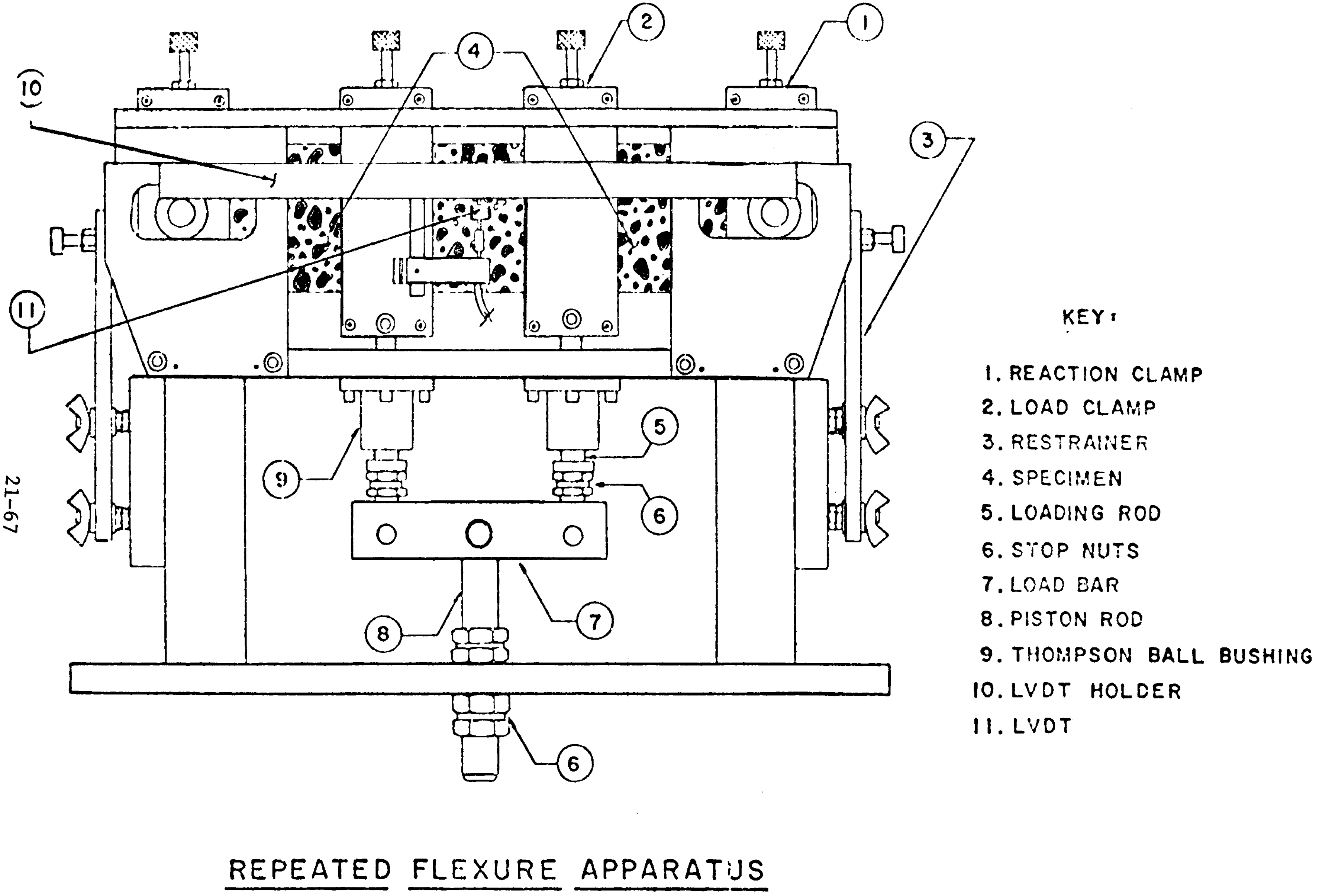


FIGURE 8 - DRAWING OF REPEATED FLEXURE APPARATUS (AFTER REF. 4)

E_s = flexural stiffness modulus, psi,

a = reaction span length -4 = 1/2 in.,

p = dynamic load applied at third points, lbs.,

b = specimen width, in.,

h = specimen depth, in.,

ℓ = reaction span length, in.,

I = moment of inertia of specimen, in^4,

and d = dynamic deflection of beam at the center, in.

Indirect Tension

The indirect tensile test is performed by loading a cylindrical specimen with a single or repeated compressive load which acts parallel to and along the vertical diametral plane (Figure 9). This loading configuration develops a relatively uniform tensile stress perpendicular to the direction of the applied load and along the vertical diametral plane, which ultimately causes the specimen to fail by splitting along the vertical diameter.

The development of stresses within the cylindrical specimen subjected to load is reported by Kennedy and Hudson (Refs. 13 and 14).

Most of the work in this area has been done at the University of Texas at Austin as a part of two research projects titled "Evaluation of Tensile Properties of Subbases for Use in New Rigid Pavement Design" and "Tensile Characterization of Highway Materials". The series of reports from the initial project cover the static and dynamic fatigue testing of stabilized materials. Reports from the second project are concerned with both the static and dynamic characteristics of pavement materials.

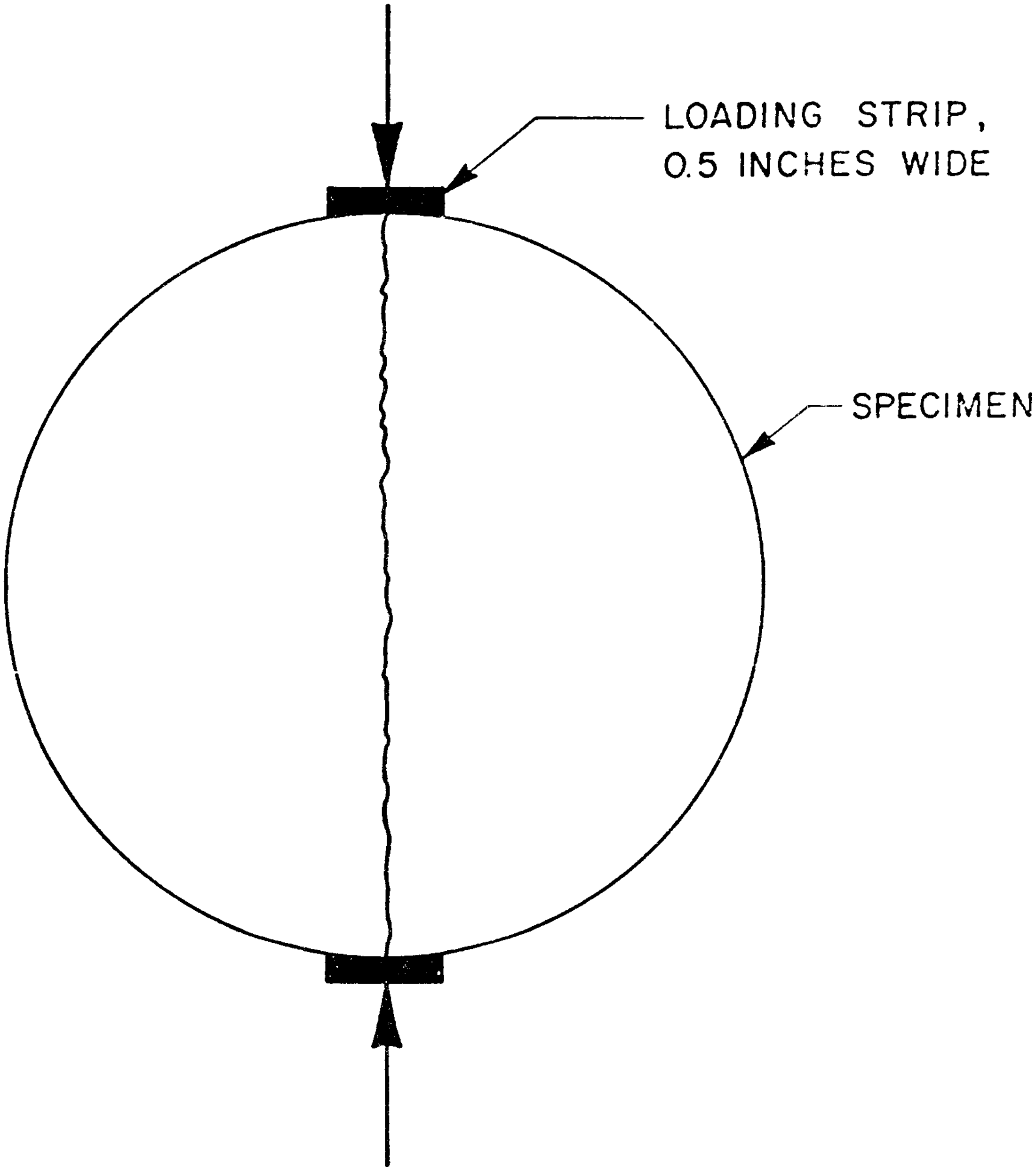


FIGURE 9 - THE STATIC OR DYNAMIC INDIRECT TENSILE TEST.

A 0.5-inch curved loading strip is used because the stress distributions are not altered significantly and because calculations of modulus of elasticity and Poisson's ratio are facilitated by maintaining a constant loading width rather than having a constantly changing loading width (which would occur with a flat load strip, as shown in Ref. (15) and (16)).

The development of equations that permitted the computation of the tensile strain at failure, the modulus of elasticity, and Poisson's ratio are reported in Refs. (15, 16 and 17). The equations are as follows:

$$S_T = \frac{2P_{Fail}}{\pi ah} (\sin 2\alpha - \frac{a}{2R})$$

$$E = \frac{P}{X}\left[\int_{-r}^{+r} \frac{\sigma_{rx}}{P} - \nu \int_{-r}^{+r} \frac{\sigma_{\theta x}}{P}\right]$$

$$\nu = \frac{\left[\int_{-r}^{r} \sigma_{ry} + R\int_{-r}^{+r} \sigma_{rx}\right]}{\left[R' \;{}_{-r}^{\;r}\, \sigma_{\theta x} + {}_{-r}^{\;r}\, \sigma_{\theta y}\right]}$$

$$\epsilon_T = \frac{X}{\ell} \frac{\left[\int_{-\frac{\ell}{2}}^{+\frac{\ell}{2}} \frac{\sigma_{rx}}{P} \quad \nu\int_{-\frac{\ell}{2}}^{+\frac{\ell}{2}} \frac{\sigma_{\theta x}}{P}\right]}{\left[\int_{-r}^{+r} \frac{\sigma_{rx}}{P} \quad \nu\int_{-r}^{+r} \frac{\sigma_{\theta x}}{P}\right]}$$

where: E = modulus of elasticity

S_T = tensile strength

P_{Fail} = total load at failure

P = load

a = width of loading strip

h = height of specimen

α = angle (radians) subtended by one-half the width of the loading strip

$\frac{P}{X}$ = least squares line of best fit between load P and total horizontal deformation X, for loads up to 50 percent of the maximum load

X = total horizontal deformation at any given load

ℓ = length over which strain is estimated

ν = Poisson's ratio

R = radius of specimen

ε_T = tensile strain at any given load

$R' = \frac{Y}{X}$ = least squares line of best fit between vertical deformations Y and the corresponding horizontal deformation X

$\int \frac{\sigma_x}{P}$, $\int \frac{\sigma_{\theta x}}{P}$, $\int \frac{\sigma_{ry}}{P}$, and $\int \frac{\sigma_{\theta y}}{P}$ = integrals of unit stresses

T = total tensile strain at failure

$\int_{-r}^{r} \sigma_{ry}$ and $\int_{-r}^{r} \sigma_{rx}$ = integrals of radial stresses in the y and x directions respectively

$\int_{-r}^{r} \sigma_{\theta x}$ $\int_{-r}^{r} \sigma_{\theta y}$ = integrals of radial stresses in the x and y directions respectively

$\int_{-r}^{+r} \frac{\sigma_{rx}}{P}$ $\int_{-r}^{+r} \frac{\sigma_{\theta x}}{P}$ = integrals of unit stresses σ_{rx} and $\sigma_{\theta x}$.

These equations require that the integrations be carried out using a computer and a computer program MODELAS. However, for a given diameter and width of loading strip these equations can be simplified and used without the aid of a computer. Table 2 presents these equations for 4- and 6-inch diameter specimens and a 0.5-inch loading strip (17).

In the static test method a cylindrical specimen is loaded on generators at the top and bottom of the specimen at a relatively slow rate (generally 2 in. per min.). The temperature used is 75°, although other temperatures can be used to characterize behavior if desired. A special transducer is used to measure the total horizontal (tensile) deformation while the vertical deformations are measured using an LVDT.

In the dynamic or repeated load indirect tensile test method the same equations are used except that it is not necessary to characterize the entire load-deformation relationships. A typical load pulse for the repeated load test is shown in Figure 10. The resulting load-deformation relationships are shown in Figures 11 and 12. A complex or a resilient indirect tensile modulus can be obtained by measuring the total vertical and horizontal deformation occurring under the applied load or the recoverable vertical and horizontal deformation and assuming a linear relationship between load and deformation (Figure 11). In addition, this method also provides an estimate of permanent deformation which occurs under repeated loads (Figure 12). Any level of stress less than the static strength can be used and applied in the form of a haversine (Figure 10).

Work using the dynamic indirect tensile test has been conducted both by Kennedy at the University of Texas at Austin and Schmidt at Chevron Oil Corporation, California.

STIFFNESS MODULUS BY INDIRECT MEANS

One of the most widely used methods of determining stiffness

TABLE 2. EQUATIONS FOR CALCULATION OF TENSILE PROPERTIES,AFTER REF. (17)

	Diameter of Specimen	
Tensile Property	4-Inch	6-Inch
Tensile strength S_T, psi	$0.156 \frac{P_{Fail}}{h}$	$0.105 \frac{P_{Fail}}{h}$
Poisson's ratio ν	$\frac{0.0673DR - 0.8954}{-0.2494DR - 0.0156}$	$\frac{0.04524DR - 0.6804}{-0.16648DR - 0.00694}$
Modulus of elasticity E, psi	$\frac{S_H}{h} [0.9976\nu + 0.2692]$	$\frac{S_H}{h} [0.9990\nu + 0.2712]$
Total tensile strain at failure ϵ_T	$X_{TF} \left[\frac{0.1185\nu + 0.03896}{0.2494\nu + 0.0673} \right]$	$X_{TF} \left[\frac{0.0793\nu + 0.0263}{0.1665\nu + 0.0452} \right]$

P_{Fail} = total load at failure (maximum load P_{max} or load at first break point) in pounds;

h = height of specimen, in inches;

X_{TF} = total horizontal deformation at failure (deformation at the maximum load or at first break point), in inches;

DR = deformation ratio $\frac{Y_T}{X_T}$ (the slope of line of best fit* between vertical deformation Y_T and the corresponding horizontal deformation X_T up to failure load P_{Fail});

S_H = horizontal tangent modulus $\frac{P}{X_T}$ (the slope of the line of best fit* between load P and total horizontal deformation X_T for loads up to failure load P_{Fail}).

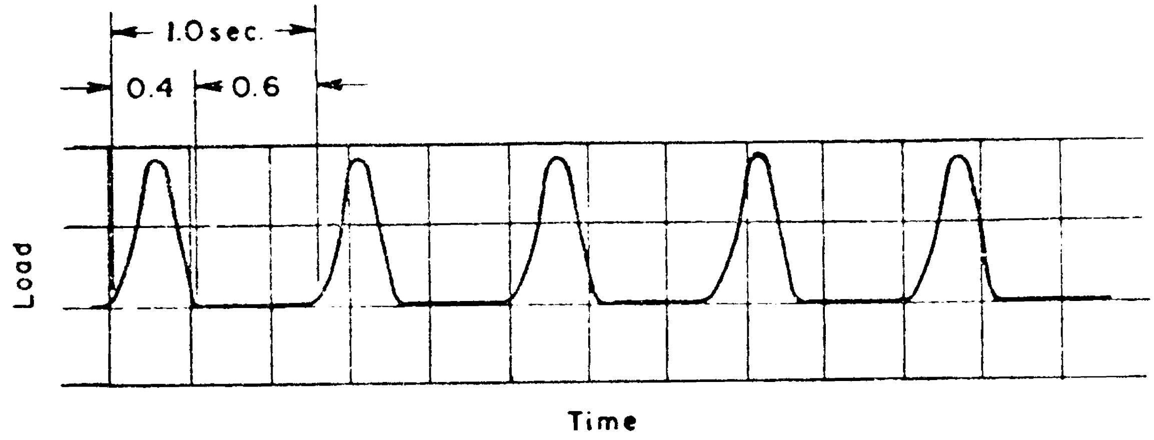


FIGURE 10 - LOAD - TIME PULSE FOR REPEATED LOAD INDIRECT TENSILE TEST

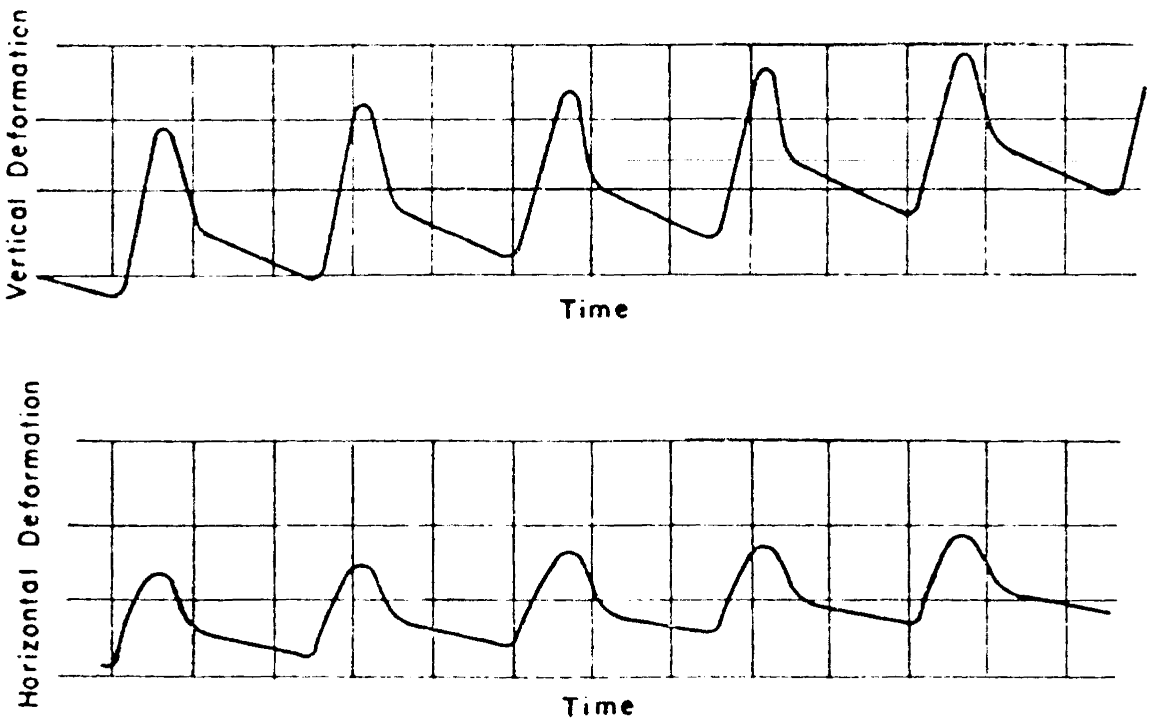


FIGURE 11 - DEFORMATION vs TIME RELATIONSHIPS FOR REPEATED LOADING INDIRECT TENSILE TEST

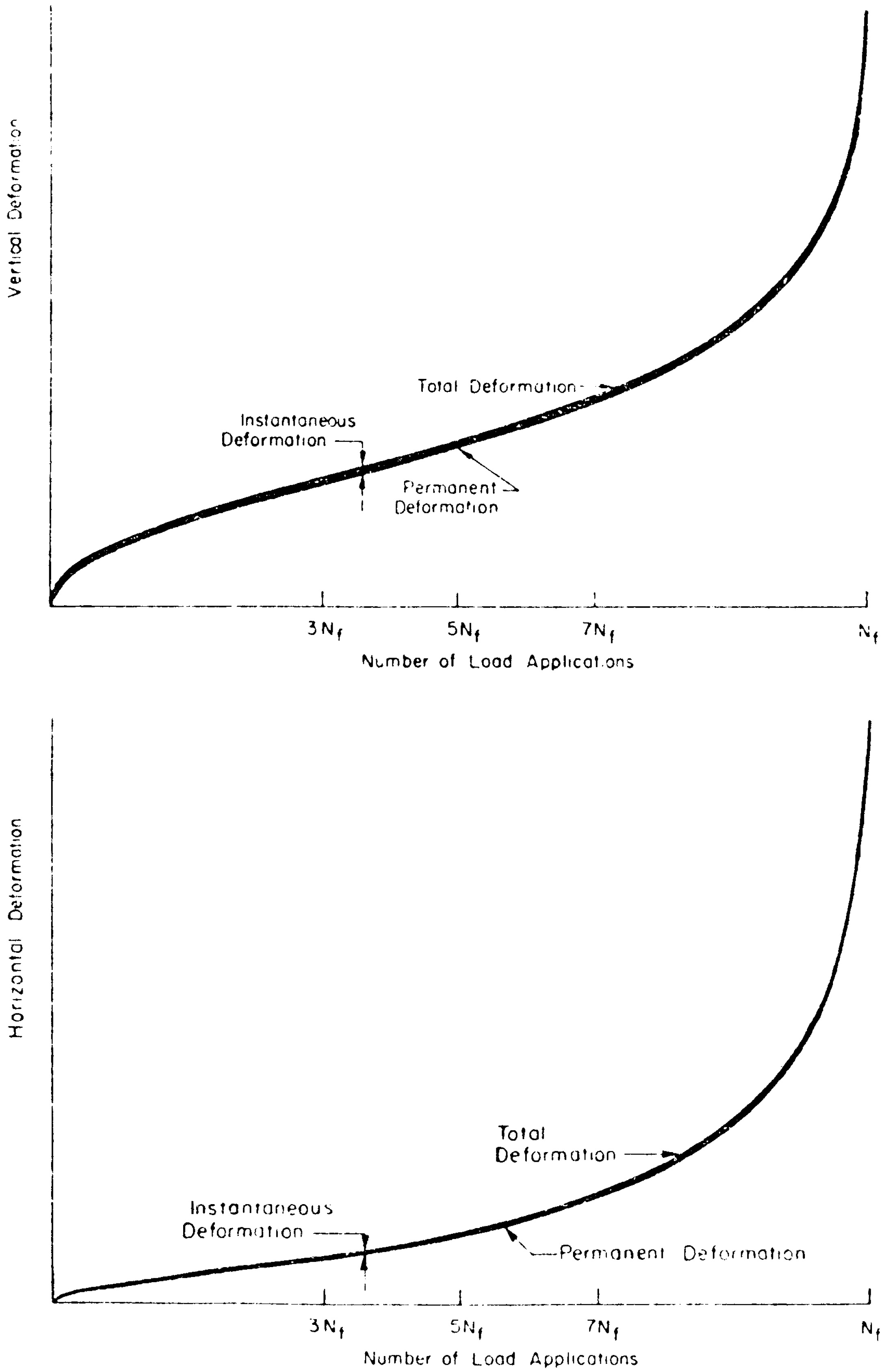


FIGURE 12 - TYPICAL RELATIONSHIPS BETWEEN DEFORMATION AND NUMBER OF REPEATED LOAD APPLICATIONS FOR THE INDIRECT TENSILE TEST

modulus of conventional bitumens and bituminous mixtures was originally developed by van der Poel (19). Stiffness modulus is defined as

$$S(t,T) = \frac{\sigma}{\varepsilon}$$

where S(t,T) = stiffness modulus, usually as psi or Kg/cm^2, of the material for a particular time of loading, t, and for a particular temperature, T

σ = stress at t and T, and

ε = unit strain at t and t

The procedure involved has been described in numerous references. Ref.(20) contains a complete description, including the modifications developed by Heukelom and McLeod. It is quite quick and simple to use, employing firstly values of penetration and softening point to derive bitumen stiffness, for the desired time and temperature; then, the bitumen stiffness is used to derive the mix stiffness.

It should be emphasized though, that because correlations are involved with index properties, the derived stiffness value of the mix may be quite approximate. Such limitations are fully discussed in Ref.(20).

LABORATORY TESTS - VISCOELASTIC

Viscoelastic tests are used primarily to evaluate asphalt-treated materials, clays and silty soils, and, to a lesser extent, granular subbase materials. The only current pavement design procedure utilizing viscoelasticity is VESYS IIM which was developed for the Federal Highway Administration (Refs. 22, 23 and 24) and has been described by Kenis and McMahon (Refs. 25-28). VESYS IIM involves a computer program in which materials properties are expressed in terms of both elastic and viscoelastic (creep) functions determined from laboratory tests.

The elastic properties can be determined from the previously discussed elastic tests; however, the viscoelastic characteristics must be determined from other tests. These viscoelastic properties are expressed in terms of creep compliance. Although VESYS IIM is still in the process of being evaluated, the tests and techniques for establishing creep compliance values and other related properties should be reviewed.

Creep Compliance

Normally creep compliance is determined by applying a constant axial load to a specimen and measuring the time-dependent deformation which occurs. Creep compliance J_t is then calculated by dividing the strain by the applied stress as follows:

$$J_t = \frac{\varepsilon_t}{\sigma} \quad \text{at any test temperature } T$$

ε_t = strain at time t

σ = applied stress, psi

A typical deformation-time relationship and creep compliance-time relationship is shown in Figures 13 and 14. For asphalt-treated materials and clay soils the test usually does not involve a confining pressure; however, for granular materials the specimen must be confined.

Since the creep behavior is dependent on the loading history of

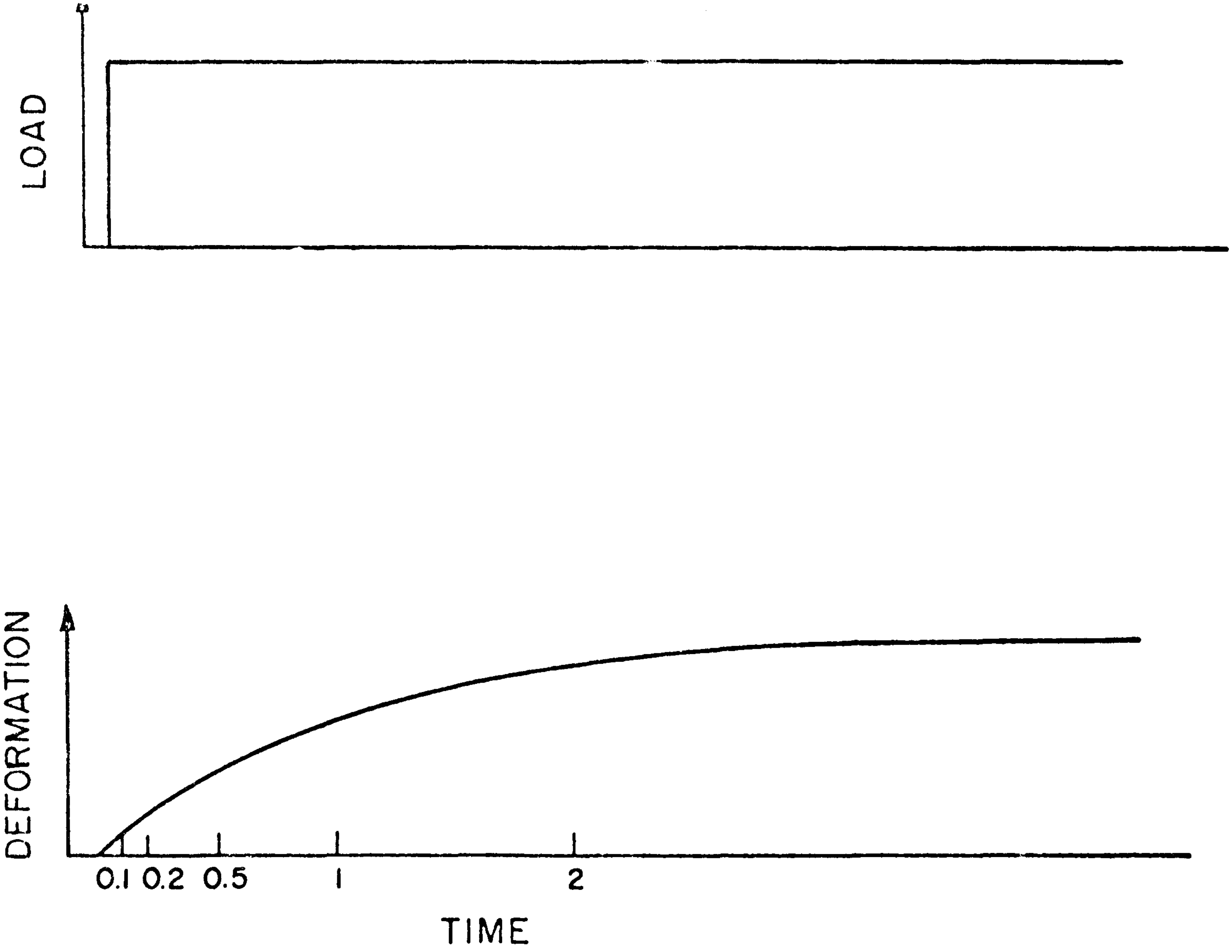


FIGURE 13 - TYPICAL CREEP DEFORMATION - TIME RELATIONSHIP FOR ASPHALT CONCRETE

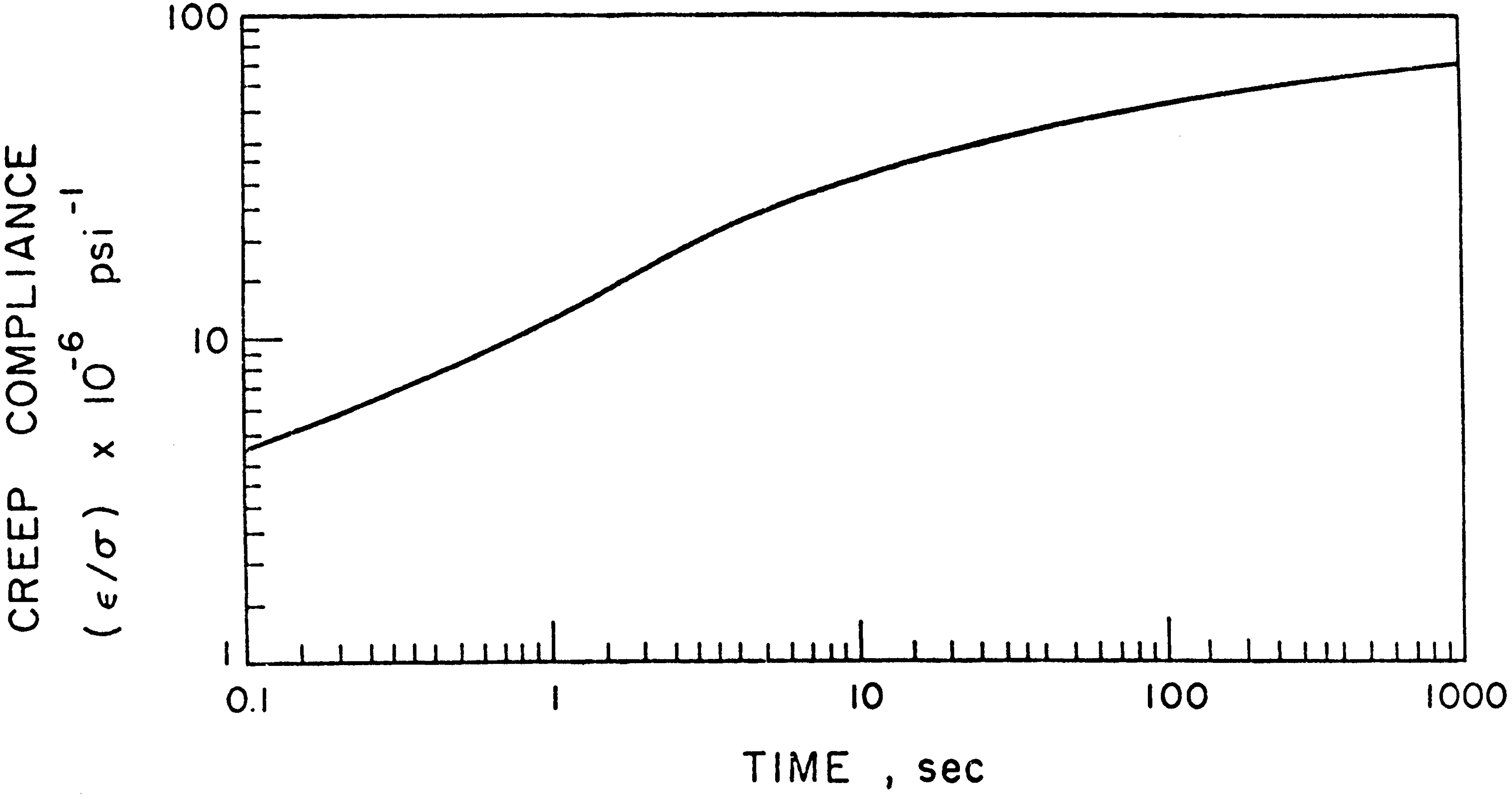


FIGURE 14 - TYPICAL CREEP COMPLIANCE - TIME RELATIONSHIP FOR ASPHALT CONCRETE

the test specimen, asphalt specimens are preconditioned by three cycles of the test load with load held constant for ten minutes. The specimen is then loaded for a 20-minute period during which deformation measurements are recorded.

The test load is selected so that the specimen will not exhibit more than one percent deformation to assure that the linear range is not exceeded. The test temperature must be held constant through the test; however, the tests should be conducted over a range of temperatures. The resulting creep data can then be extended to longer time periods by means of the time-temperature superposition concept.

Time-Temperature Superposition

Since it is not practical to test over the complete time range involved in design, the principal of time-temperature superposition can be used to extend the test data to much longer periods of time.

The creep data for each test emperature are plotted versus time on logarithmic paper (Figure 15). The horizontal distance (time) required to superimpose the curves at a master temperature is graphically determined. This distance is called the shift factor α_T. Shift factors for typical mixes are shown in Figure 16.

Relaxation

Relaxation tests are not used in design methods currently in use or being developed (Reference 4). This type of test, however, may be utilized in the future. The test is similar to a creep test except that the deformation is maintained constant and the load is allowed to decrease with time.

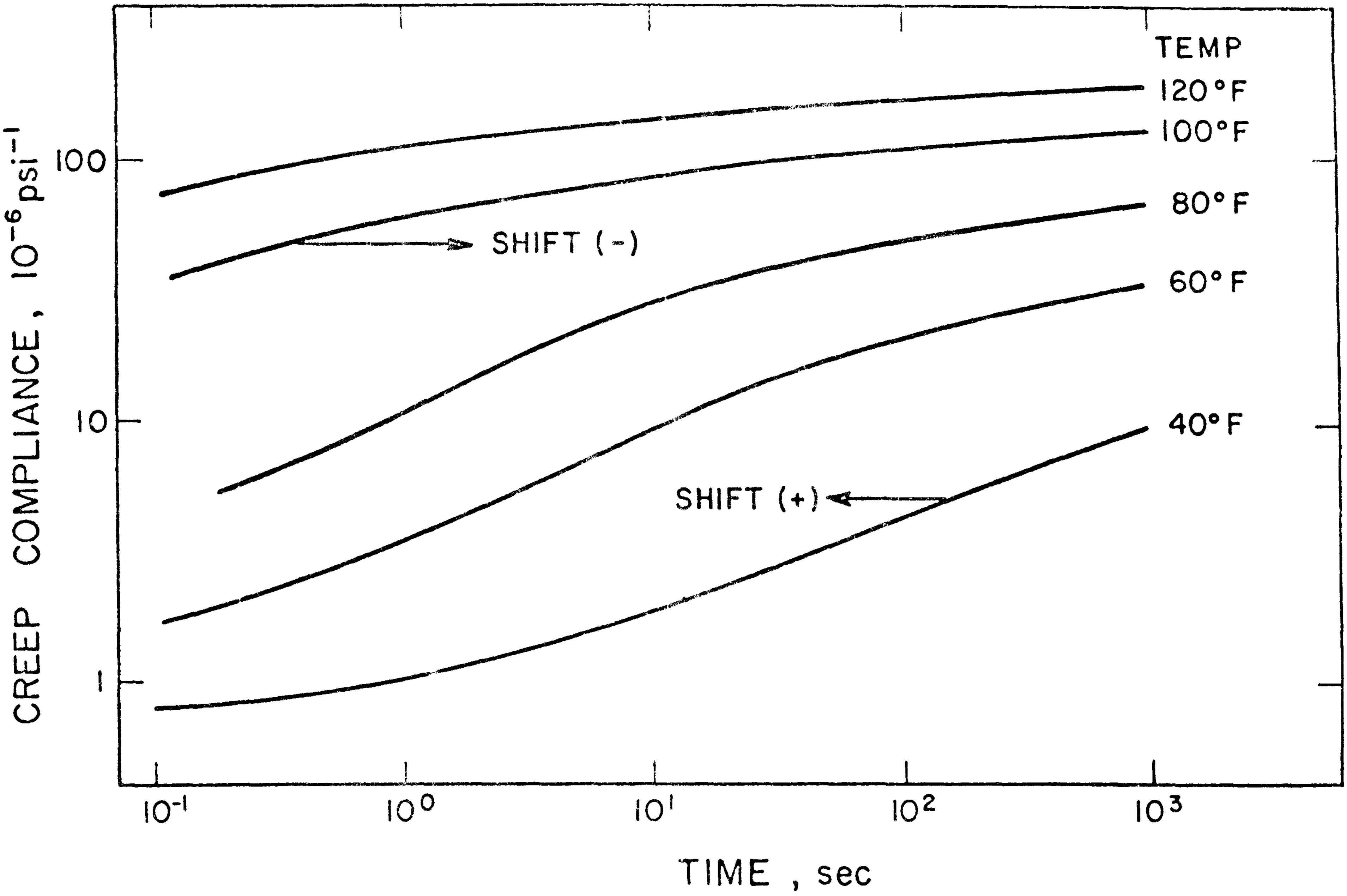


FIGURE 15 - TIME - TEMPERATURE SUPERPOSITION (REF.4)

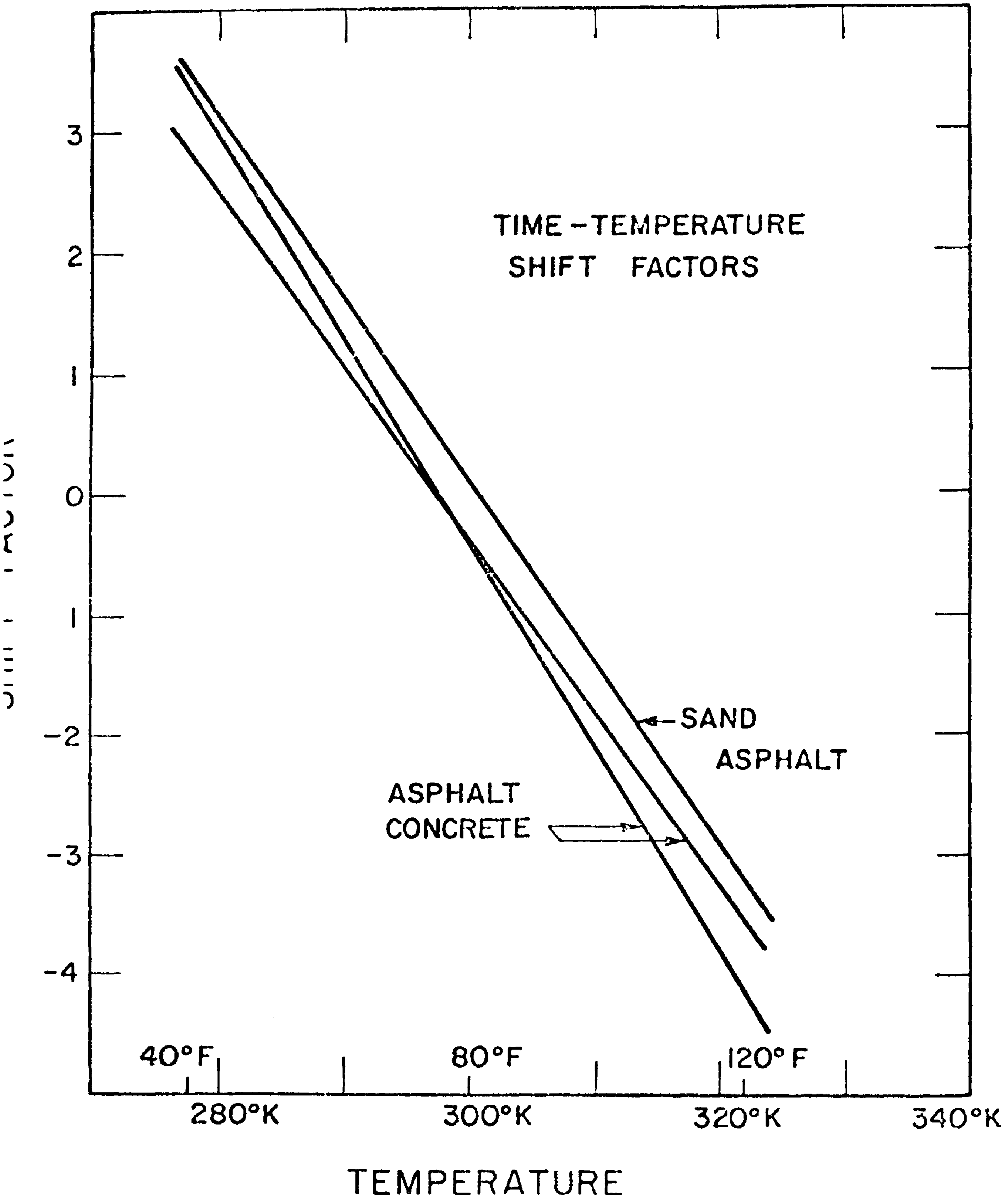


FIGURE 16 - SHIFT FACTORS FOR ASPHALT CONCRETE (REF. .)

Complex Modulus Tests

The complex modulus test has been previously discussed as a method of obtaining elastic moduli. The absolute value of the complex modulus $|E^*|$ is a measure of the elastic stiffness. The phase lag ϕ is a measure of the viscous response. However, additional work is needed if the phase lag ϕ is to be used in viscoelastic analyses.

EFFECTS OF VARIATION

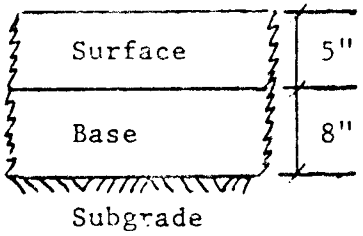
The foregoing sections have described briefly the more common procedures available for determining materials properties to use in layer analysis of a pavement. Results of such types of analysis have been related to observed pavement behavior and performance by a number of investigators, including Ref. (1, 18), and tentative design relationships have been suggested.

However, significant variations in the input variables to the analysis can similarly result in significant variations in the design thicknesses selected, or in the number of loads that can be carried by a particular design thickness. In the case of materials characteristics, such variations can be large, due to lack of uniformity, due to error in the test method, or due to differences between tests.

The effects of such typical variation are considered in an example using an elastic analysis (Figure 17). A factorial arrangement of high and low values for subgrade, base and surface moduli is shown. The values selected are representative of the range that might occur on a typical pavement section.

A layer analysis program known as BISAR (developed by the Shell

Surface, s	Hi (200,000)		Low (100,000)	
Base, b	Hi (60,000)	Low (30,000)	Hi (60,000)	Low (30,000)
Subgrade, S				
Hi (30,000)	1 380,000	2 360,000	3 450,000	4 410,000
Low (15,000)	5 350,000	6 310,000	7 420,000	8 380,000



Note: 1. Numbers in parentheses ( ) are moduli values in psi
2. Numbers in squares are the equivalent 18 kip axle loads the pavement can carry to the end of its service life for the particular combination.
3. Pavement structural section considered for evaluation shown at right

FIGURE 17. EXAMPLE OF A FACTORIAL ARRANGEMENT FOR EVALUATING THE EFFECTS OF MATERIAL VARIATION ON THE NUMBER OF EQUIVALENT 18 KIP AXLE LOADS CARRIED TO THE END OF THE SERVICE LIFE OF THE PAVEMENT.

Oil Co. Ltd.) was employed to calculate the vertical stresses on the subgrade, using the following inputs:

- Moduli values as shown in Figure 17
- Layer thicknesses as shown in Figure 17
- Wheel load 9,000 lbs. and tire pressure 70 psi,
- Radius of loaded area 6.4 in.
- Poisson's ratio of 0.38, 0.42 and 0.46 for the surface, base and subgrade materials respectively.

The values in the squares in Figure 17 show the number of equivalent 18 Kip single axle load equivalents that the pavement section shown could carry during its service life, for the particular moduli combination. Suggested design curves in Ref. (1) were used for the calculations.

The range of 18 Kip equivalent loads in Figure 17 is from 310,000 for combination 6, to 450,000 for combination 3. This could represent several years in the life of the pavement.

It is important to note, as illustrated by Figure 17 that an effect of a high or low modulus (or other property) of a material should not be considered in isolation. That is, its combined effect with the properties of the other materials should be considered for a complete assessment. For example, in Figure 17 a high surface modulus combined with a low base modulus and a low subgrade modulus (combination 6) give the worst condition. A low surface modulus, combined with the same low subgrade and base moduli gives a substantial improvement (i.e., combination 8). These examples should not be taken as universally applicable generalizations, but they serve to illustrate the importance of considering all possibilities for a given situation.

CONCLUSIONS

The major points of this paper may be summarized as follows:

1. Pavement design technology is making increased use of structural layer analysis techniques. These require as inputs materials properties of a fundamental nature.
2. There are a wide variety of techniques available for measuring these fundamental properties. The paper categorizes them, briefly describes the more common ones that are used and suggests a number of criteria for their application.
3. Variation in materials properties, which can occur due to non-uniformity, errors or differences in testing technique, can significantly affect the number of loads that a pavement can carry during its service life. The paper presents some examples of the effects of such variation.

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LESSON OUTLINE
GENERATING ALTERNATIVE PAVEMENT DESIGN STRATEGIES

Instructional Objectives

1. To introduce the student with the design strategy concept as opposed to the traditional design approach.

2. To explain the different design options when generating alternative pavement design strategies.

3. To present sample computer based algorithm used to generate pavement design strategies in working design systems.

Performance Objectives

1. The student should be able to explain the concept of design strategy.

2. The student should be able to generate pavement design strategies which consist of combinations of the different design options.

Abbreviated Outline	Time Allocations, min.
1. Introduction	10
2. Structural Options	5
3. Policy Alternatives	20
4. Automation in Generating Alternative Design Strategies	15
	50 minutes

Reading Assignments

1. Haas & Hudson - Chapter 15

2. Principles & Practices/ Course Notes

GENERATING ALTERNATIVE PAVEMENT DESIGN STRATEGIES

1.0 INTRODUCTION

1.1 Design Strategy Concept (Visual Aid 22.1 and 22.2)

Pavement Type selection should be based on an engineering and economic analysis. Most past design practice has been concerned with layer thickness selection. A more comprehensive concept is required, which includes consideration of materials types and expected policies of construction, maintenance, and rehabilitation throughout the design period.

1.2 Design Options (Visual Aid 22.3)

Therefore, when designing a pavement, the following options should be considered:

1.2.1 Structural Options.

(a) materials, and

(b) thicknesses.

1.2.2 Policies.

(a) construction,

(b) maintenance, and

(c) rehabilitation.

1.2.3 Design Period. A general guideline for selecting the length of design period is that it should not extend beyond the period of reliable forecasts. For traffic, 20 years is often used as an upper limit. Most transportation studies use a range of 20 to 30 years, and this seems reasonable for pavements.

1.3 Generation of Alternative Pavement Strategies

By combining the different design options, a set of alternative designs are generated from which the best design can be selected.

2.0 STRUCTURAL OPTIONS

The selection of materials and thickness for each of the layers in a pavement structure is limited by local availability and practical considerations; however, the possible combinations of structural options render it essential to use automated solutions.

2.1 Materials

Each material type has different mechanical properties and cost than the others. Different materials should be considered for each of the various pavement layers; for instance, asphalt and PC concrete, or more than one asphalt cement may be considered for the surface layer.

2.2 Thickness

The material types can be combined with incremental thickness to further multiply the number of possible solutions for a design problem.

Visual Aid 22.4 is an example schematic representation of the range of initial possible structural section alternatives for a pavement design problem.

3.0 POLICY ALTERNATIVES (VISUAL AID 22.5)

Different materials and thicknesses are obvious design options; however, without including policies, there may be appreciable error in selecting an optimum design.

3.1 Construction

Success of a design strategy in performing as expected is largely dependent on the construction policies to build it. Policies that might be considered include the following:

(a) tolerances for thicknesses, materials properties, initial roughness, etc.,

(b) traffic handling method,

(c) time of day, and season, for construction operations, and

(d) materials sources.

3.2 Maintenance

It is not yet possible to consider adequately alternative levels of maintenance in terms of their cost and benefit effects on a design strategy. Nevertheless, the designer should indicate policies and costs expected for the recommended strategy.

3.3 Rehabilitation (Visual Aid 22.6)

Overlays and seal coats are the most common type of rehabilitation; either type, or both, may be applied up to several times during the design period.

3.3.1 Structural Aspects. A large number of structural combinations and timing can exist.

3.3.2 Policies. Similar to construction policies. They are very important because of their effect in extra users cost. (Visual Aid 22.7).

4.0 AUTOMATION IN GENERATING ALTERNATIVE DESIGN STRATEGIES

4.1 Solution Algorithms

The computer costs and storage requirements involved in generating, analyzing, and evaluating several hundred strategies for one design problem can be significant. Therefore, existing design programs make use of solution algorithms to eliminate infeasible strategies.

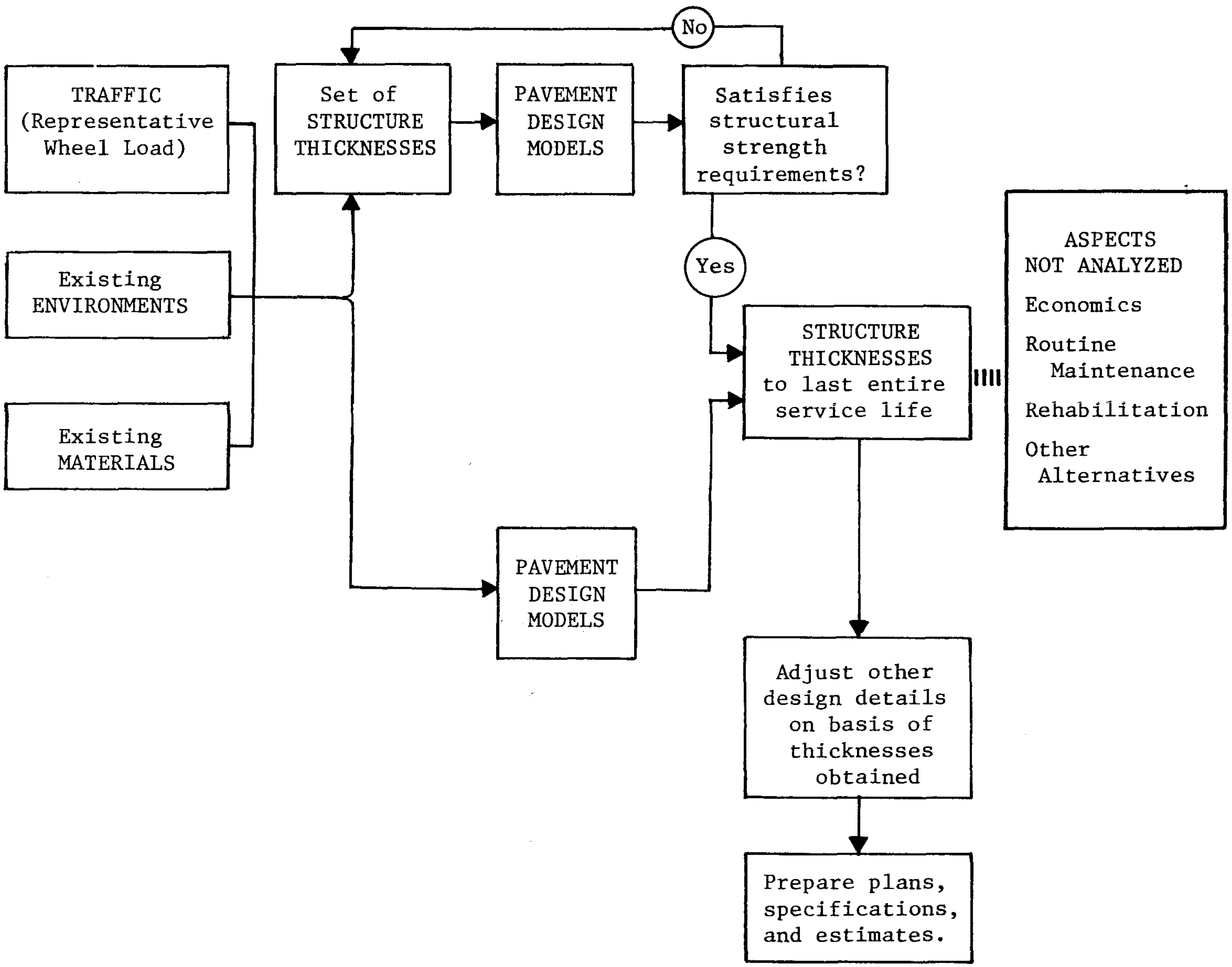
4.2 Sample Computer Programs

4.2.1 Flexible Pavement Design System (FPS). This computer program generates alternate flexible pavement design strategies and prints out the optimal from an economics standpoint. Visual Aid 22.8 indicates the process of generating design strategies built into FPS.

4.2.2 Rigid Pavement Design System (RPS). This program generates alternate rigid pavement design strategies and prints out the optimal from an economics standpoint. Visual Aid 22.9 indicates the process of generating designs in RPS.

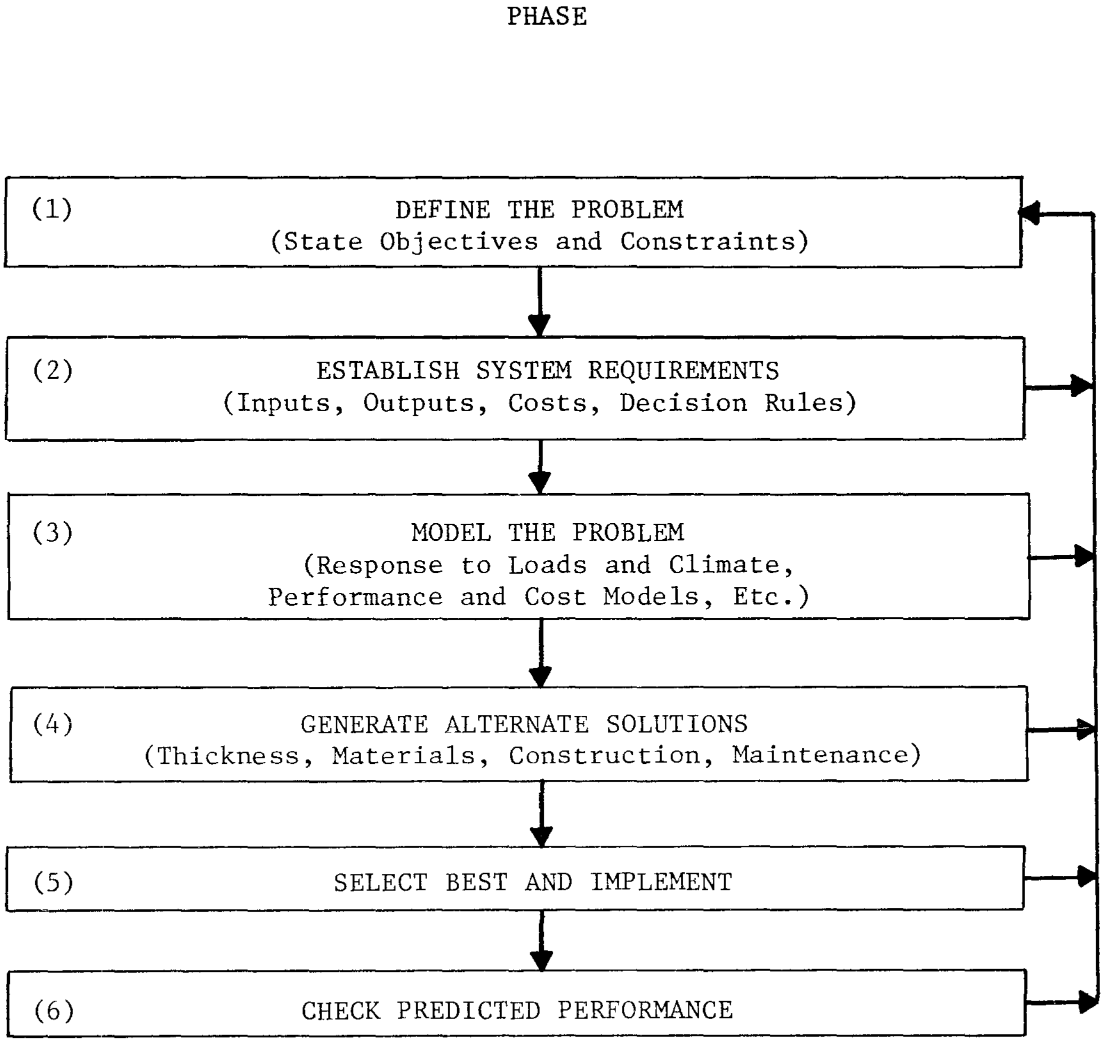
LESSON OUTLINE
GENERATING ALTERNATIVE PAVEMENT DESIGN STRATEGIES

VISUAL AID	TITLE
Visual Aid 22.1.	Schematic diagram of post pavement.
Visual Aid 22.2.	Major phases of the systems analysis methods.
Visual Aid 22.3.	Key components of generating alternative strategies.
Visual Aid 22.4.	Range of initial possible structural section alternatives.
Visual Aid 22.5.	Effect of policies on user costs.
Visual Aid 22.6.	Illustrative performance patterns of overlays.
Visual Aid 22.7.	Traffic handling methods.
Visual Aid 22.8.	Process of generating designs of FPS.
Visual Aid 22.9.	Process of generating designs of RPS.

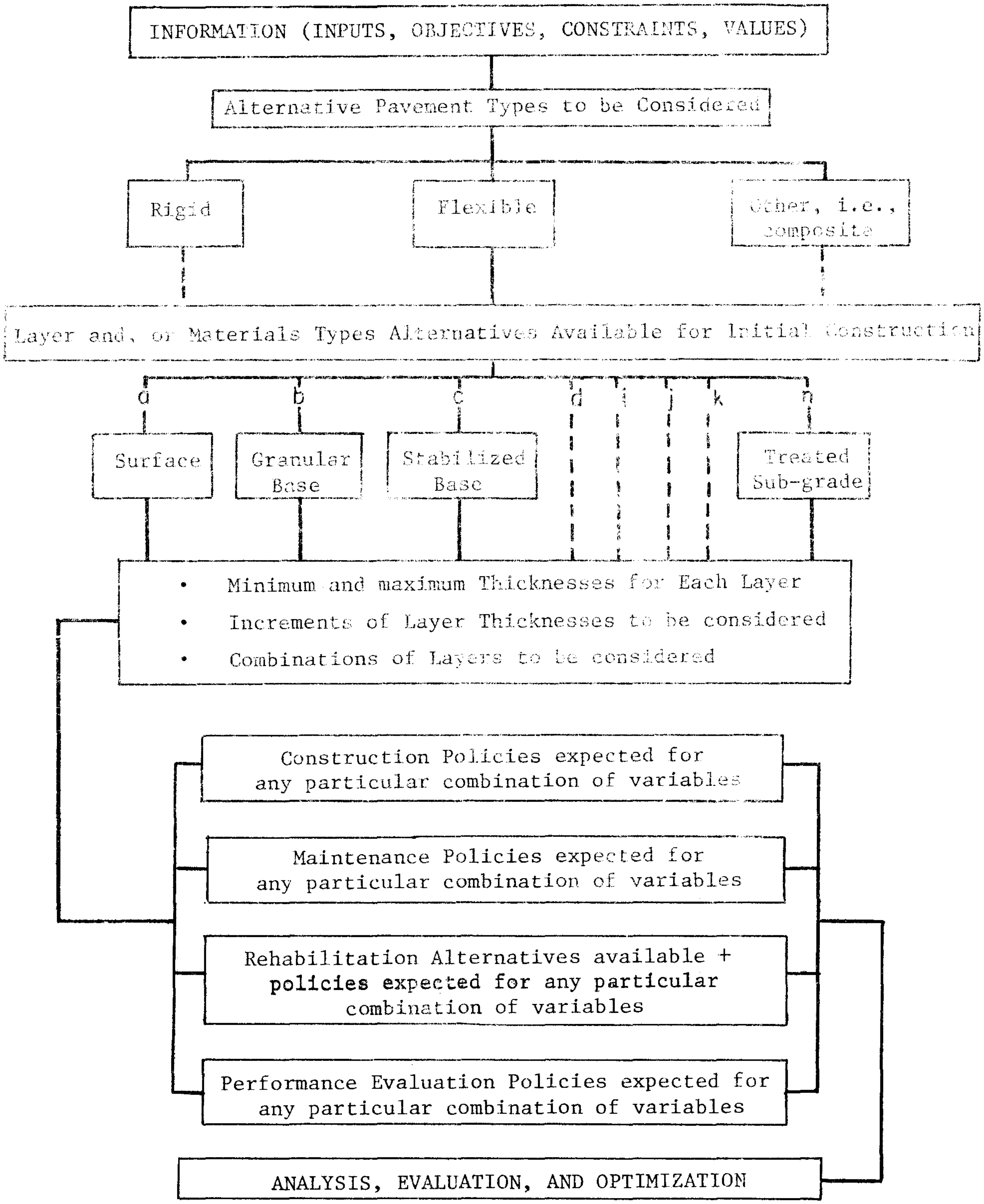


Visual Aid 22.1. Schematic diagram of post pavement.

Visual Aid 22.2. Major phases of the systems analysis methods.



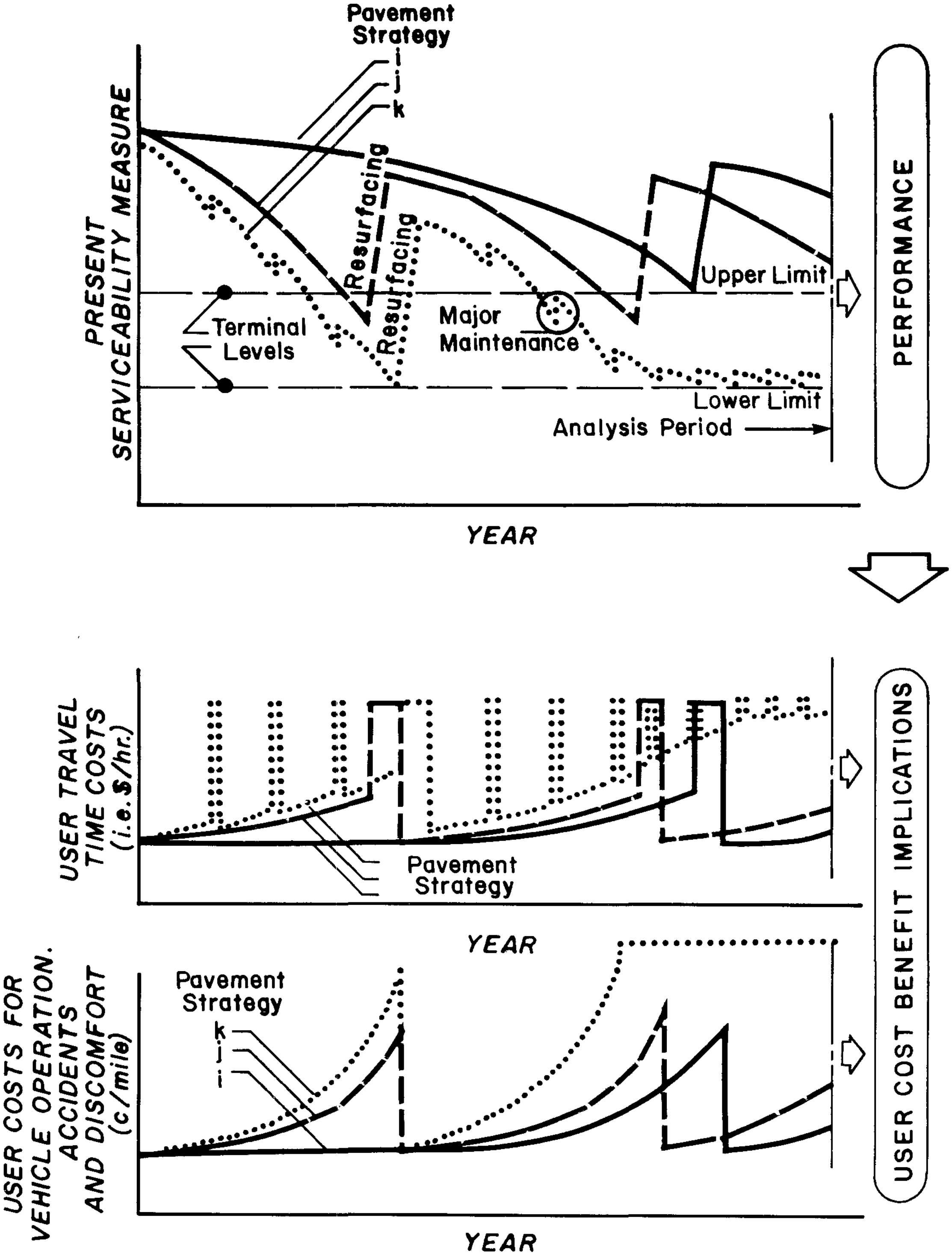
Visual Aid 22.3. Key components of generating alternative strategies.

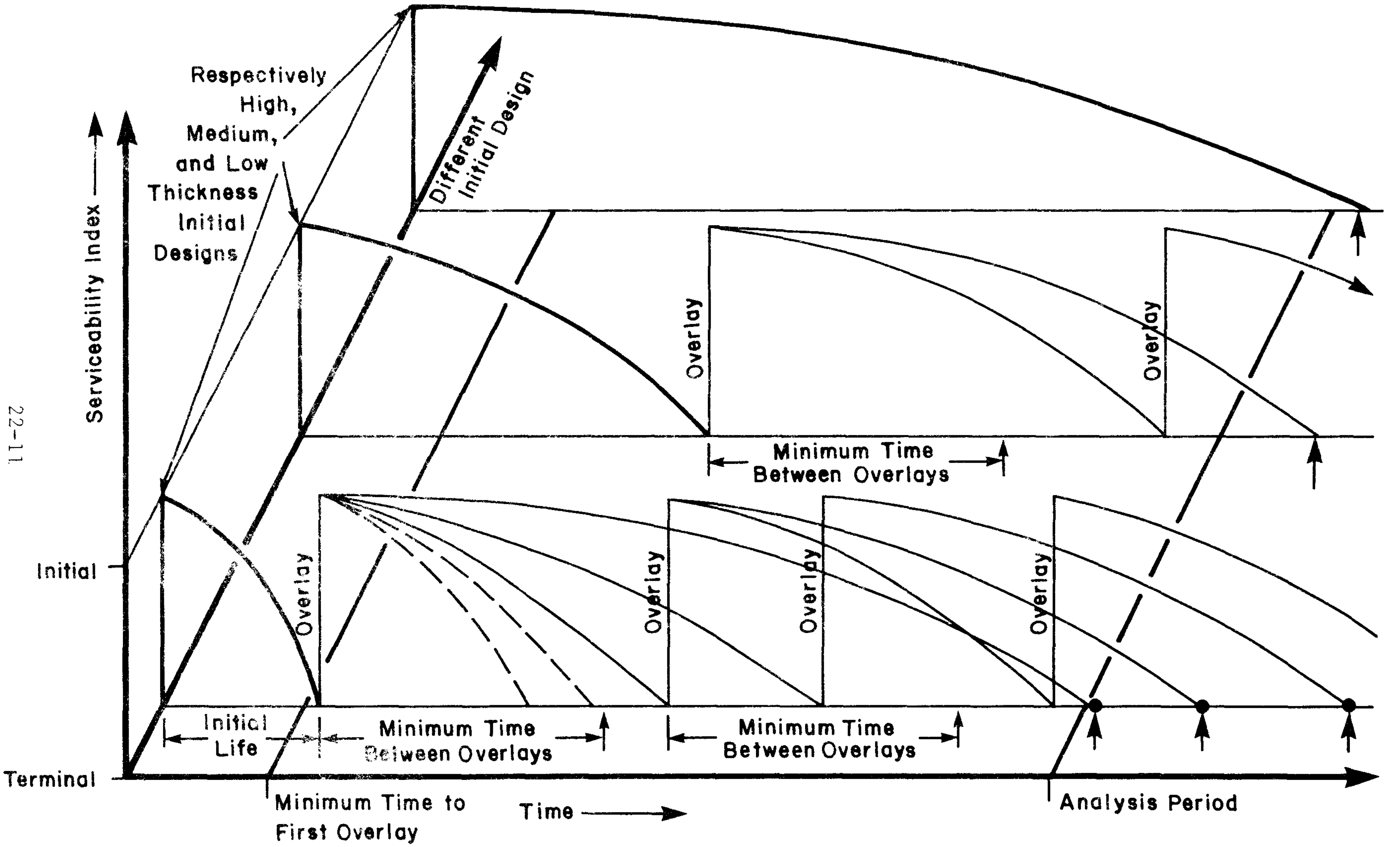


Visual Aid 22.4. Range of initial possible structural section alternatives.

Layer Type	Is layer to be included in all Alternatives	Thickness Range (in.)		Alternative Possible Initial Structural (in.)												
		MIN	MAX	1	2	→	i	→	k	l	→	n	→	p	q	→
Asphalt Concrete Surface	YES	2	4	2	2½	→	4	→	2	2½	→	4	→	2	2	→
Unbound Granular Base	YES unless (c) is used	4	8	4	4	→	4	→	0	0	→	0	→	4½	5	→
or																
Asphalt Treated Base	YES unless (b) is used	2	4	0	0	→	0	→	2	2	→	2	→	0	0	→
Unbound Granular Sub-base	YES unless (c) is used	4	8	4	4	→	4	→	4	4	→	4	→	4	4	→
TOTAL PAVEMENT THICKNESS				10	10½		12		8	8		10		10½	11	

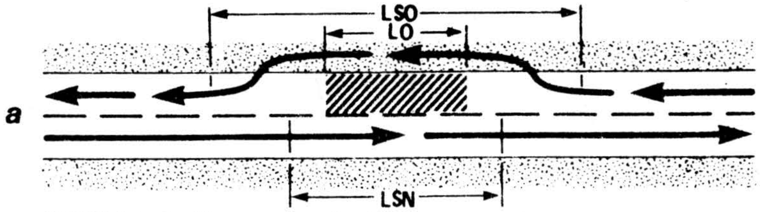
Visual Aid 22.5. Effect of policies on user costs.



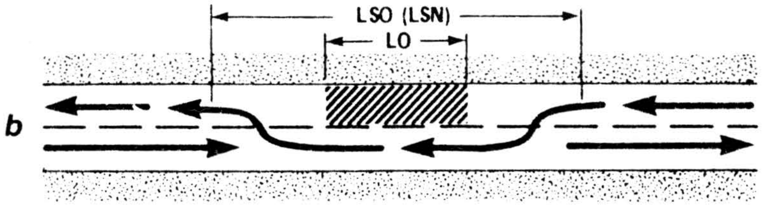


Visual Aid 22.6. Illustrative performance patterns of overlays.

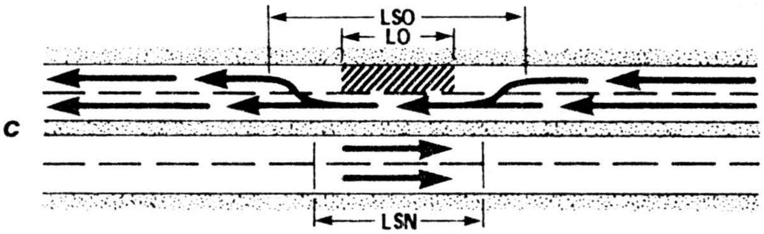
Visual Aid 22.7. Traffic handling methods.



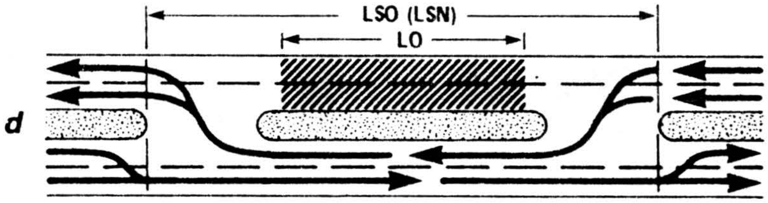
METHOD I: Traffic routed to shoulder



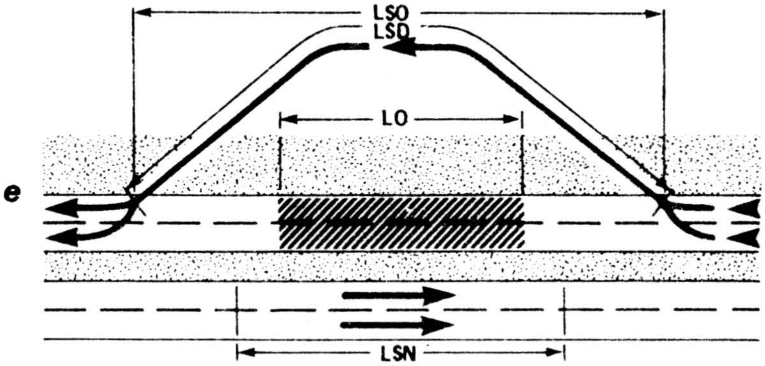
METHOD II: Alternating traffic in one lane



METHOD III: Two lanes merge, non-overlay direction not affected

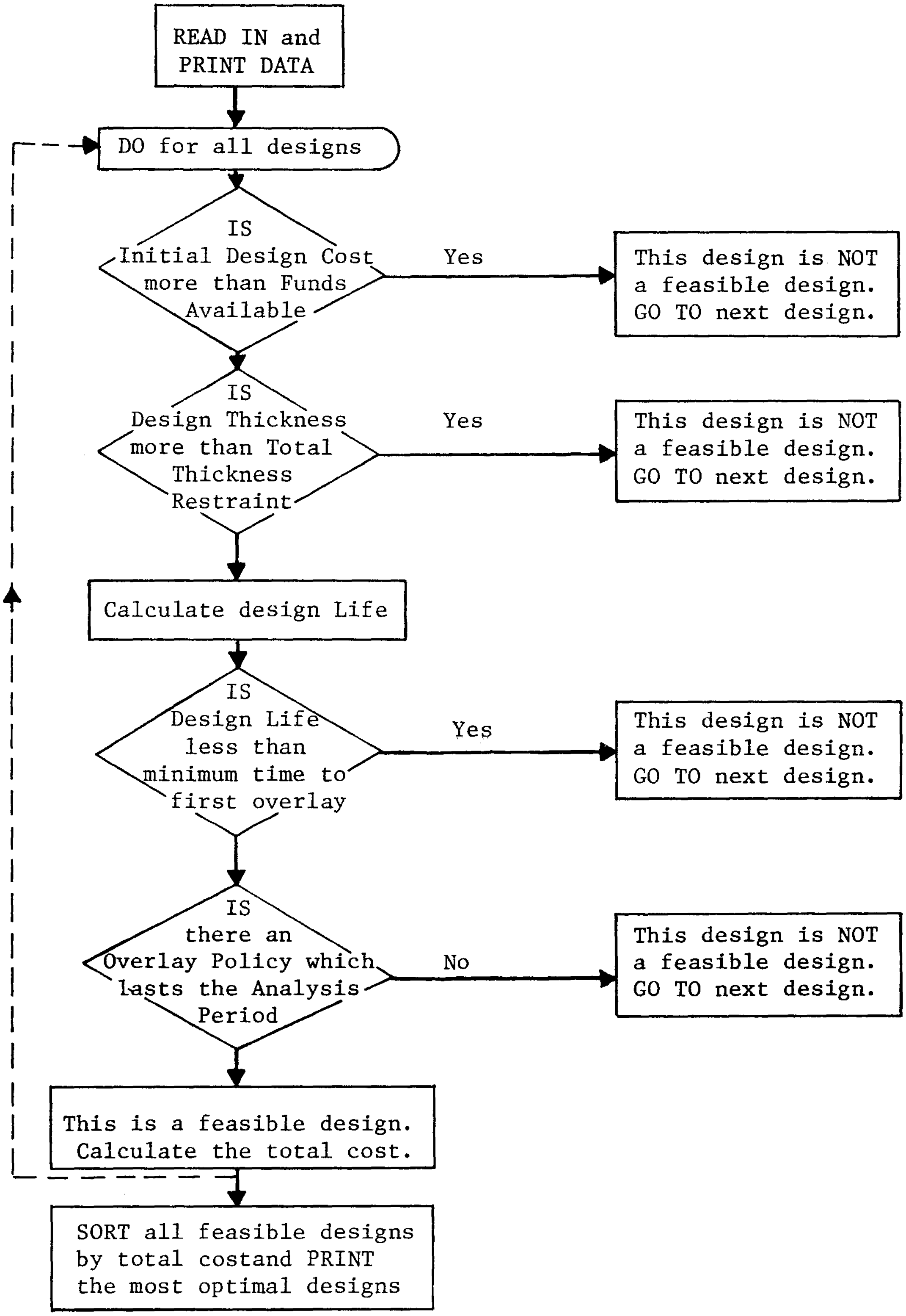


METHOD IV: Overlay direction traffic routed to non-overlay lanes

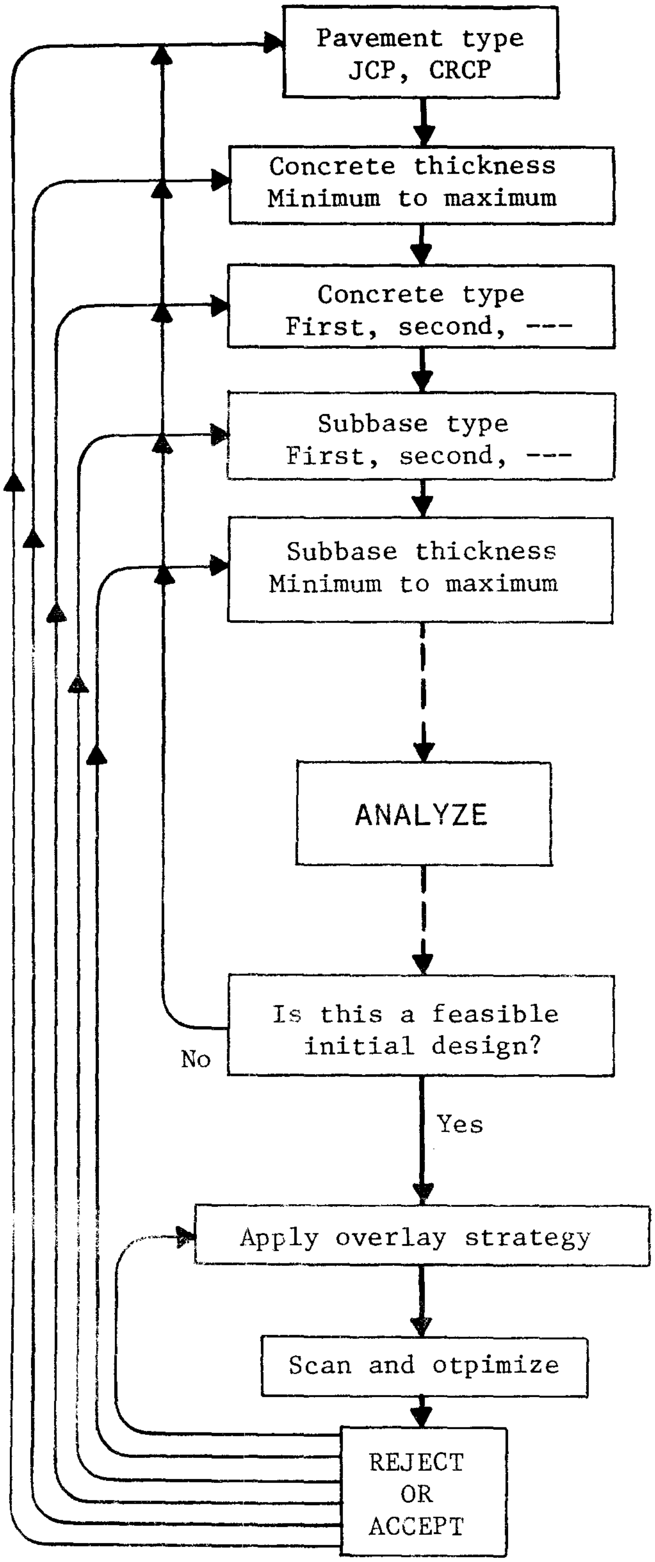


METHOD V: Overlay direction traffic routed to frontage road or parallel route

Visual Aid 22.8. Process of generating designs of FPS.



Visual Aid 22.9. Process of generating designs in RPS.



LESSON OUTLINE

LIFE CYCLE COSTING AND ECONOMIC ANALYSIS

Instructional Objectives

1. To explain the concept of life cycle costing.
2. To present economic analysis methods based on life cycle cost comparisons.
3. To pinpoint the importance of economic analysis in the decision making process.

Performance Objectives

1. The student should be able to calculate the life cycle cost of a pavement project.
2. The student should be able to explain the principles of each economic analysis method.
3. The student should know the advantages and disadvantages of the different economic analysis methods.

Abbreviated Summary	Time Allocations, min.
1. Background	5
2. Time Value of Money Equations	15
3. Life Cycle Costing of a Project	10
4. Methods of Economic Analysis	20
	50 minutes

Reading Assignment

1. Haas & Hudson - Chapter 16, Pages 199 to 216
2. RTAC-Canadian Guide - Part 2 and Part 3, Pages 2.1 - 3.22
3. Instructional Text

Additional Reference

1. Grant, E. L., Ireson, W. G., "Principles of Engineering Economy", Wiley, Sixth Edition, 1976 - Chapter 4, 6, 7.

LESSON OUTLINE
LIFE CYCLE COSTING AND ECONOMIC ANALYSIS

1.0 BACKGROUND

1.1 Definition

Life cycle cost of a venture is the summation of all expenditures and incomes occurring over the lifetime or analysis period of the venture.

1.2 Elements in Life Cycle Costing

The elements usually included in life cycle costs in pavement projects are as follows:

1.2.1 Agency Costs.

(a) initial cost,

(b) rehabilitation,

(c) maintenance, and

(d) salvage value among others.

1.2.2 Users Costs.

(a) delays,

(b) extra operational costs, and

(c) safety.

1.3 Time Value of Money

The estimation of life cycle cost is made somewhat complicated by the fact that money changes value with time. The time value of money can be computed using the equations shown in Visual Aid 23.1.

1.3.1 Interest Rate. Interest may be defined as money paid for the use of borrowed money. The rate of interest is the ratio between the interest changeable or payable at the end of a period of time, and the money owed at the beginning of that period.

1.3.2 Inflation. The question of how to take inflation into account is a economic, evaluation is of concern to engineers and administrators. Basically, the answer is that inflation is not used in the evaluation, except when substantial evidence exists that real prices will change.

1.3.3 Design Period. Time period over which an economic analysis are computed. This period should not extend beyond time of reliable forecasts.

2.0 TIME VALUE OF MONEY EQUATIONS

For the life cycle cost anlysis of pavement sections, the most used equations are the ones for present worth; however, the other approaches are also applicable.

2.1 Present Worth of Single Pavement (Visual Aid 23.2)

The equation to compute the present worth of a single payment answers the question: How much future sum F, "invested" (n) years from now, is worth if "invested" today at an interest rate (i)?

$$P = F \left[\frac{1}{(1 + i)^n} \right]$$

2.2 Present Worth of a Series of Equal Payments (Visual Aid 23.3)

How much a series of payments of A, "invested" at the end of each of (n) periods at an interest rate (i), is worth today?

$$P = A \left[\frac{(1 + i)^n - 1}{i(1 + i)^n} \right]$$

2.3 Examples

2.3.1 Sample Computation of PW of Single Payment.

How much is it worth today - a sum of $200,000, obtained 10 years from now, if the interest rate is 5%?

$$P = 200{,}000 \left[\frac{1}{(1 + 0.05)^{10}} \right] = 200{,}000\ (0.6139) = 122{,}783.$$

2.3.2 Sample Computation of PW of Series of Equal Payments.

How much is it worth today - a series of payments of $3,000.00 invested at the end of 10 periods, at an interest rate of 5%?

$$P = 3{,}000 \left[\frac{(1 + 0.05)^{10} - 1}{0.05\ (1 + 0.05)^{10}} \right] = 3{,}000\ (7.7217) = 23{,}165$$

3.0 LIFE CYCLE COSTING OF A PROJECT

Using the time value of money, the different costs occurring through the lifetime or analysis period of a project can be transferred to an equivalent cost at some reference year.

For instance, the present value of a pavement project, which will last (n) years, with an initial cost C, a yearly maintenance cost M, and a salvage value S, is equal to:

$$P = C + M \left[\frac{(1 + i)^n - 1}{i(1 + i)^n} \right] - S \left[\frac{1}{(1 + i)^n} \right]$$

3.1 Examples of Life Cycle Costing

Calculate the present worth of a pavement section where the following investments need to be made:

(a)	Initial construction cost	$1,000,000.
(b)	Maintenance costs	3,000./per year
(c)	Salvage value	200,000.

Assume a design life of 10 years and an interest rate of 5%.

$P = 1,000,000. + 3,000\ (7.7217) - 200,000\ (0.6139)$

$P = 1,000,000. + 23,165 - 122,780$

$P = 900,385.$

4.0 METHODS OF ECONOMIC ANALYSIS

By comparing the life cycle costs of mutually exclusive project, a selection can be made of the most economical.

4.1 Basic Principles

(a) Economic analysis provides the basis for a management decision but does not by itself represent a decision.

(b) An economic evaluation should consider all possible alternatives, within the constraints of the problem.

(c) All alternatives should be compared over the same time period.

(d) The economic evaluation of pavements should include agency and user costs, and benefits if possible.

4.2 Description of Methods

There are a number of methods of economic analysis that are applicable to the evaluation of alternative strategies. Visual Aid 23.4 presents a summary of the methods of economic analysis.

The following paragraphs briefly consider the essential characteristics of each method:

4.2.1 Equivalent Annual Cost Method. This method combines all initial capital costs and all recurring future expenses into equal annual payments over the analysis period. (Visual Aid 23.5).

4.2.2 Present Worth Method. This method can consider either cost alone, benefits alone, or costs and benefits together. It involves the discounting of all future sums to the present, using an appropriate discount rate.

The combination of both benefits and costs is known as the net present value method (NPV). It is simply the difference between the present worth of benefits and the present worth of costs. Obviously benefits must exceed costs if a project is to be justified on economic grounds. (Visual Aid 23.6).

4.2.3 Rate of Return Method. This method, which is used by a number of highway agencies, considers both costs and benefits and determines the discount rate at which the costs and benefits and determines the discount rate at which the costs and benefits for a project are equal. (Visual Aid 23.7).

4.2.4 Benefit-Cost Ratio Method. The benefit-cost ratio is perhaps the most used approach for economic analyses in the highway field. It involves expressing the ratio of the present worth of benefits of an alternative to the present worth of costs. (Visual Aid 23.8).

4.2.5 Cost-Effectiveness Method. Where significant nonmonetary outputs are involved, the cost-effectiveness method can be used to compare alternatives. It involves a determination of the advantages or benefits to be gained, in subjective terms, of additional expenditures.

4.3 Considerations When Selecting the Appropriate Method

(a) How important is the initial capital expenditure in comparison to future expected expenditures?

(b) What method is most understandable to the decision maker?

(c) What method best suits the requirements of the particular agency involved?

(d) Are benefits to be included in the analysis?

LESSON OUTLINE
LIFE CYCLE COSTING AND ECONOMIC ANALYSIS

VISUAL AID	TITLE
Visual Aid 23.1.	Time value of money equations.
Visual Aid 23.2.	Present worth of single payment.
Visual Aid 23.3.	Present worth of a series of equal payments.
Visual Aid 23.4.	Methods of economic analysis.
Visual Aid 23.5.	Equivalent annual cost method.
Visual Aid 23.6.	Net present value method.
Visual Aid 23.7.	Rate of return method.
Visual Aid 23.8.	Benefit-cost ratio method.

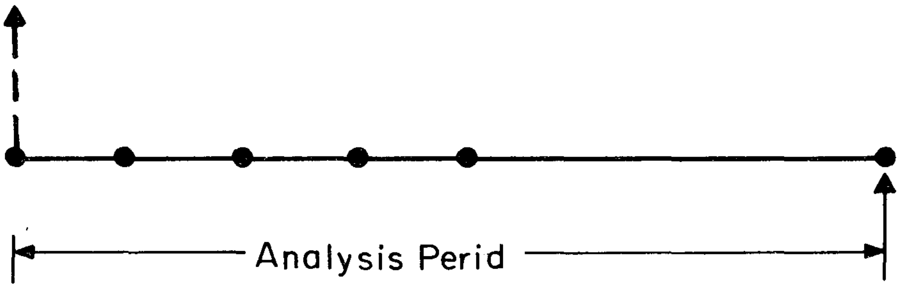
Visual Aid 23.1. Time value of money equations.

Equation Number			
1	Present worth	Single payment present worth factor	$P = F \frac{1}{(1 + i)^n}$
2		Equal payment series present worth factor	$P = A \frac{(1 + i)^n - 1}{i(1 + i)^n}$
3	Future worth	Single payment compound amount factor	$F = P (1 + i)^n$
4		Equal payment series compound amount factor	$F = A \frac{(1 + i)^n - 1}{i}$
5	Equal annual payment	Equal payment series sinking fund factor	$A = F \frac{i}{(1 + i)^n - 1}$
6		Equal payment series capital recovery factor	$A = P \frac{i(1 + i)^n}{(1 + i)^n - 1}$

Symbols: i = annual interest rate, discount rate, or inflation rate
n = number of annual interest periods
P = a present principal sum
A = a single payment in a series of n equal payments, made at the end of each annual interest period
F = a future sum, n annual interest periods hence equal to the compound amount of a present principal sum P or equal to the sum of

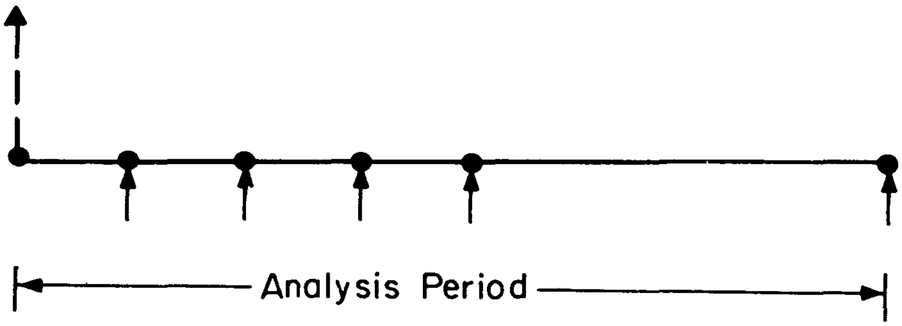
Visual Aid 23.2. Present worth of single payment.

$$P = F\left[\frac{1}{(1+i)^n}\right]$$



Visual Aid 23.3. Present worth of a series of equal payments.

$$P = A\left[\frac{(1+i)^{n}-1}{1(1+i)^{n}}\right]$$



Visual Aid 23.4. Methods of economic analysis.

- EQUIVALENT ANNUAL COST
- PRESENT WORTH FOR
  - COSTS
  - BENEFITS
  - NET PRESENT VALUE
- RATE OF RETURN
- BENEFIT-COST RATIO
- COST EFFECTIVENESS

Visual Aid 23.5. Equivalent annual cost method.

$$EAC = (CRF)\ ICC + AAMO + AAUC - (CRF)\ \frac{SV}{(1 + C)^{N}}$$

EAC = EQUIVALENT ANNUAL COST
CRF = CAPITAL RECOVERY FACTOR
ICC = INITIAL CAPITAL COST
AAMO = AVERAGE ANNUAL MAINTENANCE PLUS OPERATION COST
AAVC = AVERAGE ANNUAL USER COST
SV = SALVAGE VALUE

Visual Aid 23.6. Net present value method.

$$NPV = TPWB - TPWC$$

NPV = Net Present Value

TPWB = Total Present Worth of Benefits

$= \Sigma\,(PWF)\,\left[DUB + IUB + NUB\right]$

= Total Present Worth of Costs

$ICC + \Sigma\,(PWF)\,\left[CC + MO + US\right] - (PWF)SV$

Visual Aid 23.7. Rate of return method.

FIND INTEREST RATE WHICH SATISIFIES:

TPWB = TPNC

TPWB = TOTAL PRESENT WORTH OF BENEFITS

TPWC = TOTAL PRESENT WORTH OF COSTS

Visual Aid 23.8. Benefit-Cost ratio.

$$BCR = \frac{TPWB}{TPNC}$$

BCR = Benefit-Cost Ratio

TPWB = Total Present Worth of Benefits

TPWC = Total Present Worth of Costs

INSTRUCTIONAL TEXT

PAVEMENT TYPE SELECTION - LIFE CYCLE COST

PAVEMENT TYPE SELECTION - LIFE CYCLE COST

Introduction

One of the areas of pavement management that has received a lot of attention over the years has been pavement type selection. It has been stated that the highway engineer or administrator does not have at his disposal generally acceptable theoretical or rational methods that give an absolute and indisputable comparison of the competitive pavement types for set conditions.

The 1960 AASHTO Guidelines on Pavement Type Selection noted that there was no magic formula, where certain figures could be inserted and a definite answer to pavement type would result.

The highway engineer has faced the problem of selecting pavement types from the beginning of modern highway construction. It is an essential part of the engineering process of developing a facility. The issue became quite serious at the start of the Interstate System. There was a debate in the Congress as to whether asphaltic concrete or portland cement concrete was the best type of pavement to use. There were of course industry advocates on each side of the issue and no doubt they had valid statistics to back up their position. The debate has continued in importance to the present.

Many groups and individuals are interested these days in the types of pavement being selected and constructed, even though they do not always understand the problems involved in making the selection. The engineer finds himself, to an increasing extent, required to justify and document his decisions. This is a healthy situation, for although the highway program is vast in extent, with large sums of money being expended, there is a great need for economy. Never before has the engineer been able to realize such large dividends, as can come from the proper selection of pavement type - dividends in construction and maintenance costs, as well as in service to the user. The taxpayer and those who represent him need to be assured that valid and correct decisions are being made by the engineer.

The preceding comment was taken from a paper by W. J. Liddle of the FHWA Pavement Branch in 1961. It is interesting to note that the same situation exists today.

With the decreasing amount of new construction and the effects of the current recession, industry appears to be becoming more actively involved in the selection of the pavement type. They are questioning our design procedures, selection guidelines and performance data. They are developing performance data of their own to illustrate the benefits of their product.

For example, we recently adopted the Revised AASHTO Pavement Guide which revised Chapter III on rigid pavements. The concrete and cement industries opposed this adoption because under certain conditions, it would require thicker concrete sections and the industry felt that it would place them at an unfair disadvantage in competitive bidding against asphalt pavements. They were also concerned that the flexible portion of the guide was not revised at the same time.

While we have maintained our adoption of the Revised Guide, we have encouraged the revision of the entire AASHTO Guide. AASHTO is contracting a project through

NCHRP to accomplish this. This will take some time to complete and you can be certain that it will be closely monitored by both industries.

Concern has not been limited to the concrete industry. There is a push to use full depth asphalt pavements and higher structural layer coefficients that result in thinner sections. The asphalt industry is trying to increase the structural layer coefficients as a result of a restudy of the AASHO Road Test. Everybody is trying to modify the design procedures and practices, sometimes not based on performance, but on a desire to make their product more competitive in the pavement type selection process.

Why are we in this position? Why have things gotten out of control? Some of it can be traced back to the end of the AASHO Road Test in the early 1960's. The Road Test was an accelerated full scale test with fixed axle loads being applied to varying thickness of the two pavement types on a common subgrade for a short time period. It was intended that after the Road Test, each State would take the results and conduct satellite studies to verify or modify the results to fit their own individual conditions.

However, this was never fully implemented. We didn't get the performance data that was needed. Thus over 20 years later, we are still relying on the same ASSHTO design equation and asking the same questions? Which is better, asphalt or concrete?

With decreasing highway revenues, increasing construction costs, and inflation, the emphasis has been on pavement management over the past few years. Highway administrators have been trying to get the best pavement for the highway dollar. With this emphasis, they have taken a hard look at pavement type selection. And this emphasis has initiated a lot of interest in a new term, life cycle cost.

It is recognized by States and Federal Government that it is more desirable to measure the costs of a highway improvement over a certain analysis period instead of just comparing initial costs. This involves an economic analysis that compares all costs (initial, maintenance, rehabilitation, and road user) over a chosen analysis period.

Before I get into a detailed·discussion of life cycle cost, I would like to discuss our policy on pavement type selection. It is hard to understand our emphasis on life cycle cost without a discussion of all phases of the pavement type selection process.

FHWA Policy

The FHWA policy on pavement type selection is of long standing and is designed to provide the public with acceptable highway service at a minimum cost while permitting opportunity for the use of competing materials and different design details. We recognize that alternate materials are available for use in the design of pavement structures and that acceptable designs can be prepared utilizing these materials.

The pavement is to be designed in accordance with procedures that have been found by experience to provide an economical, durable and otherwise satisfactory highway structure under the conditions that will prevail on the highway section under consideration. The design is to be based on traffic volumes and axle loads estimated to occur by the design year established for the project and on engineering and economic evaluation of all governing factors.

For many years, FHWA has recommended that pavement selection be based upon an engineering analysis using the factors listed in the AASHTO publication entitled "An Informational Guide on Project Procedures." Some time ago, portions of the Guide not related to pavement selection became obsolete and the guide was no longer available through AASHTO. Due to the emphasis on pavement management, AASHTO recently reprinted the chapter on pavement type determination separately and it is now available from AASHTO.

From time to time proposals have been made by State Highway Departments whereby the selection of pavement would be determined by direct bidding competition between alternative types. In 1968, a policy on alternate bids was issued in a memorandum from Mr. Frank Turner to Mr. A. C. Taylor.

This memorandum reaffirmed FHWA's use of the factors listed in the AASHTO "Informational Guide on Project Procedures." It stated that when an engineering analysis showed that several specific types of pavement structures will equally serve the highway needs, alternate bids may be taken. There were two cases where alternate bids were permitted. Under the first case, full 20-year designs could be bid against each other with the lowest bidder being accepted. Under the second case, an initial stage of a staged construction could be bid against a 20-year design. Costs for future stages during the 20-year design period were added to the bid for the initial stage to determine the lowest successful bidder. Separate PS&E's were required, maintenance costs could not be used in the analysis, and no future Federal-aid funds were permitted in the design period for case 1. This policy remained in effect for many years. Over this period, there were many interpretations of the policy; however, the use of alternate bids was limited until recently.

In 1974, Mr. Lindberg rescinded the requirement for 2 PS&E's. In 1976, we surveyed the Regions on pavement type selection processes. It was felt that the AASHTO "Informational Guide on Project Procedures" was still generally valid and suitable for use. There were no changed in the policy on alternate bids.

In 1980, we issued an ANPRM in the <u>Federal Register</u> in an attempt to update our policy on pavement type selection. This notice asked several questions to stimulate discussion on the subject. All States responding supported the use of the AASHTO Informational Guide.

About this time the use of alternate bids started increasing, especially in the southeast. The interpretations of Turner's memorandum were varied and there was no uniformity. Some States were bidding full designs against each other and some were bidding staged designs againt full designs. There was concern that the designs were not equivalent. Some States were using various methods to add on future costs. We started receiving numerous inquiries from the field. Also, industry started getting involved in the process, questioning a lot of the FHWA field decisions. They were questioning the designs saying they were not equivalent.

Based on these concerns, it was decided to issue a policy statement as soon as possible on pavement type selection. A policy statement was issued on October 8, 1981. The policy was designed to provide the public with acceptable highway service at a minimal annual life cycle cost while permitting maximum flexibility. It was written with the intention of taking advantage of fluctuating material prices while not compromising good design and pavement management practices.

The policy has the following four key points:

1. Pavement type selection should be based upon an engineering evaluation considering the factors contained in the 1960 AASHTO publication entitled "An Informational Guide on Project Procedures." The consideration of alternate designs and strategies were encouraged in the pavement type selection process.

2. The pavement type determinations should include an economic analysis based on life cycle costs of the pavement type. Estimates of life cycle costs should become more accurate as pavement management procedures begin providing historical cost, serviceability, and performance data. States without this data are encouraged to obtain it.

   It is this point that has generated the interest for presentation today. We will go into more detail later on.

3. The third point is that an independent engineering and economic analysis and final pavement type determination should be performed or updated a short time prior to advertising on each pavement type being considered. Some agencies do the type determination years before a project is advertised and never update it. It should be updated to take advantage of any fluctuating market prices.

   There has been some confusion as to what we mean by independent. Some people have misinterpreted this term to mean that the determination has to be conducted by a separate agency or department. Our intention was that each pavement type would be reviewed and considered separately or independent of the other pavement types until the final determination is made.

4. The last point was that when the analysis reflects that two or more initial designs and their forecasted performance are determined to be comparable (or equivalent), then alternate bids may be permitted if requested by the contracting agency. The Division Administrator shall review the analysis and concur in the finding of equivalency prior to PS&E approval.

   The policy permits the use of alternate bids when an engineering and economic life cycle analysis results in no clear-cut choice between alternatives and when two or more designs are equivalent or comparable. The two prerequisites for the use of alternate bids are:

   1) Initial designs must be comparable or equivalent. Each pavement type is to be designed using the same traffic over the same analysis period. We use the "AASHTO Interim Guide for Design of Pavement Structures" to evaluate the adequacy of the initial designs. Please note that this

is a full initial design and not a stage construction design.

2) The second prerequisite is that the forecasted performance must be equivalent or comparable. For alternate bids, it is essential that the State have adequate data to document the performance of each pavement type being used in the State. This will include current performance and life cycle cost data that reflect comparable or equivalent service life.

   The PS&E shall not include future costs as adjustment or add-on factors. This has been allowed by Turner's memorandum to Taylor in 1968, but it was decided that this should not be allowed anymore.

It is difficult, if not impossible to even develop equivalent full designs between rigid and flexible pavements, much less equivalent designs between a stage construction and a full design. It is impractical to bring the cost of future overlays back to present worth for deciding the low bidder. With fluctuating interest rates, inflation factors, and materials costs, it is impossible to predict with any accuracy the cost for future improvements as stages. It was decided that it is unfair to the contractor to include this cost in the determination of the lowest bidder. If we are going to allow alternate bids then the performance or service life must be equivalent. We are not going to allow unequal designs to be bid against each other.

A review of the overall policy statement reveals two key ingredients, an engineering analysis and an economic analysis. Each one of these is essential to insure that the proper pavement type is selected.

Engineering Analysis

The engineering evaluation is to consider the factors contained in the 1960 AASHTO publication entitled "An Informational Guide on Project Procedures." These factors are found under the section entitled "Paving Type Determination and Documentation" on pages 49-54. The guide states,

> "To avoid criticism, if that is possible, any decision should be firmly based. Judicious and prudent consideration and evaluation of the governing factors should result in a firm base for a decision on paving type."

The fifteen governing factors are divided into two groups. The principle factors are those which may be considered to have a major influence. The secondary factors are those which have a lesser or only occasional influence. The order of magnitude or influence is considered interchangeable within the groups and between the groups, as no single order is held to apply in all cases. The factors are generally applicable to both new and reconstructed pavements.

The first principle factor is traffic. The volume of passenger cars generally affects only the geometric or lane requirement. The percentage of commercial traffic and frequency of heavy load application generally has the major direct effect on the structural design of the pavement. Existing heavy-duty highways constitute sufficient evidence that both flexible and rigid pavement designs can meet requirements under given conditions.

If a cost comparison between competitive paving types is to be of value, it is imperative that the structural designs compared have equal capacity to carry loads.

Another factor is soil characteristics. The characteristics of native soils not only directly affect the pavement structure design, but may, in certain cases, dictate the types of pavement economically justified for a given location.

Weather is a factor that affects the subgrade as well as the pavement wearing course. The amount of rainfall, snow and ice and frost penetration will seasonally influence the bearing capacity of subgrade materials. Moisture and freezing and thawing affect a pavement's performance. In drawing upon performance record of pavements elsewhere, it is most important to take into consideration the conditions pertaining in the particular climatic belt.

To a large degree, the experience and judgment of the highway engineer is based on the performance of pavements in the immediate area of his jurisdiction. Past performance is a valuable guide, provided there is good correlation between conditions and service requirements between the reference pavements and the designs under study. This factor should not be allowed to develop into blind prejudice. Caution must be urged against reliance on short-term performance records, and on those long-term records of pavements which may have been subjected to much lighter loadings for a large portion of their present life. The need for periodic reanalysis is apparent.

One of the most critical factors is the cost comparison. This is where the analysis of life cycle cost comes in and will be discussed later. In any cost comparison of paving types, the matter of availability of local or commerically produced materials and the existence and proximity of manufacturing or processing plants will be of significant importance.

The secondary factors are:

1. Adjacent existing pavements.

2. Stage construction - This is a definite advantage of flexible pavements and has been used by many highway agencies in their pavement management scheme. This factor should be considered as it may be more economical and provide for a smoother ride since you are scheduled to overlay within a certain time period.

3. Depressed, surface, or elevated design.

4. Highway system.

5. Conservation of aggregate - This factor is important today with emphasis on rehabilitation. The recycleability of a material has become an important consideration.

6. Stimulation of competition.

7. Construction consideration. Such considerations as speed of construction, reduction of traffic maintenance during construction, ease of replacement, anticipated future widening, need for minimum of surface maintenance in highly congested locations, seasons of the year when construction must be accomplished, and perhaps others may have a strong influence on paving type selection.

   One of the key considerations today is how long a pavement will last before requiring rehabilitation. For example, in an urban area you want the pavement to last as long as possible to avoid handling traffic during rehabilitation. The cost of traffic maintenance is a large part of rehabilitation and you want to choose the pavement type that will keep the need for maintenance of traffic to a minimum.

8. Municipal Preference, Participating Local Government Preferences and Recognition of Local Industry.

9. Traffic Safety.

10. Availability of and Adoption of Local Materials or of Local Commerically Produced Paving Mixes.

These are the factors listed in the 1960 AASHTO Guide. There may be other factors you may want to consider in an engineering evaluation. Now the guide was developed in 1960 and has served us for well over 20 years. The AASHTO Joint Task Force on Pavements has just completed an update of these factors. The new guidelines were approved by AASHTO but will not be published at this time. The guidelines will be included in the future update of the AASHTO Pavement Design Guide. The new guidelines list the following items as principle factors:

1. Traffic
2. Soil Characteristics
3. Weather
4. Construction Consideration
5. Recycling
6. Cost Comparison

The secondary factors are:

1. **Performance of similar pavements in the area.**
2. **Adjacent existing pavements.**
3. **Conservation of materials and energy.**
4. **Availability of local materials or contractor capabilities.**
5. **Traffic safety.**
6. **Incorporation of experimental factors.**
7. **Stimulation of competition.**
8. **Municipal preference, participating local government.**

The factors in the engineering analysis apply not only to new construction, but also to rehabilitation. You need to conduct a pavement evaluation and identify the distress and determine the cause of the distress. Then you can consider primary factors such as traffic, soils, construction consideration, weather. From this analysis, alternates are developed that address the cause of the distress. Life cycle cost analysis can then be conducted on each alternative strategy.

The engineering evaluation must also evaluate the structural design. FHRPM 6-2-4-1 requires that the design of a particular pavement be evaluated to see if it coincides with the AASHTO Pavement Design Guide.

Economic Analysis

To properly discuss the costs of pavements, we should consider some of the principles of economics. Webster defines economics as, "The Science that investigates the conditions and laws affecting the production, distribution, and consumption of wealth." He further defines economy as, "The thrifty use of material resources, frugality in expenditures, and efficient and sparing use of the means available for the end proposed.

Engineering economy has been recognized for years. In the field of transportation the subject goes back at least to 1877 when Mr. Arthur Wellington, a well known expert in railroading wrote the following: "It would be well if engineering were less generally thought of and even defined as the art of constructing. In a certain important sense it is rather the art of not constructing or to define it rudely, but not inaptly, it is the art of doing well with one dollar that which any bungler can do with two, after fashion." I believe we can all agree that Mr. Wellington has correctly, if somewhat rudely, stated the need for economic as well as engineering considerations in the design of an engineering project.

For proper pavement type selection, in addition to an engineering evaluation, you should conduct an economic analysis based on life cycle costs of the pavement type. As we started evaluating a pavement over its service life, we have begun to consider all the costs over this period. With the different performance of pavement types, we need to select a common analysis period and evaluate all the costs over this period for all pavement types being considered.

What is life cycle cost? Unfortunately, there is not much guidance available on it. I would define it as the economic analysis of all costs associated with a pavement type over a chosen analysis period.

This form of analysis of measuring costs is not a system of precise calculations. It is rigid in concepts and procedures, but is based on estimates of future cash flows. Therefore, it is subject to a certain amount of question. Nevertheless, the analysis is being used more and more.

What is the objective of all this analysis? The objective is not to find the decision between a set of alternatives, but to assemble the information on economy to aid in the decision-making process. By making an economic analysis of each structural design, the engineer is in a position to better evaluate which alternative will provide the desired service at the least cost.

You have to consider all the factors of cost and benefit. In other words, you cannot be just selective and take the costs that appeal to you. You have to include all costs that are apt to be experienced in the analysis period.

The question everyone asks is what costs should be considered and collected? The first cost is the initial construction cost. This is a cost that has been collected and used in pavement type selection for many years. This data is obtained from bid tabulations. The cost should be kept current and should represent pavements of similar types in some geographical location of the State. Current typical construction costs reported to FHWA Construction Price Index are $24.29/ton for bituminous pavements and $12.75/square yard for portland cement concrete pavement.

As noted earlier, construction costs are dependent upon the availability of local materials and in some cases on the proximity of manufacturing or processing plants. These costs can usually be estimated with reasonable accuracy. Careful study is required whenever a new type of pavement is considered, particularly when a history of building only one type of pavement has been established.

The next cost to consider is future rehabilitation and heavy maintenance cost. How much is it going to cost to rehabilitate the pavement to keep it at an acceptable level of serviceability over the analysis period. There are several items that must be addressed to determine these costs. First, what is the minimum level of serviceability you want to maintain. This will vary from agency to agency and with the type of highway facility. For example, on the Interstate system, you will probably use a present serviceability index (PSI) value of 3.0 to determine when to initiate rehabilitation. On a farm to market road, you may be able to live with a threshold value of 2.0. These values will vary with what the motorist is willing to accept and what the agency can afford.

Choosing a threshold value is also a matter of economics. You may think that using a low value will mean that you don't have to rehabilitate as often and you will save money. However, it has been shown that by doing rehabilitation or heavy maintenance sooner, when the pavement is at a higher threshold value, substantial savings can be realized. As a pavement deteriorates, the cost to rehabilitate it increases significantly.

To determine when a pavement reaches the threshold value, you need to set up procedures to conduct periodic conditions surveys. These vary from State to State. Some are based on ride only, others on distress, and still others on combination of ride and distress.

Another item that must be considered is performance data. How long does the pavement last before it needs rehabilitation? You would be surprised with the number of States that don't have this data. I recently conducted a review of a southeastern State that based the selection of pavements on an old performance study by FHWA. When I reviewed this study, I found that it was a summary of performance data for western States. Here was a State that was basing its selection of pavement type on outdated performance data from other States with different design practices, materials, and environmental conditions.

How long does your asphalt pavement last before needing overlays or how long does your concrete pavement last before needing joint repair? Each State should have this data. If you don't, then you are encouraged to obtain it. Historical data is essential in a pavement management process.

The last item to address is what are the rehabilitation or heavy maintenance techniques that are currently being used by your State. Each State has had success with different rehabilitation techniques. The condition survey I mentioned previously determines the distress type. An engineering evaluation is made to determine the cause of the distress and then based upon past performance a rehabilitation technique is selected to address the cause.

Also important is the timing of the techniques. How often are overlays placed or joints repaired? Here you need additional performance data on how long the rehabilitation technique will last. When evaluating a pavement type over a long analysis period, the rehabilitation technique may have to be applied several times over the period. For example, you may have to place overlays two or three times during the analysis period. For concrete pavement, you may repair joints, grind, or underseal. This will last a certain time before an overlay or additional rehabilitation is needed. We need to know how long these techniques will last. This is a very weak link in our analysis. We just don't have very good data. Some techniques are new and have not been down long enough to make a good estimate of the service time. A rehabilitation technique may work in one area and be a failure in another. A good example is the different fabric treatments for reflective cracking. While your State may not have the necessary performance data now, steps should be taken to insure that this data is collected and available for future analysis.

Once you know threshold value, performance of pavement types, types of rehabilitation work and their performance and timing, you can estimate cost for rehabilitation work. Cost data may be obtained from bid tabulations like initial costs. You may have to project some since data may not be available because some of the techniques may not have been used before.

Another cost that should be considered in a life cycle cost analysis is minor maintenance cost. This is routine maintenance cost, the cost to preserve the pavement. This cost is presently collected by highway agencies, though it may not be in a form that is useful to us.

Some maintenance data are collected by control section with all costs lumped together. So the cost you have may include mowing and litter pickup. We must be able to relate the cost to a specific pavement type.

As with rehabilitation and heavy maintenance costs, we need to know what type of minor maintenance is being performed on each pavement type such as joint sealing, pothole patching, etc. Also, we need to know how often, how long each treatment lasts, the timing of these applications. This relates back to performance data for the pavement types. How well a pavement is performing depends on how often maintenance is required. As with rehabilitation, proper timing can result in a significant savings in cost. Preventive maintenance can extend the serviceability of a pavement.

Minor maintenance costs may vary for new designs. The maintenance costs we are collecting today are for the older designs. There have been changes in our design practices over the years. With the newer, improved designs, we will have different maintenance needs,treatments, and performance; therefore, the maintenance costs will be different. This will require a little engineering judgement to estimate these costs as accurately as possible.

Estimates of maintenance costs may be toublesome. Industry has been known to question the rehabilitation and maintenance costs developed by State highway agencies. You can't satisfy everyone so you have to make the best estimate you can.

In the course of seeking an improved decision-making tool for pavement management, it became apparent that direct agency costs of construction, rehabilitation, and annual maintenance did not provide a sufficient basis for determining the pavement structure design. The cost implications of lowered service to the public in terms of additional user operation costs, due to rougher pavements, and delay costs, due to traffic impedence during rehabilitation and maintenance should also be included in the economic analysis.

When you have to rehabilitate a pavement or perform a maintenance activity, you normally have to close a lane or disrupt the traffic flow. This delay or inconvenience to traffic causes a cost to the traveling public. These activities disrupt traffic flow and cause vehicle speed fluctuation, stops and starts, and time losses. This indirect, non-agency cost has not in the past been given due consideration. The extra user cost is an expense to the road users and, therefore, should be included in the economic analysis. Cost is comprised of user time and vehicle operating values resulting from driving slowly, fluctuating speeds, stopping, accelerating, and idling. This cost includes vehicle running costs, time value to motorist; and accident cost.

Here again, you need to know performance data of your pavement type. How often do you need to rehabilitate or perform maintenance? How often will traffic be delayed?

Each alternative pavement design is associated with a number of indirect costs that accrue to the road user and must be included in a rational economic analysis. Similar to pavement costs, user costs are related to the performance history of the pavement. A pavement design that provides an overall high level of roughness over a longer time period will result in a higher user cost than a design that provides a relatively smooth surface for most of the time.

Road user costs include the sum of motor vehicle running cost, the value of vehicle user travel time, traffic accident cost, and discomfort. The motor vehicle running cost includes the expenses of fuel, tire, engine oil, maintenance, and that portion of vehicle depreciation attributable to highway mileage traveled. The roughness of the pavement surface contributes to additional tire wear and influences the maintenance and repair expenses incurred in keeping the vehicle in operation.

As a pavement becomes rough, the operating speeds of vehicles are reduced. Lower speeds and rough pavements increase traveling time, level of discomfort, and other user costs. Since level of roughness for a pavement design depends on its initial construction thicknesses and materials, the extent of rehabilitation, ad the extent of major and minor maintenance provided during its service life, user cost is interrelated with all of these factors.

There is a lack of extensive data on user costs as related to a pavement's riding quality. One of the most recent reports available is the report entitled "Vehicle Operating Costs, Fuel Consumption, and Pavement Type and Condition Factors" published in March 1982 by our Office of Highway Planning.

There is no doubt that road user or indirect costs should be considered in the economic analysis of pavement types. However, there is concern over how much weight should be given to these costs. If you give them the same weight as initial and rehabilitation costs, then you are assuming that the highway agency has all the money it needs. In this case, the road user cost may govern. However, this is not a practical assumption, since there is limited money available to a highway agency. Realistically, we must consider direct (agency) and indirect (road user) cost separately in a two phase approach. Let the decision rest with the real direct costs unless there is no real difference between alternatives, then indirect costs can enter the decision-making process.

Another cost considered is salvage value. While it is not an expense, it is a dollar value that we must consider. Salvage value, which also called residual or terminal value, has been an area of confusion and there has been disagreement over whether to consider it. Some engineers try to place some kind of value on the years of serviceability left in the highway at the end of the analysis period and no materials were to be salvaged. With conservation of materials in mind, the salvage value could be taken as the recycling value of the pavement materials less the cost of reclaiming the materials. Some people consider the salvage to be equal for both pavement types although as reconstruction/rehabilitation costs become better known, salvage can be better estimated.

In a report by the National Asphalt Pavement Association, they did not include a salvage value because, first, it is difficult to estimate a salvage value.

Second, the terminal or salvage value would have to be quite large to have a significant effect on the analysis since it would be converted to a present worth at time zero using a discounting factor for 40 years at 8%.

Salvage value refers to the residual value of the road after it has served its planned useful service life. The residual value represents the remaining value of the materials incorporated in the base, surfacing, and shoulder structure of the roadway. Minnesota just completed a study in which they determined that the value depends on many factors, both known and unknown, such as:

1) Road condition at the end of analysis period,
2) Will road be needed to continue to provide for vehicular transportation?
3) What materials,technologies, and/or equipment will be available at the end of analysis period to reutilize or prolong the utility of the materials in the existing facility.

Because of the unknowns, they decided not to consider salvage value in their economic model at this time.

However, I tend to agree with Robley Winfrey, who in a recent conference in Minnesota stated that you must consider salvage value. To make a true economic analysis, you must consider all factors. If your study shows that the salvage values for the pavement types are equal or insignificant, then they can be omitted, but at least study them.

These are the costs that should be considered in a life cycle cost analysis. Now each of these costs occur over various years of the analysis period. Therefore, an economic comparison must be made of these costs. There are several methods for making economic comparisons of alternate proposals. The present worth, equivalent uniform annual cost, rate of return and cost-benefit ratio methods are the most common. Of the four methods, the present worth and equivalent annual cost concepts are better suited for pavement cost evaluations.

The most easily understood economic analysis is the present worth method. It is based on exactly the same principles that businessmen use in evaluating discounted cost flows in investment projects. In the present worth method, all cash disbursements that will be made within the analysis period are reduced to their present worth at the beginning of the analysis period. The initial cost, of course, is already in terms of present worth, but annual maintenance costs, resurfacing, resealing, or any other expenditures made at varying periods of time must be reduced to the present worth value by use of the appropriate interest rate and period of time involved. The alternate having the lowest present worth value is the logical choice from an economical standpoint.

Where the alternates under study have unequal total lives, the equivalent uniform annual cost basis is convenient. This method varies from the present method in that lump sum payments including initial costs are converted to uniform annual sums by use of the capital recovery factor and added to the other annual recurring costs, such as maintenance. The alternate having the lowest equivalent uniform annual cost is the logical choice from the standpoint of economics.

These methods are described in any economic textbook. Mr. Charles Dale will go into these methods in detail later on and in the workshop session this afternoon.

During my presentation, I have been discussing analysis period. The analysis period is the period of time selected for making an economic analysis of pavement costs. All the costs associated with a pavement type are analyzed over this period. There is no set length for this period, it may vary from State to State. The analysis period will depend on the method selected for economic comparison, service life of alternate pavement type, and salvage value determinations. The useful life used in an economic analysis of a pavement is a judgement decision in which consideration must be given to both the estimated time the facility will be used as intended, and the estimated time during which more desirable investments will not arise.

It is convenient to use an analysis period equal to the estimated total life of a pavement; however, the period may be shortened provided proper allowance is made for salvage value of the pavement at the end of the period. It is common practice to use analysis periods in the range of 24 to 40 years for new construction and 5 to 20 years for rehabilitation so that at least one resurfacing of rigid pavement will be included and from one to three resurfacings of flexible pavement. In a recent study of their pavement type selection, Minnesota determined that they will use a 35-year analysis period.

We have now come to the final stage of life cycle cost analysis and perhaps the most controversial. During my research on this subject, I came to the conclusion that economists are as prone as engineers to disagree, and may be even more so. The factors which provoke the most discussion among economists and administrators are whether or not discount rates or inflation should be used in an economic analysis and if so, what rate. This is particularly true where public works projects are concerned and the funds are derived from tax revenue.

A discount rate is used to reduce future expected costs or benefits to present-day terms. It provides the means to compare alternative uses of funds, but it should not be confused with interest rates, which is associated with borrowing money.

The actual rate to be used in the agency's calculations is a policy decision. Also, this rate could vary with the element under evaluation to reflect the associated degree of uncertainty. Most agencies, however, use a single rate. In the pavement field, discount rates between about 4 and 10 percent have been typically used.

In a paper at the 1965 Highway Research Board meeting by Lee and Grant, the following factors were listed when considering inflation:

1) Long-term inflation is difficult to forecast.
2) The Federal Government is committed to price stabilization.
3) Future dollars to pay for future expenses will likewise be inflated.

The problem with considering inflation over a long-term period is the difficulty in estimating what the rate is going to be. Lee and Grant list this difficulty in forecasting long-term inflation as a "main reason" for not including an inflation rate in engineering economy studies.

One older study had suggested that a long-term rate of inflation of 2 percent per year was a reasonable figure. As inflation rates over the past several years have been considerably greater than the 2 percent figure, it is appropriate to reconsider these recommendations on inflation.

Recent works suggest the need for considering an inflation factor in engineering economic analysis if market interest rates are used. The interest actually charged is commonly called the market interest rate.

Recent literature suggests a method of taking the rate of inflation into account, without forecasting what it is going to be. This method recognizes that the market interest rate is made up of the anticipated rise in prices (inflation) plus an amount to add a real increase in the purchasing power of the investment. As an example: Assume a friend asks to borrow $100 for one year. You agree, but want to be compensated for not having the use of the $100 for the coming year. You decide that you will charge an interest rate that, one year from now, will enable you to buy $104 worth of goods in today's dollars. The rate would be 4 percent if prices do not rise. However, suppose you anticipate prices to rise 6 percent due to inflation during the year. Then to ensure that you can buy $104 worth of goods (which would cost $104 x 1.06 = $110) you must adjust the interest rate to account for the expected inflation. You would have to charge 4% + 6% = 10% interest.

The interest rate actually charged (10% in the example) is commonly called the "market interest rate." The percentage real increases in purchasing power (4 percent in the example) is referred to as the "real interest rate", the "real cost of capital" or the "real cost of money." The percentage increase in prices is the "inflation rate." The "real interest rate" is nominally equal to the "market interest rate" minus the "inflation rate."

AASHTO in a 1977 report entitled A Manual on User Benefits Analysis of Highway and Bus Transit Improvement recommends using the real cost of money in economic analysis.

That report states:

> "...the common practice of calculating benefits in constant dollars (usually at prices prevailing when the economic study is made) and discounting benefits at market rates of interest is in error, because the market or nominal rate of return includes (1) an allowance for expected inflation, as well as (2) a return that represents the real cost of capital. Thus if future benefits or costs are in constant dollars, only the real cost of capital should be represented in the discount rate used."

AASHTO states that "Constant dollars refers to an expression of costs stated at price levels prevailing at a particular constant date in time..." If you use constant cost and not inflated costs, then the real cost of money should be used in the analysis.

They further state that the real cost of money is a function of the "riskiness" of the investment and "the inflation adjustment is the investor's expectations as to the long term outlook for inflation." If the investor's expectation for inflation was always accurate, then the real cost of money should be virtually constant for investments of the same riskiness. However, because of estimates of future inflation are not always accurate, the real cost of money does in fact, vary somewhat. Fortunately, this variation appears to be relatively small over the long term.

Economics textbooks and recent engineering reports suggest that is is justifiable to use a constant "real interest rate" for economic analyses of investments of a given constant risk. Pavement surfacing is undoubtedly such an investment. What that interest rate should be is discussed in several references.

Hirshleifer and Shapiro state that "...the anticipated real rate of return that enters into investor's calculations has remained in the neighborhood of 4 percent." AASHTO states "...a rate of about 4 to 5 percent seems appropriate for projects of average risk ,evaluated in constant dollars." Based on a study of literature, Minnesota has decided the interest rate used in the economic formula for pavement selection will be the real interest rate of 4.5 percent and that future costs will be expressed in constant dollars, that is, those prices prevailing at the time of the economic study.

Robley Winfrey, an expert in economics, disagrees somewhat and finds it quite acceptable and reasonable to adopt a discount rate which may include a small inflation factor. This position comes from the fact that inflation is hard to detect in the rate and what you would call a pure, real interest rate, that is referred to in the literature so often, cannot be precisely defined. The pure interest is not definable and not identifiable. If you look around the country, the pure interest rate varies with the person and with the geographical location involved. In the end, we see interest rates that are in use, but how much of the rate is pure interest, how much is inflation, and how much risk we do not know. Mr. Winfrey sees rates of 8-9% being used.

If benefits and costs are projected in inflated or "current" dollars, then the full current market rate of interest should be used. A range of 8 percent to 12 percent is common to represent the average long-term market interest rate in recent economy studies of public projects. If current dollar valuations are employed, use of an average rate of inflation for all price changes is recommended unless there are good grounds to expect highly signifiant differences in the rate of price changes for some major resources or benefits of a project. It is felt that the constant dollar approach is preferable, in most cases, to using inflated or current dollars for economic analysis, since it avoids the need for speculation about future inflation in arriving at the economic merit of the project.

A study conducted by the Florida Senate Committe on Transportation recommended that the Florida Department of Transportation should use a cost inflation factor based on the forecast of the firm with which the State Government contracts.

Pennsylvania uses a 6% discount rate. Some economists argue that private interest rates reflect the risk of investment and recommend the use of long term government securities as a basis for the discount rate.

It is obvious that the choice of a discount rate has an important influence on investment decisions. Too low a rate understates the value of current consumption, leading to projects with a high initial cost (and often low maintenance costs). Too high a rate results in less initial investment that is worthwhile often with high maintenance costs.

I have tried to just present some of the arguments. There is no clear-cut answer and if that is what you are expecting, then you are going to be disappointed. I think various interest rates should be used to test sensitivity. I do not like selecting one rate for all projects. I like the three discount calculations presented by Robley Winfrey at a recent conference in Minnesota. This is a system of calculating at three discount rates --- a low, medium, and high. It gives you the significance of the rate. That is, how it affects the answer. And that is important to know. If this three-rate analysis always selects the same alternative, you can safely conclude that a change in the discount rate within a reasonable range will not change the indicated winner in the competition for design.

Now the results of the economic analysis along with the engineering analysis are used in the decisionmaking process to select the proper pavement type. The decision can be made by one individual or can be made by a pavement selection committee. These committees have been set up in a number of States, and are composed of representatives of construction, design, maintenance, materials, etc.

Summary

I have discussed our policy on pavement type selection and a lot of factors related to the engineering and economic analyses. The question is how involved should the division Office be in the pavement type selection process? What is the proper role for FHWA in this element of pavement management? It is not necessary that you review all pavement type determinations. You should insure that the State has adequate procedures. Don't second guess the State.

If they select rigid on a particular project, don't say it should be flexible. Evaluate their procedure.

If a selection is made based on rational established procedures then accept it. Your concern should be with insuring that the State has adequate and established procedures. Your periodic reviews should focus on these procedures to improve them and to eliminate any identified deficiencies. This policy applies to CA as well as non-CA States.

If there is one thing that should be clear from my discussion, it is that we do not have a consensus of opinion or uniformity on life cycle cost factors and issues. It may be desirable to have uniformity, but the fact is that we don't have any very good information and we are in an area where we are still defining the factors. AASHTO has not addressed the subject of life cycle cost yet. If we are ever going to get uniformity, then we must get AASHTO involved.

Because of the lack of guidance on life cycle cost, a research project has been initiated for Fiscal Year 1984 pavement research program. This report is the result of concern by many people in the FHWA field and headquarters offices. The study will evaluate the agency and user costs to consider in life cycle cost analysis.

Life cycle cost analysis is a new concept to the pavement type selection process. I have tried to present some of the background that led to our emphasis on it. There are a lot of unknowns and questions. I would suggest that each State study the subject as thoroughly as Minnesota has in their pavement type selection process. One important point to remember in closing, the selection of a pavement type should not be based solely on economics. The selection has two distinct parts, an engineering analysis and an economic analysis. The results of these analyses should provide the highway administration with the information he needs to substantiate his decision of selection of the best buy for the public dollar.

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10. Spencer, Milton H., Comtemporary Economics, Worth Publishers, Inc. 1974.

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12. Baldock, R. H., "The Annual Cost of Highways," Highway Research Record No. 12, 1963.

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LESSON OUTLINE
ECONOMIC EVALUATION OF ALTERNATIVE PAVEMENT STRATEGIES

Instructional Objectives

1. To discuss the elements of pavement cost analysis.
2. To present the application of economic analysis to evaluate alternative pavement strategies.
3. To present some of the computer pavement design systems.

Performance Objectives

1. The students should be able to perform an economic analysis to evaluate alternate pavement designs.
2. The students should be able to understand the capabilities of the computer pavement design systems.

Abbreviated Summary	Time Allocations, min.
1. Elements of Pavement Cost Analysis	15
2. Application of Economic Analysis	15
3. Automation of the Analysis	15
4. Selection of a Strategy	5
	50 minutes

Reading Assignments

1. Haas & Hudson - Chapter 16, pages 216 to 227
2. Haas & Hudson - Chapter 19, pages 290 to 294
3. RTAC-Canadian Guide - Parts 2 and 3, pages 2.1 to 3.22
4. Instructional Text

Additional Reference

1. Grant, E. L., Ireson, W. G., "Principles of Engineering Economy", Wiley, Sixth Edition, 1976 - Chapter 9, 11, 19.

LESSON OUTLINE
ECONOMIC EVALUATION OF ALTERNATIVE PAVEMENT STRATEGIES

1.0 ELEMENTS OF PAVEMENT COST ANALYSIS

1.1 Initial Capital Cost

The initial capital cost involves the cost of in-place materials in a pavement structure, including quality of material to be provided, and equipment and labor necessary to prepare, place, and finish the pavement structure.

1.2 Rehabilitation Costs

1.2.1 Definition. Rehabilitation cost includes future overlays or upgradings made necessary when the pavement distress, skid resistance, structural capacity, or riding quality reach the limits of acceptability.

1.2.2 Forecasting Rehabilitation. Essential to the determination of resurfacing costs are the algorithms that predict the number of years at which a pavement reaches terminal condition after initial or overlay construction.

1.3 Maintenance Cost

A comprehensive economic analysis should include the estimation of all costs that are essential to maintain the pavement at a desirable level of service and deterioration. Visual Aid 24.1 indicates the effect of maintenance in the performance of a pavement section.

1.4 Salvage Value

Salvage return of a strategy is the value of a pavement at the end of its analysis period. Computation of this cost allows for comparison of designs with different deterioration and serviceability at the end of the analysis period.

1.5 Traffic Delay Cost (Visual Aid 24.2)

Overlay and maintenance operations disrupt traffic flow and cause vehicle speed fluctuations, stops and starts, and time losses. The user cost thus incurred is often a significant portion of the total cost and may warrant its inclusion in the economic analysis.

1.6 Extra Operational Costs (Visual Aid 24.3)

Each pavement design involves costs to the user and that must be included in a rational economic analysis. User costs are related to the serviceability and deterioration history of the pavement.

1.6.1 Vehicle Operating Cost. This cost consists of fuel consumption, tire wear, vehicle maintenance, oil consumption, vehicle depreciation, and parts replacement.

1.6.2 Travel Time Cost. This cost is computed based on the extra travel time a user incurs as a result of a road not being built or a road in poor condition.

1.6.3 Accident Cost. The cost users experience due to accidents because a roadway is not built or improved.

1.6.4 Discomfort Cost. Costs associated with a user experiencing discomfort on a roadway section.

Each of the operational costs is a function of roughness, skid, and deterioration of the road and of the resulting vehicle speed.

2.0 APPLICATION OF ECONOMIC ANALYSIS PRINCIPLES

This section consists of a worked out example which is intended:

(a) To demonstrate the practical application of the economic analysis principles discussed in the previous lecture, and

(b) To stress the difference in results, in comparing alternative pavement designs, when considering the initial pavement cost or the life cycle costs of the alternatives.

2.1 Example Problem on Economic Analysis

Visual Aid 24.4 to 24.6 describe, in summary form, an example problem. Five different pavement design strategies have been analyzed to select the most appropriate from an economic standpoint.

Visual Aid 24.4 presents the five initial thickness designs. Visual Aid 24.5 indicates the overlay timings chosen for each strategy. Finally, Visual Aid 24.6 is a summary of the various cost components; inside a block, the least cost figures have been indicated.

On a first cost basis alternative C appears to be the best solution; however, after considering its life cycle costing, it results in the most expensive alternative. Alternative D, the most expensive on a first cost basis, appears to be the least expensive alternative in the long run.

3.0 AUTOMATION OF ANALYSIS

In order to generate, analyze, and sort all the feasible design strategies, a computer is mandatory.

3.1 Existing Pavement Design Systems

Several pavement design systems have been developed such as SAMP, FPS, RPS, OPPC, RPRDS. They vary widely in details and applications; however, getting acquainted with them can provide ideas for the development of specific pavement design system.

3.2 Sample Input

Visual Aid 24.7 presents a sample input of a pavement design system. The input values correspond to imposed limitations to layer thicknesses and policies, plus material properties, traffic, environmental inputs, and materials and operation costs.

3.3 Sample Output

Visual Aid 24.8 presents a sample output of a pavement design system. The most optimal designs have been printed out, detailing the various cost components, the initial structure, and the timing and thickness of overlays.

4.0 SELECTION OF A STRATEGY

An economic analysis provides a basis for making a management decision to select the best strategy. However, other factors, in addition to economics, may be used to make this judgment.

4.1 Role of the Decision Maker

The results of analyses do not make decisions, this is the role of the "manager." The analyses are efficient tools to expand the scope and efficiency of the decision maker.

4.2 Selecting the Optimal Strategy

The final selection of a design strategy for implementation is largely subjective. No hard-and-fast decision rules exist that can be followed to the letter. The computerized design systems provide the decision maker with a list of nearly optimum strategies so he has enough flexibility in choosing the "best" alternative.

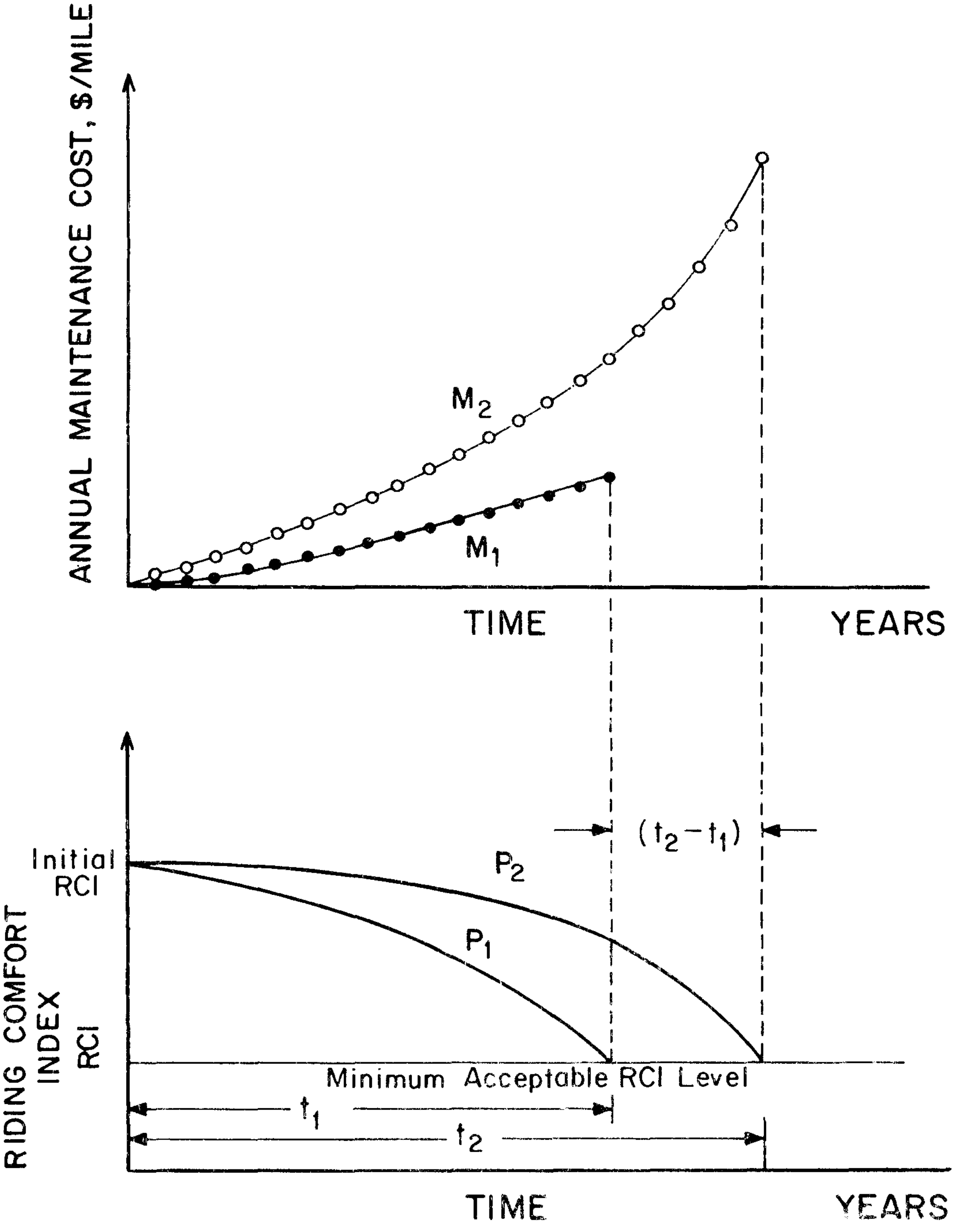
4.3 Communicating Design Strategies

Once an optimal strategy has been selected, the designer should document this strategy for implementation. That is, other phases of pavement management need to become aware of the structural and policy aspects of a selected design.

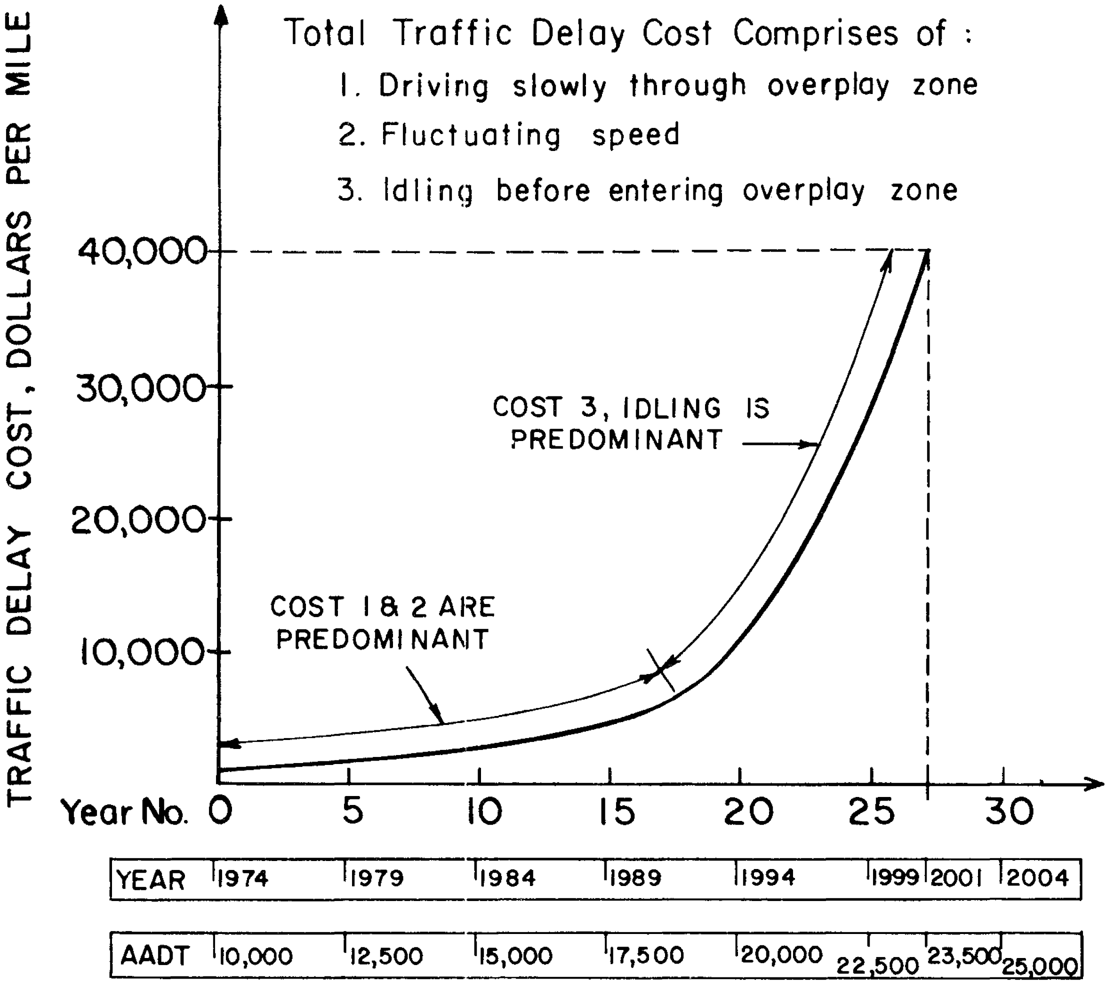
LESSON OUTLINE
ECONOMIC EVALUATION OF ALTERNATIVE PAVEMENT STRATEGIES

VISUAL AID	TITLE
Visual Aid 24.1.	Performance as a function of maintenance.
Visual Aid 24.2.	Traffic delay costs during overlay construction.
Visual Aid 24.3.	User costs related to PSI for rural roads.
Visual Aid 24.4.	Initial thickness designs for the example.
Visual Aid 24.5.	Performance histories for sample designs.
Visual Aid 24.6.	Summary of cost components for the example.
Visual Aid 24.7.	Design system - sample input.
Visual Aid 24.8.	Design system - sample output.

Visual Aid 24.1. Performance as a function of maintenance level.



Visual Aid 24.2. Traffic delay costs during overlay construction.



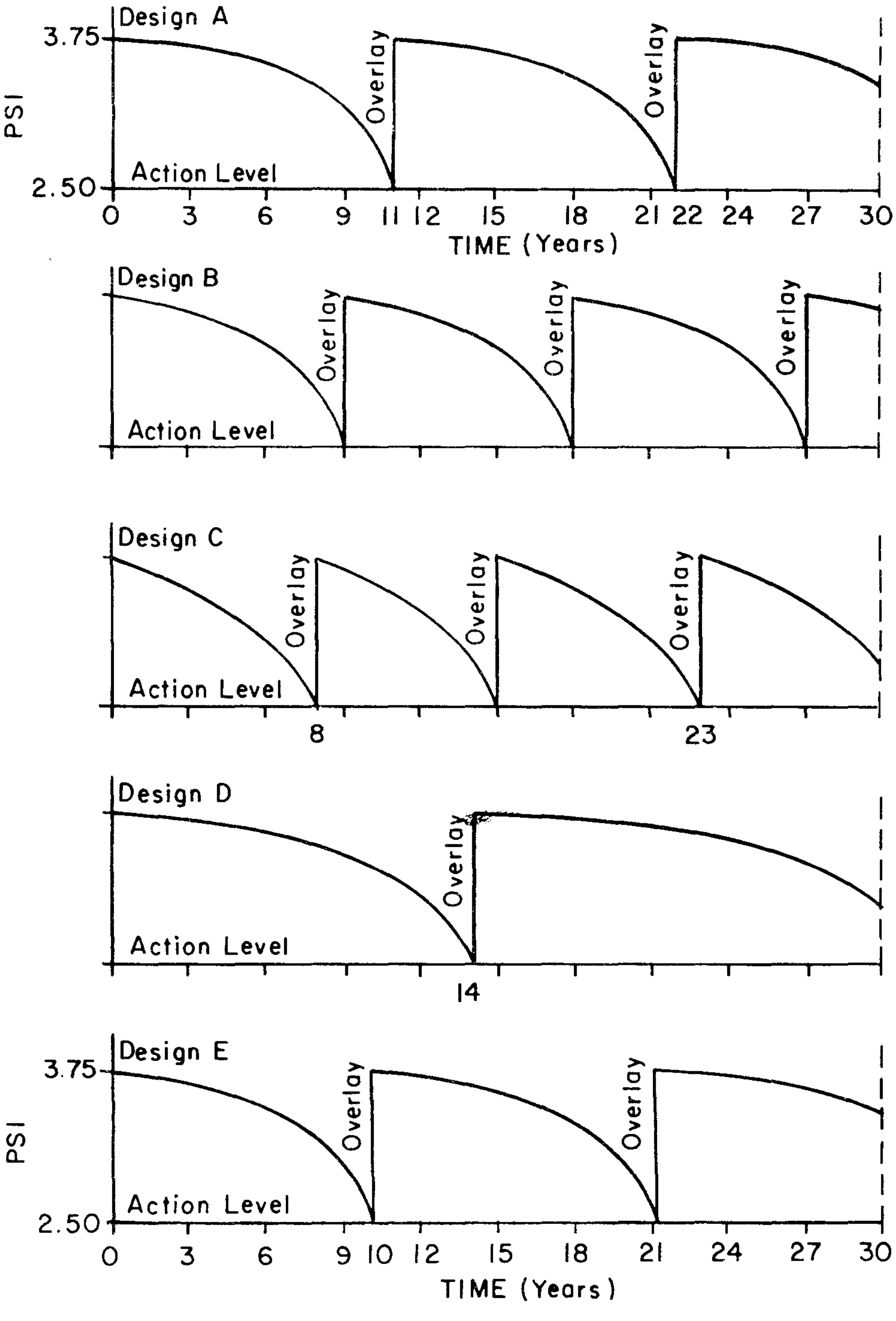
Visual Aid 24.3. User costs related to PSI for rural roads.

Present Serviceability Index	Type of Cost				
	Time	Operating	Accident	Discomfort	Total
			Two Lane Rural Roads		
1.5	9.86	7.95	0.86	2.20	20.87
2.0	8.74	7.84	0.75	1.80	19.13
2.5	7.93	7.73	0.68	1.40	17.74
3.0	7.50	7.37	0.63	0.95	16.45
3.5	7.25	7.06	0.61	0.50	15.42
4.0	7.13	6.75	0.60	0.20	14.68
4.5	7.07	6.58	0.59	0.00	14.24
5.0	7.07	6.43	0.59	0.00	14.09
			Four Lane Rural Roads, Undivided		
1.5	9.75	8.05	2.80	2.25	22.85
2.0	8.57	8.00	2.06	1.90	20.53
2.5	7.71	7.96	1.55	1.50	18.72
3.0	7.25	7.70	1.25	1.05	17.25
3.5	7.01	7.34	1.07	0.55	15.97
4.0	6.84	7.03	1.00	0.20	15.07
4.5	6.73	6.84	0.96	0.00	14.53
5.0	6.73	6.67	0.96	0.00	14.36
			Four or More Lane Rural Roads, Divided		
1.5	9.64	8.14	0.44	2.30	20.52
2.0	8.48	8.10	0.38	1.95	18.91
2.5	7.65	8.05	0.34	1.54	17.58
3.0	7.07	7.97	0.32	1.15	16.51
3.5	6.78	7.65	0.31	0.65	15.39
4.0	6.63	7.30	0.30	0.25	14.48
4.5	6.52	7.11	0.30	0.00	13.93
5.0	6.52	6.88	0.30	0.00	13.70

Visual Aid 24.4. Initial thickness designs for the example.

		Actual Thicknesses, in.		
Design	Description	Surface	Base	Subbase
A	Conventional	5	10	9
B	Conventional	4	6	15
C	Conventional	6	6	6
D	Deep strength	8	12	--
E	Full depth	12.5	--	--

Visual Aid 24.5. Performance histories for example designs



Design	Initial Capital Cost of Pavement a	Overlay Costs b	Subtotal a + b	Maintenance Cost c	Subtotal a + b + c	Extra User Costs During Overlays d	Subtotal a + b + c + d	Salvage Return e	Subtotal a + b + c + d − e	Extra User Costs Due to Speed Reductions f	Total a + b + c + d − e + f
A	$232,950	$49,300	$282,250	$35,100	$317,350	$ 5,100	$322,450	$13,550	$308,900	$ 92,950	$401,850
B	203,950	70,450	$\boxed{274,400}$	31,400	$\boxed{305,000}$	10,300	316,100	15,100	301,000	108,900	409,900
C	$\boxed{198,600}$	81,300	279,900	27,500	307,400	7,100	$\boxed{314,500}$	16,300	$\boxed{298,200}$	140,050	438,250
D	259,350	27,300	296,950	41,400	338,350	1,450	339,800	13,100	326,700	64,350	$\boxed{391,050}$
E	244,850	52,700	297,600	34,300	331,900	4,550	336,450	17,200	319,250	83,750	403,000

☐ = least cost

Visual Aid 24.6. Summary of various cost components (present worth) for the example.

Visual Aid 24.7. Design system - sample input

```
PROB      1                    A SAMPLE PROBLEM

        THE CONSTRUCTION MATERIALS UNDER CONSIDERATION ARE

   MATERIAL          COST/C.Y.   ST.COEF.  MIN.DEPTH  MAX.DEPTH  SALV.PCT.
ASPHALTIC CONCRETE    10.00        .82       1.00       10.00      45.00
CR. LIMESTONE-1        5.00        .55       6.00       16.00      75.00
GRAVEL-1               3.00        .35       6.00       16.00     100.00
SUBGRADE               0.00        .22       0.00        0.00       0.00

  NUMBER OF OUTPUT PAGES DESIRED(8 DESIGNS/PAGE)                              3
  NUMBER OF INPUT MATERIAL TYPES                                              3
  MAX FUNDS AVAILABLE PER SQ.YD. FOR INITIAL DESIGN (DOLLARS)              5.00
  LENGTH OF THE ANALYSIS PERIOD (YEARS)                                    20.0
  INTEREST RATE OR TIME VALUE OF MONEY (PERCENT)                            5.0
  ASPHALTIC CONCRETE PRODUCTION RATE (TONS/HOUR)                           75.0
  ASPHALTIC CONCRETE COMPACTED DENSITY (TONS/C.Y.)                         1.80
  MAXIMUM ALLOWED THICKNESS OF INITIAL CONSTRUCTION (INCHES)               36.0

  DISTRICT TEMPERATURE CONSTANT                                            30.0
  SERVICEABILITY INDEX OF THE INITIAL STRUCTURE                             4.2
  SERVICEABILITY INDEX P1 AFTER AN OVERLAY                                  3.8
  MINIMUM SERVICEABILITY INDEX P2                                           3.0
  SWELLING CLAY PARAMETERS -- P2 PRIME                                     1.50
                                  B1                                      .0800

  ONE-DIRECTION ADT AT BEGINNING OF ANALYSIS PERIOD (VEHICLES/DAY)        12000
  ONE-DIRECTION ADT AT END OF ANALYSIS PERIOD (VEHICLES/DAY)              18000
  ONE-DIRECTION 20-YR ACCUMULATED NO. OF EQUIVALENT 18-KIP AXLES        2000000

  MINIMUM TIME TO FIRST OVERLAY (YEARS)                                     2.0
  MINIMUM TIME BETWEEN OVERLAYS (YEARS)                                     5.0
  MIN TIME TO FIRST SEAL COAT AFTER OVERLAY OR INITIAL CONST.(YEARS)        5.0
  MINIMUM TIME BETWEEN SEAL COATS (YEARS)                                   3.0
  NUMBER OF OPEN LANES IN RESTRICTED ZONE IN O.D.                             1
  NUMBER OF OPEN LANES IN RESTRICTED ZONE IN N.O.D.                           2

  C.L. DISTANCE OVER WHICH TRAFFIC IS SLOWED IN THE O.D. (MILES)            .50
  C.L. DISTANCE OVER WHICH TRAFFIC IS SLOWED IN THE N.O.D. (MILES)          .50
  PROPORTION OF ADT ARRIVING EACH HOUR OF CONSTRUCTION (PERCENT)            6.0
  OVERLAY CONSTRUCTION TIME (HOURS/DAY)                                    10.0
  THE ROAD IS IN A RURAL AREA.

  PROPORTION OF VEHICLES STOPPED BY ROAD EQUIPMENT IN O.D. (PERCENT)        2.0
  PROPORTION OF VEHICLES STOPPED BY ROAD EQUIPMENT IN N.O.D. (PERCENT)      0.0
  AVERAGE TIME STOPPED BY ROAD EQUIPMENT IN O.D. (HOURS)                   .100
  AVERAGE TIME STOPPED BY ROAD EQUIPMENT IN N.O.D. (HOURS)                 .100
  AVERAGE APPROACH SPEED TO THE OVERLAY ZONE (MPH)                         60.0
  AVERAGE SPEED THROUGH OVERLAY ZONE IN O.D. (MPH)                         40.0
  AVERAGE SPEED THROUGH OVERLAY ZONE IN N.O.D. (MPH)                       55.0
  TRAFFIC MODEL USED IN THE ANALYSIS                                          3

  FIRST YEAR COST OF ROUTINE MAINTENANCE (DOLLARS/LANE MILE)              50.00
  INCREMENTAL INCREASE IN MAINT. COST PER YEAR (DOLLARS/LANE MILE)        20.00
  COST OF A SEAL COAT (DOLLARS/LANE MILE)                               1500.00
  WIDTH OF EACH LANE (FEET)                                               12.00
  MINIMUM OVERLAY THICKNESS (INCHES)                                         .5
  ACCUMULATED MAXIMUM DEPTH OF ALL OVERLAYS (INCHES)                        8.0
```

Visual Aid 24.8. Design system - sample output.

```
  PROB      1                  A SAMPLE PROBLEM

                         SUMMARY OF THE MOST OPTIMAL DESIGNS
                          IN ORDER OF INCREASING TOTAL COST

                                1      2      3      4      5      6      7      8
*********************************************************************************
DESIGN NUMBER                   3      3      2      3      3      2      2      3
INIT. CONST. COST           2.000  2.278  1.944  2.306  2.347  2.222  2.292  2.375
OVERLAY CONST. COST          .882   .543   .882   .532   .517   .543   .517   .517
USER COST                    .203   .125   .203   .123   .120   .125   .120   .121
SEAL COAT COST               .233   .384   .233   .380   .374   .384   .374   .374
ROUTINE MAINT. COST          .166   .190   .166   .190   .192   .190   .192   .192
SALVAGE VALUE               -.679  -.711  -.612  -.715  -.730  -.644  -.663  -.734
*********************************************************************************
*********************************************************************************
TOTAL COST                  2.804  2.810  2.816  2.816  2.821  2.821  2.832  2.845
*********************************************************************************
*********************************************************************************
NUMBER OF LAYERS                3      3      2      3      3      2      2      3
*********************************************************************************
LAYER DEPTH (INCHES)
     D( 1)                    1.0    1.0    1.0    1.0    1.0    1.0    1.0    1.0
     D( 2)                    8.5   10.5   12.0   11.0   11.0   14.0   14.5   11.5
     D( 3)                    6.5    6.5    0.0    6.0    6.5    0.0    0.0    6.0
*********************************************************************************
*********************************************************************************
NO.OF PERF.PERIODS              4      3      4      3      3      3      3      3
*********************************************************************************
PERF. TIME (YEARS)
     T( 1)                  4.906  6.250  4.969  6.406  6.531  6.281  6.563  6.656
     T( 2)                  9.945 12.383 10.008 12.773 13.094 12.453 13.164 13.453
     T( 3)                 16.195 20.039 16.336 20.703 21.336 20.148 21.445 21.969
     T( 4)                 23.812  0.000 24.031  0.000  0.000  0.000  0.000  0.000
*********************************************************************************
OVERLAY POLICY(INCH)
(EXCLUDING LEVEL-UP)
     O( 1)                    1.0     .5    1.0     .5     .5     .5     .5     .5
     O( 2)                     .5     .5     .5     .5     .5     .5     .5     .5
     O( 3)                     .5    0.0     .5    0.0    0.0    0.0    0.0    0.0
*********************************************************************************
NUMBER OF SEAL COATS            2      3      2      3      3      3      3      3
*********************************************************************************
SEAL COAT SCHEDULE
   (YEARS)
   SC( 1)                   9.906  5.000  9.969  5.000  5.000  5.000  5.000  5.00
   SC( 2)                  14.945 11.250 15.008 11.406 11.531 11.281 11.563 11.65
   SC( 3)                   0.000 17.383  0.000 17.773 18.094 17.453 18.164 18.45
*********************************************************************************
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INSTRUCTIONAL TEXT

ECONOMIC ANALYSIS

Introduction

The application of principles of engineering economy to pavement management occurs basically at two levels. First, there are the management decisions required to determine the feasibility and timing of a series of projects; second there is the requirement to achieve the maximum economy within each project. Project feasibility is determined at the network level, by comparison with other potential projects, whereas within-project economy is achieved by considering a variety of alternatives capable of satisfying the overall project requirements.

The only major economic evaluation difference between these two levels of pavement management concerns the specific type and amount of detail of information required. The basic principles involved are the same. This unit considers both these principles and their incorporation into models or methods of economic evaluation. Such models are a vital part of the pavement management system.

Decision Criteria and Constraints

Every highway agency faces constraints which limit the size and scope of the services they may provide. The most visible of these constraints is generally economic: the available budget for a district, department or program. Many other constraints are usually present, though, including limits on manpower, materials and equipment, minimum service levels to be maintained, stability of manpower and equipment usage in each department over several years, environmental limitations, length of construction season, testing capabilities, minimum time between overlays, and so on. No strategy can be approved unless all applicable constraints are met, so a primary function of the analysis methodology is to test proposed strategies against a complete set of constraints.

Decision Criteria and Selection. The decision criteria applied to the various alternatives from the project analysis subsystem, in order to select the best one, may involve both quantitative and non-quantitative factors. These factors should reflect the needs of the network as perceived by the decision maker. A least cost or maximun benefit alternative may be selected, or previous experience, judgment, etc., may be combined with the economic based criterion.

Decision Criteria and Budget Constraints Applied. The decision criteria and budget constraints applied to the initial program resulting from the network analysis subsystem may simply involve a selection of those projects and that maintenance program which can be done within some available budget. This budget may have been fixed at the higher management level, or several alternative budget levels may be considered.

The projects or parts of the maintenance program falling below the budget cutoff would then be put back on the candidate list for consideration the following year.

Some agencies designate separate budgets for new construction, rehabilitation and maintenance, while others, for example, have new construction projects "compete" with rehabilitation projects. As well, some transportation departments allocate budgets by region or district.

The non-quantitative aspects of the decision criteria might involve, for example, an engineering judgment to move a project up in the priority list, or political decisions to include certain projects.

Basic Principles of Economics

A considerable amount of literature is available on the principles of engineering economy and methods of economic evaluation see for example References 1-3, 11, 12, 14. Those principles that are applicable to pavement management may be summarized as follows:

1. The management level at which the evaluation is being performed should be clearly identified.

2. The economic analysis provides the basis for a management decision but does not by itself represent a decision. Criteria, rules, or guides for such decisions must be separately formulated before the results of the economic evaluation are applied.

3. The economic evaluation itself has no relationship to the method or source of financing a project. Such financing considerations can either limit the number of feasible projects, or limit the amount available for a particular project, but they do not affect the methodology or principles controlling the economic evaluation.

4. An economic evaluation should consider all possible alternatives, within the constraints of time and other planning and design resources. This includes the need for comparing alternatives not only with a base or existing situation but also with each other.

5. All alternatives should be compared over the same time period. This time period should be chosen so that the factors involved in the evaluation can be forecast with some reasonable degree of reliability.

6. The economic evaluation of pavements should include agency costs and user costs, and benefits if possible.

Pavement Cost and Benefit Factors

Many economic factors should be considered in planning pavement investments. These factors include both costs and benefits associated with alternative strategies. Not all costs and benefits may be included in a particular economic analysis, however, for the following reasons:

- Not all costs and benefits are easily quantifiable. Non-quantifiable factors are excluded from the analysis even though they may be important.

- Some measures of benefit involve primarily non-economic factors and are treated during the technical analysis.

- Time and budget constraints may not allow detailed consideration of more than a handful of economic factors for each alternative strategy.

- Some factors, though quantifiable and important, may not vary appreciably among the possible alternative strategies under consideration and may therefore be excluded.

In general, costs and benefits employed in pavement management may be classified in three groups: (1) factors affecting the transportation agency, such as maintenance costs; (2) factors affecting the road user, such as vehicle operating costs; and (3) factors affecting the public in general, such as changes in the prices of transported good. As a general rule, selected factors from only the first two categories are used in the economic analysis for pavement management. The third category is, however, recognized to some extent by decision-makers, and is indirectly included in the decision process in a non-quantitative manner.

<u>Identification</u> <u>of</u> <u>Pavement</u> <u>Costs</u>. The major initial and recurring costs that a highway agency may consider in the economic evaluation of alternative pavement strategies include the following:

1. Agency costs:

    a. Initial capital cost of construction
    b. Future capital costs of construction or rehabilitation (overlays, seal coats, reconstruction, etc.)
    c. Maintenance costs, recurring throughout the design period
    d. Salvage return or residual value at the end of the design period (which may be a "negative cost")
    e. Engineering and administrative
    d. Costs of investments

2. User Costs:

    a. Travel time
    b. Vehicle operation
    c. Accidents
    d. Discomfort
    e. Time delay and extra vehicle operating costs during resurfacing or major maintenance

3. Nonuser costs (Ref. 4)

Identification of Pavement Benefits. The benefits of a transport project can accrue from direct or indirect cost reductions, and from advantages or gains in business, land use and values, aesthetics, and community activities in general. Pavement benefits would accrue primarily from direct reductions in transportation costs of the user.

In order to measure or calculate pavement benefits, it is necessary to define those pavement characteristics that will affect user costs. These could include level of serviceability, slipperiness, light reflection characteristics, appearance, color, etc. However, the first two factors of serviceability (as it affects vehicle operating costs, travel time costs, accident costs, and discomfort costs) and slipperiness (as it affects accident costs) would have the major influence.

Figure 14.1 is a schematic representation of the effects of different pavement design strategies on user costs. Considering only the variation in serviceability, for example, the diagram shows the following:

1. As serviceability decreases, travel time costs increase because average travel speed decreases (in a nonlinear manner).
2. When rehabilitation occurs (i.e., major maintenance, resurfacing, or reconstruction), high travel time costs can occur because of traffic delays.

The other three components of user costs, shown aggregated, also illustrate two major points:

1. As pavement serviceability approaches a terminal level, user costs increase at an increasing rate.
2. Pavement strategies that do not call for surfacing or other rehabilitation until a lower limit of terminal serviceability is reached will result in higher user costs.

Quantification of User Costs and Benefits for Pavement Projects. A considerable amount of reference material is available on user cost data for various highway types and design characteristics. Those relating to vehicle operation can be found in such sources as the AASHO Red Book (Ref. 6), which is now somewhat outdated, Winfey (Ref. 3), and Claffey (Ref. 7). A recent study by the Stanford Research Institute (Ref. 8) has updated and expanded these sources. In addition, there is a variety of material available on the costs of travel time and accidents. However, it was McFarland (Ref. 1) who first quantified the effects of varying pavement serviceability on user costs, providing the information required to

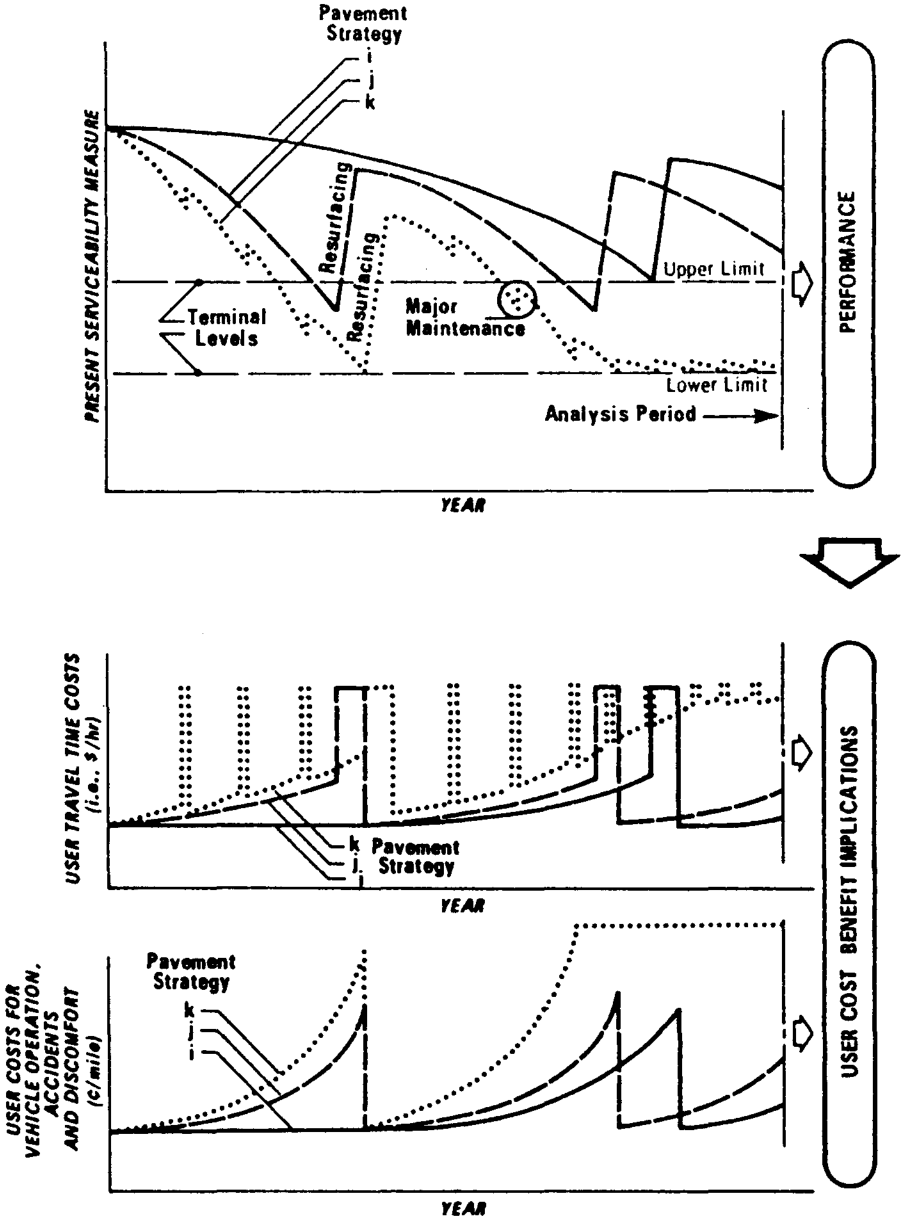


Figure 14.1. Effects on user costs of pavement strategies with varying performance profiles

evaluate pavement benefits. More recently, an extensive UNDP-sponsored study in Brazil (Ref. 16), and a similar research effort in Kenya (Ref. 17), have produced considerable data relating user costs to roadway roughness.

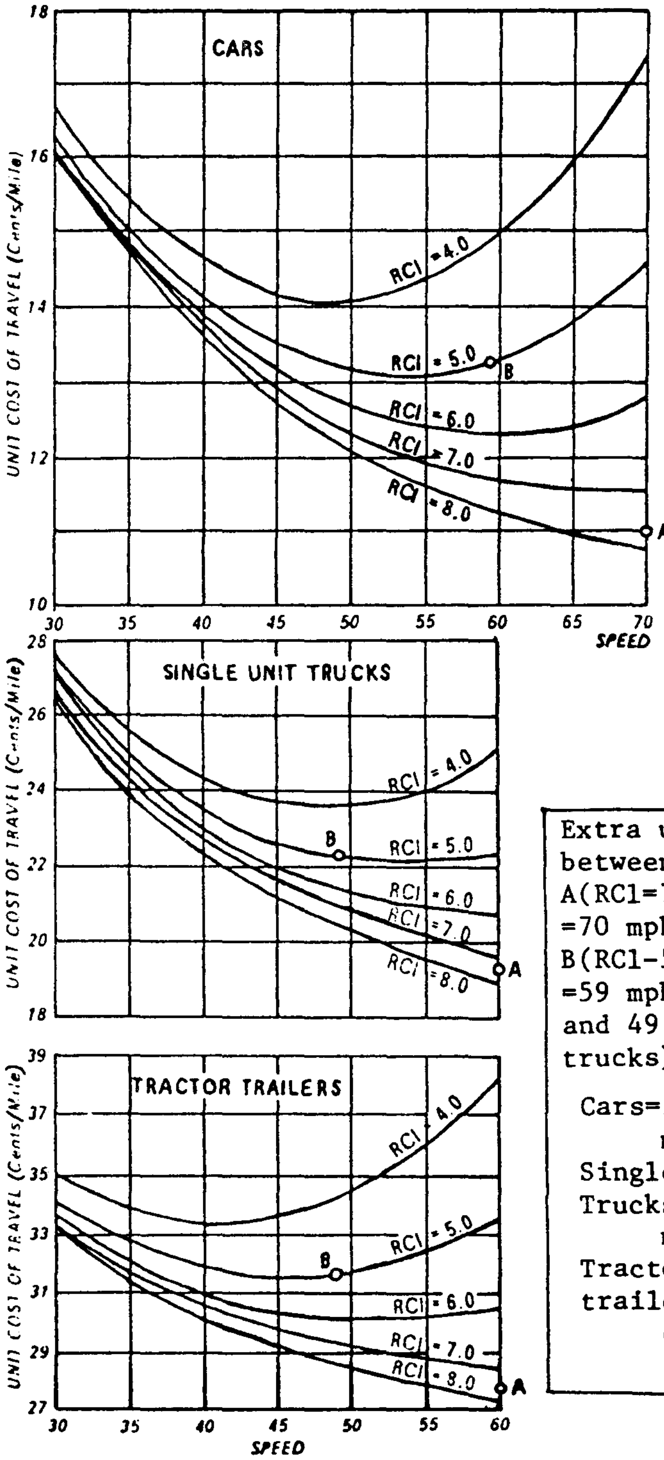
An example of the quantification of user cost variation with pavement serviceability and speed is shown in Figure 14.2 (Ref. 2). The measure of present serviceability is the Canadian Riding Comfort Index (RCI).

A portion of McFarland's original data is given in Tables 14.1 and 14.2. Table 14.1 is for urban roads, without including accident costs. Also not included are extra costs associated with changing speeds and stopping at traffic lights or stop signs. These extra costs should, however, be independent of any pavement characteristics.

Table 14.2 is for rural roads, with accident costs included. Because the data for both rural and urban condition relate to only one (average) speed for each level of serviceability, McFarland also developed data to show the variation of total user costs with varying average speed (Ref. 1).

Example. Consider a 10-mi. portion of two-lane rural highway that has tentatively been programmed for resurfacing this year. It is desired to calculate the extra user costs incurred if the project is delayed for one year. This delay could be achieved by sufficient maintenance to keep the serviceability index at its present level of 2.0. Average daily traffic on the road over the year is expected to be 2,000 vehicles. Resurfacing is expected to raise the serviceability index to 3.5.

Table 14.2 shows total user costs of 19.13 and 15.42 cents per vehicle mile for serviceability index levels of 2.0 and 3.5, respectively. This would result in total user costs of 0.1913 x 10 x 2,000 x 365 = \$1,396,490 for the nonresurfaced case and 0.1542 x 10 x 2,000 x 365 = \$1,125,660 for the resurfaced case. The difference, \$270,830 in total or



Extra unit cost between points A(RC1=7.5, Speed =70 mph) and B(RC1-5.0, Speed =59 mph for cars and 49 mph for trucks) is:

Cars=1.75 cents mile
Single Unit Trucks=3.0 cents mile
Tractor trailers=3.75 cents mile

Figure 14.2. Unit costs (vehicle operating + travel time) of speed reductions at different RCI levels

Table 14.1. User Costs in Cents per Vehicle Mile Related to Present Serviceability Index for Urban Roads

Present serviceability index	Type of cost			
	Time	Operating	Discomfort	Total
	two lane Urban roads			
1.5	24.74	5.90	0.70	31.34
2.0	19.80	5.43	0.53	25.76
2.5	17.22	5.03	0.40	22.65
3.0	16.50	4.91	0.20	21.61
3.5	15.84	4.86	0.08	20.78
4.0	15.84	4.83	0.00	20.67
4.5	15.84	4.82	0.00	20.66
5.0	15.84	4.82	0.00	20.66
	four lane Urban roads, undivided			
1.5	18.00	5.60	0.93	24.53
2.0	14.67	5.30	0.75	20.72
2.5	12.77	5.10	0.55	18.42
3.0	12.00	4.94	0.35	17.29
3.5	11.65	4.85	0.13	16.63
4.0	11.48	4.77	0.02	16.27
4.5	11.31	4.74	0.00	16.05
5.0	11.31	4.73	0.00	16.04
	four or more lane Urban roads, divided			
1.5	13.66	5.66	1.25	20.57
2.0	11.65	5.45	1.05	18.15
2.5	10.42	5.26	0.75	16.43
3.0	9.66	5.12	0.45	15.23
3.5	9.21	5.02	0.20	14.43
4.0	9.00	4.94	0.05	13.99
4.5	8.80	4.88	0.00	13.68
5.0	8.80	4.84	0.00	13.64

After McFarland (1).

Table 14.2. User Costs in Cents per Vehicle Mile Related to Present Serviceability Index for Rural Roads

Present serviceability index	Type of cost				
	Time	Operating	Accident	Discomfort	Total
			two lane Rural roads		
1.5	9.86	7.95	0.86	2.20	20.87
2.0	8.74	7.84	0.75	1.80	19.13
2.5	7.93	7.73	0.68	1.40	17.74
3.0	7.50	7.37	0.63	0.95	16.45
3.5	7.25	7.06	0.61	0.50	15.42
4.0	7.13	6.75	0.60	0.20	14.68
4.5	7.07	6.58	0.59	0.00	14.24
5.0	7.07	6.43	0.59	0.00	14.09
			four lane Rural roads, undivided		
1.5	9.75	8.05	2.80	2.25	22.85
2.0	8.57	8.00	2.06	1.90	20.53
2.5	7.71	7.96	1.55	1.50	18.72
3.0	7.25	7.70	1.25	1.05	17.25
3.5	7.01	7.34	1.07	0.55	15.97
4.0	6.84	7.03	1.00	0.20	15.07
4.5	6.73	6.84	0.96	0.00	14.53
5.0	6.73	6.67	0.96	0.00	14.36
			four or more lane Rural roads, divided		
1.5	9.64	8.14	0.44	2.30	20.52
2.0	8.48	8.10	0.38	1.95	18.91
2.5	7.65	8.05	0.34	1.54	17.58
3.0	7.07	7.97	0.32	1.15	16.51
3.5	6.78	7.65	0.31	0.65	15.39
4.0	6.63	7.30	0.30	0.25	14.48
4.5	6.52	7.11	0.30	0.00	13.93
5.0	6.52	6.88	0.30	0.00	13.70

After McFarland (1)

about $27,000 per mile, represents the savings to users that could be realized by resurfacing this year.

Such user savings or benefits, plus savings in maintenance through resurfacing this year instead of next, may be compared to the costs of resurfacing this year. To be accurate, the savings should be reduced by the difference between the present worth of next year's resurfacing costs and this year's resurfacing costs. If the net benefits exceed the costs of resurfacing, then such resurfacing should occur this year. Actually, benefits should be calculated over the expected service period of the resurfacing and discounted to present worth. These total net benefits might well exceed costs, especially in the example given. However, normally budget constraints limit an agency to programming investments only for those projects yielding the highest net benefits. Even if the example were economically justifiable, it might still have to be delayed one or more years because of such budget constraints.

For roads with high traffic volumes, user delay costs due to resurfacing can be appreciable and can significantly affect the programming of such rehabilitation.

Analysis Period

A general guideline for selecting the length of analysis period is that it should extend over the expected service life of the improvement, but should not extend beyond the period of reliable forecasts. Some tradeoff between these two goals may be needed in the event that the former exceeds the latter. For traffic, 20 years is often used as an upper limit. For other factors, 30 years may not be unreasonable; however, the present worth of costs or benefits at such future times may be insignificant, depending on the discount rate used. In general, an analysis period in the range of 10 to 30 years is reasonable. The particular period chosen is basically a policy decision for the agency concerned and can vary with a number of factors.

The choice of a relatively short analysis period may be partially compensated through considertion of salvage value.

Discount Rate and Interest Rate

A discount rate is used to reduce future expected costs or benefits to present-day terms. It provides the means to compare alternative uses of funds, but it should not be confused with interest rate, which is associated with borrowing money.

The actual rate to be used in the agency's calculations is a policy decision. Also, this rate could vary with the element under evaluation to reflect the associated degree of uncertainty. Most agencies, however, use a single rate. In the pavement field, discount rates between about 4 and 10 percent have typically been used. It should be emphasized that discount rate is a highly significant factor and can have a major influence on the results of an economic analysis.

The discount rate does not include consideration of inflation. In fact, inflation is not generally recommended for inclusion in economic evaluation of pavement strategies. This is due to several factors, such as the difficulty in forecasing inflation rates and the balancing effect of the inflation of both benefits and costs.

Salvage or Residual Value

Salvage or residual value is used by some agencies in economic evaluation. It can be significant in the case of pavements because it involves the value of reuseable materials at the end of the design period. With depleting resources, such materials can become increasingly important in the future, especially when used in a new pavement by reworking or reprocessing.

Salvage value of a material depends on several factors such as volume and position of the material, contamination, age, or durability, anticipated use at the end of the design period, and so on. It can be represented as a percentage of the original cost or as an estimate of the benefit remaining due to previous improvements, or in a number of other ways.

Methods of Economic Evaluation

There are a number of methods of economic analysis that are applicable to the evaluation of alternative pavement strategies. They can be categorized as follows:

1. Equivalent uniform annual cost method, or the annual cost method

2. Present worth method for:

    a. Costs
    b. Benefits
    c. Benefits minus costs, usually termed the net present worth or net present value method

3. Rate-of-return method

4. Benefit-cost ratio method

5. Cost-effectiveness method

These methods have the common feature of being able to consider future streams of costs or of costs and benefits, so that alternative investments may be compared. Differences in the worth of money over time, as reflected in the compound interest equations used, provide the means for such comparisons.

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LESSON OUTLINE

PAVEMENT INVENTORY DATA COLLECTION, EQUIPMENT AND PROCESSING

Instructional Objective

1. To introduce the student to the inventory data system approach concept.
2. To explain all the data needed for implementing the pavement data system.
3. To provide the general picture about the pavement management system in the planning stage.

Performance Objective

1. The student should be able to understand the concept of the inventory data.

Abbreviated Summary	Time Allocations, min.
1. Pavement Inventory Data Collection	15
2. Major Types of Pavement Evaluation	25
3. Inventory Data Processing	10
	50 minutes

Reading Assignment

1. RTAC-Canadian Guide - Part 4 and Part 8
2. Haas & Hudson - Chapter 23
3. Instructional Text

LESSON OUTLINE
PAVEMENT INVENTORY DATA COLLECTION, EQUIPMENT, AND PROCESSING

1.0 MONITORING AND EVALUATION

1.1 Distinction

1.1.1 Monitoring. Monitoring is taking measurements or observations.

1.1.2 Evaluation. Making a judgment based on measurements or observations.

1.2 Purposes (Slides 25.1 - 25.5)

(a) Provide data base for all pavement management functions.
(b) Provide information for improving design, construction, and maintenance Practices and standards.

2.0 TYPES OF EVALUATION

2.1 Roughness - Serviceability

2.2 Structural

(a) strength or deflection, and
(b) cores for thickness of layers.

2.3 Distress or Surface Condition (Transverse Profile and Rut Depths)

3.0 PAVEMENT INVENTORY DATA COLLECTION

3.1 Data Collection (Visual Aid 25.1) (Slides 25.6 and 25.7)

The first basic requirement for data collection is some reference system for identifying locations. It is desirable to have a common location indexing scheme across an agency so data from planning, construction, maintenance, etc., can be linked.

(a) geographical coordinates,

(b) highway number, section and mileage, and

(c) contract or project number and mileage.

3.2 File Structure (Visual Aid 25.2)

There are several alternative ways of creating a file structure for the data items, but they should relate to the basic structure of the pavement management system involved.

3.2.1 Master File. As-built data on geometry of the pavement structure, location, dates, material quantities, and quality control data, costs, subgrade type and properties, shoulders, etc. (Slide 25.8)

3.2.2 Evaluation File. Periodic data on deflection, roughness, condition, skid resistance, traffic, environment, etc. (Slide 25.9)

3.2.3 Maintenance and Rehabilitation File. Periodic data on maintenance types, locations and costs; also data on the location, dates, geometry, material quantities, and costs, quality control, etc. of rehabilitation (i.e., overlays, seal coats, etc.). (Slide 25.10)

3.3 Guidelines for Selecting Inventory Sections and Frequency of Measurements (Slide 25.11 and 25.12)

The section of inventory sections for periodic pavement evaluation and the frequency of measurements required involve a number of considerations.

(a) type of measurement (i.e., roughness, skid resistance, deflection, etc.),

(b) type of facility (i.e., freeway, local road etc.),

(c) purpose of measurement (i.e., for detailed project evaluation, for mass inventory at the network levels, etc.),

(d) users of evaluation information (design, maintenance or construction people, administrators, researchers),

(e) resources of the agency,

(f) age and condition of the section,

(g) physiographic and topographic features of the area traversed by the route,

(h) traffic and geometric conditions of the route, and

(i) maintenance history on the route.

3.4 Selecting Section and Subsection Boundaries and Length (Slide 25.13 & 25.14)

Inventory sections should be relatively homogeneous over their length with respect to traffic and roadway geometrics.

3.4.1 Basis for Selecting Boundaries of the Section (Slide 25.16)

(a) beginning and end of original construction contract,

(b) intersection with another major facility, and for a major change in traffic volume, and

(c) beginning or end of maintenance district or county.

3.4.2 Basis for Selecting Boundaries of Subsections. (Visual Aid 25.3) Subsections are usually selected within these sections for roughness, condition survey, structural and skid resistance measurement purposes. (Slide 25.15)

(a) section beginning or end,

(b) major change in subgrade soil type or drainage characteristics, and

(c) change in pavement structure (thickness and/or type).

3.5 Selecting Frequency of Measurement (Visual Aid 25.4)

It is not possible to develop absolute standards for the frequency with which evaluation measurements should be taken. Nevertheless, it is possible to develop some very general guidelines related to mass inventory evaluation, as shown in Visual Aid 25.4.

3.6 Indexing Sections, Subsections and Measurements

It is imperative that evaluation measurements be properly indexed by section and subsection, for efficient data management.

3.6.1 Sections. By geographical coordinates (i.e., geocoding), or by contract number, or by highway number and mileage, offset from a landmark.

3.6.2 Subsections. By geographic coordinates, or by assigned number within section identification, or by mileage within section identification.

3.6.3 Measurements. In addition to by date, by subsection as a whole, or by graphic coordinates, or mileage within subsection (where precise location is desired).

3.6.4 Compatibility. It is highly desirable that evaluation sections and subsections be located and indexed so they are completely compatible with design, construction, and maintenance.

4.0 MAJOR TYPES OF PAVEMENT EVALUATION

The evaluation of pavements can involve one or more of the following: structural capacity, physical deterioration or distress, user-related factors such as riding comfort, safety and appearance, and user related costs and benefits associated with varying serviceability and with various rehabilitation measures.

4.1 Evaluating Pavement Structural Capacity

Pavement structures can be divided into three separate classes for the evaluation of structural capacity; a rigid pavement structure, a composite pavement structure, and a flexible pavement structure.

4.1.1 Laboratory Tests. These tests include grain-size distribution, density, and moisture content. In addition, the properties of the pavement can be determined by use of split tensile tests, compression tests, etc. In order to perform many of these tests in the laboratory, extensive investment in facilities is required.

4.1.2 California Bearing Ratio (CBR). For many problems it is possible to obtain an estimate of the strength of a subgrade from original CBR tests that were made prior to the original design and construction. However, densification of the road under traffic coupled with environmental factors of ten make these estimates unreliable.

4.1.3 Plate Bearing Tests. These tests on in-service pavements require that test pits of substantial size be dug, and, hence, this type of test is time consuming and often expensive.

4.1.4 Non-destructive Field Tests. Except deflection measurement instruments, there are methods of evaluating structural adequacy with instruments which apply vibratory forces to the pavement, and then by means of velocity transducers, the response of the pavement is measured.

4.1.5 In-place Density and Moisture Tests. Moisture and density data can be obtained using standard sand cone, water balloon, or nuclear techniques.

4.1.6 The Benkelman Beam. (Visual Aid 25.5) The Benkelman Beam Test has been used to quantify experience, simplify theory, and assess construction quality control. This test was developed for the purpose of measuring pavement deflections under static wheel loads.

4.1.7 The Dynaflect. (Visual Aid 25.6 and 25.7) The system provides rapid and precise measurements of roadway deflections at five points on the pavement surface using a cyclic force of known magnitude and frequency which is applied to the pavement through two steel wheels.

4.2 Evaluating Pavement Serviceability

The serviceability of a pavement is largely a function of its roughness. There are several methods for measuring serviceability at any particular time, these will be covered in detail in future lessons.

(a) Measuring riding comfort index by panel rating procedure (Visual Aid 25.8).
(b) Measuring pavement roughness by car road meter (Visual Aid 25.9 and 25.10).
(c) Measuring pavement roughness by more precise or sophisticated methods (Visual Aid 25.11).

  (1) U.S. Bureau of Public Road type of roughometer (BPR)
  (2) CHLOE type profilometer CHLOE
  (3) Rolling Straightedge (RSE)
  (4) British Transport and Road Research Laboratory type of profilometer (TRRL)
  (5) Surface Dynamic Profilometer (SDP)
  (6) Precise levelling method for profile determination (LEVEL)

4.3 Evaluating Pavement Safety

The evaluation of pavement safety is usually thought of in terms of skid resistance. However, there are several safety components. Skid resistance will be covered in detail in future lessons.

(a) Skid resistance measurements and methods (Visual Aid 25.12).
(b) Surface ruts.
(c) Pavement color, light reflectivity and lane demarcation.

4.4 Evaluating Pavement Distress

The section is primarily concerned with condition surveys used to periodically measure and evaluate distress.

4.4.1 Condition Survey Approach. Condition survey, together with RCI, skid resistance and other measurements, are used in determining the maintenance needed to prevent accelerated future distress, or to determine the rehabilitation measures needed.

4.4.2 Components of condition surveys and procedures for field measurements. (Visual Aid 25.13)

(a) surface defects,

(b) permanent deformation or distortion,

(c) cracking, and

(d) patching.

4.5 Traffic and Load Data

Traffic data is essential for investment programming and design purposes. It is also required for certain aspects of construction and maintenance functions. Volumes, loads and classifications of traffic need to be known so costs and benefits can be evaluated in investment programming and in project economic evaluation, so structural designs can be analyzed and construction and maintenance operations properly scheduled. It is beyond the scope of this lecture to describe these data in detail.

5.0 INVENTORY DATA PROCESSING

The development of computer programs for data editing, storage, updating and retrieval is a main part of a pavement data system.

5.1 Basic Functional Requirements of a Pavement Data System. (Visual Aid 25.14) (Slides 25.17 - 25.20)

(a) planning requirements,

(b) design requirements,

(c) construction requirements,

(d) maintenance requirements, and

(e) research requirements.

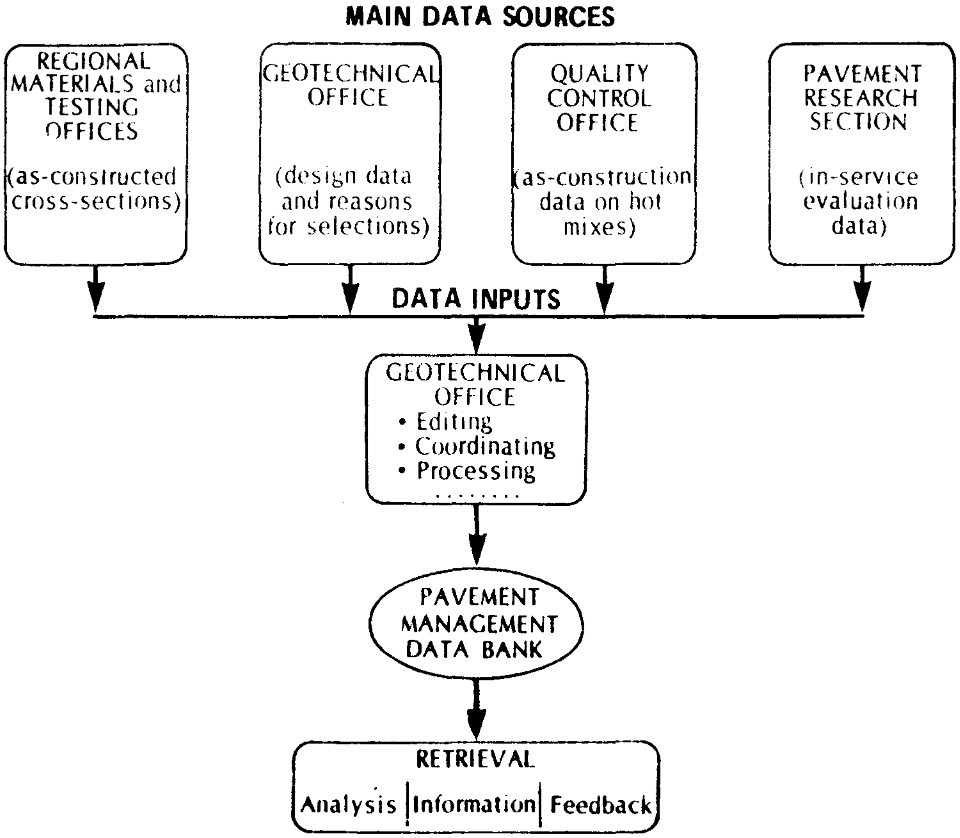
5.2 Basic Steps in Development and Implementation (Visual Aid 25.15)

Visual Aid 25.15 gives a comprehensive development plan for implementing a data system into pavement management.

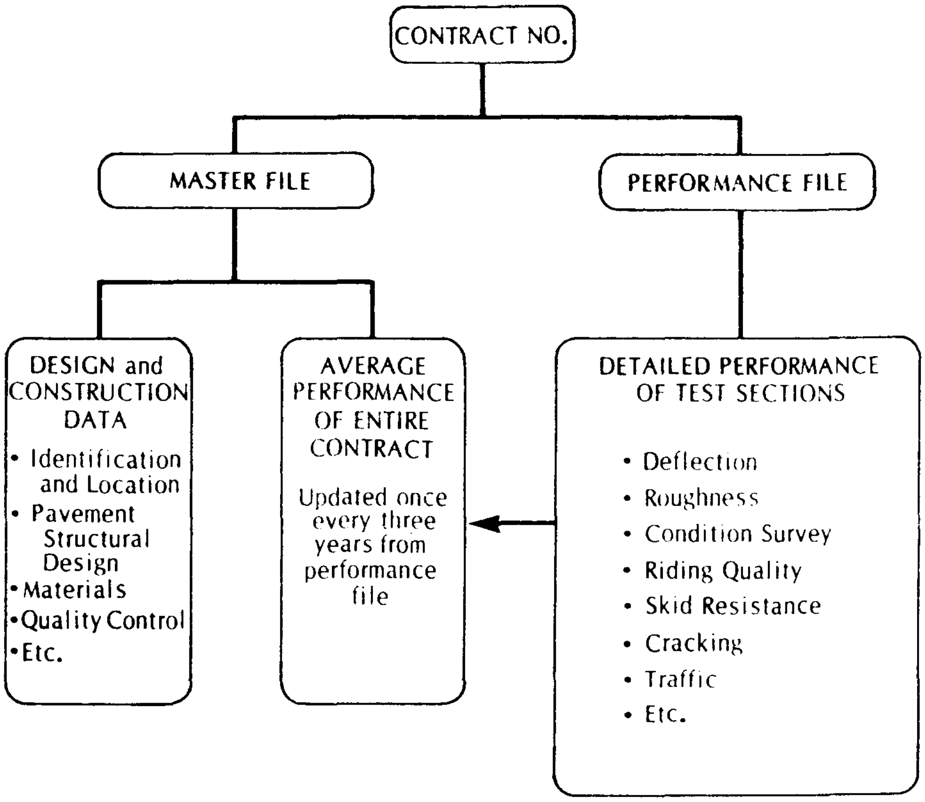
LESSON OUTLINE
PAVEMENT INVENTORY DATA COLLECTION, EQUIPMENT AND PROCESSING

VISUAL AID	TITLE
Visual 25.1	Data Collection in the Ontario Pavement Data Bank System
Visual 25.2	File Structure for the Ontario Pavement Data Bank System
Visual 25.3	Example of Pavement Evaluation Measurements by Section and Subsection
Visual 25.4	Guidelines for Selecting Frequency of Pavement Evaluation Measurements for Mass Inventory Purposes
Visual 25.5	Benkelman Beam and California Continuous Deflectometer
Visual 25.6	Dynaflect and Schematic Illustration of the Dynaflect Force Application and Deflection
Visual 25.7	Example Correlations Between Benkelman Beam and Dynaflect
Visual 25.8	Evaluation Forms for Individual, Subjective Pavement Ratings
Visual 25.9	Schematic of Frame for B.C. Photo Inventory
Visual 25.10	A Typical Output of the Mays Ride Meter Type or Car Road Meter
Visual 25.11	Areas of Applicability and Uses for Various Types of Roughness Measurements
Visual 25.12	(a) Variation of Skid Resistance with Time (Traffic) as a Measure of Pavement Performance (b) Example of Short Term Change in Skid Resistance Due to Rain
Visual 25.13	Major Distress Factors for Condition Surveys
Visual 25.14	General Functional Nature of A Pavement Data System in Operation
Visual 25.15	Major Steps in Developing and Implementing a Pavement Data Bank

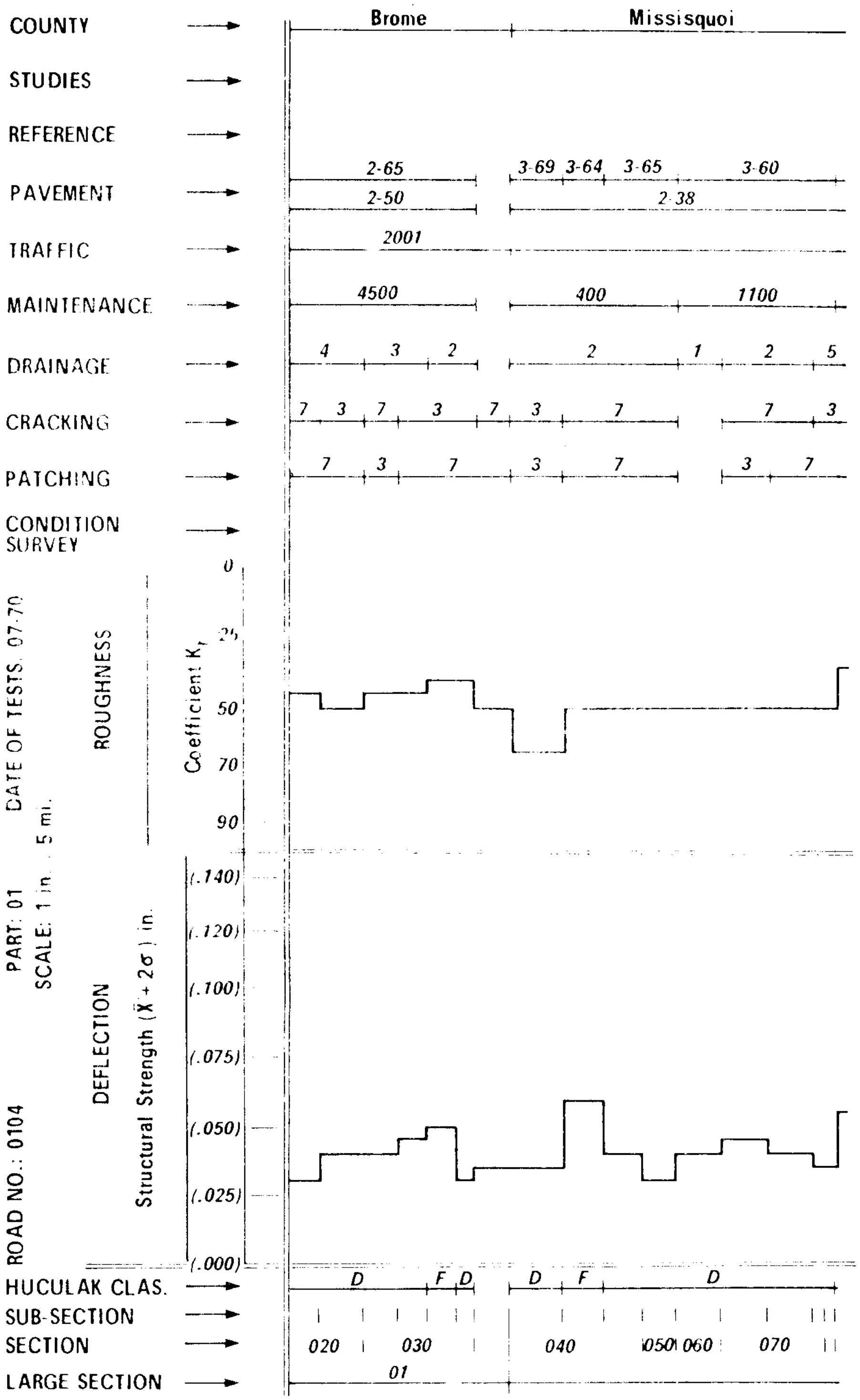
Visual Aid 25.1. Data collection in the Ontario pavement data bank system.



Visual Aid 25.2. File structure for the Ontario pavement data bank system.



Visual Aid 25.3. Example of pavement evaluation measurements by section and subsection.

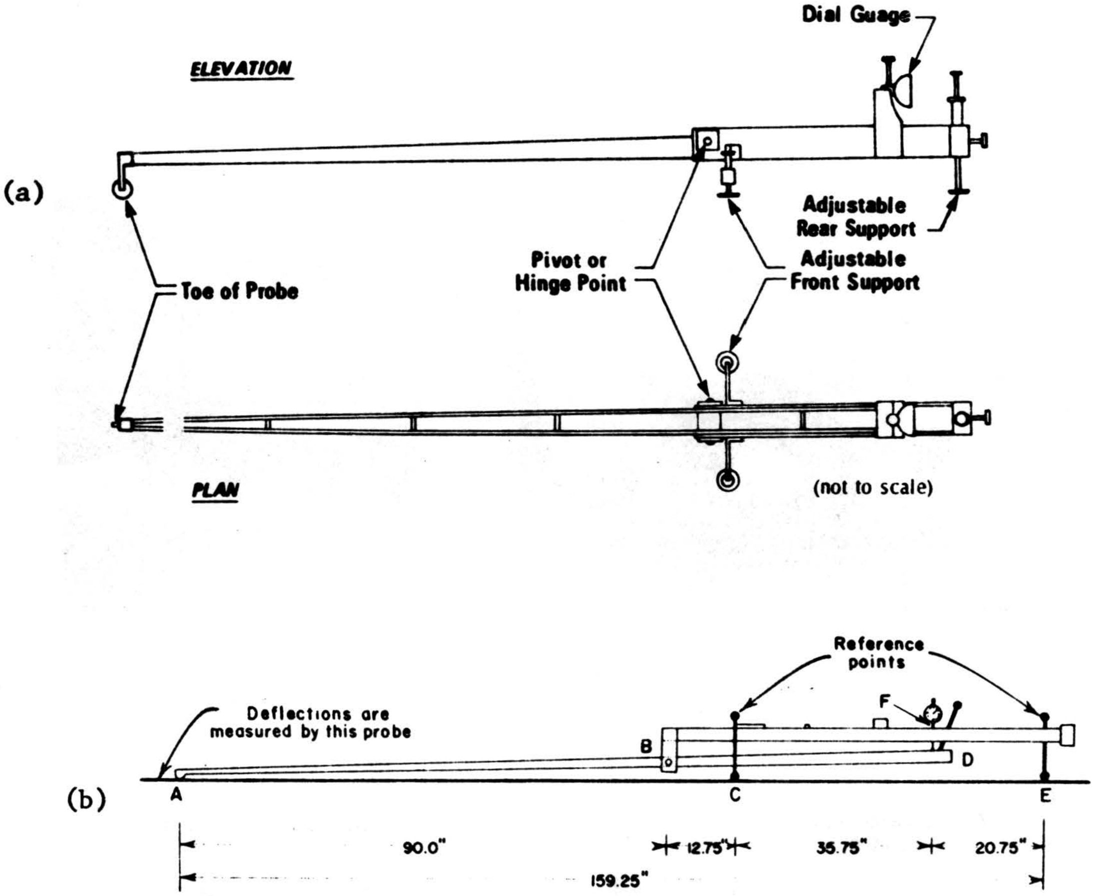


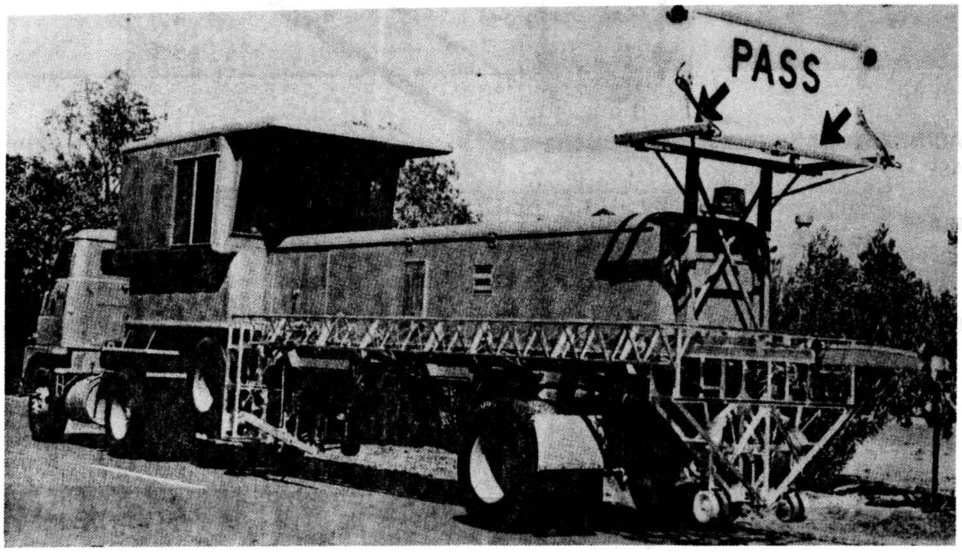
Transparency 25.4
Guidelines for Selecting Frequency of Pavement Evaluation Measurements for Mass Inventory Purposes

		Type of Evaluation Measurement on Subsection			
Facility		Deflections (Spring)	Roughness	Condition Survey	Skid Resistance
Freeway or Primary Rural Highways	More than 10,000 AADT	After constr.; then every 6 years; every 3 years when RCI reaches 6.0	Immed. after constr.; then every 5 years until RCI is 6.0; then every 3 years to end of service life	Every year after RCI drops to 6.5	At accident sites; every year where pilot testing indicates values close to agency's min. guideline
	Less than 10,000 AADT	After constr.; then every 6 years; every 3 years when RCI reaches 6.0	Immed. after constr.; then every 5 years until RCI is 6.0; then every 3 years until end of service life	Every 2 years after RCI drops to 6.0	"
Secondary Rural Highways	More than 5,000 AADT	After constr.; then every 5 years; every 2 years when RCI reaches 5.5	Immed. after constr.; then every 5 years until RCI is 5.5; then every 2 years until end of service life	Every 2 years after RCI drops to 5.5	"
	Less than 5,000 AADT	After constr.; then every 5 years; every 3 years when RCI reaches 5.0	Immed. after constr.; then every 5 years until RCI is 5.0; then every 2 years until end of service life	Every 2 years after RCI drops to 5.0	"
County or Local Rural Highways	More than 1,000 AADT	After constr.; then every 4 years; every 2 years when RCI reaches 4.5	After constr.; then every 5 years until RCI is 4.5; then just before rehabilitation	Every 2 years after RCI drops to 4.5	"
	Less than 1,000 AADT	After constr.: then every 4 years	After constr.: just before rehabilitation	Every 2 years After RCI drops to 4.0	"

Visual Aid 25.4. Guidelines for selecting frequency of pavement evaluation measurements for mass inventory purposes.

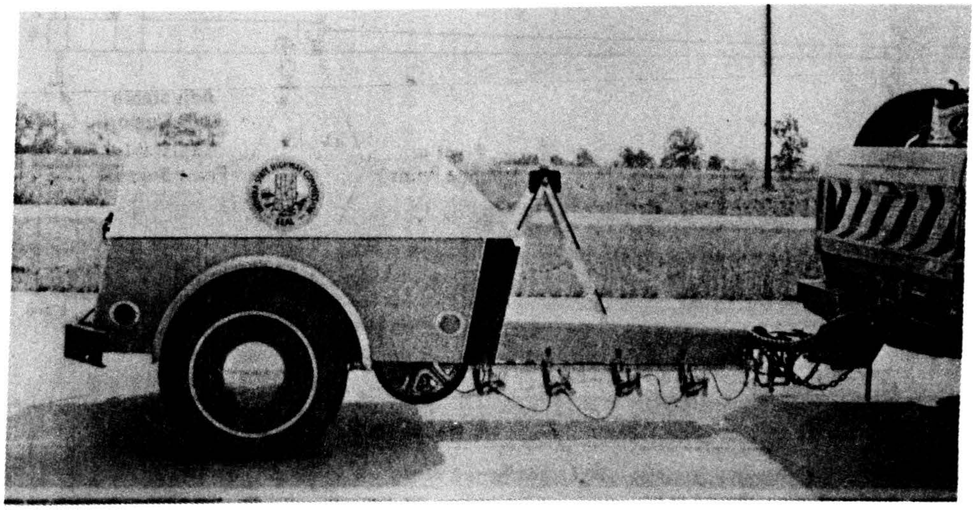
Visual Aid 25.5. Benkelman beam and California continuous deflectometer.



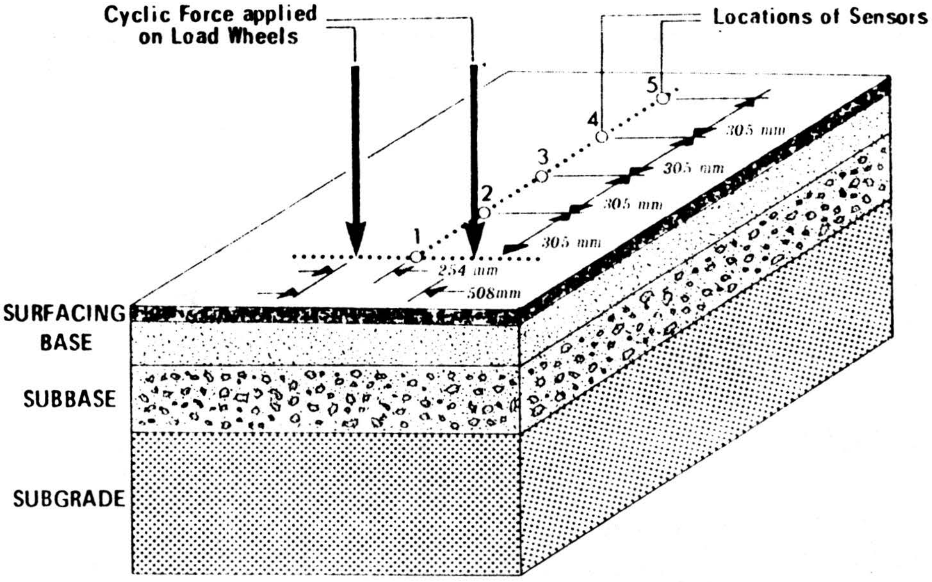


(c) California continuous deflectometer

Visual Aid 25.6. Dynaflect and schematic illustration of the Dynaflect force application and deflection.

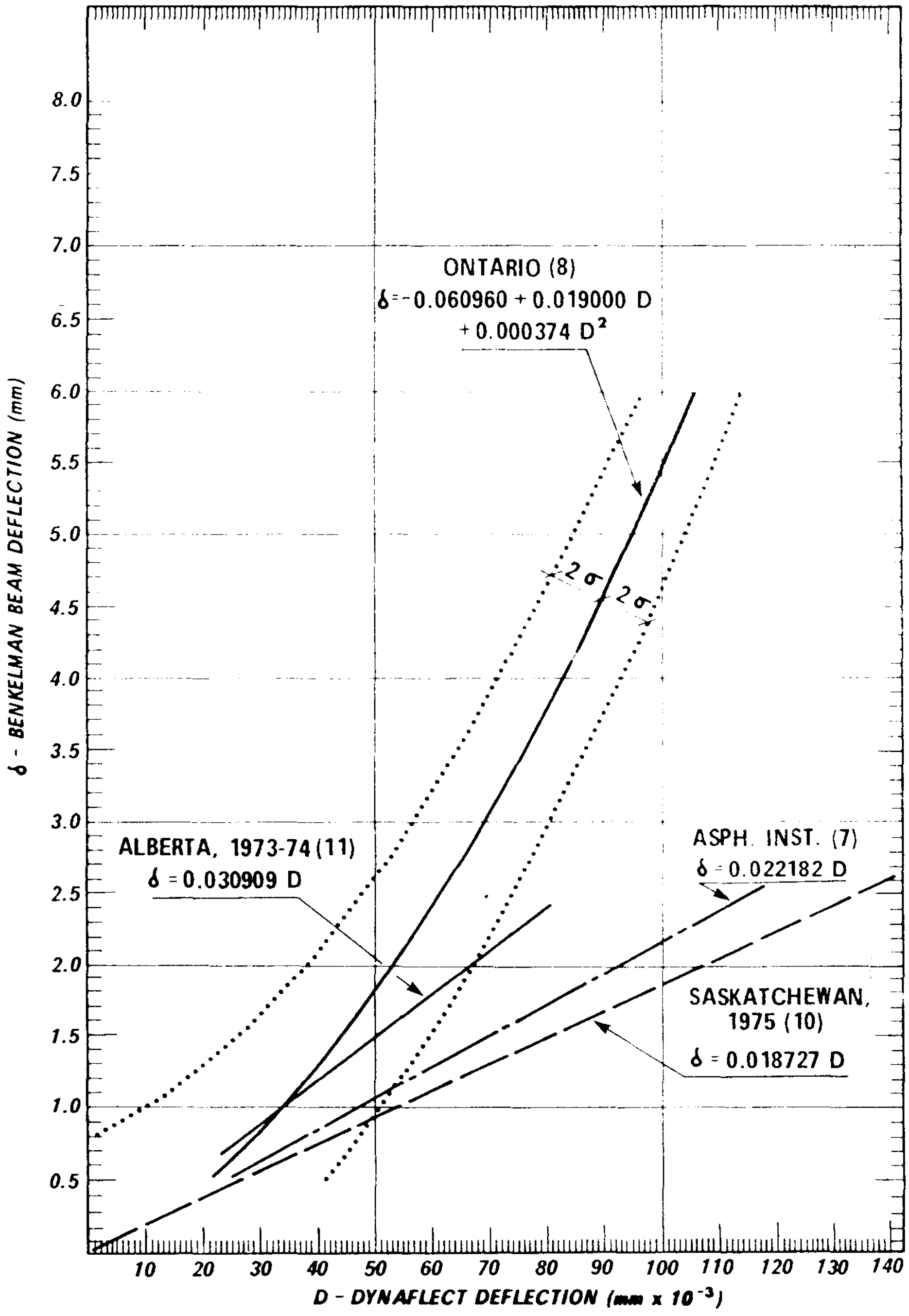


(a) Dynaflect

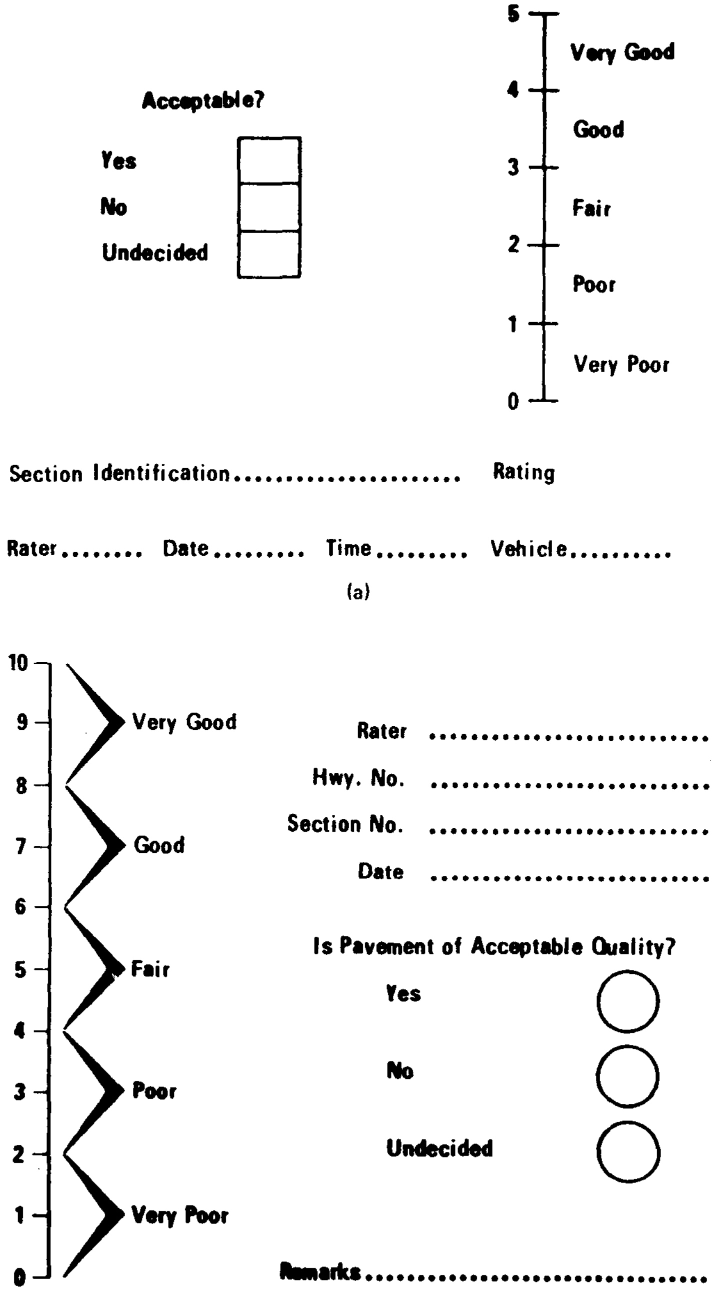


(b) Schematic Illustration of the Dynaflect Force Application and Deflection

Visual Aid 25.7. Example correlations between Benkelman beam and dynaflect.

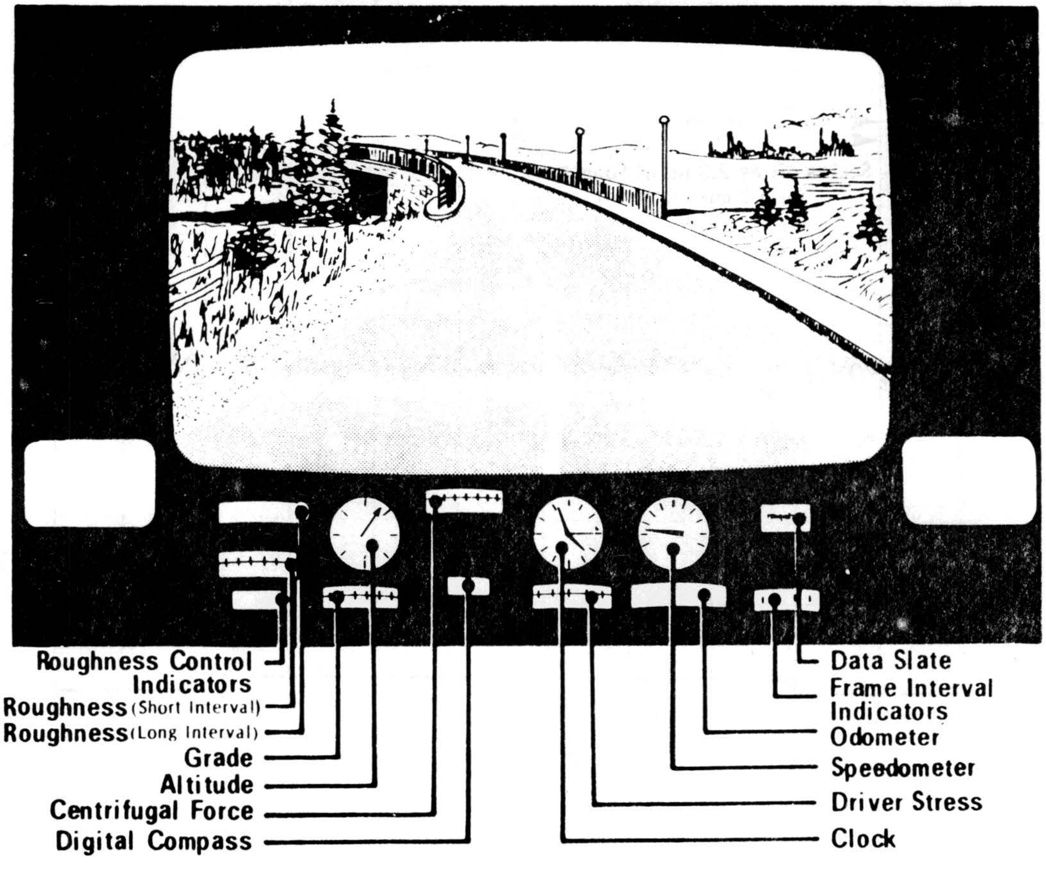


Visual Aid 25.8. Evaluation forms for individual, subjective pavement ratings.

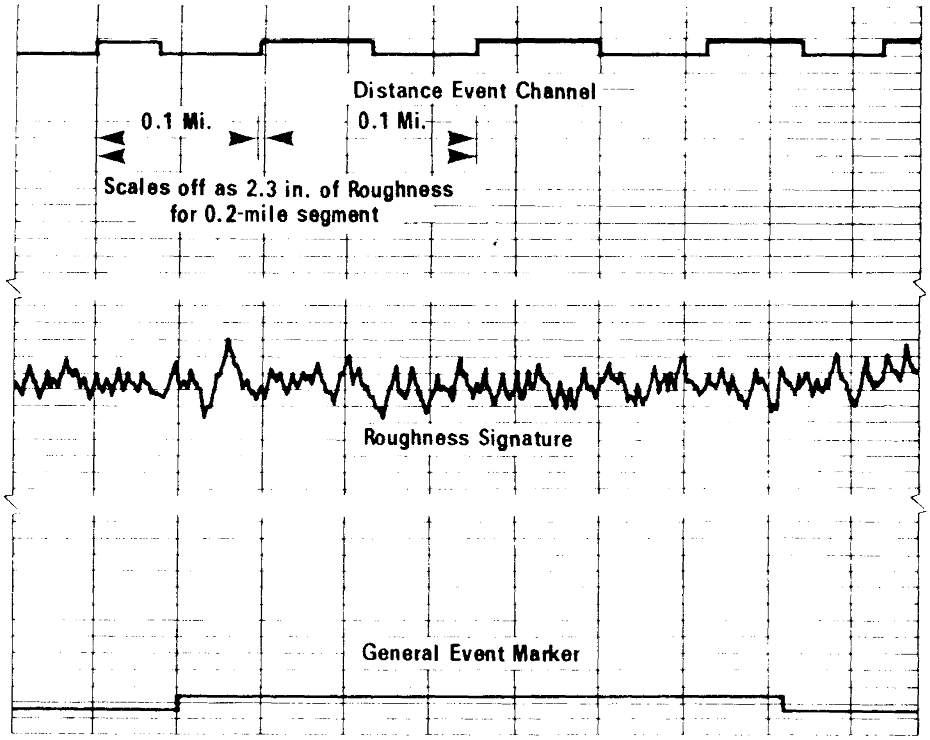


Evaluation forms for individual, subjective pavement ratings. (a) Individual Present Serviceability Rating (PSR) form used at AASHO Road Test. (b) Present Performance Rating (now Riding Comfort Index) form developed by the Canadian Good Roads Association (now the Roads and Transportation Association of Canada).

Visual Aid 25.9. Schematic of Frame for B.C. Photo Inventory.



Visual Aid 25.10. A Typical output of the Mays Ride Meter type or Car Road Meter.



Visual Aid 25.11. Areas of applicability and uses for various types of roughness measurements.

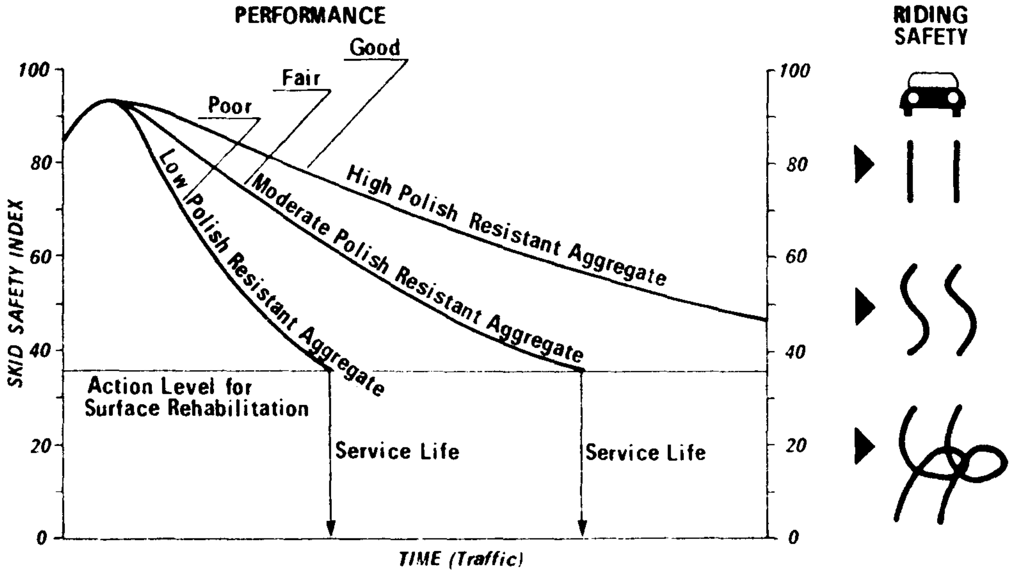
	Classes of Measurement by Purpose		
Facility Type	Initial Roughness	Periodic Ride	Final Roughness
Freeway or Primary Highway	BPR, SDP, CRM RSE[1] (TRRL, CHLOE)[2]	CRM, SDP (TRRL, CHLOE)	CRM, SDP (CHLOE, TRRL)
Secondary (Rural) Highway	BPR, CRM, RSE (SDP, TRRL, CHLOE)	CRM (SDP, TRRL, CHLOE)	CRM (SDP, CHLOE, TRRL)
County or Local Rural Highway	CRM, BPR, RSE (SDP)	CRM	CRM
Runways	SDP, TRRL, CRM	CRM, SDP, TRRL (LEVEL)	SDP, TRRL, LEVEL
Uses of Roughness Measurements, for All Facility Types			
Construction Monitoring	Yes[3]		
Maintenance Programming		Yes	Yes
Inventory and Network Programming		Yes	Yes
Research	Yes	Yes	Yes

[1] See section 4.3.3 and 4.3.4 for explanations of abbreviations for roughness devices

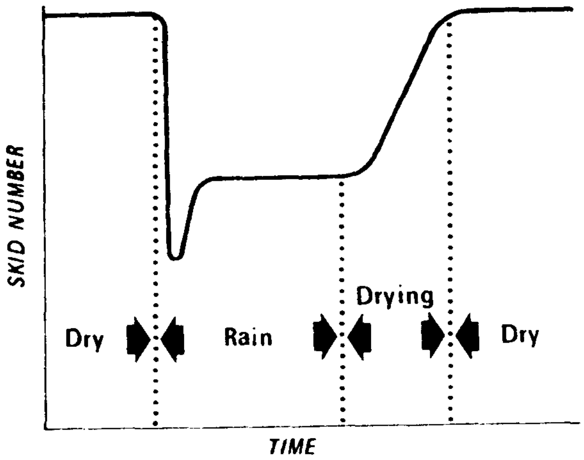
[2] Parentheses denote applicability primarily for special purposes or control sections

[3] These indicate the primary applicability of the class of measurement (i.e., initial roughness measurements are primarily applicable to construction monitoring, for all facility types).

Visual Aid 25.12



(a) Variation of Skid Resistance with Time (Traffic) as a Measure of Pavement Performance

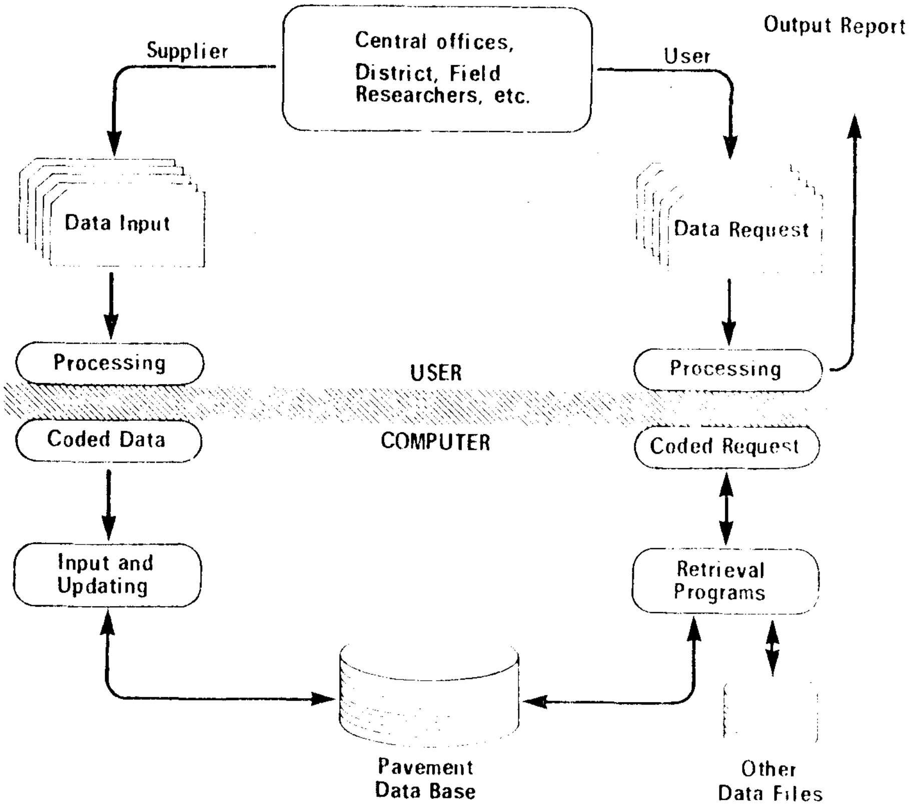


(b) Example of Short Term Change in Skid Resistance Due to Rain

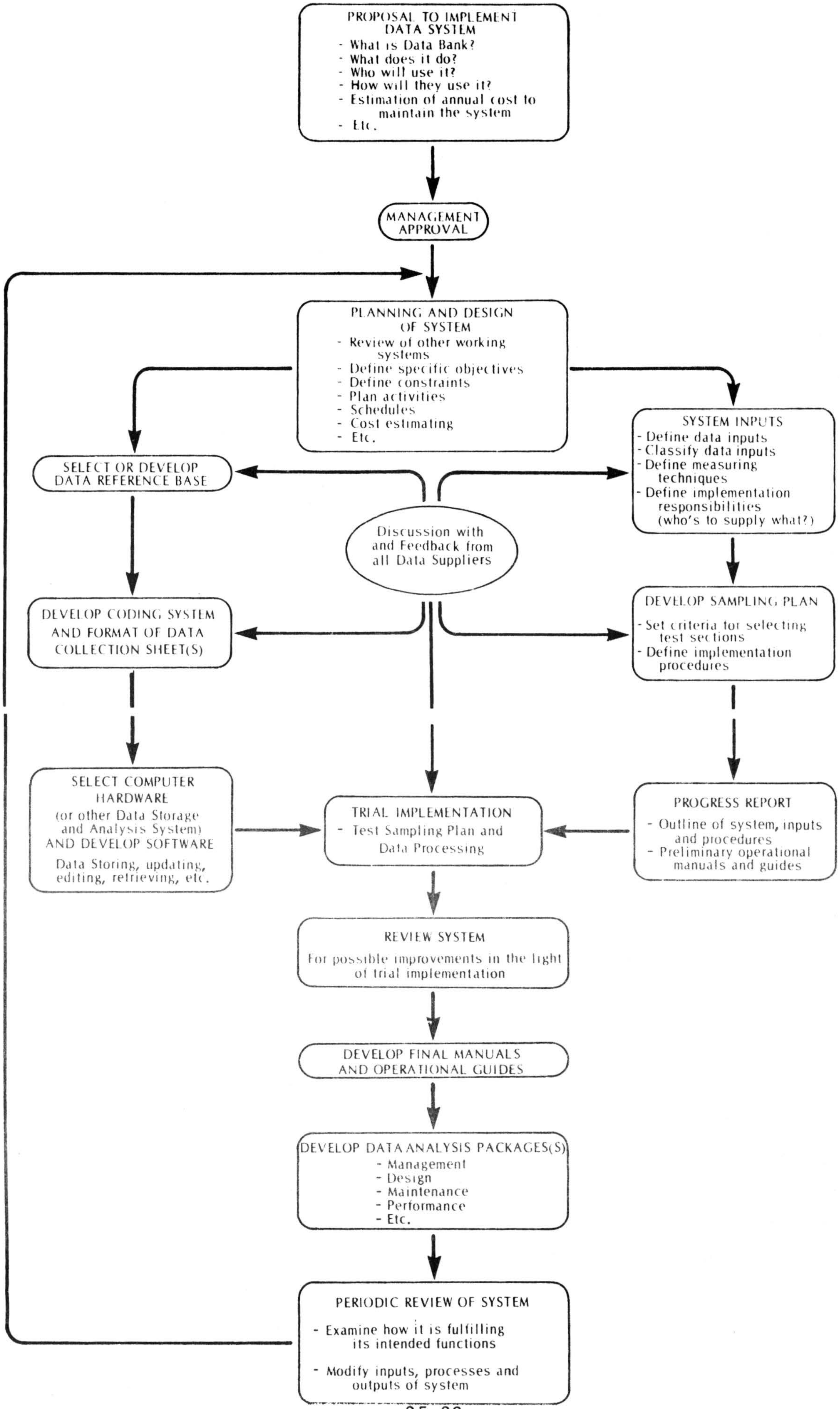
Visual Aid 25.13. Major distress factors for condition surveys.

PAVEMENT DISTRESS MANIFESTATION	EVALUATION		
	Severity	Density	Other Character.
Surface Defects			
• Coarse Aggregate Loss			
• Ravelling			
• Flushing			
Surface Deformation			
• Rippling			
• Shoving			
• Wheel Track Rutting			
• Distortion			
Cracking			
• Longitudinal Wheel Track			
Midlane			
Centre Line			
Pavement Edge			
• Meandering			
• Transverse			
• Alligator			
• Random			
• Slippage			
• Other			
Maintenance Patching			
• Spray			
• Skin			
• Hot-Mix			

Visual Aid 25.14. General functional nature of a pavement data system in operation.



Visual Aid 25.15. Major steps in developing and implementing a pavement data bank.



INSTRUCTIONAL TEXT

MONITORING AND EVALUATING PAVEMENTS: DISTRESS, STRUCTURAL CAPACITY, SAFETY, GEOMETRICS

Introduction

The evaluation phase of pavement management involves the determination and continuous monitoring of the condition of the roadways within the agency's purview. Evaluation provides the primary source of information for use at all levels and in all activity areas of a pavement management system. Monitoring involves the routine collection of field data and recording such data in a useful form. Evaluation, as described in this Session, encompasses monitoring, but involves a judgment or determination of the meaning of the information collected.

It is the function of pavement evaluation in a pavement management system to measure pavement condition periodically in order to:

1. Provide data for checking and updating predictions
2. Reschedule rehabilitation, maintenance, etc. as indicated by these updated predictions
3. Provide data for improving the prediction model
4. Provide data for improving construction and maintenance techniques
5. Provide information for updating network improvement programs

Pavement condition information needed for rehabilitation involves five main components: (1) serviceability or riding comfort, (2) load-carrying capacity, (3) safety, (4) distress or surface condition, and (5) geometrics. The specific information to be recorded is a function of its use, which may involve project or network level applications.

Pavement condition varies with time. In this sense it must be considered now and in the future. Planning, design, and other areas of a pavement management system must be concerned with both the present and future (and to some extent, the past) values of the "outputs" of a pavement, including the condition components noted above. Figure 5.1 is a schematic representation of the variations of the major types of pavement outputs with time. These outputs are predicted over the analysis period and are of course actually measured when the pavement is in service.

Measures of Pavement Outputs

Various measures are used to represent the pavement outputs identified in Figure 5.1. Some typical measurement methods are listed in Figure 5.2.

Physical structure and material strength can be monitored by physical testing and sampling.

Behavior can be defined as the immediate response of the pavement to load. Thus, load-deflection testing of all types, including plate load tests, static deflection measurements such as those using the Benkelman Beam, and dynamic deflection measurements, fall into this category. Although information about the physical structure of the pavement is often inferred from behavioral evaluations, it should be remembered for purposes of clarity that these load-testing techniques evaluate only the behavioral response of the pavement and not the physical properties directly.

Further information on structural evaluation may be found in Unit 2.6 of the Reference Notebook, and in Reference 1.

Safety may be measured in an empirical fashion, e.g., through determination of those locations with high accident rates. However, this may not be due to pavement-related factors, but could, for example, indicate an alignment problem. Such factors may be included in the

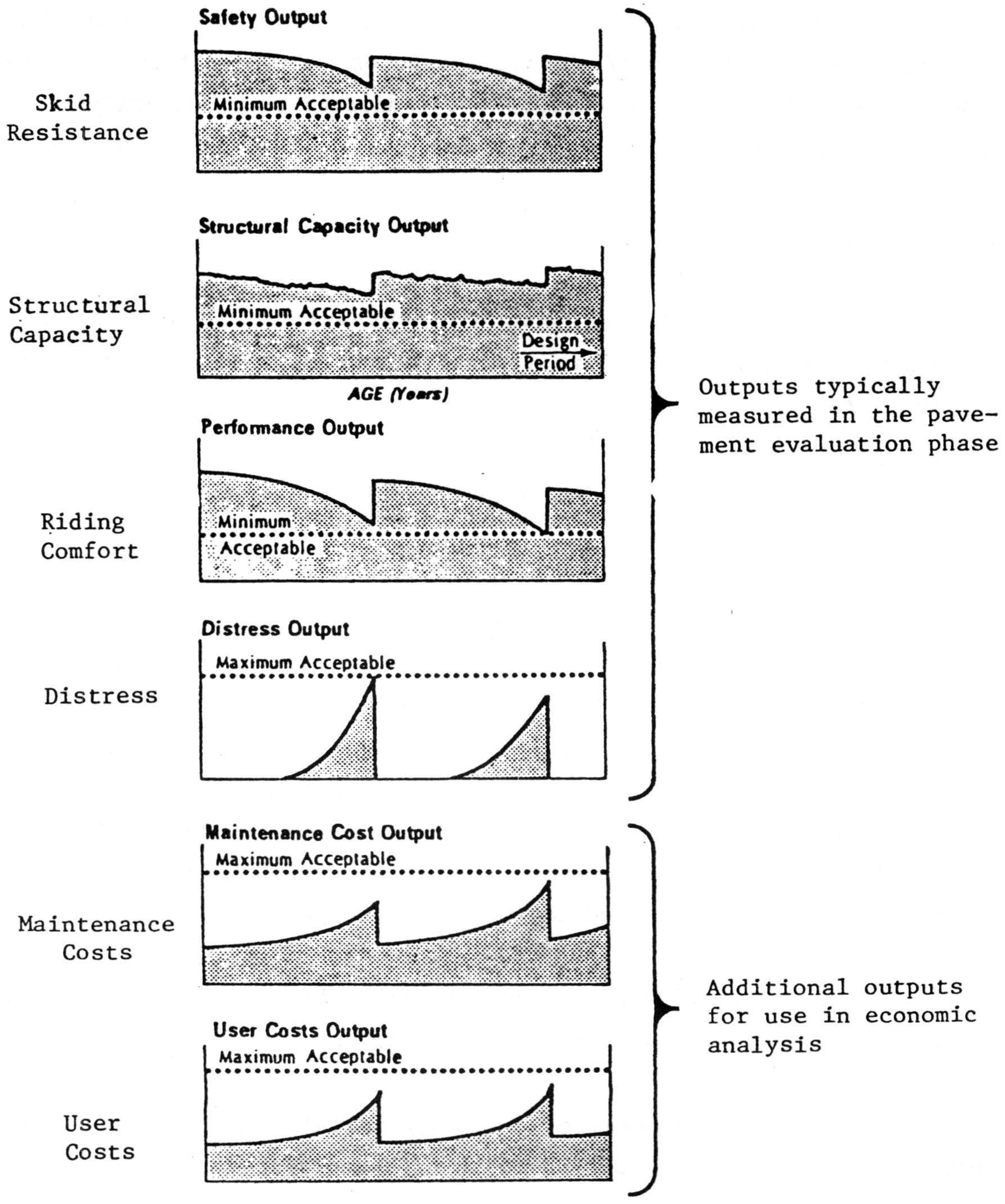


Figure 1 Major Types of Pavement Outputs or Performance Indicator (After Ref. 1)

Figure 2 Example Measures of the Four Major Pavement Outputs

Pavement Output or Performance Indicator	Example Monitoring Method
Safety	Skid Number, Accident Rate
Structural Capacity	Cores, Deflections
Serviceability	PSR, PSI
Distress	Condition Survey

pavement management system, at the discretion of the agency involved. The typical current practice is to use skid resistance as the primary measure of safety related to pavements.

Distress can be defined as observable deterioration or damage in the pavement. Thus, the accumulated damage that the pavement has suffered is monitored and evaluated. Because maintenance may have been performed on some of the distress, the evidences of this maintenance in the form of patches and sealed areas should also be monitored. Such monitoring is done routinely by many agencies in the form of condition surveys, and the data accumulated can provide important pavement evaluation information.

The identification of various distress types for measurement in a routine pavement condition survey is generally made on the basis of the experience of the individual agency regarding which distress types are most important. Thus, the specific variables recorded, and the units in which they are measured will vary from agency to agency. A sample condition survey form is shown in Figure 5.3, but this is just a sample and is not to be considered suitable for all agencies.

It is, of course, desirable to have consistent pavement condition measurements carried out by all states, using the same techniques, units and categories of measurement. This would facilitate comparison of pavement performance, prediction models, etc., and would allow better determination of the effects of climatic variation, etc. A recent FHWA report by Smith, Herrin, and Darter provides detailed information on an extensive list of pavement distress variables, along with suggested severity levels and measurement techniques (Ref. 3). The widespread use of this report would represent a giant step toward compatible pavement condition measurement.

Another category of pavement evaluation of major interest is pavement serviceability and performance. This is discussed in Sessions 4 and 4A.

PAVEMENT DISTRESS MANIFESTATION	EVALUATION		
	Severity	Density	Other Character.
Surface Defects			
• Coarse Aggregate Loss			
• Ravelling			
• Flushing			
Surface Deformation			
• Rippling			
• Shoving			
• Wheel Track Rutting			
• Distortion			
Cracking			
• Longitudinal Wheel Track			
Midlane			
Centre Line			
Pavement Edge			
• Meandering			
• Transverse			
• Alligator			
• Random			
• Slippage			
• Other			
Maintenance Patching			
• Spray			
• Skin			
• Hot-Mix			

Figure 3 Example Form for Pavement Condition Evaluation

There is a growing feeling that the word "performance" should be reserved to mean the over-all service history of the pavement, incorporating not only serviceability, but structural adequacy, distress, etc. Some feel that safety, life cycle costs, etc. should also be included in the definition of performance. These course notes tend toward a broad definition. In a sense, the choice is a matter of semantics, but it is clear that a word is needed to denote the combined, over-all service adequacy of a pavement over a period of time.

Other factors, such as costs and aesthetics, are not generally measured as a part of pavement evaluation. Each activity area is generally charged with recording costs incurred in carrying out its own specific functions. Routine maintenance costs, for example, are reported by the maintenance division. Aesthetic factors are generally not treated in a quantitative way, but may be included in a subjective manner by the decision-maker.

There is, of course, considerable overlap among the evaluation measures discussed here. However, they should not be confused or used indiscriminantly. For example, the fact that some people evaluate serviceability level using a serviceability equation that involves cracking and patching does not mean that the equation provides an adequate evaluation of distress. Likewise, the fact that dynamic deflection measurements may be used to estimate pavement structural thicknesses and/or properties should not confuse the user. These behavioral measurements can be used to estimate structural inputs only in conjunction with some type of theory or model. The structural input values themselves can be evaluated directly only with a destructive test or sampling procedures. Figure 5.4 is a schematic diagram of the Dynaflect load wheel and sensor configurations. A block diagram illustrating the relationship of the different inputs to a rehabilitation design subsystem from monitoring and evaluation is shown in Figure 5.5.

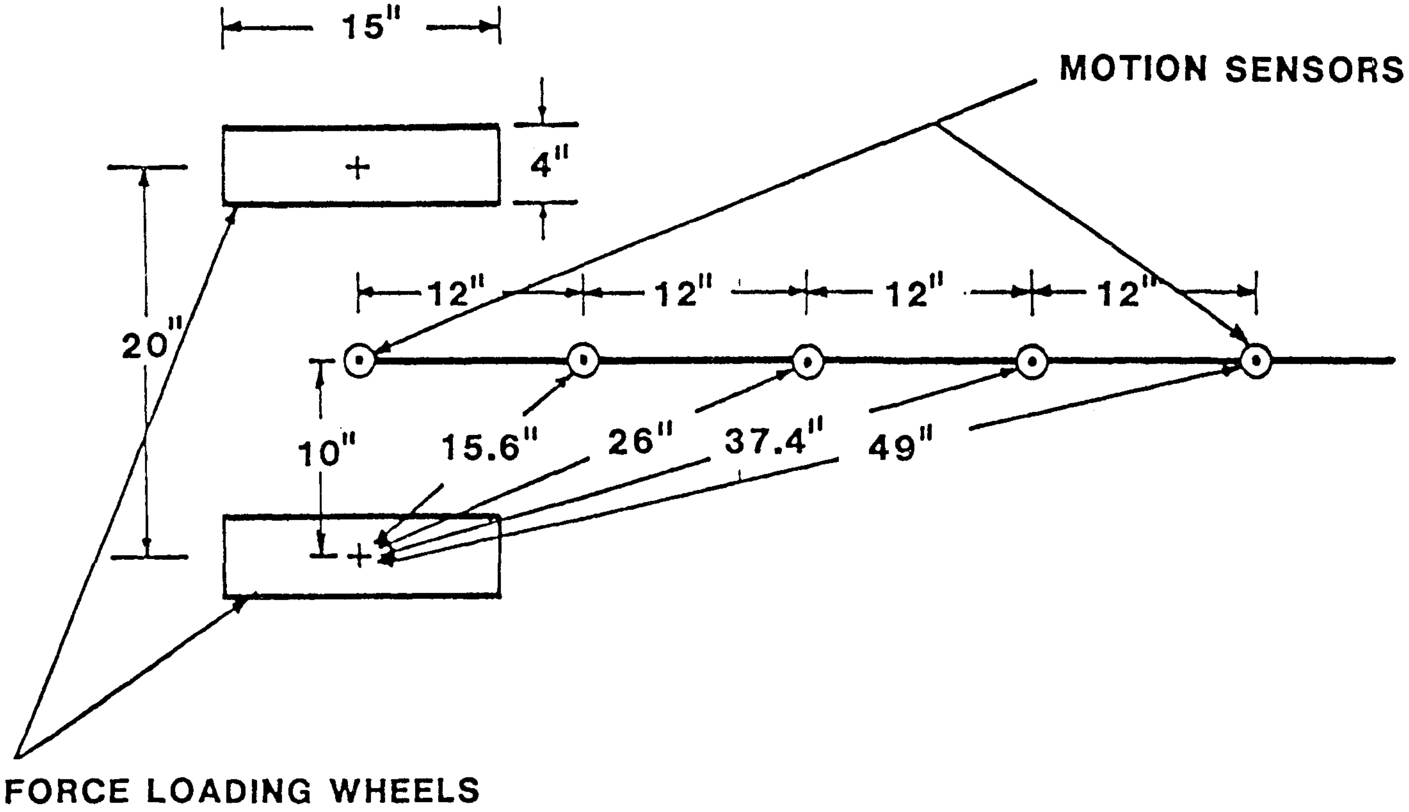


Figure 4 Dynaflect Load and Sensor Configuration

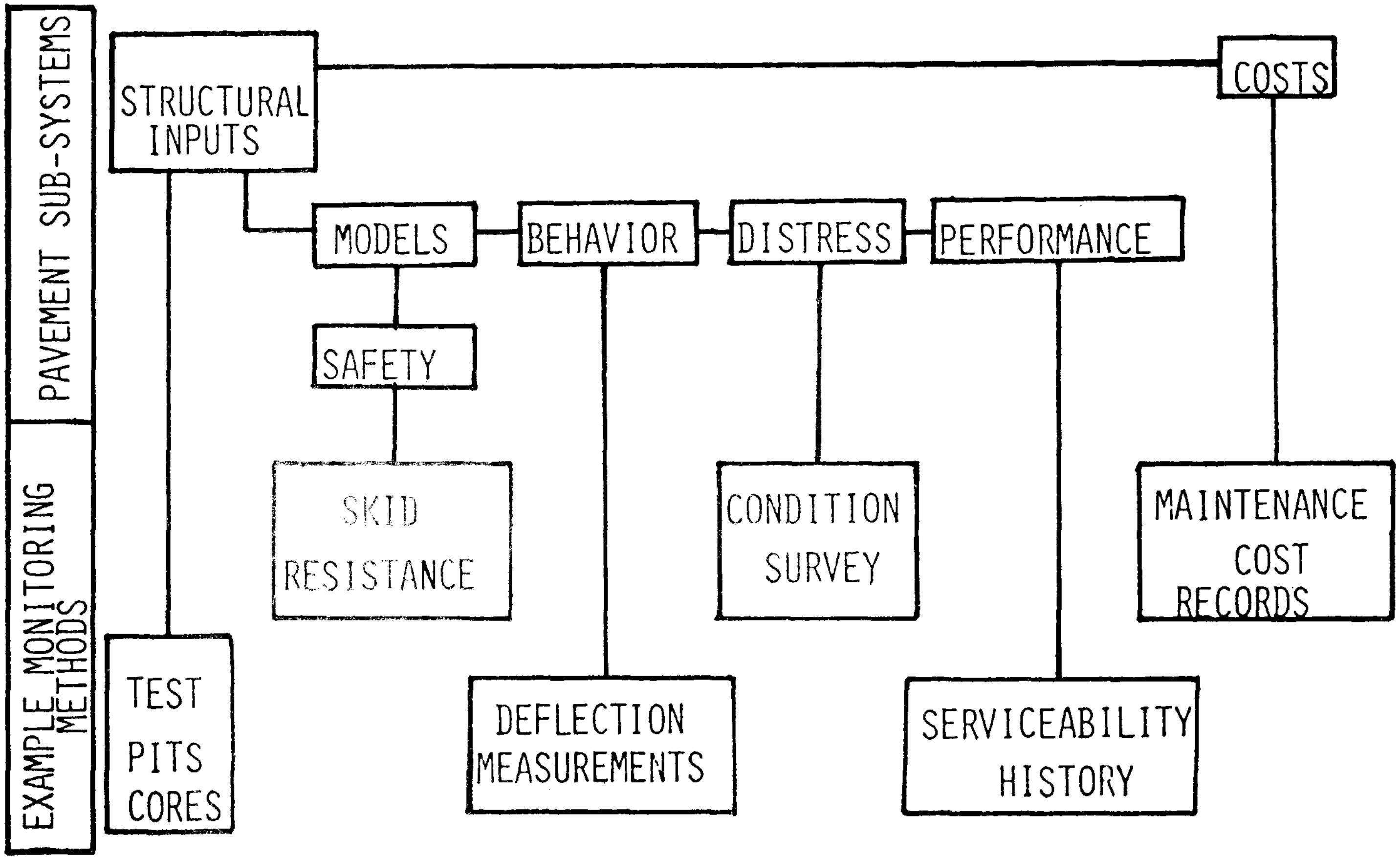
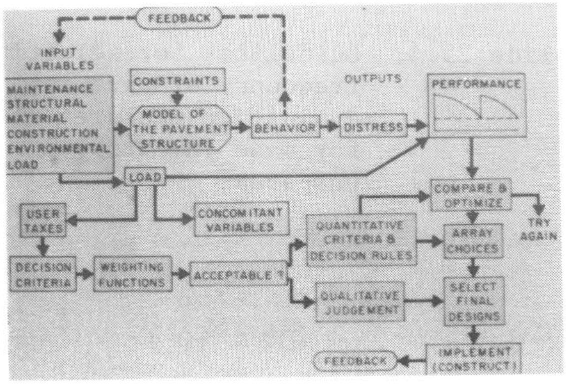


Figure 5 Input to the Rehabilitation Design Subsystem from Monitoring and Evaluation



FEEDBACK

INPUT VARIABLES

MAINTENANCE
STRUCTURAL
MATERIAL
CONSTRUCTION
ENVIRONMENTAL
LOAD

CONSTRAINTS

MODEL OF THE PAVEMENT STRUCTURE

OUTPUTS

BEHAVIOR

DISTRESS

PERFORMANCE

LOAD

USER TAXES

CONCOMITANT VARIABLES

COMPARE & OPTIMIZE

TRY AGAIN

QUANTITATIVE CRITERIA & DECISION RULES

ARRAY CHOICES

DECISION CRITERIA

WEIGHTING FUNCTIONS

ACCEPTABLE ?

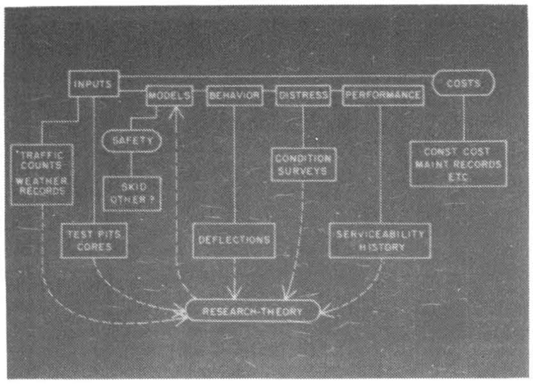
QUALITATIVE JUDGEMENT

SELECT FINAL DESIGNS

FEEDBACK

IMPLEMENT (CONSTRUCT)

Slide 25.1. Data collection in the Ontario pavement data bank system.



INPUTS

MODELS

BEHAVIOR

DISTRESS

PERFORMANCE

COSTS

SAFETY

TRAFFIC COUNTS
WEATHER RECORDS

SKID
OTHER ?

CONDITION SURVEYS

CONST COST
MAINT RECORDS
ETC

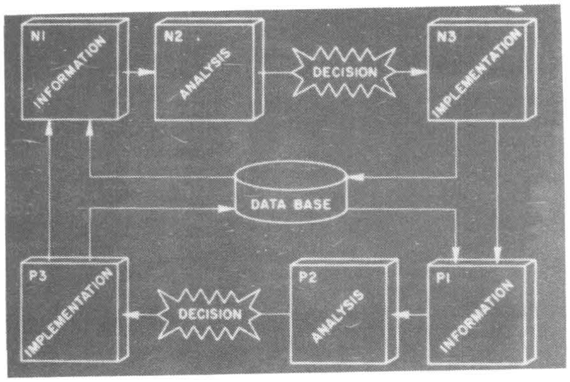
TEST PITS
CORES

DEFLECTIONS

SERVICEABILITY HISTORY

RESEARCH-THEORY

Slide 25.2. File structure for the Ontario pavement data bank system.



N1 INFORMATION

N2 ANALYSIS

DECISION

N3 IMPLEMENTATION

DATA BASE

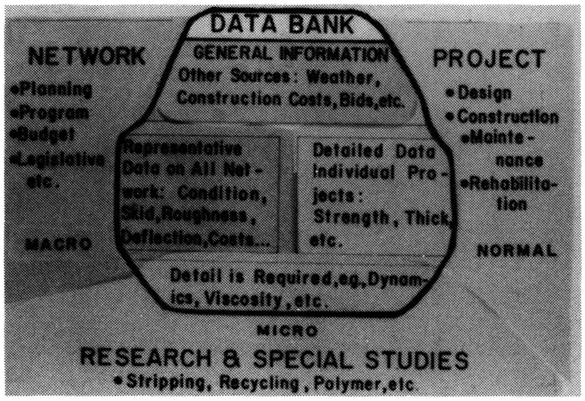
P3 IMPLEMENTATION

DECISION

P2 ANALYSIS

P1 INFORMATION

Slide 25.3. Example of pavement evaluation measurements by sections and subsection.



DATA BANK

GENERAL INFORMATION
Other Sources: Weather,
Construction Costs, Bids, etc.

NETWORK
- Planning
- Program
- Budget
- Legislative etc.

MACRO

Representative Data on All Network: Condition, Skid, Roughness, Deflection, Costs...

Detailed Data Individual Projects: Strength, Thick, etc.

PROJECT
- Design
- Construction
- Maintenance
- Rehabilitation

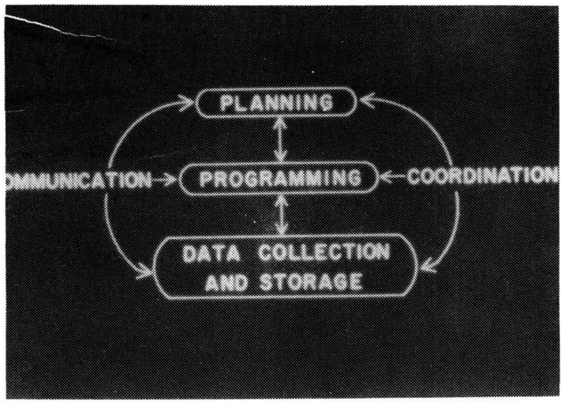
NORMAL

Detail is Required, eg, Dynamics, Viscosity, etc.

MICRO

RESEARCH & SPECIAL STUDIES
- Stripping, Recycling, Polymer, etc

Slide 25.4. Guidelines for selecting frequency of pavement evaluation measurements for mass inventory purposes.



PLANNING

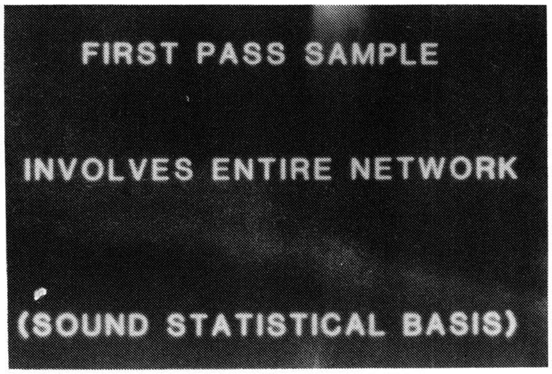
PROGRAMMING

DATA COLLECTION AND STORAGE

OMMUNICATION

COORDINATION

Slide 25.5. Data collection system.

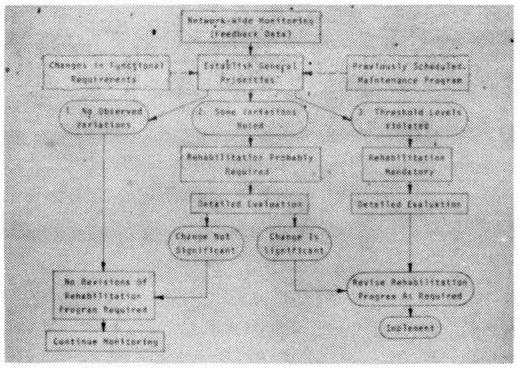


FIRST PASS SAMPLE

INVOLVES ENTIRE NETWORK

(SOUND STATISTICAL BASIS)

Slide 25.6. Sampling in the network system.



Network-wide Monitoring (Feedback Data)

Changes in Functional Requirements

Establish General Priorities

Previously Scheduled Maintenance Program

1. No Observed Variations

2. Some Variations Noted

3. Threshold Levels Violated

Rehabilitation Probably Required

Rehabilitation Mandatory

Detailed Evaluation

Detailed Evaluation

Change Not Significant

Change Is Significant

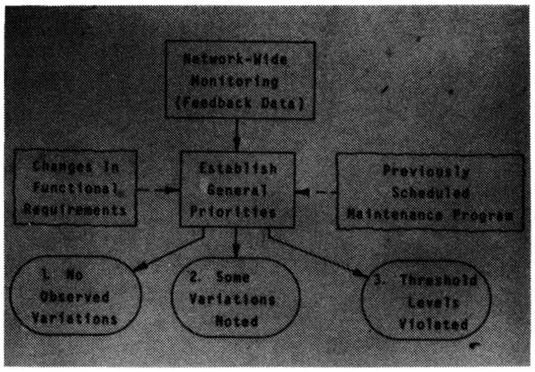
No Revisions Of Rehabilitation Program Required

Revise Rehabilitation Program As Required

Continue Monitoring

Implement

Slide 25.7. Data collection in the network system.



Network-Wide Monitoring (Feedback Data)

Changes In Functional Requirements

Establish General Priorities

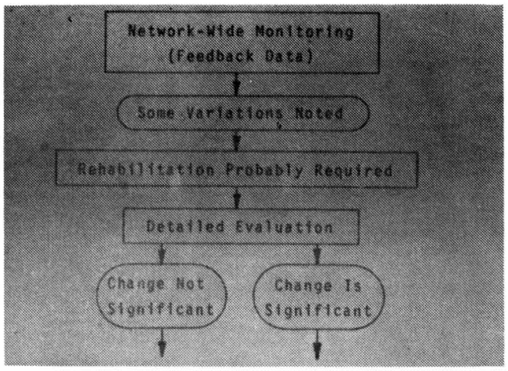
Previously Scheduled Maintenance Program

1. No Observed Variations

2. Some Variations Noted

3. Threshold Levels Violated

Slide 25.8. Master file as-built data.



Network-Wide Monitoring (Feedback Data)

Some Variations Noted

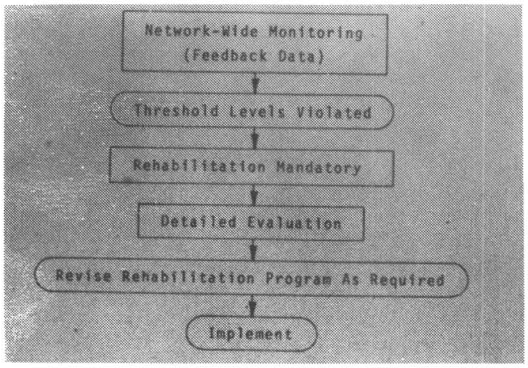
Rehabilitation Probably Required

Detailed Evaluation

Change Not Significant

Change Is Significant

Slide 25.9. Evaluation file.



Network-Wide Monitoring (Feedback Data)

Threshold Levels Violated

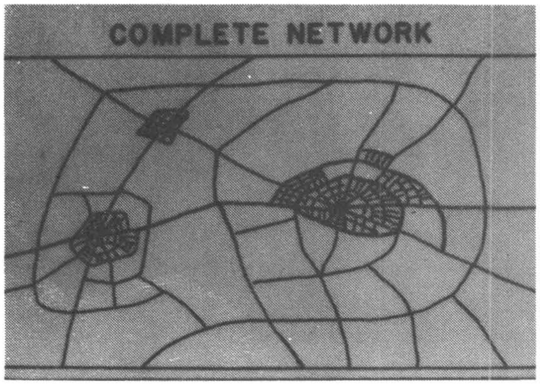
Rehabilitation Mandatory

Detailed Evaluation

Revise Rehabilitation Program As Required

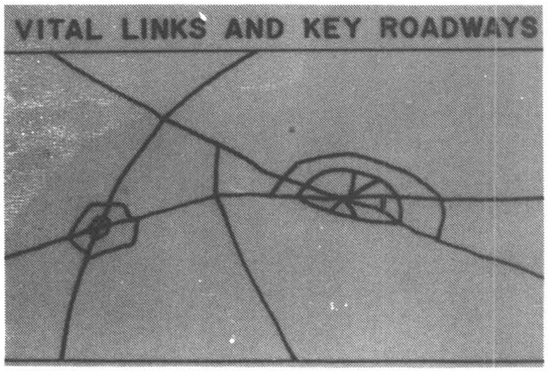
Implement

Slide 25.10. Maintenance and rehabilitation file.



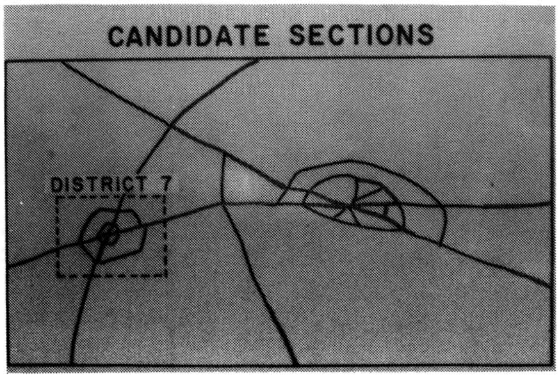
COMPLETE NETWORK

Slide 25.11. Complete network system.



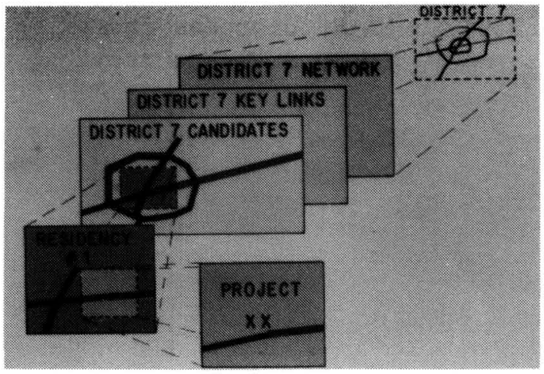
VITAL LINKS AND KEY ROADWAYS

Slide 25.12. Vital links and key roadways.

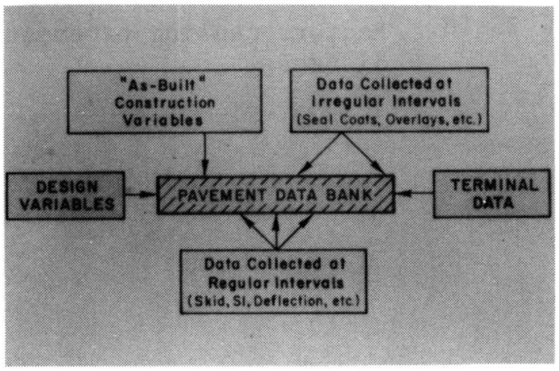


CANDIDATE SECTIONS

Slide 25.13. Selecting sections.



Slide 25.14. Selecting subsections boundaries and lengths.



"As-Built"
Construction
Variables

Data Collected at
Irregular Intervals
(Seal Coats, Overlays, etc.)

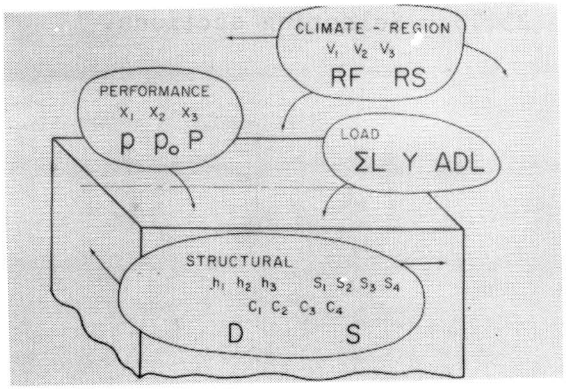
DESIGN
VARIABLES

PAVEMENT DATA BANK

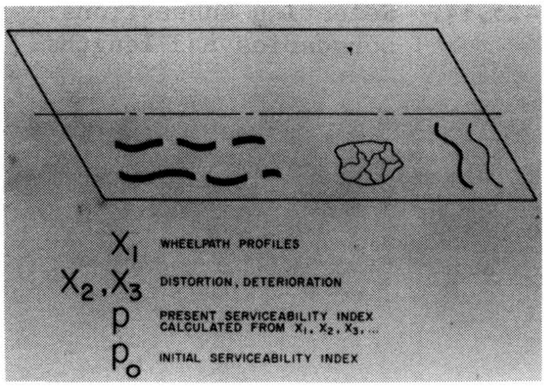
TERMINAL
DATA

Data Collected at
Regular Intervals
(Skid, SI, Deflection, etc)

Slide 25.15. Basis for selecting boundaries of the subsections.



Slide 25.16. Basis for selecting boundaries of subsections.



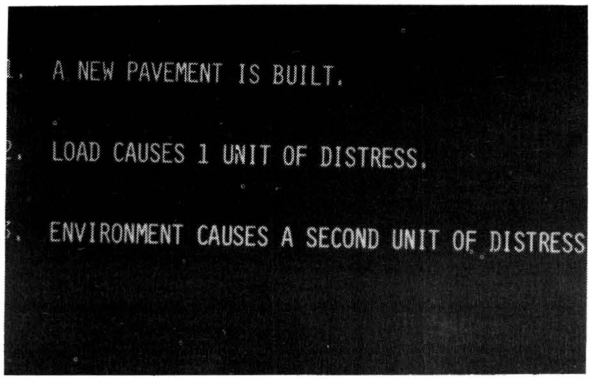
X_1 WHEELPATH PROFILES

X_2, X_3 DISTORTION, DETERIORATION

p PRESENT SERVICEABILITY INDEX CALCULATED FROM X_1, X_2, X_3, ...

p_o INITIAL SERVICEABILITY INDEX

Slide 25.17. Major distress factor in condition survey.

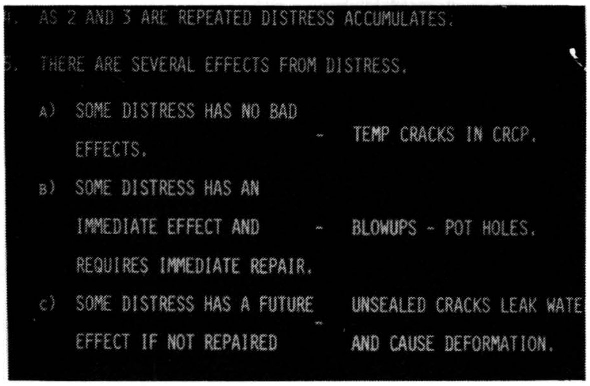


A NEW PAVEMENT IS BUILT.

LOAD CAUSES 1 UNIT OF DISTRESS.

ENVIRONMENT CAUSES A SECOND UNIT OF DISTRESS

Slide 25.18. Factors causing pavement distress.

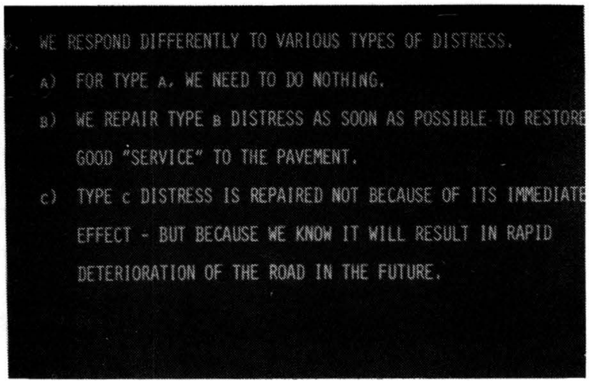


AS 2 AND 3 ARE REPEATED DISTRESS ACCUMULATES.

THERE ARE SEVERAL EFFECTS FROM DISTRESS.

A) SOME DISTRESS HAS NO BAD EFFECTS.	- TEMP CRACKS IN CRCP.
B) SOME DISTRESS HAS AN IMMEDIATE EFFECT AND REQUIRES IMMEDIATE REPAIR.	- BLOWUPS - POT HOLES.
C) SOME DISTRESS HAS A FUTURE EFFECT IF NOT REPAIRED	- UNSEALED CRACKS LEAK WATE AND CAUSE DEFORMATION.

Slide 25.19. Variations in the effect from distress types.



WE RESPOND DIFFERENTLY TO VARIOUS TYPES OF DISTRESS.

A) FOR TYPE A, WE NEED TO DO NOTHING.

B) WE REPAIR TYPE B DISTRESS AS SOON AS POSSIBLE TO RESTORE GOOD "SERVICE" TO THE PAVEMENT.

C) TYPE C DISTRESS IS REPAIRED NOT BECAUSE OF ITS IMMEDIATE EFFECT - BUT BECAUSE WE KNOW IT WILL RESULT IN RAPID DETERIORATION OF THE ROAD IN THE FUTURE.

Slide 25.20. Remedial actions as responses to distress types.

Revised DS/lg 1/9/84
Lesson 26

LESSON OUTLINE
FIELD MEASUREMENT OF SERVICEABILITY AND ROUGHNESS

Instructional Objectives

1. The participant will be able to illustrate the importance of adequate field measurements of pavement roughness, serviceability, and performance.

2. The participant will be able to discuss the problems which arise to interfere with such measurements.

Performance Objectives

1. The student shall understand how the serviceability concept is translated into practice through the measurement of roughness.

2. The student should be able to discuss the use of roughness in evaluating pavement performance and relate that to other factors such as "distress".

Abbreviated Summary	Time Allocations, min.
1. Development of the Serviceability Concept	10
2. Concept of Ratings	10
3. Developement of a Serviceability Index	30
	50 minutes

Reading Assignment

1. Haas & Hudson - Chapter 7

2. NCHRP 7

LESSON OUTLINE
FIELD MEASUREMENT OF SERVICEABILITY AND ROUGHNESS

1.0 COMPONENT OF PAVEMENT ROUGHNESS (Slides 26.1 & 26.2)

1.1 Roughness Defined

The distortion of the road surface which contributes to an undesirable, unsafe uneconomical, or uncomfortable ride.

1.2 Factors Influencing Perception of Roughness

1.2.1 Road Profile. (Slides 26.3 - 26.8) To define the pavement roughness function completely, some evaluation of the roughness of the entire surface area of the pavement should be made. However, for the most purposes this roughness can be divided into three components:

(a) transverse variations,
(b) longitudinal variation, and
(c) horizontal variations of pavement alignment.

1.2.2 Vehicle Response. (Visual Aid 26.1) (Slides 26.9 & 26.10) Many previous studies have shown that the longitudinal variations are the major contributing factor to undersirable vehicle forces. The next greatest offender is transverse roughness. The ride sensation is however, a function at the road profile, the vehicle parameters, and the vehicle speed.

2.0 ROUGHNESS MEASUREMENT

2.1 Fundamental Uses of Roughness Measurement

(a) To maintain construction quality control.
(b) To locate abnormal changes in the roadway.
(c) To establish a statewide basis for allocation of road maintenance resources.
(d) To identify road serviceability.

2.2 Methods of Measuring Roughness

There are a variety of methods or devices for measuring roughness that have found common use for highways and airports in North America. These range from the simple to the sophisticated and include:

2.2.1 US Bureau of Roads Type of Roughometer (BPR).(Slides 26.20-26.24) The BPR roughometer is one of the best-known devices This roughometer essentially simulates one wheel of a passenger car and is comprised of a press, spring, and damper combination. Displacement of the wheel with respect to the mass is recorded by an integrator coupled to an electronic counter.

2.2.2 CHLOE Type of Profilometer (CHLOE).(Slides 26.14 - 26.16) The CHLOE device was developed at the AASHO Road Test as a simplified modification of the AASHO slope profilometer. Pavement roughness is measured by the change in angle between two reference lines, one of which is determined by two small wheels and the other which is determined by a 20-foot long frame member supported by two large rear wheels and a trailer hitch on the front.

2.2.3 Rolling Straightedge (RSE). (Slides 26.34 & 26.35) The rolling straightedge or profilograph has been used by several agencies. The device records a continuous chart profile in each wheel track. Two sets of bogey wheels 30-feet apart provide reference points from which a vertical displacement is measured by a recording wheel at the midpoint. The cumulative vertical displacement per mile is termed the roughness index.

2.2.4 British Road Research Laboratory Profilometer (RRL). (Slide 26.36) This device consists of an articulated carriage with four 4-wheel bogies of total width of 4-feet and wheel base length of 21-feet. The detector assembly at the center consists of a detector wheel mounted centrally on a vertical shaft postioned in vertical guides and trailed by two flanking wheels. A profile is plotted of the road surface in a natural vertical scale. Also, the number of bumps of different sizes are measured by means of a classificaiton.

2.2.5 Surface Dynamics Profilometer (SDP). (Slides 26.10 - 26.13) (Slides 26.25 - 26.28) The SDP is a system consisting of two road following wheels mounted on trailing arms beneath a van. Relative motion between the vehicle and the wheel is measured by a potentiometer. An accelerometer measures the acceleration of the vehicle itself. The signals go into an analog computer in the vehicle. A detailed evaluation of the SDP is contained in the next lession.

2.2.6 Car Road Meter, PCA, or Maysmeter (CRM). (Slides 26.37 - 26.49) The CRM type devices have become very popular with highway agencies during the past few years. It is a simple electromechanical device that measures the number and magnitude of vertical deviations between the body of the automobile and the center of the rear axle.

2.2.7 Precise Leveling Method for Profile Determination (LEVEL). This is the only viable method for determining actual profile information for pavements. Although this method is very simple and very accurate, it is extremely slow and painstaking.

3.0 CORRELATING THE OUTPUTS OF ROUGHNESS-MEASURING DEVICES

3.1 Reasons for Correlation (Slides 26.50 - 26.53)

(a) Calibration using a repeatable device to provide periodic checks for another device that may vary with time or use.
(b) Estimation using one device to estimate the output of another.

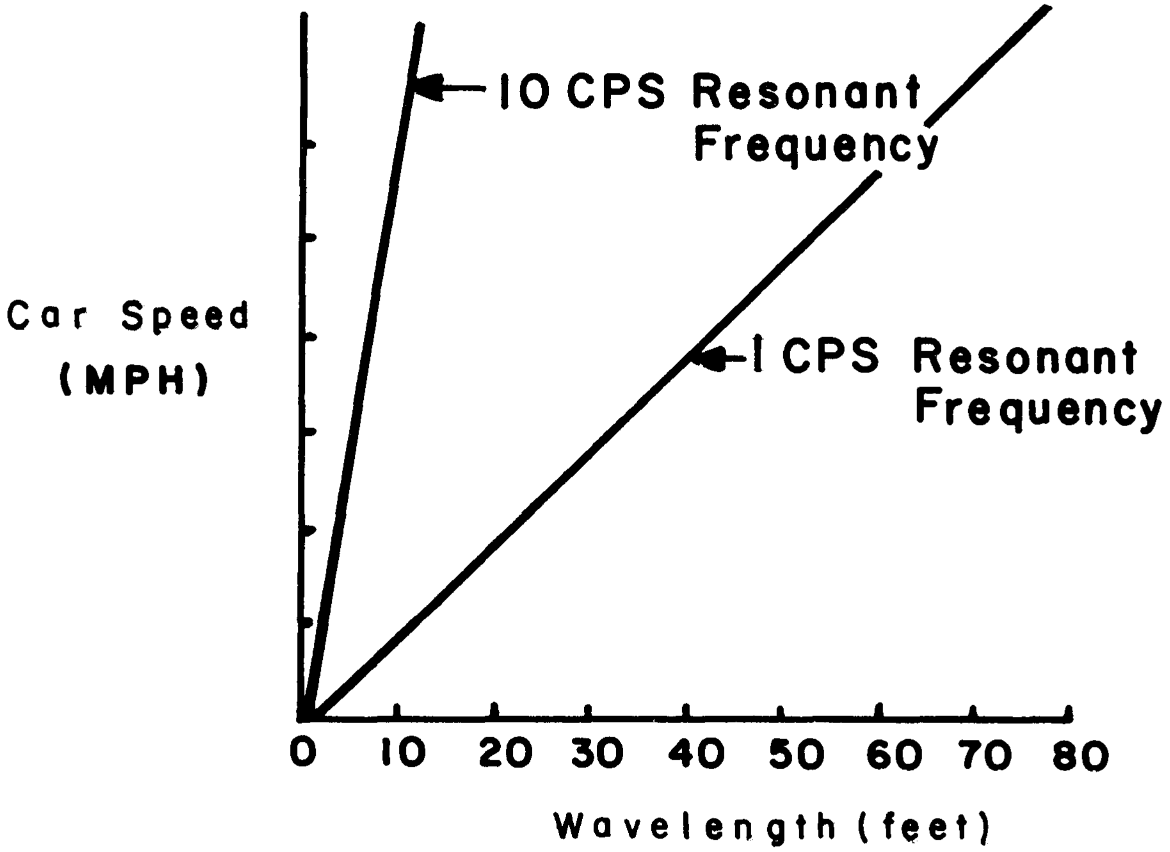
3.2 Areas of Applicability and Uses for Various Types of Roughness Measurement

Visual Aid 26.2 provides a tabular listing of the applicability of roughness measurements. It suggests that the overall approach should be concerned with purpose of measurement, applicable facility, use of data, and whether the primary interest is in estimating serviceability or some other purpose.

LESSON OUTLINE
FIELD MEASUREMENT OF SERVICEABILITY AND ROUGHNESS

VISUAL AID	TITLE
Visual Aid 26.1.	Relationships amoung resonant frequencies of cars, car speed, and pavement surface wavelength.
Visual Aid 26.2.	Areas of applicability and uses for various types of roughness measurements.

Visual Aid 26.1. Relationships among resonant frequencies of cars, car speed, and pavement surface wavelength.



Visual Aid 26.2. Areas of applicability and uses for various types of roughness measurements.

	Classes of measurement, by purpose		
Facility type	**Initial ride**	**Periodic ride**	**Terminal ride**
1. Expressway or primary highway	BPR, SDP, CRM, RSE[a] (RRL, CHLOE)[b]	CRM, SDP (RRL, CHLOE)	CRM, SDP (CHLOE, RRL)
2. Secondary (rural) highway	BPR, CRM, RSE (SDP, RRL, CHLOE)	CRM (SDP, RRL, CHLOE)	CRM (SDP, CHLOE, RRL)
3. County or local rural highway	CRM, BPR, RSE (SDP)	CRM	CRM
4. Runways	SDP, RRL, CRM	CRM, SDP, RRL (LEVEL)	SDP, RRL, LEVEL

Uses of roughness measurements, for all facility types	**Initial ride**	**Periodic ride**	**Terminal ride**
A. Construction monitoring	Yes[c]	–	–
B. Maintenance programming	–	Yes	Yes
C. Inventory and network programming	–	Yes	Yes
D. Research	Yes	Yes	Yes

INSTRUCTIONAL TEXT

FIELD MEASUREMENT OF SERVICEABILITY AND ROUGHNESS

FIELD MEASUREMENT OF SERVICEABILITY-PERFORMANCE AND ROUGHNESS

Use of Roughness Measurements in Estimating Pavement Serviceability

The major use of roughness measurements, which are objective, is for estimating pavement serviceability, which is subjective. Carey and Irick (Ref. 1) provided the most widely known means for this purpose in developing the Present Serviceability Index (PSI) equation at the AASHO Road Test. The original form of this equation is as follows:

$$PSI = C + (A_1R_1 + \ldots) + (B_1D_1 + B_2D_2 + \ldots) \quad (1)$$

where:

C = coefficient (5.03 for flexible pavements and 5.41 for rigid pavements)

A_1 = coefficient (-1.91 and -1.80 for flexible and rigid respectively)

R_1 = function of profile roughness $[\log(1 + \overline{SV})]$, where $\overline{SV}$ = mean slope variance obtained from the Road Test profilometer

B_1 = coefficient (-1.38 for flexible and 0 for rigid)

D_1 = function of surface rutting ($\overline{RD}^2$, where $\overline{RD}$ = mean rut depth as mesured by simple rut depth indicator)

B_2 = coefficient (-0.01 for flexible and -0.09 for rigid)

D_2 = function of surface deterioration ($C = P$), where $C + P$ = amount of cracking and patching, determined by procedures described in Reference 2

Given this general form of equation, it is necessary to determine the coefficients for a particular set of input variables. This was done at the Road Test for several sets of variables as reported by Carey and Irick (Ref. 1). It is important to understand that the resulting equation for the particular variables selected is a best-fit equation based on all observed data used in the equation. Other variables that were candidates for inclusion in the equation proved in the regression to add no significance in predicting PSI when added to the equation.

It should be remembered that a regression equation is not a causative relationship and that covariance between terms can account for very small coefficients on a variable that alone is only slightly less well correlated with the dependent variable. For example, a great deal of the observed roughness in a pavement is due to cracking, and therefore the two factors are highly correlated. Consequently, once the roughness term is included in the equation, little variation remains to be explained by adding the cracking terms and thus the coefficient is small. This does not indicate a lack of concern for cracking and is merely the form the equation takes. It is not satisfactory for users of the equation to alter the equation arbitrarily because they intuitively "feel" that cracking is more important. In fact, any such alteration is erroneous and produces unpredictable results.

Other forms of the PSI equation have been developed for other pavements and for other input variables.

Any change in measurement methods or units will result in a modified equation. This can be done either by performing an entirely new regression if all data are available, or by comparing the old measurement to the new and making an appropriate substitution in the equation. This was done at the Road Test for the BPR roughometer by comparing the roughometer output, R, in inches per mile, to mean slope variance, $\overline{SV}$. The resulting equation is

$$PSI = 5.41 - 1.80 \log (0.40R - 30) - 0.09\sqrt{C + P} \qquad (2)$$

with terms as previously defined.

The actual numerical values noted for all the coefficients in the equations were determined by Carey and Irick by multiple regression techniques for over 120 data points observed on existing pavements in 1958 and 1959. It must be emphasized, however, that the PSI model represented by Eq. (1) is not an end in itself. Carey and Irick made this quite clear in pointing out that it is intended to predict PSR to a satisfactory approximation. Unfortunately, this intention and use of the concept has been forgotten by many engineers in the ensuing years. Engineers are somewhat inherently "hostile" to the concept of a completely subjective evaluation as represented by PSR. They prefer to evaluate their structures by mesurable physical criteria that can be determined objectively. Consequently, the PSI concept also largely served the purpose of making available to engineers a type of tool with which they were more familiar and amenable to using.

The PSI equation was developed by multiple regression techniques, as previously noted. That is, a set of physical measurements were related to the subjective, user evaluations in terms of the mean panel rating values, PSR, described in Session 4. Although these physical measurements include condition or distress data (i.e., mean rut depth plus cracking and patching), it is roughness that provides the major correlation variable (i.e., correlation coefficients between PSR and PSI are increased by only about 5 percent after adding in the condition data). Thus, it should be emphasized that whenever PSI is calculated from physical measurement data, this is really only an estimate of PSR; that is,

$$PSI = PSR \pm E \qquad (3)$$

where E is an error term. In other words, contrary to what is all too often implied or stated in the literature, PSI and PSR are not two different ways of obtaining pavement serviceability. PSI is not an end within itself. It represents a means of using objectively obtained data to estimate a subjectively based parameter, as originally pointed out by Carey and Irick (Ref. 1) and subsequently by Haas and Hudson (Ref. 10)

The original Canadian evaluation studies previously noted (Refs. 4,5) also tried to relate panel ratings tp physical measurement data by multiple regression techniques (roughness data was not included). Although these efforts were relatively successful in explaining performance variations, the regressions were not significant enough as a predictive design tool for many pavement groups. Consequently, most agencies continued to make direct, periodic subjective ratings until the mid-1960s. At that time, a major program was initiated on relating these subjective ratings of Riding Comfort Index (RCI) to roughness measurements, primarily using the CRM type of devices (Ref. 6). Figure 4A-1 contains example correlations from the Canadian studies. A result of these studies was a set of recommendations relating to correlation and calibration procedures and to operating methods for the CRM (Refs. 6, 7, 8).

It should be noted that correlations such as those shown in Figure 4A-1 can change significantly among regions and with time. Thus, the recommendations noted in the preceding paragraph include periodic recalibration experiments.

Most efforts by U.S. agencies to correlate CRM output with serviceability have involved several steps. First, slope variance of a number of evaluation sections is measured with a CHLOE profilometer. This data is then used to calculate PSI. Next, CRM measurements are taken of the sections and these are correlated with the calculated PSI's. Figure 4A-2 is an example of such correlations for flexible and rigid sections in Wisconsin (Ref. 9).

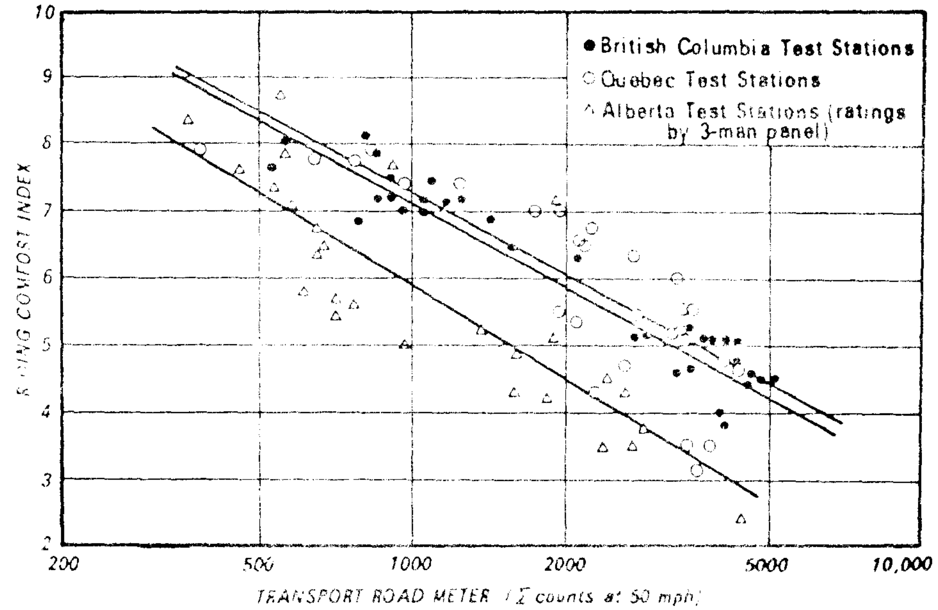


Figure 4A.1 Example correlation of Riding Comfort Index with CRM roughness measurements, from Canadian studies. After (8).

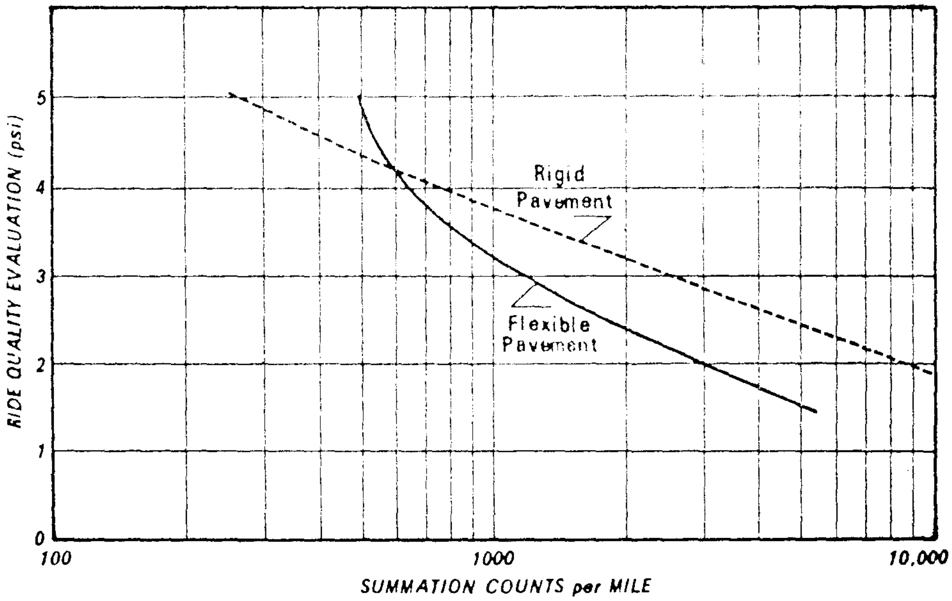


Figure 4A.2 Example correlations of PSI with CRM roughness measurements, for Wisconsin pavements. After (9)

There are two major questions that can be raised concerning approaches that go through a number of transformations to estimate PSI from CRM measurements:

1. The PSI equation itself (which is supposed to estimate PSR) may no longer be valid for the particular area of application
2. Transformations can compound errors, as demonstrated by Haas and Hudson (Ref. 10)

As a result, several agencies in the United States have developed their own serviceability equations, rather than use the PSI equation. Reference 11 is an example of this approach. The work by Canadian agencies (Ref. 6) is similar in approach. These efforts are based on the premise that it is necessary to conduct new rating panel sessions at periodic intervals (say, every 3 or 4 years) and to correlate the results with roughness measurements. The roughness device itself may have to be calibrated at much more frequent intervals.

It should be strongly emphasized that the serviceability-performance concept, as originally advanced by Carey and Irick (Ref. 1) has as its principal purpose the modeling or simulation of subjective user response or opinion. In other words, acceptance of the serviceability-performance concept as the primary output characterization of a pavement does not require acceptance or use of the PSI equation at all. There will undoubtedly continue for some time to be a variety of equations used to estimate user opinions, combined with the changes of these opinions with time. Unfortunately, there are still some misconceptions with regard to the foregoing concepts and principles. These seem to arise mainly over the fact that performance (i.e., the serviceability-age history) has a precise meaning in the Carey-Irick formulation, and over the fact that the PSI equation developed at the AASHO Road Test represents only one of the many possible means of estimating serviceability.

Precautions in Using Subjective Measures

If one accepts the premise that pavements are provided for the user, then one must employ some measure of user response in analysis and evaluation. This user response is in terms of an entirely subjective opinion, as indicated in the serviceability-performance concept. Because the methodology for modeling such subjective opinions or ratings has been developed primarily in the field of psychology, engineers are often unaware of its features and its limitations.

The literature on this subject, termed psychophysical scaling, is extensive. Of particular interest to the pavement engineer is the work of Stevens (Ref. 12), who classified measurements on the basis of the transformations that leave the scale form invariant. Hutchinson (Ref. 3), and subsequently Haas and Hudson, have shown that the considerations presented by Stevens are particularly relevant to the pavement field in terms of the validity of certain statistical manipulations that are performed on evaluation data. These considerations should be carefully reviewed when devising experiments to relate subjective user opinions to objective mechanical measurements, and when the results are interpreted and applied to design.

There are also several major assumptions involved in acquiring or modeling user opinions themselves. Such as the PSR's of the AASHO Road Test or the RCI's of the Canadian studies. These assumptions neglect the following systematic errors that can occur:

1. Leniency error (i.e., a rater's tendency, for various reasons, to rate too high or too low)
2. Halo effect (i.e., rater's tendency to force a particular attribute rating toward his or her overall impressions of the object)

3. Central tendency error (i.e., a rater's hesitation to give extreme judgments, thereby tending ratings toward the mean of the rating panel)

A number of guidelines for constructing rating scales, and a discussion of the precautions to be used in interpretation, have been presented by Hutchinson (Ref. 3) and by Haas and Hudson (Ref. 10). They have suggested that careful consideration of these guidelines and precautions can lessen the incompatibilities in pavement evalution that often exist both within and between agencies.

Hidden Errors in PSI Estimates

One of the sources of potentially large errors in current methods of present serviceability evaluation has escaped the attention of many users. The reason is that the errors are "hidden" by using or assuming previous correlations to be perfect. To illustrate this situation, we can recall that the initial present serviceability equations as previously discussed are multiple regression equations, with correlation coefficients of about 0.8 and 0.9 and a standard error of ±0.3 to 0.4 PSI units. But this correlation is valid only for the original AASHO Road Test profilometer. That device has been used only for research, because it is too big to be of practical use on highways. Consequently, the most widely used PSI equation invovles the CHLOE profilometer. This equation was obtained by correlating the roughness estimate from the CHLOE device with the AASHO Road Test device on several pavement sections during the Road Test in 1961.

Because there is error in both mesurements, the true correlation coefficient of CHLOE profilometers with the panel ratings becomes more erroneous. Recently, users of the new instruments, as subsequently illustrated, have gone one step further by correlating their device with a CHLOE device that is not the original but a later model. Thus, if they use the original PSI equations, as most do, they are three or four steps away from the original correlation data. That true correlation of such a process is quite low, and the probable error of such estimates is quite high.

The concept of these hidden cumulative errors can be expressed most simply by looking at the mathematics involved, taking the estimated relationships for PSI, and moving through a correlation between the AASHO profilometer and the CHLOE profilometer to some third device as follows. Rewriting the original AASHO PSI euation (Eq. 1) in the form

$$PSI = A_0 + A_1 \log (1 + \overline{SV}) \pm e_1 \qquad (4)$$

where e_1 is the error of estimate, and A_0, A_1 are coefficients, and using a correlation between CHLOE and AASHO profilometers,

$$\log (1 + \underline{SV}) = B_0 + B_1 [f(CHLOE)] \pm e_2 \qquad (5)$$

where e_2 is the error of estimate, and B_0, B_1 are coefficients, it follows, by direct substitution, that

$$PSI = C_0 + C_1 [f(CHLOE)] \pm A_1 e_2 \pm e_1 \qquad (6)$$

where C_0, C_1 are coefficients. It can be seen that the importance of the term $A_1 e_2$ and e_1 is not their radomness but their magnitude. If, in addition, we correlate a third type of roughness device, RM, as follows:

$$f(CHLOE) = D_0 + D_1(RM) \pm e_3 \qquad (7)$$

where e_3 is the error of estimate, and D_0, D_1 are coefficients, it follows that

$$PSI = F_0 + F_1(RM) \pm [C_1 e_3 \pm A_1 e_2 \pm e_1] \qquad (8)$$

where F_0, F_1 are coefficients. The total error therefore is that given in brackets $[C_1e_3 \pm A_1e_2 \pm e_1]$, not e_3 alone.

The best way to eliminate this problem is to form a new pavement rating panel and to correlate the results directly with the particular roughness instrument of interest. This has been done in Texas for both the CHLOE profilometer (Ref. 13) and the surface dynamics profilometer (Ref. 11), and in Canada.

Toward Achieving Better Compatibility in Serviceability-Performance Evaluation

Highway agencies are increasingly becoming conscious of the importance of pavement performance evaluation. Many agencies have put considerable effort into developing, applying, and analyzing serviceability measuring schemes. This is certainly encouraging; however, it has also led to a proliferation of methods and data, many of which are unfortunately incompatible with other data. This lack of compatibility is essentially dual in nature:

1. "External" compatibility, relating to whether the results of one agency's work have any quantitative relation or meaning to those of another agency, and

2. "Internal" compatibility, relating to correlating results, achieving repeatability, etc., within an agency.

It seems apparent from various conferences and studies, and from engineering reason based on experience with other structures, that better compatibility in pavement performance evaluation is desirable. Consequently, the following suggestions are directed toward this goal:

1. Performance evaluation of pavements should be established on a planned basis to become an integral part of the overall pavement management system.

2. An automated data feedback system is a most useful and perhaps necessary component of the performance evaluation scheme.

3. The existing definitions of serviceability, and its components, should be clearly understood, as well as the underlying assumptions. Moreover, it should be explicitly recognized that serviceability measures, such as those developed at the AASHO Test and in Canada, are not ends within themselves; they exist to estimate the road user's opinion.

4. There are a variety of possible errors in subjective evaluations of serviceability. These can be significant, and it is important that the principles underlying subjective rating scale design and analysis are well understood. Because such principles have not been a "normal" part of engineering analysis, they have been somewhat neglected in much of our current methodology. It seems necessary, however, that such understanding be achieved if we are to make significant progress toward better compatibility. This book has presented some pertinent discussion on this problem area and has noted the major references that should be examined by those involved with pavement serviceability analyses.

5. Serviceability measures can be conveniently approximated, for many practical purposes, by condition surveys, roughness measurements, or a combination of the two. However, it must be realized that any serviceability predicted from a "unique" method of surveying structural condition is only qualitatively compatible with any other measure. Predictions from roughness measurements can be quantitatively compatible, but it must be

recognied that because of the nature of subjective evaluations, the interpretation and use of a serviceability measure is unique to the particular region.

6. The problems of internal compatibility often seem to be related to lack of correlations and replications. These can perhaps be largely controlled by carefully designed experiments, so that proper statistical analyses may be conducted.

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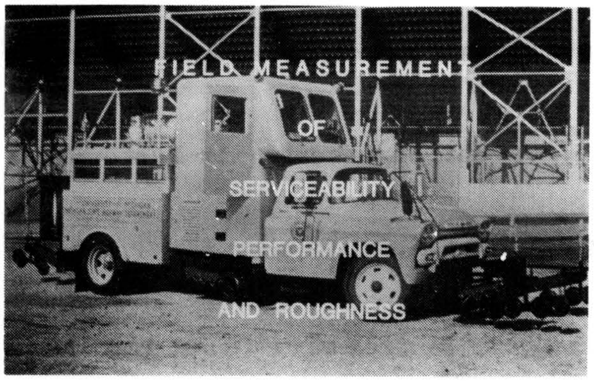
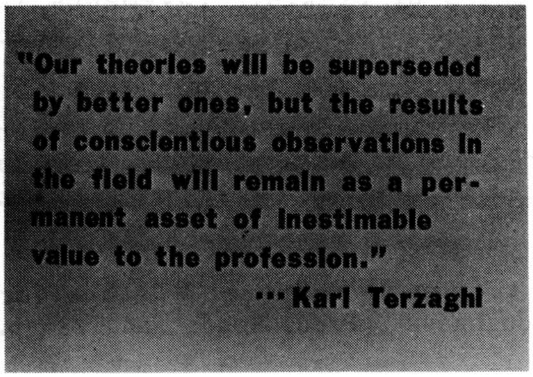


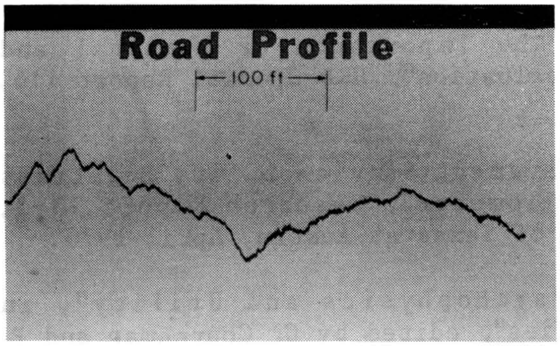
FIELD MEASUREMENT
OF
SERVICEABILITY
PERFORMANCE
AND ROUGHNESS

Slide 26.1. Title Slide - Field Measurements



"Our theories will be superseded by better ones, but the results of conscientious observations in the field will remain as a permanent asset of inestimable value to the profession."
··· Karl Terzaghi

Slide 26.2. Question from Karl Terzaghl, Father of Soil Mechanics.



Road Profile

Slide 26.3. Roughness depicted graphically through road profile.

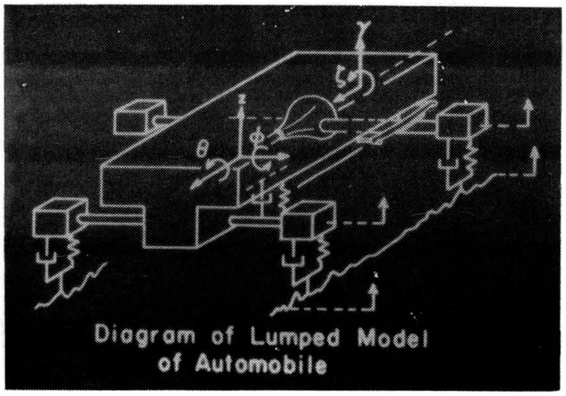
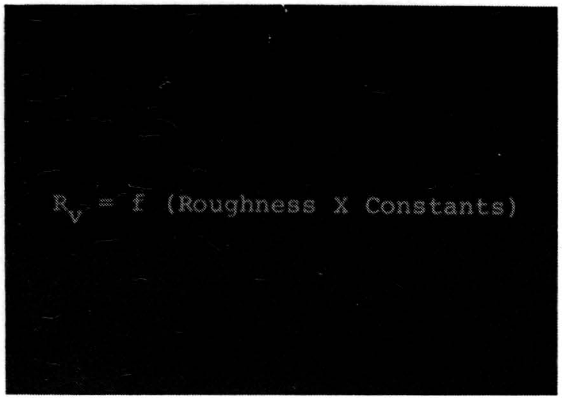


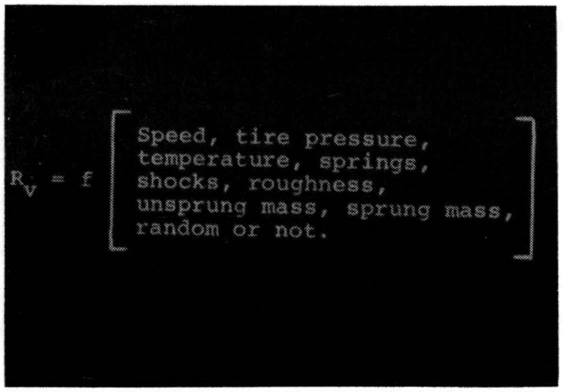
Diagram of Lumped Model of Automobile

Slide 26.4. Lumped model of an automobile - illustrates how road roughness will affect rideability.



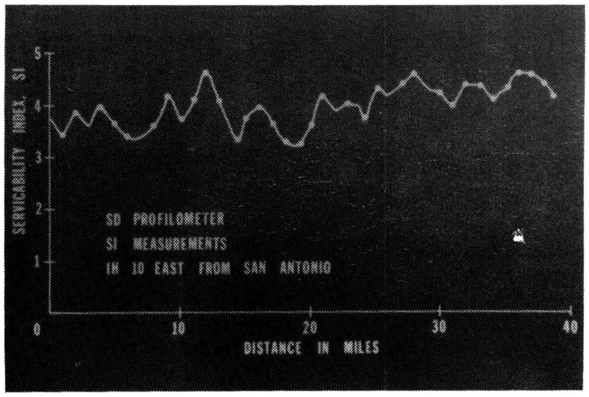
R_V = f (Roughness X Constants)

Slide 26.5. Ride quality is a function of roughness.

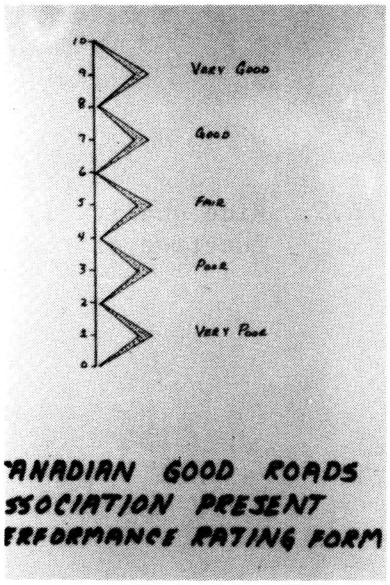


R_V = f [Speed, tire pressure, temperature, springs, shocks, roughness, unsprung mass, sprung mass, random or not.]

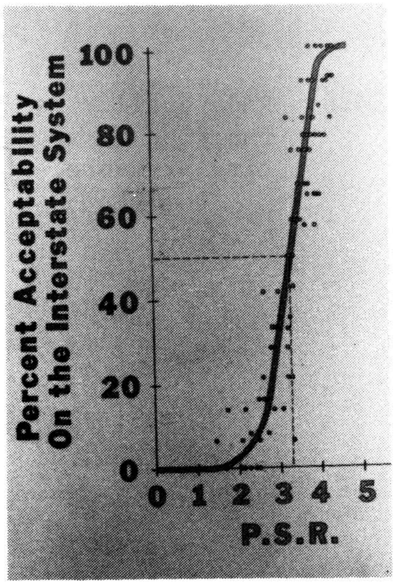
Slide 26.6. Ride quality is a function of speed, tire pressure, etc.



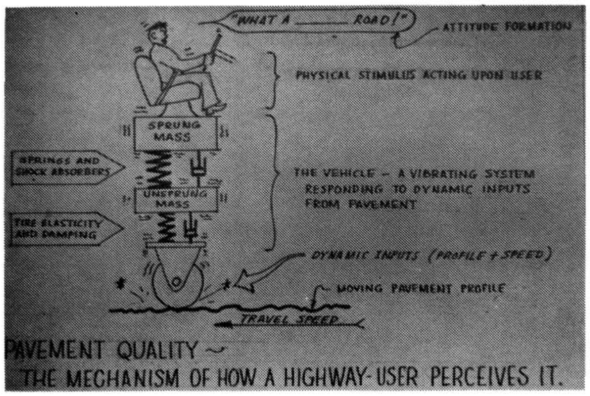
Slide 26.7. Profilometer generated plot - sample.



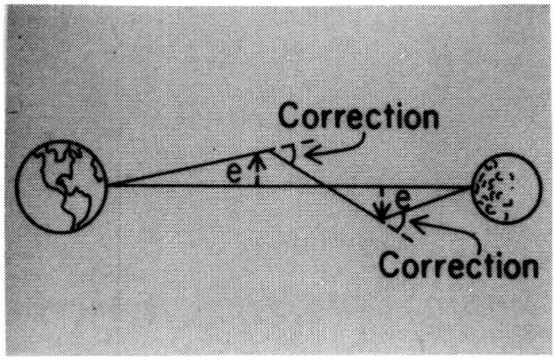
Slide 26.8. 10-point scale of road serviceability rating - used in Canada.



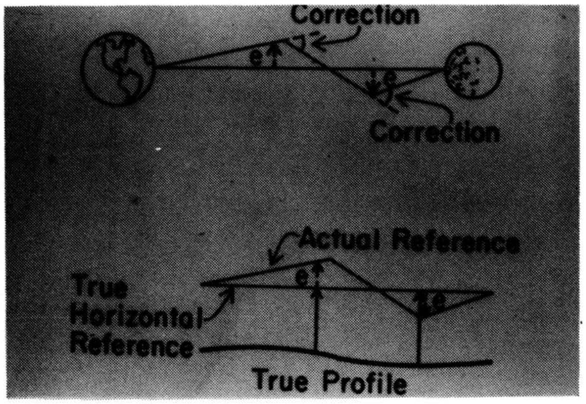
Slide 26.9. Relationship between highway class and acceptable level of PSR.



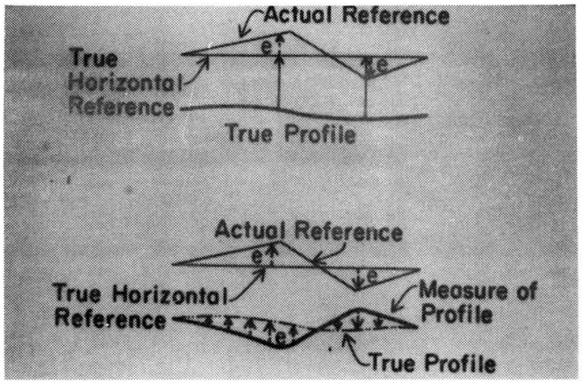
Slide 26.10. Factors affecting ride quality.



Slide 26.11. CHLOE profilometer principle.

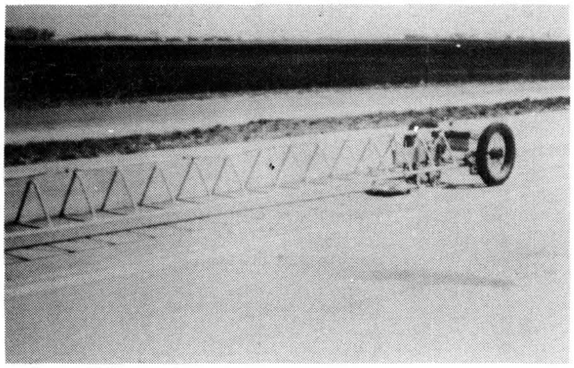


Slide 26.12. CHLOE profilometer principle.

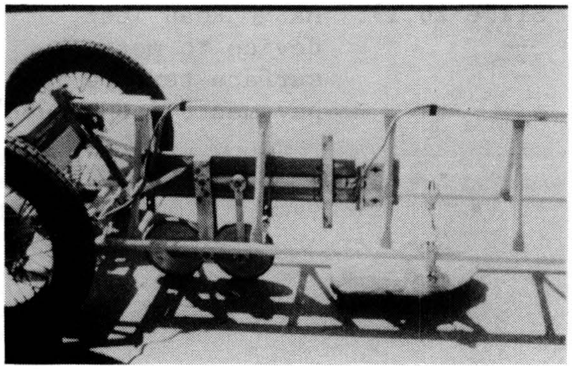


Slide 26.13. CHLOE profilometer principle.

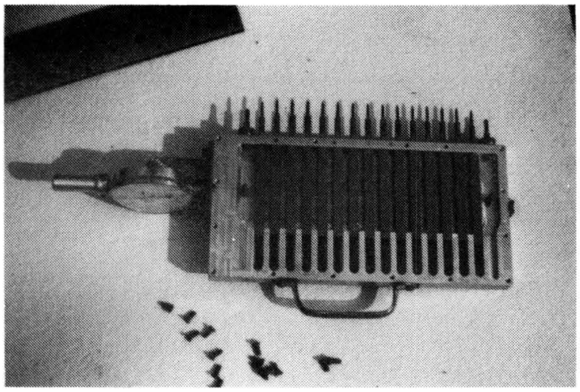
Slide 26.14. CHLOE profilometer - ready for use.



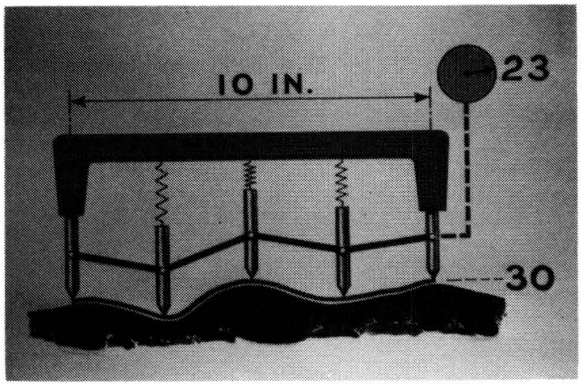
Slide 26.15. CHLOE profilometer - close up.



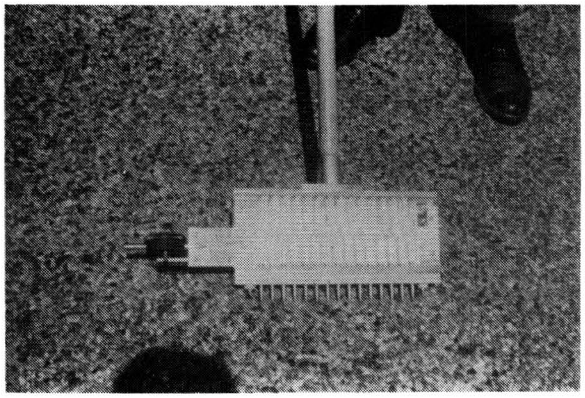
Slide 26.16. CHLOE profilometer - details.



Slide 26.17. AASHO Road Test device to measure surface texture in pavements.

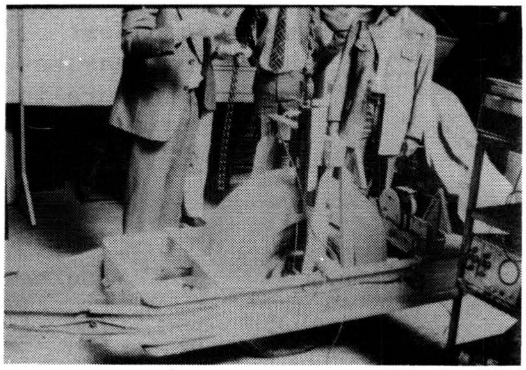


Slide 26.18. AASHO Road Test device to measure surface texture in pavements (continued).

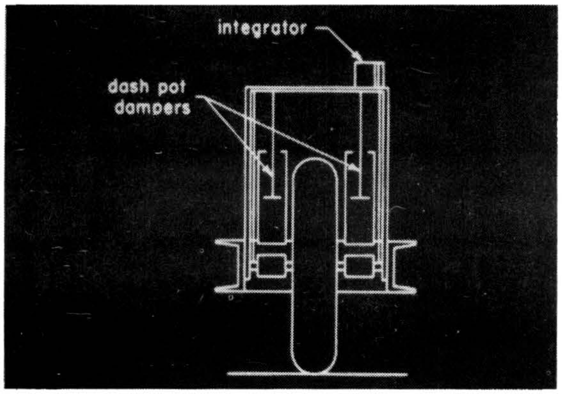


Slide 26.19. AASHO Road Test device to measure surface texture in pavements (continued).

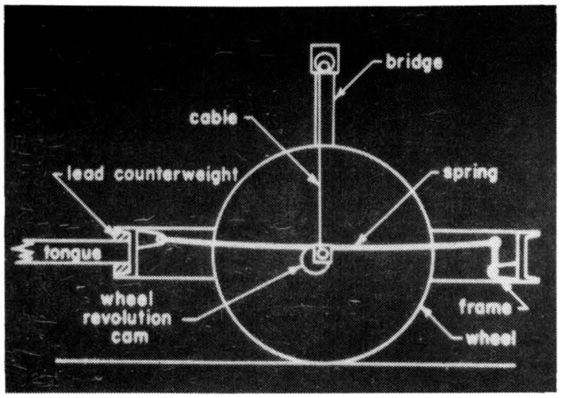
Slide 26.20. The BPR roughometer - ready for use.



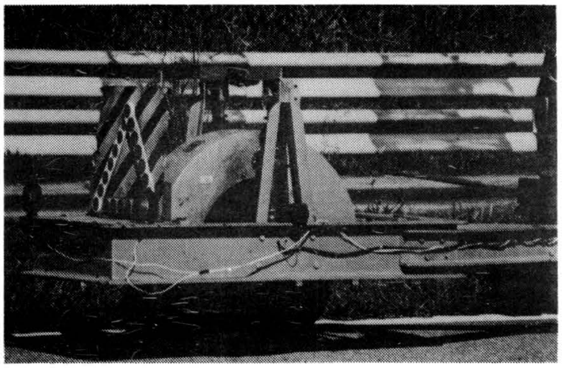
Slide 26.21. The BPR roughometer details.



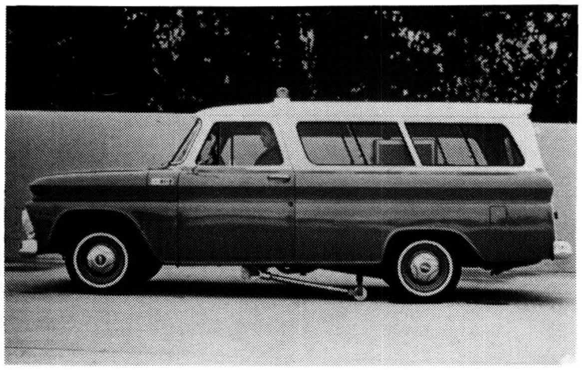
Slide 26.22. The BPR roughometer schematic diagram.



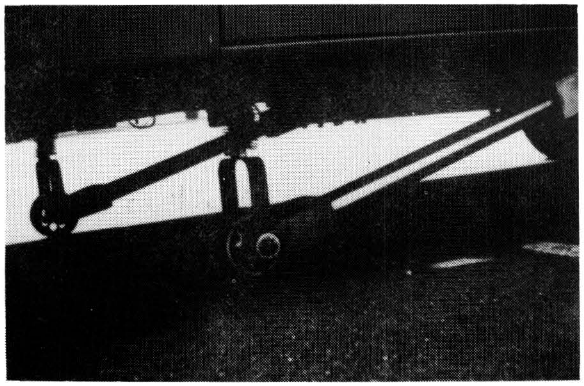
Slide 26.23. The BPR roughometer schematic diagram.



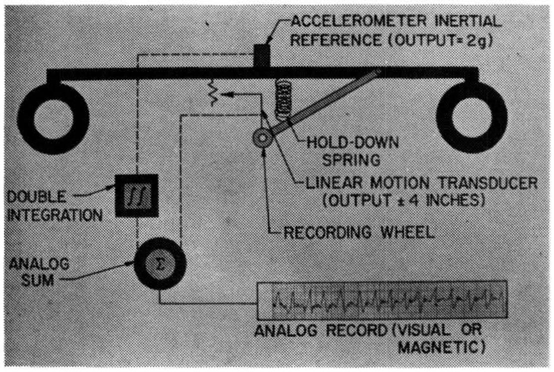
Slide 26.24. The BPR roughometer.



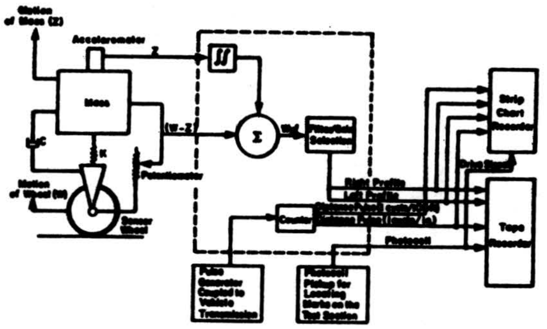
Slide 26.25. Surface Dynamics Profilometer - mounted under the vehicle.



Slide 26.26. Surface Dynamics Profilometer - close up view.

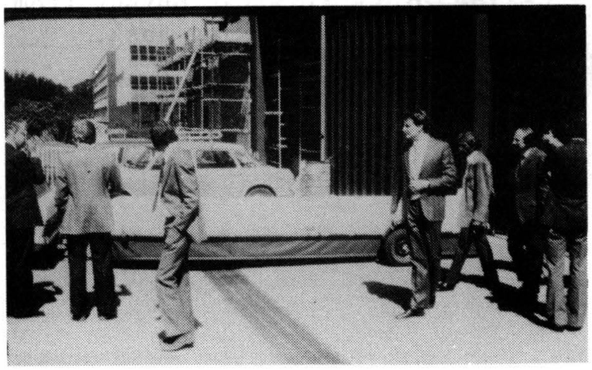


Slide 26.27. Surface Dynamics Profilometer - working principle.

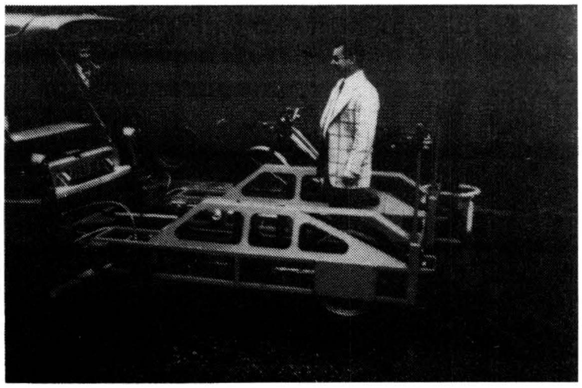


Slide 26.28. Surface Dynamics Profilometer - theory of operation.

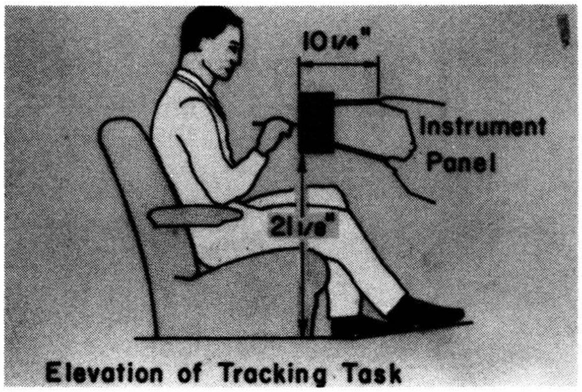
Slide 26.29. The TRRL high-speed profilometer uses four laser beams.



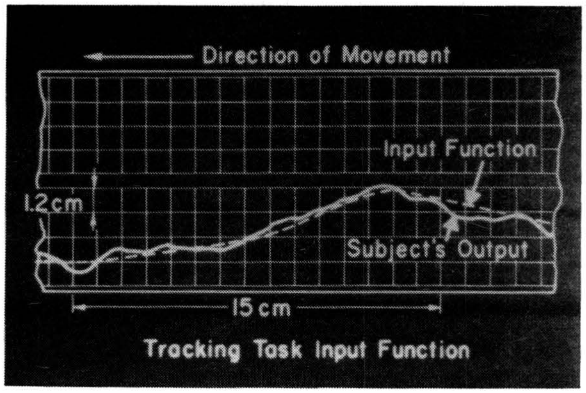
Slide 26.30. Another view of the TRRL profilometer.



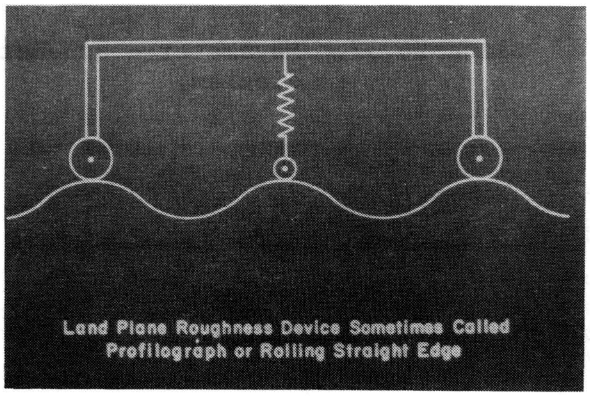
Slide 26.31. The French profilometer LPC which uses a mechanical pedulum.



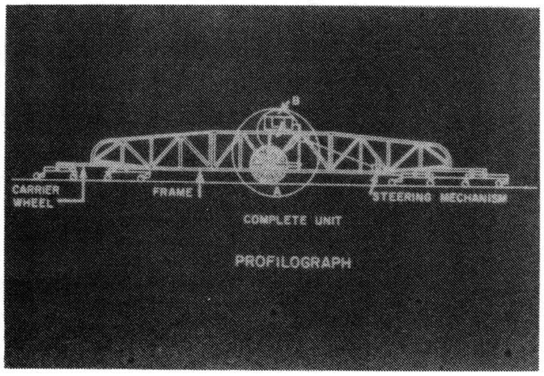
Slide 26.32. Operation position and instrument panel.



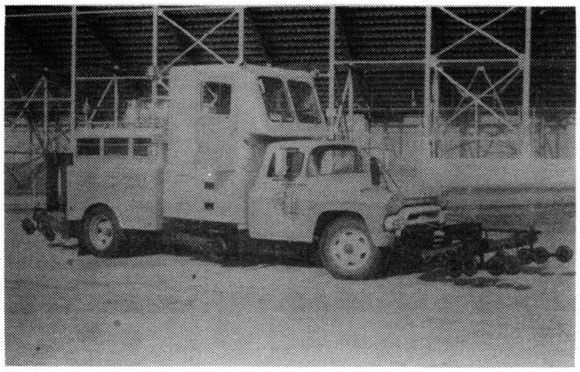
Slide 26.33. Graphic output from tracking task.



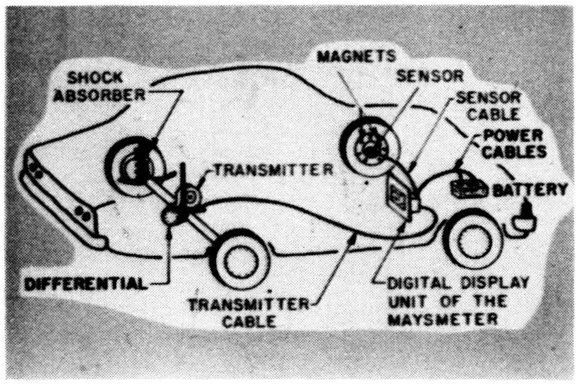
Slide 26.34. Rolling straight edge profilometer - principle.



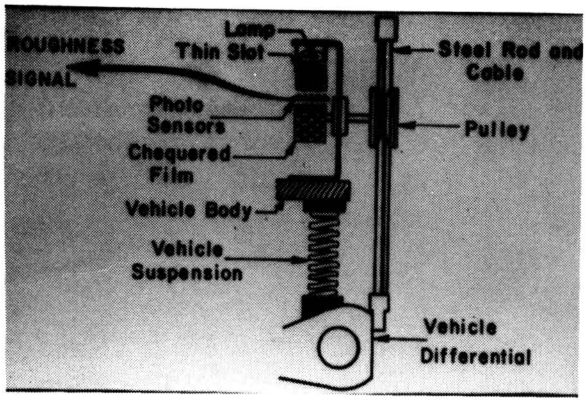
Slide 26.35. Rolling straight edge profilograph.



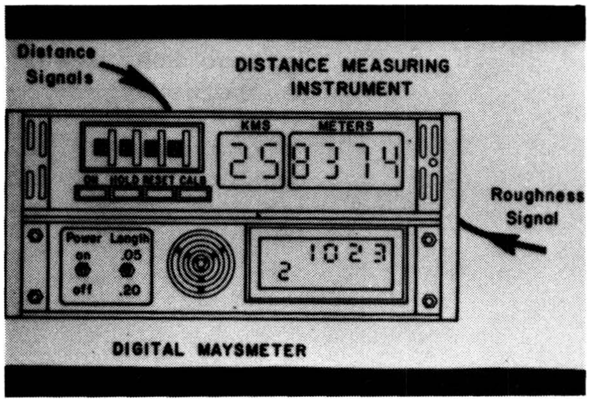
Slide 26.36. British Road Research Laboratory profilometer.



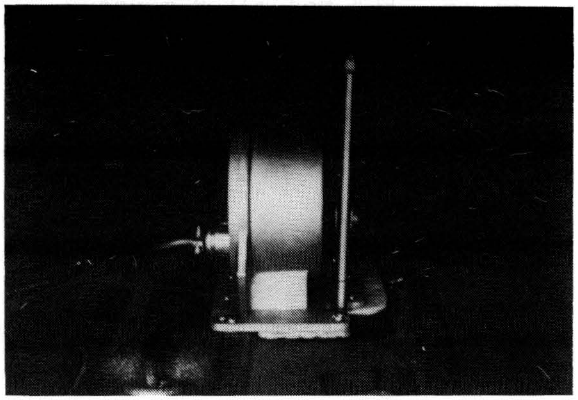
Slide 26.37. Maysmeter - schematic diagram.



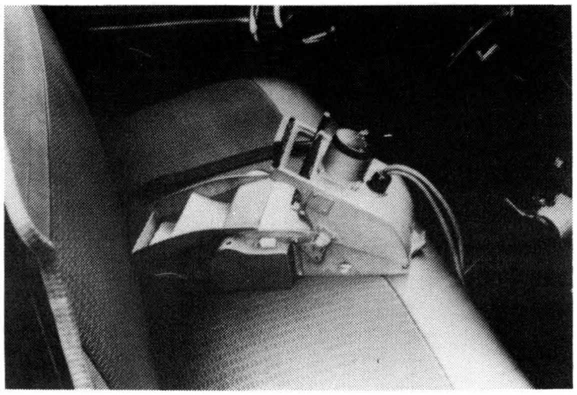
Slide 26.38. Detail description of the Maysmeter functioning.



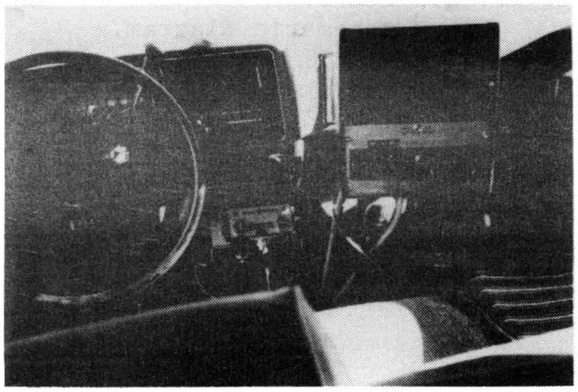
Slide 26.39. Maysmeter - digital display.



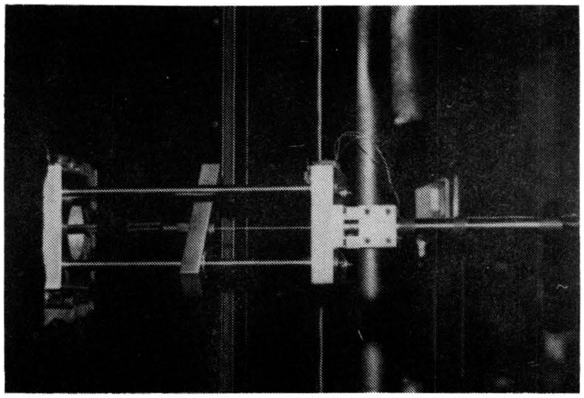
Slide 26.40. Actual Maysmeter mounting on a car.



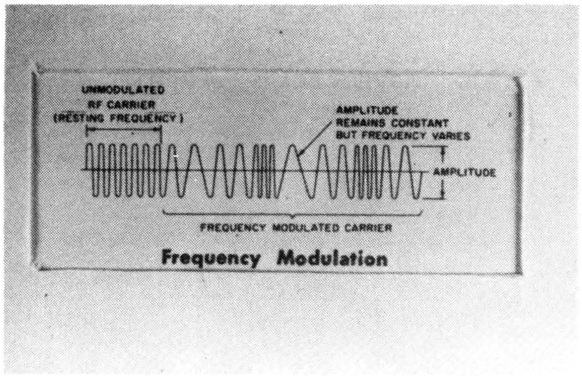
Slide 26.41. Maysmeter - graphic output.



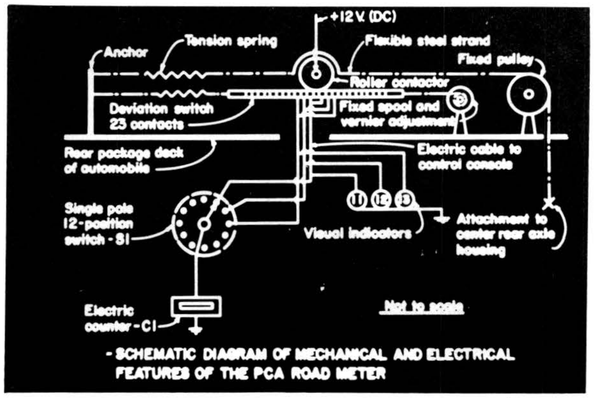
Slide 26.42. Digital output of the PCA roadmeter.



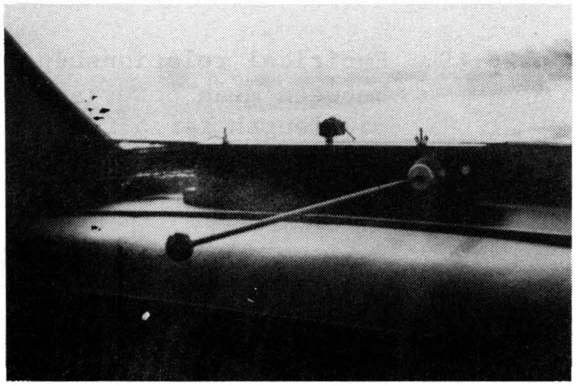
Slide 26.43. PCA roadmeter mounted on a car.



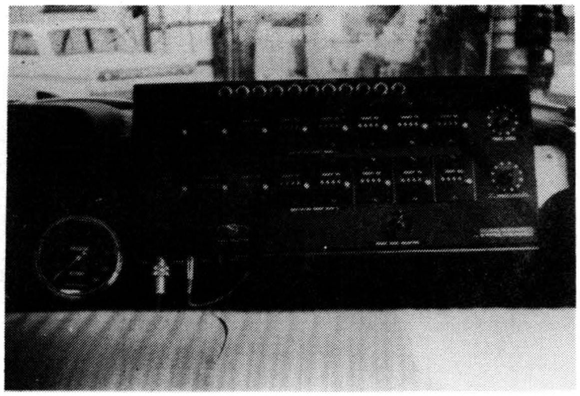
Slide 26.44. The effect of frequency modulation.



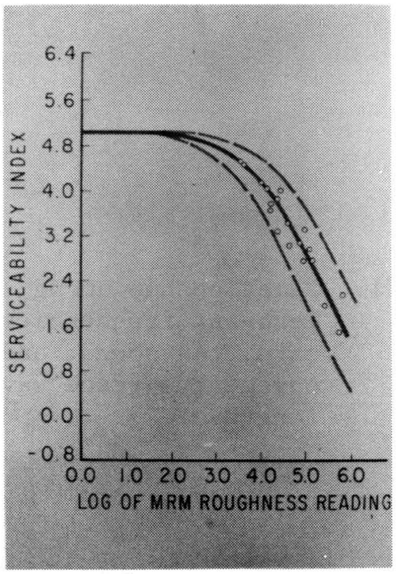
Slide 26.45. PCA roadmeter - schematic diagram.



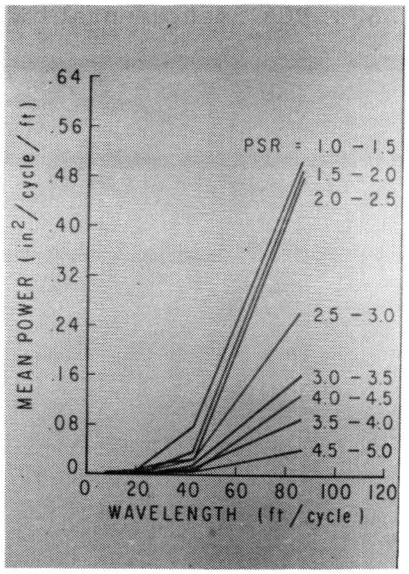
Slide 26.46. PCA instrumentation.



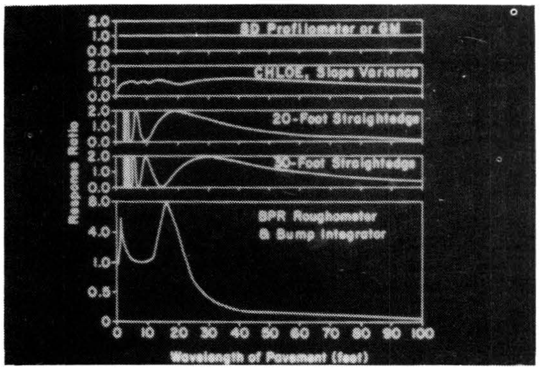
Slide 26.47. PCA instrumentation.



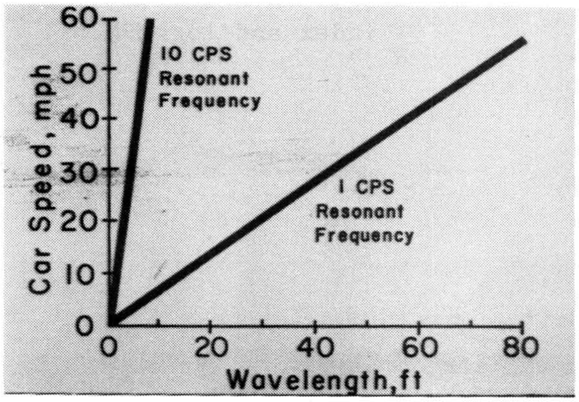
Slide 26.48. Empirical relationship between serviceability index and log MRM.



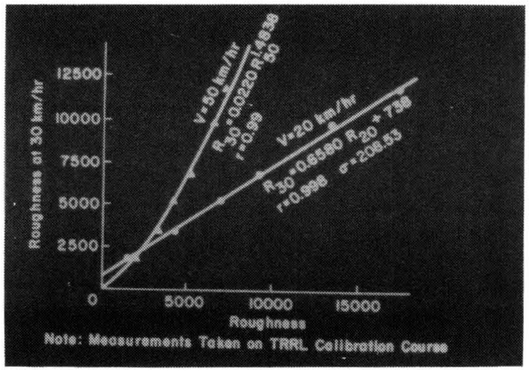
Slide 26.49. **Empirical relationship** between mean power and wavelength for different PSR values.



Slide 26.50. Theoretical differences between SD profilometer, CHLOE, Rolling straight edge, and Seis roughometer.

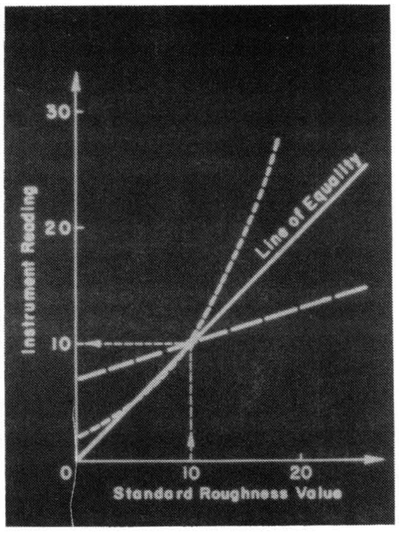


Slide 26.51. Relationship among resonant frequencies of cars, car speed, and pavement surface wave length.



Note: Measurements Taken on TRRL Calibration Course

Slide 26.52. Relationship between surface roughness and speed of vehicle.



Slide 26.53. Single-point BPR calibration problem.

LESSON OUTLINE
ROUGHNESS MEASUREMENT TECHNIQUES

Instructional Objectives

1. To introduce the SD profilometer as a stable profile device.
2. To discuss the use of the SD profilometer to calibrate response type roughness meters.

Performance Objectives

1. The student should be able to explain briefly how the accelerometer-potentiometer system in the SD profilometer measures road profile.
2. The student should be able to explain how the RMSVA statistics are computed and how they are used.

Abbreviated Summary	Time Allocation, min.
1. SD Profilometer	10
2. Root Mean Square Vertical Acceleration	20
3. Mays Ride Meter Calibration	15
4. Summary	5
	50 minutes

Reading Assignment

1. Haas and Hudson, Chapter 7
2. RTAC - Canadian Guide, Chapter 4; Section 4.3
3. Instructional Text

LESSON OUTLINE
ROUGHNESS MEASUREMENT TECHNIQUES

1.0 SURFACE DYNAMICS PROFILOMETER (Slide 27.1)

With the development of the SD profilometer computerized, repeatable, stable roughness measurements can easily be obtained.

1.1 Objectives of Roughness Evaluations (Slides 27.2 - 27.4)

Define a set of roughness indices with the following properties:

(a) reflects the degree of roughness in different frequency ranges,
(b) simple to compute,
(c) insensitive to the particular profile measuring device or method,
(d) definition in conceptionally simple,
(e) can be compared by statistical methods that shows high correlations to roughness measuring devices (Slides 27.5 & 27.6).

1.2 Basic Concepts of the SD Profilometer

The SD profilometer is complex electromechanical device consisting of an accelerometer-potentiometer configuration utilizing an on-board computer for data collection and analysis.

(a) Accelerometer - measures the vertical motion of the vehicle.
(b) Potentiometer - measures the change on the distance from the bottom of the vehicle to the road surface.
(c) On-board Computer - double integrates the signal from the accelerometer and adds the resulting displacement to the potentiometer signal. This gives a dynamic measurement of road profile which can be analyzed.

2.0 ROOT MEAN SQUARE VERTICAL ACCELERATION (RMSVA)

RMSVA is a road profile statistic which provides measure of different frequencies which make up a road profile.

2.1 Base Length (Slide 27.7)

The base length is the distance between two data points. The shortest base length use for calculations currently is 6 inches. The range of base lengths considered are:

(a) 0.5 ft
(b) 1.0 ft
(c) 2.0 ft
(d) 4.0 ft
(e) 8.0 ft
(f) 16.0 ft
(g) 32.0 ft
(h) 64.0 ft
(i) 128.0 ft

2.2 RMSVA Defined (Slides 27.8 & 27.9)

RMSVA at base length (b) is proportional to the root-mean-square difference between adjacent slopes connecting points that are "B" distance apart. The result is calculated for each base length.

3.0 CORRELATION WITH MAYS RIDE METER (Slide 27.10 and Slides 27.26 - 27.32)

The response of the Mays Ride Meter has been shown to be sensitive to only some of the road profile frequencies.

3.1 Linear Correlation (Slide 27.14)

A reasonable linear correlation exists between the Mays Ride Meter and the RMSVA. The correlation best fits with a combination of the four and sixteen foot wavelengths.

3.2 Mays Ride Meter Calibration (Slides 27.11 - 27.22)

The State of Texas uses 25 test sections for Maysmeter calibration. These sections are profiled quarterly with the SD profilometer. The RMSVA statistics are then used to calibrate the Maysmeters which are used for highway inventory.

3.3 Correlation with Rod-and-Level Survey (Slide 27.13)

The SD profilometer has been correlated to rod-and-level surveys using the established test sections. The correlation is reasonably linear.

4.0 SUMMARY (Slide 27.23)

The SD profilometer can provide a stable reference with which to calibrate less expensive response type roughness inventory devices.

4.1 RMSVA (Slide 27.24)

The Root-Mean-Square Vertical Acceleration is a useful statistic has successfully characterized pavement roughness.

4.2 Road Profile (Slide 27.25)

A road section exhibits a spectrum of frequencies making up overall roughness. The frequencies can be measured and analyzed separately using RMSVA.

Revised RF/lg 1/9/84
Lesson 27

INSTRUCTIONAL TEXT

RIDE QUALITY EVALUATION FOR PAVEMENT PERFORMANCE

by

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The Dewitt C. Greer Centennial
Professor of Transportation Engineering
The University of Texas at Austin
Austin, Texas

Submitted for

Publication in the
Transportation Journal of
The American Society
of Civil Engineers

(Presented at the National ASCE Meeting
Philadelphia, PA - May 16, 1983)

ABSTRACT

This paper is one of a group of four papers summarizing the evaluation of existing pavements and determination of appropriate overlay designs. Roughness, along with condition evaluation and non-destructive structural evaluation provide important inputs into the overall pavement evaluation process. In particular, roughness as it relates to serviceability and performance is an extremely useful evaluation technique for long term monitoring of pavement performance. If compatible roughness measurements are taken and faithfully recorded over a long period of time, they can be extremely useful in upgrading pavement evaluation methods.

The paper also summarizes the importance of rational and compatible road roughness measurements and points out some of the problems and possible methods for making such compatible measurements. The problems of calibrating and correlating roughness measurements with relatively simple devices such as the Mays meter, are also covered. Reference is made to the Texas method of analyzing rod and level surveys or other true profiles to provide calibration techniques for the so-called simple devices.

INTRODUCTION

One of the primary operating characteristics of a road, whether paved or unpaved, at any particular time is the level of service that it provides to its users. In turn, the variation of this level of service or serviceability with time provides one measure of the road's performance. This performance, and the cost and benefit implications thereof, are the primary outputs of a pavement management system. In 1960, Carey and Irick (Ref 5) showed that surface roughness was the primary variable needed to explain the driver's opinion of the quality of serviceability provided by a pavement surface, e.g., its desirability for use. More recently, research has shown that user costs are also related to roughness, particularly on rougher paved roads and unpaved roads. The Kenya Highway Design Standards Study conducted by the U. K. Transport and Road Research Laboratory and the World Bank from 1971 to 1975 demonstrated the relationship of vehicle operating costs to road roughness (Ref 34). Preliminary results of a similar study in Brazil give the same general conclusions (Ref 35).

From the time the first highway was built, both the engineers and users of highways have made judgments relative to the quality of the highway. In the very early days these were very general statements, related primarily to whether the road was passable or impassable. In general the "service life" or how long the roadway continued to adequately serve the riding public was the major criteria for whether or not a road was "good" or "bad". This type of quality evaluation scheme is best typified in Fig 1. This pavement evaluation scheme was used for many years, from the earliest Roman roads until the mid-1950's.

At the WASHO Road Test it was essential to define pavement quality, in order to compare the service life (performance), of pavement sections subjected to either identical traffic but with slightly different surface thicknesses, or identical pavement design sections subjected to different applications of axle loads. Mr. William N. Carey, Jr., the Chief Engineer of the WASHO Road Test, recognized the need for defining pavement quality but was unable to implement a new definition until the beginning of the AASHO Road Test in 1958 (Ref 5). The "serviceability-performance concept" initiated by Carey and Irick (Ref 5) quantitatively outlines a method for pavement performance evaluation which includes:

(1) the measure of surface roughness,

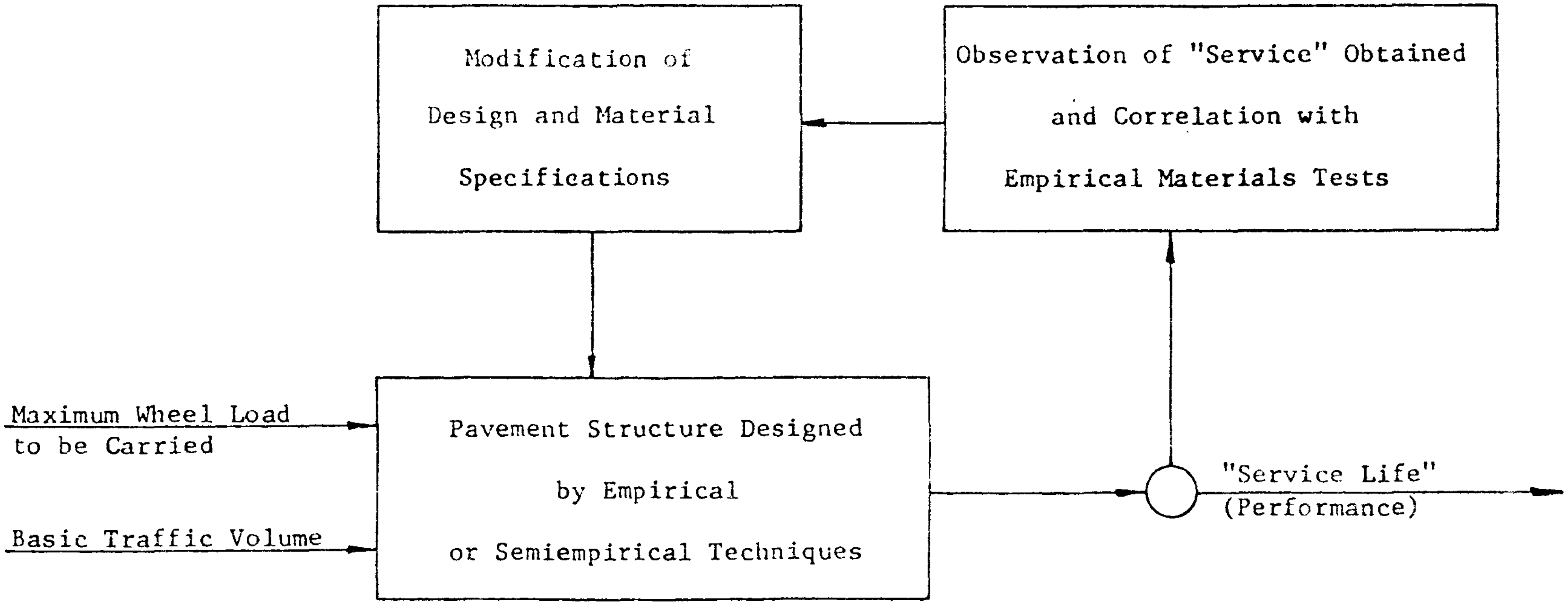


Figure 1. Block Diagram illustrating service life evaluation of pavement quality.

(2) the measurement of surface distress such as cracking and patching, and

(3) the combination of these factors into a serviceability "index".

This index is an objective measurement intended to approximate the subjective rating of a group of typical highway users and engineers.

Since the AASHO Road Test many new pavement evaluation methods have developed. These basically fall into three categories:

(1) Evaluation of pavement distress or surface condition.

(2) Evaluation of pavement riding quality and objective roughness measurements.

(3) Evaluation of pavement serviceability rating by a panel of one or more human evaluators.

The relationship for these methodologies are shown in Fig 2 and 3. A similar evaluation concept has been shown in the book by Haas and Hudson and is reproduced in Fig 4 (Ref 40). In either case, it is important to realize that evaluation of surface damage or distress does not replace roughness or serviceability evaluation, but rather supplements and combines with it to yield a better overall picture of the pavement quality and pavement performance.

Specifically roughness is one of three major approaches involved in pavement evaluation. People often misunderstand these approaches and feel that they replace each other or interfere with each other; they do not. As shown in Fig 4 condition surveys evaluate pavement distress, deflection or non-destructive methods evaluate pavement behavior and roughness evaluates pavement serviceability and thus performance. Other papers treat the aspect of condition surveys and non-destructive testing. Thus, in this paper we will consider only roughness.

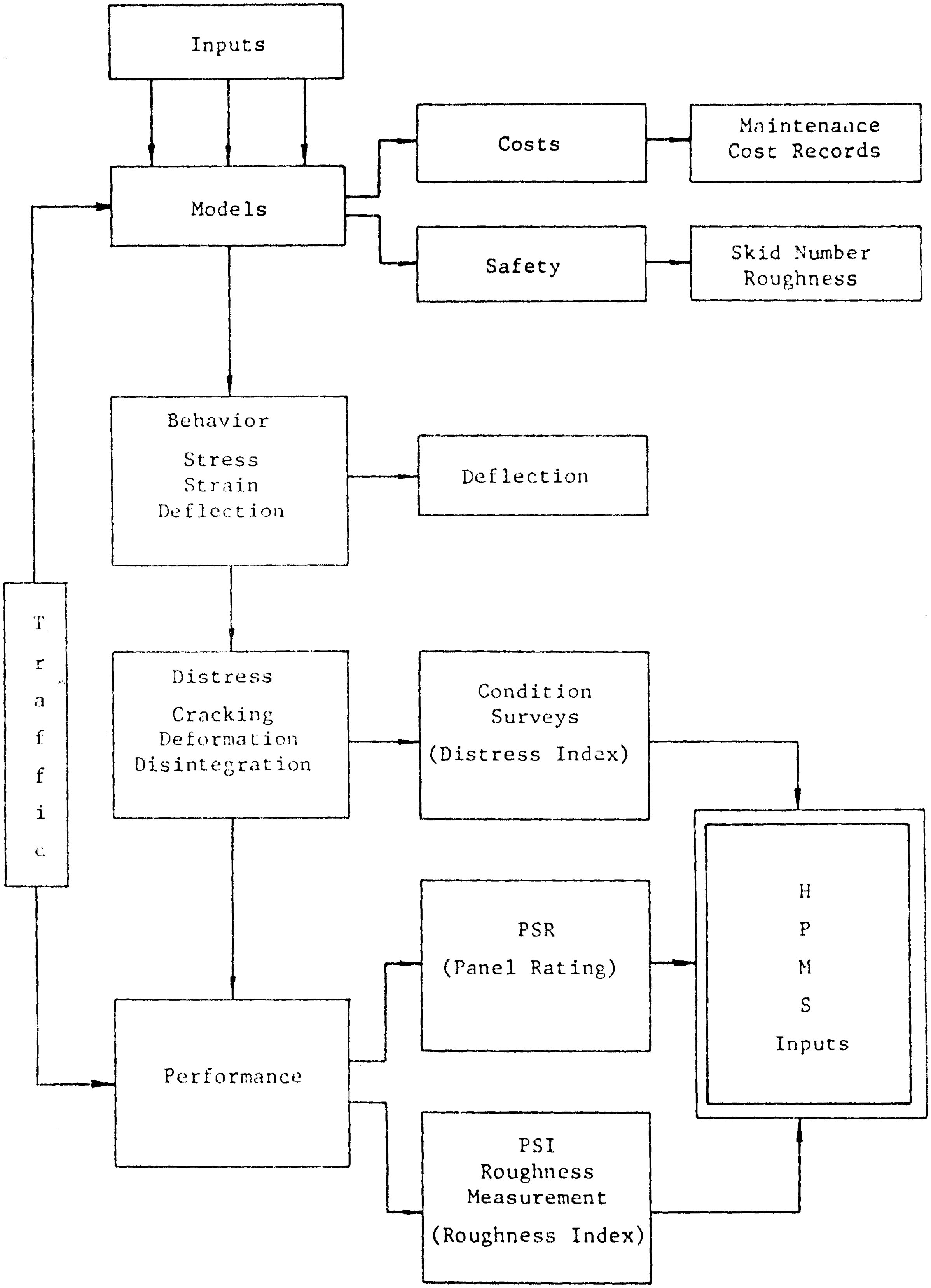


Figure 2. Flow of pavement condition information.

	USE IN PAVEMENT EVALUATION	
TYPES OF CONDITION SURVEYS	PROJECT LEVEL	NATIONWIDE NETWORK LEVEL
Pavement Distress		
Windshield Surveys		X
Microprocessor Summary		X
Σ Condition Index	X	as available
Deduct values	X	as available
PSI Present Serviceability Index		
1) Roughness correlated to Road Test PSI - large rating panel		(Use depends on quality & stability of measurements)
2) Roughness devices calibrated only		(Useful for short term comparisons within a small set of sections e.g. one district)
3) Roughness calibrated & correlated to Road Test or large rating panel		(Good for annual or long term comparisons - long term monitoring)
PSR Present Serviceability Rating		
(Accuracy increases with number of ratings (see Ref. 2, 3, 6))		
1 rater		Not useful
2 raters		Poor
.		
.		
.		
4 or 5 rater teams		Satisfactory

Figure 3. Uses of pavement conditon information.

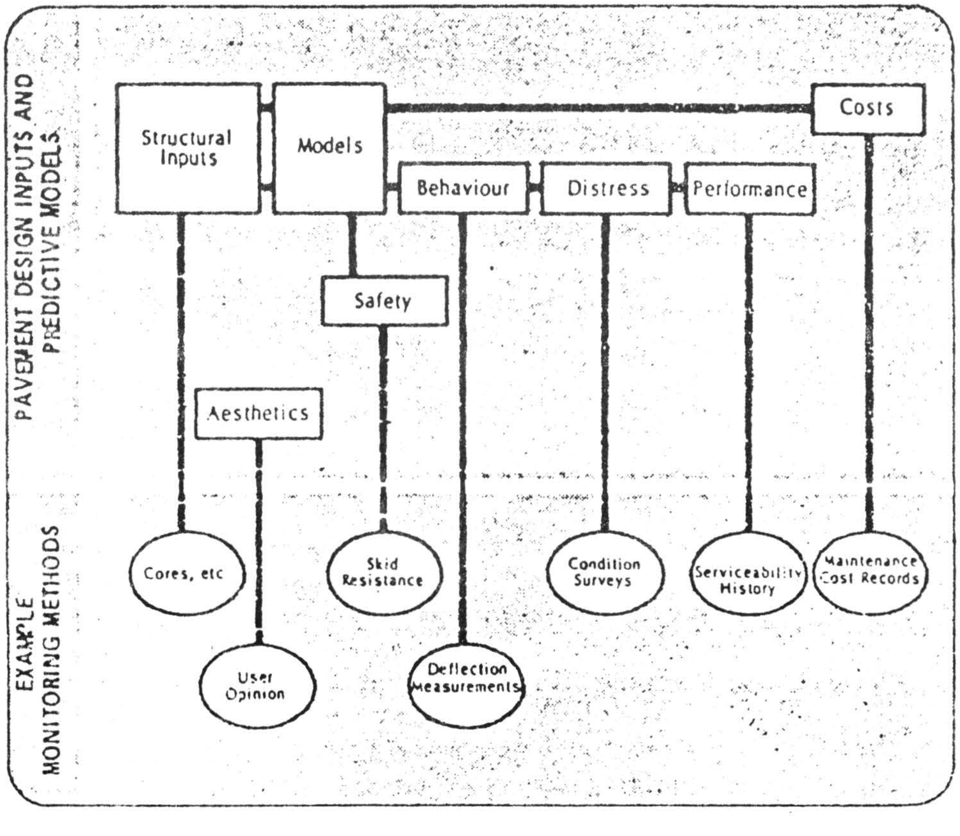


Figure 4. Simplified predictive portion of pavement design and related examples of types of periodic evaluation measurements.

What is road roughness and how can it best be defined? Some people talk about smoothness; other, serviceability. The Canadians talk of "riding comfort" and there are national committees in the United States to evaluate "riding quality." Still others talk of surface profile. In this paper, then, road roughness and smoothness will be defined as opposite ends of the same scale. A general definition of roughness must include "those surface characteristics of a pavement which affect vehicle operating costs and the riding quality of the pavement as perceived by the highway user."

Roughness is important in terms of evaluating road surfaces and their performance. It is also important in terms of evaluating vehicle operating costs as outlined above. The accuracy in measurement required for these various purposes may vary, as it may also vary between very rough roads, such as gravel and earth roads, and relatively smooth or paved roads. In the face of these diverse needs, it is important that a compatible roughness scale be made available for worldwide use.

Road Roughness

Road serviceability or riding quality is largely a function of road roughness. Studies made at the AASHO Road Test (Ref 5) have shown that about 95 percent of the road user's perception of the serviceability of a road is contributed by the roughness of its surface profile. That is to say, the correlation coefficients in the present serviceability or PSI equation studies improved only about 5 percent when other factors were added (Ref 5) to the index. Francis Hveen discusses this problem in several papers (Ref 14). He states that "there is no doubt that mankind has long thought of road smoothness or roughness as being synonymous with pleasant or unpleasant." New economic engineering research has shown that the effect of roughness on transportation costs may be more important than the effect on riding comfort. This aspect is of overwhelming importance in low-income, developing countries. Road surface roughness is not easily described or defined, and the effects of a given degree of roughness vary considerably with the speed and characteristics of the vehicle using the road.

The Need for Compatibility

Diverse measurements of roughness are used around the world. It is not feasible to compare equality among these measurements since there is no roughness measuring system capable of giving equal results for all conditions.

Rather, it is essential to insure that we have compatible measurements. Given proper consideration, a compatibility among the various measuring systems can be provided. This compatibility involves two levels of concern:

(1) "External" Compatibility -- relating to whether the results of one agency's or country's work have a quantitative relationship or meaning with those of another agency, and

(2) "Internal" Compatibility -- relating to correlating results, achieving repeatability, etc., within an agency.

This second aspect of compatibility means that all measurements with a particular agency must be compatible, not only on a given day, but from day to day and over a period time. In other words, you must be able to take a reading on pavement section number 237 on October 3, 1983 and be able to compare that same measurement with a measurement made on Highway 339 on December 3, 1985.

The problems of external compatibility relate to comparing results between agencies. For example, if we are to take the results of pavement performance measurements from several State Highway Departments and put them together to have a broader data base, those measurements must be compatible on a quantitative basis.

Roughness Defined

Road roughness is a phenomenon which results from the interaction of road surface profiles and any vehicle traveling over that surface and is experienced by the vehicle, its operator, and any passengers or cargo. This roughness is a function of the road surface profile and certain parameters of the vehicle, including tires, suspension, body mounts, seats, etc., as well as of the sensibilities of the passengers and driver to acceleration and speed.

Hudson and Haas (Ref 8) refer to "pavement roughness" as the "distortion of ride quality". This definition is intended to refer to the road surface, whether paved or unpaved. Safety considerations also influence the acceptance of roughness, and the important economic aspects of roughness on vehicle operating costs should also be recognized. For purposes of this paper, then, the following definition or road roughness is suggested:

> "the distortion of the road surface which contributes to an undesirable, unsafe, uneconomical, or uncomfortable ride". A similar but slightly different definition is "distortion of the road surface which imparts undesirable vertical accelerations and forces to the vehicle or to its riders, thus contributing to an undesirable, uneconomical, unsafe, or uncomfortable ride".

To define completely a road roughness function, some evaluation of the roughness of the entire surface area of the road should be made. However, for practical purposes this roughness can be divided into three components: (1) transverse variations, (2) longitudinal variations, and (3) horizontal variations of road alignment. In other words, any functional roadway parameter which imparts acceleration to the vehicle or to the riders should be examined. Of most interest, however, are those functions which influence the deterioration of the vehicle and/or the comfort and safety of the rider.

Previous studies have shown that longitudinal roughness is the major contributing factor to undesirable vehicle forces (Ref 29). The next greatest offender is transverse roughness (e.g., the roll component transmitted to the vehicle). The horizontal curvature of the roadway, which imparts yaw forces to the vehicle, is considered to be the least offensive and is normally handled by following good highway alignment practices. Since most vehicles (approximately 70 percent) travel in a well-defined wheel path, with their right wheels located about one meter (3.28 feet) from the outside lane line, we conclude that measurements of longitudinal profile in the two respective wheel paths, which are 1.83 meters (6 feet) apart, might provide the best sampling of roadway surface roughness. Furthermore, comparison between the two wheel paths can provide some measurement of the cross slope or transverse variations, which are also important.

A rider in a vehicle passing over a road surface experiences a ride sensation. This ride sensation is a function of (1) the road profile, (2) the vehicle parameters, and (3) the vehicle speed. A variation of any one of these three variables can make a rough road profile appear smooth, or rough. Therefore, we might say that from a passenger's viewpoint roughness is an undesirable combination of road profile, vehicle parameters, and speed. Riding characteristics of airplanes are also affected by the properties of airfield surfaces and of the aircraft. Vertical accelerations of sufficient magnitude to critically affect safety or aircraft operations are sometimes obtained over poor surfaces.

Most drivers have experienced the sensation of improving their ride on a particular road by either slowing down or speeding up. This indicates that the road surface profile contains roughness waves or undulations of a length which, when driven over at a particular speed, produce an excitation in the vehicle at one of the vehicles's resonant frequencies. Since a normal vehicle is a simple mechanical vibrating system made up of the mass of the vehicle, the springs on which it rides, and the shock absorbers, there is a particular frequency of vibration or bouncing of any vehicle at which the vibrations tend to increase in amplitude. This is normally called the resonant frequency. The typical passenger car has resonant frequencies of between one and ten cycles per second (Fig 5). This relationship indicates that at any particular speed of travel there is a road profile wavelength that will excite the vehicle at one of its resonant frequencies and thus cause excessive vibration or bouncing. If the amplitude of that resonant wavelength is large, the vibration or vertical accelerations imparted to the vehicle may be quite noticeable. Since vertical accelerations import significant vertical force, these wavelengths result in significant forces applied to the vehicle, which can result in damage to vehicle components and increase operating costs, as well as in an unsafe and uncomfortable ride.

In general, most vehicles in a particular class, i.e., passenger cars as one class and trucks as another class, possess similar characteristics and for any particular road surface, most vehicles in the same class will be driven at about the same speed. With two of these variables held relatively fixed, the excitation in the vehicle and thus the riding quality and vertical

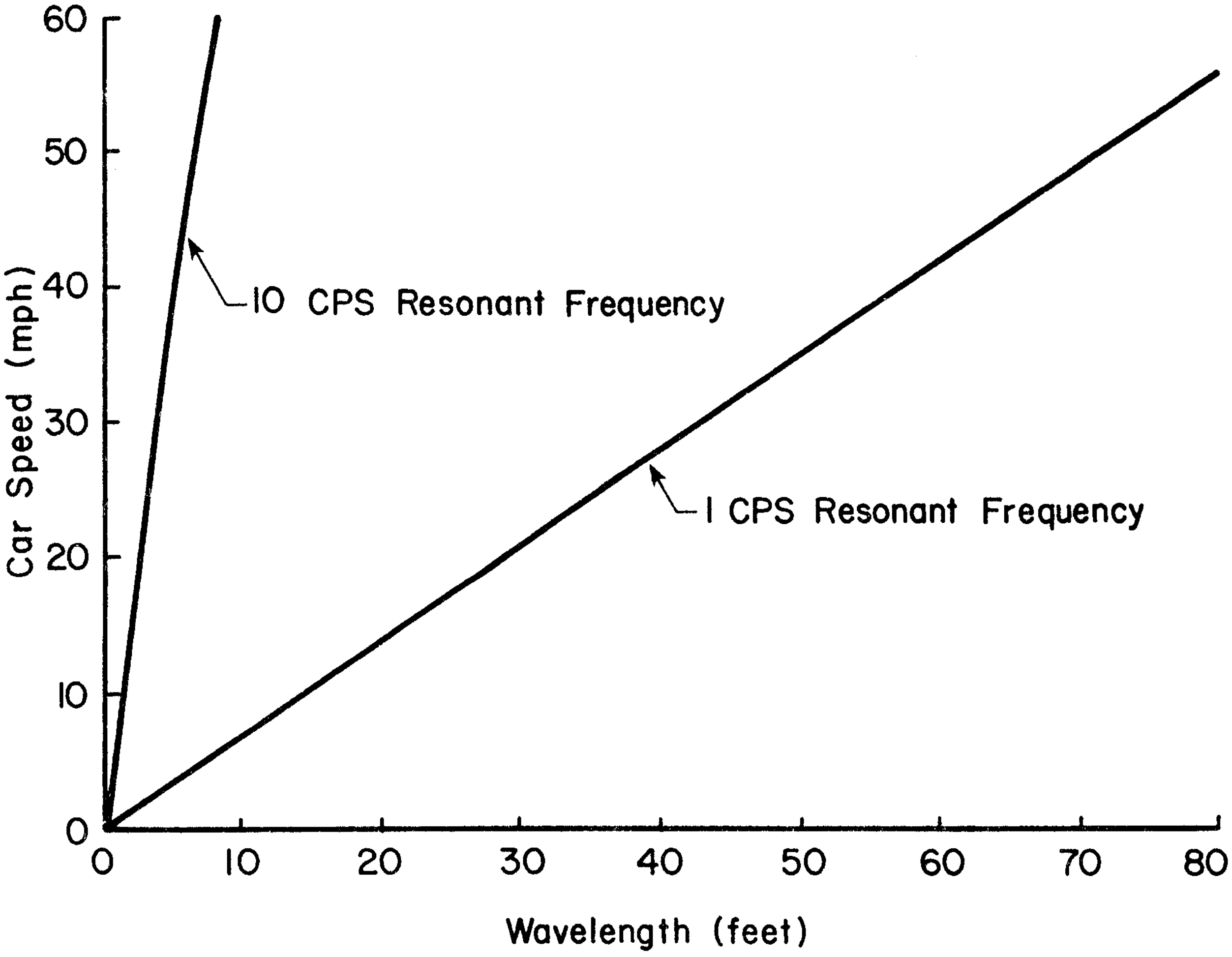


Fig 5. Relationship between resonant frequencies of cars, car speed and pavement surface wavelength.

forces on the vehicle become primarily a function of the wavelength content of the road profile surface.

Road Roughness Evaluation

Roughness evaluation has received considerable attention from many highway and airport agencies in North America in the last three decades. Roughness is the primary component of pavement serviceability and a large number of different roughness measures are in current use to evaluate such serviceability. Some of the more widely used methods of measuring roughness, correlating measurements, and applying the results are outlined in Ref 37. Many of these measurements involve perception by the highway user as a very important factor and thus roughness measurements have generally excluded surface texture and microtexture of surface aggregates since these are not perceived by the user to affect riding comfort.

Road Profile

Many authors, such as Darlington (Ref 6) and Carey (Ref 3), feel that pavement profile does the best job of characterizing roughness. In terms of pavement profile, roughness can be defined as "the summation of variations in the surface profile of the pavements". Profiles in this sense do not include the overall geometry in the road but are limited to wavelengths in the surface of the pavement between approximately 0.031 meters (0.1 feet) and 152.4 meters (500 feet) in length. In Darlington's terms, roughness is "the analysis of the pavement profile or of the random signal known as profile".

Carey (Ref 3) points out four fundamental uses of pavement surface profiles or roughness measurements:

(1) to maintain construction quality control,

(2) to locate abnormal changes in the highway, such as drainage or subsurface problems, extreme construction deficiencies, etc.,

(3) to establish a system wide basis for allocation or road maintenance resources, and

(4) to identify road serviceability-performance life histories for evaluation of alternative designs.

In summary, then, a road profile is a detailed recording of surface characteristics, and roughness or smoothness is a statistic which summarizes these characteristics and provides a measure of riding quality of a road.

Once the surface characteristics of a road are summarized, it is essential to establish a scale for this statistic or summary value. This can be done in may ways, as pointed out by Darlington (Ref 6). Traditionally there are two basic ways of determining this statistic:

(1) mechanical integration and

(2) mathematical integration or analysis.

The first of these methods is the most common, that is, the use of some mechanical instrument or device such as the BPR Roughometer or TRRL Bump Integrator (Fig 6) to mechanically filter and summarize the data in a specified way. The second method involves recording the profile as faithfully as possible and then analyzing and/or integrating this profile mathematically with some standard mathematical procedure such as that outlined by Walker and Hudson (Refs 25 and 26), Roberts and Hudson (Refs 20 and 21), Quinn (Ref 19), and Darlington (Ref 6). The most common methods in current use for mechanical measurement and summary include the BPR Roughometer (Refs 11 and 12), the very similar TRRL Bump Integrator (Ref 33), the PCA Roadmeter (Refs 1 and 2), the Mays Meter (Fig 8) (Refs 26 and 27), the CHLOE Profilometer (Ref 4), and the land plane, profilograph or rolling straight edge (Fig 7) (Ref 24).

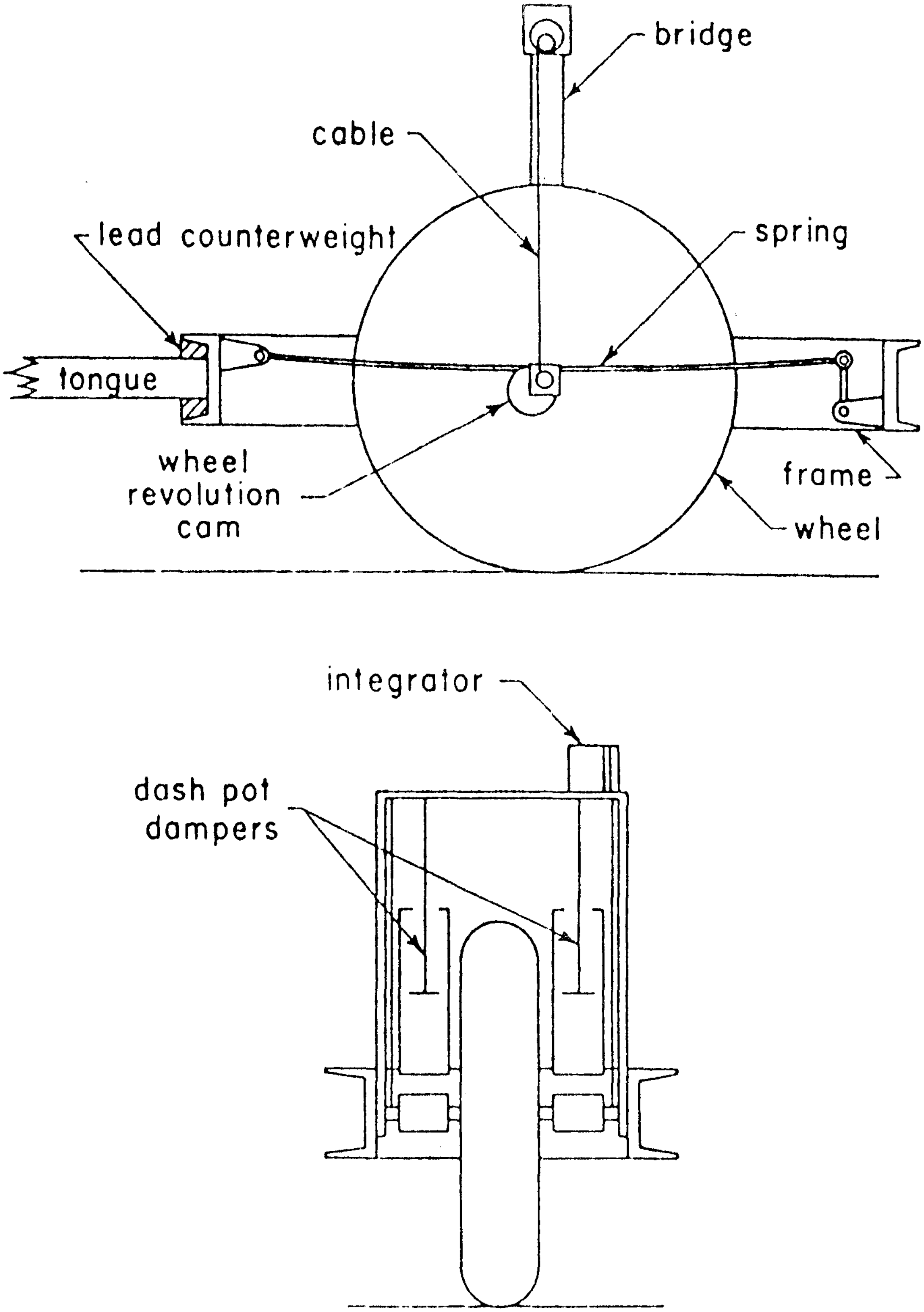


Fig 6. Schematic diagram - BPR Roughometer.

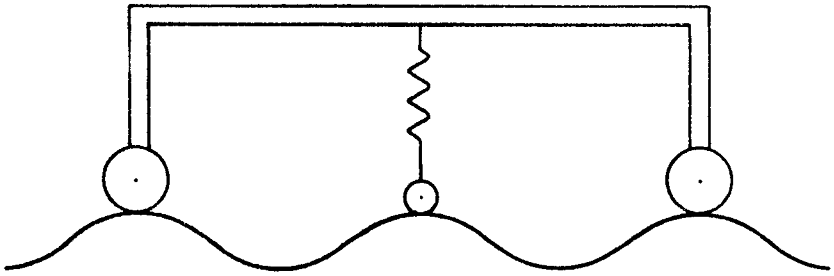


Fig 7. Land plane roughness device sometimes called profilograph or rolling straight edge.

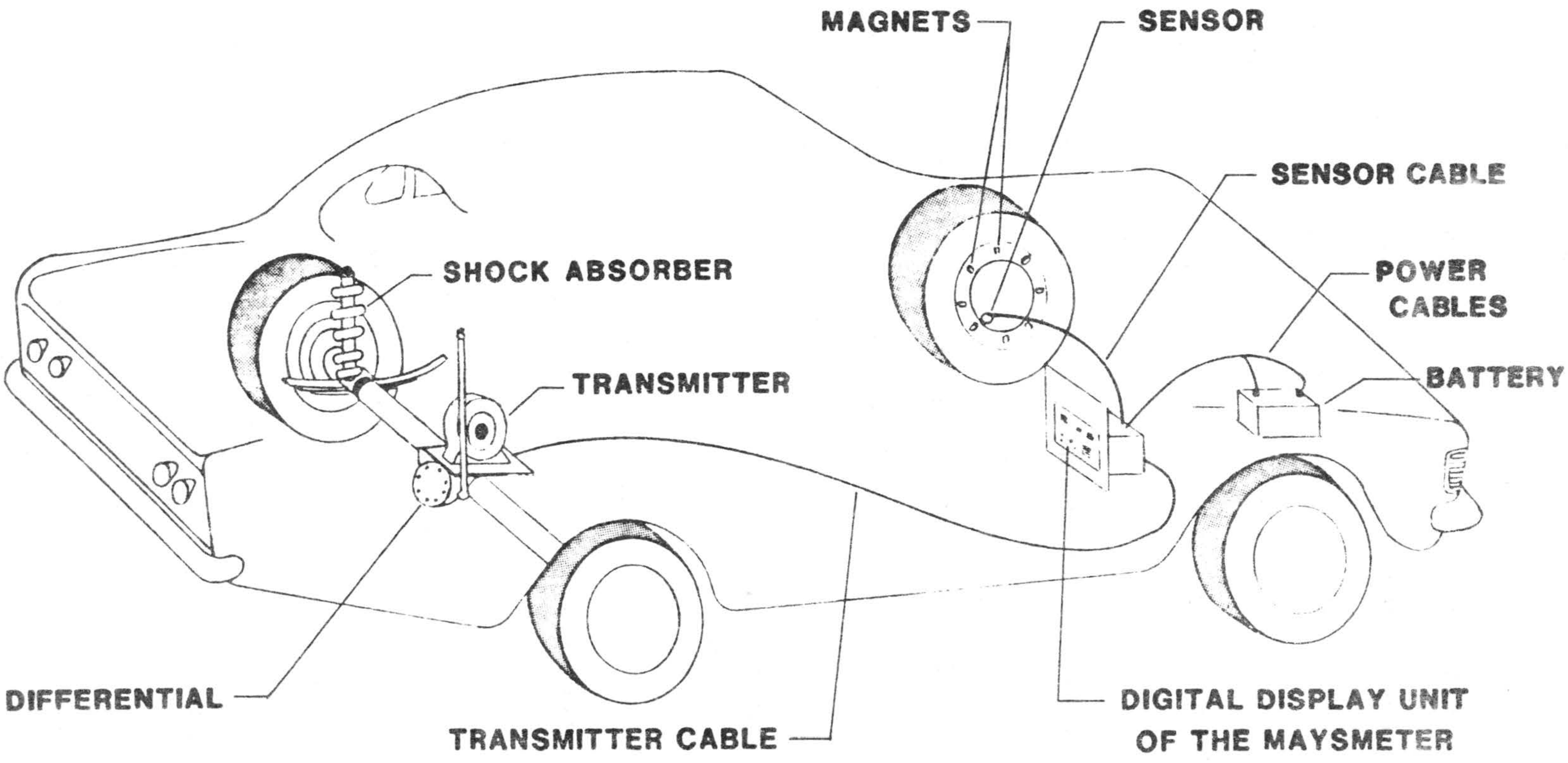


Figure 8. Mays Meter.

A number of studies have been made to compare these instruments as outlined in attached references (Refs 6 and 12).

A word of elaboration is needed on the term "mechanically filtered," mentioned above for the BPR Roughometer. Instruments such as the BRP Roughometer, the PCA Road Meter, and the Mays Meter use the vehicle itself as a mechanical filter for processing the profile and summarizing in effect the response of a particular vehicle (in its specific condition) to the road profile.

If the mechanical characteristics of the measuring vehicle could be previously set and maintained at a desired preselected level, then the resulting summary statistics could be directly related to the economics and/or safety of a specific vehicle class. Unfortunately, due to the many parameters and the great variability involved, the use of the Bump Integrator or BPR Roughometer concept rather than the profile itself introduces great measurement and analytical complications.

Since so much has been written about the various instruments available we will not attempt in this short paper to review all these measurement methods in detail.

Comparison of Measurement and Summary Techniques

Regardless of the measurement and type of summary technique used, it is essential that a good reference be established and maintained. It is equally important that accuracy in summation be maintained. Every different instrument has a different readout scale and even seemingly identical instruments must be calibrated so that the observed readout is meaningful. This readout scaling and consistency are central to this paper.

Darlington (Ref 6) points out that three basic reference methods have been used historically:

(1) a so-called rolling straight edge or land plane, as illustrated in Fig 7,

(2) an inertial mass as used in the BPR Roughometer (Fig 6), the Mays Meter, and the PCA Road Meter (in the latter two cases, the automobile forms the inertial mass), and

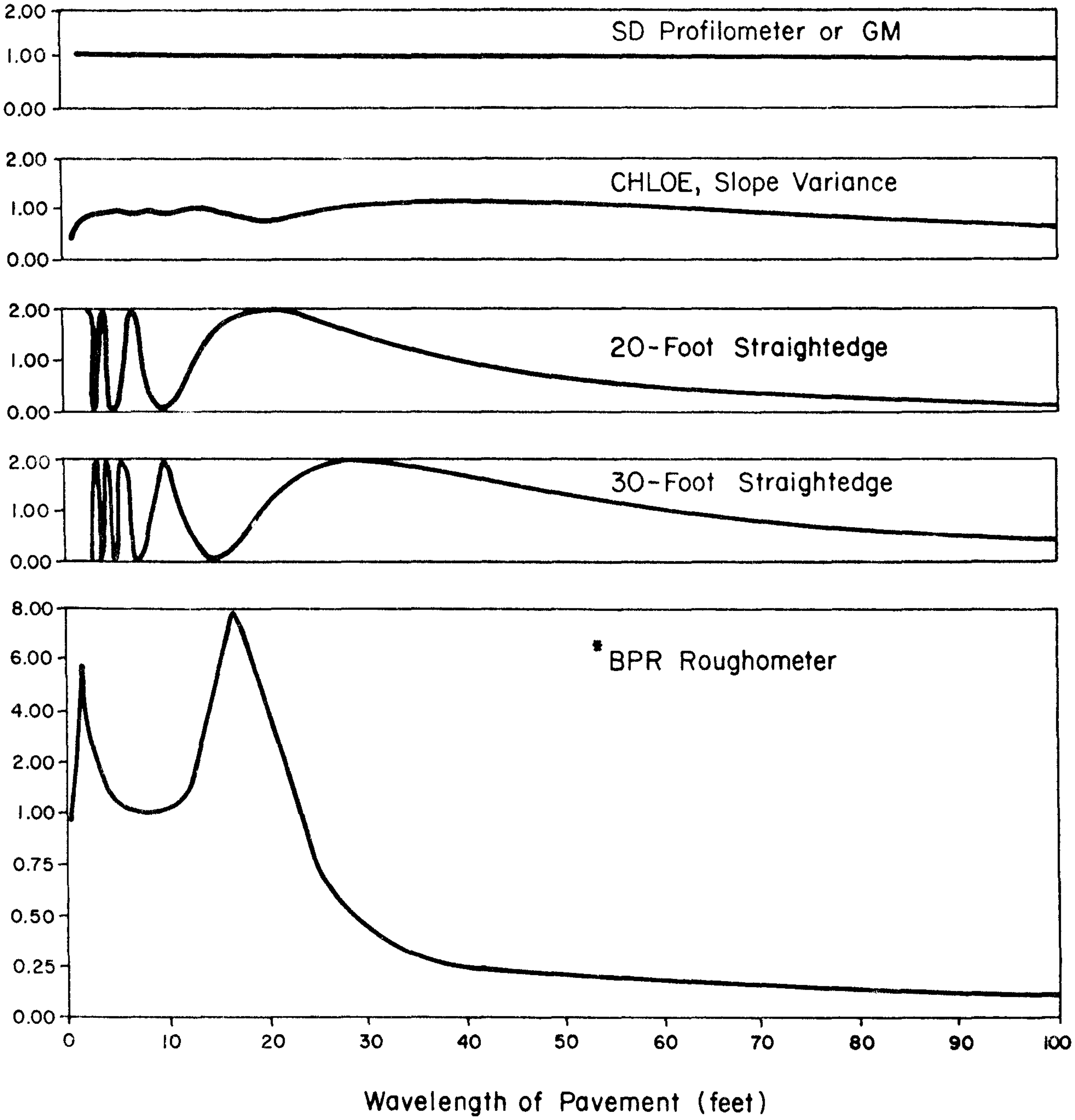
(3) an inertial reference profilometer such as the Surface Dynamics or General Motors Profilometer, where an external reference is provided.

Figure 9 illustrates by means of a Bode plot the transfer function or response of several types of profilers to the input of road roughness. The so-called "rolling straight edge" or land plane device is so erratic in its response as to be relatively useless, as shown in Fig 9, since roughness wavelengths which are any fraction of the length of the straight edge result in zero output from the device.

Darlington simulated the response of the BPR Roughometer, or seismic reference device, on an analog computer using measured physical characteristics of the instrument. His analysis shows that the roughometer type device yields reasonable results for wavelengths in the range of 1.22 to 4.26 meters (4 to 14 feet). Wavelengths in the range of 4.26 to 5.48 meters (14 to 18 feet) are badly distorted, and wavelengths beyond 6.70 meters (22 feet) rapidly attenuate to zero response.

ROUGHNESS CALIBRATION AND CORRELATION

The earliest roughness measurements were reported by Hogentogler, as far back as 1923 (Ref 37). Early development of the Roughometer was reported by Public Roads in 1926 (Ref 16). Even in these early developments the need for calibration was readily recognized. From 1941, when the BPR Roughometer became "standardized," the Bureau of Public Roads (now the Federal Highway Administration) maintained a "standard calibration section" for testing any new or modified BPR Roughometer. It was observed from the beginning that



*Note the Mays Meter and PCA meters have a response very similar to the BPR Roughometer.

Fig 9. Theoretical differences between SD profilometer, Chloe, rolling straight-edges and seismic roughometers.

instruments manufactured as nearly alike as possible did not record the same roughness value for the same pavement.

The fallacy of this calibration section is discussed by Hudson and Hain (Ref 11). It is not possible to calibrate a dynamic instrument at a single point over its range and expect the calibration to be satisfactory for use of the instrument over a full range of roughness. This is illustrated in Fig 10, where a standard roughness section with a value of 10 has been set up. We might assume that any other instrument which reads 10 would be calibrated to the standard value. In fact, this assumption is depicted by the solid "line of equality," No. 1 in the figure. This line assumes that if an instrument reads 10, it is "calibrated" and thus will read 20 when the standard instrument reads 20, 30 when the standard instrument reads 30, etc. Alternatively, line No. 2 illustrates a plausible case of a linear relationship, where instrument No. 2 is calibrated to the standard instrument on the section with value 10. Without additional test points we would not realize that the slope of the calibration line is really different from the assumed line of equality. Dotted line No. 3 illustrates a more complex case of nonlinear relationship which would, of course, also be missed with the single point calibration.

Roughometer Calibration Course - AASHO Road Test

As reported by Hudson and Hain (Ref 11) there was a need to use the Roughometer in the AASHO Road Test but it became obvious very early, with the AASHO Profilometer to compare to, that the BPR Roughometer was a variable instrument difficult to keep in calibration. In work at the AASHO Road Test we were not only involved in measuring the roughness of all pavements with AASHO Profilometer and in developing and operating the BPR Roughometer, we also checked and calibrated at least six additional roughometers from states

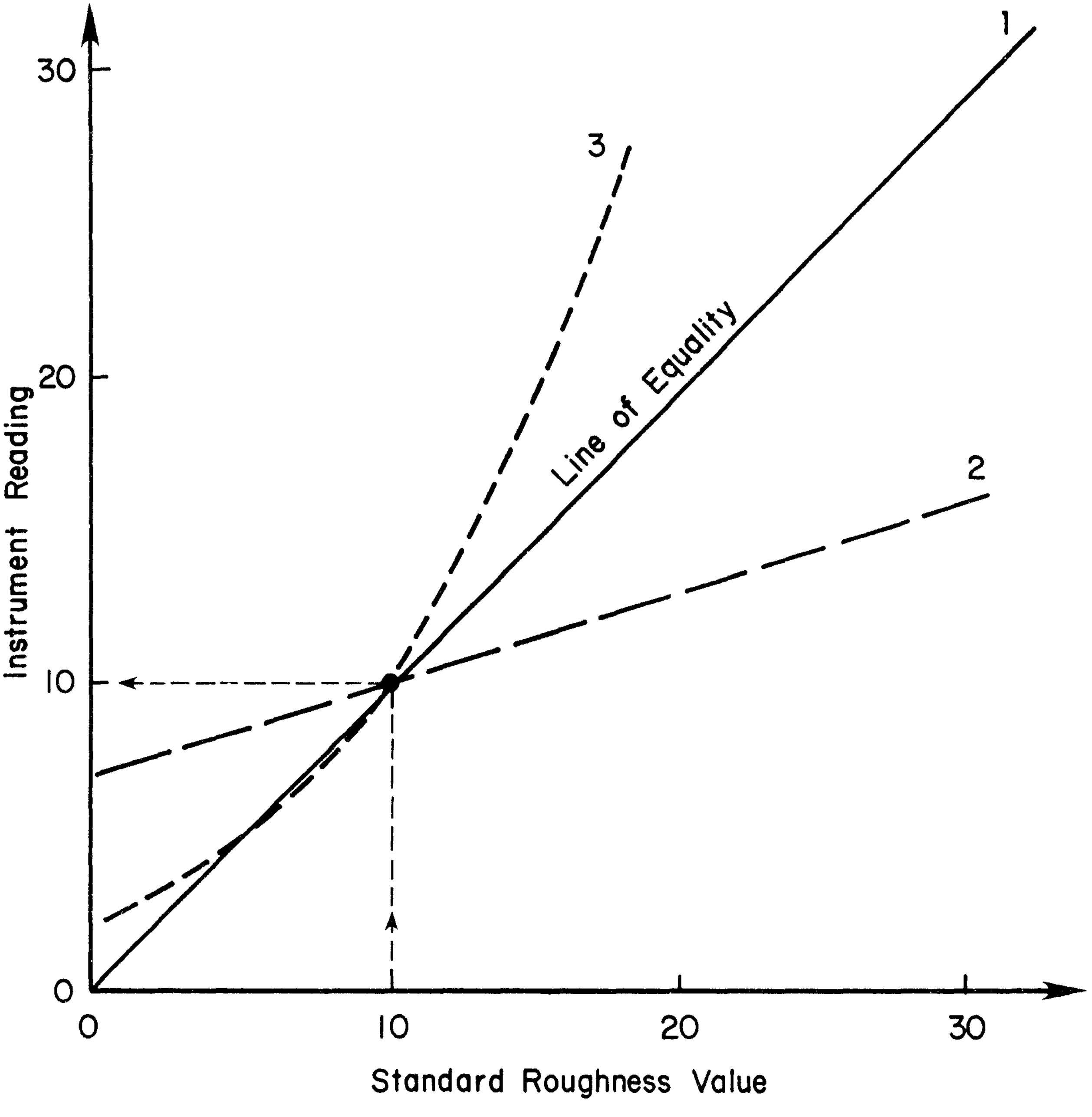


Fig 10. Single point BPR calibration problems.

such as Michigan, North Dakota, Minnesota, and Wisconsin which brought their instruments to the Road Test for calibration against the AASHO Profilometer for determining serviceability.

Basically, the method involved the installation of aluminum bars on the surface of a smooth rigid pavement to establish four separate test sections of different but known roughness. The roughometer could then be checked against the standard sections at any required time.

Use of a "Standard" Device for Calibration

Probably the most widely used method of calibration and correlation has involved some type of so-called "standard device." Really, this approach should be divided into two parts. The first involves the selection of one replicate from a group of similar devices being used and using this copy of the device for calibration purposes, so that it presumably does not "wear out." I liken this approach to gold plating a crowbar. If you have two dozen crowbars and select the one that appears to be more perfect in shape and weight than the others and plate it with gold as a reference, what do you have? Still a crowbar, albeit a shiney and expensive one.

The only validity of this approach is lack of wear of the "standard" in routine use. However, many of the errors we must deal with do not result from wear alone. There is little evidence that this type of "standard device" has been successful in use for calibration and correlation.

The second part involves the use of a master device which is itself calibratible or which has a standard of accuracy which is perhaps a magnitude greater than the other devices for which it is to be the master control. The AASHO Road Test Profilometer was such a device; it became a standard against which dozens of CHLOE Profilometers and BPR Roughometers were calibrated during and soon after the AASHO Road Test. This approach is discussed below as the Texas Calibration Course.

Texas Calibration Course Using Surface Dynamics Profilometer

The Center for Highway Research and the Texas Department of Highway and Public Transportation use the Surface Dynamics Profilometer (SDP) or General Motors Profilometer as a master calibration device for a series of Mays Meters which are used routinely throughout the state. This approach is reported by Walker, Hudson, and Williamson (Refs 26, 27, and 28). To some degree, a similar approach has been taken by the Michigan Highway Department, as reported by Holbrook and Darlington (Refs 9 and 10). A similar approach is also being taken at the present time in the UNDP Brazil Study (Ref 15). A Surface Dynamics Profilometer was purchased and is used for measuring a set of calibration sections. These sections are run regularly by several Mays Meters to insure that their calibrations remain stable. A control chart procedure and regular check procedure similar to that outlined by Williamson are followed.

Basically, Texas maintains a group of 25 pavement sections which together exhibit a range of roughness. Every three months the profile of each of these sections is measured and analyzed with the SDP Profilometer. In this way a set of pavements with known roughness is always available for use in checking and calibrating any other roughness instrument. Any instrument which appears to be giving erroneous readings is regularly run on several check sections and the values are plotted on a typical control chart. If a device is "out-of-control" on three or four sections it is thoroughly checked mechanically and recalibrated.

Rod and Level Surveys

Many people feel that it is possible to establish vehicle roughness calibrations over standard pavement sections by running control rod and level surveys of the calibration sections to see if and how their profiles are changing. There are two basic problems associated with this methodology. First, the response of the vehicle and most roughness measuring instruments to a profile is an integration of everything the measuring instrument sees on the road surface. This is a continuous process and not one involving discrete points such as are used in a rod and level survey. This problem is magnified by the fact that even the best manual leveling techniques make it expensive to make measurements of test sections 300 meters (985 feet) long at spacings closer than about one-half meter (1.6 feet). Even in this case a total of 600 measuring points is required each time a calibration section is checked.

Certainly these discrete rod and level surveys have some practical advantages, particularly in developing countries where labor-intensive methods are economical. It might be far more practical to obtain detailed, discrete profiles with rod and levels of say, 10 or 12 pavement test sections on a regular basis than to maintain a high-technology, expensive electronic device for continuous profile measurements. Such a method is be practical with data analysis techniques developed and automated for easy use by Hudson et al (Ref 38).

Rating Panel Approach - Canadian Good Roads Association

Immediately following the AASHO Road Test, the Canadian Good Roads Association desired to put the findings of the AASHO Road Test into practice. In order to summarize the Mays Meter and rod and level correlation study we developed a profile statistic based on Root Mean Square Vertical Acceleration

(RMSVA) at base lengths 4 ft and 16 ft. It was successful in explaining approximately 97 percent of the response variation between 5 trailer-mounted Mays Meters on 29 pavement test sections. This corresponds to a prediction standard error of about 10 percent of the Mays Meter reading (inches/mile), which compares favorably to what would be achieved if an actual Mays Meter has been singled out as the reference device.

The Mays Meter simulation has proved to be an effective standard (Ref 39) for Mays Meter calibration; however, the individual RMSVA indices (base lengths 1, 2, 4, 8, 16, 32, 65, and 130 feet) are genuine roughness traits which are useful for defining problems in simple measurements. Therefore, to make such comparisons easier, rescaled versions of these indices are usually provided which resemble a sequence of serviceability or "SI_b values" in the range 0 to 5. This is accomplished simply by replacing term MO (the standard Mays Meter value) with a least-squares fitting of vertical acceleration against SI or serviceability index (SIV).

The main advantage of such scalings is that their means, as determined on 31 test sections near Austin, are approximately the same, making it easier to judge their significance on other pavements. The test sections encompass a variety of roughness conditions, exhibiting a SIV range of .63 to 4.83, with mean 3.12 and standard deviation of 1.23.

The RMSVA (vertical acceleration) summary data can be used for comparing pavements and for detecting changes in different components of roughness.

To illustrate RMSVA data for two sections known to be subject to deterioration from expansive clays are shown in Fig 11 (SI_b versus baselength), along with the corresponding values obtained periodically during the previous 1-1/2 years (dashed lines). Notice that the spectra of SI_b values are distinctive traits which, in this case, changed very little during

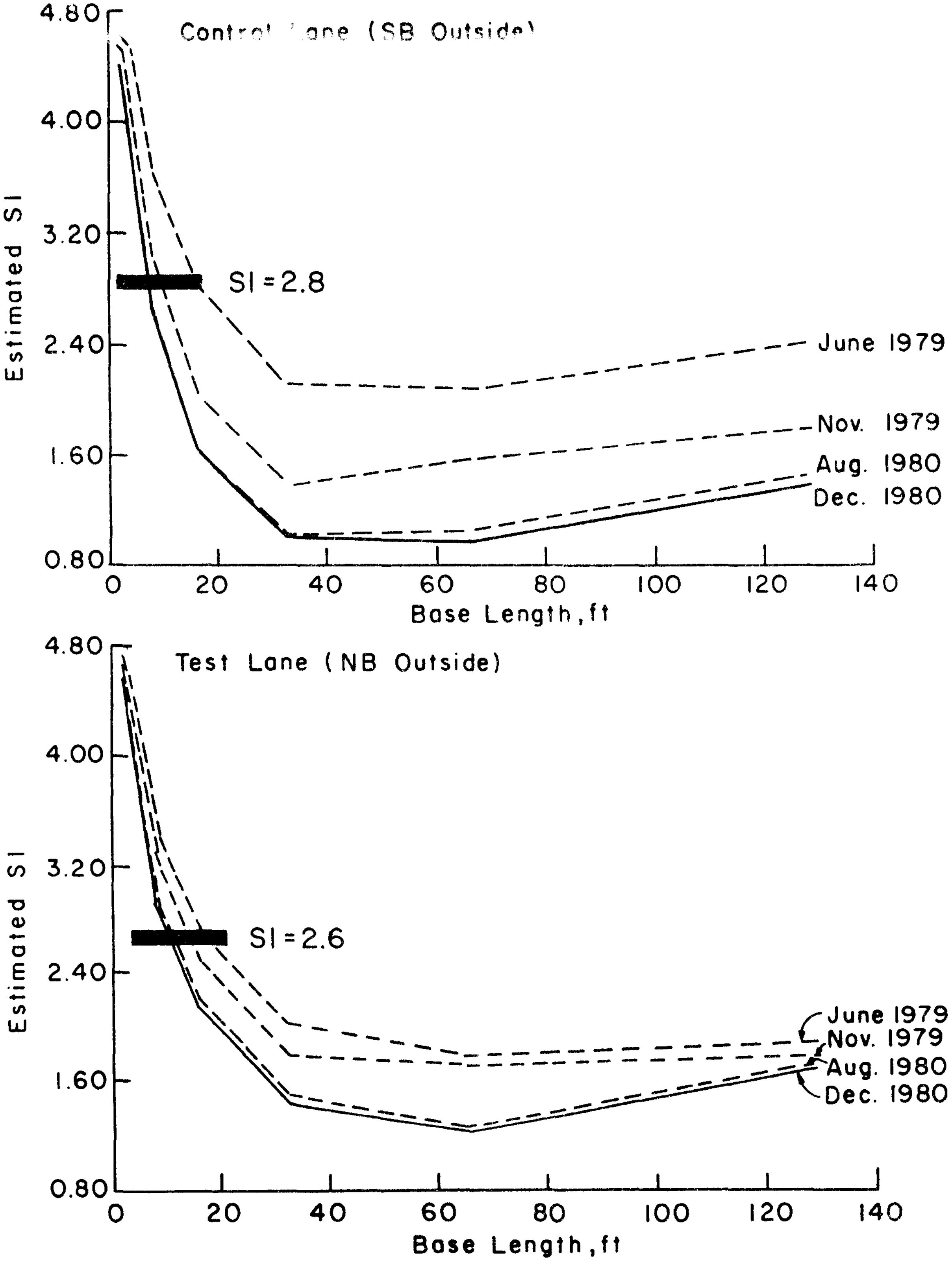


Fig 11. RMSVA signatures for untreated (top) and treated (bottom) ACP sections in a swelling clay environment -- Loop 410, San Antonio, Texas.

the observed 4-month time period. The test section shows the effect of treatment by a fabric moisture seal sometime prior to the first profilometer run in June 1979. The differences, however, are confined to the longer RMSVA baselengths and were not noticed in readings from a Mays Meter.

A more typical situation is that of Austin test section No. 23, where RMSVA "signatures" taken one year apart (Fig 12) show the effect on short wavelength roughness of an intervening overlay. In this case, Mays Meters did detect a distinct increase in serviceability (SIV), from 2.6 to 4.0.

These data illustrate why "simple" devices like PCA meters and Mays Meters do not always give results which agree with the engineers judgement. They are not capable of summarizing a full spectrum of information.

It is important not to confuse the problem of calibrating a group of instruments with the problem of interpreting their measurements. When the Texas Mays Meter calibration method was first devised, the Serviceability Index (SI) was the best available estimate of Present Serviceability Rating (PSR), a measure of roughness which is meaningful. Since serviceability estimates were from the Mays Meters, SI was chosen as the standard against which different units were to be calibrated. This was a good approach, however, only if Mays Meters were capable of measuring SI with as much accuracy as their evident precision would indicate. Unfortunately, this is not true. At best, Mays Meters can be assigned scalings so that different units give comparable "Mays Meter roughness" ratings. How the ratings should be used to predict other things, such as ride quality, is a problem to be considered apart from the calibration process itself.

To help clarify this point with an analogy, suppose that the readings of several homemade thermometers, when inserted in lakes of a given region, correlated fairly well with the number of fish caught during that day. It

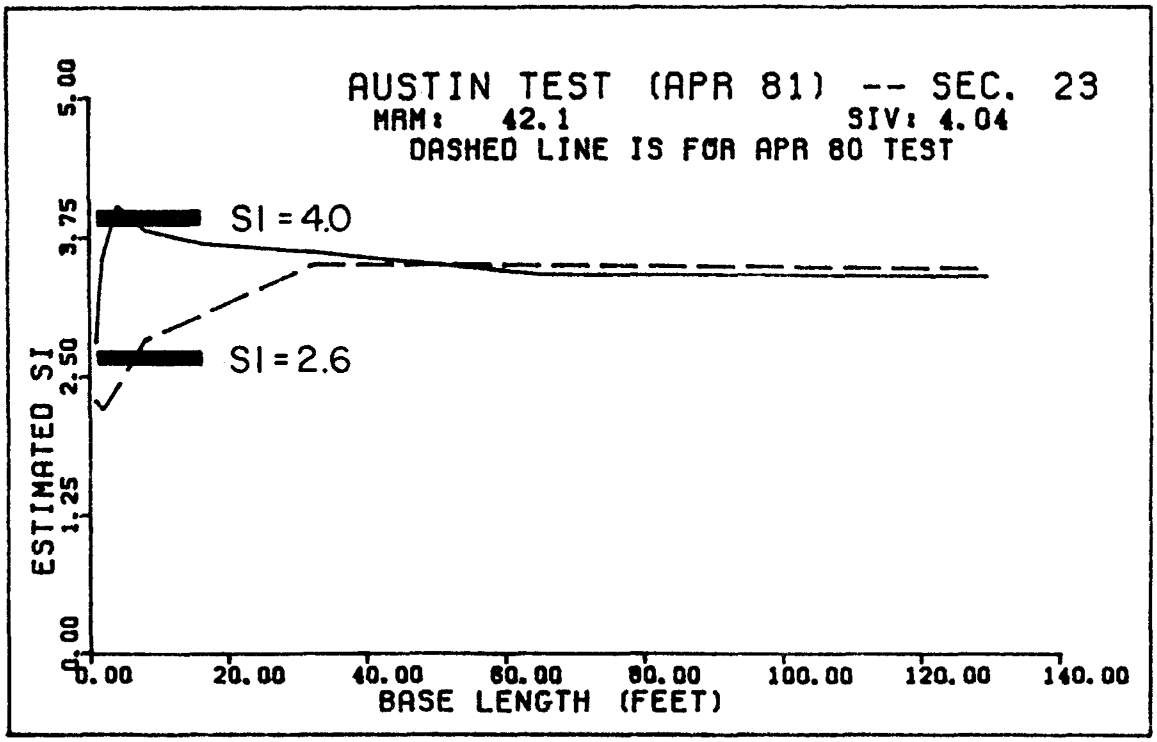


Fig 12. RMSVA signatures one year apart showing effect on short-wavelength roughness of intervening overlay.

would be desirable to know that one lake is a better fishing prospect than another, even though they were measured with different thermometers. Obviously, the best approach to "calibration" is not to compare performances on representative lakes, but to use a "standard" thermometer to correlate each homemade device with temperature, i.e., with something it is capable of measuring precisely. Then, with the benefit of results from all of the calibrated instruments, one could seek a useful relationship between temperature and number of fish caught.

The analogy between Mays Meters and thermometers is not perfect, for it is not obvious what the equivalent of temperature, in pavements, should be. Our study of the Texas Mays Meters, suggests, however, that a simple profile statistic based on RMSVA can serve effectively as a calibration standard. When the statistic is rescaled by regression techniques to approximate a serviceability rating, we find that different Mays Meters that are calibrated against it can measure roads and agree to within one or two tenths of a serviceability unit. This precision, of course, says nothing about the accuracy of such measurements as predictors of subjective serviceability ratings as the Mays Meter is necessarily limited in its response, like the thermometer in the above analogy. However, quite apart from providing imperfect estimates of serviceability, it is evident that the Mays Meter is capable of measuring a certain kind of roughness with good precision. The obvious benefit of this is in making comparisons -- for example, revealing differences in separate pavements and showing trends in deterioration or the effects of rehabilitation on roughness. It is for this purpose, especially, that a good calibration method based on a stable and valid reference is necessary.

SUMMARY

In summary roughness is one of the three important ways of evaluating pavements. It is not intended to replace condition surveys or deflection testing, which are also very important in evaluating other aspects of pavements such as behavior and distress. Roughness is used to evaluate serviceability and when accumulated over time performance.

To be most useful roughness measurements must be stable over time, calibrated to some type of standard measuring scale and also correlated to a scale which can be used directly or related with measurements taken by other agencies. This paper has outlined the general aspects of roughness measurements. The references cited covered much more detail for those interested in applying the results.

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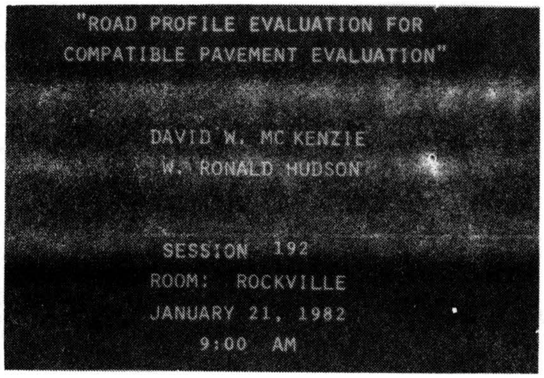
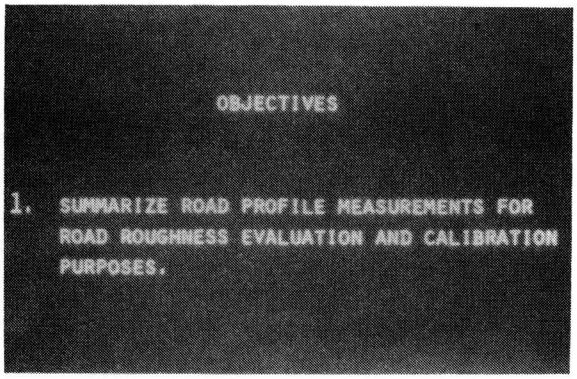


"ROAD PROFILE EVALUATION FOR
COMPATIBLE PAVEMENT EVALUATION"

DAVID W. MC KENZIE
W. RONALD HUDSON

SESSION 192
ROOM: ROCKVILLE
JANUARY 21, 1982
9:00 AM

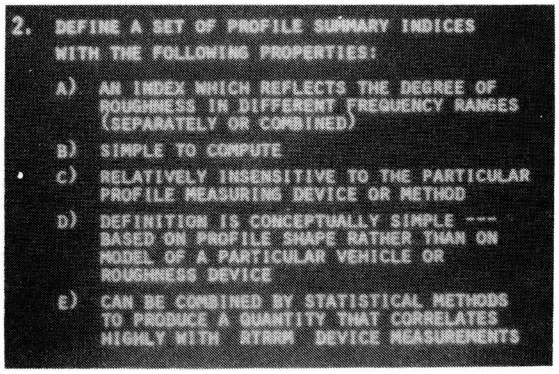
Slide 27.1. Title Slide - Introduction



OBJECTIVES

1. SUMMARIZE ROAD PROFILE MEASUREMENTS FOR ROAD ROUGHNESS EVALUATION AND CALIBRATION PURPOSES.

Slide 27.2. Objectives of roughness evaluation.



2. DEFINE A SET OF PROFILE SUMMARY INDICES WITH THE FOLLOWING PROPERTIES:

A) AN INDEX WHICH REFLECTS THE DEGREE OF ROUGHNESS IN DIFFERENT FREQUENCY RANGES (SEPARATELY OR COMBINED)

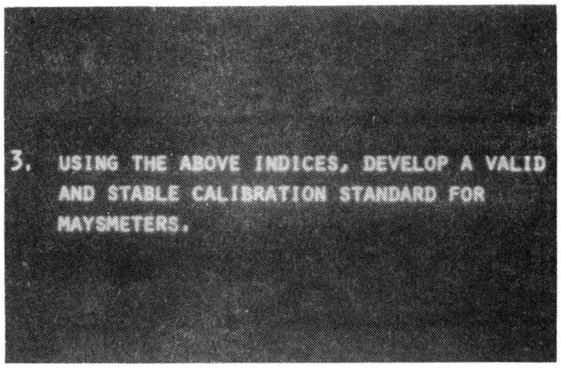
B) SIMPLE TO COMPUTE

C) RELATIVELY INSENSITIVE TO THE PARTICULAR PROFILE MEASURING DEVICE OR METHOD

D) DEFINITION IS CONCEPTUALLY SIMPLE --- BASED ON PROFILE SHAPE RATHER THAN ON MODEL OF A PARTICULAR VEHICLE OR ROUGHNESS DEVICE

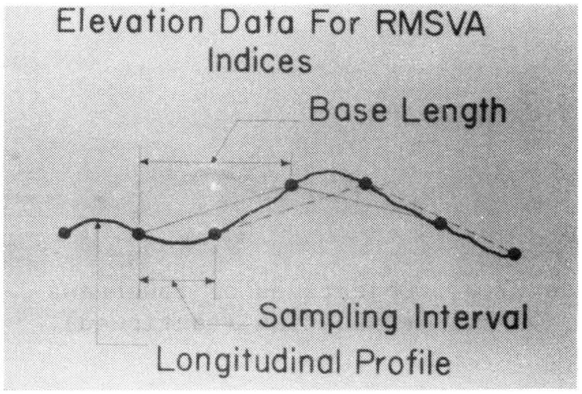
E) CAN BE COMBINED BY STATISTICAL METHODS TO PRODUCE A QUANTITY THAT CORRELATES HIGHLY WITH RTRRM DEVICE MEASUREMENTS

Slide 27.3. Objectives of roughness evaluation (continued).



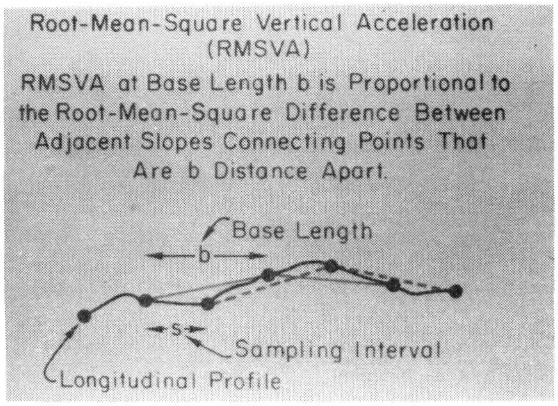
3. USING THE ABOVE INDICES, DEVELOP A VALID AND STABLE CALIBRATION STANDARD FOR MAYSMETERS.

Slide 27.4. Objectives of roughness evaluations (contined).



Elevation Data For RMSVA Indices

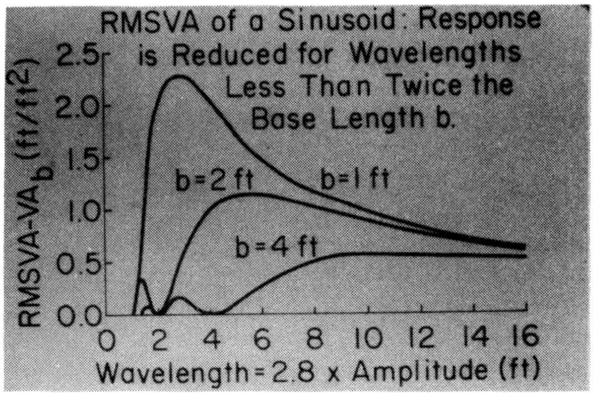
Slide 27.5. Base length and sampling interval.



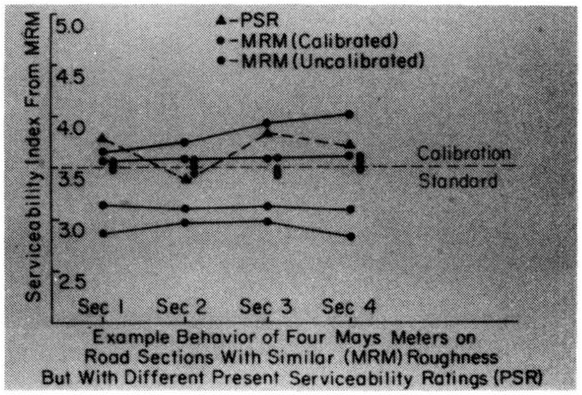
Root-Mean-Square Vertical Acceleration (RMSVA)

RMSVA at Base Length b is Proportional to the Root-Mean-Square Difference Between Adjacent Slopes Connecting Points That Are b Distance Apart.

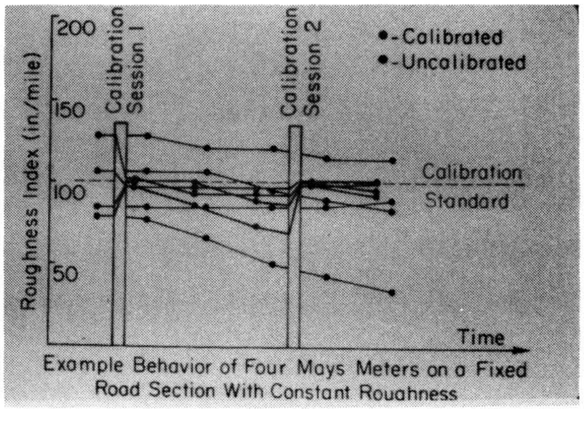
Slide 27.6. Definition and illustration of RMSVA.



Slide 27.7. Response of RMSVA to a regular waveform at different base lengths.

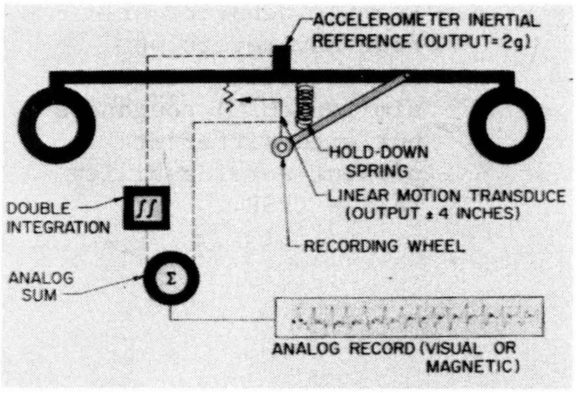


Slide 27.8. Example behavior of four Maysmeters on road sections with similar (MRM) roughness but with different present serviceability rating (PSR).

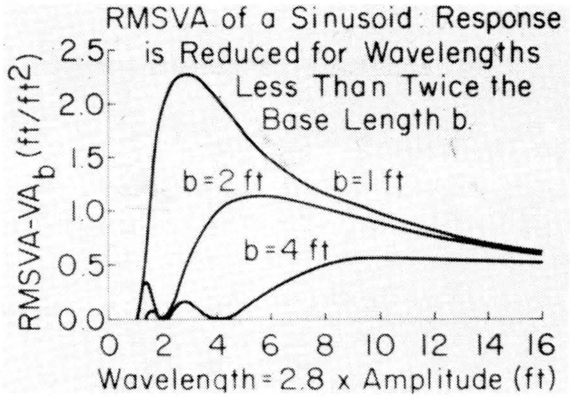


Slide 27.9. Example behavior of four Maysmeters on a fixed road section with constant roughness.

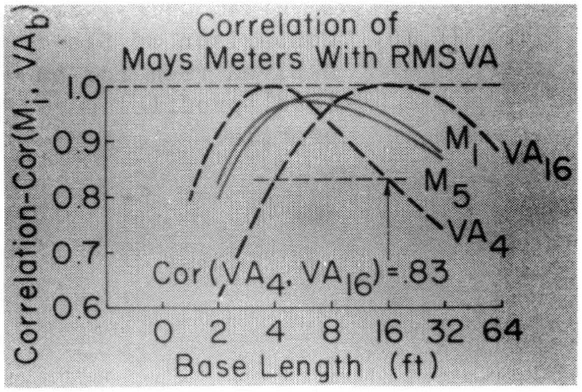
Slide 27.10. Profilometer.



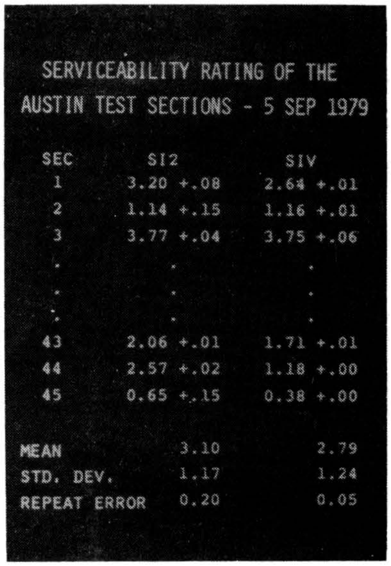
Slide 27.11. Profilometer schematics.



Slide 27.12. RMSVA defined.



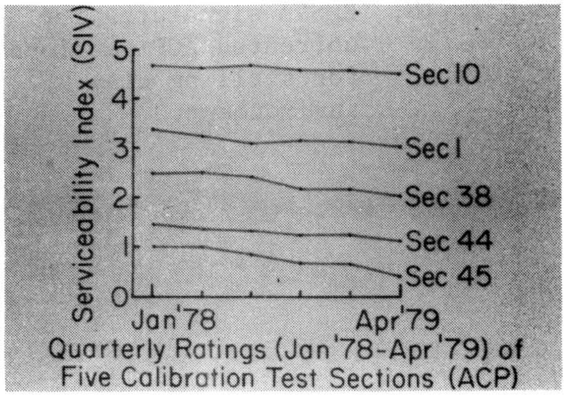
Slide 27.13. Correlations of Texas Maysmeter measurements with RMSVA.



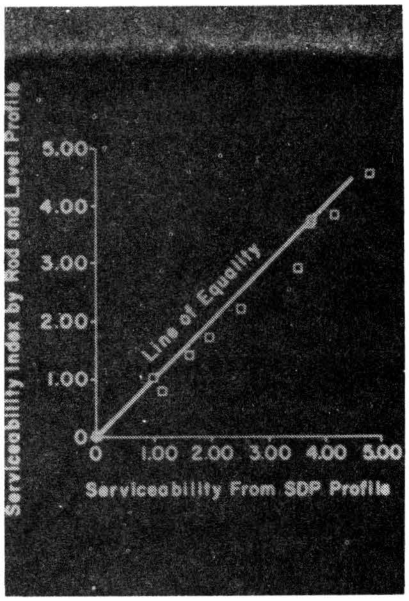
SERVICEABILITY RATING OF THE
AUSTIN TEST SECTIONS - 5 SEP 1979

SEC	SI2	SIV
1	3.20 +.08	2.64 +.01
2	1.14 +.15	1.16 +.01
3	3.77 +.04	3.75 +.06
.	.	.
.	.	.
.	.	.
43	2.06 +.01	1.71 +.01
44	2.57 +.02	1.18 +.00
45	0.65 +.15	0.38 +.00
MEAN	3.10	2.79
STD. DEV.	1.17	1.24
REPEAT ERROR	0.20	0.05

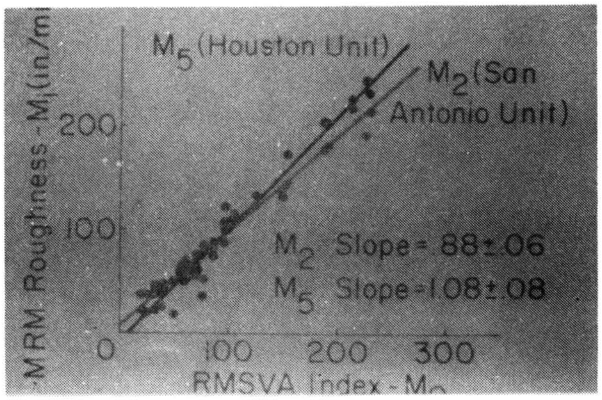
Slide 27 .14. Maysmeter - calibration.



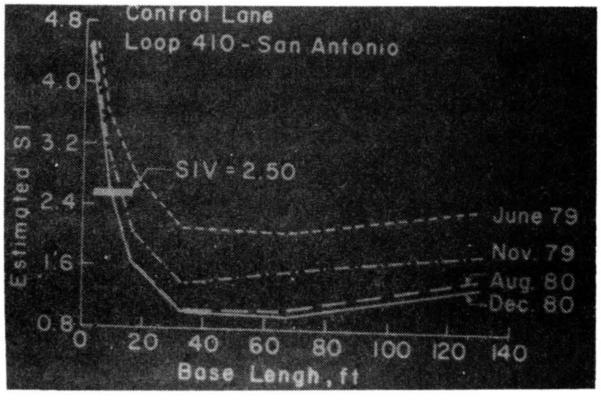
Slide 27.15. Serviceability - performance relationship for the Austin Test Sections.



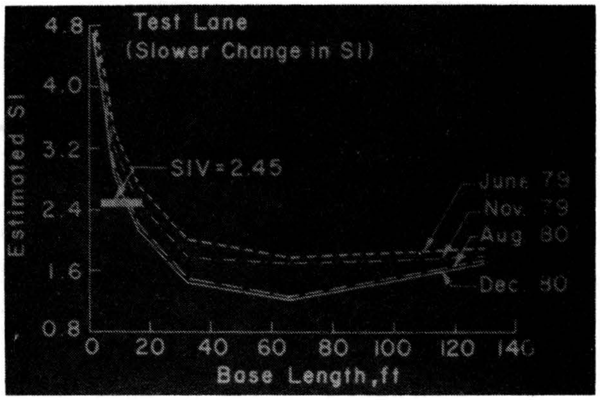
Slide 27.16. Comparison of SIs derived from rod and level profile and SDP profile.



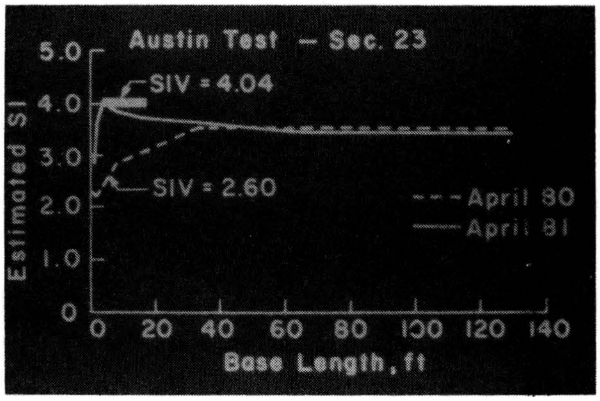
Slide 27.17. Comparison of two Maysmeters in calibration runs.



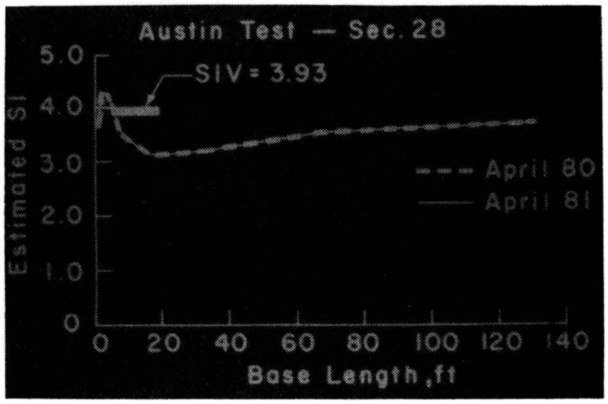
Slide 27.18. RMSVA signatures for untreated ACP sections for swelling clay environment.



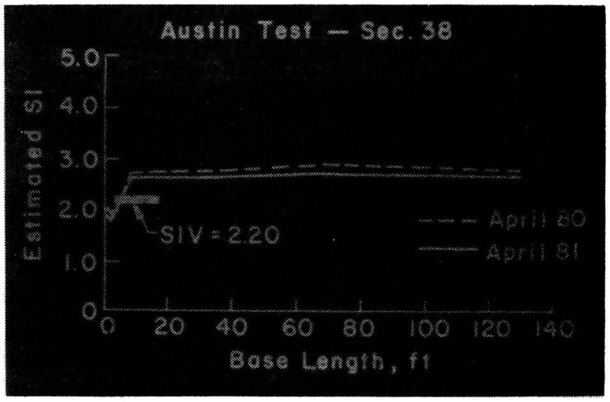
Slide 27.19. RMSVA signatures for treated ACP section in swelling clay environment.



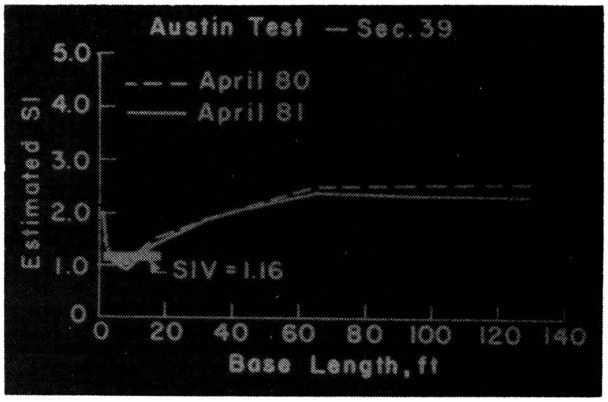
Slide 27.20. RMSVA signatures one year apart showing effect on short-wave length roughness of intervening overlay.



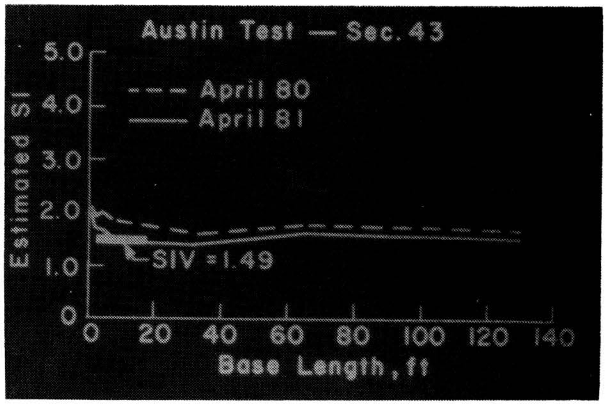
Slide 27.21. RMSVA signature one year apart showing no effect in any of the wave length roughness.



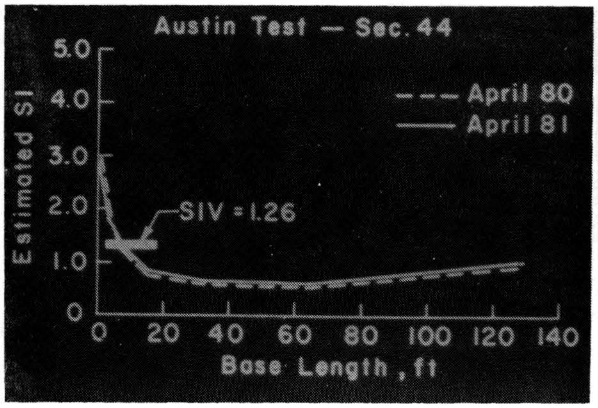
Slide 27.22. RMSVA signatures one year apart showing effect in the long-wave length roughness.



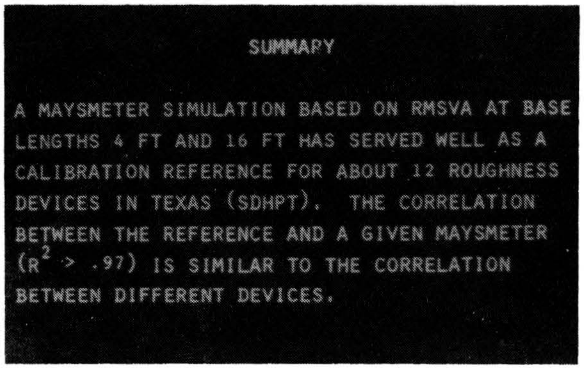
Slide 27.23. RMSVA signature one year apart showing effect on the long-wave length roughness.



Slide 27.24. RMSVA signature one year apart showing effect on the all-wave length roughness.



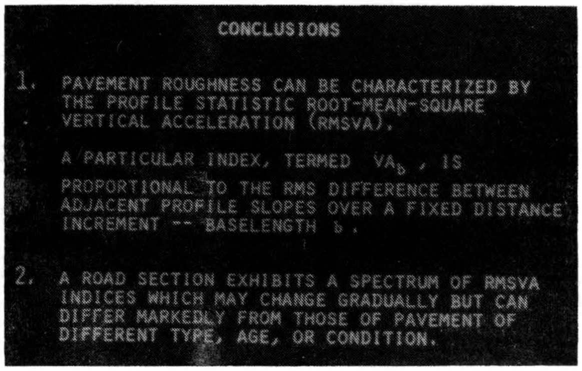
Slide 27.25. RMSVA signature one year apart showing no effect on the wave length roughness.



SUMMARY

A MAYSMETER SIMULATION BASED ON RMSVA AT BASE LENGTHS 4 FT AND 16 FT HAS SERVED WELL AS A CALIBRATION REFERENCE FOR ABOUT 12 ROUGHNESS DEVICES IN TEXAS (SDHPT). THE CORRELATION BETWEEN THE REFERENCE AND A GIVEN MAYSMETER (R^2 > .97) IS SIMILAR TO THE CORRELATION BETWEEN DIFFERENT DEVICES.

Slide 27.26. Summary.



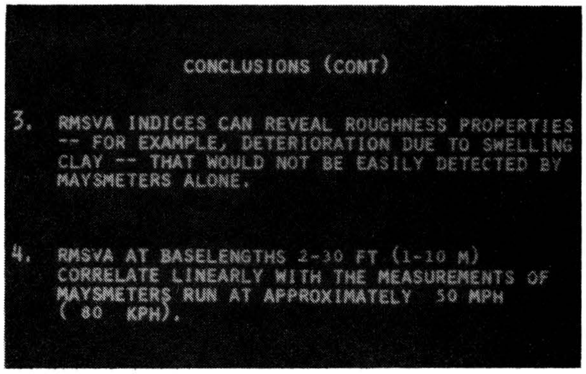
CONCLUSIONS

1. PAVEMENT ROUGHNESS CAN BE CHARACTERIZED BY THE PROFILE STATISTIC ROOT-MEAN-SQUARE VERTICAL ACCELERATION (RMSVA).

   A PARTICULAR INDEX, TERMED VA_b, IS PROPORTIONAL TO THE RMS DIFFERENCE BETWEEN ADJACENT PROFILE SLOPES OVER A FIXED DISTANCE INCREMENT -- BASELENGTH b.

2. A ROAD SECTION EXHIBITS A SPECTRUM OF RMSVA INDICES WHICH MAY CHANGE GRADUALLY BUT CAN DIFFER MARKEDLY FROM THOSE OF PAVEMENT OF DIFFERENT TYPE, AGE, OR CONDITION.

Slide 27.27. Conclusions.

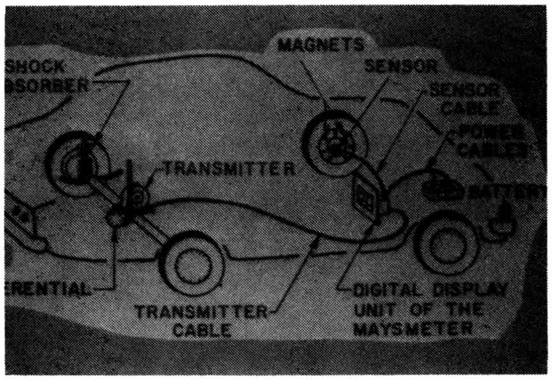


CONCLUSIONS (CONT)

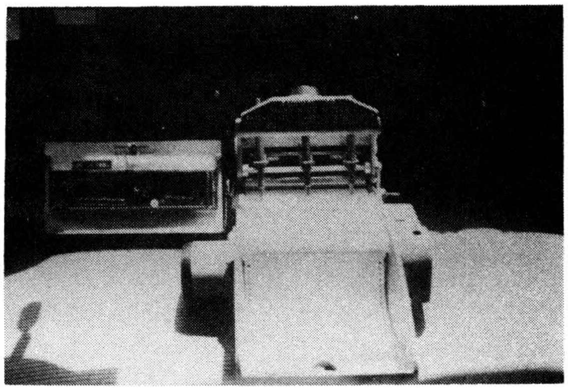
3. RMSVA INDICES CAN REVEAL ROUGHNESS PROPERTIES -- FOR EXAMPLE, DETERIORATION DUE TO SWELLING CLAY -- THAT WOULD NOT BE EASILY DETECTED BY MAYSMETERS ALONE.

4. RMSVA AT BASELENGTHS 2-30 FT (1-10 M) CORRELATE LINEARLY WITH THE MEASUREMENTS OF MAYSMETERS RUN AT APPROXIMATELY 50 MPH (80 KPH).

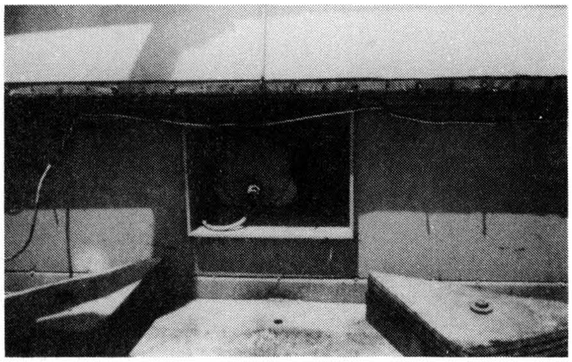
Slide 27.28. Conclusions (continued).



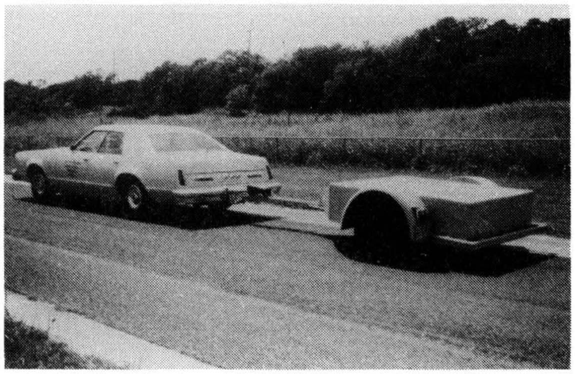
Slide 27.29. Mays Ride Meter - Schematic diagram.



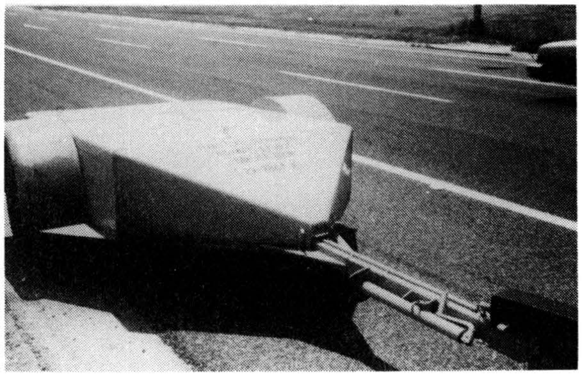
Slide 27.30. Mays Ride Meter - graphic output.



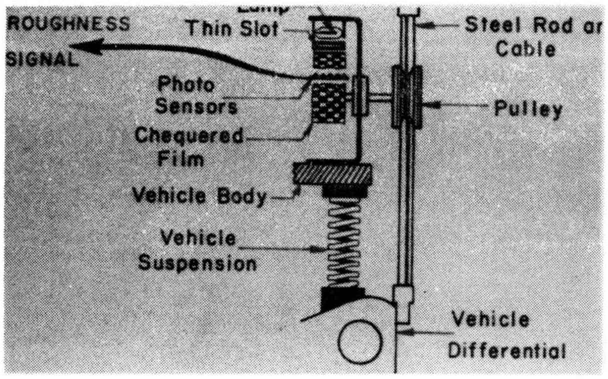
Slide 27.31. Maysmeter mounted on a trailer.



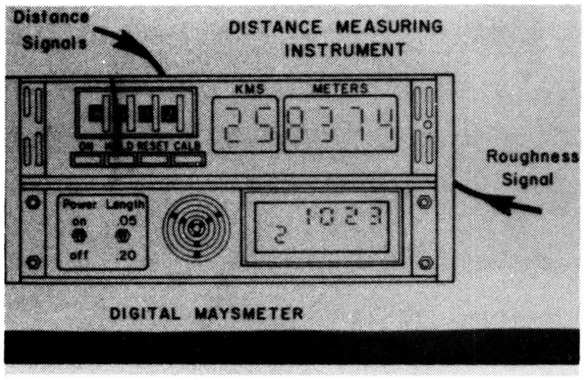
Slide 27.32. Texas Maysmeter measuring roughness on a road section.



Slide 27.33. Maysmeter trailer.



Slide 27.34. Detail description of the Maysmeter functioning.



Slide 27.35. Maysmeter - digital output.

LESSON OUTLINE

CONDITION SURVEY METHODS AND USE

Instructional Objectives

1. To demonstrate varying pavement condition survey methods and their basic purposes.

2. To present the principal components of condition surveys for flexible and rigid pavements.

3. To illustrate typical forms used in condition survey and give guidance for field survey procedures.

Performance Objectives

1. The student should be able to explain the basic purposes of pavement condition surveys.

2. The student should be able to describe general condition survey components and the principal types of pavement distresses for flexible and rigid pavements.

3. The student should be able to develop a typical condition survey form and explain the different field procedures used.

Abbreviated Summary	Time Allocations, min.
1. Condition Survey Introduction	5
2. Principal Components of Condition Surveys	20
3. Forms and Field Procedure	15
4. Relating Distress and Performance	10
	50 minutes

Reading Assignment

1. Haas & Hudson - Chapter 9, pages 97-106

2. Instructional Text A

3. Instructional Text B

Additional Reading

1. *Ontario. Ministry of Transportation and Communications, "Manual for Condition Rating of Flexible Pavements."

2. *Ontario. Ministry of Transportation and Communications," Manual for Condition Rating of Rigid Pavements."

3. *Highway Research Board, Special Report 30, "Pavement Condition Surveys."

LESSON OUTLINE
CONDITION SURVEY METHODS AND USE

1.0 CONDITION SURVEY INTRODUCTION

1.1 Definition

Condition surveys are mechanistic measurements of distress for the use of the manager of the pavement to assess the maintenance measures needed to prevent accelerated, future distress, or the rehabilitation measures needed to improve the pavement.

1.2 Purposes of Pavement Condition Survey

Condition surveys are related to the user insofar as distress is the cause of both present and future loss of serviceability. However, distress measurements should not be taken to represent user response. The uses of user responses of condition surveys are:

(a) correlation with serviceability index,
(b) establishing structural capacity, and
(c) assessing maintenance and rehabilitation needs.

2.0 PRINCIPAL COMPONENTS OF CONDITION SURVEYS

Condition surveys measure various types and degrees or severity of distress. There is some degree of commonality between the different methods with respect to the components of factors that are usually measured.

2.1 General Classification of Factors

Condition surveys may include the following general classes of factors:

(a) surface defects,
(b) permanent deformation or distortion,
(c) cracking, and
(d) patching.

2.2 Description of Pavement Distress Manifestation

2.2.1 Flexible Pavements.

(a) Surface Defects

1. coarse aggregate loss,
2. ravelling, and
3. flushing.

(b) Surface Deformation

1. rippling,
2. shoving,
3. wheel track rutting, and
4. distortion.

(c) Cracking

1. longitudinal,
2. meandering,
3. transverse,
4. alligator, and
5. random.

(d) Maintenance Patching

1. spray,
2. skin, and
3. hot-mix.

2.2 Rigid Pavements

(a) Surface Defects

1. polishing,
2. loss of coarse aggregate,
3. pot holes,
4. scaling, and
5. ravelling.

(b) Surface Deformation

1. faulting, and
2. settlement.

(c) Joint Deficiencies

1. joint creeping,
2. joint sealant loss,
3. joint spalling, and
4. joint failures.

(d) Cracking

1. longitudinal,
2. meandering,
3. corner,
4. D,
5. transverse,
6. diagonal, and
7. edge crescent.

(e) Maintenance

1. full width joint repair,
2. full depth pressure relief joint,
3. precast slab,
4. cold mix patching, and
5. hot mix patching.

(f) Miscellaneous Distresses

1. lane separation,
2. slab warping, and
3. wheel track wear.

3.0 FORMS AND FIELD PROCEDURE

3.1 Forms (Visual Aid 28.1)

The format of the necessary reporting forms for actually conducting condition surveys and processing the data varies from agency to agency. The flexible pavement evaluation form as used by the Ontario Ministry of Transportation and Communications is shown in Visual Aid 28.1.

3.2 Field Procedure (Slides 28.1 - 28.3)

(a) reconnaissance surveys,
(b) statistical surveys,
(c) semi-detailed surveys,
(d) detailed strip map surveys, and
(e) photographic surveys (Visual Aid 28.2).

4.0 RELATING DISTRESS TO PERFORMANCE

One of the important reasons for evaluating pavement distress with condition surveys involves the problem of relating distress to performance. (Visual Aid 28.3).

4.1 Background

In 1970 the HRB Workshop on Pavement Systems concluded that the first research priority in pavements was to develop better relationships between pavement distress and pavement performance.

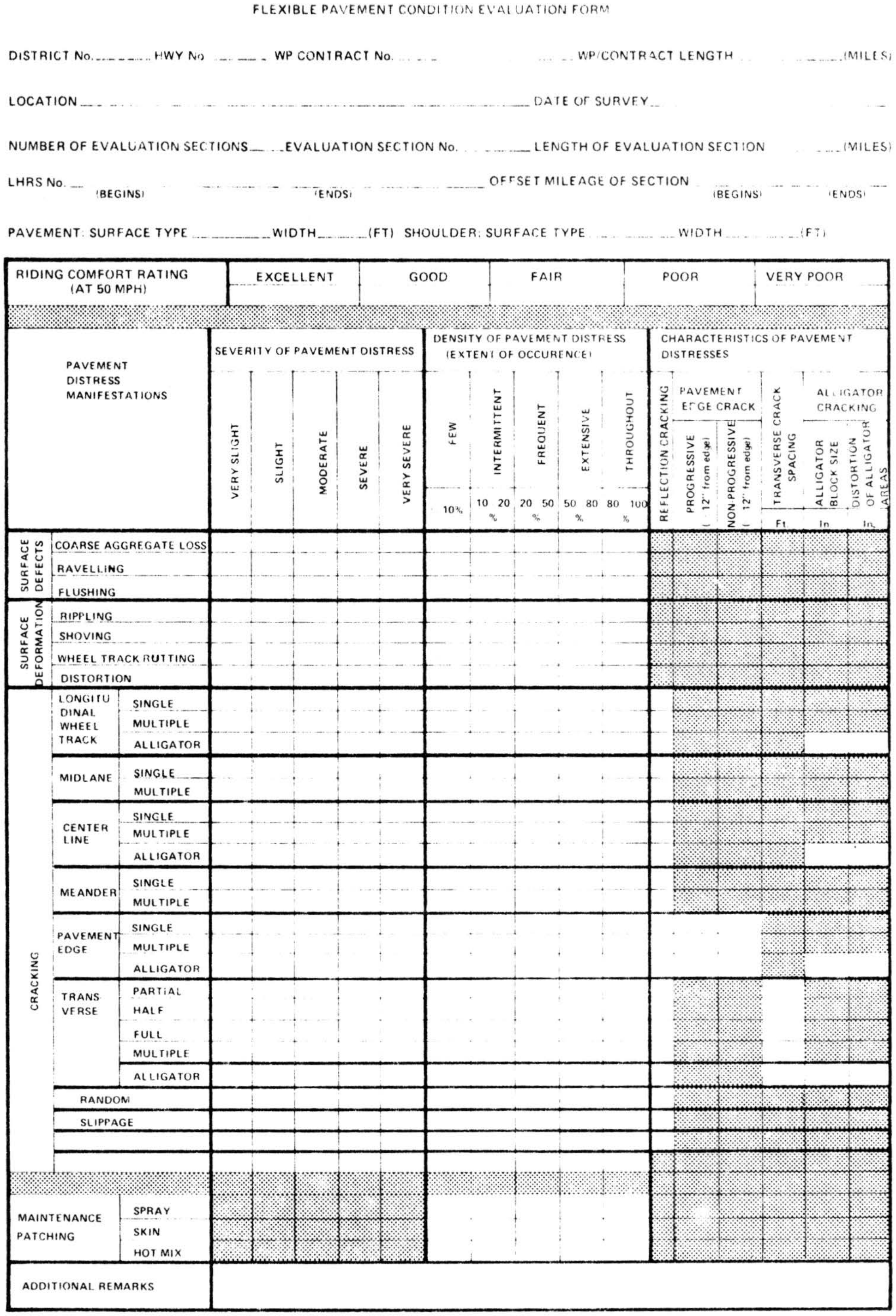
4.2 Basic Considerations

(a) prediction of type and degree of distress,
(b) prediction of component effect of a form of distress, and
(c) prediction of effect of maintenance strategies.

LESSON OUTLINE
CONDITION SURVEY METHODS AND USE

VISUAL AID	TITLE
Visual Aid 28.1.	Flexible pavement condition evaluation form.
Visual Aid 28.2.	GERPHO.
Visual Aid 28.3.	Effects of periodic major maintenance on performance curve.

Visual Aid 28.1. Flexible pavement condition evaluation form.



FLEXIBLE PAVEMENT CONDITION EVALUATION FORM

DISTRICT No.______ HWY No _____ WP CONTRACT No. _____ WP/CONTRACT LENGTH _____ (MILES)

LOCATION _____ DATE OF SURVEY _____

NUMBER OF EVALUATION SECTIONS _____ EVALUATION SECTION No. _____ LENGTH OF EVALUATION SECTION _____ (MILES)

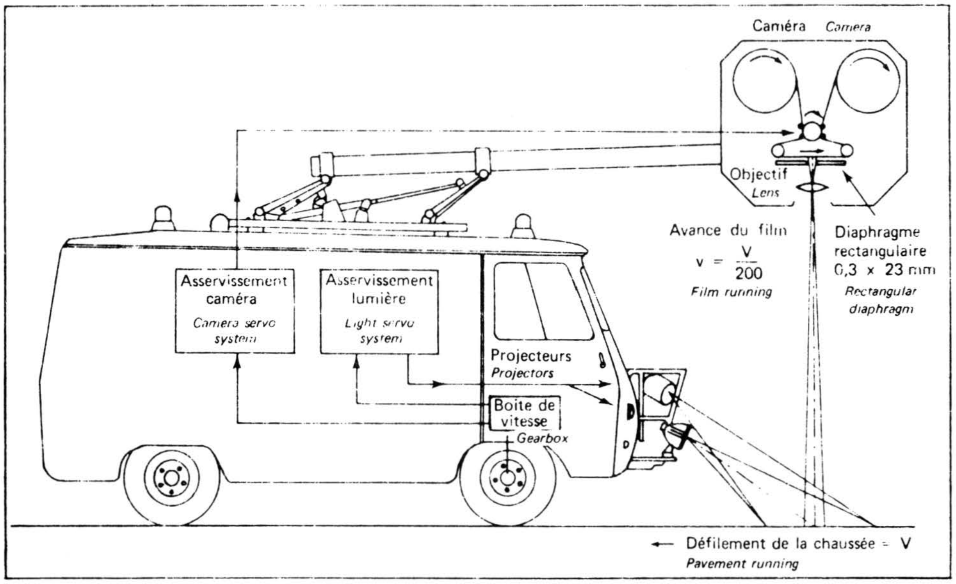
LHRS No. _____ (BEGINS) _____ (ENDS) _____ OFFSET MILEAGE OF SECTION _____ (BEGINS) _____ (ENDS)

PAVEMENT: SURFACE TYPE _____ WIDTH _____ (FT) SHOULDER: SURFACE TYPE _____ WIDTH _____ (FT)

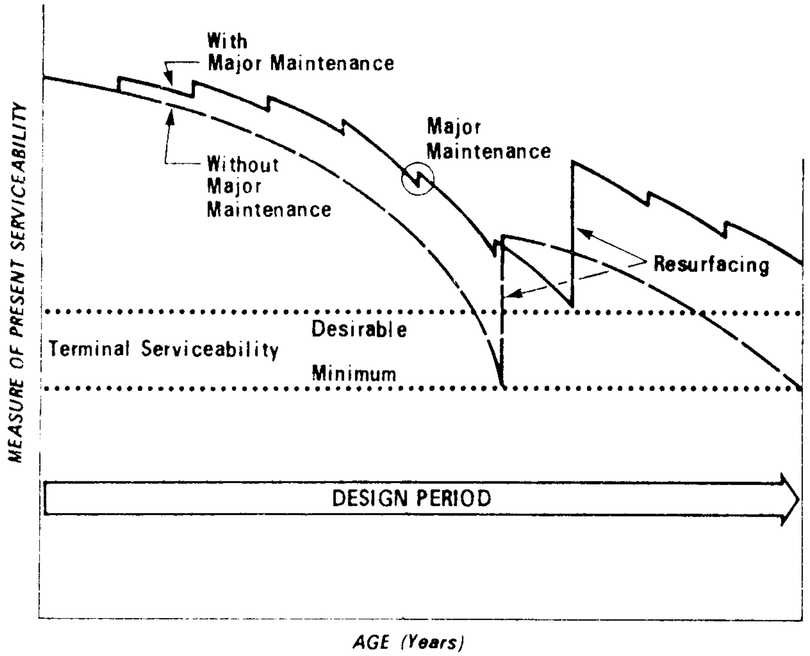
RIDING COMFORT RATING (AT 50 MPH)	EXCELLENT	GOOD	FAIR	POOR	VERY POOR

PAVEMENT DISTRESS MANIFESTATIONS			SEVERITY OF PAVEMENT DISTRESS					DENSITY OF PAVEMENT DISTRESS (EXTENT OF OCCURENCE)					CHARACTERISTICS OF PAVEMENT DISTRESSES					
			VERY SLIGHT	SLIGHT	MODERATE	SEVERE	VERY SEVERE	FEW	INTERMITTENT	FREQUENT	EXTENSIVE	THROUGHOUT	REFLECTION CRACKING	PAVEMENT EDGE CRACK		TRANSVERSE CRACK SPACING	ALLIGATOR CRACKING	
								10%	10 20 %	20 50 %	50 80 %	80 100 %		PROGRESSIVE (12" from edge)	NON PROGRESSIVE (12" from edge)		ALLIGATOR BLOCK SIZE	DISTORTION OF ALLIGATOR AREAS
																Ft	In	In.
SURFACE DEFECTS	COARSE AGGREGATE LOSS												▨	▨	▨	▨	▨	▨
	RAVELLING												▨	▨	▨	▨	▨	▨
	FLUSHING												▨	▨	▨	▨	▨	▨
SURFACE DEFORMATION	RIPPLING												▨	▨	▨	▨	▨	▨
	SHOVING												▨	▨	▨	▨	▨	▨
	WHEEL TRACK RUTTING												▨	▨	▨	▨	▨	▨
	DISTORTION												▨	▨	▨	▨	▨	▨
CRACKING	LONGITUDINAL WHEEL TRACK	SINGLE												▨	▨	▨	▨	▨
		MULTIPLE												▨	▨	▨	▨	▨
		ALLIGATOR												▨	▨	▨		
	MIDLANE	SINGLE												▨	▨	▨	▨	▨
		MULTIPLE												▨	▨	▨	▨	▨
	CENTER LINE	SINGLE												▨	▨	▨	▨	▨
		MULTIPLE												▨	▨	▨	▨	▨
		ALLIGATOR												▨	▨	▨		
	MEANDER	SINGLE												▨	▨	▨	▨	▨
		MULTIPLE												▨	▨	▨	▨	▨
	PAVEMENT EDGE	SINGLE														▨	▨	▨
		MULTIPLE														▨	▨	▨
		ALLIGATOR														▨		
	TRANSVERSE	PARTIAL												▨	▨		▨	▨
		HALF												▨	▨		▨	▨
		FULL												▨	▨		▨	▨
		MULTIPLE												▨	▨		▨	▨
		ALLIGATOR												▨	▨			
	RANDOM													▨	▨	▨	▨	▨
	SLIPPAGE													▨	▨	▨	▨	▨
														▨	▨	▨	▨	▨
													▨	▨	▨	▨	▨	▨
▨	▨	▨	▨	▨	▨	▨	▨	▨	▨	▨	▨	▨	▨	▨	▨	▨	▨	▨
MAINTENANCE PATCHING		SPRAY	▨	▨	▨	▨	▨						▨	▨	▨	▨	▨	▨
		SKIN	▨	▨	▨	▨	▨						▨	▨	▨	▨	▨	▨
		HOT MIX	▨	▨	▨	▨	▨						▨	▨	▨	▨	▨	▨
ADDITIONAL REMARKS																		

Visual Aid 28.2. GERPHO.



Visual Aid 28.3. Effects of periodic major maintenance of performance curve.



INSTRUCTIONAL TEXT A
ADVANCED CONDITION SURVEY METHODS

By

Sukumar Nair

Tech Memo 256-11

INTRODUCTION

The development of data bases is vital to the efficient implementation of a pavement management system. These data bases need to be updated on a regular basis to enable intelligent and reasonable decision-making on pavement management strategies. Typically, ground crews perform condition surveys at speeds ranging from one to ten miles per hour. This method has serious drawbacks in that

(a) It is labor intensive and therefore costly;

(b) It is time consuming;

(c) It exposes personnel to traffic hazards; and

(d) If the road network to be maintained is large, the method is uneconomical.

Therefore, the need exists for alternate (and more advanced) methods to be explored. Aerial photography, the Micrologging Photologging System and the Gerpho continuous strip method offer potential alternatives.

Aerial photography has been applied to condition surveys in only a few states, although it is extensively used by highway departments in various other aspects of highway engineering.

The Micrologging Photologging System has found diverse applications in transportation engineering, such as equipment inventory, sign visibility, traffic control, project design and planning. By obtaining a series of photographs of the pavement, the system provides a potential method of conducting condition surveys.

The Gerpho, developed and used in France, evaluates the surface condition of the pavement by using continuous strip photographs of pavement sections. This method has been used effectively and may offer a better alternative to manual condition surveys than aerial photography.

These methods are described in the following sections.

According to Rib (Ref. 1), only 15 organizations used air photo interpretation for road condition and inventory and damage surveys to varying extent. Of these, just two used this method extensively. This information was taken from the results of a questionnaire reported in 1962. Though the questionnaire results are outdated, indications are that air photo interpretation is not extensively used by many state highway organizations for condicondition surveys at the present time.

Studies of the uses of aerial photography in condition surveys of highways have been made by the State Highway Commissions of Maine and Kansas. Airfield runway pavement surveys using aerial photography have veen done in Greenland for the U. S. Air Force.

The purpose of Stoeckeler's study (Ref.2) was to determine the optimum film type and scale combination from which adequate information for flexible pavement evaluations can be obtained. He concluded that the type of film which gives the most information on flexible pavement distress features is an infrared color transparency with a photoscale of 1 in. equals 200 ft. or larger.

Two pieces of literature were found pertaining to aerial photography work done in Kansas. Stallard and Biege (Ref. 3) were interested in comparing the usefulness of color film as opposed to black and white. Meyers and Stallard (Ref.4) reported on using aerial photography to detect staining in portland cement concrete pavements associated with "D-cracking." Stallard and Biege concluded that Agfacolor Negative film was superior to others for the purposes of road condition surveys. Meyers and Stallard determined that low level aerial photographs can be successfully used as an indicator of pavement sections which require maintenance resulting from D-cracking in rigid pavements. They recognized a correlation between D-cracking and staining of the pavement which was discernable from the photographs. The various types of film which were used were all deemed satisfactory for the purposes of their research.

The technique developed by the Rome Air Development Center and the Calspan Corporation under the Special Color Analysis Techniques Program for runways (Ref. 5) appears to be promising. They were able to find regions of cracking, depressions, and high surface roughness by using a photointerpretation console which allows subtle differences in spectral bands of color film to be enhanced. This device also contains interpretation keys that relate the spectral differences to properties desired such as material type and surface deterioration. The research performed at Thule Air Force Base in Greenland showed that the results obtained from this technique agree very well with surface measurements made at the site. Since this method apparently performs successfully for runway pavements, it may be possible to adapt it for use with highway pavements.

The groundwork for the application of computer enhancement techniques to the analysis of aerial photographs is available (Ref.6). If the photograph is taken immediately after a shower, water marks associated with the cracks become visible as the pavement dries. Thus the cracks are easily identified by those water marks, which are used for estimating the locations and spacings of cracks quite accurately.

The approach consists of locating the water marks by detecting the boundaries of the water marks. The edges corresponding to region boundaries are detected by using an edge operator (the Kirsch operator has been utilized). In order to reduce noise the edge operator is coupled with a non-maxima suppression technique. The Hough transform is applied to the binary edge picture so obtained to find the direction of the road and road sections. Next, algorithms are developed to detect water marks and estimate crack spacings. The procedure, then, entails the following four steps:

(1). Picture digitization

(2). Edge detection

(3). Road detection

(4). Labelling of potential edge lines and potential boundaries.

A flow chart of the procedure & detect cracks is shown in Fig. 1.

Best results are obtained with a helicopter at low levels (50 to 100 feet) immediately after a rain.

THE MICROLOG PHOTOLOGGING SYSTEM

The Microlog Photologging System provides a color photographic record of the "driver's eye view" of highways as a function of distance traveled. Because of the particular disposition of its components that are arranged to provide the driver's eye view of the road, the Photologging System allows pictures of the pavement to be taken at different angles of incidence, from 0° to +15°.

The basic system consists of a specially designed 35 mm Automax cine/pulse camera with suitable lens, camera mount, control unit, camera actuation device and a power converter, all mounted inside a van.

The Automax camera, originally designed for flight test aircraft, has been appropriately modified for photolog applications. The camera uses Auto-Nikkor lenses that cam be interchanged and offers an exposure range from 1/64 to 1/1000 second. A 400-foot film roll provides about 6800 frames.

The camera mount is two-axis adjustable with a release for removal of the camera. It allows adjustment in elevation from +15° to -15° and in azimuth from +30° to -30°.

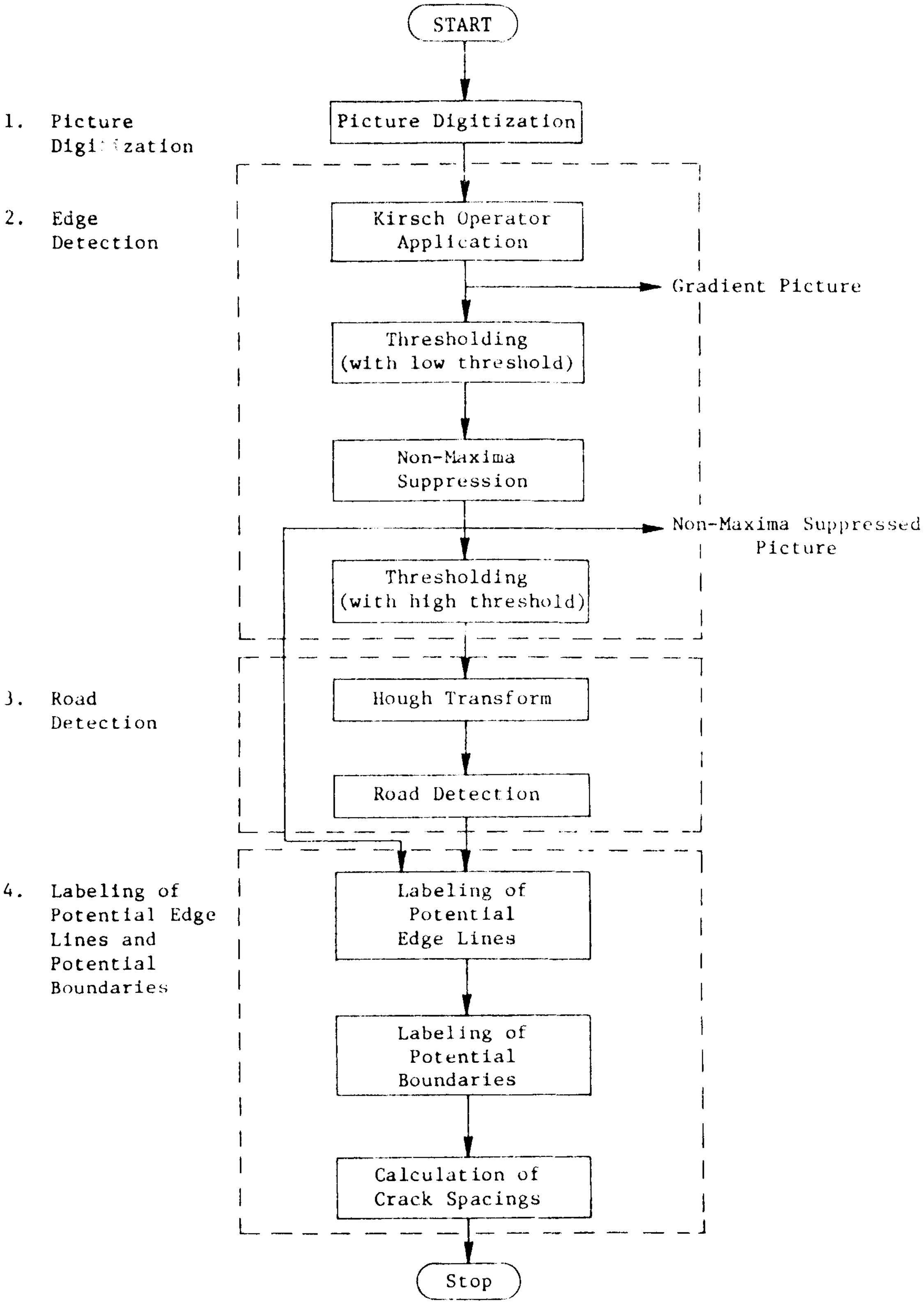


Fig 1. Flow chart of the system to detect cracks on a highway pavement.

The control unit mounted at driver's reach, contains the main power control, camera functions control and display that allows the operator to verify mileage count while driving.

The camera actuation device is mounted to the odometer shaft. It can be regulated to give a pulse every given distance between a wide range (1 to 10,000 feet). The frequency of the pulse is only a function of the distance traveled, being independent of the speed of the vehicle, which can vary while traveling and can be as high as 60 mph.

A study was conducted to evaluate the possibility of using the Microlog System to perform pavement condition surveys. Two sections in Austin, Texas were selected: a jointed portland cement concrete pavement and an asphalt concrete pavement, each about 300 ft. long.

Two sets of pictures were taken. The first set was intended to determine the maximum vertical angle of the lens axis, suitable to obtain pictures without interference of the front of the van. Angles from 6° to a maximum of 15° were tried in increments of 3°. The front of the van did not interfere at any of these angles, therefore a 15° angle was selected to obtain a second set of pictures.

In both cases, the sunlight was illuminating the pavement laterally, and the pavement was completely dry.

To evaluate the resolution of the cracks of the pavement in the pictures, a section of the road with many cracks of different widths was selected. Pictures were taken

(1) with the van stationary,

(2) with all cracks upto 30 ft. from the camera mapped with white chalk, and

(3) with the van moving at approximately 30 mph (continuous set of pictures every 25 ft.).

RESULTS

Observations with the MPS showed that less than 20 percent of the cracks are clearly seen in the case of concrete pavements and less than 30 percent in the case of asphalt pavements. It was also observed that longitudinal cracks appeared more clearly (on both pavements) than transverse cracks, due to the direction of sunlight. However, longitudinal cracks of small width did not appear in the picture, even those close to the camera.

The difference in scale between portions of the picture at different distances of the camera does not allow a relative comparison of the crack widths, since similar cracks appear wider when closer to the camera.

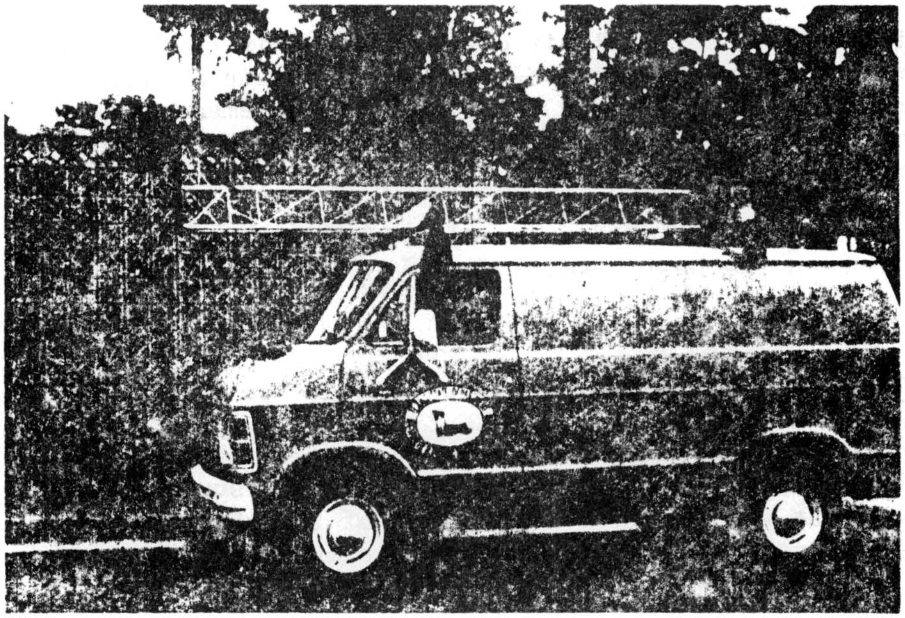


Fig 2. Camera mounted outside van (District 10 Study).

Patches, potholes and such pavement distresses are displayed more prominently than cracks but due to the scale problem, quantitative evaluation of their sizes is ruled out.

A difference in the quality of the photographs taken while the van was stationary and while moving was also observed.

An important consideration in the analysis of photographs is the continuity of pictures so that there are no gaps between frames. To obtain a continuous picture of the pavement a fix reference on or near the pavement should be selected. However, if the pavement does not show distresses it is difficult to select a reference.

Work done by District 10 with the camera mounted outside at a 90° angle of incidence (Fig. 2) has shown the best results yet.

CONTINUOUS STRIP METHODS

In 1960, the New York State Department of Public Works (Ref. 7) developed a technique wherein continuous photographs of the pavement surface were obtained. They determined that the degree of resolution of the photographs was adequate for class 4 surveys in which the features are sketched to scale in detail on a strip map.

THE GERPHO

The Groupe Examen Routier Photographic (GERPHO) system (Ref. 8) was developed by the French Ministere de C'Equipment (Laboratoire Central Des Ponts et Chaussees et Laboratoire Regional de Nancy) to accomplish the same purpose. It has been used in France primarily for the evaluation of urban freeways.

The GERPHO system basically is comprised of a 35 mm. continuously running (strip film) camera, mounted on a van - originally on a Peugeot J-7 vehicle - which has a mounted light source that illuminates the pavement (Fig. 3).

The camera is mounted on the van by means of a support that is attached at the roof. The boom mount allows the height of the camera to be varied with respect to the pavement (Fig. 4). The camera is fitted with 14.5 mm. lens, with an aperture of F-3.5. Automatic cartridges hold 120 meters (394 ft.) of 35 mm. film and can photograph 24 km. (15 miles) of pavement continuously. The scale chosen was 1/200 (film useful width divided width of filmed pavement) which means that the camera lens shoud be placed at 2.90 m. (9.5 ft.) above the pavement (focal length/heigth = 1/200) (Fig. 5) and because the effective width of the film is 23 mm., with 1/200 scale, the width of the pavement filmed is 23 mm. x 200 = 4.6 m. (15 ft). So the picture covers the entire traffic lane along which the van moves, together with part of the adjacent lane and/or part of the shoulder.

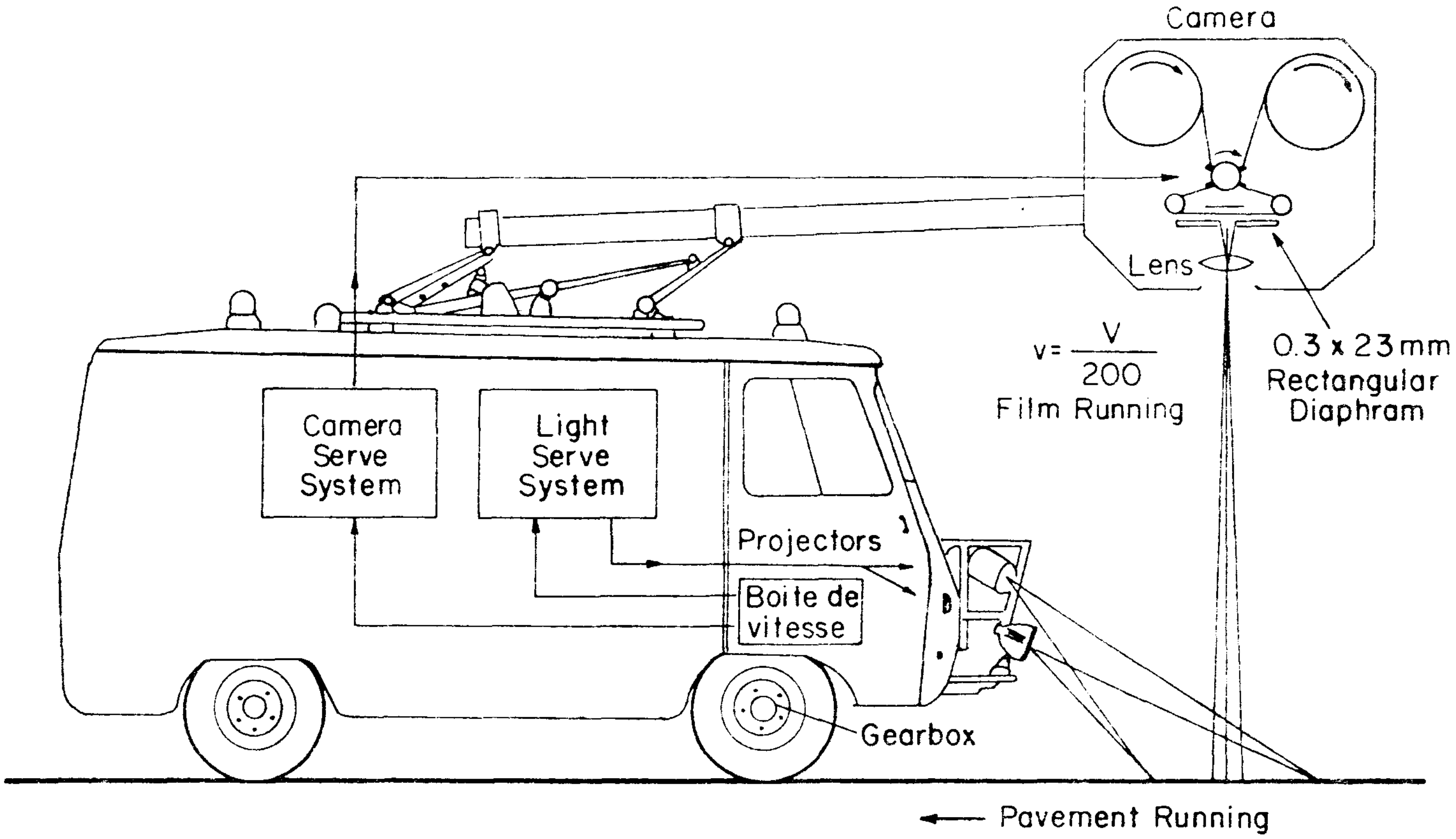


Fig 3. Elements of the GERPHO System.

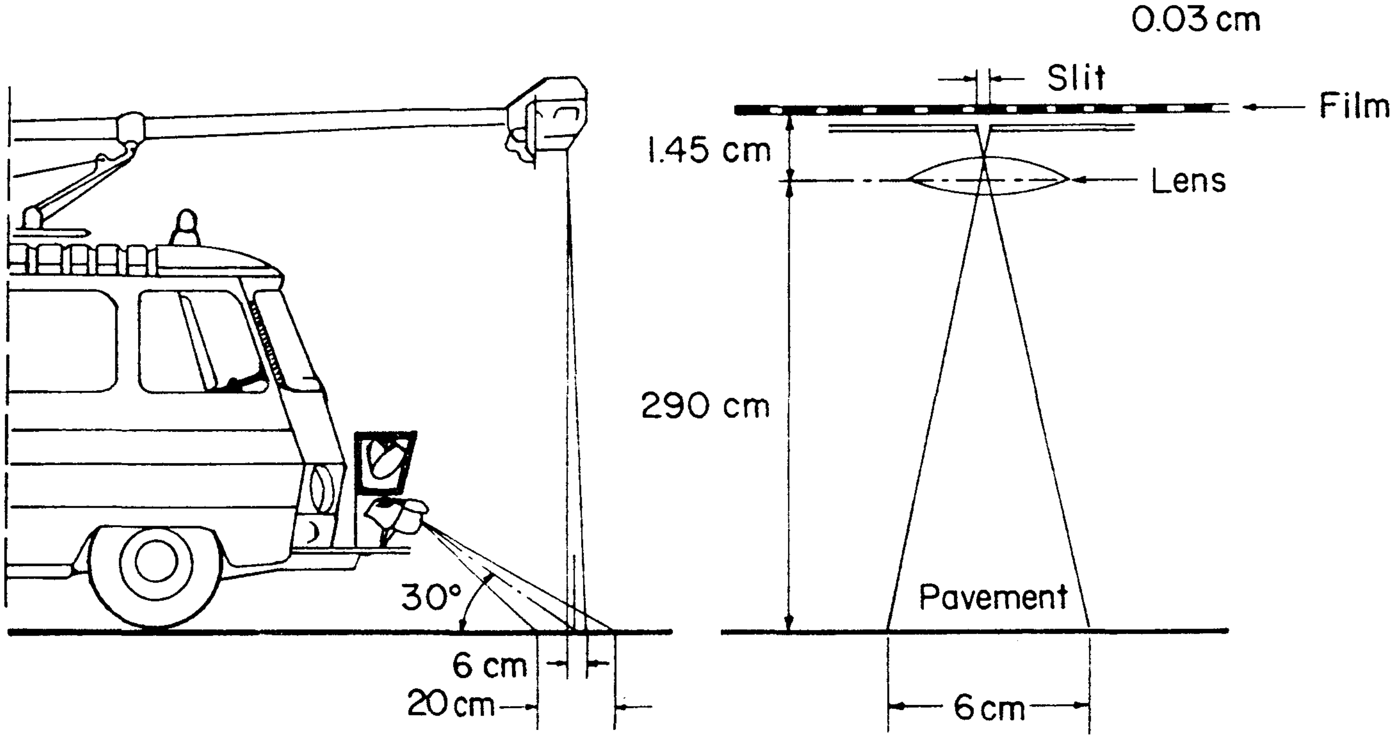


Fig 4. Camera details (The GERPHO System).

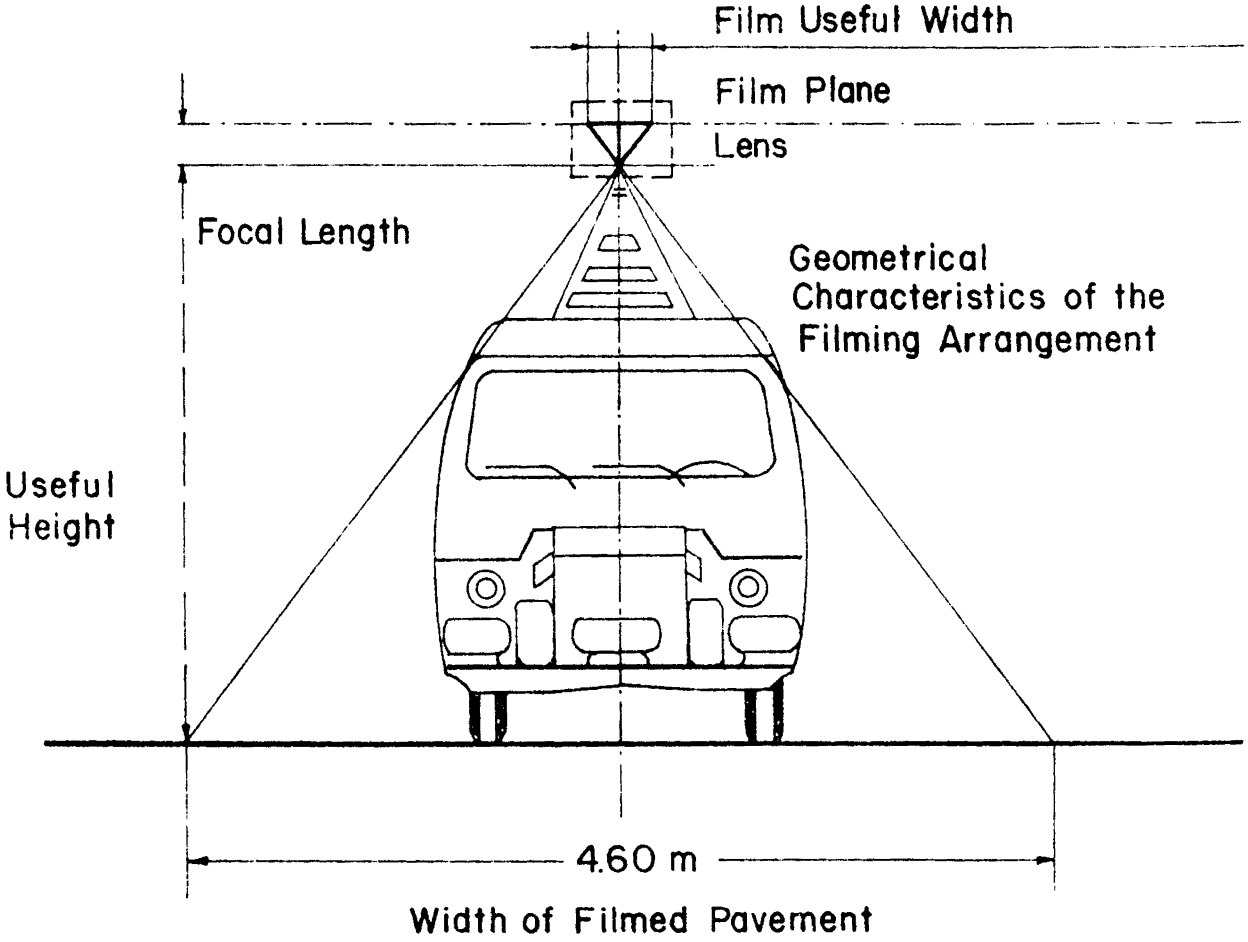


Fig 5. Height adjustment of GERPHO camera for spanning the full pavement width.

If the vehicle speed is 60 kmh (37.5 mph), for example, the film must run at 60/200 = 0.3 kmh (0.27 ft/sec). A motor impulses the film and a special system is used to insure synchronization between the advance of the film and the movement of the vehicle, such that there will not be any overlap or gap between exposures.

The light source for illumination of the pavement is comprised of five 1,000 Watt iodine projectors that illuminate at an angle of 30°. The intensity of the light source is adjusted by means of a special system that assures an uniform exposure of the film even if the speed of the vehicle is not precisely constant.

The photographic survey with the GERPHO device is performed at night, at a speed up to 60 kmh (37.5 mph.). If the pavement is in good condition, it is possible to film 100 to 200 km (62 to 125 miles) of pavement per night.

The crew is composed of two operators that are responsible for the complete work. A control panel installed inside the vehicle helps to continuously check the camera, projectors, vehicle lights and other parts of the system. A record sheet that indicates special features is completed by one of the operators along with the photographic survey such that the film can be identified properly at any point by means of a special device in the control panel.

The recording and processing processes are indicated schematically in Fig. 6.

The analysis of the pavement surface is made with the use of a special viewer that allows the operator to identify different types of distresses according to a standard classification and record them in a suitable form. The data is stored in computer files and the computer is capable of tracing this data automatically in another form.

The projector can show two sets of films at the same time, so the records at different times of the same section of pavement can be carefully analyzed. Figures 7, 8 and 9 are representative of the quality pictures obtained.

The total cost of the device and the vehicle would be $250,000 to $300,000. There is a possibility of mounting the GERPHO device in an American vehicle; however this will require a study of the vehicle and significant advantages are not apparent at the present time.

The projector which shows two films simultaneously is very important for the evaluation of the data and it will be vital for the whole process. The cost of this device is about $50,000 to $75,000. So the minimum initial investment will be about $300,000 to 375,000.

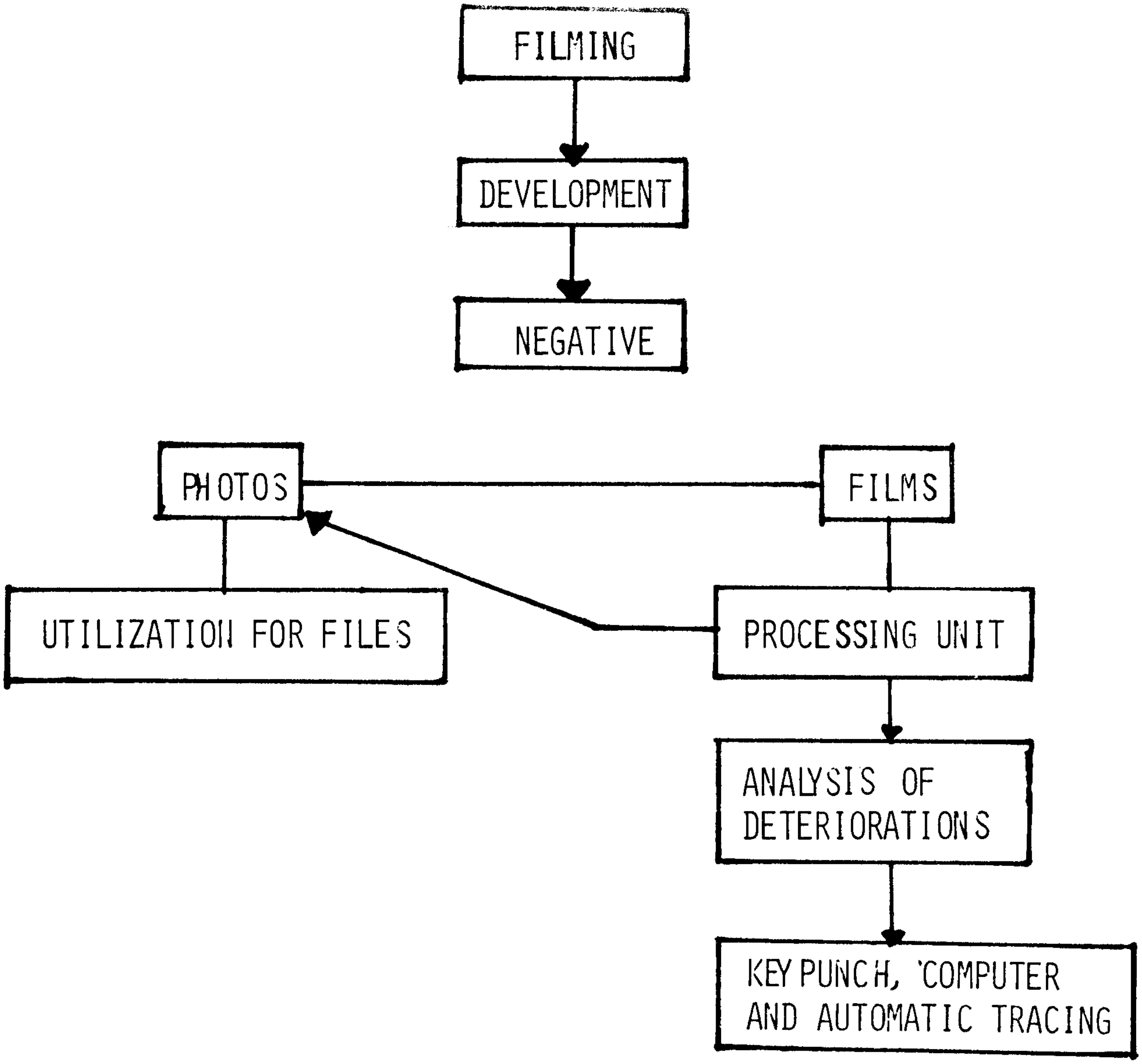


Fig 6. Schematic of the GERPHO system processing.

CODE :

EPAUF
CASS
ANGL

EPAUF
CASS
ANGL

FAIE
BAU
J DAL

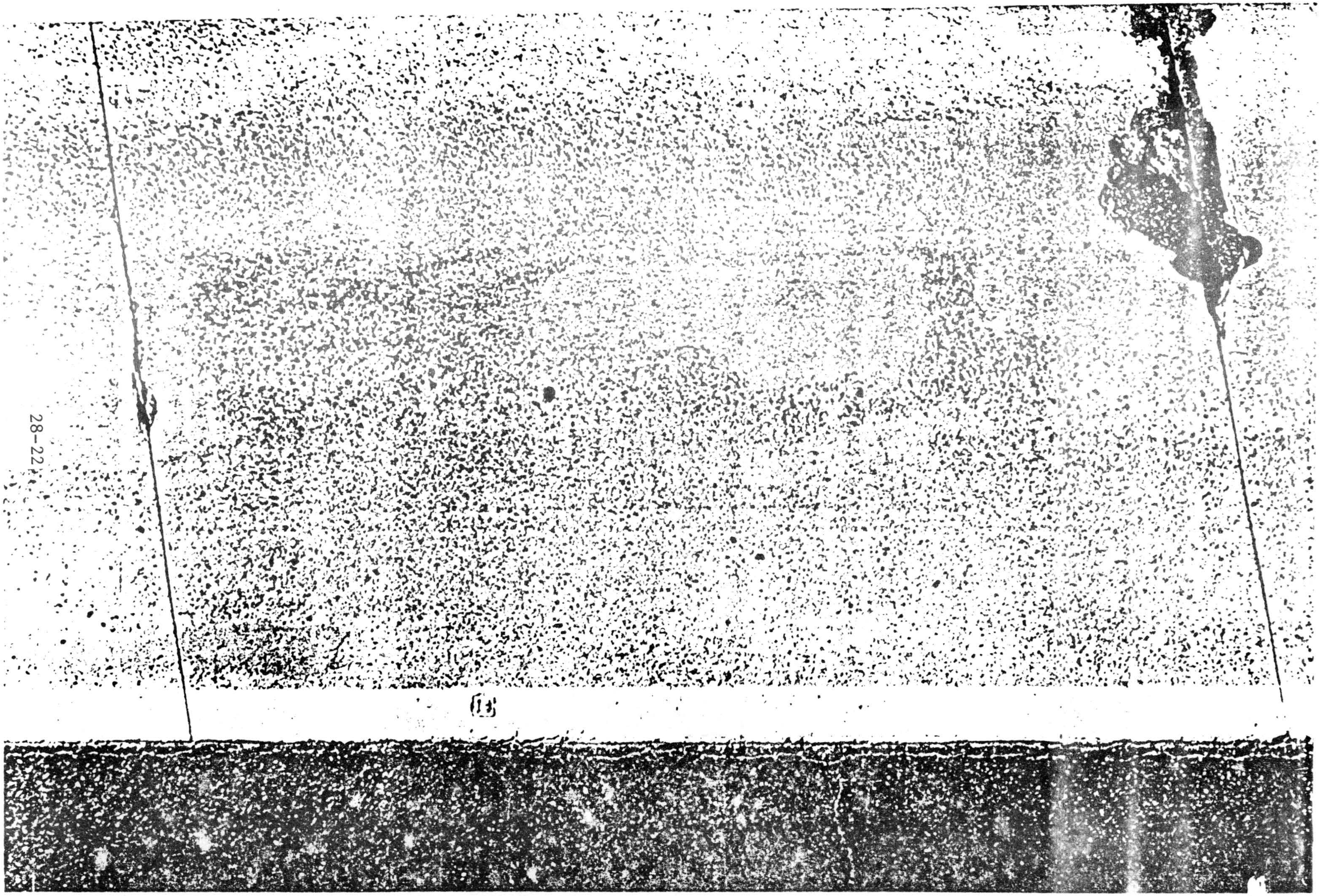


Fig 7. GERPHO photograph showing spalling.

CODE :

FISS
OBLI

EPAUF
CASS
ANGL

FAIEN
BAU
J DAL

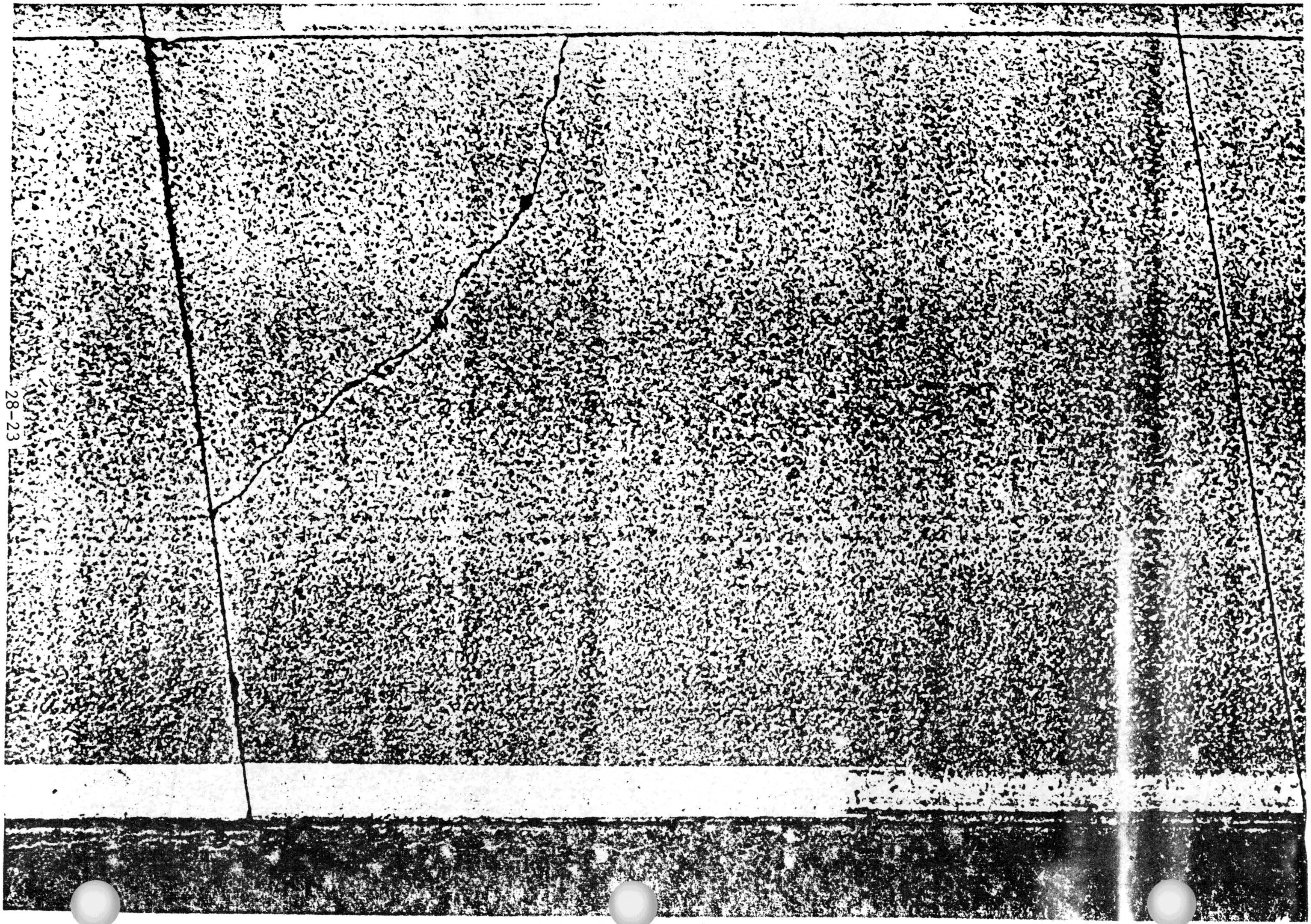


Fig 8. GERPHO photograph showing an edge crack.

CODE:

FISS
LONG

FAIEN
B A U
J DAL

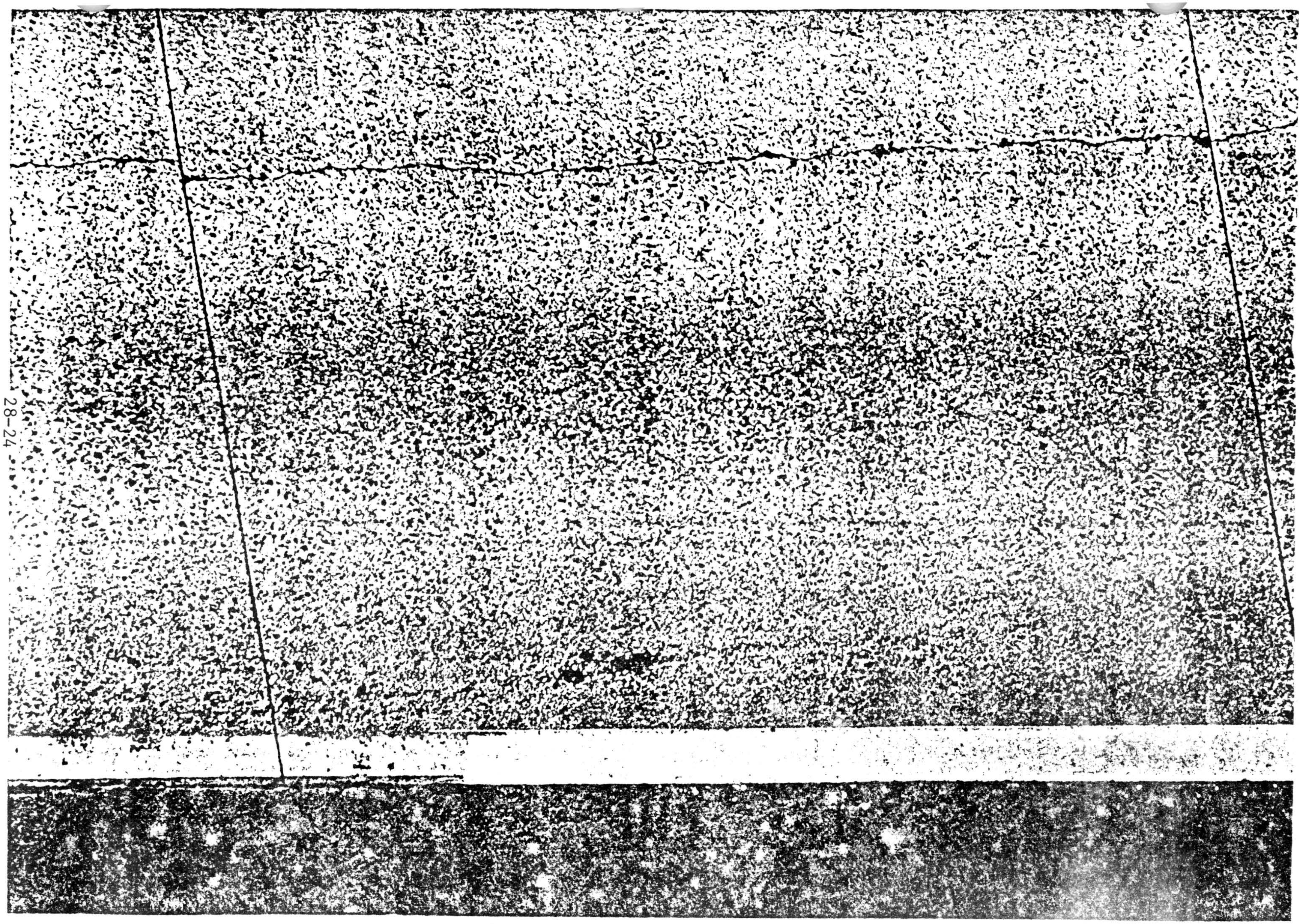


Fig 9. GERPHO photograph showing a longitudinal crack.

CONCLUSIONS

Aerial Photography

This is a method that could be implemented for use in Texas for condition surveys because of the availability of the equipment and previous experience with aerial photography in other areas of highway engineering.This makes aerial photography an attractive alternative to using manual condition surveys. This method has several disadvantages, however, including the need for good weather and limitations on how much information can be gathered by visual inspection of the photographs alone.

In the water marks approach it is difficult to recognize each individual watermark because of the undistinguishable nature of closely spaced cracks, a well-defined boundary (for water marks) and the presence of road film stripes in the center of lanes. Moreever, cracks will be visible only after the pavement is wetted; i.e. the success of the method hinges on the occurrence of a shower.

There are also problems with shadows and the inability to photograph the entire pavement from above due to vehicles on the road. The drone of a helicopter 100 feet above can prove to be an annoying experience to highway users; there is also a potential hazard to the driver; and the operating and processing costs are high.

The Microlog Photologging System

Because of all the problems outlined previously, the Microlog Photologging System under its original configuration is not suitable for mapping crack patterns in either concrete or asphalt pavements and produce data for a meaningful evaluation of pavement distresses. However, if the equipment can be readapted to obtain pictures at a 90 ° angle of incidence (as studied by Districts 10 and 11) the pictures may be more useworthy. Even though a 90 ° angle is better than other angles of incidence because it eliminates distortion, the overlap and location problems remain. This makes it a time-consuming, labor intensive and expensive method.

GERPHO

Although the initial cost of purchasing or manufacturing a system such as GERPHO would be high, it appears to be the best alternative due to its distinct advantages:

(1). Greater capacity, 100 to 200 km. (63 to 125 miles) per working night.

(2). Only two operators are required and they do not have to be highly skilled.

(3). Interpretation errors are avoided.

(4). Operated at night using an onboard light source, hence avoids shade problems.

(5). System can be moved rapidly and safely.

(6). Presence of other vehicles on the roadway is not a problem.

(7). Low operating costs.

(8). Continuous strip, no overlap problems.

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INSTRUCTIONAL TEXT B
THE ROLE OF VISUAL CONDITION SURVEYS IN A PAVEMENT MANAGEMENT SYSTEM (PMS)

By

Frank Carmichael

Tech Memo RI-1

Visual condition surveys are the oldest and probably the most widely used form of pavement evaluation. To keep these activities useful and meaningful in a pavement management activity, it is necessary to try and reduce their subjective nature as much as possible. This memorandum reviews the state-of-the-art for visual condition surveys data collection issues, data use in pavement management systems, current RIDOT capabilities. and possible alternatives.

A recent ARE Inc study (Ref 1) for the FHWA cited some of the more common problems associated with accurate collection and use of condition survey data in a PMS. First of all, there are problems with collecting the data and collecting it accurately. Some of these problems include:

1. The danger associated with having field crews survey heavily trafficked roads.
2. The overall cost of conducting field surveys and reducing the data.
3. What data to collect; Lytton et. al. (Ref. 2) present some possible pavement distresses in Table 1.
4. How much and how often (every mile, every year, or 100 ft/mile every other year).
5. What data can be collected subjectively (and to what extent) and what data requires objective measurements.
6. How to train crews to obtain consistent data (i.e., how to get good repeatability).

Table 1. Pavement Distress Indicators
(after Reference 2)

Flexible Pavement

Transverse cracking
Longitudinal cracking
Multiple Cracking (beginning of alligator cracking)
Alligator Cracking
Rutting
Raveling
Patching
Flushing (or bleeding)
Corrugations

Rigid Pavements

All rigid pavements
Surface deterioration
  - raveling
  - scaling
Spalling
Longitudinal cracking
Patching
Faulting
Pumping
Failures per mile
Blowups

Continuously reinforced concrete
Crack spacing
Percent intersecting cracks

Jointed concrete
Spalled joints
Faulted joints
Cracked panels
Broken panels
Transverse cracking

Secondly, there is a class of problems dealing with evaluating the field data and then relating it to some overall measure of pavement condition that can be used as part of a priority ranking system for maintenance and/or rehabilitation. Some of these problems include:

1. How to handle error or variation in the data.
2. What interactions between distress parameters to consider.
3. What weighting factors to assign to the different distress parameters.
4. How to interpret differences between overall ratings in terms of priority.

Although it will be impossible to solve all these problems in the development of a highway condition rating function, they should be kept in mind in order to minimize their effects. For example, by requiring that only certain significant distress parameters be collected, the volume of condition survey data can be minimized, thereby reducing the effects of several problems. These questions are considered in the formulation of the summarized approach for Rhode Island Department of Transportation.

SUMMARY

Our review of the literature concerning the evaluation of pavement surface condition and its quantification into a measure of overall distress turned up several similar models. One developed by Shahin and Kohn (Refs. 6 and 7.), which uses the "summation of deducts" approach, contained most of the better features used by individual states. This approach can be used to develop a combined condition rating score; however, it is suggested that pavement distresses be identified and recorded individually for other uses.

Since it is considered desirable to have a model which has some practical meaning in terms of the amount of damage sustained by a particular pavement, work can be initiated on the development of relationships between fatigue damage and measurable pavement distress. It is the belief of the authors that the use of such a procedure will also help in the determination of weighting coefficients for the surface condition term of the highway condition rating function.

BACKGROUND

There are four major categories of pavement surface distress which are usually recognized by the various State methods employed:

1. Cracking (alligator, longitudinal, transverse, map. reflection)
2. Disintegration (raveling, stripping, spalling, scaling)
3. Permanent Deformation (rutting. faulting etc.), and
4. Distortion (settlement, heave, etc.).

Different distress indicators are evaluated as a function of pavement type as indicated by Lytton. et. al. (see Table 1). A review of the data collection forms published by the various states indicates that there are two primary components which define the individual distresses.

1. Density - How much of the distress exists?
2. Severity - How bad is the distress?
   To what condition has it progressed?

As the compendium of example data collection forms in Appendix A shows, there are many number of ways to define these parameters.

Density

Density or extent can be rated subjectively, or each distress can be measured. There are cost and data quality tradeoffs between these two different approaches. The most common approach is a subjective estimate rather than a physical measurement. However, some distresses like rut depth are commonly measured with simple instruments like the one shown in Figure 1.

The data collection forms for flexible pavement of the States of Washington, Texas and Ohio are summarized in Table 2. As Table 2 shows, there are different approaches used to define the density of a given distress type.

Severity

The degree of damage or point to which each particular distress has deteriorated must also be expressed in the condition survey procedure. There are many approaches and definitions used. Table 3 shows a sample of three state systems. The best approach seems to be a simple estimate rather than actual measurement due to the costs involved.

State Experience

There is considerable disparity in the level of evolution of surface condition evaluation procedures in the different states. Review of the Pavement Management Workshop (Ref. 3) held in Tumwater, Washington, in 1977 showed that Arizona, Florida Utah, and Washington have the most advanced methods for conducting condition surveys and evaluating pavement distress (Table 4). Although all states are capable of conducting condition surveys many (such as Kentucky, New York and Pennsylvania) do not conduct them on a routine basis as part of an overall pavement management system.

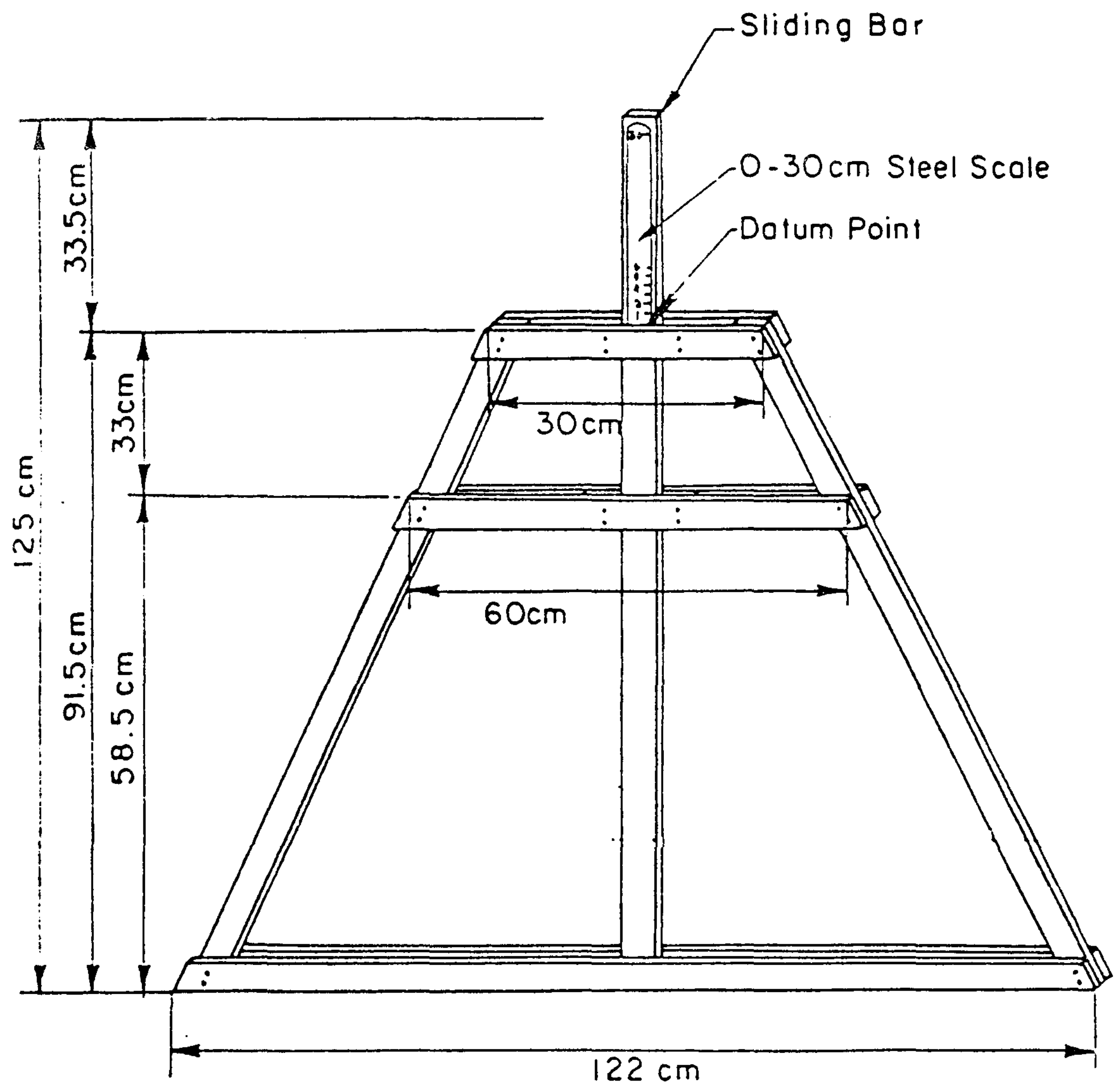


Fig 1. AASHO rut depth gauge.

Table 2. Density Rating Definitions and Levels in from Three Condition Survey Procedures

	Density Definition		
Distress	**Texas**	**Washington**	**Ohio**
Alligator Cracking	% of Area	% of Wheel Track Area/Station	Verbal
	1-5%	1-24%	occasional
	6-25%	25-49%	frequent
	+25%	75-100%	extensive
Rutting	% of Area	Measured	Verbal
	1-15%	average depth	occasional
	16-30%		frequent
	+30%		extensive
Transverse Cracking	Number per Station	Number per Station	Verbal
	1-4	1-4	occasional
	5-9	5-9	frequent
	10+	10+	extensive
Longitudinal Cracking	Linear Feet per Station	Lineal Feet per Station	Verbal
	10-99	1-99	occasional
	100-199	100-199	frequent
	+200	+200	extensive

Table 3. Severity Rating Definitions in Three Condition Survey Procedures

	Severity Definition		
Distress	Texas	Washington	Ohio
Alligator Cracking	Verbal	Verbal	Verbal
	slight	hairline	low
	moderate	spalling	medium
	severe	spalling	high
Rutting	Verbal	Average Width, in.	Verbal
	slight	1/4 - 1/2 in.	low
	moderate	1/2 - 3/4 in.	medium
	severe	over 3/4 in.	high
Transverse Cracking	Verbal	Average Width, in.	Verbal
	slight	1/8 - 1/4 in.	low
	moderate	+ 1/4 in.	medium
	severe	spalled	high
Longitudinal Cracking	Verbal	Average Width, in.	Verbal
	slight	1/8 - 1/4 in.	low
	moderate	1/4 in.	medium
	severe	spalled	high

Table 4. Comparison of Surface Condition Evaluation (after Ref. 3).

State	Comments
Arizona	Crack survey is primary evaluation. Compared to standard photos. Other distress parameters determined to be too time consuming 1000 sq. ft. of each 1/3 mile evaluated annually.
California	Structural defects such as cracking, rutting, etc. rated for extent and severity similar to Washington. Annually for interstate, 3-4 years min. of others. Different type rating for flexible and rigid pavements.
Florida	Structural defects including rutting, cracking and patching are rated for 100 ft. as representative of 1 mi. sections. Determine defect rating (DR) as part of overall.
Kentucky	Use surface condition rating as feedback for design deficiencies rather than routine monitoring.
New York	None made routinely.
Ontario	Pavement Condition Rating, PCR determined by rater as set forth in manuals. One to two year frequency. Ride and distress combined to determine Distress Index, DI.
Pennsylvania	Not made at present.
Saskatchewan	Annual surface condition rating on selected projects only.
Texas	Structural defects measured objectively based on visual rating. Vehicle mounted camera provides basis for distress rating on candidate project only.
Utah	Detailed evaluation of cracking, rutting, patching, wear, weathering, etc. on 500 ft. of 1 mi. sections made from photologging. Both subjective and objective analysis. Eleven parameters included.
Washington	Structural defects such as cracking and rutting measured every other year on a subjective basis. 200 ft. section within each 1 mi. section.

Pavement surface condition surveys are sometimes confused with structural capacity evaluations. For instance, the terms structural condition and structural capacity are used interchangeably and in other cases deflection is used as measure of surface condition. There are two probable reasons for this problem with confounding; one is that structural condition and capacity are not adequately defined. Another is that the two are often combined when there is little available data. For the purposes of this report, however, surface condition will relate only to measurable or otherwise observable distress in the pavement surface layer. Structural capacity, on the other hand, will relate to the pavement's load-carrying capacity, and will be covered in another technical memorandum. Although the two can be considered as separate pavement fitness measures, they do interact with each other and this interaction should be considered.

Lytton, et. al., (Ref 2) summarized the maximum percent that visual condition survey distress factors effected the overall pavement performance rating for various agencies. As Table 5 shows the states are probably grouped into the following broad categories:

% Effect	Comment
70-100%	States using only visual condition surveys at this time for PMS (example, Maine)
40-69%	States using visual condition surveys in combination with roughness measurements for PMS (example, Washington)
1-39%	States using measurements in all four primary area such as deflection, condition survey, roughness, and skid resistance measurements (example, Arizona)

Table 5. Maximum Percent Distress Factors Influence Overall Rating by Agency* (after Ref 2).

		Flexible Pavements	Rigid Pavements
1.	Arizona	17.0	17.0
2.	California	78.3	--
3.	Florida	50.0	--
4.	Georgia	37.5	--
5.	Indiana	22.0	22.0
6.	Kansas	44.0	50.0
7.	Louisiana	30.0	30.0
8.	Maine	100.0	--
9.	Maryland	40.0	40.0
10.	Minnesota	50.0	50.0
11.	Nebraska	40.0	--
12.	New Mexico	40.0	40.0
13.	North Dakota	75.5	--
14.	Tennessee	50.0	50.0
15.	Texas	80.4	85.7
16.	Virginia	48.0	42.0
17.	Washington	50.0	50.0

*In general, the table does not utilize distress measured by ride meters in the computation of percentages.

-- Indicates one of two items: The agency does not use a rating system for rigid pavements or distress factors are not numerically weighted.

Table 6 lists the distresses which various states rated at the time of Lytton's study. Figures 2 and 3 (Refs 4 and 5) contain a comprehensive list of all the distress manifestations commonly rated for flexible and rigid pavement respectively.

U.S. Army Corps of Engineers Condition Rating Procedure

Shahin and Kohn of U.S. Army Corps of Engineers' Construction Engineering Research Laboratory (CERL) have developed a procedure (Ref. 6) which represents the present state-of-the-art in rating pavement surface condition. It has many of the attributes of the more advanced procedures used by the progressive states pointed out earlier.

The CERL procedure is similar to the procedures used by Florida and Washington (and probably other states) in that deduct values are assigned to certain observed distress types, according to their extent and severity. and then subtracted from a perfect score to give PCI (Pavement Condition Index) and the pavement rating. The procedure basically consists of six steps which are reproduced below (for an individual sample unit) and illustrated in Figure 4.

Step 1 - Each sample unit (of pavement) is inspected and distress data recorded. (Distress data includes observed distress types and their severity and density).

Step 2 - Determine deduct values using deduct curves for each distress type and severity. (Longitudinal and transverse cracking, a, and alligator cracking are illustrated in Figure 4).

Step 3 - Compute total deduct value, TDV, by summing all individual deduct values.

Step 4 - Determine corrected deduct value, CDV, for multiple distress combinations.

Table 6. Maximum Percent Individual Distress Factors Influence Pavement Rating for Each Agency (after Ref. 2).

	Flexible Pavements												Rigid Pavements								
	General	Cracking	Rutting	Raveling	Deflection	Corrugations	Patching	Pitting	Distortion	Shoving	Pot Holes	Flushing	General	Cracking	Faulting	Patching	Blow Ups	Spalling	Undulations	Scaling	Pumping
Arizona	17.0												17.0								
California		41.0	11.7	14.0			5.9														
Florida		30.0	13.3				6.7														
Georgia					37.5																
Indiana	22.0												22.0								
Kansas	4.0	18.0	12.0				10.0						4.0	24.0	4.0			4.0	10.0	4.0	
Louisiana	30.0												30.0								
Maine	9.0	44.0	14.0			5.0		14.0	14.0												
Maryland	40.0												40.0								
Minnesota		27.5	7.5				15.0							15.0	5.0	15.0		12.5		2.5	
Nebraska	20.0				20.0																
New Mexico	40.0												40.0								
North Dakota		49.0	10.2	4.1						6.1	6.1										
Tennessee	>50												>50								
Texas		27.4	5.9	7.8		7.8	7.8				15.7	7.8		24.1	9.2	4.6	9.2	13.8		13.8	13.8
Virginia		35.0	5.0	1.0					2.0			5.0		7.0	10.0			25.0			
Washington,		18.1	11.1			2.7	4.2		11.1		2.7			9.8	5.9	2.9	2.9	9.8		9.8	8.8
King County, Washington		12.5	6.3	6.3		3.1			3.1	3.1		3.1									
Avg. Among Agencies Using the Distress Factor	25.8	30.2	9.7	6.8	28.8	4.6	8.3	14.0	7.6	4.6	8.2	5.3	29.0	16.0	6.8	7.5	6.0	13.0	10.0	7.5	11.3
Avg. Among Agencies Using Flexible (18) or Rigid (12) Rating Systems	12.9	16.8	5.4	1.9	3.2	1.0	2.8	0.8	1.7	0.5	1.4	0.9	16.9	6.7	2.8	1.9	1.0	5.4	0.8	2.5	1.9

PAVEMENT DISTRESS MANIFESTATION	EVALUATION		
	SEVERITY	DENSITY	OTHER CHARACTERISTIC
SURFACE DEFECTS			
● COARSE AGGREGATE LOSS			
● RAVELLING			
● FLUSHING			
SURFACE DEFORMATION			
● RIPPLING			
● SHOVING			
● WHEEL TRACK RUTTING			
● DISTORTION			
CRACKING			
● LONGITUDINAL WHEEL TRACK			
● MIDLANE			
● CENTER LINE			
● PAVEMENT EDGE			
● MEANDERING			
● TRANSVERSE			
● ALLIGATOR			
● RANDOM			
● SLIPPAGE			
● OTHER			
MAINTENANCE PATCHING			
● SPRAY			
● SKIN			
● HOT-MIX			

Figure 2. List of Flexible Pavement Distress Manifestations (After Ref. 4).

PAVEMENT DISTRESS MANIFESTATION	EVALUATION		
	SEVERITY	DENSITY	OTHER CHARACTERISTIC
SURFACE DEFECTS • Polishing • Loss of Coarse Aggregates • Pot Holes • Scaling • Ravelling			
SURFACE DEFORMATION • Faulting (Stepping) • Settlement (Sagging)			
JOINT DEFICIENCIES • Joint Creeping • Joint Sealant Loss • Joint Spalling • Joint Failures			
CRACKING • Longitudinal • Meandering • Corner • D • Transverse • Diagonal • Edge Crescent • Miscellaneous			
MISCELLANEOUS DISTRESSES • Lane Separation • Slab Warping • Wheel Track Wear			
MAINTENANCE • Full Width Joint Repair (With Concrete or Asphalt) • Full Depth Pressure Relief Joint • Precast Slab • Coldmix Patching • Hot Mix Patching	▨		▨

Figure 3. List of Rigid Pavement Distress Manifestations (After Ref. 5).

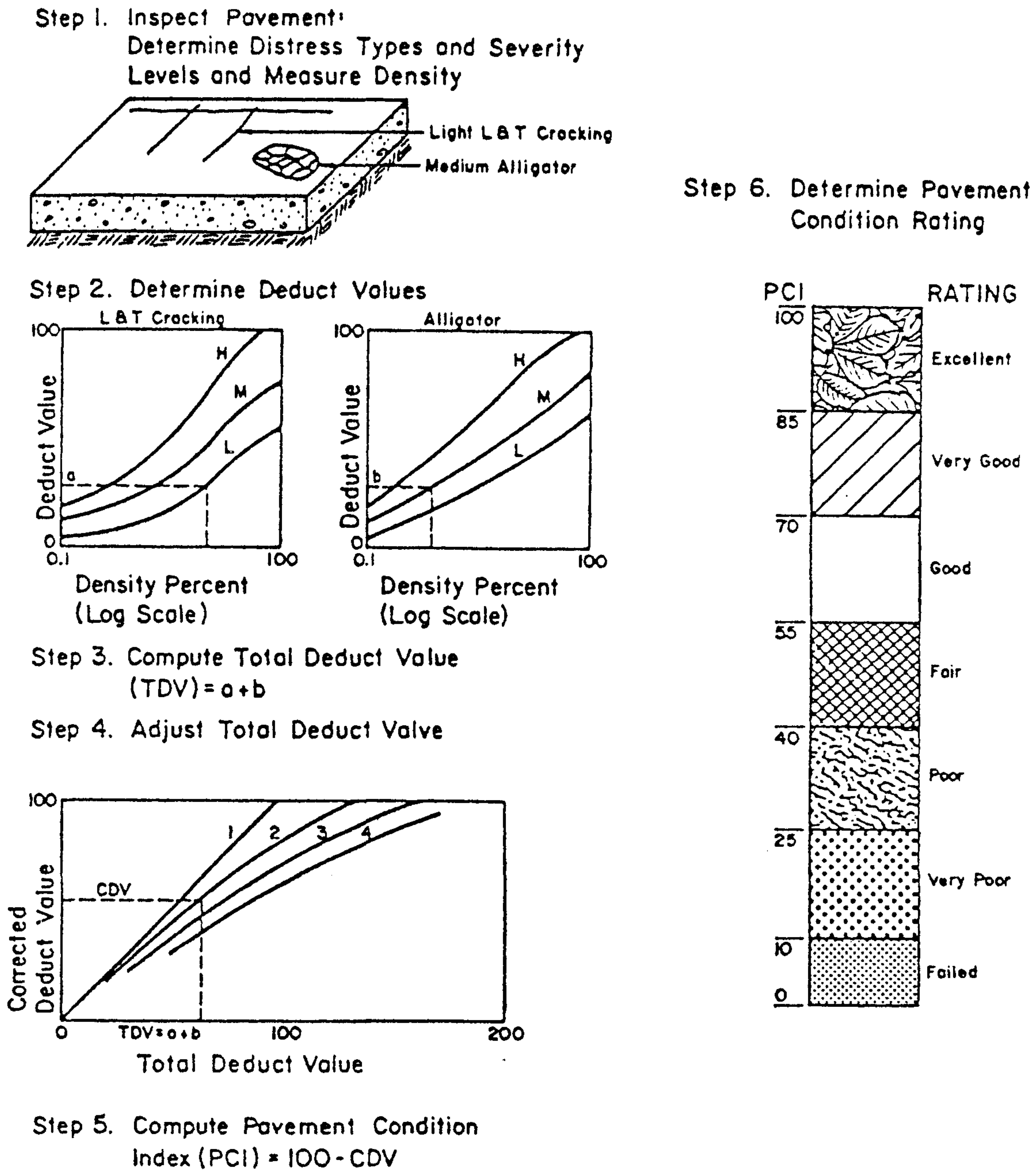


Figure 4. Steps for calculating CERL pavement condition index, PCI (after Ref. 7).

Step 5 - Compute PCI = 100-CDV.

Step 6 - Determine pavement condition rating according to level of PCI.

In order to reduce the inherent variation in the subjective data obtained Shahin and Kohn also provide a supplementary report (Ref. 7) which helps in the identification and classification of pertinent distress information. The report contains many photographs which illustrate the severities of the different distresses. Other states. such as Washington and Florida, use a similar technique as an aid in training field crews in the collection of distress data.

DATA COLLECTION ISSUES

There are several issues in the determination of the best approach and procedures to implement, including the following topics: 1) which distresses to select for evaluation, 2) automated versus non-automated data collection, 3) total network versus sampling measurements, and 4) decentralized versus centralized condition survey crews. These are the same issues which face every agency and, therefore some experience exists in each of these area.

Selection of Which Distresses to Evaluate

As previously pointed out, there have been many attempts to organize distress terminology and descriptions; however, local conditions affected by provincial terminology, material types, design characteristics, construction procedures and the effects of climate contribute to make a common understanding or "universal system" most difficult. If time and money are spent to physically measure pavement distress, many states use the AASHO Road Test definitions, so that the information can be used in Present Serviceability Index (PSI) calculations as follows:

Rut depth, inches
Cracking and Patching, SF/1000SF

These distresses are also the most widely evaluated even when physical measurements are not made; however, as indicated by Tables 2 and 3, the severity and density specifications vary widely. Arizona, for example, initially measured the percent area of pavement cracked in 100 SF per mile post (Ref 8), but upon finding this process too laborious, switched to their current crack rating guide. This process simply requires the rater to compare the pavement with a series of photos to determine the percent cracking. The rater interpolates between the photos to an accuracy of the nearest whole percent if cracking is less than ten percent and to the nearest five percent if cracking is greater than ten percent. Photos are given for 2.5, 7.4, 11, 35, and 64 percents of cracking.

A recent report from Alaska indicates some of the justifications for limiting the number of distresses rated (Ref 9);

1. Some distresses are hard to quantify.
   (Example, raveling, plucking, longitudinal cracks, thermal cracking, shoving, bleeding, etc.).
2. Some distresses were rare in Alaska.
   (Example, bleeding and shoving).
3. Some distresses were easy to quantify, but time consuming.
   (Example, counting potholes).

The recent selection of variables in the pavement evaluation system (PES) of Texas (Ref 10,) as shown below, is also limited when compared with Figures 2 and 3;

1. For flexible pavements: longitudinal and transverse cracking, alligator cracking, rutting, raveling, patches, and flushing; and
2. For rigid pavements: punchouts, patches, minor and severe spalling, minor and severe pumping.

Automated versus Nonautomated Data Collection

Automated methods have been developed for the collection of surface condition information.

Photologging. The most commonly used equipment is photologging equipment made by Tech-West in British Columbia. This device produces a photographic record of the existing condition of the road. It also measures other features such as cross slope. Several agencies have implemented these devices and reported the following problems;

1. Undesirable subjectivity in surveys due to present techniques and/or human factors, i.e., photos must be interpreted to produce a rating for the pavement section. In addition, the angle of the sun (i.e., time of day), season, temperature, and presence of water all influence the ability of the camera.
2. Expense of equipment purchase and maintenance.
3. Feelings that the engineer must "walk the road" to get an accurate evaluation for the distress present.
4. Expense of view equipment.

In spite of these problems, this equipment has the following advantages;

1. Can operate in traffic stream without blocking or stopping flow as is the case with physical measurements.
2. Can adequately record photos and information in correlation with distance measurement instrument thereby insuring correct location orientation.
3. Can reduce field labor costs in the long run.
4. Can probably provide adequate detail for a first evaluation at the network level. (For example, Arizona simply uses a percentage estimate of the area cracked).
5. Provides a permanent record. which is more explicit than just numbers on a data form.

Microprocessing. The only automated device for collecting visual distress estimations, cross slope, horizontal curvature, vertical curvature, etc., is the ARAN, Automated Road Analyzer, made by the same company who manufacturers the Dynaflect. This device has a microprocessor on board into which the operator can key in varying types of distress to be stored. This device has not been purchased by a state agency, although Nevada currently has one on order.

Non-Automated Procedures. Non-automated data collection procedures vary from windshield surveys where the driver and rater or raters do not stop, to set procedures where sample segments of the road are walked on foot while a form is filled out. Windshield surveys are very gross at best because of the inability to estimate properly all the items in one pass, and factors such as fatigue, sun angle, etc. Walking surveys, on the other hand, are time consuming and costly, although they provide a higher quality of data. Certain problems have been noted based on the experience various agencies have had with their visual condition surveys (Ref 2);

1. Undesirable subjectivity in surveys due to present techniques and/or human factors,
2. Absence of valid, workable statistical sampling procedures for highway surveys,
3. Inadequate delineation of established survey areas for repetitive survey purposes,
4. Lack of uniformity in severity weighting techniques for distress types,
5. Inability with current data storage and retrieval methods to achieve a valid and workable inventory of pavement condition, and
6. Hazardous and disruptive nature of condition surveys, as currently conducted.

In spite of the above mentioned problems with condition surveys, due to the cost of photologging or more sophisticated equipment such as the ARAN, most highway agencies have adopted visual condition survey procedures.

Total Network Monitoring versus Sampling

There are advantages to be gained by the highway agency if the question of sampling can be addressed and used. It is, however, usually necessary to make a complete network survey in the beginning to establish the base line condition of the network.

There are ways to reduce the impact of preparing this initial data base. For example, a windshield survey or general classification scheme using Photologger data may be used to select only a percentage of the network for more detailed data collection.

The following ideas were presented by CALTRANS (Ref 11) with respect to the the sampling versus non-sampling question:

> "This PMS is oriented toward decision making at both the program and the project level. In order to address project level decisions and make comparative judgements between specific projects total system survey coverage is required. Consequently, statistical sampling of the survey was dismissed.
>
> CALTRANS conducts a pavement condition survey each two years. Every lane of every mile (47,000 miles) of the entire state highway system is surveyed.
>
> The 1975-76 pavement condition survey data collection and machine processing cost approximately $362,000. Consideration was given to the possibility of reducing the cost by eliminating certain select portions of the State Highway System from alternate surveys, in effect, surveying portions of the system on a four year frequency instead of the present biennial basis.
>
> Candidates considered for less frequent surveys were:
>
> 1. Sections with very minor problems in the last survey. Results of the four biennial pavement condition surveys made since 1969 have been reviewed. Many sections of pavement with minor problems deteriorated within a two year period to a condition which could trigger remedial maintenance or rehabilitation.
>
> If these sections with good ratings were evaluated on a four year cycle, it is probable that many would deteriorate before detection. Identification of progressive pavement deterioration at an early point in time can allow remedial maintenance to extend the life of pavement serviceability resulting in a more cost effective approach.
>
> 2. Sections that have been resurfaced or rehabilitated subsequent to the last pavement condition survey.
>
> The monitoring and evaluation of recently completed projects is important in determining the progressive deterioration rate of pavement and the development of performance information.
>
> 3. Sections which are in geographic or environmental areas where pavement deterioration is very gradual.
>
> Investigations to date have not established a correlation between pavement deterioration rates and environmental conditions. Until such a correlation is established, there is no basis on which to reduce survey frequency.
>
> 4. Sections which experience low traffic volumes.

> Investigations to data have not established a direct correlation or trend relating ADT and pavement deterioration rate. This applies to both structural condition and ride quality ratings.
>
> 5. Sections which are not of statewide significance.
>
> Routes of statewide significance include Interstate, Rural Principal Arterials, and Urban Connecting Lines. Many high volume urban freeways are not included in the above category and need to be evaluated concurrently with the rest of the State Highway System. There are several thousand miles of non-statewide significant routes in which it is essential to protect the existing investment and insure the application of cost effective repair strategies in order to maximize pavement serviceability.
>
> 6. Conclusions
>
> Considering the cost and time required to collect and process survey data for California's State Highway System, it is deemed infeasible to survey annually. It is also generally inappropriate to survey at three or four year intervals considering the deterioration rates of some pavement types as discussed in Section IV of this report. A two year frequency seems most appropriate for uniform data collection, supplemented by other surveys for special studies or unique problems.
>
> Following full implementation of a PMS in California and the development of an adequate historical data base for performance and trend analyses, consideration will be given to the adviseability of reducing the extent and frequency of condition surveys. There is currently insufficient historical information to take the risk."

To overcome some of the problems expressed by California, an approach being considered in Texas is stratified sampling (Ref 10). For stratified sampling, the parent population of roads is divided into mutually exclusive and exhaustive subsets based on important characteristic differences such as classification, environmental zone, traffic level, age, etc. A random sample of elements is chosen independently from each subset for monitoring.

The subsets into which the population of the highway sections are divided are called strata or subpopulations. To be mutually exclusive and exhaustive, every population element must be assigned to one and only one strata and no population elements are omitted in the assignment procedure. For example, suppose that the parent population is composed of the total number of pavement segments within a district. For stratified sampling, these highway sections are divided into various strata on the basis of functional class, from which random samples may be drawn independently. A detailed stratification of the highway sections may be advantageous for investigating the characateristics of particular strata. The division of sections should be selected in such a way that the variability within each strata is minimized in order to obtain acceptable results. This may involve consideration of such factors as traffic, enviromment, pavement type, and functional class. In addition, a decision may have to be made on whether to make a proportionate or a disproportionate stratification.

In a proportionate stratified sample, the number of observations in the total sample is allocated among the strata in proportion to the relative number of elements (or in this case mileage) in each strata in the population. For example, a strata containing one-tenth of all the population elements would account for one-tenth of the total sample observations.

Disproportionate stratified sampling requires that the variances of the individual strata be taken into account when allocating the sample observations among strata. With a fixed sample size, strata exhibiting more variability are sampled more, and conversely those strata that are very homogeneous are sampled less than proportionately. This scheme will produce more efficient estimates, but it requires information on strata variances which may or may not be available. For condition survey use, therefore, the use of disproportionate stratified sampling would perhaps require that variances of pavement condition scores for each of the functional classes be known. It is possible in such cases for estimates to be made from past records, and the experience and judgment of the RIDOT engineers would certainly be useful.

Decentralized versus Centralized Pavement Condition Surveys

Centralized pavement condition survey teams are used by some highway agencies to insure consistency of the raters in evaluating and estimating distress. The number of raters is minimized in this way as well as some of the rating variations. Offsetting problems are the cost of mobilization and subsistence in a large state as well as the time required for a centralized crew to collect the data.

Decentralized crews are often more familiar with the local area and the relative condition of pavements in their network. This may allow for a better rating if steps are followed to insure that decentralized crews have all been trained. In addition, decentralized crews allow for the collection of networkwide information quicker due to their ability to work simultaneously.

CALTRANS reported the following experience on costs and features of each approach (Ref 11):

"1. Decentralized Survey - Current Practice

> The pavement condition survey is conducted every two years using Transportation District personnel, trained in headquarters, who survey all lane miles of flexible and rigid pavements. The survey is completed in all districts within a six months time frame. Recent statewide pavement condition survey costs are estimated at:
> 75-76 18 man years; $362,000 (including EDP costs)
> 77-78 12 man years; $335,000 (including EDP costs)
> Survey operations have been in a state of refinement as the PMS program developed. Future surveys will be more routine in nature. For example, the major task of bridge and roadway section identification has been accomplished, and need not be repeated. Routine operation should reduce 1979-80 survey costs by an estimated 20% from the 1977-78 survey costs.
>
> 2. Centralized Survey
>
> Consideration has been given to revising the manner in which the present pavement condition surveys are conducted. A continuous survey conducted over a two year period with a limited staff of pavement raters from headquarters has some attractive features.
>
> The limited staff would require much less training effort and more consistent survey results could be expected. The continuous survey approach would partially eliminate some district manpower and bugeting difficulties, particularly where district priorities interfere with assigning experienced raters to the periodic surveys. Surveillance frequency could be more easily adjusted for specific segments of highway, resulting in more efficient use of raters. On the other hand, it may prove difficult to retain a headquaters rating staff who would be amenable to travel throughout the state on a continuous basis.
>
> It is estimated that a 1979-80 centralized survey approach would require about nine man years and cost on the order of $315,000, including EDP costs.

The present decentralized pavement condition survey presents a complete picture of the state of deterioration of highways based on information gathered within a six month time span. A centralized survey could be conducted on a continuous basis to provide new information on a two year frequency. Various areas of the state would be examined at substantially different times. There may be no valid comparison between a December 1978 pavement in Los Angeles and an April 1980 pavement in Eureka, yet both pavements may be in competition for the same funding for rehabilitation.

Pavement condition surveys should be conducted following the most damaging season of the year, before problems are hidden by maintenance operations. In California, this would be in late Spring. The centralized survey would preclude this since it would have to be conducted on a year around basis due to the limited staffing approach.

Work load reductions due to reduced length of roadway being sampled, improved measuring equipment, and standardization of data collection operations will result in lower costs for either type survey. These factors, therefore, should not be determinants on the issue of a centralized or decentralized survey.

The primary purpose of PMS is to improve the decision making process as it relates to the establishment of pavement repair program levels, project priorities and cost effective repair measures. Although the total cost of the biennial pavement condition surveys are appreciable, it must be considered in light of the current annual funding level of over $90 million for California's pavement reconstruction, resurfacing and maintenance programs.

The decision to centralize or decentralize pavement condition surveys should not be made solely on an economic basis. The cost differential is not as significant as some of the other factors discussed above. Following the full implementation of PMS in California and the develoment of a strong data base consideration will again be given to the best course of action to follow in future surveys."

USE OF CONDITION SURVEY DATA IN A PMS

One of the problems with relating observed distress to condition ratings is the selection of approximating weighting factors for the different types of distress and their severity and extent. Obviously, some types of distress are more critical than others in their influence on all the different pavement fitness measures, including surface condition. The problem is to determine how much more critical. In the more advanced procedures, the weighting values were estimated using regression analysis and trial and error. Unfortunately, many seasonal and regional factors come into play such that these weighting values are not the same from one state to the next. This was illustrated in Table 6, prepared by Lytton (Ref 2) in a study of pavement evaluation and evaluation of equipment. As can be seen, the states are not consistent on what they consider to be the most influential distress factors on pavement rating. Apparently, these environmental and/or regional factors can have a positive or negative influence on the development of certain types of distress so that frequency of distress occurrence is as important (in determining weighting values) as severity and extent. In other words, the states tend to give more weight to the types of distress that their maintenance crews encounter the most.

Most states use a single numeric score such as that presented in Figure 1 to express the pavement condition. In general, this value is expressed on a scale of 0-100, with zero being the worst pavement and 100 being a pavement with no visible distress. Table 7 shows the deduct values used in Texas for the different distress types observed in flexible pavements. These values apply to a 0-100 scale, and as can be seen, they are dependent on both the density and servity of distress.

Not all states base their ratings on a 0-100 scale, nor do they all use deduct values to arrive at a condition rating. Illinois, for example, uses a simple zero to nine scale where the programmatic definitions for different rangs of the scale are as follows:

Table 7. Deduct values for flexible pavement (After Ref. 25).

Type of Distress	Degrees of Distress	Extent or Amount of Distress		
		(1)	(2)	(3)
Rutting	Slight	0	2	5
	Moderate	5	7	10
	Severe	10	12	15
Raveling	Slight	5	8	10
	Moderate	10	12	15
	Severe	15	18	20
Flushing	Slight	5	8	10
	Moderate	10	12	15
	Severe	15	18	20
Corrugations	Slight	5	8	10
	Moderate	10	12	15
	Severe	15	18	20
Alligator Cracking	Slight	5	10	15
	Moderate	10	15	20
	Severe	15	20	25
Patching	Good	0	2	5
	Fair	5	7	10
	Poor	7	15	20

Deduct Points for Cracking

Longitudinal Cracking

	Sealed			Partially Sealed			Not Sealed		
	(1)	(2)	(3)	(1)	(2)	(3)	(1)	(2)	(3)
Slight	2	5	8	3	7	12	5	10	15
Moderate	5	8	10	7	12	15	10	15	20
Severe	8	10	15	12	15	20	15	20	25

Transverse Cracking

Slight	2	5	8	3	7	10	3	7	12
Moderate	5	8	10	7	10	15	7	12	15
Severe	8	10	15	10	15	20	12	15	20

Failures		20	30	40

Mays Meter	Deduct Points	50	40	30	20	10	5	0
	SI	2.4	2.7	2.9	3.1	3.3	3.5	4.7

less than 4.5 = critically deficient--pavement in need of immediate improvement
4.6 - 6.0 = approaching a condition that will likely necessitate improvement over the short term
6.1 - 7.5 = acceptable condition (low end) to good condition (high end)--not in need of improvement
7.6 - 9.0 = high quality condition

Table 8 presents the descriptions of the pavement surface corresponding to the different rating levels for bituminous overlays on both the non-Interstate and Interstate/Freeway system in Illinois.

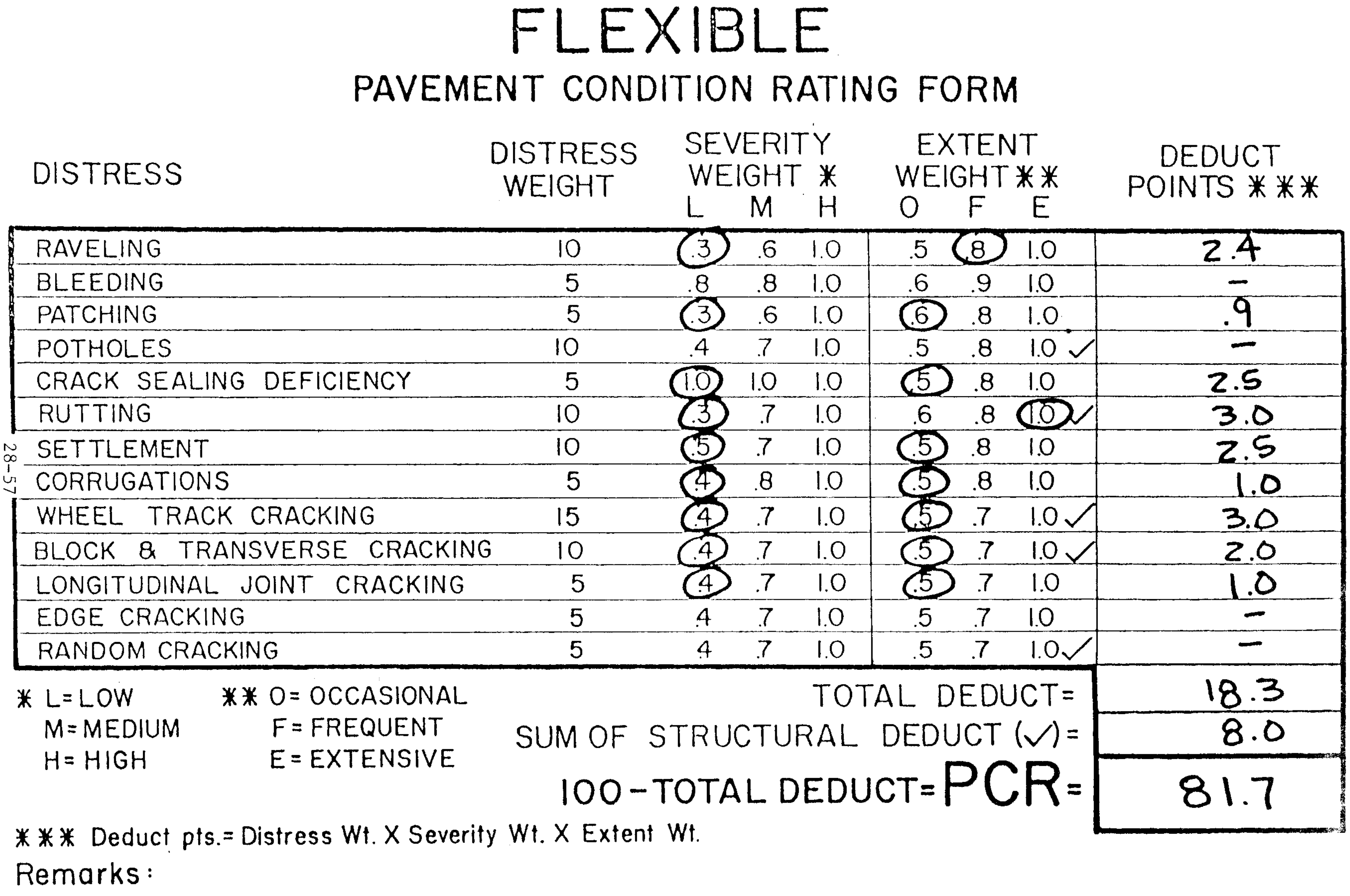
An example of a more detailed pavement condition rating procedure is that used by Ohio, where multiplicative weighting factors for severity and density are applied to a weighted distress measure to arrive at deduct values (or points) to subtract from a maximum score of 100. The procedure used by Ohio is illustrated in Figure 5.

In contrast to these somewhat similar rating procedures is the method adopted by California. They found that using the actual values for ride and extent and severity of pavement distress has simplified their approach in determining required rehabilitation. Furthermore, they believe that prior ratings systems which determine single numeric values for pavement condition do not provide unique condition information, nor do they permit interpretation of the observed pavement distress, or meaningful assessment of the pavement performance.

The conceptual pavement condition evaluation procedure used by California for flexible pavements is presented in Figure 6. Decision trees such as that shown in Figure 7 are used to evaluate pavement condition and identify problems and appropriate rehabilitation strategies. In this figure, the limiting values (or trigger values) directing the decision path are adjustable, thus providing flexibility for adjusting the levels of service.

Table 8. Illinois Pavement Condition Rating Score (After Ref. 12).

Rating	Description
9.0	Hairline cracks only and very few of these. Newly resurfaced pavement.
8.0	Beginning of reflecting widening crack near pavement edge--only narrow crack, however. Very slight reflection of transverse cracks, no more than 15-25 feet of reflective transverse cracking per 1000 square feet of pavement. (Frequency of initial reflective cracking is dependent upon distance between expansion joints of underlying pavement.) No indication that any serious problems from the underlying PCC have begun to surface.
7.0	The widening crack, if any, is definitely observable. Reflective cracks are more common, still narrow, with perhaps 50 lineal feet of cracking per 1000 square feet of pavement. Some minor rutting may be noticeable.
6.0	Reflective cracks are wider and there is some block cracking now observed. Minor patching is also possible; old blowups are probably showing up infrequently.
5.0	Many reflective cracks now show through. Block cracking is common and weathering is noticeable with some deteriorating effects to pavement. Around 100 lineal feet of cracking per 1000 square feet of pavement--actual amount of lineal cracking is possibly difficult to determine because of commonness of block cracking and lineal cracking. Reflective cracks may be slightly upheaved. There is some rutting over 0.5 inch deep. Alligator cracking is not uncommon.
4.0	Alligator cracking is more common as is the amount of patching. Rutting is noticeable but it may not be the sole factor responsible for this rating. Rutting may not be present and this rating would still apply. Cracks are wider and the degree of cracking in the wheelpath is probably extensive.
3.0	The majority of cracks are wide; severe disintegration is found at the pavement edge and over 200 or more lineal feet of cracking or a surface exhibiting block-alligator cracking is observed. The pavement is undoubtedly rough riding. Rutting should be common.
2.0	Pieces of surface have fallen out in many areas. The entire surface exhibits alligator cracking. The ride is very rough.
1.0	The entire surface is cracked and disintegrated. Traffic operation is severely affected. This condition will probably not occur on the marked State system.



FLEXIBLE

PAVEMENT CONDITION RATING FORM

DISTRESS	DISTRESS WEIGHT	SEVERITY WEIGHT *			EXTENT WEIGHT **			DEDUCT POINTS ***
		L	M	H	O	F	E	
RAVELING	10	[✓ .3]	.6	1.0	.5	[✓ .8]	1.0	2.4
BLEEDING	5	.8	.8	1.0	.6	.9	1.0	–
PATCHING	5	[✓ .3]	.6	1.0	[✓ .6]	.8	1.0	.9
POTHOLES	10	.4	.7	1.0	.5	.8	1.0 ✓	–
CRACK SEALING DEFICIENCY	5	[✓ 1.0]	1.0	1.0	[✓ .5]	.8	1.0	2.5
RUTTING	10	[✓ .3]	.7	1.0	.6	.8	[✓ 1.0] ✓	3.0
SETTLEMENT	10	[✓ .5]	.7	1.0	[✓ .5]	.8	1.0	2.5
CORRUGATIONS	5	[✓ .4]	.8	1.0	[✓ .5]	.8	1.0	1.0
WHEEL TRACK CRACKING	15	[✓ .4]	.7	1.0	[✓ .5]	.7	1.0 ✓	3.0
BLOCK & TRANSVERSE CRACKING	10	[✓ .4]	.7	1.0	[✓ .5]	.7	1.0 ✓	2.0
LONGITUDINAL JOINT CRACKING	5	[✓ .4]	.7	1.0	[✓ .5]	.7	1.0	1.0
EDGE CRACKING	5	.4	.7	1.0	.5	.7	1.0	–
RANDOM CRACKING	5	.4	.7	1.0	.5	.7	1.0 ✓	–

* L = LOW
M = MEDIUM
H = HIGH

** O = OCCASIONAL
F = FREQUENT
E = EXTENSIVE

TOTAL DEDUCT = 18.3
SUM OF STRUCTURAL DEDUCT (✓) = 8.0
100 – TOTAL DEDUCT = **PCR** = 81.7

*** Deduct pts. = Distress Wt. X Severity Wt. X Extent Wt.

Remarks:

Figure 5. Ohio flexible pavement condition rating form (After Ref. 13).

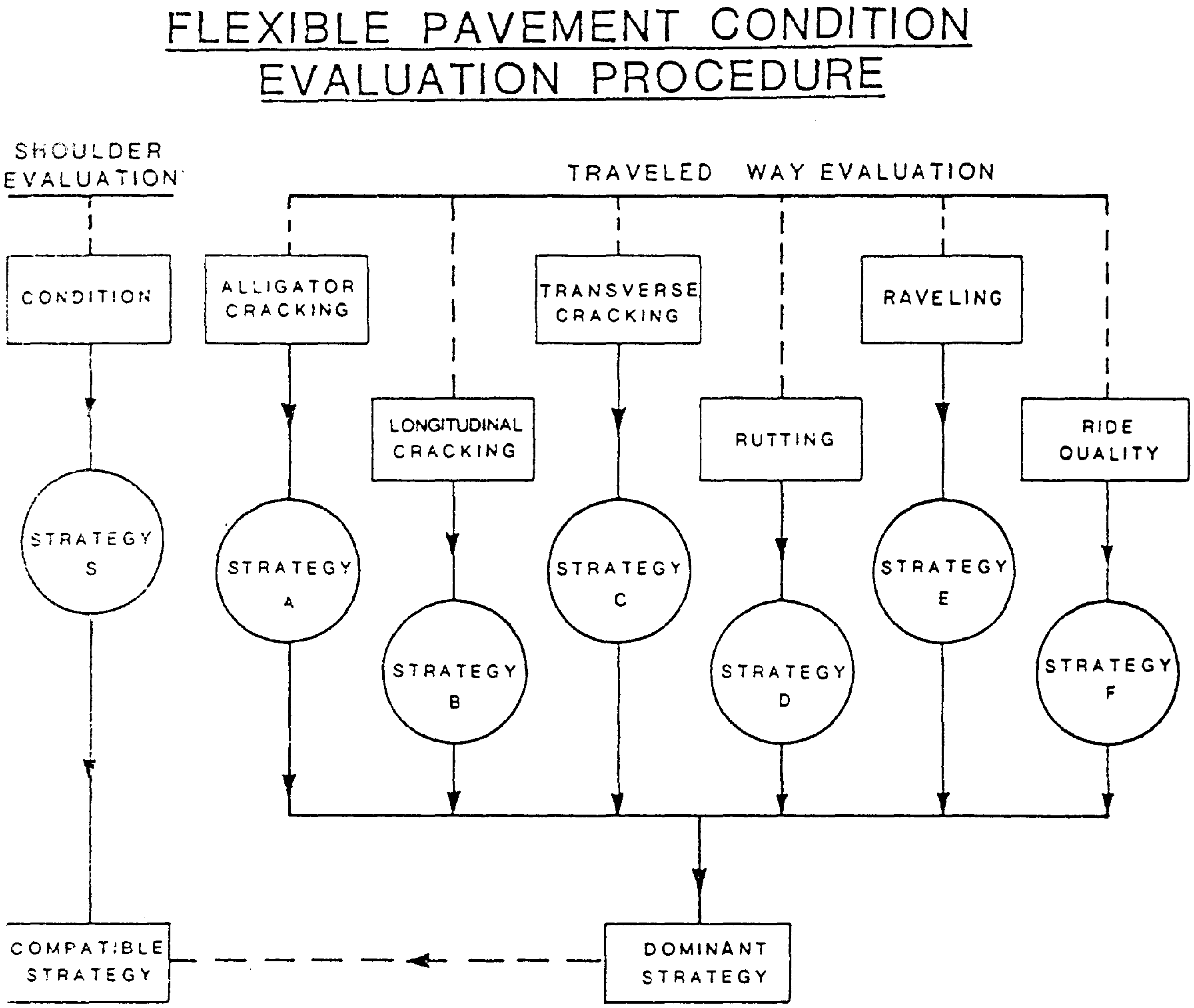


Figure 6. California flexible pavement condition evaluation procedure (After Ref. 11).

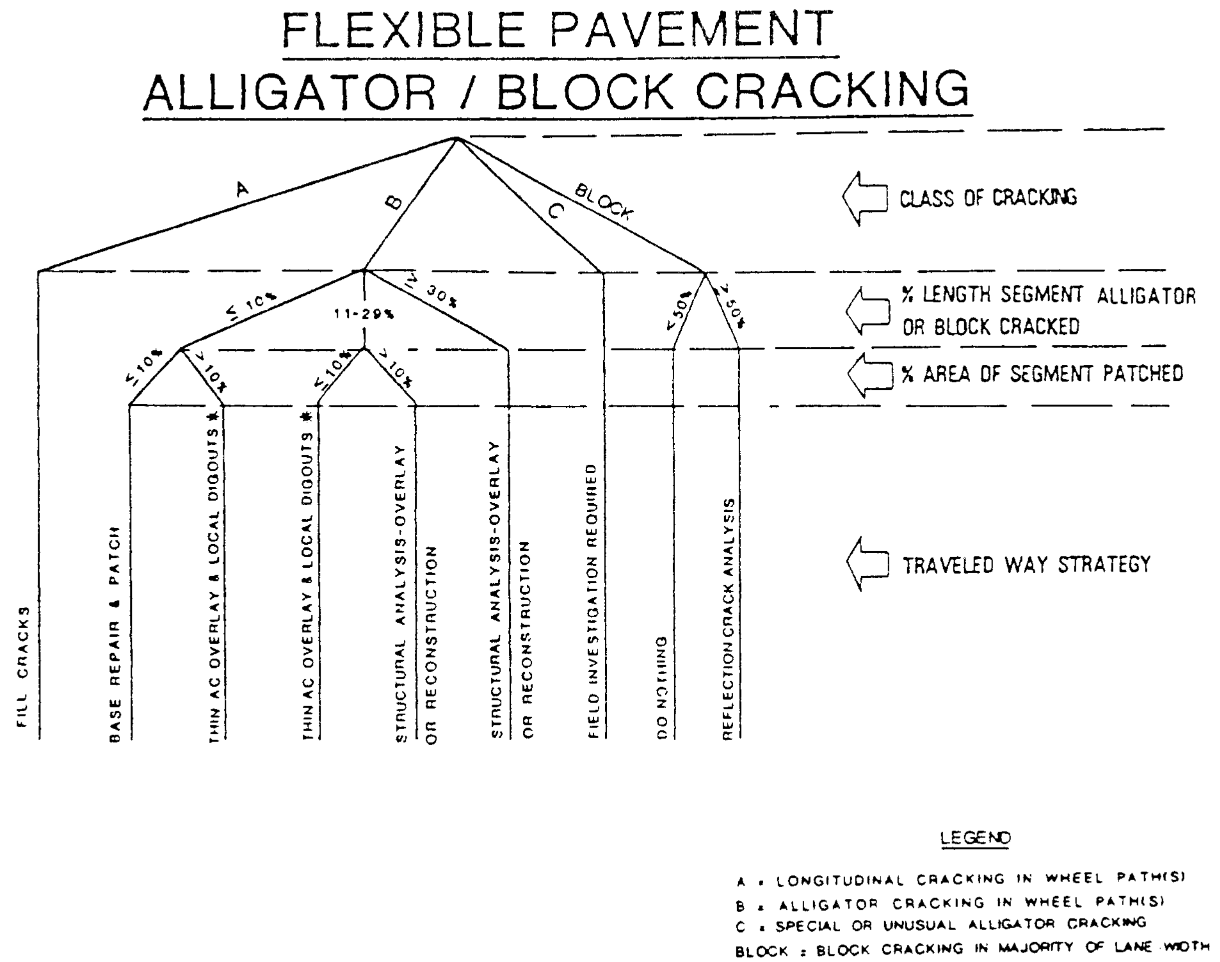


Figure 7. California PMS decision tree approach to using visual condition survey information and relating it to the appropriate rehabilitation strategy (After Ref. 11).

The amount of variability in subjective ratings has been reviewed by a number of agencies. Figure 8 shows the variation between a student and instructor for a second rating of some sections. The first rating produced even more variation. Although these types of variations can be expected, there is some evidence that the ratings are an adequate representation of the subjective feelings of field engineers. Figure 9 shows the results of Ohio ratings which were assigned and calculated.

Network Evaluation Models.

Some work has been accomplished at the network level in relating pavement condition rating scores to maintenance costs, current network rehabilitation needs, and predicting future needs. Current network needs and estimated costs can be calculated based on the first survey. Because the data base will contain pavements in all conditions from best to worst, preliminary models can be made to estimate future deterioration and, consequently, rehabilitation needs.

Current State-of-the-Art. Figure 10 shows some results from PMS work in Arizona, one of the states with a long time data base. Figure 10 indicates that the standard error in the estimated percent cracking increases over time; thus, greater error occurs with attempts to predict future cracking. However, even considering these problems, it was concluded in a recent Arizona report (Ref 26), "the cracking model in its present form represents a valuable tool to predicting cracking for individual miles of highway up to 20 years." Figure 11 shows the model used by Arizona for predicting the time to first cracking. Arizona has also been able to develop models to predict the change in the amount of cracking as a function of previous cracking and routine maintenance for newly constructed roads (Ref 16).

$$C_N = 0.198 + 0.56C_P + 0.05C_P^2 + 0.009R_G^2 + 0.04R_GC_0 - 0.0035C_0^2$$

in which:

C_0 = percent cracking in previous year

C_N = change in percent cracking in next year

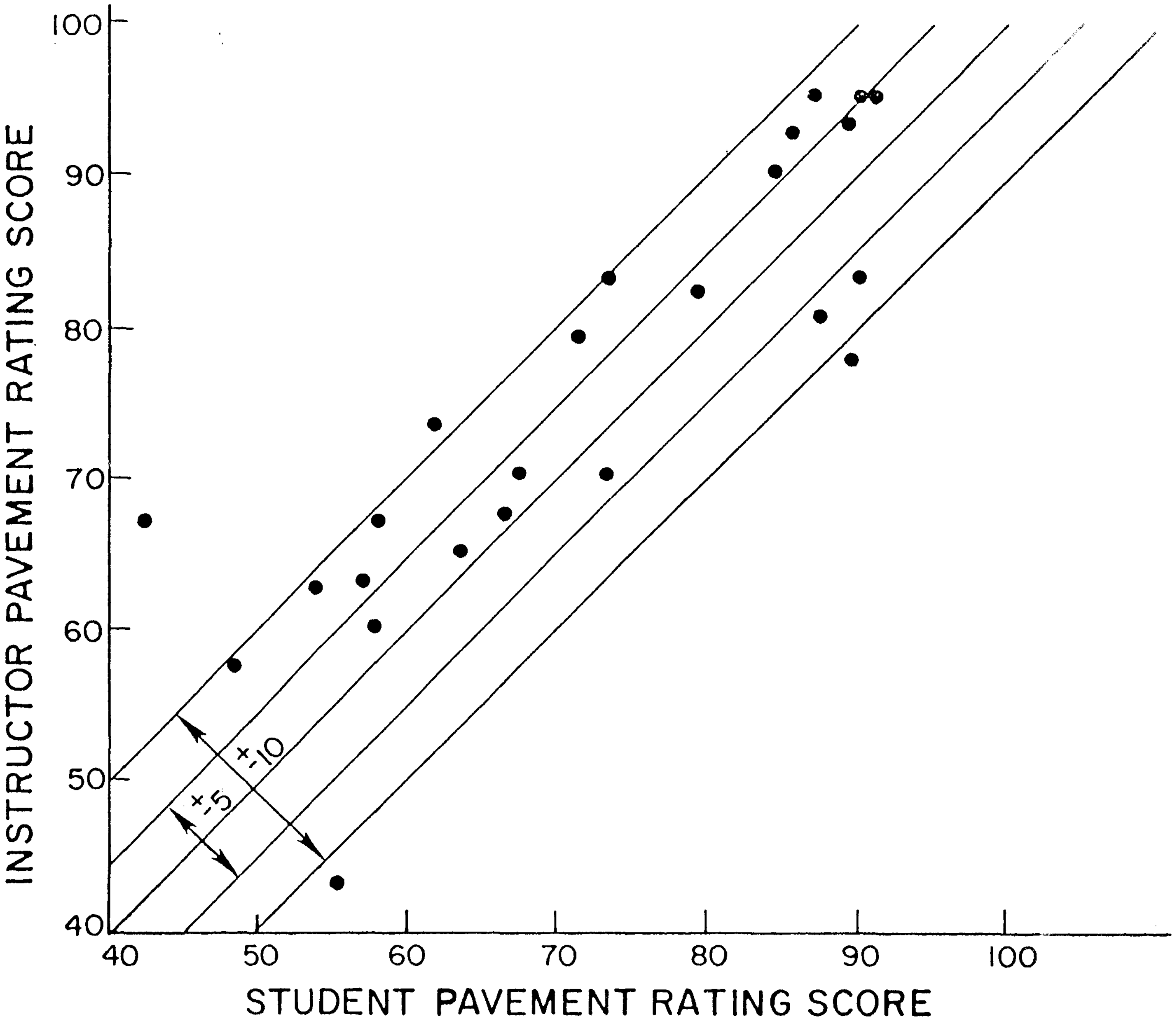


Figure 8. Comparison of instruction and student rating score - second rating (After Ref. 15).

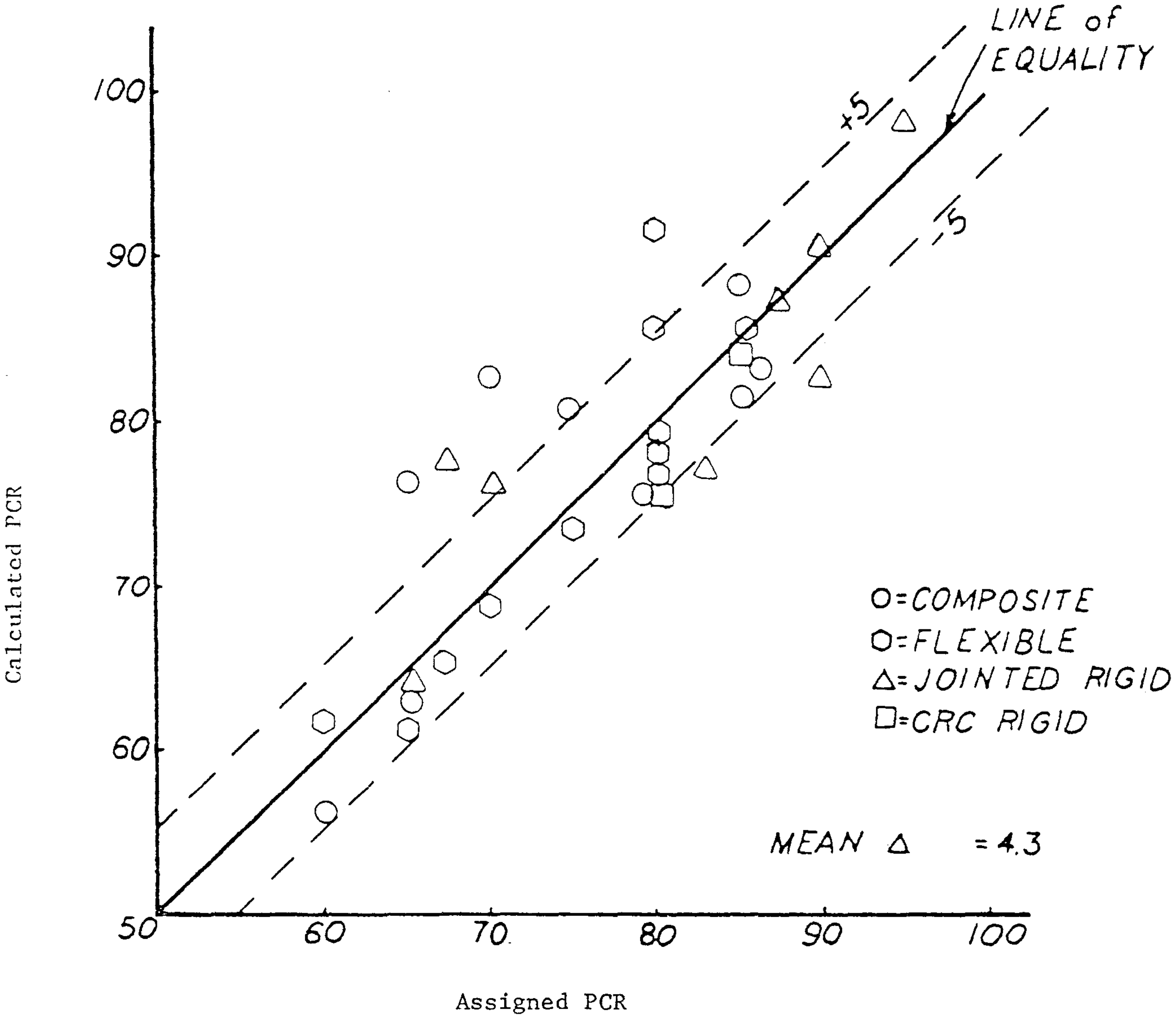


Figure 9. Suitability of PCR to reflect judgements of ODOT Engineers (After Ref. 13).

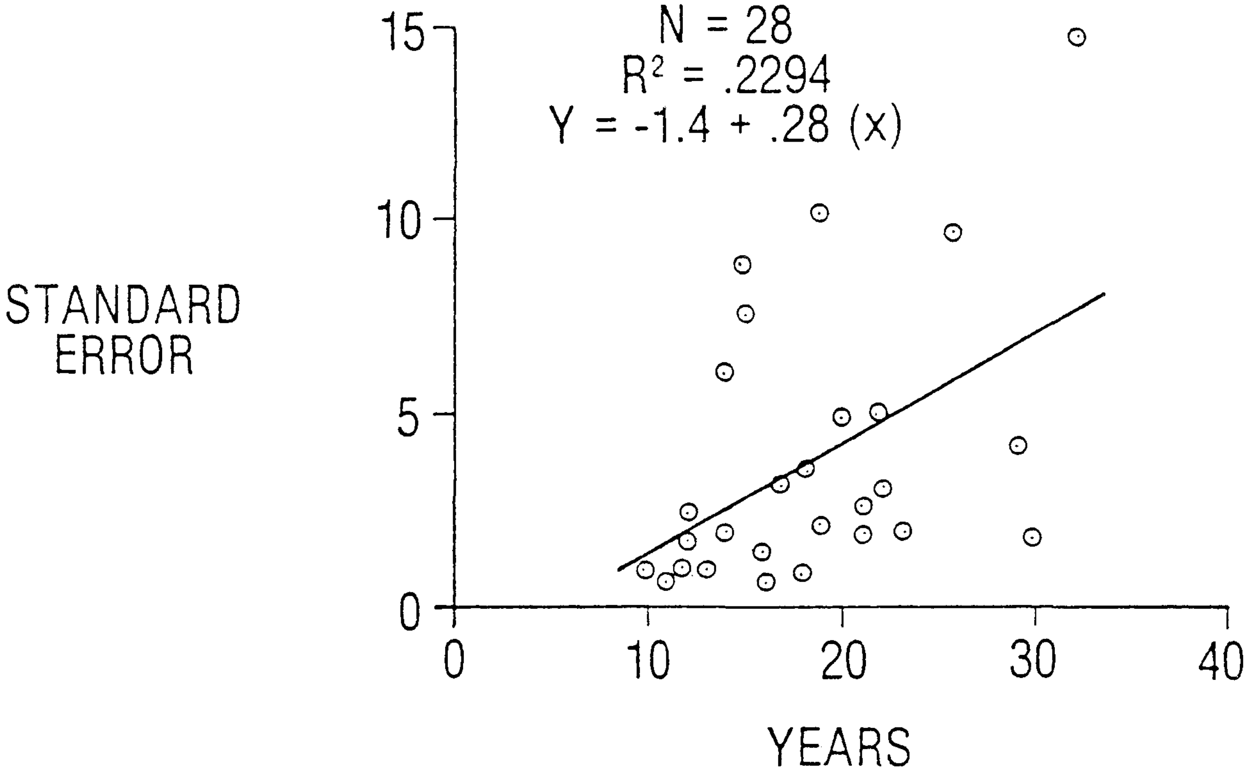


Figure 10. Standard error in the estimate of percent cracking as a function of time (After Ref. 26).

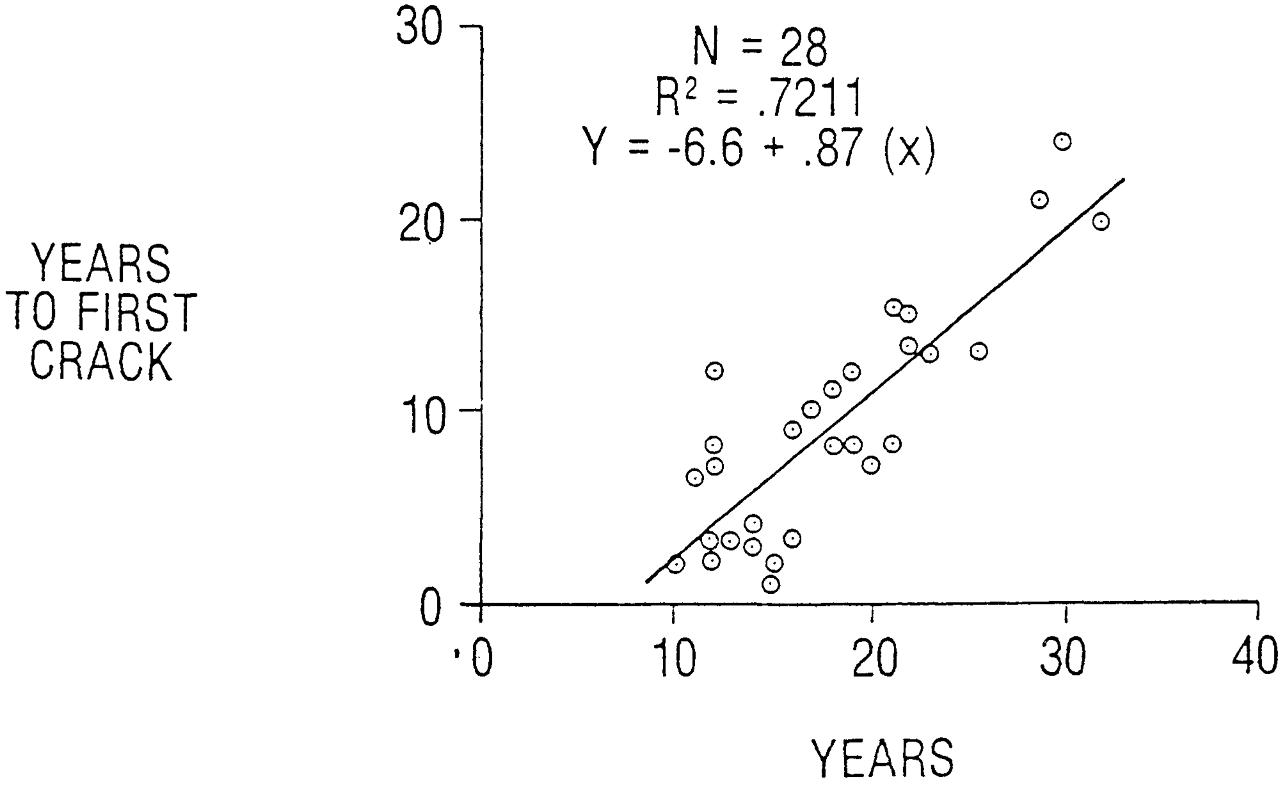


Figure 11. Time to First Cracks (After Ref. 26).

C_P = change in percent cracking in previous year

R_G = regional factor

Other states have developed similar models as shown by the Kentucky model given in Figure 12. These are examples of how pavement condition rating scores have been used separately in developing decision criteria.

Other states combine the condition survey rating scores with roughness, structural, and/or safety measurements to make "combined" scores. Washington combines its condition rating scores with roughness measurements to provide an overall pavement rating score as follows (Ref 3);

$$0 = D\ (1 - R/10)^{.5}$$

where

0 = overall final pavement rating

R = pavement ride score from roughness measurements

D = distress rating

= 100 - summation (defect deduction values)

As cited earlier, California simply uses the field data to directly estimate needs based on the particular distresses present, not a combined score. As can be seen, there are many approaches and uses for the network level condition rating information.

Some recent research by ARE, Inc (1) developed another approach which we believe makes the data useful at the netowrk level and the project level. The following concepts are taken from this work.

Use for Condition Data in PMS. Our initial meetings on the development of performance models to be used as pavement fitness measures, indicated that our emphasis should be placed on models which provided some physical or practical meaning to pavement evaluation. It was thought that such an approach would help in the process of determining weighting values for both the individual distress parameters and the fitness measure that would be used in the future RIDOT highway condition rating function.

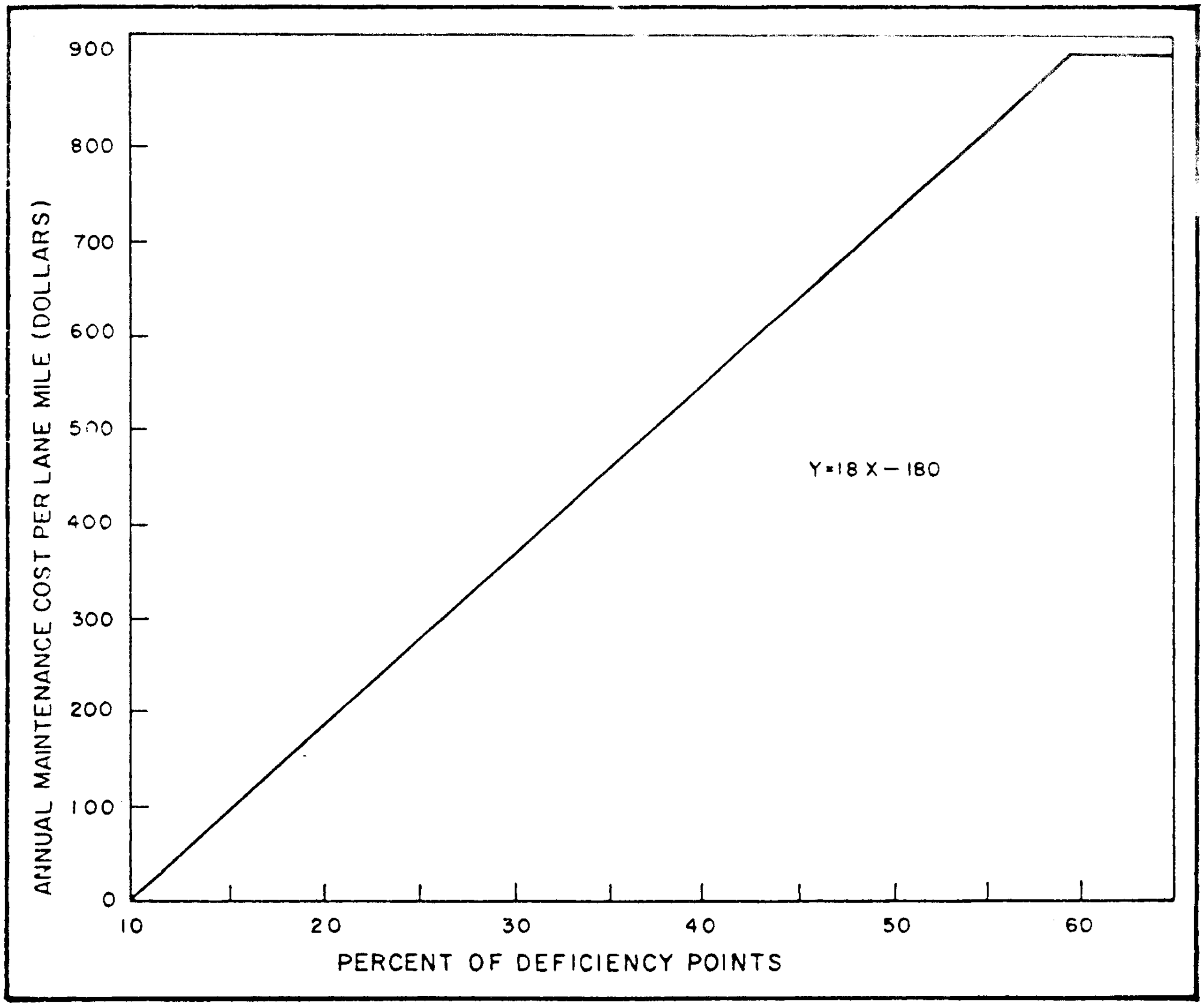


Figure 12. Relationship between Annual Maintenance Cost and Deficiency Rating (After Ref. 17).

From our experience in developing pavement overlay design procedures for the FHWA (Refs 18, 19, and 20) and applying the modified procedures developed by the Texas SDHPT (Refs 21 and 22), we recognized a need to relate pavement fatigue damage to pavement condition and distress so as to minimize (or substitute) the need for accurate past traffic information. We have developed a model for rigid pavements based on regression analyses of AASHO Road Test (Ref 23) data and have also established some tentative criteria for use in a model for flexible pavements. Of course, accurate traffic records make the process even more accurate and reduce the need for deflection measurements.

Before moving on to discuss the actual damage-distress models, it is useful to first discuss the meaning of fatigue damage and its usefulness in pavement evaluation. The term fatigue, as it relates to pavements, refers to the deterioration of the surface layer that results from cyclic loading. Fatigue damage, D, then, is a number which quantifies the amount of fatigue that has occurred and is expressed mathematically as:

$$D = n/N_f$$

where:

N_f = number of allowable cyclic load stresses before the pavement reaches some failure criteria

n = number of actual cyclic load stresses applied to date

Figure 13 represents a plot of damage versus the number of stress cycles. The assumption inherent in the linear relationship, known as Miner's linear damage hypothesis, is that the damage due to a single load stress application is the same, regardless of when it is applied. (Note that load stress and not just load is used here, since the effect of a fixed load can vary with changes in the environment).

With the aid of a relationship between damage and distress, then, it should be apparent from Figure 13 that the damage could be used to predict past traffic and more importantly, future allowable traffic (i.e.,

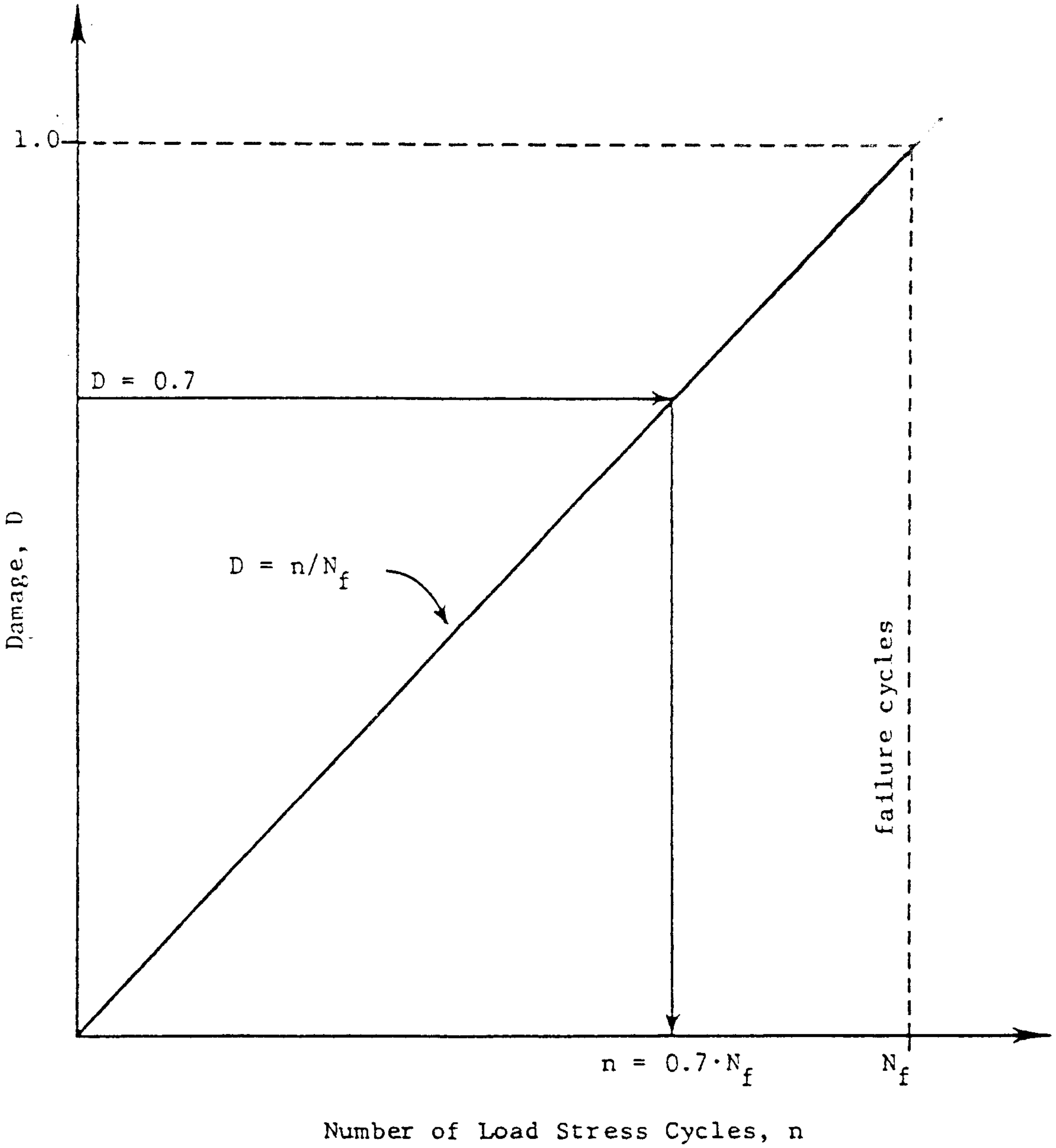


Figure 13. Plot of pavement damage versus load stress applications showing the Damage versus stress application relationship (After Ref. 1).

remaining life). For example, if the fatigue damage was determined to be 0.7 from field observation, then the remaining number of applications the structure can carry before it reaches its failure criteria would be:

$$N_f - n = N_f - (D \cdot N_f) = N_f - 0.7N_f = 0.3N_f$$

In the approach we are developing, N_f represents the full structural capacity of the existing pavement and is determined using known structural parameters and measured surface deflection (see Technical Memo RI1-06).

Because deflection measurements are only expected to be a secondary or tertiery development at this time in the RIDOT PMS, they cannot be initially used. Therefore, the approach would be to use the initial design traffic value for N_f. This, of course, is crude; however, it allows the field condition survey information to be related to remaining life in terms of traffic.

In order to develop a relationship between fatigue damage and pavement distress, it is obviously necessary to use data which consists of a comprehensive history of distress and traffic. Only the AASHO Road Test data provided the required parameters in the volume necessary to develop a damage to distress relationship in the ARE Inc study (Ref 1). Consequently, this data was used to develop the PCC pavement fatigue damage model.

Figure 14 provides an example of a rigid pavement performance record from the AASHO Road Test (Ref 23). As can be seen, traffic was recorded in the form of 18-kip equivalent single axle loads (ESAL) while the basic distress data recorded was cracking, where both extent and severity (i.e., class 1 and 2, class 3 & 4 and asphalt patched) were measured. Figure 15 (Ref 23) illustrates different levels of the extent of cracking in terms of a cracking index which is the sum of the extents of all cracking and patching.

A review of several plots similar to the one presented in Figure 16 for the data presented in Figure 14 indicated a definite relationship

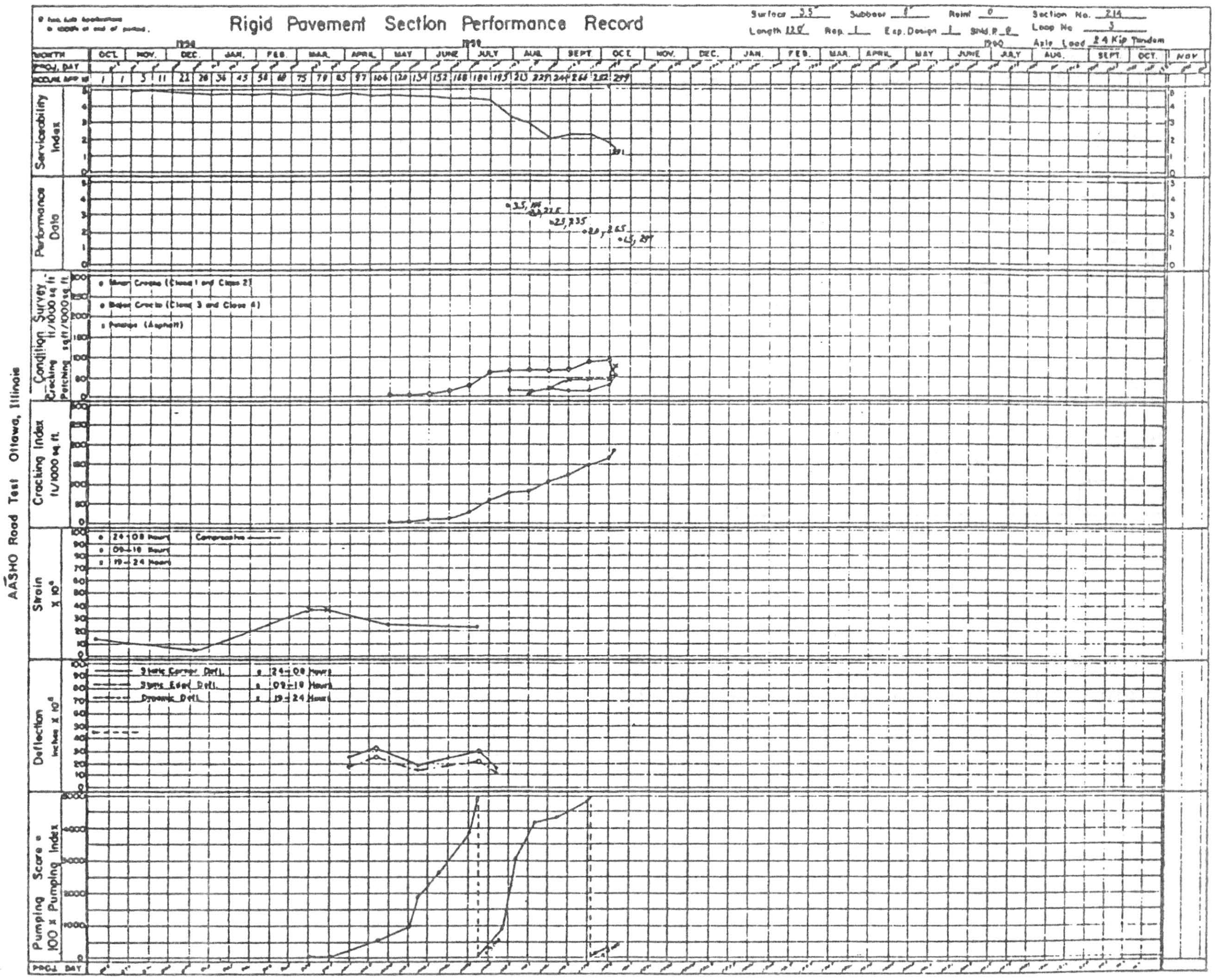


Figure 14. Example of section history chart (after Ref. 1).

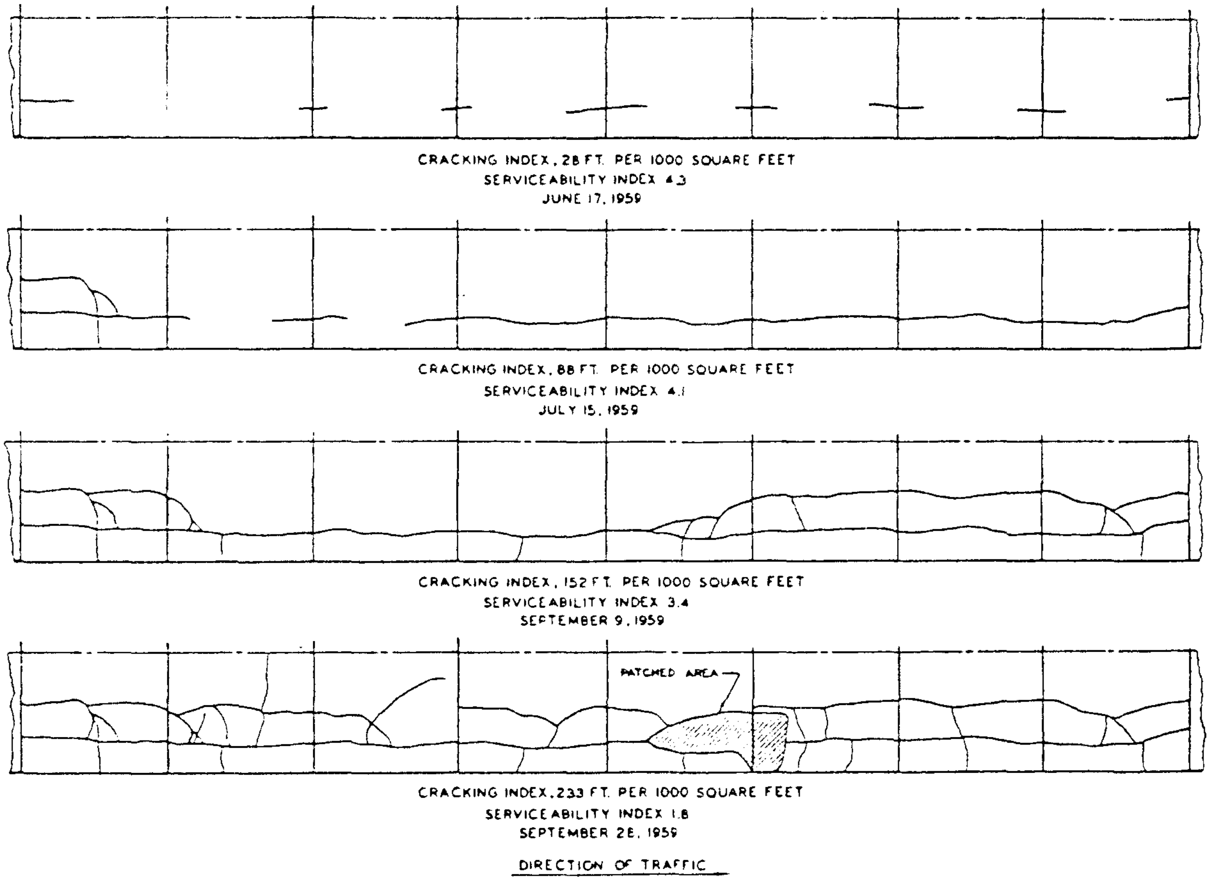


Figure 15. Progression of cracking in a 3.5-in. nonreinforced section with paved shoulders on 6.0 in. of subbase, 24-kip tandem axle load (after Ref. 1).

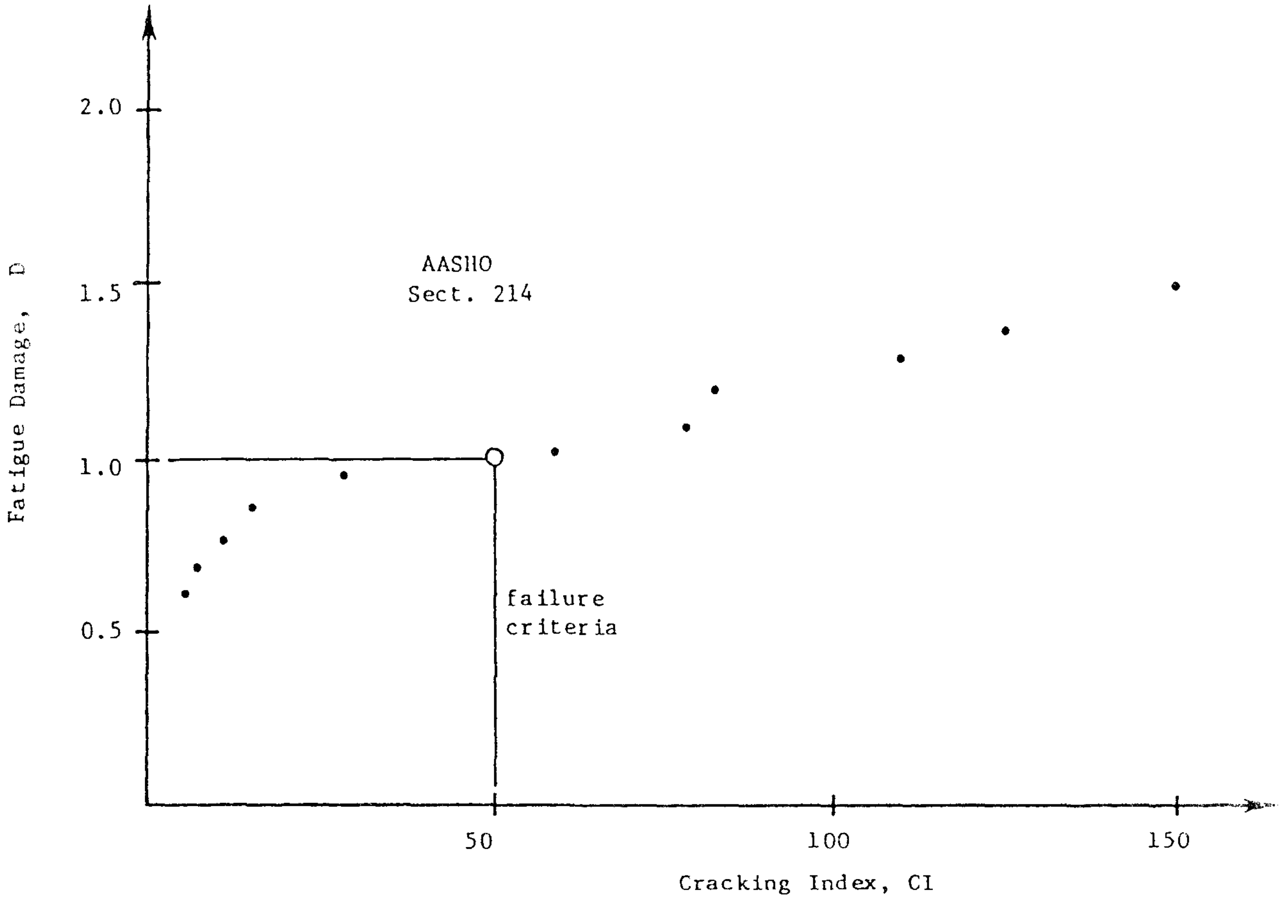


Figure 16 . Plot of Fatigue Damage versus Cracking Index for AASHO Rigid Section 214 (After Ref. 1).

between fatigue damage and cracking for any given PCC section. The failure criteria used was a cracking index of 50 ft/ft^2, since it was apparent in many sections that the rate of cracking increased dramatically beyond this point. The plots also showed that pumping (or pumping score) was also a significant factor affecting damage and cracking. Therefore, its affect was also considered in the analysis. Failure criteria will need to be established by RIDOT engineers and the records of condition survey correlated with traffic data in the PMS data base.

A computer program, STEP01 (Ref 24), for stepwise linear regression was used to develop the equation which relates fatigue damage to cracking index and pumping score. This equation is presented in Figure 17. Although it seems to have a good correlation ($R^2 = 0.65$), inspection of the residual plot for the dependent variable, fatigue damage, indicates that there is a tendency for the equation to overpredict damage when it is in fact low and underpredict damage when it is high. This suggests that there is some factor or affect not considered in the equation, perhaps a nonlinear interaction between the independent variables or a nonlinear transformation of the dependent variable. In either case, further development should be performed. A similar approach can be used to correlate RIDOT condition survey data to traffic for different types of pavements. This approach will give a meaning to th econditino survey data with respect to the pavement's performance. It can be accomplished with the available data, and even if rough will be useful in making network level estimates for the legislature.

Rigid Pavement Fatigue Damage Relationship

$$\begin{aligned} D = -0.6221 &- 0.01888 \times T_1 \\ &+ 0.000007190 \times PS \\ &+ 0.3872 \times \ln(CI + 1) \\ &+ 0.1343 \times \ln(PS + 1) \\ &- 0.3147 \times \ln(CI + 1) \times \ln(PS + 1) \end{aligned}$$

Coefficient of Determination, $R^2 = 0.65$

Equation Parameters:

D = rigid pavement fatigue damage

T_1 = slab thickness, inches

PS = pumping score [100 x (volume/unit length) in in^3/in]

CI = cracking index, linear $feet/feet^2$

Figure 17 . Regression model for the prediction of rigid pavement fatigue damage as a function of slab thickness, cracking and pumping. (Ref. 1).

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Appendix A.

ILLUSTRATIVE PAVEMENT CONDITION RATING FORMS FROM VARIOUS SELECTED STATES

GEORGIA

☐ Resurfacing Project ☐ Seal Project ☐ District Priority Number

GEORGIA
PAVEMENT CONDITION

RATER ____________________ TITLE ____________ DATE ________

DISTRICT NUMBER ____________ STATE ROUTE NUMBER ____________

COUNTY ____________ SURFACE TYPE ____________

LOCATION ________________________________

LENGTH ________ PAVEMENT WIDTH ________ SHOULDER WIDTH ________

BRIDGE WIDTH ____________ AADT ____________

RATING SCALE

10
9 Very Poor
8
7 Poor
6
5 Fair
4
3 Good
2
1 Very Good
0

RELATIVE INFLUENCE ON RATING

TYPE OF DISTRESS	None	Slight	Moderate	Severe	Deteriorated
STRUCTURAL:					
Transverse Cracking					
Longitudinal Cracking					
Alligator Cracking					
Patching					
Rutting					
Edge Rutting					
Ride (Roughness)					
SURFACE:					
Oxidation					
Wear					
Bleeding					
Skid Resistance					
Missing Stone					

COMMENTS & RECOMMENDATIONS ________________________________

STATE HIGHWAY COMMISSION OF KANSAS
MAINTENANCE DEPARTMENT

BITUMINOUS SURFACE MAINTENANCE RECORD

Division ____________
District ____________
Route ____________
County ____________
Proj. No. ____________
From ____________
To ____________
PROJECT LENGTH ____________ Miles

- ☐ 2 Lane
- ☐ 4 Lane
- ☐ 6 Lane
- ☐ Shoulders

Type of Resurfacing
- ☐ Machine Seal
- ☐ Slurry Seal
- ☐ Conventional Seal

SURFACE INFORMATION

TYPE	YEAR	AADT	Score
A.		A.	
B.		B.	
C.		C.	
D		D.	
E.		E.	

4 = Asphaltic Concrete
3 = Machine Seal
2 = Slurry Seal
1 = Conventional
Score = Type-Surface Age

ORIGINAL ROADWAY DESIGN

Score
Asphalt Concrete = 5
P. C. Concrete = 4
Base Tx = 3
Base Tx-S = 2
No Design Base = 1
A. ______

STRUCTURAL ADEQUACY

Score
Adequate = 4
Some Distress = 3
Map Cracking = 2
Inadequate = 1
A. ______
B. ______
C. ______
D. ______
E. ______

TRANSVERSE CRACKS

Number per 100 feet
10 = 0
9 = 1
8 = 2
7 = 3
6 = 4
5 = 6
4 = 8
3 = 10
2 = 12
1 = over 12

Score
A. ______
B. ______
C. ______
D. ______
E. ______

TRANSVERSE CRACK TYPE

Score
Level = 3
Sag = 2
Hump = 1
A. ______
B. ______
C. ______
D. ______
E. ______

LONGITUDINAL CRACKING

Score
None = 5
¼ = 4
½ = 3
¾ = 2
Over ¾ = 1
A. ______
B. ______
C. ______
D. ______
E. ______

CRACK POURING

Score
Full = 15
¼ = 12
½ = 9
¾ = 6
None = 3
A. ______
B. ______
C. ______
D. ______
E. ______

TEMPORARY SIGNING PASS—DO NOT

	Yes or No	
A.		
B.		
C.		
D.		
E.		

WHEEL RUTS

Score
0-¼" = 12
¼"-½" = 9
½"-1" = 6
over 1" = 3
A. ______
B. ______
C. ______
D. ______
E. ______

SURFACE REPAIR COMPLETED

Score
None = 6
⅛ = 5
¼ = 4
½ = 3
¾ = 2
over ¾ = 1
A. (______)*
B. (______)*
C. (______)*
D. (______)*
E. (______)*

* Not Used for Rating

SURFACE REPAIR REQUIRED

Score
None = 18
⅛ = 15
¼ = 12
½ = 9
¾ = 6
over ¾ = 3
A. ______
B. ______
C. ______
D. ______
E. ______

DILUTE SEAL

Score
Not Needed = 4
Needed = 0
A. ______
B. ______
C. ______
D. ______
E. ______

UNIFORMITY OF SURFACE TEXTURE & COLOR

Score
Excellent = 10
Good = 8
Fair = 6
Poor = 4
Non-uniform due to patching, etc. = 2
A. ______
B. ______
C. ______
D. ______
E. ______

SKID RESISTANCE

Score
Excellent = 10
Good = 8
Fair = 6
Poor = 4
Inadequate = 2
A. ______
B. ______
C. ______
D. ______
E. ______

RATING

Total of Scores
A. ______
B. ______
C. ______
D. ______
E. ______

RATE OF APPLICATION

Asphalt	Aggregate
Type Asphalt	Type Aggregate
Gal./Cu. Yd.	Cu. Yd./Mile

Completion Date

DATE	RATED BY	COMMENTS
A.		
B.		
C.		
D		
E.		

PROPOSED FORM TD 71-103

KENTUCKY DEPARTMENT OF TRANSPORTATION
CENTRAL OFFICE EVALUATION FOR SURFACING OR REPAIR

(b)
Front

District	County	Route Number	Road Name

Project Description:
From:
To:

Length	Width	Sq. Yds.	Type Code	Project Number	Last Treatment Date:

I. SERVICE:

Points

AADT--------------- Posted Speed----------------

II. PAVEMENT CONDITION:

	DENSITY				SEVERITY			
	Nil	Few	Inter-mediate	Exten-sive	Slight	Mode-rate	Severe	Points
Cracking	0	2	4	6	1	2	3	----
Base Failures	0	1	2	3	1	2	3	----
Raveling (Spalling)	0	1	2	3	1	2	3	----
Edge Failures	0	.7	1.4	2.1	.6	1.2	1.9	----
Out of Section	0	.6	1.3	2	.3	.6	1	----
Patching	0	1.3	2.6	4	-	-	-	----
Appearance	Good - 0		Fair - 1	Poor - 2		Very Poor - 3		----

Subtotal -----
Average Rutting Depth ------------in.
Roughness Index -------------

III. SAFETY: Skid Number -------------

Inspected By: Date: Total Points

IV. NOTES:

☐ PCC ☐ Bit. ☐ PCC With
☐ Curb & Gutter Bit. Overlay
☐ Man Holes
☐ Shoulders High Low
Width --------
Type ---------
☐ Industrial Haul
Type ----------

Recommended Treatment
☐ Resurface
☐ Patching
Tons per mi.-----
☐ Grinding
Depth ----------
☐ Maintenance
☐ Other ----------

Recommended Type
☐ Bit Conc.
☐ Open Graded
☐ Sand Asphalt
☐ Slurry Seal
☐ Other ---------

V. COST:

Estimated Cost --------------------------------
Recommended Treatment (District) ------------------------------------
District Priority Rank -------------------------

Remarks:

I. SERVICE: Traffic Volume and Posted Speed

AADT	Points at Posted Speed Less than 55 mph	55 mph
500 or less	0	5
501- 1,500	2	7
1,501- 3,500	4	9
3,501- 6,500	6	11
6,501- 10,500	8	13
10,501 or higher	10	15

III SAFETY: Skid Resistance

Skid Number	Points
40 or higher	0
37-39	2
35-36	5
33-34	8
31-32	11
29-30	14
28* or less	114

*Castatrophic Failure – requires remedial action.

II. PAVEMENT CONDITION: Roughness

Roughness Index Bit. Pavements	PCC or PCC with Bit. Overlay	Points
400 or less	425 or less	0
401- 450	426-450	2
451- 500	451-475	4
501- 550	476-500	6.5
551- 600	501-525	8.5
601- 650	526-550	11
651- 700	551-575	13
701- 750	576-600	15
751- 800	601-625	17.5
801- 850	626-650	20
851- 900	651-675	22
901 or higher	676 or higher	24

Ride Quality Assessment	Points
Smooth	0
Medium Rough	5
Medium Rough to Rough	10
Rough	16
Severely Rough	22

NOTE: For roads with traffic speeds below 50 mph, assess ride quality by driving the section at prevailing traffic speed and rate the pavement as being Smooth to Severely Rough (0 to 22 points)

(b)
Back

NOTE: Add 100 points when Roughness Index or depth of rutting for a given volume of traffic exceeds the cited values.

AADT	Roughness Index Bituminous	PCC or PCC with Bit. Overlay	Rutting (inches)
100 or less	-	-	-
101 - 200	1,030	740	1 5/8
201 - 500	1,000	725	1 1/2
501 - 1,000	990	715	1 3/8
1,001 - 2,000	960	705	1 1/4
2,001 - 3,000	930	690	1 1/8
3,001 - 4,000	900	675	1
4,001 - 5,000	870	660	1
5,001 - 6,000	845	645	1
6,001 - 7,000	815	630	7/8
7,001 - 8,000	790	615	7/8
8,001 - 9,000	760	600	7/8
9,001 - 10,000	730	585	7/8
10,001 - 12,000	700	570	3/4
12,001 - 14,000	645	545	3/4
14,001 - 16,000	590	515	3/4
16,001 or higher	555	500	3/4

Rutting (inches)	Points
1/4 or less	0
3/8	2
1/2	4
5/8	6
3/4	9
7/8	12

FLEXIBLE PAVEMENT CONDITION SURVEY

MAINE

SURVEY TEAM I.D. NO. ☐

SECTION I.D. NO. ☐

DIST./LOW NODE ☐ . ☐ .

/ /
DAY MO. YR.

WORK AREA

TIME IN ☐ :

OUT ☐ :

REMARKS

ONS			0	25	50	75	100	
CRACKING	TRANSV.	HAIRLINE	.	.	.	.		.
		< 1/4"	.	.	.	.		.
		> 1/4"	.	.	.	.		.
	LONG.	HAIRLINE	.	.	.	.		.
		< 1/4"	.	.	.	.		.
		> 1/4"	.	.	.	.		.
	LOAD ASSOC.	NONE L / R						
		INIT. L / R					>	☐
		ADV. L / R					>	☐
		SEV. L / R					>	☐
PATCHING		GOOD						☐
		FAIR						☐
		POOR						☐
RUTS		< 1/2" L / R						
		1/2"-1" L / R					>	☐
		> 1" L / R					>	☐
CROWN		NORM.						☐
		1"-2"						☐
		2"-4"						☐
		> 4"						☐
		REV.						☐
SHLD.		BOX CURB						
		NORM.						☐
		MARG.						☐
		DEF.						☐
		NONE						☐

NORTH CAROLINA
FLEXIBLE PAVEMENT
SURFACE CONDITION RATING SHEET

Date: ------------ County: ------------------ Route:---------------- From: ----------------------

To: ---------------- Mileage: ------------- Type Surface: ----------------------------------

Date of Placement: ------------------- Additional information regarding pavement surface: -------------

----

	Gen. Str. Condition	Surface Wear	Uniformity	Rutting	Typical Section	Riding Quality
1	--- Good	--- None	--- Good	--- 0"	--- Excellent	--- Excellent
2	--- Long. Crk.	--- Slight	--- Strkd.	--- 1/8"	--- Good	--- Good
3	--- Map Crk.	--- Moderate	--- Cr. Fill.	--- 1/4"	--- Fair	--- Fair
4.	--- Allig. Crk.	--- Severe	--- Blotchy	--- 1/2"	--- Poor	--- Poor
5	--- Eros.	--- Abrasion	--- Non. Unif.	> 1/2"	--- Very Poor	--- Very Poor

Total Points -------------

NORTH DAKOTA STATE HIGHWAY DEPARTMENT
PAVEMENT CONDITION RATING (PCR) FORM
(1976)

	Month	Year
Date	☐☐ (1)	☐☐ (4)
District	☐ (5)	
Highway	☐☐☐☐ (6–9)	
Mile	☐☐☐ (10–12)	

☐ (17) Mark Left or Right if 4-lane Highway
(Running south to north and west to east)

			Point Value	
SURFACE CRACKING	TRANSVERSE CRACKING (Deep seated, shoulder to shoulder)	None or Occasional (More than 90 ft. spacing)	0	☐ 20
		Minor (60-90 ft. predominantly)	1	
		Moderate (20-60 ft. predominantly)	2	
		Severe (20 ft. spacing and less)	3	
	FATIGUE CRACKING (Mat edge & Minor transverse)	None	0	☐ 21
		Occasional Groups	1	
		Frequent Groups	2	
		Continuous Groups	3	
	TRANSVERSE CRACK WIDTHS (Sealed & Unsealed)	Nonexisting	0	☐ 22
		Up to 1/4"	1	
		About 1/2"	2	
		1" or more	3	
	LONGITUDINAL CRACKING	None	0	☐ 23
		Occasional Spots	1	
		Frequent Spots	2	
		Extended Lengths	3	
	CRACK SPALLING	None Evident	0	☐ 24
		Noticeable	4	
	MAP CRACKING	None	0	☐ 25
		Occasional Spots	2	
		Extended Lengths	3	
	ALLIGATOR CRACKING	None	0	☐ 26
		Occasional Spots	4	
		Extended Lengths	5	
SURFACE DISTORTION	RUTTING (Continuous depression of wheel paths)	None	0	☐ 27
		Minor (up to 1/2" depth)	1	
		Moderate (1/2" to 1" depth)	3	
		Severe (more than 1" depth)	5	
	SHOVING (Washboard)	None	0	☐ 28
		Occasional Spots	2	
		Extended Lengths	3	
	POT HOLES (Pavement breaks-rough spots)	None	0	☐ 29
		Occasional Spots	2	
		Extended Lengths	3	
	RAVELLING (Dislodging of surface layers)	None	0	☐ 30
		Occasional Spots	1	
		Extended Lengths	2	
MAINTENANCE EFFORT	SCOTCH PATCHING	None	0	☐ 31
		Occasional Spots	1	
		Extended Lengths	2	
	MIX PATCHING	None	0	☐ 32
		Occasional Spots	3	
		Extended Lengths	4	
OTHER CONDITIONS	SEAL CONDITION (Retention of seal aggregate)	Good	0	☐ 33
		Fair	2	
		Poor	3	
	SHOULDER CONDITION	Good	0	☐ 34
		Cracked	1	
		Broken-Up (Crumbling)	2	
		Grassed Over or Nonexisting	3	

OHIO
EVALUATION BITUMINOUS SURFACED PAVEMENTS

District ______________ County ______________ Route ______________

Begin SLM ______________ End SLM ______________ City Village ______________

Present Surface Type ☐ 404 ☐ 405 ☐ Other ______________

Previous Treatment Year ______________ Length ______________ mi. Width ______________

PAVEMENT RATING		
CRACKED and/or ALLIGATORED 1 2 3 4 5 6 7 8 9 10	X 2 =	
RAVELING 1 2 3 4 5 6 7 8 9 10	X 2 =	
PATCHED and/or CRACK SEALED 1 2 3 4 5 6 7 8 9 10	X 1 =	
ROUGHNESS 1 2 3 4 5 6 7 8 9 10	X 4 =	
RUTTED 1 2 3 4 5 6 7 8 9 10	X 1 =	
	TOTAL	

1 2 3 4 5 6 7 8 9 10

Very Poor Poor Fair Good Very Good

FINAL EVALUATION	
1. PAVEMENT RATING (from above)	
2. ON SLIPPERY PAVEMENT LIST	
3. AVERAGE DAILY TRAFFIC	
(a) total ______________	
(b) number B & C commercial	
4. DEFERMENT OF RESURFACING	
(a) severely reduce routine maintenance (yes or no) ______________	
(b) beyond capability to further maintain (yes or no)	

REMARKS: ______________

____

____

____

______________ INSPECTED BY

______________ DATE

FLEXIBLE

PAVEMENT CONDITION RATING FORM

DISTRESS	DISTRESS WEIGHT	SEVERITY WEIGHT *			EXTENT WEIGHT **			DEDUCT POINTS ***
		L	M	H	O	F	E	
RAVELING	10	[✓ .3]	.6	1.0	.5	[✓ .8]	1.0	2.4
BLEEDING	5	.8	.8	1.0	.6	.9	1.0	–
PATCHING	5	[✓ .3]	.6	1.0	[✓ .6]	.8	1.0	.9
POTHOLES	10	.4	.7	1.0	.5	.8	1.0 ✓	–
CRACK SEALING DEFICIENCY	5	[✓ 1.0]	1.0	1.0	[✓ .5]	.8	1.0	2.5
RUTTING	10	[✓ .3]	.7	1.0	.6	.8	[✓ 1.0] ✓	3.0
SETTLEMENT	10	[✓ .5]	.7	1.0	[✓ .5]	.8	1.0	2.5
CORRUGATIONS	5	[✓ .4]	.8	1.0	[✓ .5]	.8	1.0	1.0
WHEEL TRACK CRACKING	15	[✓ .4]	.7	1.0	[✓ .5]	.7	1.0 ✓	3.0
BLOCK & TRANSVERSE CRACKING	10	[✓ .4]	.7	1.0	[✓ .5]	.7	1.0 ✓	2.0
LONGITUDINAL JOINT CRACKING	5	[✓ .4]	.7	1.0	[✓ .5]	.7	1.0	1.0
EDGE CRACKING	5	.4	.7	1.0	.5	.7	1.0	–
RANDOM CRACKING	5	.4	.7	1.0	.5	.7	1.0 ✓	–

28-86

* L = LOW
M = MEDIUM
H = HIGH

** O = OCCASIONAL
F = FREQUENT
E = EXTENSIVE

TOTAL DEDUCT = 18.3

SUM OF STRUCTURAL DEDUCT (✓) = 8.0

100 – TOTAL DEDUCT = **PCR** = 81.7

*** Deduct pts. = Distress Wt. X Severity Wt. X Extent Wt.

Remarks:

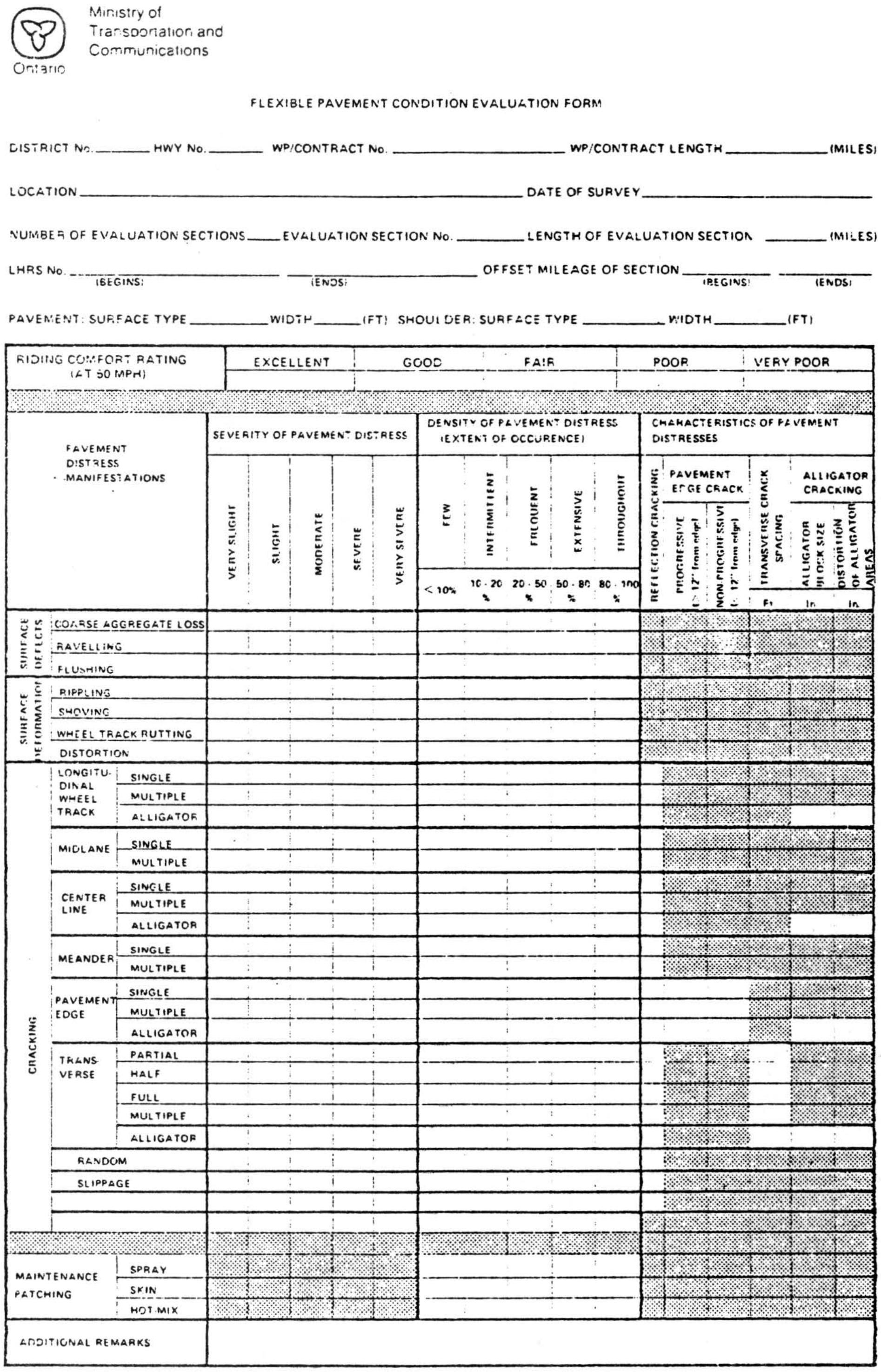
OHIO

Maintenance Patching

ONTARIO

Maintenance patching should be reported by its type and the density of occurrence by checking off the appropriate spaces (see Figure A-1).

The performance summary based on information gathered in this "Flexible Pavement Condition Evaluation Form" is used to formulate the "Condition Rating" as given in Appendix B.



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FLEXIBLE PAVEMENT CONDITION EVALUATION FORM

DISTRICT No.______ HWY No.______ WP/CONTRACT No. ____________ WP/CONTRACT LENGTH _________ (MILES)

LOCATION ________________________ DATE OF SURVEY ____________

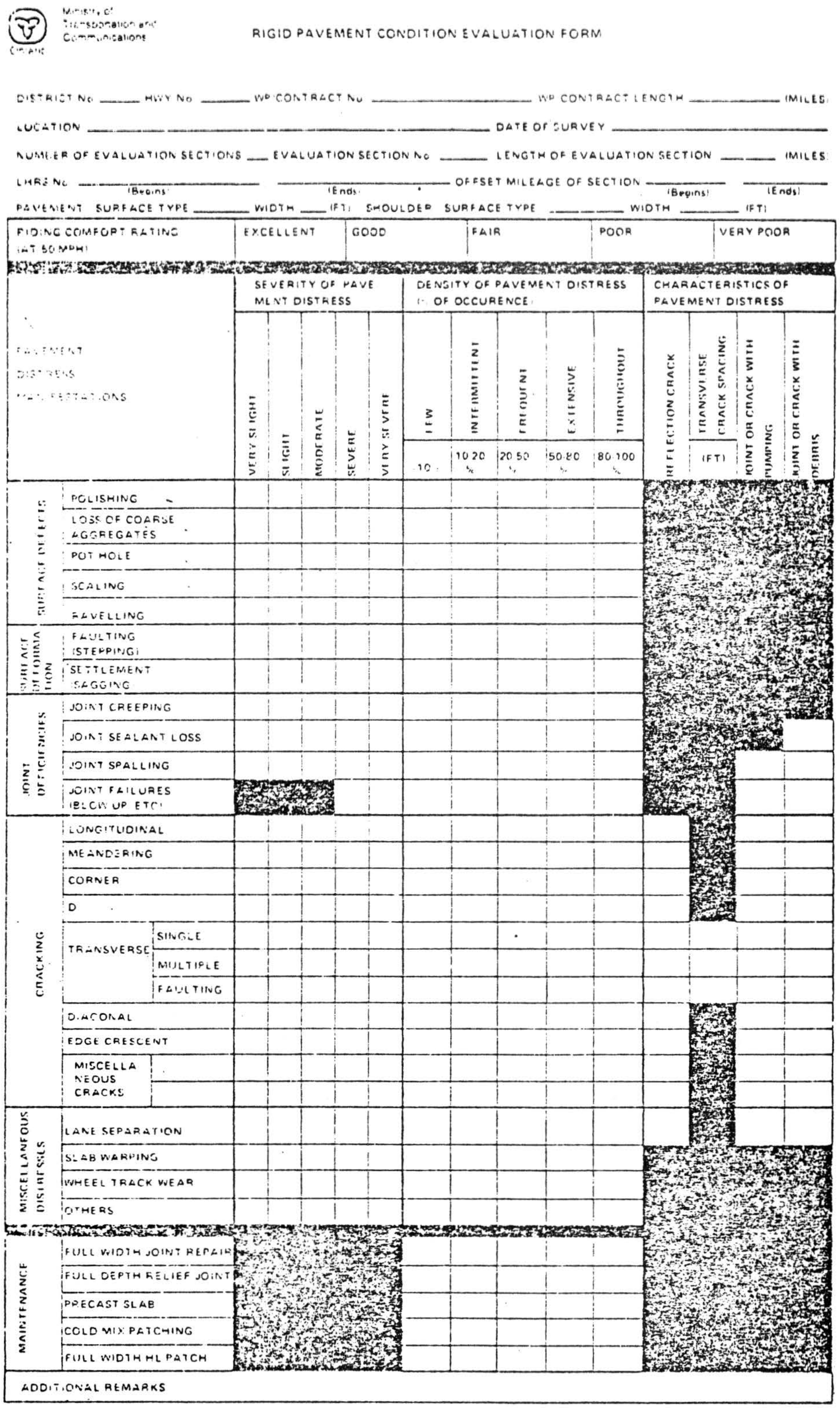
NUMBER OF EVALUATION SECTIONS ______ EVALUATION SECTION No. ______ LENGTH OF EVALUATION SECTION ______ (MILES)

LHRS No. __________ (BEGINS) __________ (ENDS) OFFSET MILEAGE OF SECTION ______ (BEGINS) ______ (ENDS)

PAVEMENT: SURFACE TYPE ________ WIDTH ______ (FT) SHOULDER: SURFACE TYPE ________ WIDTH ______ (FT)

RIDING COMFORT RATING (AT 50 MPH)	EXCELLENT	GOOD	FAIR	POOR	VERY POOR

PAVEMENT DISTRESS MANIFESTATIONS			SEVERITY OF PAVEMENT DISTRESS					DENSITY OF PAVEMENT DISTRESS (EXTENT OF OCCURENCE)					CHARACTERISTICS OF PAVEMENT DISTRESSES					
			VERY SLIGHT	SLIGHT	MODERATE	SEVERE	VERY SEVERE	FEW < 10%	INTERMITTENT 10 - 20 %	FREQUENT 20 - 50 %	EXTENSIVE 50 - 80 %	THROUGHOUT 80 - 100 %	REFLECTION CRACKING	PAVEMENT EDGE CRACK PROGRESSIVE (> 12" from edge)	PAVEMENT EDGE CRACK NON PROGRESSIVE (< 12" from edge)	TRANSVERSE CRACK SPACING Ft	ALLIGATOR CRACKING ALLIGATOR BLOCK SIZE In	ALLIGATOR CRACKING DISTORTION OF ALLIGATOR AREAS In
SURFACE DEFECTS	COARSE AGGREGATE LOSS												▨	▨	▨	▨	▨	▨
	RAVELLING												▨	▨	▨	▨	▨	▨
	FLUSHING												▨	▨	▨	▨	▨	▨
SURFACE DEFORMATION	RIPPLING												▨	▨	▨	▨	▨	▨
	SHOVING												▨	▨	▨	▨	▨	▨
	WHEEL TRACK RUTTING												▨	▨	▨	▨	▨	▨
	DISTORTION												▨	▨	▨	▨	▨	▨
CRACKING	LONGITUDINAL WHEEL TRACK	SINGLE												▨	▨	▨	▨	▨
		MULTIPLE												▨	▨	▨	▨	▨
		ALLIGATOR												▨	▨	▨		
	MIDLANE	SINGLE												▨	▨	▨	▨	▨
		MULTIPLE												▨	▨	▨	▨	▨
	CENTER LINE	SINGLE												▨	▨	▨	▨	▨
		MULTIPLE												▨	▨	▨	▨	▨
		ALLIGATOR												▨	▨	▨		
	MEANDER	SINGLE												▨	▨	▨	▨	▨
		MULTIPLE												▨	▨	▨	▨	▨
	PAVEMENT EDGE	SINGLE														▨	▨	▨
		MULTIPLE														▨	▨	▨
		ALLIGATOR														▨		
	TRANSVERSE	PARTIAL												▨	▨		▨	▨
		HALF												▨	▨		▨	▨
		FULL												▨	▨		▨	▨
		MULTIPLE												▨	▨		▨	▨
		ALLIGATOR												▨	▨			
	RANDOM													▨	▨	▨	▨	▨
	SLIPPAGE													▨	▨	▨	▨	▨
														▨	▨	▨	▨	▨
													▨	▨	▨	▨	▨	▨
▨	▨	▨	▨	▨	▨	▨	▨	▨	▨	▨	▨	▨	▨	▨	▨	▨	▨	▨
MAINTENANCE PATCHING	SPRAY		▨	▨	▨	▨	▨						▨	▨	▨	▨	▨	▨
	SKIN		▨	▨	▨	▨	▨						▨	▨	▨	▨	▨	▨
	HOT MIX		▨	▨	▨	▨	▨						▨	▨	▨	▨	▨	▨
ADDITIONAL REMARKS																		



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RIGID PAVEMENT CONDITION EVALUATION FORM

DISTRICT No ______ HWY No ______ WP CONTRACT No ____________________ WP CONTRACT LENGTH ____________ (MILES)

LOCATION ____________________________________ DATE OF SURVEY ____________________

NUMBER OF EVALUATION SECTIONS ___ EVALUATION SECTION No ______ LENGTH OF EVALUATION SECTION ______ (MILES)

LHRS No ____________ (Begins) ____________ (Ends) OFFSET MILEAGE OF SECTION ________ (Begins) ________ (Ends)

PAVEMENT SURFACE TYPE ________ WIDTH ____ (FT) SHOULDER SURFACE TYPE ________ WIDTH ________ (FT)

RIDING COMFORT RATING (AT 50 MPH)	EXCELLENT	GOOD	FAIR	POOR	VERY POOR

	PAVEMENT DISTRESS MANIFESTATIONS		SEVERITY OF PAVEMENT DISTRESS					DENSITY OF PAVEMENT DISTRESS (% OF OCCURENCE)					CHARACTERISTICS OF PAVEMENT DISTRESS			
			VERY SLIGHT	SLIGHT	MODERATE	SEVERE	VERY SEVERE	FEW <10 %	INTERMITTENT 10-20 %	FREQUENT 20-50 %	EXTENSIVE 50-80 %	THROUGHOUT 80-100 %	REFLECTION CRACK	TRANSVERSE CRACK SPACING (FT)	JOINT OR CRACK WITH PUMPING	JOINT OR CRACK WITH DEBRIS
SURFACE DEFECTS	POLISHING												▨	▨	▨	▨
	LOSS OF COARSE AGGREGATES												▨	▨	▨	▨
	POT HOLE												▨	▨	▨	▨
	SCALING												▨	▨	▨	▨
	RAVELLING												▨	▨	▨	▨
SURFACE DEFORMATION	FAULTING (STEPPING)												▨	▨	▨	▨
	SETTLEMENT (SAGGING)												▨	▨	▨	▨
JOINT DEFICIENCIES	JOINT CREEPING												▨	▨	▨	▨
	JOINT SEALANT LOSS												▨	▨	▨	
	JOINT SPALLING												▨	▨		
	JOINT FAILURES (BLOW UP ETC)		▨	▨	▨								▨	▨		
CRACKING	LONGITUDINAL													▨		
	MEANDERING													▨		
	CORNER													▨		
	D													▨		
	TRANSVERSE	SINGLE														
		MULTIPLE														
		FAULTING														
	DIAGONAL													▨		
	EDGE CRESCENT													▨		
	MISCELLANEOUS CRACKS													▨		
														▨		
MISCELLANEOUS DISTRESSES	LANE SEPARATION													▨		
	SLAB WARPING												▨	▨	▨	▨
	WHEEL TRACK WEAR												▨	▨	▨	▨
	OTHERS												▨	▨	▨	▨
MAINTENANCE	FULL WIDTH JOINT REPAIR		▨	▨	▨	▨	▨						▨	▨	▨	▨
	FULL DEPTH RELIEF JOINT		▨	▨	▨	▨	▨						▨	▨	▨	▨
	PRECAST SLAB		▨	▨	▨	▨	▨						▨	▨	▨	▨
	COLD MIX PATCHING		▨	▨	▨	▨	▨						▨	▨	▨	▨
	FULL WIDTH HL PATCH		▨	▨	▨	▨	▨						▨	▨	▨	▨

ADDITIONAL REMARKS

Rated By................Date.........

TENNESSEE SURFACE CONDITION RECORD

COORDINATED DATA FILE SYSTEM

CARD NUMBER	COUNTY NUMBER	ROUTE NUMBER	SC	COUNTY SEQ. NO.
1 2	3 4	5 6 7 8 9	10	11 12
1 1				

BEGINNING LOG MILE	ENDING LOG MILE	CROSS SECTION GRADE	CROSS SECTION POINT RATING	PROFILE GRADE	PROFILE POINT RATING	SURFACE CHARACTERISTICS GRADE	SURFACE CHARACTERISTICS POINT RATING	TOTAL POINT RATING
13 14 15 16	17 18 19 20	21	22 23	24	25 26	27	28 29	30 31 32

CROSS SECTION

Uniformity of crown
Superelevation of curves
Raveling and/or spalling of pavement edges
Rutting in wheel paths

Grade	Condition	% Meeting Accept. Cond.	Pt Rating
A	Excellent	95–100	25
B	Very Good	85–95	20
C	Good	75–85	15
D	Fair	65–75	10
E	Poor	50–65	5
F	Very Poor	less than 50	0

PROFILE

Corrugations
Humps
Dips
Riding quality

Grade	Condition	% Meeting Accept. Cond.	Pt. Rating
A	Excellent	95–100	25
B	Very Good	85–95	20
C	Good	75–85	15
D	Fair	65–75	10
E	Poor	50–65	5
F	Very Poor	less than 50	0

SURFACE CHARACTERISTICS

Pot holes
Surface raveling and disintegration
Blow-ups
Pumping
Bleeding
Patching
Cracking
Bumps

Grade	Condition	% Meeting Accept. Cond.	Pt. Rating
A	Excellent	95–100	50
B	Very Good	85–95	40
C	Good	75–85	30
D	Fair	65–75	20
E	Poor	50–65	10
F	Very Poor	less than 50	0

REMARKS: ____

DATE ______

DISTRICT NO.			LOCATION	
COUNTY NO.				
HIGHWAY NO.				
CONTROL				
SECTION				
FROM				
TO				
LANE NO.				
RATER NO.				
SLIGHT	① 1-15	%AREA	RUTTING	PAVEMENT
MODERATE	② 16-30			
SEVERE	③ >30			
SLIGHT	① 1-15	%AREA	RAVELING	
MODERATE	② 16-30			
SEVERE	③ >30			
SLIGHT	① 1-15	%AREA	FLUSHING	
MODERATE	② 16-30			
SEVERE	③ >30			
SLIGHT	① 1-15	%AREA	CORRUGATIONS	
MODERATE	② 16-30			
SEVERE	③ >30			
SLIGHT	① 1-5	%AREA	WAVES, SAGS, HUMPS	
MODERATE	② 6-25			
SEVERE	③ >25			
SLIGHT	① 1-5	%AREA	ALLIGATOR CRACKING	
MODERATE	② 6-25			
SEVERE	③ >25			
SLIGHT	① 0-99	LIN. FT. PER STA.	LONGITUDINAL CRACKING	
MODERATE	② 100-199			
SEVERE	③ >200			
SLIGHT	① 1-4	NO. PER STA	TRANSVERSE CRACKING	
MODERATE	② 5-9			
SEVERE	③ >10			
GOOD	① 1-5	%AREA	PATCHING	
FAIR	② 6-15			
POOR	③ >16			
RIDE			PAVED	SHOULDER
CONTRAST				
PAVEMENT EDGE				
SHOULDER EDGE				
CRACKS				
RAVELING				
VEGETATION				
PAVEMENT EDGE			UNPAVED	
RUTTING, CORRUGATIONS, LOOSE ROCK				
LITTER			ROADSIDE	
MOWING				
VEGETATION				
SLOPE EROSION				
ENCROACHMENTS				
CULVERTS			DRAINAGE	
DITCHES, OUTFALL, CHANNELS				
ROADSIDE DRAINAGE				
GUARDRAILS			TRAFFIC SERVICES	
SIGNS				
DELINEATORS				
STRIPING				
AUXILIARY MARKINGS				
SIGNALS				
ROADWAY MAINTENANCE SCHEDULE				

DATE ____________

LOCATION

DISTRICT NO.

COUNTY NO.

HIGHWAY NO.

CONTROL

SECTION

FROM

TO

LANE NO

RATER NO

PORTLAND CEMENT CONCRETE PAVEMENT	JOINTED ONLY	CRACKING	CRACKS > 1/8" PER PANEL	1 1-2 2 3-5 3 5+	1-25% PANELS 25-50% PANELS OVER 50% PANELS
		JOINT SPACING		1< 20' 2 >20'	SAWED CONSTRUCTION
	ALL PAVEMENTS	JOINT SPALLING or TRANSVERSE CRACK SPALLING	WIDTH OF SPALL IN INCHES	1 0-1 2 1-3 3 3+	1-16% JOINTS 16-50% JOINTS OVER 50% JOINTS
		RAVELLING DISENTIGRATION POP OUT SCALING		1 SLIGHT 2 MODERATE 3 SEVERE	1-25% AREA 26-75% AREA OVER 75% AREA
		PUMPING BLOWING	% SECTION LENGTH	1 < 10% 2 10-50% 3 > 50%	1-15% SECTIONS 16-35% SECTIONS OVER 35% SECTIONS
		FAULTING CURLING WARPING		1 0-1/4" 2 1/4-1/2" 3 1/2" +	1-15% SECTIONS 16-35% SECTIONS OVER 35% SECTIONS
		PATCHING	% AREA PER SECTION	1 1-5% 2 6-25% 3 25% +	1-5% SECTIONS 6-20% SECTIONS OVER 20% SECTIONS
	CRCP ONLY	TRANSVERSE CRACK SPACING	3 < 2' OR >15'	1 5-10' 2 2-5' OR 10-15'	< 1/16" WIDE > 1/16" WIDE
		LONGITUDINAL CRACKING	LINEAL FEET PER STATION	1 0-99' 2 100-199' 3 200 +	< 1/8" WIDE 1/8"-1/4" WIDE SPALLED
		% INTERSECTING CRACKS		1<10% 2>10%	NO LOOSE PIECES LOOSE PIECES
SHOULDER	PAVED				RIDE CONTRAST PAVEMENT EDGE SHOULDER EDGE CRACKS RAVELLING VEGETATION
	UNPAVED				PAVEMENT EDGE RUTTING, CORRUGATIONS, LOOSE ROCK
	ROADSIDE				LITTER MOWING VEGETATION SLOPE EROSION ENCROACHMENTS
	DRAINAGE				CULVERTS DITCHES, OUTFALL, CHANNELS ROADSIDE DRAINAGE
	TRAFFIC SERVICES				GUA SIGNS DELINEATORS STRIPING AUXIL RY MARKINGS SIGNALS
					ROADWAY MAINTENANCE SCHEDULE

WASHINGTON
PAVEMENT CONDITION RATING
BITUMINOUS PAVEMENTS
DEFECT DEDUCTIONS

Negative Values Are Assigned
To The Failures By Degree

			Throughout Rated Section				
			None	1/4-1/2	1/2-3/4	3/4+	
RUTTING PAVEMENT WEAR	Average Depth in Inches	(1) 1/4-1/2"		5			
		(2) 1/2-3/4"			12		Negative
		(3) Over 3/4"				20	Values

			Change Per 10 Feet in Inches				
			None	0-2	2-4	4+	
CORRUGATIONS WAVES SAGS HUMPS	Percent of Roadway	(1) 1-25		1	2	3	
		(2) 26-75		2	3	4	Negative
		(3) 76+		3	4	5	Values

			Percent of Wheel Track Per Station					
			None	1-24	25-49	50-74	75+	
ALLIGATOR CRACKING		(1) Hairline		2	5	10	15	
		(2) Spalling		5	10	15	20	Negative
		(3) Spalling & Pumping		10	15	20	25	Values

			Local-ized	Wheel Paths	Entire Lane	
RAVELING OR FLUSHING		(1) Slight	2	5	10	
		(2) Moderate	5	10	15	Negative
		(3) Severe	10	15	20	Values

			Average Width in Inches				
			None	1/8-1/4	1/4+	Spalled	
LONGITUDINAL CRACKING	Lineal Feet Per Station	(1) 1-99		10	15	20	
		(2) 100-199		15	20	25	Negative
		(3) 200+		20	25	30	Values

			Average Width in Inches				
			None	1/8-1/4	1/4+	Spalled	
TRAVERSE CRACKING	Number Per Station	(1) 1-4		8	10	15	
		(2) 5-9		9	12	17	Negative
		(3) 10+		10	15	20	Values

			Average Depth in Inches				
			None	0-1/2	1/2-1	1+	
PATCHING	Percent Area Per Station	(1) 1-5		2	5	7	
		(2) 6-25		5	7	10	Negative
		(3) 26+		7	10	15	Values

WASHINGTON
PAVEMENT CONDITION RATING
CEMENT CONCRETE PAVEMENT
DEFECT DEDUCTIONS

Negative Values Are Assigned
To The Failures By Degree

CRACKING AVERAGING 1/8+	Units Per Panel Length		Percent of Panels				
			None	1-25	26-50	51+	
		(1) 1-2		5	10	20	
		(2) 3-4		10	20	35	Negative
		(3) 4+		15	30	50	Values
RAVELING DISINTEGRATION POPOUTS SCALING			Percent of Area				
			None	1-25	26-75	76+	
		(1) Slight		5	10	20	
		(2) Moderate		10	20	35	Negative
		(3) Severe		15	30	50	Values
SPALLING AT JOINTS AND CRACKS	Average Width in Inches		Percent of Joints				
			None	1-15	16-50	51+	
		(1) 0-1		5	10	20	
		(2) 1-3		10	20	35	Negative
		(3) 3+		15	30	50	Values
PUMPING BLOWING	Percent of Panel Length		Percent of Panels				
			None	1-15	16-35	36+	
		(1) 1-9		5	20	35	
		(2) 10-50		10	25	40	Negative
		(3) 51+		15	30	45	Values
BLOW-UPS	Number Per Mile		Blowups Per Mile				
			None	1	2-3	4+	
		(1) 1		5			
		(2) 2-3			10		Negative
		(3) 4+				15	Values
FAULTING CURLING WARPING SETTLEMENT	Average Displacement in Inches		Percent of Panels				
			None	1-15	16-35	36+	
		(1) 0-1/4		0	10	20	
		(2) 1/4-1/2		5	15	25	Negative
		(3) 1/2+		10	20	30	Values
PATCHING	Percent of Panels		Percent of Area Per Panel				
			None	1-5	6-25	26+	
		(1) 1-5		2	5	7	
		(2) 6-20		5	7	10	Negative
		(3) 21+		7	10	15	Values
RUTTING PAVEMENT WEAR	Average Depth in Inches		Throughout Rated Section				
			None	1/4-1/2	1/2-3/4	3/4+	
		(1) 1/4-1/2"		5			Negative
		(2) 1/2-3/4"			12		Values
		(3) Over 3/4"				20	

PAVEMENT CONDITION DATA

7		10 11		15	18	22	23	24	25
CONTROL DATA									RIDE
NOT USED	STATE ROUTE	FUNCT. CLASS	CONTROL SECTION	CONTROL SECTION SEQ.	ENDING CONTROL SECTION MILEPOST FOR RATED SECTION	PAVEMENT TYPE "A", "B", "C"	• PAVEMENT EXEMPTION	MULTI-LANE (L), (R), (B)	01 2 3 4 5 6 7 8 9 Very Good Good Fair Poor
	28-94								
	DUPLICATE								

27	28			31				35				39			42			45		
BITUMINOUS PAVEMENTS																				
RUTTING PAVT. WEAR	CORRUGATION, WAVES, SAGS, HUMPS % Roadway (1) 1 - 25% (2) 26 - 75% (3) Over 75%			ALLIGATOR CRACKING (1) Hairline (2) Spalling (3) Spalling & Pumping				RAVELING OR FLUSHING (1) Slight (2) Moderate (3) Severe				LONGITUDINAL CRACKING Lineal Ft/Sta (1) 0 - 99 (2) 100 - 199 (3) 200 Plus			TRANSVERSE CRACKING No./Station (1) 1 - 4 (2) 5 - 9 (3) 10 Plus			PATCHING % Area/Sta. (1) 1 - 5% (2) 6 - 25% (3) Over 25%		
••	0 - 2" CHANGE/10FT.	2 - 4" CHANGE/10 FT.	OVER 4" CHANGE/10 FT.	1 - 24% WHL. TRK/STA.	25 - 49% WHL. TRK/STA.	50 - 74% WHL. TRK/STA.	75 - 100% WHL. TRK /STA.	LOCALIZED	WHEEL PATHS	ENTIRE LANE	R = RAVELING F = FLUSHING	LESS THAN 1/4"	OVER 1/4" WIDE	SPALLED	LESS THAN 1/4"	OVER 1/4" WIDE	SPALLED	0 - 0.50" THICK	0.50 - 1.0" THICK	OVER 1.0" THICK

48			51			54			57			60	61			64			67
PORTLAND CEMENT CONCRETE																			
CRACKING Units Panel Length (1) 1 - 2 (2) 3 - 4 (3) 4 Plus			RAVELING DISINTEGRATION, POP OUT, SCALING (1) Slight (2) Moderate (3) Severe			JOINT SPALLING Width (1) 0 - 1" (2) 1 - 3" (3) 3 Plus Inches			PUMPING BLOWING % Panel Lgth (1) 0 - 9% (2) 10 - 50% (3) Over 50%			BLOWUPS	FAULTING, CURLING, WARPING, SETTLEMNT (1) 0 1/4" (2) 1/4" 1/2" (3) 1/2" Plus			PATCHING % Panels (1) 1 - 5% (2) 6 - 20% (3) Over 20%			PAVT WEAR
1 - 25% PANELS	26 - 50% PANELS	OVER 50% PANELS	1 - 25% AREA	26 - 75% AREA	OVER 75% AREA	1 - 15% JOINTS	16 - 50% JOINTS	OVER 50% JOINTS	1 - 15% OF PANELS	16 - 35% OF PANELS	OVER 35% OF PANELS	NO./MILE	1 - 15% OF PANELS	15 - 35% OF PANELS	OVER 35% OF PANELS	1 - 5% AREA/PANEL	6 - 25% AREA/PANEL	OVER 25% AREA/PANEL	••

WASHINGTON

WEST VIRGINIA DEPARTMENT OF HIGHWAYS

PAVEMENT RATING WORKSHEET

DATE: ------------

ROUTE: ------------

FROM: ------------

ADT: ------------

TO: ------------

LENGTH: ------------

SUFFICIENCY RATING

	SCORE (1-5)	$(\text{SCORE})^2$
RIDEABILITY:	-------------	--------------.
LEVELING:	-------------.	--------------.
WATERPROOFING:	-------------.	--------------.
GEOMETRICS:	-------------.	--------------.
SHOULDERS:	-------------.	--------------.

SUFFICIENCY RATING--------------.

$$\text{NEED INDEX} = \frac{\text{ADT}}{\text{SUFFICIENCY RATING}} = \text{--------------.}$$

PROJECT REVIEW NOTES: --

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LESSON OUTLINE
FRICTION MEASUREMENT METHODS

Instructional Objectives

1. To provide the student with a knowledge of the basic concepts of friction resistance phenomenon.
2. To present the methods of friction resistance measurement most commonly used, its advantages and disadvantages.
3. To present the parameters that influence the friction resistance of a pavement and how they vary under different conditions.

Performance Objectives

1. The student should be able to explain the basic concepts of the friction resistance phenomenon.
2. The student should be able to describe the most commonly used methods of friction resistance measurement.
3. The student should be able to explain how friction resistance changes and what are the parameters that affect it most.

Abbreviated Summary	Time Allocation, min.
1. Basic Friction Resistance Concepts	10
2. Friction Resistance Measurement	25
3. Deterioration of Friction Resistance	15
	50 minutes

Reading Assignment

1. Haas & Hudson - Chapter 10, pages 107-115
2. RTAC-Canadian Guide - Section 4.4, pages 4.22-4.29
3. Instructional Text

LESSON OUTLINE
FRICTION MEASUREMENT METHODS

1.0 BASIC CONCEPTS OF FRICTION IN HIGHWAY PAVEMENTS

1.1 Nature of Friction

The phenomenon of skidding involves a very complex interrelationship among pavement factors, vehicle factors (mainly tires), and driving factors.

(a) When friction is developed
(b) Adhesion and deformation forces
(c) Coefficient of friction

1.2 Dry versus Wet Pavements

Skidding accidents occur not only by direct forward sliding, but also by jackknifing and by breaking away or sliding off curves. Thus, friction-resistance measurements are taken under wet pavement conditions.

2.0 FRICTION MEASUREMENT METHODS

2.1 Introduction

Most of the past research has been directed at identifying frictional needs of the vehicle under various conditions or various devices to resistance.

2.2 Methods of Measurement

ASTM Committee E17 on Skid Resistance deals with the development of standard methods of measuring available friction. These methods should be used, unless there is valid reason for using others. All are detailed in the ASTM manuals.

2.2.1 Locked-wheel Trailer Methods ASTM E 274.

(a) Standard Tire - ASTM E 249,

(b) Application of water to the pavement, and

(c) Source of error in the measurements.

2.2.2 Automobile Methods ASTM E 445.

- (a) Standard Tire - ASTM E 249,
- (b) Diagonal wheel lock,
- (c) No standard vehicle, and
- (d) Environment.

2.2.3 Portable Field Testers. These generally involve dropping a spring-loaded rubber shoe attached to a pendulum.

- (a) The California skid tester,
- (b) The Drag tester (Keystone tester), and
- (c) British Pendulum Device.

2.2.4 Slip Mode Methods. This refers to the phenomenon that occurs when a wheel is gradually braked, with increasing friction factor, to the point of "critical slip" beyond which the wheel locks and the friction factor drops.

- (a) "Critical Slip" concept and maximum friction factor (Visual Aid 29.1), and
- (b) Critical slip variability (Visual Aid 29.2).

2.2.5 Yaw Mode Methods. The wheels are turned at some angle to the direction of motion and the slide or cornering force is measured and it peaks at some critical yaw angle.

- (a) Basis of the method, and
- (b) Critical yaw angle (Visual Aid 29.3).

2.3 Correlation Between Testers

Each type of tester measures a different aspect of the friction developed on a pavement surface. Therefore, it should not be expected that there will be a 1:1 correlation between the results obtained with different types of testers. (Visual Aid 29.4).

- (a) Shows the correlation for ASTM Method E274 vs. Automobile Methods (Visual Aid 29.4 and 29.5), and
- (b) Shows the correlation for ASTM Method E274 vs. Mu-Meter. (Visual Aid 29.5).

3.0 DETERIORATION OF SKID RESISTANCE

The skid resistance offered by a pavement surface to a vehicle tire is determined by three groups of parameters: those associated with the pavement surface, those associated with the vehicle and those associated with operating conditions.

3.1 Surface Texture

3.1.1 Macrotexture. Refers to the roughness of the pavement surface as a whole, It is generally influenced by the coarse aggregate in asphalt pavements and the texture finish of concrete pavements.

3.1.2 Microtexture. Refers to the fine-scale roughness contributed to by the asperities of aggregate particles on the pavement surface.

3.1.3 Texture Measurements. Several methods are currently in use to determine the surface texture characteristics. Most of these generally involve using a known quantity of material and filling the grooves or holes in the surface.

(a) outflow meter
(b) sand patch,
(c) putty impression, and
(d) others.

3.2 Skid Resistance Parameters

3.2.1 Associated with Pavement Surface. Considerations of time/traffic/climate-based changes in friction resistance require periodic measurements. Various changes in the nature of the pavement surface should be recognized as possible contributing factors to such friction resistance changes.

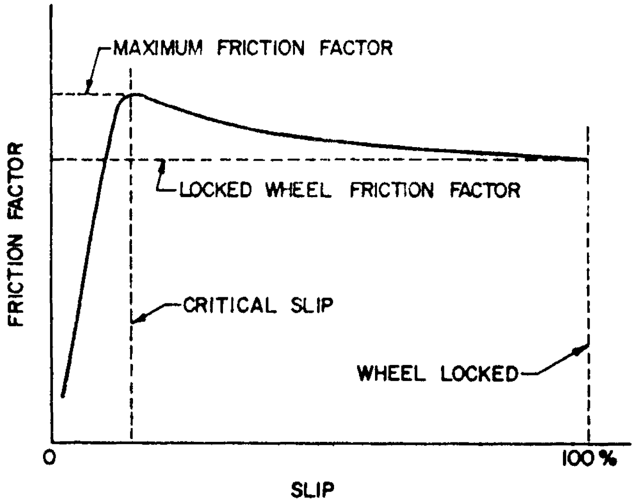
(a) wear (Visual Aid 29.6),
(b) polishing,
(c) texture depth,
(d) water depth,
(e) pavement cross-slope,
(f) bleeding of asphaltic pavement,
(g) compaction, rutting, particle loss,
(h) contamination (Visual Aid 29.7), and
(i) roughness.

3.2.2 Associated with the Vehicle. Obviously there are certain factors pertaining to the maintenance and operation of all vehicles which contribute to the degree of friction resistance available to the vehicle.

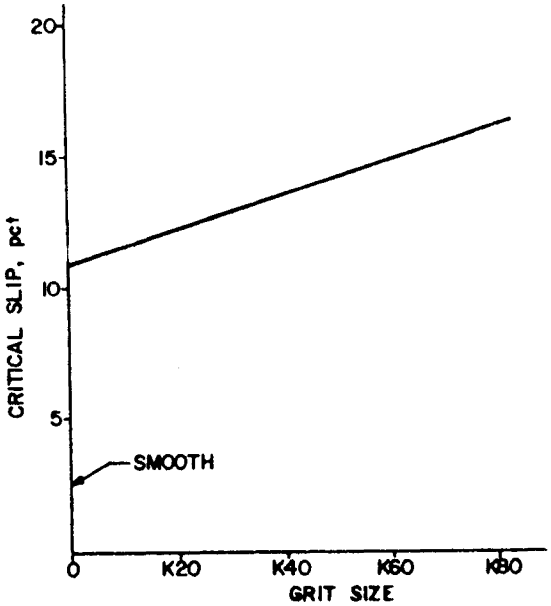
(a) rubber properties,
(b) tread pattern,
(c) tire pressure, and
(d) temperature.

LESSON OUTLINE
SKID MEASUREMENT METHODS

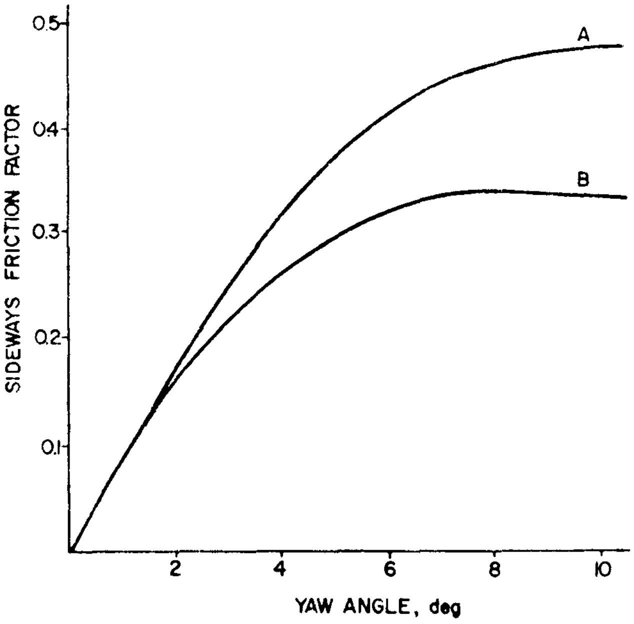
VISUAL AID	TITLE
Visual Aid 29.1.	Friction factor as a function of slip.
Visual Aid 29.2.	Change of critical slip with texture.
Visual Aid 29.3.	Sideways friction factor versus yaw angle.
Visual Aid 29.4.	Skid number versus stopping distance friction factor correlation.
Visual Aid 29.5.	Two-wheel trailer versus Mu Meter correlation.
Visual Aid 29.6.	Deterioration of skid resistance with exposure to traffic.
Visual Aid 29.7.	Change of skid resistance during shower.



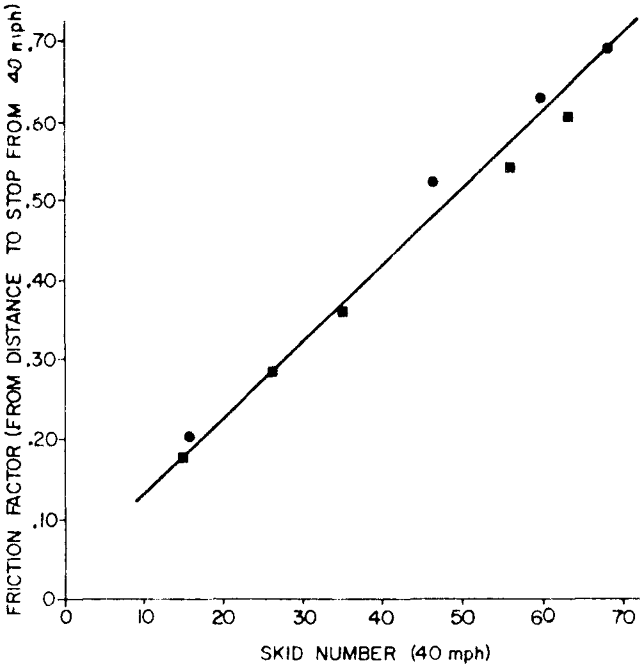
Visual Aid 29.1. Friction Factor as a Function of Slip.



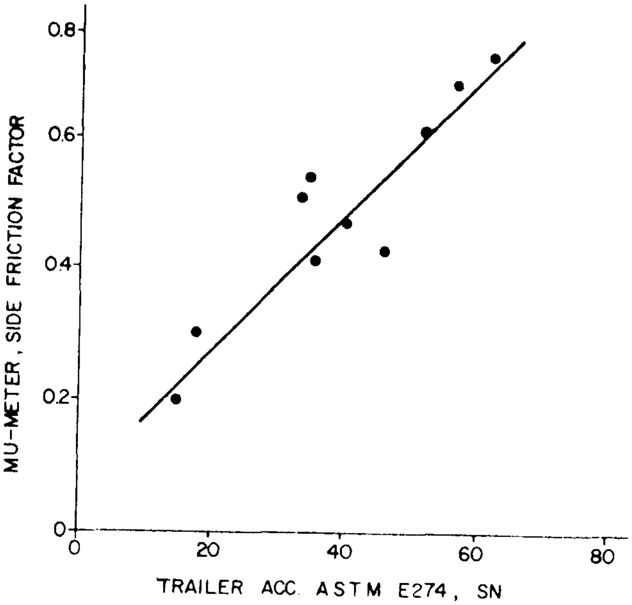
Visual Aid 29.2. Change of Critical Slip with Texture.



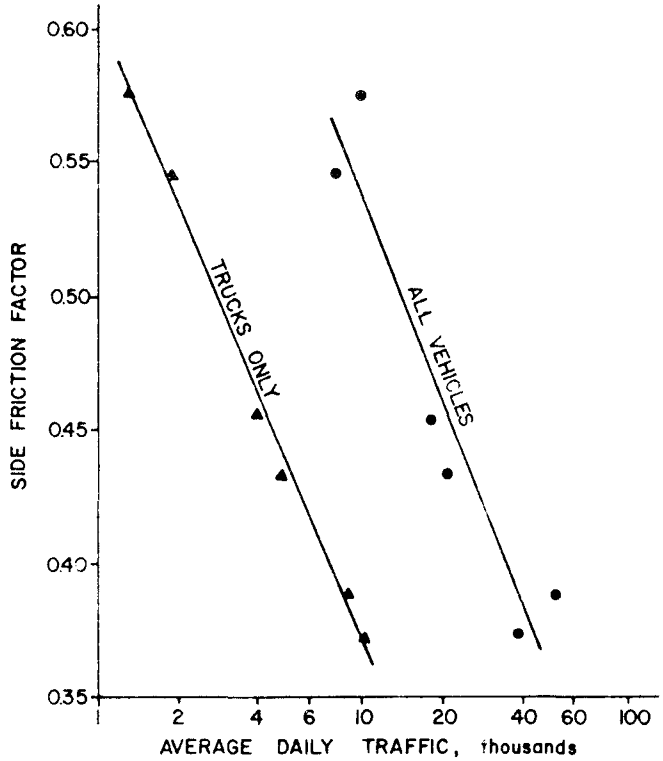
Visual Aid 29.3. Sideways Friction Factor vs. Yaw Angle.



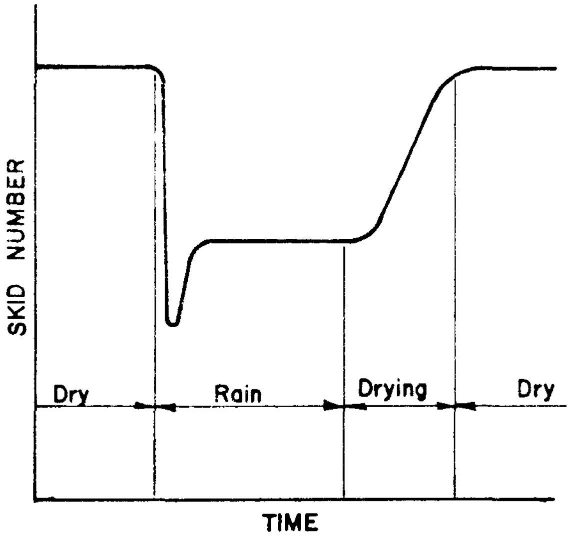
Visual Aid 29.4. Skid Number vs. Stopping Distance Friction Factor Correlation.



Visual Aid 29.5. Two-Wheel Trailer vs. Mu Meter Correlation



Visual Aid 29.6. Deterioration of Skid Resistance with Exposure to Traffic.



Visual Aid 29.7. Change of Skid Resistance During a Shower

INSTRUCTIONAL TEXT

USE OF FRICTION (SKID RESISTANCE) MEASUREMENTS IN A PAVEMENT MANAGEMENT SYSTEM (PMS)

By

Frank Carmickael

Technical Memorancum
April 20, 1982

The purpose of skid resistance measurements is to evaluate the safety of the pavements being measured. These measurements are, however, also important components in overall systematic pavement management and pavement performance predictions.

BACKGROUND

Skid resistance measurements have been in use many years by highway agencies. NCHRP Report 37 (Ref 1) published in 1966 referenced 58 reports dating from 1943 concerning the measurement and use of skid resistance measurements, though most of the references were from the early 60s. Over the past 20 years many reports of research activities dealing with skid resistance have been published. Many of these references are summarized in NCHRP Synthesis Report 14 (Ref 2).

Most of the research has been directed at identifying frictional needs of the vehicles under various conditions or various devices to measure "skid resistance" either directly (locked wheel trailers, stopping distances, etc.) or indirectly (British Portable Tester, texture depth, etc.). Balmer (Ref 3) summarized the influence of pavement textures on several of the performance aspects of pavements, such as speed gradients, hydroplaning, skid numbers, accident rates, noise, and wear.

Researchers generally agree that a safety evaluation of pavement requires some form of skid measurement and some measure of texture depth. Skid resistance is defined as the force that develops when a tire that is prevented from rotating slides along the pavement surface (Ref 4). Obviously, one would expect that the "rougher" the pavement surface is, the greater the resistance to skidding will be. However, as will be discussed later, there are several parameters that can influence the skid characteristics of a pavement surface and lead to deterioration of skid resistance.

The skid resistance quality of a pavement surface can be characterized by its surface texture. Pavements with a smooth surface texture generally can be expected to have lower values of skid resistance than those with a rougher texture. Two terms are used to discuss surface texture; macrotexture and microtexture. Macrotexture refers to the roughness of the pavement surface as a whole. It is generally influenced by the coarse aggregate in asphalt surfaces and the texture finish in concrete pavement. On the other hand, microtexture refers to the fine-scale roughness contributed by individual small asperities of aggregate particles on the pavement surface which may not be discernible to the eye, but apparent to the touch (grittiness). A pavement surface may exhibit good macrotexture and yet have poor microtexture, or the reverse may be true.

Regardless of pavement type, dry pavements typically exhibit satisfactory and similar skid resistance. The main concern for good skid resistance occurs when the surface is wet due to hydroplaning problems. The term hydroplaning is defined as a loss of tire traction due to the presence of water at the tire-pavement interface.

The formula for the commonly used skid number, SN, is as follows:

$$SN = 100\ f$$

where f = friction factor

The friction factor, f, is analogous to the coefficient of friction, Mu, commonly defined in solid mechanics as follows:

$$f = F/L$$

where

F = Frictional resistance to motion in the plane of an interface
L = Load acting perpendicular to the interface

The friction factor depends on the contact area between the pavement surface and the me[illegible] tire, and [illegible] is, [illegible] rolling, slipping, or skidding or the tire, particularly when water is present. A high coefficient of friction is indicative of good skid resistance. The maximum value of the coefficient of friction is 1.0

SKID RESISTANCE MEASUREMENT TECHNIQUES

Skid resistance measurements depend on many pavement surface, tire, and environmental characteristics such as existing texture, rut depth, dust accumulation, drainage path length, measurement speed, specific measurement location, water film thickness, temperature, wind velocity, tire wear, tire inflation, etc. There are a number of different measurement techniques for measuring skid resistance as follows;

1. Locked-wheel Methods - ASTM Type Skid Trailers
2. Yaw Mode Methods - Mumeter Trailers
3. Portable Field Testers - British Portable Tester
4. Slip Mode Methods - Swedish Road Research Skidometer

These equipment are summarized in Table 1 (Ref 5).

General State of the Art

Each type of skid resistance measurement equipment measures a different aspect of the friction developed between the test tires and the pavement surface. Therefore, it should not be expected that there will be a 1:1 correlation between the results obtained with different types of testers. In an attempt to calibrate and correlate friction measurement devices, two primary reference surfaces have been constructed at Texas Transportation Institute near College Station, Texas, and Ohio State University in East Liberty, Ohio. Identical materials and construction methods were used at these calibration facilities.

Many highway agencies make some form skid measurement, some on a more or less periodic basis and others on an as needed basis. Appendix A shows an excellent summary of the skid resistance measurement equipment used by a majority of the states.

The most generally accepted method of skid testing is with a locked wheel skid trailer. While the trailer is towed at a constant speed, either one or both of its wheels are locked, and the force required to pull the trailer is recorded. ASTM Test Method E274 covers this type of testing. This test yields a skid number (SN) which is the ratio of the normal and tangential forces on the test tire multiplied by 100. Because skid resistance varies with speed, the skid number is reported along with the speed of the test.

Most agencies use a locked wheel trailer at 40 mph to obtain a skid number (SN_{40}) for comparison purposes. SN_{40} values are relatively easy to obtain and do not in most instances required blocking of a traffic lane. It is generally agreed that a SN_{40} value of 40 (Ref 1) is required for most vehicle manuevers at highway speeds of 50 to 60 mph.

Another notable skid measuring device is the Mumeter. This trailer mounted device contains two smooth treaded tires angled from the direction of travel. The wheels are angled in equal and opposite directions so that the trailer will travel in a straight line. This trailer has been correlated with other skid measuring devices (Ref 6 and 7) and is also recommended by the FAA for use in characterization of runway friction (Ref 8). Measurements with the Mumeter are said to be made in the yaw-mode due to the angle on the measurement tires.

Another type of skid measurement device widely used in both the laboratory and field is the British Portable Tester. This is a pendulum device which characterizes the frictional properties of a surface by the energy loss developed when a small rubber shoe is slid along the surface.

Equipment	Normal Operating Speed	Disadvantages	No. of Operators	What does it Measure	Data Output	Output Units	Measurement Method	Implementation
Locked-2 wheel trailer testers	40-50 mph*	Length of unit; trailer plus vehicle; availability of water; initial cost	1 or 2	Relative pavement frictional resistance	Paper tape; magnetic tape or analog tape	Skid number (SN) or torque force	ASTM E-274	Survey, evaluation of materials, and construction practices; skid accident reduction; traffic speeds
Single, locked wheel trailer (PSU** Design)	40-50 mph	Availability of water; vehicle cannot be used for other purposes; initial cost	1 or 2	Relative pavement frictional resistance	Paper tape; magnetic or analog tape	Skid number (SN)	ASTM E-274	Survey, evaluation of materials, and construction practices; skid accident reduction; traffic speeds
MU Meter (12)	10-85 mph	Availability of water	1	Continuous record of side slip force	Continuous graphical	Coefficient of friction (Mu)	ASTM E-***	Evaluation of coefficient of friction (Mu) for runway surfaces
British Pendulum Tester	Stationary (6-8 mph)	Low speed of operation; safety; correlation with skid number	1	Relative pavement frictional resistance (micro-friction)	Gauge reading	British portable (tester) number (BPN)	ASTM E-303	Evaluation of materials and construction practices; laboratory tests and polish value measurements
SCRIM (13)	25-50 mph	High initial cost; availability of water	2	Continuous record of skid resistance (or intermittent values)	Magnetic or punched tape	Sideway force coefficient (SFC)	SFC 20° angle slip	Research survey
Skidometer (13)	37-75 mph	Availability of water	1	Continuous coefficient record	Digital printer	Braking force coefficient (BFC)	BFC, locked or 13% slip	Airfield runways, texture depth effectiveness on high-speed roads

* One mph ≈ 1.61 km/h.

** The Pennsylvania State University

***Pending

Table 1 Summary Evaluation of Friction Testing Equipment and Methods (after Ref 5).

This test method is covered under ASTM E303. Correlations with other skid measuring devices is not outstanding. This may be attributed to the relatively small sample of surface that the striker contacts.

A natural method of measuring the skid resistance of a pavement surface is to lock a vehicles' wheels and measure the stopping distance. This test can be performed with either all four wheels locked or two diagonally opposite wheels locked. ASTM test methods E445 and E503 cover this testing in the full wheel and diagonal wheel lock modes, respectively. Treaded tires meeting standard specification ASTM E501 are used for this type of skid testing. This type of testing can, of course, be dangerous to the operator and requires complete traffic control.

USE OF SKID RESISTANCE MEASUREMENTS

At the 1978 Pavement Management Workshop in Tumwater, Washington (Ref 11), a majority of the participants strongly agreed that (1) safety attributes should be rated and that (2) safety aspects should include skid resistance. Of the states represented at the Tumwater Workshop, seven states used ASTM skid trailers and two states use MuMeters to measure skid resistance. Skid resistance factor was the most common factor used to estimate the "safety" of highway pavements. In many cases, skid resistance measurements are related to or used in conjunction with accident frequency and/or severity.

There are no rigidly fixed standards for skid resistance of highway pavements. This is due to legal implications placed on highway authorities by such standards.

Skid resistance data can be used for the following pavement management purposes (Ref 13):

1. Identifying areas of excessive slipperiness,
2. Planning maintenance (or future rehabilitation), and
3. Evaluating various types of materials and new construction practices.

Most pavement management systems choose to treat skid resistance separately. This course is reasonable since skid resistance has little or no effects on other pavement characteristics with the possible exception of wet weather accidents.

Network Evaluation

As stated earlier, in general highway engineers have used skid resistance measurements as a highly related, but separate factor in PMS. A representataive example, is the State of Florida. Florida reported at the Tumwater PMS Conference (Ref 11), "safety improvments, including skid overlay, highway accident location , and roadside obstacle elimination programs are considered in a separate procedure." This separate system has been developed to identify and investigate Safety Improvemnet Projects, including skid, high accident, and roadside obstacle problems. The 20 to 30 pavements with the highest Safety Ratio, not otherwise scheduled for improvement, within two years are chosen as candidates. All sections with Skid Number (SN) less than 36 are chosen as well as those with SN less than 41 which have greater than average accident rates. All sections with reports of vehicles striking roadside objects are investigated for possible improvements. A benefit-cost analysis is done to rank alternatives. A recent report on pavement condition measurement needs and methods (Ref 5) showed conceptually how skid measurements might be considered in light of structural and roughness evaluations (see Figure 2).

Performance Predictions

Skid resistance evaluation, especially for the purpose of assessing future rehabilitation needs, should consider changes on a time and/or traffic basis; as well as on a climatic effect basis. The latter can involve both short and longer periods of time (i.e., rainfall or icing of a short duration, versus seasonal changes in climate).

Consideration of time/traffic/climate based changes in skid resistance requires periodic measurements, preferably on a mass inventory basis for investment programming purposes. Various changes in the nature of the pavement surface should be recognized as possible contributors to such skid resistantc changes, and they include the following:

1. Porosity,
2. Wear (i.e., due to studded tires),
3. Polishing of surface aggregate,
4. Rutting (due to compaction, laterial distortion, or studded tire wear),
5. Bleeding,
6. Contamination (i.e., rubber, oil, water, etc.)

The skid resistance provided by a pavement surface is related to its texture. The texture contributes to skid resistance by generation of friction through adhesion and hysteresis effect of the tire rubber and by providing water escape channels. For frictional characterization, surface texture is subdivided into two components, the micro-texture and macro-texture. Micro-texture is a property of the surface of the aggregate and is sometimes described as gritty and rounded. The macro-texture is a surface mix property and is related to aggregate gradation, shape, and the amount of binder present. It is common to speak of macro-texture as being coarse or fine.

SUB-SYSTEM 1 - Evaluation of Friction Resistance

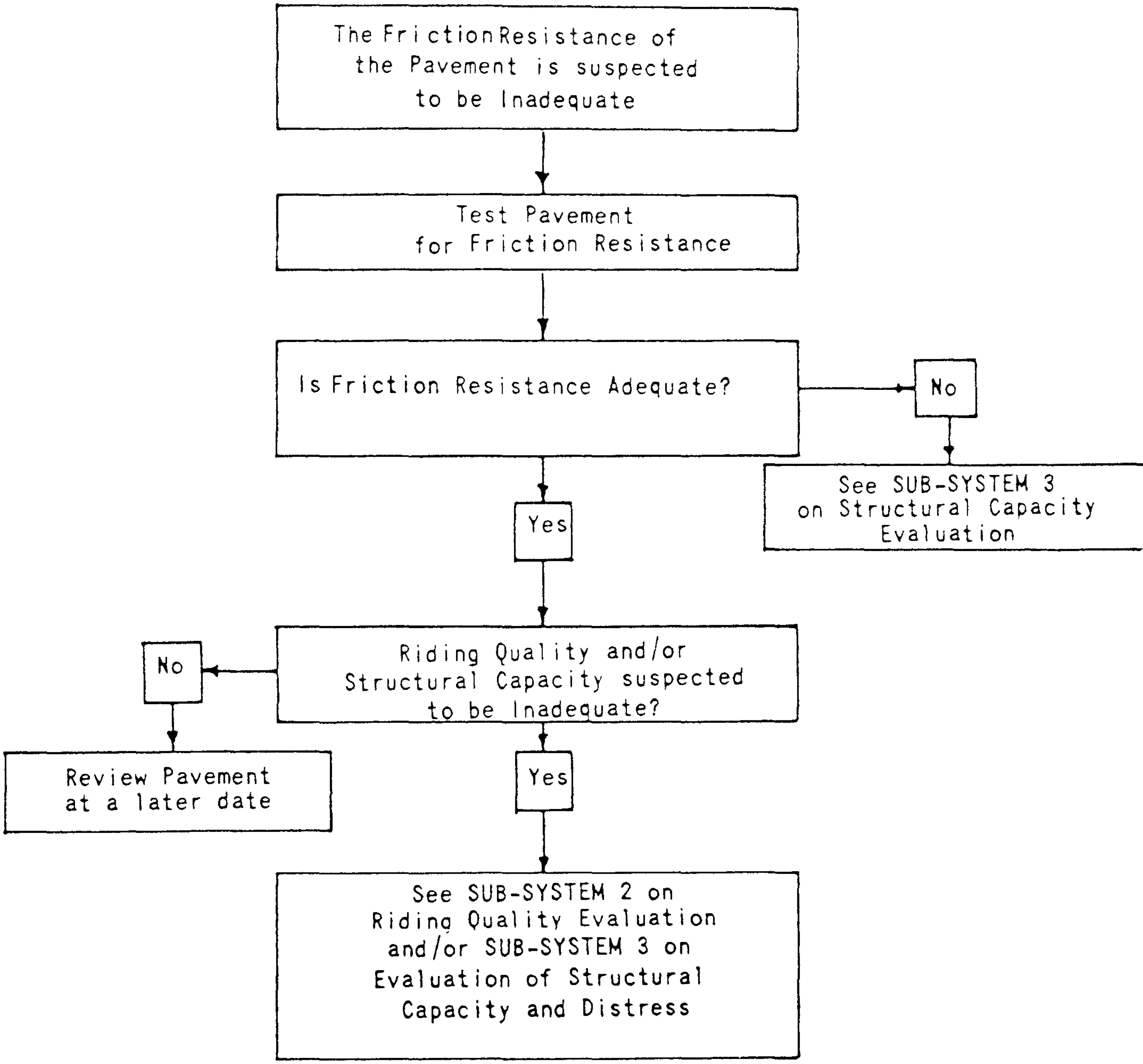


Figure 2 Example of Proposed Skid Evaluation Flow in a PMS (after Ref 5)

Figure 3 gives a schematic illustration of micro and macro-textrue and indicates its effect on skid number as a function of speed. Note the the gritty micro-texture exhibits high resistance properties at low speeds and as the speed increases the effect of the coarse macro-texture becomes more dominant. Thus, skid resistant feature of the pavement surface are highly dependent upon the aggregate characteristics. Pertinent aggregate characteristics include wear properties, texture, shape, gradation, and blending.

In addition to friction measurements, surface texture depth measurements are commonly conducted to "get a feel" for the skid qualities of a pavement surface. Again, there are several ways to measure average texture depth (ATD), and there is variation between measurements. Some of the methods used to measure texture depth include:

1. Outflow Meter
2. Sandpatch Method
3. Grease Smear Method
4. Putty Method
5. Photogrammetric Method
6. Profilograph
7. Texturemeter

One of the problems associated with texture measurements is the correlation between texture depth and skid number. Texture depth measurements usually represent only a very small sampling of the area over which friction measurments are made. Cost for some methods may prohibit adequate sampling. Also, difficulty arises in that texture cannot be described by a single characteristic (such as depth). It is not only the escape of water from the tire contact area (controlled by texture), but also the nature of contact between the tire tread and the surface which affects the skid number measured. Due to the latter effects, correlation between texture measurements and skid measurement are not entirely accurate.

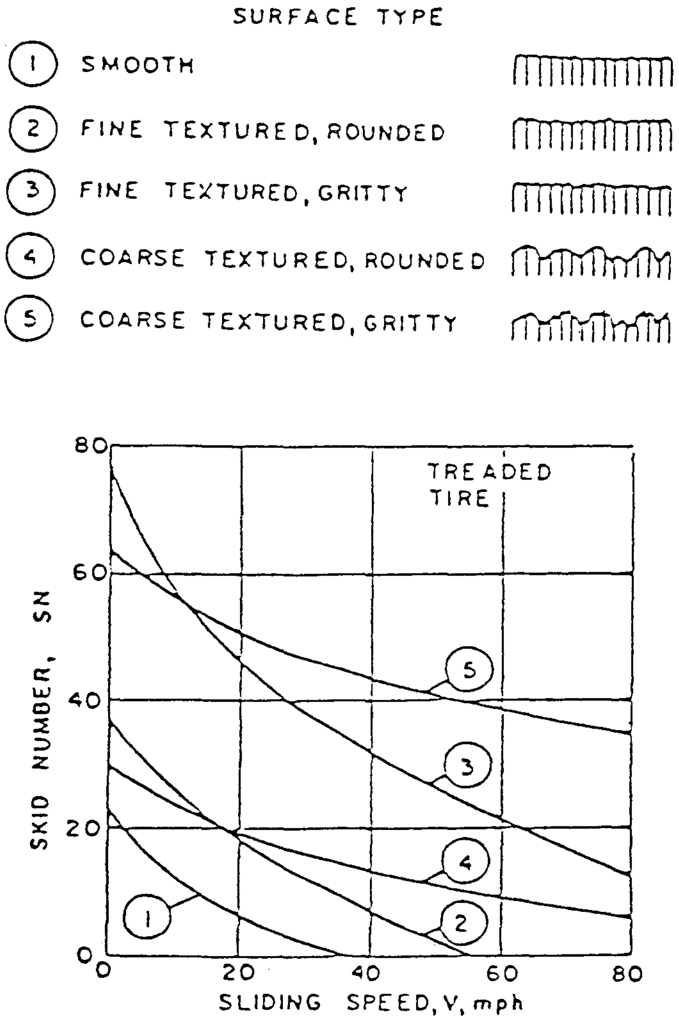


Figure 3 Road surface types and associated frictional characteristics at different speeds (after Ref.15)

As mentioned previously, there are several parameters which influence the skid resistance of pavement surfaces and, consequently, the measured value of friction;

1. surface contamination
2. pavement deterioration
3. seasonal variation

Surface contamaination is any foreign material such as water, snow, dirt, dust, oil, rubber, etc. , on a pavement surface which causes a reduction in surface friction. The presence of water on a pavement surface is probably the most common form of contamination. It significantly reduces the skid resistance of nearly every type of pavement.

Pavement deterioration occurs nonstop. Repeated traffic usage reduces pavement life by introducing various forms of distress (cracking, rutting, bleeding, surface wear, etc.).

Work by the Arizona Department of Highways indicates that there is seasonal variation in friction reading. Pavements that may have a satisfactory friction value at one time of the year could have an unsatisfactory value at another time of the year. Arizona found that the lowest skid level is reached during the summer months. For this reason, they conduct friction inventories during the summer months.

Though some attempts have been made to develop predictive equations for SN an ATD values, most have proved to be too limited to be of general value. For asphalt cement concrete (ACC pavements, changes in SN and ATD are generally related to the polishing of the coarse aggregate exposed at the surface. Various labotatory polishing techniques have been used to predict field performance. These measures have generally been found to be good indicators of the final or ultimate field polish value, but do not provide realiable information about interim performance such as rate of wear or polishing. For portland cement concrete (PCC) pavement, the SN and ATD values are a function of the texture imparted to the surface during the finishing process. The wear occurs rapidly in early life, but tends to reach a plateau that remains for a rather extended period of time.

To develop predictive mathematical models will require historical data which will be limited at best. Perhaps developing acceptable levels of SN_{40} and ATD for certain characteristics such as traffic and environment would be more appropriate at this point. These would be modified as a data base if developed and once a data base is obtained the predictive model would be more viable.

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APPENDIX A.

SUMMARY OF SKID SYSTEM FEATURES, COMPONENTS, AND OPERATIONS

(after Reference 5)

APPENDIX A

SUMMARY OF SKID SYSTEM[d] FEATURES, COMPONENTS, AND OPERATIONS

(after Reference 5)

State	Make	Year	Cost	Most Recent FHWA Evaluation Year	Water Tank Capacity Gal.	Force Sensors					Recorder						System Amplifier			Brake System										Data Reduction		
						Torque Tube	Gauged Axle	Force Transducers	Drawbar	Other	Strip Chart	Digital	Heat	Light	Printer	Other	Recorder External	Recorder Internal	No. of Channels	Hydraulic	Electric	Air/Hydraulic	Other	Automatic Timed	Manual Operated	Both Wheel Locked	Left Wheel Only	Right Wheel Only	Either Locked	Manual	Semi-Automatic	Fully Automatic
Alabama	Soiltest	1969	33,880	1976	510[a]	X					X		X				X		X			X		X		X			X			X
Alaska	b																															
Arizona	Did not respond to questionnaire																															
Arkansas	University of Arkansas	1976	30,000	1976	430	X										X						X							X			X
	University of Arkansas	1977	35,000	1977	430	X										X						X							X			X
California	K. J. Law	1969	90,000	1975	230			X			X			X			X		4			X		X	X	X					X	
	Soiltest	1970	40,000		500	X					X					X		X	2			X		X		X					X	
	Cox & Sons	1977	60,000	1977	230			X			X	X			X		X		1			X		X	X	X						X
Colorado	Soiltest	1969	23,000	1975	450	X					X	X					X	X	2		X			X	X		X			X	X	
Connecticut	Testlab	1970	22,295	1977	550	X					X							X	2			X	X	X		X			X	X		
Delaware	Soiltest	1971	28,000	1975	350	X					X				X	X			2		X			X	X						X	
District of Columbia	c																													X		X
Florida	In-house	1965	30,000		400					X												X		X						X		
	K. J. Law	1973	46,000	1976	240			X			X			X			X		4	X				X			X		X	X		
	K. J. Law	1973	50,000	1975	240			X			X			X			X		4	X				X			X		X	X		

a 1 gallon = $0.003785 m^3$

b Contracted Washington State to survey State System

c Testing has been performed by National Bureau of Standards, Maryland Roads Commission and FHWA

d See Indiana State Highway Commission Report (10)

APPENDIX A (Con't)

SUMMARY OF SKID SYSTEM FEATURES, COMPONENTS, AND OPERATIONS

(after Reference 5)

State	Make	Year	Cost	Most Recent FHWA Evaluation	Water Tank Capacity	Force Sensors					Recorder						System Amplifier			Brake System										Data Reduction		
						Torque Tube	Gauged Axle	Force Transducers	Drawbar	Other	Strip Chart	Digital	Heat	Light	Printer	Other	Recorder External	Recorder Internal	No. of Channels	Hydraulic	Electric	Air Hydraulic	Other	Automatic Timed	Manual Operated	Both Wheel Locked	Left Wheel Only	Right Wheel Only	Either Locked	Manual	Semi-Automatic	Fully Automatic
		Year	Cost	Year	Gal.																											
Florida (Cont)	K. J. Law	1977	63,000		300			X									X		4			X		X			X		X	X		
Georgia	Soiltest ML 350H	1969	25,000	1975	450	X[a]				X	X				X[b]		X		4			X		X					X		X	
	Soiltest ML355H-K	1975	45,000	1977	320			X			X				X[b]		X		4			X		X					X		X	
Hawaii	K. J. Law	1975	65,000	1977	250			X			X	X		X	X		X		4	X				X	X		X					X
	Soiltest ML-350	1970	26,500	1975	470	X					X							X	2			X		X		X			X	X		
Illinois	Soiltest	1968	23,000	1975	450	X					X	X					X		2		X			X					X[d]		X	
	Soiltest[c]	1977	67,244		300					X	X	X					X		2			X		X					X[d]			X
Indiana	Fabricated Machine Corp	1976	46,850	1976	300							X			X		X		4			X		X					X			X
	Indiana State Hwy Commis	1977	26,000		300							X			X		X		6	X				X			X					X
Iowa	Iowa Dept. of Transp.	1968	NA		250					X						X	X		2		X			X			X		X	X		
	K. J. Law Assoc.	1972	NA	1976	250			X						X			X		2	X				X			X		X		X	
	K. J. Law Assoc.	1975	NA	1976	250			X				X		X	X		X		4			X		X			X		X			X
Kansas	K. J. Law Model 1270	NA	60,000	1975	230		X	X			X	X		X	X		X		4	X				X			X	X	X		X	
Kentucky	K. J. Law Engr., Inc.	1969	34,120[e]	1976	230			X			X			X			X		5[f]			X		X	X	X					X	
	K. J. Law Engr., Inc.	1976	58,440	1976	300			X				X			X		X		5			X		X	X		X				X	

[a] Since report Torque Tubes replaced by Stop Bar

[b] Pressurized ink

[c] Scheduled for delivery in Mid-August 1977

[d] Operator selects either right or left wheel

[e] Skid number computer and averager added in 1973($2240)

[f] Can use 7 Channels

APPENDIX A (Con't)

SUMMARY OF SKID SYSTEM

FEATURES, COMPONENTS, AND OPERATIONS

(after Reference 5)

				Most Recent FHWA Evaluation	Water Tank Capacity	Force Sensors					Recorder						System Amplifier			Brake System										Data Reduction		
State	Make	Year	Cost	Year	Gal.	Torque Tube	Gauged Axle	Force Transducers	Drawbar	Other	Strip Chart	Digital	Heat	Light	Printer	Other	Recorder External	Recorder Internal	No. of Channels	Hydraulic	Electric	Air/Hydraulic	Other	Automatic Timed	Manual Operated	Both Wheel Locked	Left Wheel Only	Right Wheel Only	Either Locked	Manual	Semi-Automatic	Fully Automatic
Louisiana	La.D.O.T.D.	1974	25,000	1976	300		X					X			X						X			X			X					X
	La.D.P.T.D.	1976	26,500		300		X				X			X							X			X			X			X		
Maine	Maine Dept. of Transp.	1973	37,000	1974	150		X				X		X					X	3		X								X	X		
Maryland	Md. State Hwy. Adm.	67-68	31,716[a]	1976	500		X				X							X	4				X	X	X		X	X		X	X	X
	K.J.Law, Inc. Model 965A	1974	113,495 (Bid)	1976	300			X			X							X	4			X		X	X		X	X		X	X	X
	K.J.Law, Inc. Model 1270	1976	92,190 (Bid)	1976	300			X			X							X	4			X		X			X	X		X	X	X
Massachusetts	Stevens Institute	1973	45,000	1977	250			X			X	X	X				X		4	X				X	X				X	X		X
Michigan	K.J.Law Engineers	1971	80,000	1976	230			X			X	X		X	X		X		8			X		X	X	X	X	X		X		X
	Mich.Dept.of St.Hwys.Tran	1957[b]	2,000		400					X		X			X		X					X		X	X	X						X
Minnesota	Soiltest	1969	30,205	1975	470	X					X							X	2			X		X	X				X	X		
	K.J.Law Engineers	1974	59,800	1975	246			X			X	X		X	X		X		4	X				X	X		X					X
Mississippi	Soiltest	1973	53,000	1976	350			X			X						X		4			X		X	X				X	X		
Missouri	K.J.Law Engineers	1969	85,000[c]	1977	230			X			X	X		X	X		X		8			X		X	X	X	X	X		X	X	
Montana	K.J.Law	1972	52,000	1975	230			X			X			X			X		6				X	X					X	X		
Nebraska	K.J.Law Engr.Inc.	1974	41,500	1977	250			X			X			X			X		5				X	X	X		X			X		

a Does not reflect special tools & 20% S.R.C. overhead.

b A replacement is currently under construction.

c A data-longer for SN was installed in 1976 for $11 600.

APPENDIX A (Con't)

SUMMARY OF SKID SYSTEM

FEATURES, COMPONENTS, AND OPERATIONS

(after Reference 5)

				Most Recent FHWA Evaluation	Water Tank Capacity	Force Sensors					Recorder						System Amplifier			Brake System										Data Reduction		
State	Make	Year	Cost	Year	Gal.	Torque Tube	Gauged Axle	Force Transducers	Drawbar	Other	Strip Chart	Digital	Heat	Light	Printer	Other	Recorder External	Recorder Internal	No. of Channels	Hydraulic	Electric	Air/Hydraulic	Other	Automatic Timed	Manual Operated	Both Wheel Locked	Left Wheel Only	Right Wheel Only	Either Locked	Manual	Semi-Automatic	Fully Automatic
Ohio (Cont)	K.J. Law	1975	60,000	1977	250			X			X	X		X	X		X		5		X			X	X		X					X
Oklahoma	Oklahoma Dept.of Tran	NA	NA	1969	300		X				X							X	2		X			X			X			X		
Oregon	K.J.Law Engrs.,Inc.	1974	48,000[a]	1975	230			X				X			X		X		4	X				X	X	X						X
Pennsylvania	Wald Industs	1969	49,076	1976	250					X	X			X				X	4			X		X	X					X		
	Soiltest,Inc	1969	34,795	b	250					X	X			X				X	4			X		X	X					X		
	Soiltest,Inc	1973	36,050	1975	250					X	X			X				X	4			X		X	X					X		
	Soiltest,Inc	1973	36,050	1975	250					X	X			X				X	4			X		X	X					X		
Rhode Island	K.J.Law	1973	46,890	1976				X			X								4	X				X			X	X	X	X		
South Carolina	Soiltest,Inc	1972	36,000[c]	1976	450	X					X							X	2			X		X					X	X		
South Dakota	K.J.Law	1975	57,765	1976	300			X			X	X		X	X			X	4			X	X	X	X		X					X
Tennessee	University of Tennessee	1968	30,000	1975	500					X	X		X					X	2			X		X	X	X				X		
	K.J.Law Model 1270	NA	64,000	1976	230			X			X	X		X			X		4			X		X	X	X					X	
Texas	Fabricatedin Dept'al shop	1974	25,000	1976	300	X					X[d]	X			X	X	X		8	X				X	X		X	X[e]				X
	Fabricatedin Dept'al Shop	1974	25,000	1976	300	X					X[d]	X			X	X	X		8	X				X	X		X	X[e]				X
	Fabricated i Dept'al Shop	1974	25,000	1976	300	X					X[d]	X			X	X	X		8	X				X	X		X	X[e]				X

[a] Does not include estimated $4,000 for recorder amplifier for recorder, & air bearing calibrator system.

[b] Scheduled for 4/18/77.

[c] Approximately $2500 worth of additional equip. added.

[d] Used as auxiliary device.

[e] Rarely used.

APPENDIX A (Con't)

SUMMARY OF SKID SYSTEM

FEATURES COMPONENTS AND OPERATIONS

(after Reference 5)

				Most Recent FHWA Evaluation	Water Tank Capacity	Force Sensors					Recorder						System Amplifier			Brake System										Data Reduction		
State	Make	Year	Cost	Year	Gal	Torque Tube	Gauged Axle	Force Transducers	Drawbar	Other	Strip Chart	Digital	Heat	Light	Printer	Other	Recorder External	Recorder Internal	No. of Channels	Hydraulic	Electric	Air Hydraulic	Other	Automatic Timed	Manual Operated	Both Wheel Locked	Left Wheel Only	Right Wheel Only	Either Locked	Manual	Semi Automatic	Fully Automatic
Nevada	K.L. sic, Engrs., Inc.	1974	57,950	1977	230			X			X			X			X		5[a]			X		X	X				X			X
New Hampshire	b																															
New Jersey	Stevens Inst of Technolog	[c] 1968	50,000			X																										
	Stevens Inst of Technolog	1974	34,000	1976	350	X					X		X					X	2		X			X	X		X			X		
	Stevens Inst of Technolog	1974	34,000	1976	350	X					X		X					X	2		X			X	X		X			X		
New Mexico	Soiltest MU 350[d]	1970	26,330	1976	480	X					X								2		X			X					X	X		
New York (Engr Research)	N.Y. State Dept of Tran	NA	NA	1974	275					X	X							X	2				X	X	X		X			X		
New York (Mat'ls Bureau)	NYS DOT-Eng Res & Devpm	1974	NA	1976	275					X		X							5			X		X			X				X	
	K.J. Law Engineers[e]	1977	88,700		300			X				X			X				5			X		X					X			X
North Carolina	N. Carolina Equip. Dept.	1968	25,000		500		X				X							X	2	X				X	X		X	X		X		
	N. Carolina Equip. Dept.[f]	1975	18,000	1976	250		X				X	X			X			X	2	X				X	X		X	X				X
	N. Carolina Equip. Dept.[g]		35,000		250		X				X	X			X			X	2	X				X	X		X	X				X
North Dakota	K.J. Law	1975	58,000	1977	230			X			X	X			X			X	5	X				X			X					X
Ohio	K.J. Law	1968	85,000	1977	250	X					X			X			X		7			X		X	X		X	X	X	X		
	K.J. Law	1975	60,000	1977	250			X			X	X		X	X		X		5		X			X	X		X					X

a. Presently, one of these channels is not used.

b See Main DOT information.

c Taken out of service during last testing season.

d Torque tube was required to be replaced prior to acceptance.

e Delivery expected in May 1977

f Replaced 1968 Model.

g Now under construction.

APPENDIX A (Con't)

(after Reference 5)

SUMMARY OF SKID SYSTEM

FEATURES, COMPONENTS, AND OPERATIONS

				Most Recent FHWA Evaluation	Water Tank Capacity	Force Sensors					Recorder						System Amplifier			Brake System										Data Reduction		
State	Make	Year	Cost	Year	Gal.	Torque Tube	Gauged Axle	Force Transducers	Drawbar	Other	Strip Chart	Digital	Heat	Light	Printer	Other	Recorder External	Recorder Internal	No. of Channels	Hydraulic	Electric	Air Hydraulic	Other	Automatic Timed	Manual Operated	Both Wheel Locked	Left Wheel Only	Right Wheel Only	Either Locked	Manual	Semi-Automatic	Fully Automatic
Texas (Cont)	Fabricated in Dept'al Shops	1974	25,000	1976	300	X					X[h]	X			X	X	X		8	X				X	X		X	X[i]				X
Utah	M.L. Aviation Co. LTD	1969	5,000[j]	1976	325						X	X																		X		
Vermont	a																															
Virginia	VA Hwy & Trans Res. Council	1967	100,000		270					X	X								2				X	X	X		X			X		
	K.J. Law Engrs., Inc.	1974	48,227	1975	230			X			X	X			X				4		X			X	X		X			X	X	X
	K.J. Law Engrs. Inc. b	1977	65,000		300			X			X	X			X				4			X		X	X					X	X	X
Washington	Soiltest	1969	28,000	1976	450	X					X						X		3		X			X			X	X[c]		X		
	K.J. Law Engrs. Inc.	1974	58,000	1976	300			X			X	X		X	X		X		4	X				X	X		X					X
West Virginia	Soiltest	1969	40,000	1976	250	X					X	X	X				X	X	4			X		X		X					X	
	K.J. Law	1976	123,000	1976	250			X			X	X		X	X		X		NA			X		X		X						X
Wisconsin	Soiltest	1969	33,000[d]	1973	500	X					X						X	X	2			X		X		X[e]	X[f]			X		
Wyoming	M.L. Aviation MAidenhead Eng	1969	5,688 g		280					X	X							X	NA						X					X		

a The Vermont Dept. of Hwys does not own a skid trailer, but has a rental agreement with the FHWA in Arlington, VA to obtain measurements and tabulate basic data.

b On order

c Right wheel lock-up capability not installed at this time.

d Does not include numerous modifications that had to be made.

e Originally

f Presently

g Approximately 8200 has been spent for a new frame.

h Used as auxiliary device.

i Rarely used.

j Bought without watering system.

LESSON OUTLINE

DEVELOPING A COMBINED INDEX FOR "OVERALL PERFORMANCE" - OBJECTIVE FUNCTION

Instructional Objectives

1. To help the student understand the reason for conducting pavement evaluation, and to help him realize its importance in the overall pavement management process.
2. To explain to the student the significance of developing a combined index.
3. To review the various approaches to developing a combined index.

Performance Objectives

1. The student should be able to understand and explain the various purposes of pavement evaluation.
2. The student should develop an understanding of the various approaches for formulating a combined index.

Abbreviated Summary	Time Allocations, min.
1. Pavement Evaluation - An Overview	20
2. Developing a Combined Index	30
	50 minutes

Reading Assignment

1. Haas & Hudson - Chapter 7
2. Instructional Text

LESSON OUTLINE

DEVELOPING A COMBINED INDEX FOR "OVERALL PERFORMANCE" - OBJECTIVE FUNCTION

1.0 PAVEMENT EVALUATION - AN OVERVIEW

1.1 Introduction (Slide 30.1)

Pavement condition and performance has been a major concern in pavement management. Pavement condition involves four main components such as riding comfort, load-carrying capacity, safety and aesthetics. Evaluating of the pavement condition provides information for assessing deficiencies of existing sections, and for identifying needs at both the network and the project levels of pavement management.

1.2 Purposes of Pavement Evaluation (Slides 30.2 & 30.3)

A pavement management system consists of the comprehensive set of activities that go into the planning, design, construction, maintenance, evaluation, and research of pavements. Three purposes of the pavement evaluation are:

(a) Checking whether the intended function and expected performance of pavements are being achieved.

(b) Providing information for planning rehabilitation for existing pavements.

(c) Providing information for improving the technology of design, construction, and maintenance.

1.3 Pavement Evaluation and the Need for a Combined Index

As indicated previously, one function of pavement evaluation is to check on the performance of existing road sections. This could be facilitated by developing an index that measures the overall performance or adequacy of a pavement section.

The development of an index can also facilitate comparisons between pavement sections for rehabilitation programming purposes. This helps to fulfill the second purpose of pavement evaluation which is to provide information for planning rehabilitation activities.

2.0 DEVELOPING A COMBINED INDEX

2.1 Components of Overall Performance Index (Slide 30.4)

A combined index provides an indication of the overall adequacy of a pavement section in terms of a combination of selected attributes such as riding quality and pavement conditions.

The recommended five components of the overall performance index are:

(a) roughness or serviceability index,

(b) pavement condition or distress index,

(c) structural index expressed in structural number (SN) or remaining life,

(d) skid number or friction coefficient, and

(e) maintenance cost index.

2.2 Alternative Approaches to Formulating a Combined Index

There is no single overall index because of different decision factors like traffic and different purposes such as maintenance program, rehabilitation program and safety program. A single number (a combined index) tends to lose the identity of individual factor.

2.2.1 Review of NCHRP Report No. NA - 3/1 - "Simplified Pavement Management at the Network Level". In NCHRP Report NA - 3/1, a methodology for developing an index that can be used for establishing rehabilitating priorities at the network level PMS is presented. This index, which is called PINDEX in the report, was developed using a procedure that involved the following steps:

(a) selection of pavement attributes to include in PINDEX,

(b) categorization of the selected pavement attributes (Visual Aid 30.1),

(c) establishment of numerical values for each category of the pavement attributes, and

(d) establishment of weighting factors with which to adjust calculated values of PINDEX (Visual Aid 30.2).

2.2.2 Unique Sums Approach.

(a) An alternative way of combining pavement attributes is by the method of unique sums, which has been used in Sweden. In this approach, pavement attributes are assigned numerical values in such a way that when these values are added together, the sum is unique (Visual Aids 30.3 and 30.4).

(b) Advantage - composite rating numbers can be readily broken down into the individual components; thus, information on the individual pavement attributes can be recovered.

(c) Disadvantage - numerical values obtained are highly non-linear, allowing no meaningful comparisons to be made between different degrees of pavement condition.

2.2.3 Utility Theory.

(a) Description - Basically, the method involves the assessment of utility functions which express a decision maker's preferences over different levels of selected attributes. The utility values calculated from the utility functions are expressed on a scale from 0 to 1.

(b) Advantages - Any number of pavement attributes may be combined using utility theory. The multi-attribute utility is expressed on a scale which allows differences in utilities to be directly compared from section to section.

(c) Disadvantages - unlike the unique sums approach, the individual utility values cannot be identified given the final multi-attribute utility value.

2.2.4 Delphi Technique.

(a) One other approach to formulating a prioritization index is through the Delphi technique. This technique has been used previously in the development of a data base for Texas, and has been recently applied to develop a basis for evaluating highway surface condition in the State of Maine.

(b) In this method, an attempt to achieve a consensus of opinions among a group of experts is made through cycles of intensive questioning interspersed with controlled opinion feedback. The technique was applied by the Department of Transportation of the State of Maine in an effort to establish weights for various severity levels of selected distress categories.

2.2.5 Rational Factorial Rating Method. (Slide 30.5)

(a) Description - The rational factorial rating method is a factorial based procedure for formulating a prioritization index. This method was first applied in a research study undertaken by the Center for Transportation of the University of Texas at Austin, for the Texas State Department of Highways and Public Transportation.

(b) Factorial design - an experimental design that is employed when the effects of two or more variables are being simultaneously studied; each level of each factor is used with each level of each other factor. In the study reported in Ref 4, the following variables were used (Visual Aid 30.5): (1) degree of distress, (2) present serviceability index, (3) traffic level, (4) amount of rainfall, and (5) amount of freeze-thaw. The effect of pavement type was also studied in another factorial design (Visual Aid 30.6). (Slides 30.6 - 30.8)

(c) In this method, numerous highway engineers are asked to provide priority ratings (using a specified scale) to selected combinations of the variables included in the factorial design. Using the ratings obtained, a regression equation can be established that can be used to estimate how rehabilitation priorities are set by highway engineers. (Slides 30.9 - 30.21)

(d) Advantages:

  (1) Significant variables can be identified.

  (2) Interaction effects between variables can be estimated.

  (3) The method can provide a better insight as to how decisions on priorities are made by highway engineers.

  (4) The method is a rational way of formulating an index for establishing rehabilitation priorities.

(e) Disadvantage - problem associated with keeping the design to a manageable size; requires judicious selection of a factorial design that keeps the number of variable combinations to a desirable limit, and still allows sufficient information to be gained.

LESSON OUTLINE
DEVELOPING A COMBINED INDEX FOR "OVERALL PERFORMANCE" - OBJECTIVE FUNCTION

Visual Aid 30.1. Categories of pavement attributes used in formulation of PINDEX.

A. Serviceability Category	PSI Range	Assigned Numerical Value
Very Good	3.8 - 5.0	6
Good	2.8 - 3.7	20
Fair	2.0 - 2.8	40
Poor	below 2.0	80

B. Fatigue Cracking Category	Severity	Extent (%)	Assigned Numerical Value
Excellent	Slight	10	2
Very Good	Slight	10-25	6
	Moderate or Severe	10	
Good	Slight	25-49	20
	Moderate or Severe	10-25	
Fair	Slight	50	40
	Moderate or Severe	25-49	
Poor	Moderate or Severe	50	80

Visual Aid 30.2. Example prioritization factors based on functional classification and average daily traffic.

Functional Classification	ADT	Factor
Interstate	high	1.00
	medium	0.95
	low	0.88
Principal Arterial	high (>15,000)	0.93
	medium (5,000 - 15,000	0.87
	low (<5,000)	0.80
Minor Arterial	high (>12,000)	0.83
	medium (4,000 - 12,000)	0.75
	low (<4,000)	0.68
Major Collector	high (>8,000	0.73
	medium (2,000 - 8,000)	0.65
	low (<2,000)	0.60
Minor Collector	high (>5,000)	0.60
	medium (1,000 - 5,000)	0.53
	low (<1,000)	0.45
Local	high (>3,000)	0.55
	medium (500 - 3,000)	0.45
	low (<500)	0.35

Visual Aid 30.3. Established rating numbers for various categories of pavement attributes used in the Swedish road inventory system.

Variable	Variable Level	Class Number	Rating
Wear	None or slight	1	12
	Obvious	2	18
	Considerable	3	48
	Serious	4	80
Deformation	None or slight	1	17
	Obvious	2	24
	Considerable	3	55
	Large	4	93
Treatment in Routine Maintenance	None	1	1
	Isolated patches, sealing	2	5
	Considerable tear up	3	5
	Considerable patching	4	20

Visual Aid 30.4. Table of composite pavement ratings.

		Wear			
Treatment in Routine Maintenance	Deformation	None or Slight (12)*	Obvious (18)	considerable (48)	Serious (80)
None (1)	None or slight (17)	30	36	66	98
	Obvious (24)	37	43	73	105
	Considerable (55)	68	74	104	136
	Large (93)	106	112	142	174
Isolated patches, sealing	None or Slight (17)	34	40	70	102
Sealing (5)	Obvious	41	47	77	109
	Considerable (55)	72	78	108	140
	Large (93)	110	116	146	178
Considerable tear up (15)	None or slight (17)	44	50	80	112
	Obvious (24)	51	57	87	119
	Considerable (55)	82	88	118	150
	Large (93)	120	126	156	188
Considerable patching (2)	None or slight (17)	49	55	85	117
	Obvious (24	56	62	92	124
	Considerable (55)	87	93	123	155
	Large (93)	125	131	161	193

*Numbers in parenthesis beside variable levels are the assigned rating numbers shown in Visual Aid 30.3.

Pavement Distress Evaluation			Significant		Moderate		Minimal	
PSI			2.4	3.5	2.4	3.5	2.4	3.5
		Traffic Level*						
Wet	Freeze Thaw	High	101 X	102	103	104 X	105 X	106
		Low	107	108 X	109 X	110	111	112 X
	No Freeze Thaw	High	113	114 X	115 X	116	117	118 X
		Low	119 X	120	121	122 X	123 X	124
Dry	Freeze Thaw	High	125	126 X	127 X	128	129	130 X
		Low	131 X	132	133	134 X	135 X	136
	No Freeze Thaw	High	137 X	138	139	140 X	141 X	142
		Low	143	144 X	145 X	146	147	148 X

* Low ≈ 6000 ADT
High ≈ 100,000 ADT

Visual Aid 30.5. Initial factorial design.

Pavement Type			Rigid				Flexible			
Pavement Distress Evaluation			Significant		Minimal		Significant		Minimal	
PSI			2.4	3.5	2.4	3.5	2.4	3.5	2.4	3.5
Traffic Level*										
Wet	Freeze Thaw	High	201 X	202	203	204 X	205	206 X	207 X	208
		Low	209	210 X	211 X	212	213 X	214	215	216 X
	No Freeze Thaw	High	217	218 X	219 X	220	221 X	222	223	224 X
		Low	225 X	226	227	228 X	229	230 X	231 X	232
Dry	Freeze Thaw	High	233	234 X	235 X	236	237 X	238	239	240 X
		Low	241 X	242	243	244 X	245	246 X	247 X	248
	No Freeze Thaw	High	249 X	250	251	252 X	253	254 X	255 X	256
		Low	257	258 X	259 X	260	261 X	262	263	264 X

*Low ≈ 6000 ADT
High ≈ 100,000 ADT

Visual Aid 30.6. Second factorial design.

INSTRUCTIONAL TEXT

REVIEW AND EVALUATION OF ALTERNATIVE APPROACHES TO FORMULATING A PRIORITIZATION INDEX

by

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Technical Memorandum 307-14
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REVIEW AND EVALUATION OF ALTERNATIVE APPROACHES TO FORMULATING A PRIORITIZATION INDEX

INTRODUCTION

An important phase of rehabilitation programming is the establishment of candidate projects for road repair work. In order to carry out this function, numerous highway agencies have developed pavement rating systems to quantify the condition of each road segment in the network. In most cases, a combined rating or score is used to express the overall condition of the pavement in terms of a combination of selected attributes. However, there are also rating systems which utilize only a single attribute to quantify pavement condition. As an example, the pavement rating system of New York utilizes only pavement serviceability.

Early efforts at developing pavement rating systems began in 1946 when the Highway Research Board established a committee on Pavement Condition Surveys in the Department of Design (Ref 1). The work of this committee culminated in the publication in 1957 of HRB Special Report 30, "Pavement Condition Surveys - Suggested Criteria." This report listed the various types of condition surveys and suggested items of information to be recorded for both preliminary and final type surveys. In addition, a comprehensive list of definitions of terms pertinent to pavement condition surveys was published. Then, in 1960, the staff of the AASHO Road Test developed an altogether new concept which was called the Present Serviceability Index (PSI) that is widely used by many highway agencies today (Ref 2). Finally, in 1962, the Highway Research Board published a procedure for rating the condition of flexible pavements. This procedure assigned numerical deduct values for specific distress types depending on their extent and severity. A numerical pavement score was computed for a specific road segment by adding up the deduct values and subtracting the sum from an assumed perfect score of 100. This procedure has been adopted in several highway agencies throughout the country and Reference 3 documents pavement rating systems where the procedure is used.

Several other approaches have been used to formulate indices or scores for quantifying pavement condition, and for establishing candidate projects for rehabilitation and maintenance programs. The objective of this chapter is to briefly document other studies which were made to formulate a pavement index or score for the purposes stated previously.

REVIEW OF NCHRP RESEARCH REPORT NO. NA - 3/1

Introduction

Research Report No. NA - 3/1 entitled, "Simplified Pavement Management at the Network Level" (Ref 4) presents a simplified pavement management system at the network level and provides an example illustrating how the framework can be applied to produce a priority ranking on a network basis. Emphasis shall be placed herein on describing the procedure used to quantify the adequacy of a pavement section for establishing priorities for rehabilitation work.

Methodology for Formulation of Prioritization Index (PINDEX)

In Research Report NA - 3/1, a methodology for developing an index that can be used for establishing rehabilitation priorities at the network level PMS was presented. This index, which was called PINDEX in the report, was developed using a procedure that involved the following steps:

(1) selection of pavement attributes to include in PINDEX,

(2) categorization of the selected pavement attributes,

(3) establishment of numerical values for each category of the pavement attributes, and

(4) establishment of weighting factors with which to adjust calculated values of PINDEX.

In order to illustrate the methodology, the example provided in Research Report No. NA - 3/1 shall be discussed.

For simplicity, the set of pavement attributes selected was PSI and severity and extent of fatigue cracking. In actual practice, additional variables may be incorporated in the formulation of PINDEX depending on particular agency circumstances. However, it should also be kept in mind that the methodology discussed herein is to be applied for programming purposes only and not for any specific rehabilitation design. Consequently, the selection of a few essential pavement attributes (such as PSI and cracking) can be justified.

The next step in the procedure is the categorization of the selected pavement attributes. In connection with this, the categories used are shown in Table 1. It is emphasized that the categories established in the table are merely illustrative. In actual practice, the categories are decided upon by a selected group of experienced highway engineers within a department.

Following the categorization of the selected pavement attributes, numerical values are then assigned to each category. This procedure is very similar to using deduct values except that for this case "additive" values are employed as shown in Table 1. Again, it is noted that those values are assigned on the basis of the subjective judgments of a set of pavement engineers. Using the condition survey information for a particular highway segment, a PINDEX value is computed by summing the pertinent numerical values for that particular road segment. For example, if a pavement belongs to the "Very Good" category in terms of both serviceability and fatigue cracking, the calculated PINDEX value would then be: 6 + 6 = 12. Other possible values of PINDEX for the example problem are summarized in Table 2. It should be noted that the higher the value of PINDEX, the higher the priority assigned to a pavement.

The last step in the procedure involves the establishment of weighting factors with which to adjust computed PINDEX values. In the field, conditions are not similar for all highway segments so that it would not be reasonable to compare pavements only on the basis of PINDEX values calculated using the procedure mentioned previously. For example, given two pavements with the same PINDEX value but with different traffic levels, it may not be logical to assign the same priority for both pavements. Instead, the highway segment with the higher traffic level should, most likely, be given a higher priority than the other one with low traffic. As a consequence, weighting factors are established considering variables such as traffic, functional

TABLE 1. CATEGORIES OF PAVEMENT ATTRIBUTES USED IN EXAMPLE PROBLEM AND CORRESPONDING NUMERICAL VALUES (Ref. 4)

A. Serviceability Category	PSI Range	Assigned Numerical Value
Very Good	3.8 - 5.0	6
Good	2.8 - 3.7	20
Fair	2.0 - 2.8	40
Poor	below 2.0	80

B. Fatigue Cracking Category	Severity	Extent (%)	Assigned Numerical Value
Excellent	Slight	10	2
Very Good	Slight	10 - 25	6
	Moderate or Severe	10	
Good	Slight	25 - 49	20
	Moderate or Severe	10 - 25	
Fair	Slight	50	40
	Moderate or Severe	25 - 49	
Poor	Moderate or Severe	50	80

TABLE 2. POSSIBLE VALUES OF PINDEX FOR SAMPLE PROBLEM (Ref. 4)

	PSI Category			
Fatigue Cracking Category	Very Good	Good	Fair	Poor
Excellent	8	22	42	82
Very Good	12	26	46	86
Good	26	40	60	100
Fair	46	60	80	120*
Poor	86	100	120*	160*

* In this example, if PINDEX >100, replace by PINDEX = 100

classification and amount of rainfall. For the sample problem, prioritization factors (Table 3) were established considering functional class and Average Daily Traffic (ADT). Again, it should be mentioned that these factors are established subjectively. A modified PINDEX is then computed by multiplying the PINDEX value by the appropriate weighting factor. In addition, priority categories may be established by assigning relative priority rankings to specific ranges of the adjusted PINDEX. For the example, the following priority categories were used:

ADJUSTED PINDEX	PRIORITY CATEGORY
≥ 60	1
≥ 28 but < 60	2
< 28	3

In summary, a simple framework for establishing priority listings at the network level was presented. Because of its simplicity, the methodology may be readily implemented within a highway agency. The method is subjective, but this may be an advantage in that highway personnel can easily relate with it since the numerical values used reflect their own collective judgment. In addition, it is a procedure which can be applied in the absence of objective data with which to construct an index for establishing priority listings. In effect, the framework can be used as a first cut procedure toward establishing priority rankings within an initial PMS pending the development of more complicated models later on.

TABLE 3. EXAMPLE PRIORITIZATION FACTORS BASED ON FUNCTIONAL CLASSIFICATION AND AVERAGE DAILY TRAFFIC (ADT)*

Functional Classification	ADT	Factor
Interstate	high	1.00
	medium	0.95
	low	0.88
Principal Arterial	high (>15,000)	0.93
	medium (5,000 - 15,000)	0.87
	low (<5,000)	0.80
Minor Arterial	high (>12,000)	0.83
	medium (4,000 - 12,000)	0.75
	low (<4,000)	0.68
Major Collector	high (>8,000)	0.73
	medium (2,000 - 8,000)	0.65
	low (<2,000)	0.60
Minor Collector	high (>5,000)	0.60
	medium (1,000 - 5,000)	0.53
	low (<1,000)	0.45
Local	high (>3,000)	0.55
	medium (500 - 3,000)	0.45
	low (<500)	0.35

* after Ref. 4

UNIQUE SUMS APPROACH

The unique sums approach is characteristic of a rating system used in Sweden (Ref 5) where classification of road sections are made with respect to the following variables: (1) pavement wear; (2) deformation [roughness and cracking]; and (3) amount of treatment in routine maintenance. For each variable, four levels were established which are indicative of the extent of distress, and for each level, there is assigned a class number and a rating as shown in Table 4. Each road section is therefore characterized by three rating numbers, which are added together to give a composite rating. The rating numbers were chosen in such a way that the sum of numerical values for every combination of variable levels is unique, i.e., each sum is different from the other sums. This characteristic differentiates this rating system from other procedures that assign numerical values to established categories of selected pavement attributes. In order to verify the uniqueness of the sums, Table 5 was set up using the rating numbers given in Table 4. Examination of the sums shows that each one is different from the rest. Because of this characteristic, any composite rating number can be readily broken down into its components. For example, given a composite rating of 30, one can easily identify the component variable levels as follows (refer to Table 5):

TABLE 4 ESTABLISHED RATING NUMBERS FOR VARIOUS CATEGORIES OF PAVEMENT ATTRIBUTES USED IN THE SWEDISH ROAD INVENTORY SYSTEM (Ref. 5)

Variable	Variable Level	Class Number	Rating
Wear	None or slight	1	12
	Obvious	2	18
	Considerable	3	48
	Serious	4	80
Deformation	None or slight	1	17
	Obvious	2	24
	Considerable	3	55
	Large	4	93
Treatment in Routine Maintenance	None	1	1
	Isolated patches, sealing	2	5
	Considerable tear up	3	15
	Considerable patching	4	20

TABLE 5 TABLE OF COMPOSITE PAVEMENT RATINGS

		Wear			
Treatment in Routine Maintenance	Deformation	None or Slight (12)*	Obvious (18)	Considerable (48)	Serious (80)
None (1)	None or slight (17)	30	36	66	98
	Obvious (24)	37	43	73	105
	Considerable (55)	68	74	104	136
	Large (93)	106	112	142	174
Isolated patches, sealing (5)	None or slight (17)	34	40	70	102
	Obvious (24)	41	47	77	109
	Considerable (55)	72	78	108	140
	Large (93)	110	116	146	178
Considerable tear up (15)	None or slight (17)	44	50	80	112
	Obvious (24)	51	57	87	119
	Considerable (55)	82	88	118	150
	Large (93)	120	126	156	188
Considerable patching (20)	None or slight (17)	49	55	85	117
	Obvious (24)	56	62	92	124
	Considerable (55)	87	93	123	155
	Large (93)	125	131	161	193

*Numbers in parenthesis beside variable levels are the assigned rating numbers shown in Table 4

Component Variable	Variable Level	Rating
Wear	None or slight	12
Deformation	None or slight	17
Treatment in routine maintenance	None	1
	Composite Rating =	30

In summary then, the unique sums approach is another simple way of formulating an index for quantifying pavement condition. The procedure is also comparable to other rating systems that assign deduct values to specific categories of pavement attributes. However, the selection of numerical values is constrained by the requirement that their sums be unique. Because of this characteristic, the composite rating numbers can be readily broken down into its components, and this is a desirable feature to have in a pavement rating system.

However, unless one has had experience using a rating system based on this approach, it may be difficult to identify the components of the composite ratings without also looking at some kind of a listing of unique sums and the corresponding variable categories for each. In addition, the numerical values obtained are highly non-linear, allowing no meaningful comparisons to be made between different degrees of pavement condition.

UTILITY THEORY

The application of utility theory to develop a measure of overall pavement performance has been reported for Arizona and Texas (Refs 6 and 7).

Basically, the procedure involves the assessment of utility functions which express a decision maker's preferences over different levels of selected attributes. These functions are primarily developed by soliciting expert opinion through interviews. An example (involving a utility function for money) is discussed herein so as to illustrate a procedure for constructing utility functions. However, some terms need to be defined first. The definitions provided are based on the material reported in Reference

For the succeeding discussion, any uncertain proposition is described as a lottery. For example, a person may be offered a lottery where he receives $100 if a head comes out in a toss of a coin, and nothing if a tail comes out. This coin tossing lottery is illustrated in Fig 1. Assuming that the coin is fair, his probability of winning $100 is exactly 1/2. The expected value of the lottery is computed by multiplying the amount of each prize by its probability and summing over all prizes. As such, the expected value of the coin tossing lottery is: 1/2($100) + 1/2($0) = $50. Now suppose that another individual offers to buy the lottery from the person to whom it was given to. The minimum price with which that person is willing to part with his lottery is defined as the certain equivalent of the lottery. Below this minimum selling price, the person would rather play the lottery, and above it, he would choose to sell it. Figure 2 illustrates the meaning of certain equivalent. The symbol ∿ is used to indicate that the person is indifferent between playing the lottery or getting an amount of $30 for it.

Several methods are available for assessing utility functions (Refs 9 and 10). For the example given here, a method known as the "standard gamble" is used. In applying this method, a decision maker is asked to choose between: (1) the certainty of receiving a sum of money, or (2) a lottery in

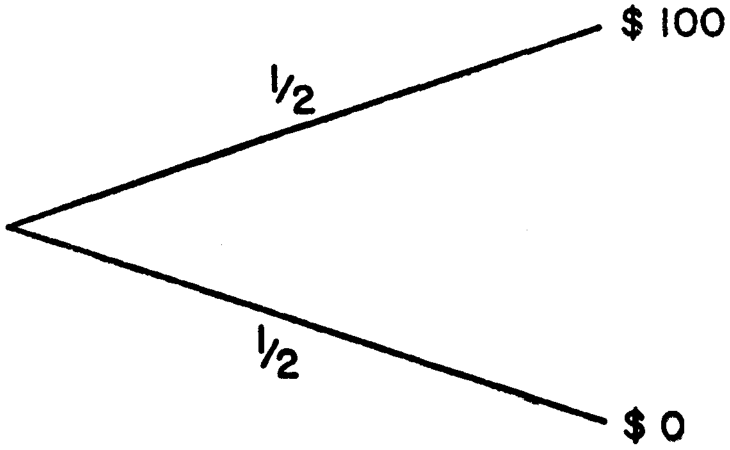


Fig 1. A coin tossing lottery.

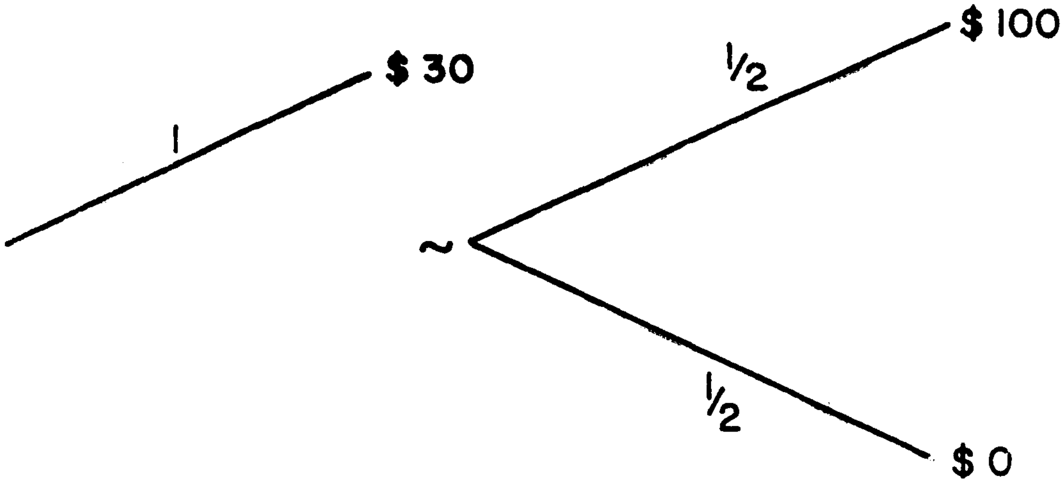


Fig 2. Certain equivalent of the coin tossing lottery.

which there is a chance of receiving or, in some cases, losing one of two sums of money in which risk is expressed in terms of the probabilities associated with winning each amount. If the decision maker expresses a preference for one of the alternatives, the probabilities are changed successively until the alternatives appear as equally desirable to the decision maker. At this point of indifference, the alternatives are equal in terms of utility. Since a utility function reflects subjective evaluations of amounts of money in relative terms, the utilities for two of the dollar amounts in the initial lottery can be chosen arbitrarily, and the utilities of other dollar amounts can be determined in relation to these utility values.

Proceeding with the example then, suppose an individual is asked to choose between (1) the certainty of receiving $3000 or (2) a lottery with a probability p = 0.15 of receiving $10,000 and a probability (1 - p) = 0.85 of getting $1000. It is assumed for this first iteration that the individual prefers Alternative 1. This seems reasonable since the value of Alternative 1 is greater than the expected value of the lottery E(u) = 0.15($10,000) + 0.85($1000) = $2350. For the next iteration, assume that the probabilities for winning $10,000 and $1000 are changed to 0.25 and 0.75 respectively, and suppose that the individual still prefers Alternative 1. This would indicate risk aversion by the individual since the expected value of the lottery ($3250) with the revised set of probabilities is greater than the value of Alternative 1. The procedure is continued, and the probabilities are changed to p = 0.30 and (1 - p) = 0.70. Given these probabilities, the individual might now indicate equal preference for or indifference between the alternatives. At this point, the individual's utility for $3000 in

Alternative 1 is equal to the utility for a 0.30 chance of receiving $10,000 and a 0.70 chance of getting $1000. To obtain utility values (in relative terms) for the monetary amounts in the alternatives, a utility value of zero for $1000 and a utility value of one for $10,000 can be assigned arbitrarily, and the utility value for $3000 can be found by solving the equation for the equal utility of the two alternatives. Thus:

$$\begin{aligned} U(\$3000) &= 0.3U(\$10{,}000) + 0.7U(\$1000) \\ &= 0.3(1) + 0.7(0) \\ &= 0.3 \end{aligned}$$

Therefore, with three points known, the individual's utility curve can already be constructed as shown in Fig 3.

The application of utility theory may be extended to the problem of formulating measures of pavement performance by construction of utility curves for selected pavement attributes. A composite measure of pavement performance can then be obtained by combining the utility curves in a single equation. For example, the following simple model may be used:

$$U(\underline{X}) = \sum_{i=1}^{n} k_i \, U_i \, (X_i)$$

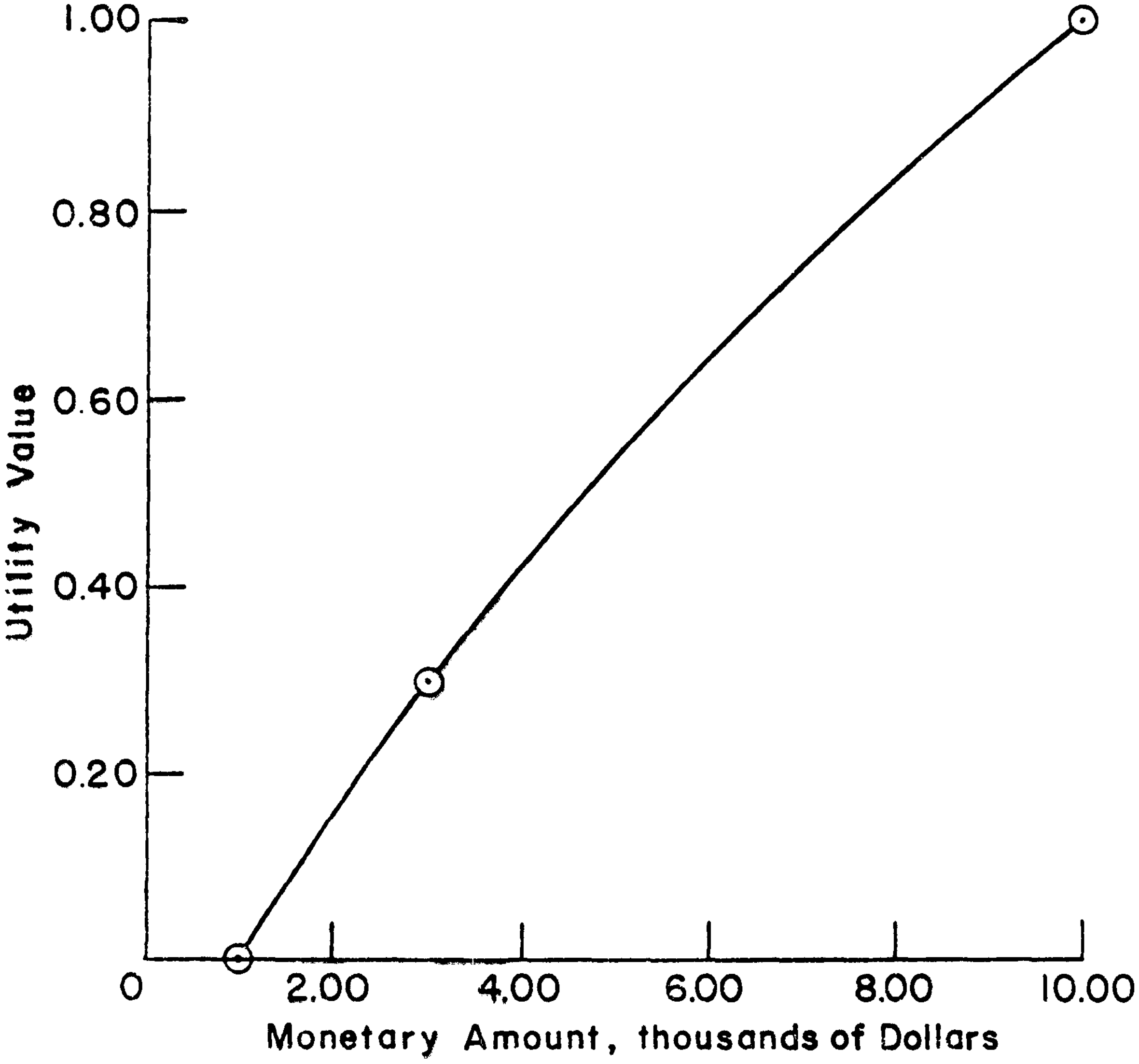


Fig 3. Example utility curve.

where

$U(\underline{X})$ = multi-attribute utility function scaled between 0 and 1,

$U_i(X_i)$ = individual utility function for the i^{th} attribute, scaled from 0 to 1, and

k_i = scaling constants with values between 0 and 1 such that

$$\sum_{i=1}^{n} k_i = 1.$$

This equation assumes mutual preferential independence between attributes. The intuitive meaning of this condition is that there is no interaction of preferences between attributes. Priorities can then be established by comparing the relative values obtained from the multi-attribute utility function $U(\underline{X})$.

DELPHI METHOD

One other approach to formulating a prioritization index is through the Delphi technique. This technique has been used previously in the development of a data base for Texas (Ref 11), and has recently been applied to develop a basis for evaluating highway surface condition in the State of Maine (Ref 12).

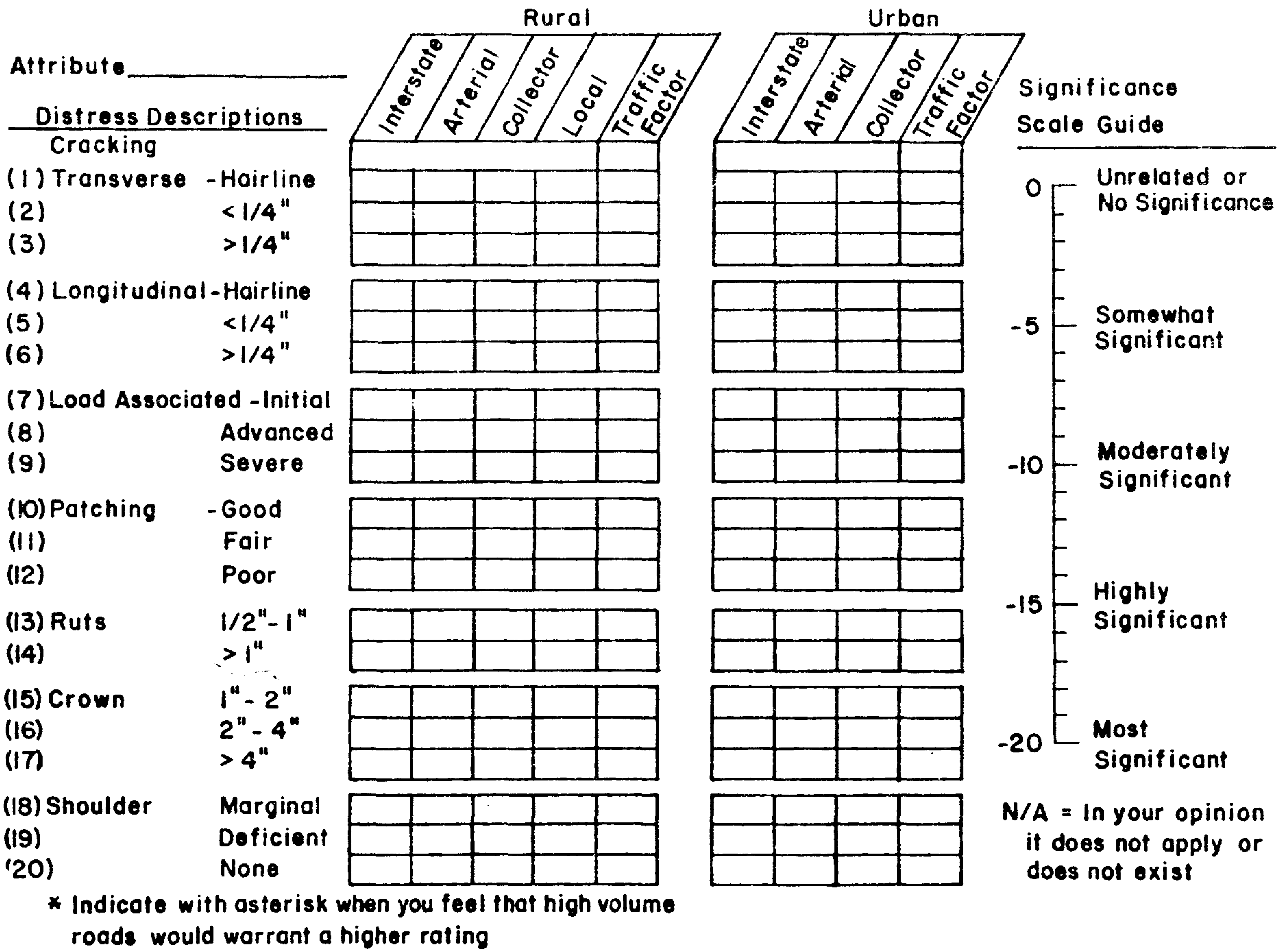
In this method, an attempt to achieve a consensus of opinion among a group of experts is made through cycles of intensive questioning interspersed with controlled opinion feedback. The technique avoids the direct confrontation of experts with one another which is the traditional method of pooling individual opinions. In this way, some of the serious difficulties inherent in face-to-face interaction are circumvented, such as (Ref 11):

(1) The spurious influence of a high status individual on the group - here, the status of an individual, which is often unrelated to his expertise on the question at hand, is given undue consideration in a face-to-face discussion.

(2) Ego commitment - after openly committing himself to a particular position, the individual is less likely to respond to facts and opinions advanced by other members of a face-to-face discussion group.

(3) Group pressure for conformity - in a face-to-face situation, the individual encounters great pressure to jump on the bandwagon and join the group.

The technique was applied by the Department of Transportation of the State of Maine in an effort to establish weights for various severity levels of selected distress categories. In connection with this, a rating form was developed, an example of which is shown in Fig 4 . Numerous pavement experts within the Department were then consulted. Each expert was asked to establish the relative importance of selected distress categories for the following attributes: (1) overall surface condition, (2) roughness, (3) safety, (4) strength, and (5) maintenance need. Scores are assigned using a scale from 0 to 20 as shown in Fig 4, and functional classification and traffic level are also considered when assigning scores. Successive iterations of the Delphi process are then made. At each iteration, the means of the ratings obtained during the previous cycle for each distiess category are fed back to the participants who are invited to make changes in their ratings in the light of the information presented. The final output of this process then is a set of importance ratings reflecting the group consensus which may be used for establishing priorities.



Attribute________________

Distress Descriptions		Rural Interstate	Rural Arterial	Rural Collector	Rural Local	Rural Traffic Factor	Urban Interstate	Urban Arterial	Urban Collector	Urban Traffic Factor
Cracking										
(1) Transverse	-Hairline									
(2)	< 1/4"									
(3)	> 1/4"									
(4) Longitudinal	-Hairline									
(5)	< 1/4"									
(6)	> 1/4"									
(7) Load Associated	-Initial									
(8)	Advanced									
(9)	Severe									
(10) Patching	-Good									
(11)	Fair									
(12)	Poor									
(13) Ruts	1/2"- 1"									
(14)	> 1"									
(15) Crown	1"- 2"									
(16)	2"- 4"									
(17)	> 4"									
(18) Shoulder	Marginal									
(19)	Deficient									
(20)	None									

* Indicate with asterisk when you feel that high volume roads would warrant a higher rating

Significance Scale Guide

0 Unrelated or No Significance

-5 Somewhat Significant

-10 Moderately Significant

-15 Highly Significant

-20 Most Significant

N/A = In your opinion it does not apply or does not exist

Fig 4. Order of significance rating form (Ref 11).

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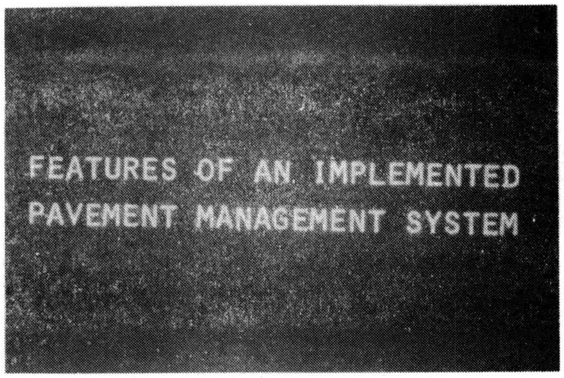
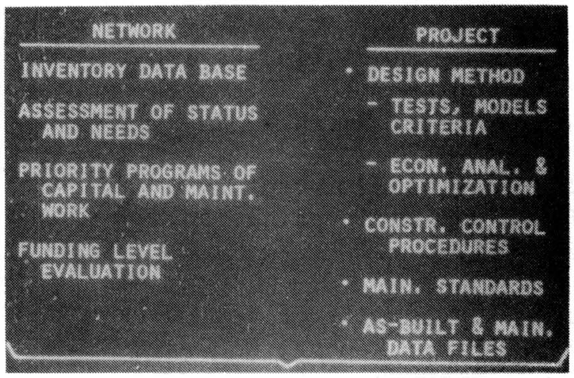


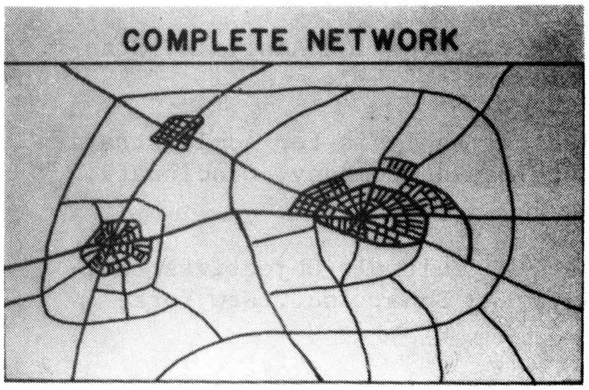
FEATURES OF AN IMPLEMENTED
PAVEMENT MANAGEMENT SYSTEM

Slide 30.1. Title Slide -
Features of an Implemented
Pavement Management System

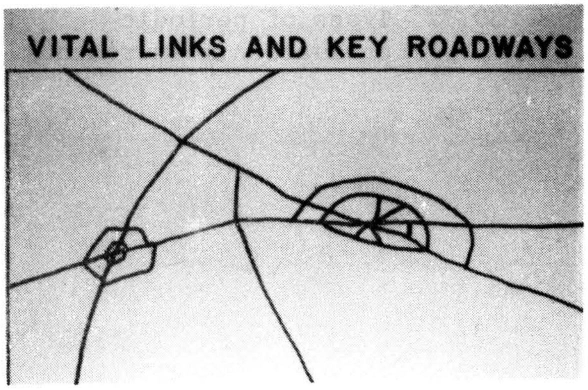


NETWORK	PROJECT
INVENTORY DATA BASE	• DESIGN METHOD
ASSESSMENT OF STATUS AND NEEDS	- TESTS, MODELS CRITERIA
PRIORITY PROGRAMS OF CAPITAL AND MAINT. WORK	- ECON. ANAL. & OPTIMIZATION
FUNDING LEVEL EVALUATION	• CONSTR. CONTROL PROCEDURES
	• MAIN. STANDARDS
	• AS-BUILT & MAIN. DATA FILES

Slide 30.2. Levels and elements of
a Pavement Management
System.

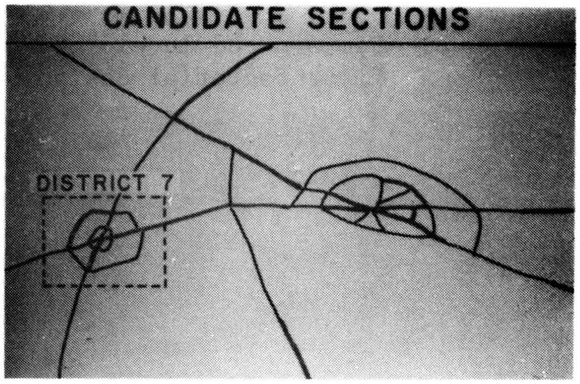


Slide 30.3. Typical network example.



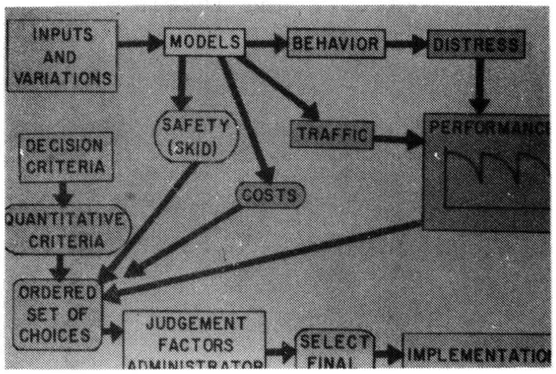
VITAL LINKS AND KEY ROADWAYS

Slide 30.4. Elements of network.



CANDIDATE SECTIONS

Slide 30.5. Candidate sections within a network.



INPUTS AND VARIATIONS

MODELS

BEHAVIOR

DISTRESS

SAFETY (SKID)

TRAFFIC

PERFORMANC

DECISION CRITERIA

COSTS

QUANTITATIVE CRITERIA

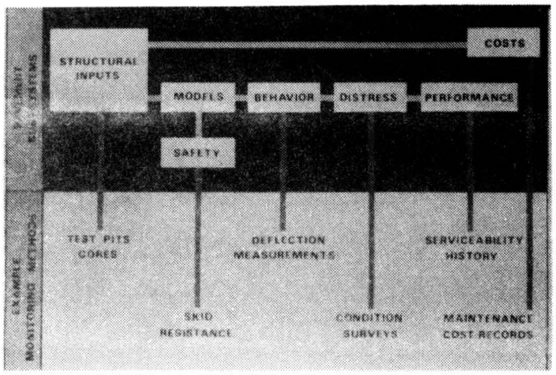
ORDERED SET OF CHOICES

JUDGEMENT FACTORS ADMINISTRATOR

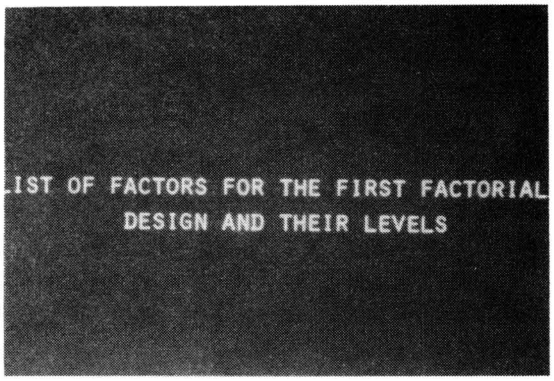
SELECT FINAL

IMPLEMENTATIO

Slide 30.6. Major components of pavement design.

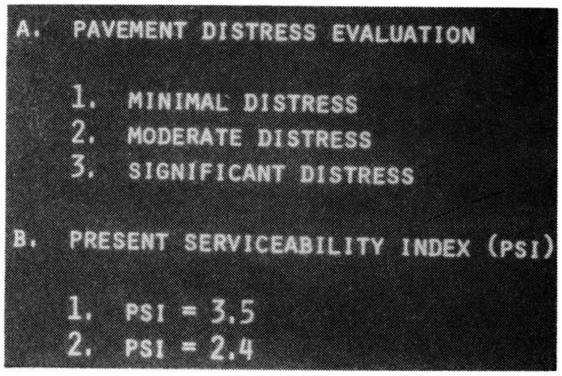


Slide 30.7. Types of periodic evaluation measurements.



IST OF FACTORS FOR THE FIRST FACTORIAL
DESIGN AND THEIR LEVELS

Slide 30.8. Factors and levels of first factorial design.



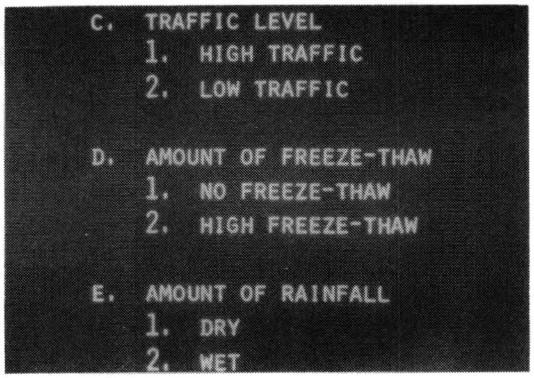
A. PAVEMENT DISTRESS EVALUATION

1. MINIMAL DISTRESS
2. MODERATE DISTRESS
3. SIGNIFICANT DISTRESS

B. PRESENT SERVICEABILITY INDEX (PSI)

1. PSI = 3.5
2. PSI = 2.4

Slide 30.9. Factors and levels.

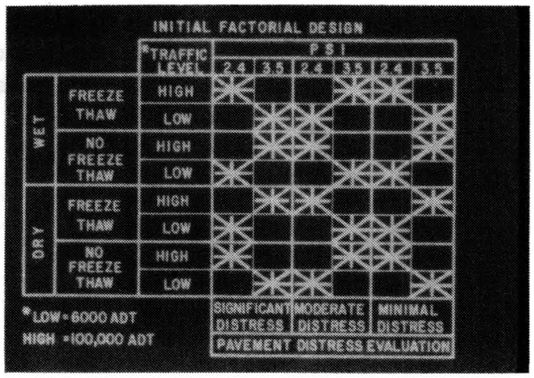


C. TRAFFIC LEVEL
  1. HIGH TRAFFIC
  2. LOW TRAFFIC

D. AMOUNT OF FREEZE-THAW
  1. NO FREEZE-THAW
  2. HIGH FREEZE-THAW

E. AMOUNT OF RAINFALL
  1. DRY
  2. WET

Slide 30.10. Factors and levels.

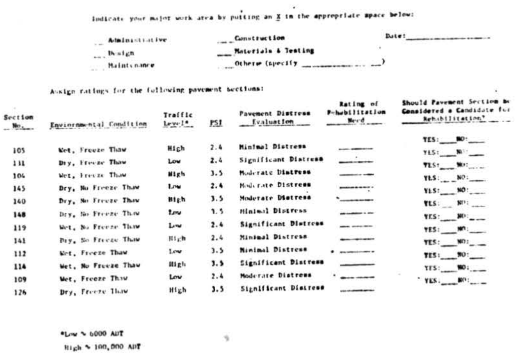


INITIAL FACTORIAL DESIGN

		*TRAFFIC LEVEL	PSI					
			2.4	3.5	2.4	3.5	2.4	3.5
WET	FREEZE THAW	HIGH	X			X	X	
		LOW		X	X			X
	NO FREEZE THAW	HIGH		X	X			X
		LOW	X			X	X	
DRY	FREEZE THAW	HIGH		X	X			X
		LOW	X			X	X	
	NO FREEZE THAW	HIGH	X			X	X	
		LOW		X	X			X
			SIGNIFICANT DISTRESS		MODERATE DISTRESS		MINIMAL DISTRESS	
			PAVEMENT DISTRESS EVALUATION					

*LOW = 6000 ADT
HIGH = 100,000 ADT

Slide 30.11. Fractional factorial from first factorial design.



Indicate your major work area by putting an X in the appropriate space below:

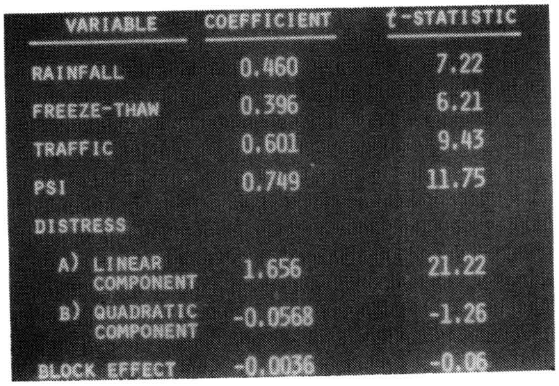
___ Administrative
___ Design
___ Maintenance
___ Construction
___ Materials & Testing
___ Others (specify ____________)
Date: ______________

Assign ratings for the following pavement sections:

Section No.	Environmental Condition	Traffic Level*	PSI	Pavement Distress Evaluation	Rating of Rehabilitation Need	Should Pavement Section be Considered a Candidate for Rehabilitation?
105	Wet, Freeze Thaw	High	2.4	Minimal Distress	______	YES:___ NO:___
111	Dry, Freeze Thaw	Low	2.4	Significant Distress	______	YES:___ NO:___
104	Wet, Freeze Thaw	High	3.5	Moderate Distress	______	YES:___ NO:___
145	Dry, No Freeze Thaw	Low	2.4	Moderate Distress	______	YES:___ NO:___
140	Dry, No Freeze Thaw	High	3.5	Moderate Distress	______	YES:___ NO:___
148	Dry, No Freeze Thaw	Low	3.5	Minimal Distress	______	YES:___ NO:___
119	Wet, No Freeze Thaw	Low	2.4	Significant Distress	______	YES:___ NO:___
141	Dry, No Freeze Thaw	High	2.4	Minimal Distress	______	YES:___ NO:___
112	Wet, Freeze Thaw	Low	3.5	Minimal Distress	______	YES:___ NO:___
114	Wet, No Freeze Thaw	High	3.5	Significant Distress	______	YES:___ NO:___
109	Wet, Freeze Thaw	Low	2.4	Moderate Distress	______	YES:___ NO:___
126	Dry, Freeze Thaw	High	3.5	Significant Distress	______	YES:___ NO:___

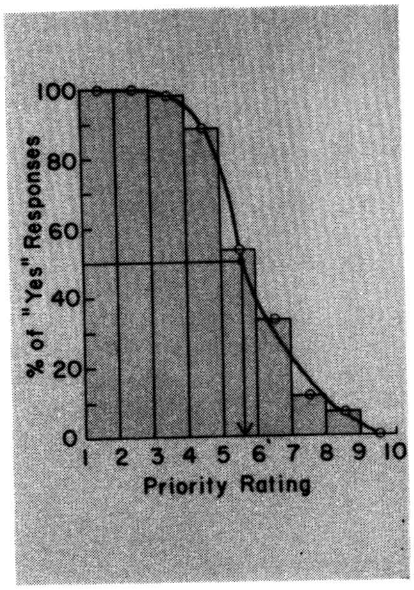
*Low ≈ 6000 ADT
High ≈ 100,000 ADT

Slide 30.12. Evaluation forms for individual ratings.

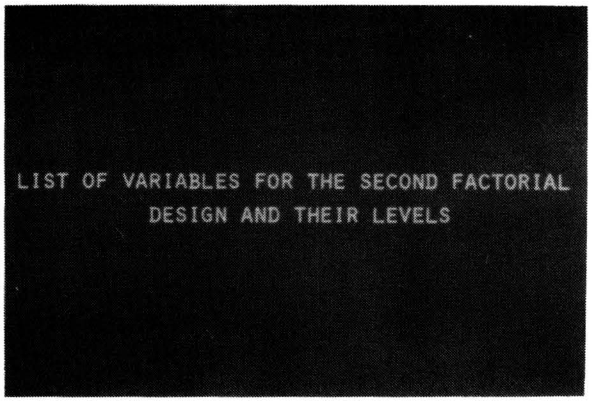


VARIABLE	COEFFICIENT	t-STATISTIC
RAINFALL	0.460	7.22
FREEZE-THAW	0.396	6.21
TRAFFIC	0.601	9.43
PSI	0.749	11.75
DISTRESS		
A) LINEAR COMPONENT	1.656	21.22
B) QUADRATIC COMPONENT	-0.0568	-1.26
BLOCK EFFECT	-0.0036	-0.06

Slide 30.13. First linear fit for prioritization index.

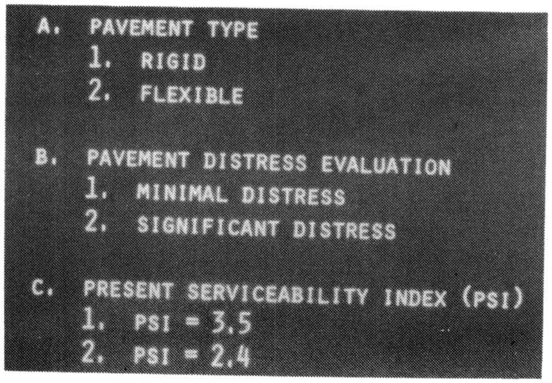


Slide 30.14. Cumulative frequency distribution of need of rehabilitation versus priority rating.



LIST OF VARIABLES FOR THE SECOND FACTORIAL DESIGN AND THEIR LEVELS

Slide 30.15. Factors and levels of second factorial design.

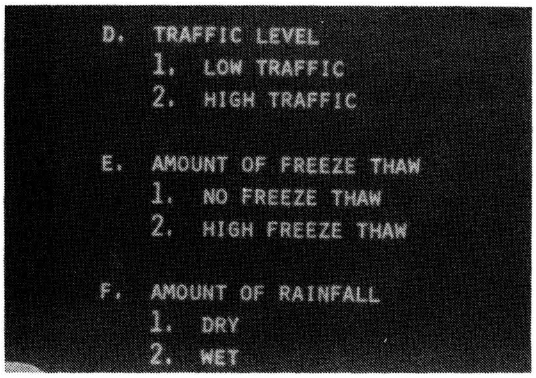


A. PAVEMENT TYPE
1. RIGID
2. FLEXIBLE

B. PAVEMENT DISTRESS EVALUATION
1. MINIMAL DISTRESS
2. SIGNIFICANT DISTRESS

C. PRESENT SERVICEABILITY INDEX (PSI)
1. PSI = 3.5
2. PSI = 2.4

Slide 30.16. Factors and levels.

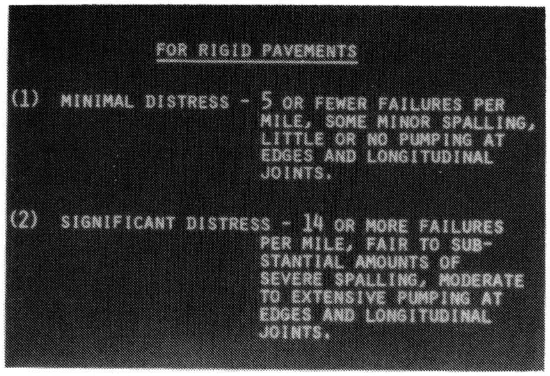


D. TRAFFIC LEVEL
1. LOW TRAFFIC
2. HIGH TRAFFIC

E. AMOUNT OF FREEZE THAW
1. NO FREEZE THAW
2. HIGH FREEZE THAW

F. AMOUNT OF RAINFALL
1. DRY
2. WET

Slide 30.17. Factors and levels.

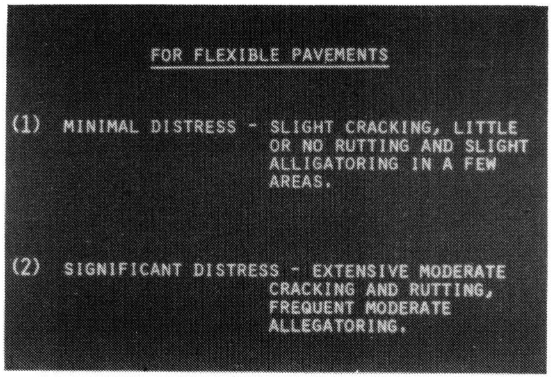


FOR RIGID PAVEMENTS

(1) MINIMAL DISTRESS - 5 OR FEWER FAILURES PER MILE, SOME MINOR SPALLING, LITTLE OR NO PUMPING AT EDGES AND LONGITUDINAL JOINTS.

(2) SIGNIFICANT DISTRESS - 14 OR MORE FAILURES PER MILE, FAIR TO SUBSTANTIAL AMOUNTS OF SEVERE SPALLING, MODERATE TO EXTENSIVE PUMPING AT EDGES AND LONGITUDINAL JOINTS.

Slide 30.18. Definition of levels for the distress factor corresponding to rigid pavements.

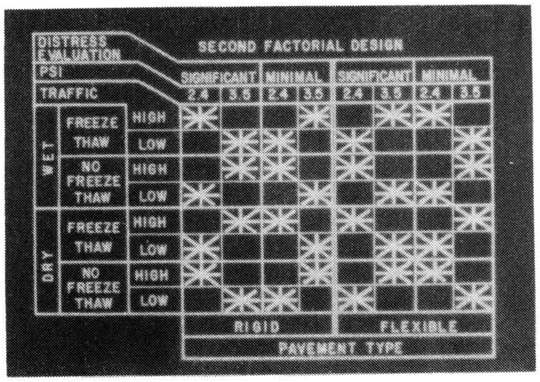


FOR FLEXIBLE PAVEMENTS

(1) MINIMAL DISTRESS - SLIGHT CRACKING, LITTLE OR NO RUTTING AND SLIGHT ALLIGATORING IN A FEW AREAS.

(2) SIGNIFICANT DISTRESS - EXTENSIVE MODERATE CRACKING AND RUTTING, FREQUENT MODERATE ALLEGATORING.

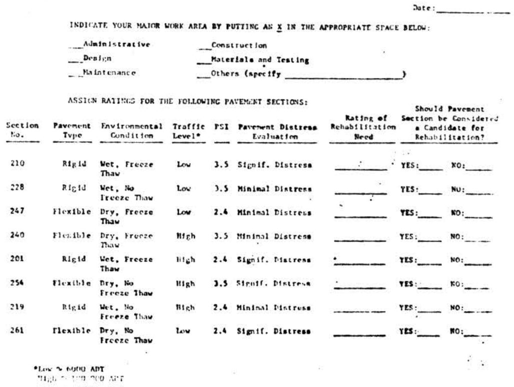
Slide 30.19. Definition of levels for the distress factor corresponding to flexible pavements.



SECOND FACTORIAL DESIGN

		PAVEMENT TYPE							
		RIGID				FLEXIBLE			
	DISTRESS EVALUATION	SIGNIFICANT		MINIMAL		SIGNIFICANT		MINIMAL	
	PSI	2.4	3.5	2.4	3.5	2.4	3.5	2.4	3.5
	TRAFFIC								
WET FREEZE THAW	HIGH	✕			✕		✕	✕	
	LOW		✕	✕		✕			✕
WET NO FREEZE THAW	HIGH		✕	✕		✕			✕
	LOW	✕			✕		✕	✕	
DRY FREEZE THAW	HIGH		✕	✕		✕			✕
	LOW	✕			✕		✕	✕	
DRY NO FREEZE THAW	HIGH	✕			✕		✕	✕	
	LOW		✕	✕		✕			✕

Slide 30.20. Fractional factorial from second factorial design.



Date:__________

INDICATE YOUR MAJOR WORK AREA BY PUTTING AN X IN THE APPROPRIATE SPACE BELOW:

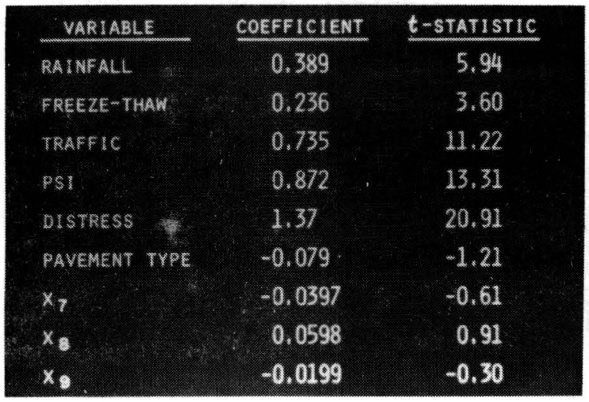
___Administrative
___Design
___Maintenance
___Construction
___Materials and Testing
___Others (specify ____________________)

ASSIGN RATINGS FOR THE FOLLOWING PAVEMENT SECTIONS:

Section No.	Pavement Type	Environmental Condition	Traffic Level*	PSI	Pavement Distress Evaluation	Rating of Rehabilitation Need	Should Pavement Section be Considered a Candidate for Rehabilitation?
210	Rigid	Wet, Freeze Thaw	Low	3.5	Signif. Distress	________	YES:____ NO:____
228	Rigid	Wet, No Freeze Thaw	Low	3.5	Minimal Distress	________	YES:____ NO:____
247	Flexible	Dry, Freeze Thaw	Low	2.4	Minimal Distress	________	YES:____ NO:____
240	Flexible	Dry, Freeze Thaw	High	3.5	Minimal Distress	________	YES:____ NO:____
201	Rigid	Wet, Freeze Thaw	High	2.4	Signif. Distress	________	YES:____ NO:____
254	Flexible	Dry, No Freeze Thaw	High	3.5	Signif. Distress	________	YES:____ NO:____
219	Rigid	Wet, No Freeze Thaw	High	2.4	Minimal Distress	________	YES:____ NO:____
261	Flexible	Dry, No Freeze Thaw	Low	2.4	Signif. Distress	________	YES:____ NO:____

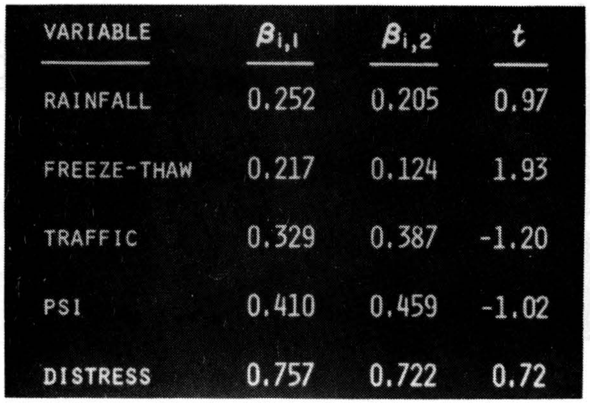
*Low < 6000 ADT
High > [illegible] ADT

Slide 30.21. Evaluation forms for individual ratings.



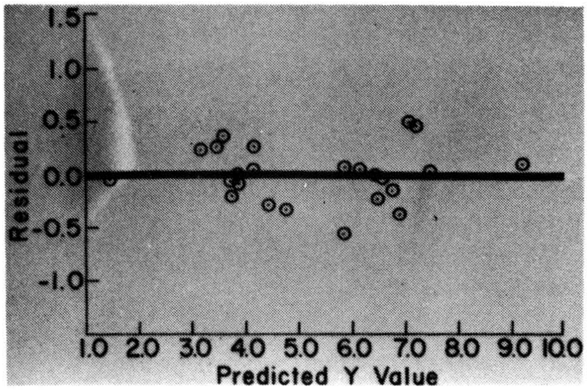
VARIABLE	COEFFICIENT	t-STATISTIC
RAINFALL	0.389	5.94
FREEZE-THAW	0.236	3.60
TRAFFIC	0.735	11.22
PSI	0.872	13.31
DISTRESS	1.37	20.91
PAVEMENT TYPE	-0.079	-1.21
X_7	-0.0397	-0.61
X_8	0.0598	0.91
X_9	-0.0199	-0.30

Slide 30.22. Second regression equation.

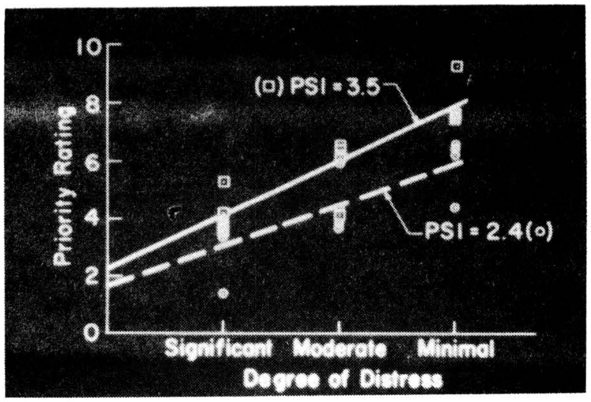


VARIABLE	$\beta_{i,1}$	$\beta_{i,2}$	t
RAINFALL	0.252	0.205	0.97
FREEZE-THAW	0.217	0.124	1.93
TRAFFIC	0.329	0.387	-1.20
PSI	0.410	0.459	-1.02
DISTRESS	0.757	0.722	0.72

Slide 30.23. Regression equation for reduced regression model.



Slide 30.24. Plot of residuals.



Slide 30.25. Priority rating as a function of degree of distress and PSI.

Sponsored by

The U.S.
Federal Highway Administration
Department of Transportation

Slide 30.26. Monogram of FHWA-DOT sponsor of this study.

REVISED CS/rh 11/1/84
Lesson 31

LESSON OUTLINE
PRIORITY PROGRAMMING

Instructional Objectives

1. To develop the basis for systematic planning of pavement investments.

2. To define and explain the factors that should be considered in priority programming or program optimization.

Performance Objectives

1. The student should be able to recognize the importance of systematic planning of pavement investments.

2. The student should be able to outline the basic steps in the systematic planning process.

Abbreviated Outline	Time Allocations, min
1. Information, analysis and decisions	15
2. Existing rehabilitation methods	15
3. Improving prioritization procedures	20
	50 minutes

Reading Assignment

1. Haas and Hudson - Chapter 6

2. RTAC - Part 2

3. Instructional Text

LESSON OUTLINE
PRIORITY PROGRAMMING

1.0 INTRODUCTION

When the individual projects have been scheduled, detail structural design and detail economic evaluation will determine the best within-project alternative. However, at network level, the selection of individual projects for construction should be made only after consideration is given to all other candidate projects with constraints of the limited fund and/or manpower availability. Therefore, some projects should be delayed due to these constraints. This situation necessitates a priority programming scheme for selection of individual projects for each year, over the program period.

2.0 BASIC CONSIDERATIONS FOR PRIORITY PROGRAMMING

2.1 Pavement Evaluation Outputs

The major outputs of a pavement provide a partial indication of what the various pavement management activities are to achieve as an end result. These outputs are such as structural capacity, riding comfort, distress, and safety.

2.2 Economic Considerations

A number of economic analysis methods include costs and benefits. In the pavement field, however, past practice has been to consider only capital and maintenance costs. User costs, for example, can vary significantly, and direct reductions can be included in benefits. Benefits are difficult to determine for pavement projects. Some can be approximately quantified in monetary terms. While direct agency costs, and direct user benefits, are given primary attention for priority programming, non-quantifiable factors may become important in some situations such as decision-making process.

2.3 Timing of investment (Visual Aid 31.1)

The delay of a pavement project changes the present value of benefits. The costs are similarly affected by the delay. A delay results in a higher extra maintenance cost that keeps the pavement at its acceptable level during the period in which the project is delayed.

3.0 Priority Programming Procedure (Visual Aid 31.2)

a) What basic data is required to make good decisions?

b) What are the good criteria to identify current needs and viable methods to identify future needs?
c) All feasible rehabilitation maintenance alternatives are considered and evaluated?
d) How to prioritize? Does priority analysis give reasonable answers?
e) Can we assess future financial implication of budget and program approvals today?
f) Are they most-cost effective programs? How about tradeoffs between maintenance and rehabilitation?
g) What are the future effects (average serviceability of network, amount of deficient mileage) for program and budget approvals?
h) How to present recommendations? Can implication of budget cuts be answered?

4.0 STATE-OF-THE-ART-PRIORITY PROGRAMMING MODELS

4.1 Priority Analysis Methods (Visual Aid 3.3)

In preparing priority programs for highway improvements, it should be emphasized that the underlying philosophy of most agencies is to protect investments. The common methods used by highway departments to assign priorities to various projects may be classified as follows.

4.1.1 Ranking. One of the most common methods used by highway agencies to prepare lists of capital improvement expenditures bases priorities on the following:
(a) ranking all candidate projects on a subjective basis using judgement, or
(b) ranking all candidate projects using the ratio of present worth of benefits to present worth of capital costs of the improvement, or
(c) ranking all candidate projects in descending order of the rate-of-return.

4.1.2 Benefit Maximization Method. The benefit maximization method uses two basic steps: first, the optimum timing of each project in the program period is calculated to maximize benefits; second, the associated costs are compared with the expected budget for each year. This can be accomplished with optimization techniques such as a linear programming.

4.1.3 Cost Minimization Method. This method is similar to the benefit maximization method, except that only costs are used for optimization. The optimum set of improvements by this method results in the least total cost to the agency. Again, various optimization techniques can be used.

4.2 Priority Programming Models Using Optimization Techniques

4.2.1 A System for Priority Programming of Investments for Road Network Improvements. This system was developed at the University of Waterloo in Canada. The research identified the essential components of a road management system for priority programming of pavement improvements for road networks. The report describes the actual development of the priority system, including the necessary performance prediction models, the determination of action levels and years for improvements, the identification of alternative improvement strategies, the economic analysis methodology and the optimization routine (linear programming) for determining priorities.

4.2.2 Rehabilitation and Maintenance System (RAMS). This system was developed at Texas A & M University. The RAMS is a set of seven computer programs that were formulated to maximize the total effectiveness of rehabilitation and maintenance activities in Texas while remaining within established resource constraints. The main purposes of these programs are:

(a) Identifying and scheduling cost effective rehabilitation and maintenance strategies,
(b) quantifying benefits of rehabilitation and maintenance strategies,
(c) deriving a rehabilitation and maintenance plan considering meaningful system constraints,
(d) determining optimal (maximum effectiveness) rehabilitation and maintenance policies.

4.2.3 Network Optimization System (NOS). The Network Optimization System was developed to assist the Arizona Department of Transportation to establish statewide pavement rehabilitation policies. Two functions of the NOS are:

(a) Determination of the rehabilitation policies that achieves prescribed performance standards at a minimum cost,
(b) by iteration, determination of the highest standards that can be maintained with a fixed budget.

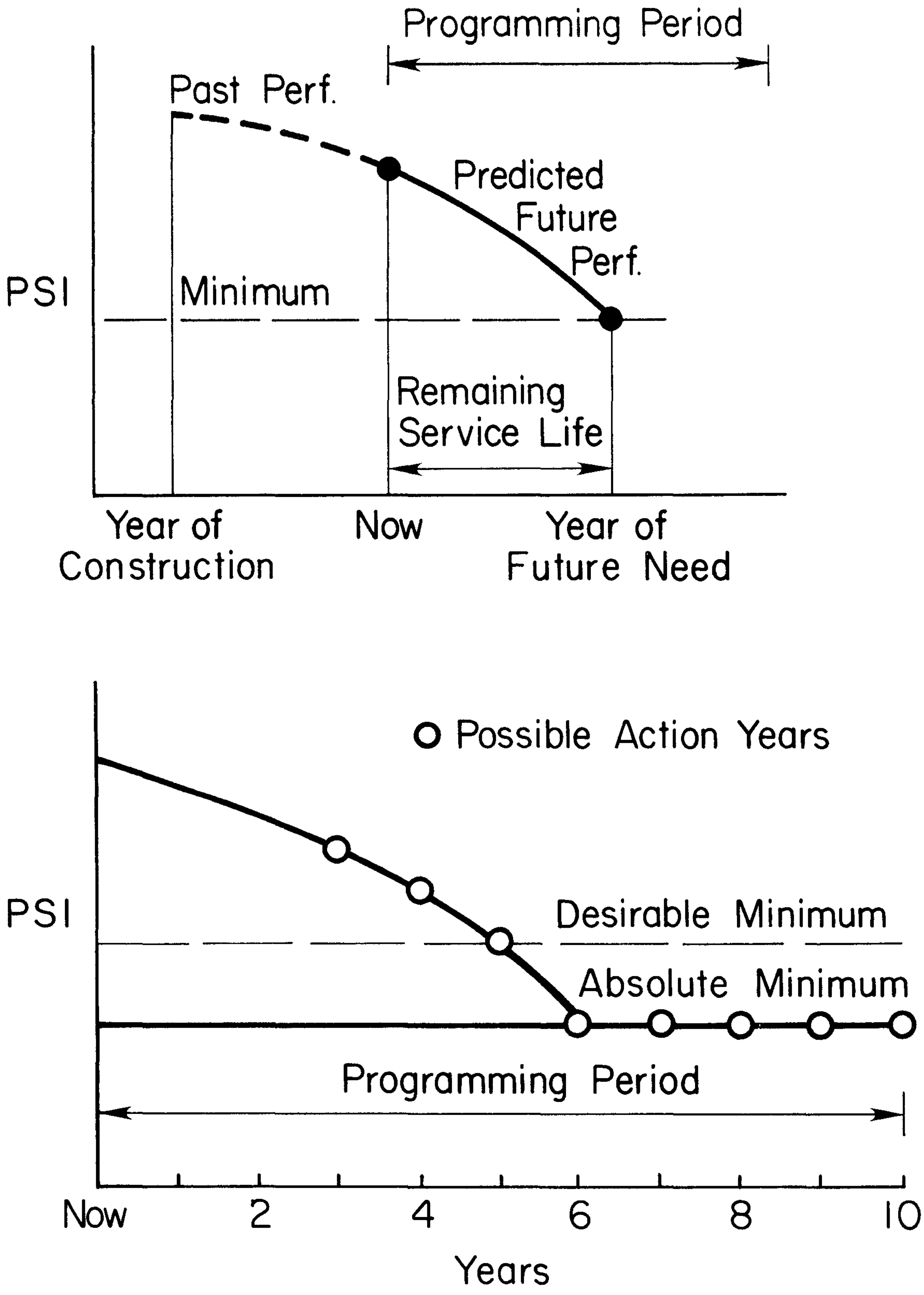
The NOS provides a systematic, consistant, and theoretically sound method for different roads in the network to achieve a desired performance standard.

A publication by FHWA titled "Pavement Management - Rehabilitation Programming: Eight States' Experiences" includes several models which are currently being used by various states to solve their rehabilitation programming problems.

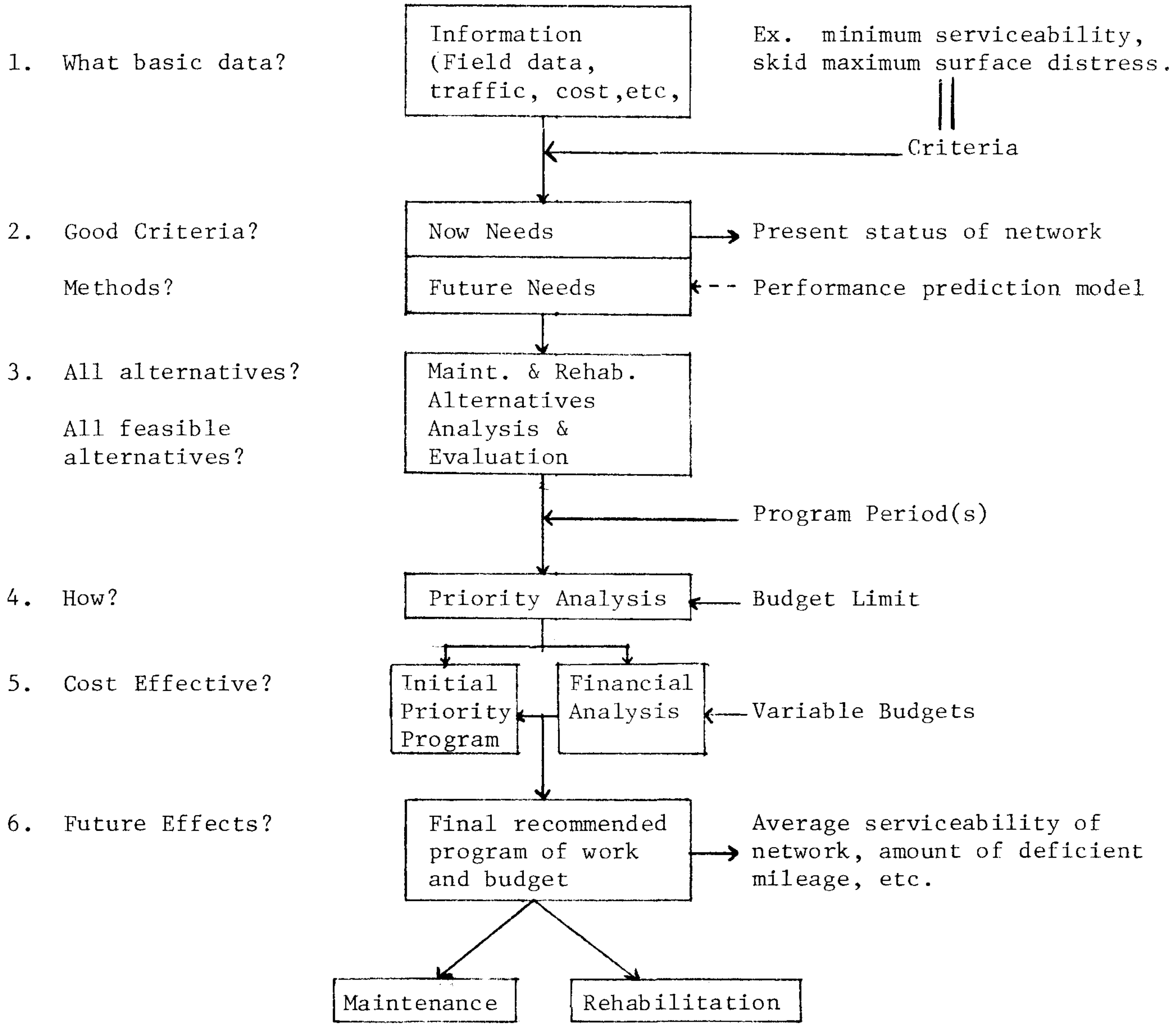
LESSON OUTLINE
PRIORITY PROGRAMMING

VISUAL AID	TITLE
Visual Aid 31.1	Programming Period and the Timing of Investment
Visual Aid 31.2	Priority Programming Procedure
Visual Aid 31.3	Priority Analysis Methods

Visual Aid 31.1. Programming period and the timing of investment.



Visual Aid 31.2. Priority programming procedure.



Visual Aid 31.3. Priority analysis methods.

TYPE OF METHOD	ADVANTAGES & DISADVANTAGES
1. SIMPLE SUBJECTIVE RANKING OF PROJECTS BASED ON JUDGEMENT	QUICK, SIMPLE; SUBJECT TO BIAS & INCONSISTENCY; MAY BE FAR FROM OPTIMAL
2. SIMPLE RANKING BASED ON SERVICE, DEFLECTION, ETC.	SIMPLE & EASY TO USE; MAY BE FAR FROM OPTIMAL
3. NO. 2 ABOVE BUT WEIGHTED BY TRAFFIC	REASONABLY SIMPLE; SHOULD BE CLOSER TO OPTIMAL
PRIORITY ANAL. (MODEL) FOR YEAR BY YEAR OF PROGRAM PERIOD	LESS SIMPLE; MAY BE CLOSE TO OPTIMAL; EFFECTS OF TIMING NOT CONSIDERED
COMPREHENSIVE PRIORITY ANALYSIS USING MATH PROGRAM MODEL; CONSIDERS TIMING ALTERN.	MOST COMPLEX; CAN GIVE OPTIMAL PROGRAM (MAX OF BENEFITS)

INSTRUCTIONAL TEXT

MUNICIPAL PAVEMENT MANAGEMENT SYSTEM

By

Mehmet A. Karan
M.Sc. (Istanbul), M.A.Sc. (Waterloo)

A thesis
presented to the University of Waterloo

Waterloo, Ontario, 1977

REVIEW OF EXISTING METHODOLOGY

2.1 INTRODUCTION

The need for a municipal pavement management system which can objectively establish pavement improvement priorities over the network, and its role in the overall road management, have been defined in Chapter 1.

In this Chapter the existing methods of programming highway improvements are briefly reviewed for their potential applicability to the municipal situation. Several examples from the current technology are given. Limitations of these methods, their advantages and disadvantages are discussed.

2.2 EXISTING METHODS OF PRIORITY PROGRAMMING

In the past, highway improvements were usually planned on the basis of current needs and projected future needs. The problem, generally, was the selection of the best alternative over a number of alternatives which were available for each specific project. In practice, selections were made with little regard to cost because it was assumed that needed funding would be available at the time of construction.

In recent years, however, highway agencies have been faced with very limited funds, which is a result of the cutbacks due to unstable economic conditions and other demands on governmental funds. But a large demand for improvements has arisen because of the size of, and the load on, the existing highway networks. The result has been a growing backlog of projects and a fragmented system of improved and unimproved highway segments.

The complexity of the problem and growing public complaints have forced engineers and economists to try and develop a system(s) which can provide a systematic solution that satisfies both the public and the agency. The result is the concept of priority analysis.

Priority analysis is a systematic process of deciding a) what projects should be built, and b) when they should be built. It is based on certain criteria which measure the degree of need, urgency, and desirability, and consider the availability of funds.

Although existing priority analysis methods vary widely in detail, they can be divided into two broad groups:

1. Ranking methods, and
2. Optimization methods.

These methods of priority analysis are briefly discussed in the following subsections.

2.2.1 Ranking Method

The ranking method is the most popular way of establishing priorities for highway improvements. It has been used widely by several highway departments (11-17) both in Canada and the United States.

In this method projects are ranked on the basis of criteria which are determined by the agency's policy. These criteria may be entirely subjective (i.e., subjective judgement) and/or they may depend on the deficiencies (i.e., deficiency ratings) of the system. In some cases ranking can also be done based on economic analysis.

The ranking method, therefore, can generally be subdivided into two main groups, such as a) sufficiency ratings, and b) ratings based on economic analysis. The following is a brief description of these methods.

2.2.1.1 Sufficiency Ratings

In this method of priority programming the approach depends on some form of sufficiency or deficiency ratings and it basically involves the following major steps (11):

1. The degree of need for an improvement, deficiencies and desirability of projects are established through a rating scheme, and
2. Projects are ordered through a ranking scheme, in accordance with their ratings and other subjective inputs.

Sufficiency ratings are the most common methods of establishing needs. Although they vary widely in detail, they can be classified in two broad groups. In the first group, each factor which affects the needs and deficiencies of the facility, is rated separately, but a single composite score is then calculated and used for project ranking. The approach used by the Arizona Highway Department is a good example of this method (12).

In the second group, projects are segregated into priority groups based on ratings of individual factors. The Tennessee (13) and Washington (14) methods are the two typical-examples for this group.

The following is a brief summary of Arizona and Tennessee methods. The limitations of these methods will be discussed in the later sections of this Chapter.

In the Arizona method a highway sufficiency rating system is used to determine the needs of the existing network, and guidelines are established to reduce the human error and personal judgement. One hundred points, which is the highest possible overall total rating, is first broken down into three major components, such as Condition, 35 points; Safety, 30 points; and Service, 35 points. These broad categories are then subdivided and the following component points are assigned:

Condition 35 pts.		Safety 30 pts.		Service 35 pts.	
Structural adequacy	17	Roadway width	8	Alignment	12
Remaining Life	13	Surface width	7	Passing Opportunity	8
Maintenance	5	Sight distance	10	Surface width	5
		Consistency	5	Ride quality	10

Highway officials inspect the network at regular intervals and collect information in terms of the points described above. Each highway section is rated separately with this rating system and a total score is then calculated to describe the deficiency of the section. A section with 67 total points, for example, is considered more deficient than a section with 75 points.

Another one hundred points are then used to describe the socio-economic factors, which are subdivided into three main groups: environment, 40 points; economic development, 35 points; and traffic safety, 25 points. Each highway section is also rated for factors and the ratings are summed and then added to the overall sufficiency rating to find one composite score which is used for priority analysis. Projects are selected for implementation, starting from the project with lowest composite score until the budget for the year is exhausted. Remaining projects are then shelved and go back on the candidate list for the following year.

In the Tennessee method, highway sections are rated from structural condition, facility of movement and safety points of view (13). The main difference of this method from the Arizona method is the fact that each of the three factors rated retains its own identity throughout the rating process; each is weighted with the others, but is not lost in a single index figure.

The first factor considered in the analysis is the structural condition which includes the condition of subgrade, drainage, base and surface. Each of these four sub-factors is rated separately on the basis of a previously established rating system. These individual ratings are then summed to find one total rating for structural condition. The same process is followed on each section and then the total ratings are arrayed in the descending order of their magnitude. They are then divided into ten groups where each group includes sections with similar structural condition ratings. Each group is given a numerical index 9 to 0 depending on the first digit of their total ratings (i.e., if the total structure condition rating is between 80 to 89, the index is 8). These indices of structural condition constitute the first digit in the final 3-digit priority index.

Deficiency of movement in hours of low traffic and high traffic is obtained by subtracting actual average design speed from standard design speed and actual operating speed from standard operating speed. The deficiency in facility of movement is then calculated by taking the average of the two differences and multiplying it by the average daily traffic volume of the section. The ratings are then arrayed in the order of their magnitudes and divided into ten groups with similar ratings. The section in these groups are then given index numbers varying from 9 to 0, to indicate the degree of deficiency. These index numbers for facility of movement constitute the second digit in the 3-digit overall priority index.

The Tennessee study measures safety in terms of accident rate per mile and uses this rate as the third digit in the section's 3-digit priority index figure.

In the final priority analysis five arrays are made by rearranging the sections. The first array which gives the sections of highest priority, consists of sections with structural deficiency ratings of 9, 8 and 7 arranged in that order. The second array consists of the remaining sections with deficiency ratings for facility of movement of 9, 8 and 7 in that order. These sections have the next highest priority.

The third array which gives the sections with the next highest priority consists of the remaining sections with deficiency ratings for safety of 9, 8 and 7 arranged in descending order. The fourth array consists of the remaining sections with a structural deficiency rating of 6 and 5, whereas the fifth array, which gives the sections with lowest priority, consists of the remaining sections in order of their rating for facility of movement and then arranging them in order of their structural condition rating and of their rating for safety.

The Tennessee study also gives a similar rating procedure for urban state highways. In urban situations, condition, congestion and route characteristics are rated separately and used in a similar approach as was briefly summarized in the foregoing paragraphs.

A different version of a ranking procedure has recently been developed by the Georgia Department of Highways (11). The procedure is based on a "scoring model approach" which analyzes the projects in terms of a number of parameters. In the Georgia method 26 parameters were chosen which were divided into eight main groups such as need, deficiency, continuity, benefit-cost, local opinion, and economic, social and environmental consequences. Each project is evaluated in terms of these parameters and a combined score, priority index, is obtained through a set of weighting factors. The projects are then ranked based on their priority indices.

In a slightly modified version of the Georgia method, two indices, priority group index and desirability index, are calculated. Projects are ranked first by their priority group indices and then the desirability index is used to re-order the projects.

In the Georgia approach, projects are categorized according to ten functional classes and nine improvement types. The projects are ranked within each category.

The priority index approach has also been used for programming arterial street improvements in urban areas. Hall and Hixon (15) in their studies in San Diego, Phoenix and Nashville have come up with different priority formulas. The basis of these formulas are the same except that different weighting factors are used to weight the parameters involved in the analyses.

The main difference between Hall and Hixon's approach and the others is the fact that they include project cost in their priority formulas. In other words, the concept of economics is considered in the analysis in terms of costs.

The following section deals with the rating methods that are based on both sufficiency ratings and economics (i.e. costs) and gives several examples.

2.2.1.2 Rating Methods Based on Economics

The priority rating system that has been developed by Thiers, et al (18) is a good example of this group of rating methods. In this method a total of 450 points are divided between the cartway (300 points), sidewalk (100 points) and curb (25 points for physical condition and 25 for the amount of curb reveal). A ratio of 2:1 is used to show the relative importance of cartway as opposed to the sidewalk-curb system. Six categories of ratings which vary from very poor to excellent are also used to reflect the condition of cartway, pavement and curbs. Table 2.1 gives the evaluation guidelines used in the study. The priority index of a street improvement is then calculated with the following formula:

TABLE 2.1

EVALUATION INDEX FOR URBAN STREET FACILITIES

CONDITION	CARTWAY	CURB CONDITION	CURB REVEAL	SIDEWALK
Very poor	0 to 50	0 to 5	0 to 6	0 to 19
Poor	51 to 100	6 to 10	7 to 12	20 to 39
Fair	101 to 150	11 to 16	13 to 18	40 to 69
Good to Fair	151 to 210	-	-	-
Good	211 to 270	15 to 22	19 to 24	70 to 95
Excellent	271 to 300	23 to 25	25	96 to 100

After Reference (18)

$$\text{Priority Index} = \frac{R_I - R_E}{C} W = \frac{IW}{C} \quad \ldots\ldots\ldots\ldots\ldots\ldots (2.1)$$

where R_I = Numerical rating of the improved condition of the facility,

R_E = Numerical rating of the existing condition of the facility,

W = Numerical weighting factor representing the relative importance of the facility,

C = Incremental cost to effect the proposed improvement, and

I = Numerical measure of incremental improvement, $R_I - R_E$.

Rank ordering the values of the priority index of each project gives a priority rating list which can be used for developing a street improvement program with budget constraints.

A similar approach which incorporates the cost of the project in priority analysis has been developed by the Ministry of Transportation and Communications of Ontario. This method has currently been used for rural, semi-urban and urban conditions by several municipalities in Ontario (19). The following is a brief description of the method that is used for urban street improvements.

In the Ontario Method a total of 100 points is divided into a number of parameters which are assumed to constitute the overall condition of the existing facility. These parameters are: level of service, 20 points; surface width, 25 points; surface condition, 10 points; structural adequacy, 20 points; drainage, 15 points; and maintenance demand, 10 points.

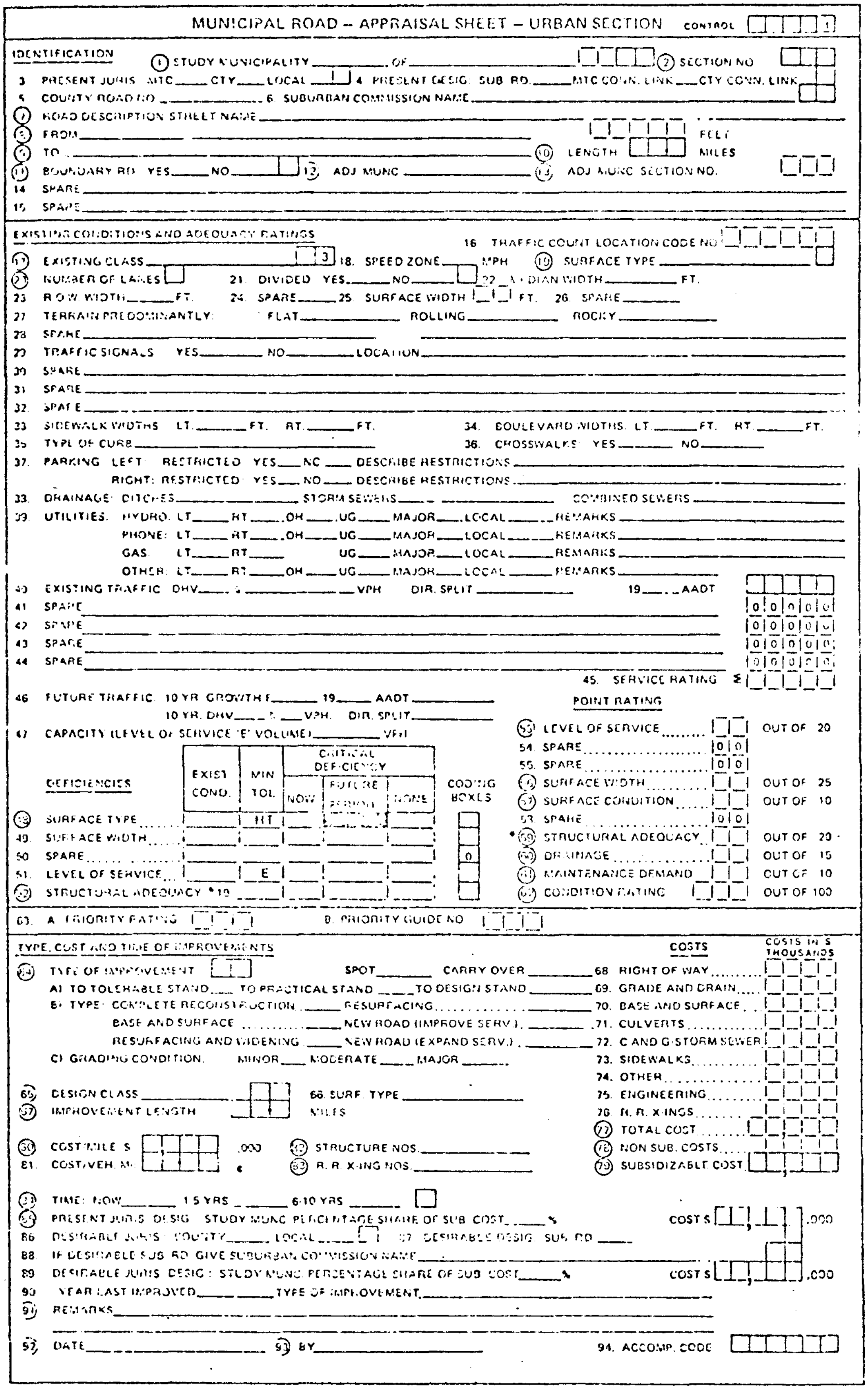
Each of these factors is rated separately on the basis of the guidelines given in Reference (19). The sum of these ratings then gives the condition rating of the street section. The total cost of performing the work required is calculated by "bench mark" cost tables. This information is then used in the following formula to find the priority guide number of each project.

$$\text{Priority Guide Number} = \frac{100 - \text{Condition Rating}}{\text{Cost per Vehicle Mile (in cents)}} \quad ..(2.2)$$

The project with the largest priority guide number gets the highest priority. All the projects are built according to the magnitude of their priority guide numbers, until the total cost equals the budget available for that year. Figure 2.1 gives an example appraisal sheet that is used for urban roads in Ontario.

In certain priority programming procedures more detailed economic analysis, mostly in terms of cost-benefit ratio or rate-of-return, is used for project ranking. Gardner and Chiles' method is a prime example of this approach (16).

The method is called "congestion approach" and is used in Pennsylvania for programming improvement priorities. In this approach the year of the structural and functional failure (i.e. demand exceeds capacity)is forecasted. It is assumed that when the structural failure occurs before the functional failure, then the year of improvement is the year of structural failure. If, however, functional failure occurs before the structural failure, then the road can either be improved by sacrificing the remaining structural life, or nothing be done and



MUNICIPAL ROAD – APPRAISAL SHEET – URBAN SECTION CONTROL ☐☐☐☐ 1

IDENTIFICATION

① STUDY MUNICIPALITY ________ OF ________ ☐☐☐☐ ② SECTION NO ☐☐☐
3 PRESENT JURIS MTC ____ CTY ____ LOCAL ____ ☐ 4 PRESENT DESIG: SUB RD ____ MTC CONN. LINK ____ CTY CONN. LINK ☐
5 COUNTY ROAD NO ________ 6 SUBURBAN COMMISSION NAME ________ ☐☐
⑦ ROAD DESCRIPTION STREET NAME ________
⑧ FROM ________ ☐☐☐☐☐ FEET
⑨ TO ________ ⑩ LENGTH ☐☐☐ MILES
⑪ BOUNDARY RD YES ____ NO ____ ☐ ⑫ ADJ MUNC ________ ⑬ ADJ MUNC SECTION NO. ☐☐☐
14 SPARE ________
15 SPARE ________

EXISTING CONDITIONS AND ADEQUACY RATINGS

16 TRAFFIC COUNT LOCATION CODE NO ☐☐☐☐☐☐
⑰ EXISTING CLASS ________ ☐ 3 18. SPEED ZONE ____ MPH ⑲ SURFACE TYPE ________ ☐
⑳ NUMBER OF LANES ☐ 21. DIVIDED YES ____ NO ____ ☐ 22 MEDIAN WIDTH ________ FT.
23 R.O.W. WIDTH ____ FT. 24. SPARE ____ 25 SURFACE WIDTH ☐☐ FT. 26. SPARE ____
27 TERRAIN PREDOMINANTLY: FLAT ________ ROLLING ________ ROCKY ________
28 SPARE ________
29 TRAFFIC SIGNALS YES ________ NO ________ LOCATION ________
30 SPARE ________
31 SPARE ________
32 SPARE ________
33 SIDEWALK WIDTHS LT. ____ FT. RT. ____ FT. 34. BOULEVARD WIDTHS LT. ____ FT. RT. ____ FT.
35 TYPE OF CURB ________ 36. CROSSWALKS: YES ____ NO ____
37. PARKING LEFT: RESTRICTED YES ____ NO ____ DESCRIBE RESTRICTIONS ________
RIGHT: RESTRICTED YES ____ NO ____ DESCRIBE RESTRICTIONS ________
38. DRAINAGE: DITCHES ________ STORM SEWERS ________ COMBINED SEWERS ________
39. UTILITIES: HYDRO: LT ____ RT ____ OH ____ UG ____ MAJOR ____ LOCAL ____ REMARKS ________
PHONE: LT ____ RT ____ OH ____ UG ____ MAJOR ____ LOCAL ____ REMARKS ________
GAS: LT ____ RT ____ UG ____ MAJOR ____ LOCAL ____ REMARKS ________
OTHER: LT ____ RT ____ OH ____ UG ____ MAJOR ____ LOCAL ____ REMARKS ________
40 EXISTING TRAFFIC DHV ____ % ________ VPH DIR. SPLIT ________ 19 ____ AADT ☐☐☐☐☐
41 SPARE ________ 0 0 0 0 0
42 SPARE ________ 0 0 0 0 0
43 SPARE ________ 0 0 0 0 0
44 SPARE ________ 0 0 0 0 0
45. SERVICE RATING Σ ☐☐☐☐☐
46 FUTURE TRAFFIC: 10 YR GROWTH F ____ 19 ____ AADT ________
10 YR. DHV ____ % ____ VPH. DIR. SPLIT ________
47 CAPACITY (LEVEL OF SERVICE 'E' VOLUME) ________ VPH

DEFICIENCIES	EXIST COND.	MIN TOL.	CRITICAL DEFICIENCY			CODING BOXES
			NOW	FUTURE PERIOD	NONE	
㊽ SURFACE TYPE		HT				☐
49. SURFACE WIDTH						☐
50 SPARE						0
51. LEVEL OF SERVICE		E				☐
(52) STRUCTURAL ADEQUACY *19						☐

POINT RATING

(53) LEVEL OF SERVICE ☐☐ OUT OF 20
54. SPARE 0 0
55. SPARE 0 0
(56) SURFACE WIDTH ☐☐ OUT OF 25
(57) SURFACE CONDITION ☐☐ OUT OF 10
58. SPARE 0 0
*(59) STRUCTURAL ADEQUACY ☐☐ OUT OF 20
(60) DRAINAGE ☐☐ OUT OF 15
(61) MAINTENANCE DEMAND ☐☐ OUT OF 10
(62) CONDITION RATING ☐☐☐ OUT OF 100

63. A PRIORITY RATING ☐☐☐ B. PRIORITY GUIDE NO ☐☐☐

TYPE, COST AND TIME OF IMPROVEMENTS

(64) TYPE OF IMPROVEMENT ☐☐ SPOT ________ CARRY OVER ________
A) TO TOLERABLE STAND ____ TO PRACTICAL STAND ____ TO DESIGN STAND ________
B) TYPE: COMPLETE RECONSTRUCTION ____ RESURFACING ________
BASE AND SURFACE ____ NEW ROAD (IMPROVE SERV.) ________
RESURFACING AND WIDENING ____ NEW ROAD (EXPAND SERV.) ________
C) GRADING CONDITION: MINOR ____ MODERATE ____ MAJOR ____

COSTS	COSTS IN $ THOUSANDS
68 RIGHT OF WAY	☐☐☐☐
69. GRADE AND DRAIN	☐☐☐☐
70. BASE AND SURFACE	☐☐☐☐
71. CULVERTS	☐☐☐☐
72. C AND G STORM SEWER	☐☐☐☐
73. SIDEWALKS	☐☐☐☐
74. OTHER	☐☐☐☐
75. ENGINEERING	☐☐☐☐
76. R. R. X-INGS	☐☐☐☐
(77) TOTAL COST	☐☐☐☐
(78) NON SUB. COSTS	☐☐☐☐
(79) SUBSIDIZABLE COST	☐☐☐☐☐

(65) DESIGN CLASS ________ ☐☐ 66. SURF. TYPE ________
(67) IMPROVEMENT LENGTH ☐☐ MILES
(80) COST/MILE $ ☐☐☐☐ .000 (82) STRUCTURE NOS. ________
81. COST/VEH. MI. ☐☐☐☐ ¢ (83) R. R. X-ING NOS. ________
(84) TIME: NOW ____ 1-5 YRS ____ 6-10 YRS ____ ☐
(85) PRESENT JURIS DESIG: STUDY MUNC PERCENTAGE SHARE OF SUB COST ____ % COST $ ☐☐☐☐ .000
86 DESIRABLE JURIS: COUNTY ____ LOCAL ____ ☐ 87 DESIRABLE DESIG: SUB RD ____
88 IF DESIRABLE SUB RD GIVE SUBURBAN COMMISSION NAME ________
89 DESIRABLE JURIS DESIG: STUDY MUNC PERCENTAGE SHARE OF SUB COST ____ % COST $ ☐☐☐☐ .000
90 YEAR LAST IMPROVED ________ TYPE OF IMPROVEMENT ________
(91) REMARKS ________
(92) DATE ________ (93) BY ________ 94. ACCOMP. CODE ☐☐☐☐☐☐

FIGURE 2.1 - EXAMPLE APPRAISAL SHEET FOR URBAN STREETS IN ONTARIO

congestion starts building up. In either case alternatives are analysed to determine the best one and the year in which the improvement should be done. In an earlier study (17) Gardner proposes a priority number for ranking projects. This priority number is calculated for each alterative by dividing the cost incurred due to congestion by the cost of performing the proposed improvement. In the early stages of testing, however, it was found that selection of the alternative with the highest priority number (i.e., highest rate of return) often failed to reduce congestion cost to desirable levels (16). It is for this reason that approximate rate-of-return and incremental rate-of-return methods have subsequently been used by Gardner and Chiles for comparing alternatives and ordering of investment priorities.

The modified version of the congestion approach works in seven main phases. In the first phase the structural and functional failure dates of the road section are calculated by the use of the existing road life studies and demand-capacity analysis, respectively. In the second phase the outputs of Phase 1 are taken to the field for verification of various elements of the output. The third phase analyses a number of alternatives that are available for the road section, and selects the best alternative on the basis of average and incremental rate-of-returns. The list of optimum improvements in sequential sections constitutes the fourth phase. In the fifth phase, road sections are combined to form projects, whereas the economics (i.e. rate-of-return) of these projects are calculated in Phase 6.

Phase 7 is the list of projects in the order of their rate-of-returns. In the final priority list, however, adjustments are made to give high priorities to the projects that fail structurally not functionally. Similarly, within the structurally failed roads, the ones that require the cheapest alternatives (i.e. null alternative) get higher priorities. The rest of the projects are then ordered according to their rates-of-return.

2.2.2 Optimization Methods

Mathematical optimization techniques have recently been used for programming investment priorities for highway improvements (20, 21, 22). The optimization approach is conceptually quite different from the other existing methods. It combines the functions of priority programming, program formulation and project scheduling into one operation which gives the optimum schedule of projects through precise analytical techniques such as linear and dynamic mathematical programming (11).

The linear programming approach (20, 21) which can be used to either maximize benefits or minimize costs, seems to be the most popular approach, but dynamic programming has also been used for programming safety improvement investments in Kentucky (22).

The following is a brief description of these methods.

2.2.2.1 Benefit Maximization With Linear Programming

In this method of optimization, which has recently been developed by the Ontario Ministry of Transportation and Communications for programming highway improvements, maximum benefit (or effectiveness) is derived from the expenditure without violating the budget constraints. The cost of the improvement, benefits accruing from the improvement and budgets are all considered in the analysis.

The cost of the improvement varies depending on the year of implementation, as shown in Figure 2.2. Similarly, benefits of the improvement also vary by the year of implementation. Figure 2.3 shows an example benefit stream, and it also shows that maximum benefits can be obtained by implementing the improvement in the year 1988. Therefore, for this project the optimum timing is 1988.

The linear programming method firstly calculates, for each improvement project, the optimum year of implementation and compares the total cost of the projects that will be implemented in a particular year with the budget available in that year. The upper portion of Figure 2.4 schematically shows this concept. In this diagram improvements are placed into their maximum benefit year and it shows the total costs for each year. Also shown is the total expected budget line. It indicates, for example, that in the year 1980 there is not enough budget available to handle all the improvement projects. In 1981, however, there are not enough improvements available to exhaust the budget.

The linear programming method then rearranges the timing of the improvements in such a way as to minimize the total benefit loss, as shown in the lower portion of Figure 2.4. The result is a priority program which does not violate budget constraints and maximizes benefits to the community.

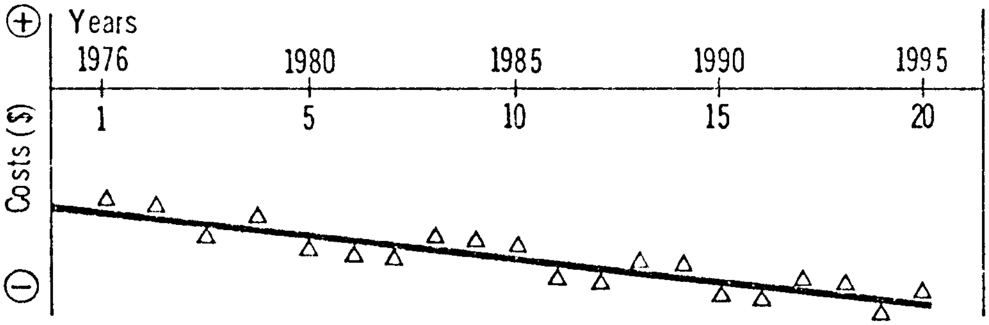


FIGURE 2.2 - TYPICAL COST STREAM (Present Day Dollars) FOR AN IMPROVEMENT FOR EACH POSSIBLE YEAR OF IMPLEMENTATION

After Ref. (20)

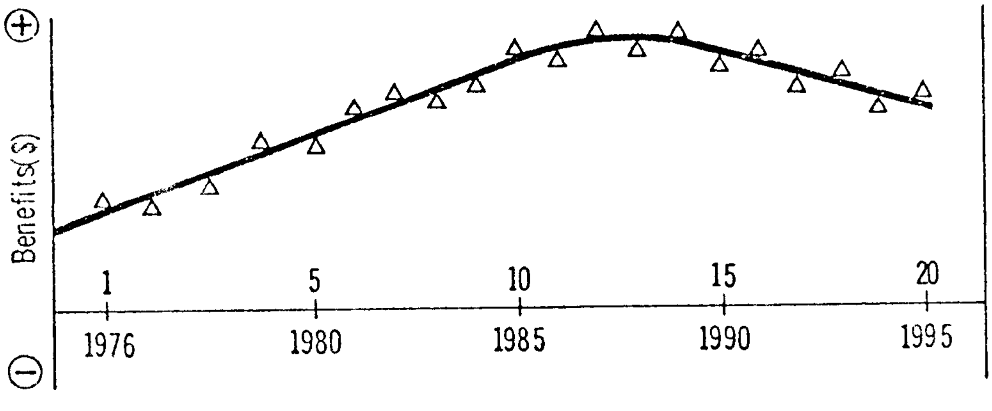


FIGURE 2.3 - TYPICAL BENEFITS STREAM (Present Day Dollars) FOR AN IMPROVEMENT FOR EACH POSSIBLE YEAR OF IMPLEMENTATION

After Ref. (20)

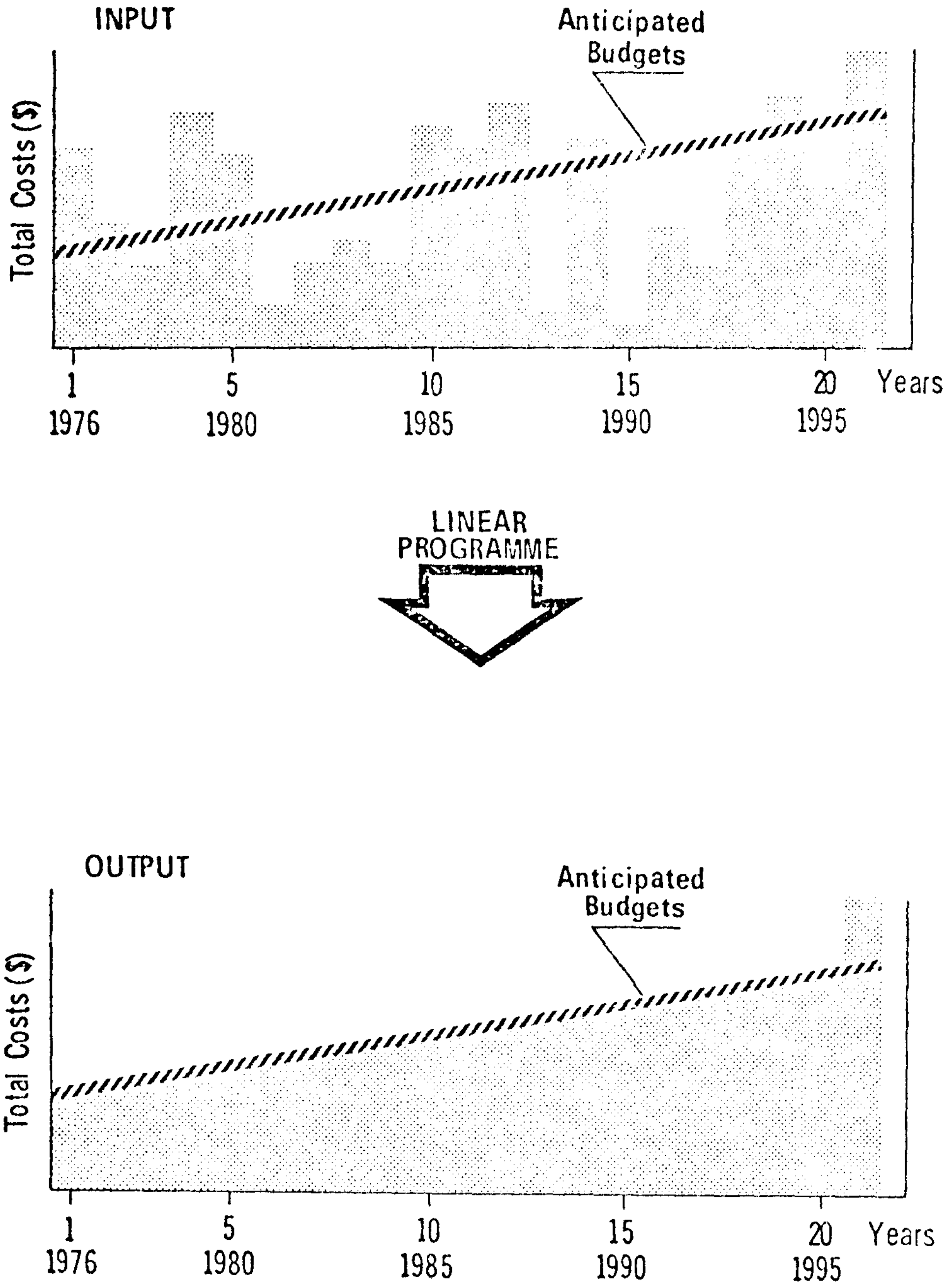


FIGURE 2.4 - LINEAR PROGRAMMING METHOD FOR REARRANGING OPTIMUM INVESTMENT TIMES FOR IMPROVEMENTS SO THAT TOTAL BENEFITS LOSS IS MINIMIZED AND BUDGET CONSTRAINTS ARE NOT EXCEEDED

After Ref.(20)

With some modifications several alternative improvements for a single project can also be tested with this method. It is also possible to consider stage construction, dependent improvements and constraints on funds that are available for a particular region or a type of improvement (20).

A similar linear programming approach has been employed in Texas for a strategic planning of pavement rehabilitation and maintenance (23). The concepts of roadway inventory, condition rating, gain of rating, minimum rating requirement, pavement survival rate, resource requirement and resource availability are used in the study with a zero-one integer linear programming model which maximizes the overall effectiveness of all proposed maintenance and rehabilitation activities subject to decision, minimum distress and pavement rating constraints, as well as available supplies, equipment, manpower and overhead constraints. The main difference of Texas' approach from the Ontario's method of priority programming is that Texas' method studies the priorities of projects in only one year. In other words it does not consider a programming period in which projects can be shifted in time, instead it works with a one year programming period and determines maintenance strategies for only that year.

2.2.2.2 Cost Minimization With Linear Programming

This method is similar to the benefit maximization method except that only costs are considered in the optimization. All other consequences of the improvement such as user and environmental benefits are ignored in the analysis. It is for this reason that the method does not necessarily give an optimum solution for the community.

The cost minimization method, however, can provide a valuable tool for the agency to measure the effectiveness of an added cost associated with a priority program determined by another method.

2.2.2.3 Dynamic Programming Approach for Priority Programming

Pigman, et al (22) have recently adopted the principles of dynamic programming for determining an optimum combination of safety improvement projects for a given budget.

The cost of the improvement in terms of construction or installation cost and maintenance costs through the expected life of the improvement, is considered in the analysis. Similarly, benefits due to the reduction in accidents are taken into account in the study.

The model can deal with a large number of projects each with one or more alternatives, and produce a solution of optimum combination of projects and/or alternatives that should be implemented for a given budget.

This model has been employed, in Kentucky, for planning 61 projects. A multi-stage dynamic programming approach has been used and has produced slightly better results than the benefit-cost ratio method.

2.3 MARGINAL ANALYSIS

An heuristic approach, based on marginal analysis, has recently been developed for the Massachusetts Department of Public Works to handle the problem of programming improvement investments (24). The objective is the maximization of net benefits under certain budget constraints. It is argued that this can only be achieved by maximizing the return of each successive dollar invested, and this requires the consideration of each project's marginal contribution to the overall program benefit.

On the basis of this concept, the iterative process starts with selecting the projects with the highest benefit-cost ratios. The budget and all other constraints (i.e. geographical constraints) are taken into account in this selection.

When a selection is made in any stage of the process, the marginal benefit-cost ratios of its mutually exclusive alternatives are calculated. These marginal benefit-cost ratios are then used in the next iteration if the budget has not been exceeded with this selection. The idea is to find the project that gives the best return for the investment.

This method has been shown to be capable of handling budget constraints, regional or area minimums, functional classification minimums, scale or sizing of projects (i.e. multiple alternatives or sizes) and project benefit interdependencies.

A Highway Investment Analysis Package, based on marginal analysis, has also been developed and used in the Federal Highway Administration (25). It is an investment programming model which produces multi-period investment programs by selecting the improvements that maximize user benefits.

2.4 LIMITATIONS OF EXISTING METHODS

The most common problem with the rating methods of priority programming is that they depend primarily on subjective judgement and opinions. The weighting factors used in these methods are mostly based on personal judgements of engineers and may vary from one person to another. There is practically no hard evidence available to show that these weightings are done in such a way as to reflect public opinion. The rating methods also have the deficiency of not including detailed economic analysis. More importantly perhaps, they do not consider the economic consequences of project timing. A project, for example, might be needed in a particular year, but when its priority is examined at the network level, it might be more economical to delay the project for a year or so. It is therefore very important to take into account the trade-offs between costs and benefits in time.

Consequently, the present ranking methods of priority programming need to be improved to include a) more systematic and objective measures for needs studies, and b) more economic considerations in project ranking.

The mathematical optimization method seems to overcome the deficiency of not including economic analysis. Some of the existing approaches, however, do have some limitations especially in terms of not dealing with a programming period. Texas' linear programming model, for example, analyses each project over a ten year analysis period and then generates an optimum priority list for the next year. This of course may be a serious limitation because of the fact that the trade-offs between costs and benefits in time may have a significant effect on the outcome of the process. It is therefore necessary to consider a programming period in order to take into consideration the effects of project timing.

Kentucky's dynamic programming approach has a similar limitation. It also deals with just a one-year program and produces a priority list for one year.

A priority program should deal with a programming period (i.e., 5 years, 10 years) and should be able to produce a priority list for each year in this period. This does not mean that these lists are final and will be implemented without any change. In fact, the program should be run each year to update the previous run.

The idea of having a programming period gives a chance for each project to be examined in the process. The effects of the project timing are also taken into account when dealing with a period rather than just one year.

One other limitation of the mathematical models is that they do not generally incorporate subjective values in their structures. They perform detailed economic analysis and the outcome of the process depends mainly on the factors that are expressable in monetary terms. In priority programming, however, some subjective issues such as public demand, politics, etc., play an important role in the decisions.

It can be argued, however, that the engineering portion of the priority programming is strictly related to the economics of the problem. In other words, engineers and planners provide a tool for decision makers to help them in making their decisions. The basic purpose of an engineering-based priority program therefore can be defined as providing an economic base for decision makers. Consequently, the limitation of not incorporating subjective issues can be overshadowed by the fact that the priority list, which is developed by any one of the existing methods of priority programming, is a tentative list which will probably be modified by the decision makers.

In summary, the existing methods of priority programming suffer from various limitations. Some of them are too subjective, and others are unnecessarily complex with respect to the mathematics involved. On the basis of the foregoing arguments, the linear programming approach seems to be more promising (in general terms) than the other approaches.

PRIORITY PROGRAMMING MODEL

7.1 INTRODUCTION

The economic analyses which provide a base for establishing pavement improvement priorities have been described in the previous Chapter.

In this Chapter, a priority programming model is described. Its purpose is to establish priorities for urban pavement improvements.

The problem of priority programming is also discussed briefly for one and multi-year programming periods, with or without budget constraints.

Several economical methods of priority programming are briefly reviewed and a linear programming (LP) approach is recommended for the urban situation. The input requirements of the method, typical results and a critical review are also included in this Chapter.

7.2 PRIORITY PROGRAMMING PROBLEM

When there is a need for a large number of pavement improvement projects and sufficient funds are not available, the agency faces the problem of selecting the "best" projects for implementation. The criteria for this selection may vary in detail from one agency to another, but it can basically be stated as follows: Maximization of road user's benefits without exceeding budgetary constraints.

From the economics point of view, therefore, the problem of priority programming is two-sided. The user or the community in a broader sense, constitute the one side of the problem. The agency and its limited funds are the other side.

The positive and/or negative impacts of a pavement rehabilitation project are consumed mostly by the road user. There may be some external effects which may influence the community. These effects, however, are usually quite hard to quantify, and are generally neglected in the analysis. But the user, being the person who enjoys or suffers the consequences of a pavement project should be taken into account in deciding what projects should be built.

The agency, on the other hand, is mostly concerned with the cost portion of the problem. The funds available for pavement improvements cannot be exceeded. It is the agency's responsibility to make sure that the budget is not violated in each year.

The problem, therefore, can be summarized as selecting projects in such a way as to maximize the total positive impacts of the projects without exceeding budgetary constraints.

Some agencies prefer to deal with this problem on a yearly basis. Stated in another way, the agency takes into account all the projects that should be built in a particular year and prepares a priority program for that year only. The projects that are not included in this program are delayed and considered in the analysis next year together with the new projects that come into the analysis next year.

To establish priorities over a programming period, however, is more realistic since it allows all the projects to compete with each other. The timing of a project, in this method, is determined through a more comprehensive analysis in which all projects are taken into account.

There are several alternative methods that may be used for one-year and/or multiple-year priority programming. A review of the existing methods of priority programming has been given in Chapter 2. The following is a brief description of some of the economic approaches that can be used for priority programming purposes. The main difference of these approaches from the methods reviewed in Chapter 2 is that they depend on pure economics. Both the user and the agency can directly be taken into account in the analysis, as benefits and costs, respectively.

7.2.1 Priority Programming for One Year Time Period

Priority programming of multiple projects for a one year time period can be done by a) calculating a rate-of-return of benefits over costs, or b) discounting a stream of costs and benefits to a reference year, and comparing them (89).

The benefit/cost ratio method and first-year rate-of-return method are the most common ones in the first group that are widely used in practice. In these methods, projects are ranked in decreasing order on the basis of their benefit/cost ratios and first-year rate-of-returns, respectively. Then, starting from the top, projects are selected until the budget for this particular time period is exhausted.

The internal rate-of-return and net present value methods, which are the most common methods in the second group, work almost on the same basis as the first group. In these methods, however, a stream of costs and benefits over an extended period of time (i.e., analysis period) are considered. Then, in the net present value method, for example, projects are ranked in decreasing order of their net present values, and a cut-off point is determined on the basis of the budget constraint.

The rate-of-return and discounting methods can be used for multi-year priority programming. The projects that are cut-off in one particular year are delayed and considered in the next year. These delayed projects compete with the new projects that come into the analysis in the next year. They may be selected for implementation or may be delayed for another year, depending on their economic implications.

The main deficiency of using these methods for programming over a number of years is that they do not consider the trade-offs between benefit losses and project timing.

7.2.2 Priority Programming Over a Programming Period

When a project needs to be built in a particular year, its economic desirability in that year is usually determined by cost-benefit analysis. Its costs and benefits are calculated with respect to this particular implementation year, and a criteria (i.e., benefit/cost ratio, net present value, etc.) is used to determine its feasibility. The project is then accepted for implementation or rejected on the basis of this criteria.

The costs and benefits of a project, however, may vary depending on the year of implementation. The delay of a project can result in substantial increases in its benefits with very little changes in the costs. The net present value of the project, therefore, may increase significantly if the project is delayed. Figure 7.1 schematically shows how the net present value of a project may vary with the implementation year.

The best time of implementation for a project is the year which maximizes the net present value of the project. The diagram in Figure 7.1, for example, indicates that the net present value of the project reaches its maximum in 1983. Thus, the implementation year of this particular project should be 1983.

Under no budgetary constraints, therefore, the priority rule is very simple. Every project must be done in the year which would result in maximum net present value. Heggie (89), shows that for most conditions net present values are maximized when first year rate-of-return of the annual benefits relative to the cost first exceeds the discount rate. Therefore, first year rate-of-return criteria could also be used for multiple year project programming under no budgetary constraints.

With budget constraints the problem becomes one of minimizing the loss in the net present value of all projects due to movements of projects from their optimum implementation years. Stated in another way, the total net present value has to be maximized without violating the budget constraints which are known for all the years in the programming period.

This maximization can be achieved in several ways. A heuristic approach can be applied in which the projects are first placed in their optimum implementation years. Then, they are advanced or postponed on the basis of their marginal net benefit-cost ratios until the budget

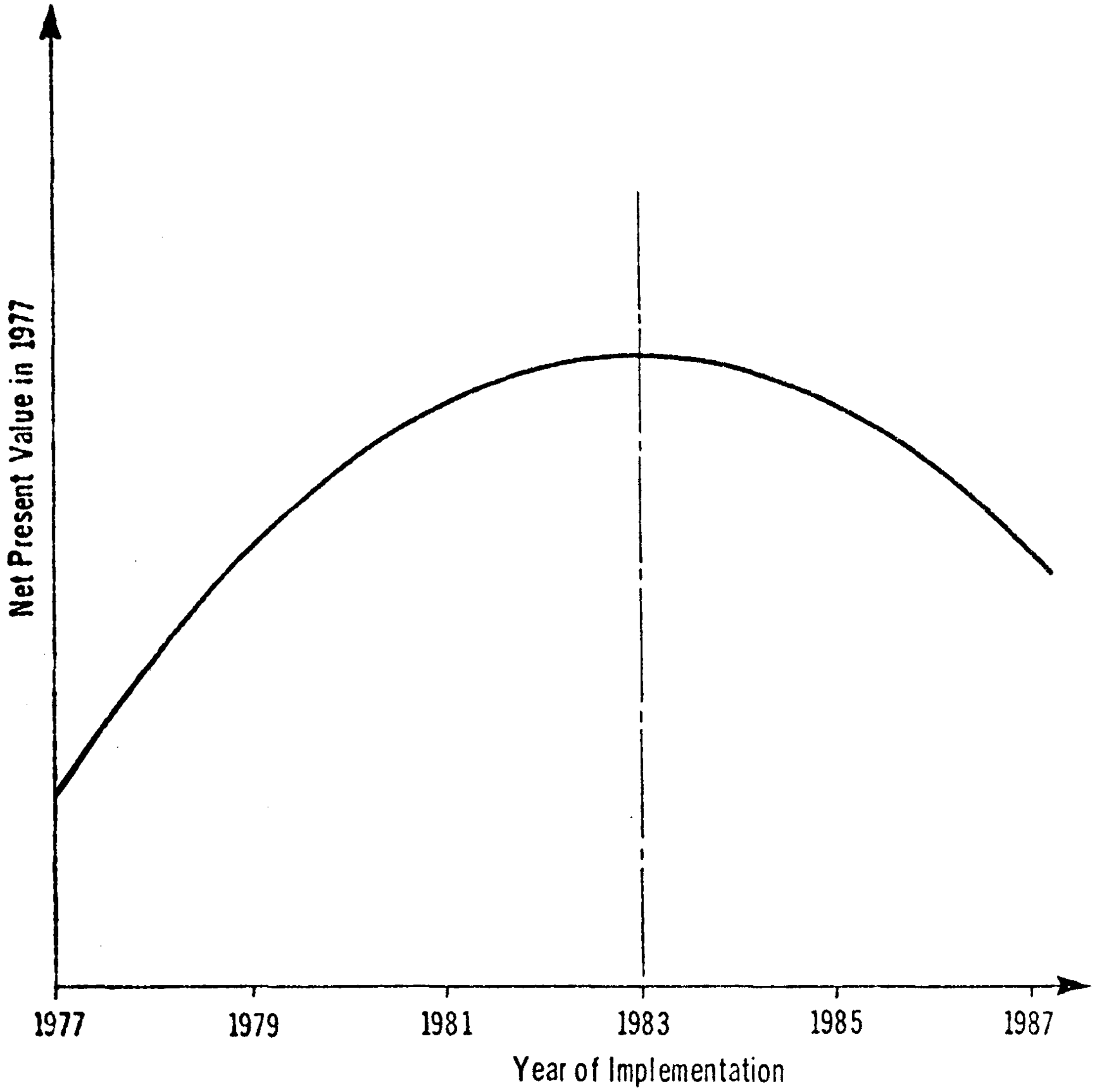


FIGURE 7.1 - VARIATION OF THE NET PRESENT VALUE IN 1977 WITH THE YEAR OF IMPLEMENTATION

constraints are satisfied. Heggie (89), for example, suggests a decision model to do this type of analysis.

Linear programming can also be used for solving the multi-year priority programming problem. In fact, it has been shown that it provides an efficient, easy to understand and easy to use approach for maximizing total net benefits under budget constraints (89, 95, 96).

It is mainly because of its simplicity and efficiency that the following linear programming model has been used in this study.

7.3 LINEAR PROGRAMMING MODEL FOR PRIORITY PROGRAMMING

7.3.1 Formulation of the Model

The formulation of a L.P. model for maximizing the total present value of m pavement improvement projects, each with k within-project alternatives, can be done in the following way, for a t years programming period:

$$\text{Maximize} \quad \sum_{i=1}^{m} \sum_{j=1}^{k} \sum_{t=1}^{t} X_{ijt} \cdot B_{ijt} \qquad \ldots\ldots\ldots\ldots\ldots(7.1)$$

Subject to

$$\sum_{t=1}^{t} \sum_{j=1}^{k} X_{ijt} \leqq 1 \quad \text{for} \quad i = 1,2,..m \qquad \ldots\ldots(7.2)$$

$$\sum_{i=1}^{m} \sum_{j=1}^{k} X_{ijt} \cdot D_{ijtt'} \leqq B_t \quad \text{for} \quad t = 1,2,\ldots 10 \qquad \ldots\ldots(7.3)$$

$$X_{ijt} \geqq 0 \qquad \ldots\ldots(7.4)$$

where: X_{ijt} = the fraction of alternative j of project i started in year t, where t is taken from 1 to 10 years in this study,

B_{ijt} = present value of annual benefits (including salvage value) of project i, with alternative j, built in year t, all discounted to base year at a discount rate of R,

$D_{ijtt'}$ = the actual construction and/or maintenance cost of project i, with alternative j, built in year t, incurred in year t',

B_t = budget for year t.

Equation (7.1) is the maximization of benefits. It should be noted that maximization of net benefits (benefits minus costs) is not necessary since construction and maintenance costs are specifically dealt with in the budget constraints (95). The L.P. model therefore implicitly takes into account the costs in the maximization process. Thus, there is no need for discounting costs back to a base year to calculate the net present value of the projects. This is one of the main advantages of the L.P. approach used in that it eliminates the use of a discount rate for costs. In fact, the L.P. model calculates an appropriate discount rate for each year in the programming period, depending on the magnitude of the budget constraints.

Equation (7.2) states that a project can be built once or may not be built at all. If a project has to be built (i.e., a "committed project") then an equality constraint has to be used.

Equation (7.3) states that the budget in any year within the programming period cannot be exceeded. Equation (7.4), which is the common constraint of all linear programming problems, states that it is not possible to recapture construction costs by not constructing the project.

Table 7.1 shows an example L.P. formulation for m pavement improvement projects, k within-project alternatives and t years programming period. The within-project alternatives are defined as various rehabilitation strategies that are available for urban pavements. In Chapter 5 these were defined as single and double-lift overlays, reconstruction and re-mix. The value of k, therefore, is equal to four in this study.

The selection of an appropriate programming period (t) is important for the accuracy of the results. A priority list determined for a relatively short programming period (i.e., 5 years) may be unrealistic because it does not give a chance to all projects to appear in the priority list. Thus, a short programming period does not allow all projects to compete with each other. This may lead to significant errors in the priority list.

In this study, a ten year programming period has been used with the assumption that most of the pavements in a network require some action in ten years. Most of the projects, if not all, should therefore appear in the priority list. A subsequent study, which will be discussed in Chapter 8, has supported this assumption.

Table 7.1SAMPLE LINEAR PROGRAMMING FORMULATION

	YEAR 1				YEAR 2					YEAR t			
Alternative / Project	1	2	3 ...	k	1	2	3 ...	k		1	2	3 ...	k
1	X_{111}	X_{121}	X_{131}	X_{1k1}	X_{112}	X_{122}	X_{132}	X_{1k2}		X_{11t}	X_{12t}	X_{13t}	X_{1kt}
2	X_{211}	X_{221}	X_{231}	X_{2k1}	X_{212}	X_{222}	X_{232}	X_{2k2}		X_{21t}	X_{22t}	X_{23t}	X_{2kt}
3													
m	X_{m11}	X_{m21}	X_{m31}	X_{mk1}	X_{m12}	X_{m22}	X_{m32}	X_{mk2}		X_{m1t}	X_{m2t}	X_{m3t}	X_{mkt}

X_{mkt} = The fraction of alternative k of project m started in year t

7.3.2 Input Requirements

The L.P. model described in the previous Section requires that the benefits and costs of each within-project alternative should be calculated separately for each possible implementation year in the programming period.

If, for example, the pavement under consideration is expected to reach its terminal serviceability level in the sixth year of a ten year programming period, then the benefits and costs of each alternative are first calculated for the sixth year, assuming that the project will be implemented in this year. Then, the project is delayed to year seven, and benefits and costs are recalculated for this new implementation year. These calculations are repeated three more times until the whole programming period is covered.

This procedure assumes that projects cannot be started until they reach their terminal serviceability levels. This can be achieved in the L.P. model by assigning high costs (i.e., 999999 dollars) and zero benefits to the years in which a project cannot be started. In the foregoing example, therefore, the benefits and costs should artificially be assigned to the first five years of the programming period.

The delay of a pavement project shifts the benefit stream to the right and changes the present value of benefits in the base year, as shown in Figure 7.2. The annual unit benefit which occurs in year 6, is shifted to year 7 and 8 by one and two year delays, respectively. Similarly, the other annual unit benefits are also shifted to the years which are determined by the amount of delay.

The costs are similarly affected by the delay, as shown in

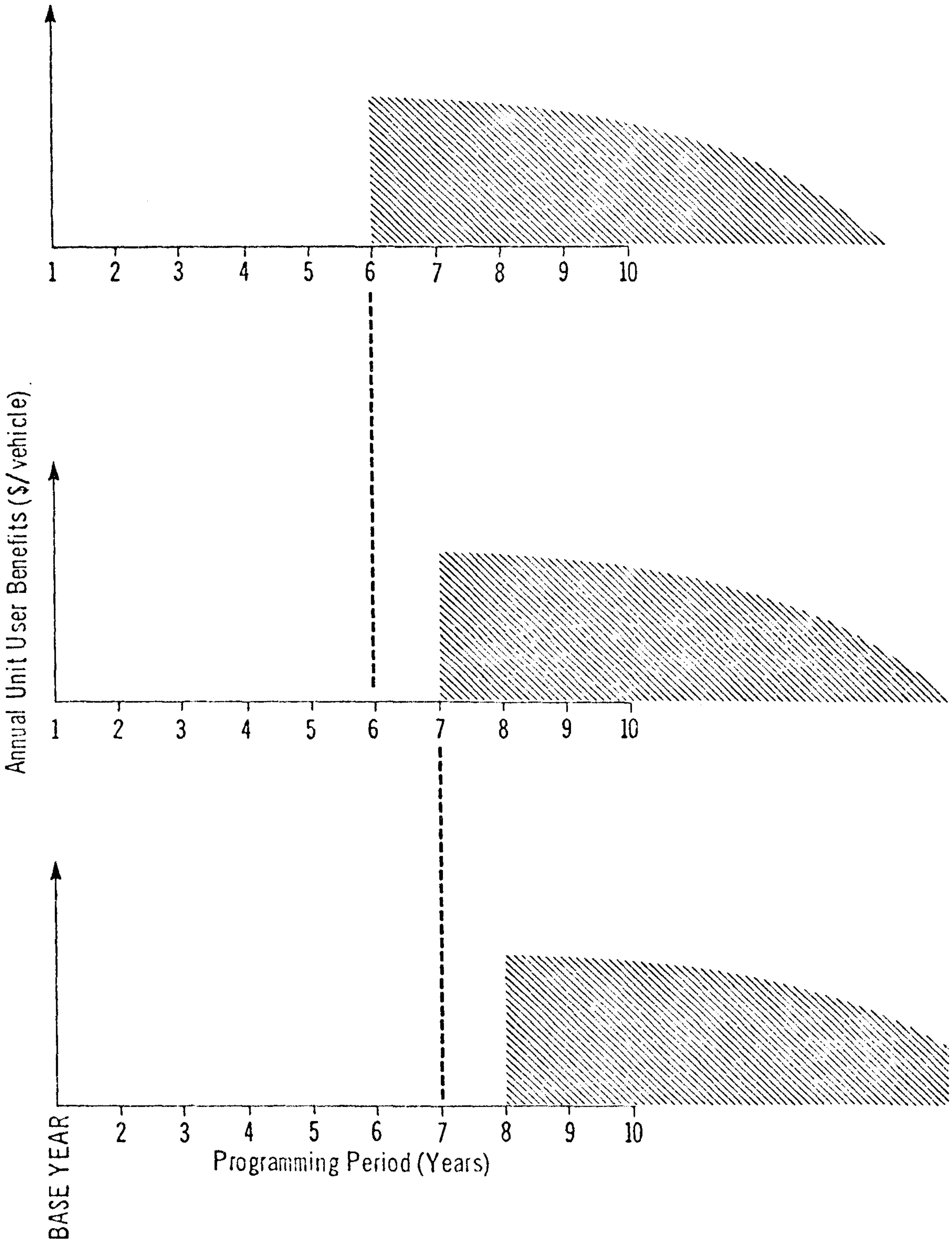


FIGURE 7.2 - THE EFFECT OF PROJECT DELAY ON USER BENEFITS

Figure 7.3. The upper portion of Figure 7.3 assumes that the pavement is constructed in the sixth year. There is a construction cost in that year followed by routine maintenance costs in the following years. A one-year delay results in a higher extra maintenance cost* in year six. This maintenance cost is spent to keep the pavement at its minimum acceptable serviceability level during the year in which the project is delayed. Then, the same costs (i.e., construction cost, routine maintenance cost) occur in the same order starting from the seventh year. A two-year delay means additional maintenance costs in the sixth and seventh years, followed by the same sequence of costs.

It should be noted that costs are considered directly in the L.P. model up to the end of the programming period. After that, costs cannot be taken into account because of the absence of budget constraints. It is for this reason that costs incurred in each year after the programming period are considered in the analysis indirectly by subtracting them from the respective annual benefits.

All the benefits and costs are calculated for each project alternative and implementation year combination as shown in Tables 7.2 and 7.3. This information is then used in the L.P. model together with the budget constraints.

Table 7.4 gives an example for calculating an element (B_{ijt}) in the benefit matrix of Table 7.2.. In this example, the project is assumed to be built in the fourth year of the programming period. Zero benefits and high costs (i.e., 999,999 dollars), therefore, are artificially assigned to the first three years in the programming period.

Annual benefits (B_t') are calculated for each year from

(*) These higher extra maintenance costs are assumed to be constant over the years.

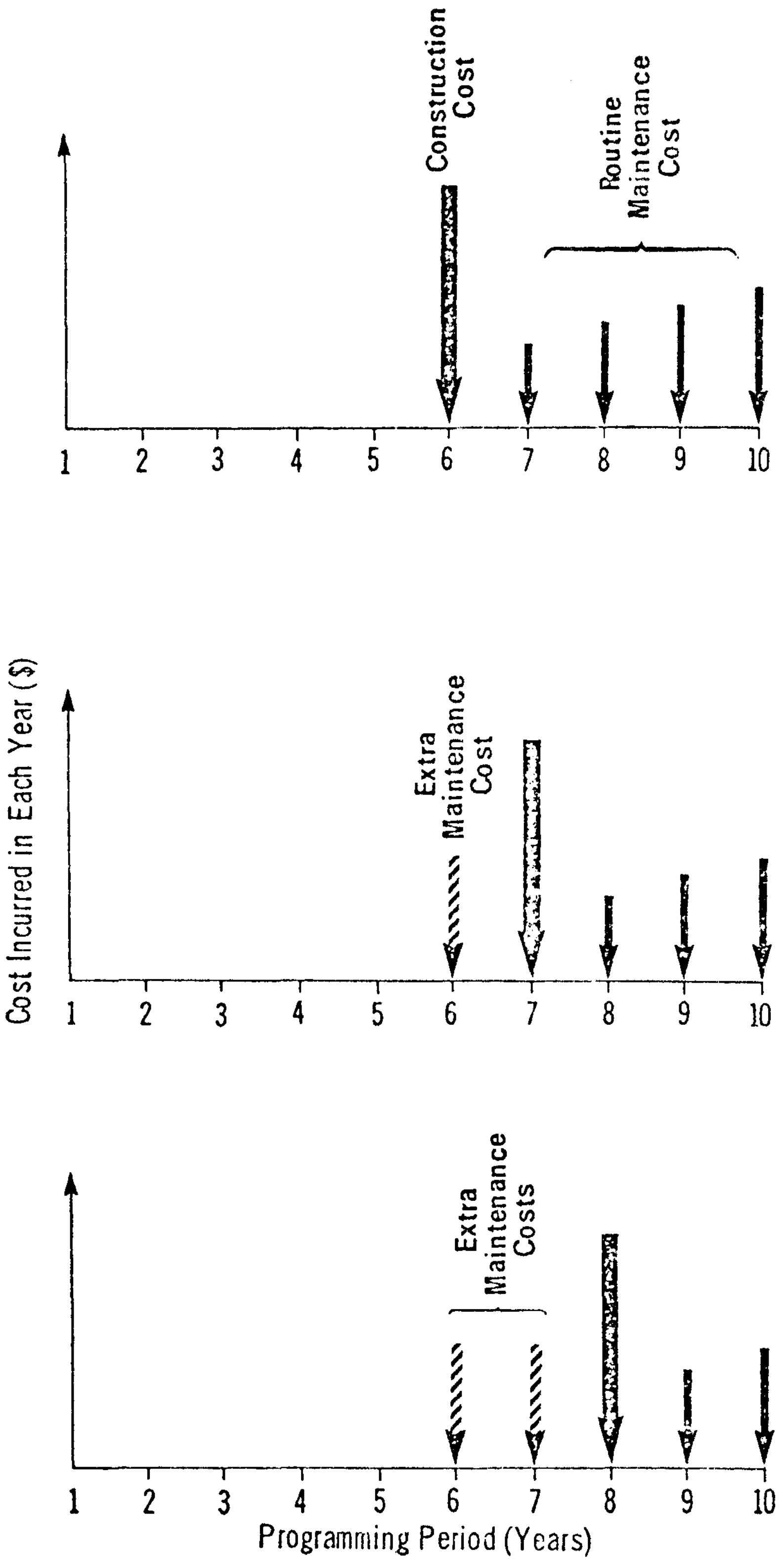


FIGURE 7.3 - THE EFFECT OF PROJECT DELAY ON COSTS

TABLE 7.2

SAMPLE BENEFIT MATRIX FOR L.P. MODEL

	YEAR 1				YEAR 2				YEAR 10			
Alternative / Project	1	2	3	4	1	2	3	4	1	2	3	4
1	B_{111}	B_{121}	B_{131}	B_{141}	B_{112}	B_{122}	B_{132}	B_{142}	B_{1110}	B_{1210}	B_{1310}	B_{1410}
2	B_{211}	B_{221}	B_{231}	B_{241}	B_{212}	B_{222}	B_{232}	B_{242}	B_{2110}	B_{2210}	B_{2310}	B_{2410}
3												
⋮												
M	B_{m11}	B_{m21}	B_{m31}	B_{m41}	B_{m12}	B_{m22}	B_{m32}	B_{m42}	B_{m110}	B_{m210}	B_{m310}	B_{m410}

where: B_{ijt} = Present value of annual benefits of project i, with alternative j built in year t, all discounted to base year at a discount rate of R.

TABLE 7.3

SAMPLE COST MATRIX FOR A PROJECT

			IMPLEMENTATION YEAR				
			1	2	3		10
PROJECT i	ALTERNATIVE	1	CC_{i111} MC_{i121} ⋮ MC_{i1101}	RM_{i112} CC_{i122} ⋮ MC_{i1102}	RM_{i113} RM_{i123} ⋮ MC_{i1103}		RM_{i1110} RM_{i1210} ⋮ CC_{i11010}
		2					
		3	CC_{i311} MC_{i321} ⋮ MC_{i3101}	RM_{i312} CC_{i322} ⋮ MC_{i3102}	RM_{i313} RM_{i323} ⋮ MC_{i3103}		RM_{i3110} RM_{i3210} ⋮ CC_{i31010}
		4					

where: $CC_{ijtt'}$ = the actual construction cost of project i, with alternative j, built in year t, incurred in year t',

$RM_{ijtt'}$ = the actual routine maintenance cost of project i, with alternative j, built in year t, incurred in year t',

$MC_{ijtt'}$ = the actual maintenance cost of project i, with alternative j, built in year t, incurred in year t'.

EXAMPLE BENEFIT CALCULATION FOR PROJECT (i) ALTERNATIVE (j), BUILT IN YEAR 4

			Year (t')	Total Annual Benefit ($B_{t'}$) in Year t'	Cost ($C_{t'}$) incurred in Year t'	$B_{t'} - C_{t'}$	Discounted Benefit	Subtotal
Programming Period			1	0	999999	-	-	
			2	0	999999	-	-	
			3	0	999999	-	-	
	Analysis Period	Project Life	4	120200	72000	-	120200(1)	
			5	115120	1600	-	106601	
			6	107318	1600	-	91972	
			7	101536	1800	-	80620	
			8	90165	2240	-	66271	
			9	82108	2240	-	55916	
			10	72218	3200	-	45497	567077(3)
			11	60567	3200	57367	53122(2)	
			12	47493	3200	44293	37959	
			13	33464	4320	29144	23140	
			14	18967	4320	14647	10766	
			15	4415	4320	95	65	
			16	12960	0	12960	8165	133217(4)
						Total Discounted Benefits		651004(3)
						B_{ij4}		516897(5)

(1) Total annual benefits ($B_{t'}$) are discounted to project implementation year 4.

(2) ($B_{t'}-C_{t'}$)'s are discounted to year 10.

(3) In 4th year dollars.

(4) Salvage value at the end of the analysis period (in 10th year dollars)

(5) In 1st year dollars.

Figure 6.2. In year 4, for example, the average unit benefit gained from the project is calculated by subtracting the vehicle operating cost which corresponds to the serviceability level in that particular year (assumed to be 7.6 in the example), from the operating cost that occurs at the terminal serviceability level of 4.0. This average benefit per vehicle mile can be determined as 0.03 dollars, from Figure 6.2. The total benefit for the year is then calculated as 120,200 dollars by multiplying the average unit benefit (0.03 dollars) by the average daily traffic (i.e., 11448 vehicles), number of days in a year (i.e., 350) and the project length (i.e., 1.0 mile).

The annual benefits that occur in the analysis period (i.e., years 4 to 10) are then discounted back (at a rate of 8 percent) to project implementation year 4.

The benefits that occur after the analysis period (i.e., between years 11 and 16) are treated separately. Costs (C_t') are deducted from the benefits, starting from year 11. The difference (B_t' - C_t') in each year is then discounted back to the end of the analysis period (i.e., year 10), and summed up to calculate the salvage value. This salvage value, which is 133,217 dollars in the example, is in 10th year dollars and includes both the salvage value of the materials at the end of the service life (i.e., 12,960 dollars) and the user benefits that occur after the analysis period.

This salvage value is then discounted back to the fourth year and summed up with the discounted annual benefits that occur in the analysis period. This total value (651,004 dollars) is the total present value of the benefits in year 4.

This total is then discounted back to the beginning of the programming period (i.e., year 1) and expressed in first year's dollars. Thus, 516,897 dollars, in the example, is the B_{ij4} element in the overall benefit matrix of Table 7.2. The rest of the benefit elements in that matrix are calculated in a similar way as shown in this example.

The other input requirement of the model is the budget for each year in the programming period. Thus budget should be only that concerned with pavement improvements. The total construction or maintenance budgets that are normally available in an urban road agency cannot be used in this priority programming formulation. The pavement improvement budget should be separated from the others and used in the analysis.

7.3.3 Results of the Model

The L.P. model establishes priorities for urban pavement improvements, over the whole network, on the basis of benefit maximization and budget constraints. Since a linear programming formulation is used, however, the solution includes fractional values (96). In other words, some split projects, in terms of time and/or alternative, may appear in the priority list. The program may suggest the implementation of a certain percentage of a project(s) in one year and the rest in other years. Or, one alternative may appear to be the best for a certain percentage of a project in one year and another alternative in other years.

Weingartner (96) shows that the number of these fractional solutions in an L.P. formulation cannot exceed the number of years in the programming period. In this study, therefore, a maximum of ten split projects may appear in the priority list. In common practice these split projects could be assigned to the year and alternative with the largest fraction. Experience has shown that this does not create any difficulty (89, 95, 96).

In addition to the priority list, the L.P. model gives some information about the discount rate for capital costs in each budget year. The dual variable of a budget constraint reflects the value of a dollar in that particular year relative to the value in other years in the programming period (i.e., shadow price). By examining these dual variables one may decide the level of budget required in a year, relative to other years.

Similarly, the dual variables of each project constraint give the "shadow price" of each project. In other words, the relative importance of a project is given by the dual variable of that particular project's constraint. The higher the value of the dual variable, the better the project is in terms of its effect on the benefit maximization.

The L.P. model also gives the maximum total benefits that will be obtained from the implementation of the priority list. The amount of budget that is used in each year, and left over, if any, is also given in the output.

The effect on the objective function of delaying a project is also given in the L.P. solution.

A sample application of the model to a simple problem can be found in References (97, 98).

7.4 GENERAL DISCUSSION OF THE PRIORITY PROGRAMMING MODEL

The L.P. approach for multi-year priority programming explicitly recognizes the trade-offs between benefit losses and project timing. It treats each possible implementation year as an independent alternative by delaying the construction of the project over the programming period. In addition to this, the within-project alternatives are also considered in the analysis. Thus, all possible combinations of project type and timing are considered and compared in the linear program, and the best ones are selected for implementation on the basis of their economic consequences.

Policy variables such as regional development policies, can be taken into account in the L.P. model by using special weighting factors. Benefits of the projects in a particular area can be weighted differently from the benefits of other projects in other areas. Thus, special consideration can be given to certain projects in the L.P. method of priority programming (95).

This can be a major advantage in urban areas in which political issues play an extremely important role in determining priorities. The weighting system, for example, can be used to spend the urban pavement improvement budget as uniformly as possible over the whole city, or to ensure that certain wards or areas are given particular consideration.

Similarly, dependent and sequentially dependent projects and staging problems can be taken into account in the L.P. model. In addition to these advantages the following major types of priorities can be considered in the L.P. approach:

1. Priorities based on identifiable economic benefits,
2. Priorities "predetermined" or committed on the basis of larger, overall improvement projects,

3. Priorities subjectively established where benefits cannot be quantified.

The pavement portion of a capacity improvement project, for example, can be taken into account as a "committed" project in the L.P. model. Similarly, a safety improvement or a subjective political or engineering decision to carry out an improvement (such as a "spot improvement") represented by the third type of priority listed, can be included in the L.P. priority programming model.

The L.P. approach used eliminates costs from the objective function. This has the advantage that the discount rate selected is used only for discounting benefits over time. Costs are not discounted; they are used in actual terms in the budget constraints.

The dual variables of the L.P. model give valuable information to the analyst about the relative importance of each project and the shadow price of a dollar in each year in the programming period. This information can be used for adjusting the priority list to avoid split projects.

A zero-one integer program would avoid the problem of fractional or split projects. It is, however, quite difficult to find a computer program which can handle large size problems (99). In this study, for example, a zero-one integer linear program by Geoffrion and Nelson (100) was tried, without success. It was not capable of being used on other than very small problems.

IBM's mathematical programming package (101, 102) could potentially be quite efficient for this problem. It's cost, however, was far in excess of the resources available to the study. Moreover, there is no assurance that it would give any better results, in a practical sense, than the L. P. formulation.

The fractional solutions of the L.P. model can be interpreted in terms of both cost and length of a project. If, for example, fifty percent of a project (i.e., a two-lift resurfacing alternative) is recommended to be built by the program in a certain year, either half of the project, in terms of length, can be built or a single-lift can be placed over the entire length of the project instead of two-lifts.

The second interpretation seems to be more logical for the urban situation. In the common application of the L.P. model, however, these split projects are usually assigned to the year with the highest function. This assignment process, which can be done in several ways (89). may violate the overall optimality, but in practice the optimums cannot always be achieved and/or implemented.

It should be realized that the output of the program is just a tentative priority list that is based purely on economic considerations. The decision maker may well modify it to take into account those subjective factors that cannot formally be considered in an engineering model. An economic analysis should only form the basis for decision making; that is, it provides a guide; it does not provide a decision by itself.

The priority program developed in this study uses the concept of terminal serviceability as a screening process. Projects are not considered until they reach their minimum acceptable serviceability levels. This may be a serious limitation because, under certain circumstances, the rehabilitation of a pavement before its terminal serviceability level is reached may be more economical. In other words, a variable terminal serviceability level may result in a true-optimum, whereas the screening process, with a fixed action level, may give a sub-optimum solution. However, as ideal as a variable terminal serviceability level may be in a theoretical sense, it may also represent an academic refinement inconsistent with practical considerations and the gross assumptions or variations occurring in other inputs and models in the total system.

It should be noted though that a screening process is always needed even with the variable terminal serviceability level criteria. It may, therefore, be argued that the true-optimum can be approached but never achieved.

On the other hand, in the urban situation where there is a huge demand for pavement improvements with very limited funds, it may be quite unacceptable to rehabilitate a relatively good pavement, even though it represents an optimal solution based on variable terminal serviceability criteria, traffic volumes, etc., while other pavements in worse condition are not improved.

It was, therefore, felt that screening with a fixed terminal level criteria is more realistic, at least at this time, for the urban situation. This fixed level might also be justified from the safety point of view.

The other limitation of the L.P. model is that the result of the model depends mainly on its inputs. For this reason, special consideration should be given to defining and calculating the costs and benefits of a project. Any error in the input may result in significant changes in the output of the model.

In conclusion, the L.P. model of priority programming developed in this study seems to be quite efficient for programming urban pavement improvements. It suffers from certain limitations, but in the current state of the technology its advantages overshadow these limitations. Previous experience (89, 95) has demonstrated that the method is practical and useful for the purposes of priority programming.

COURSE NOTES - SUPPLEMENTARY READING

Lesson 31 Priority Programming

OPTIMIZATION OF PAVEMENT REHABILITATION AND MAINTENANCE USING INTEGER PROGRAMMING

by

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ABSTRACT

An integer programming technique has been used to develop an operating computer program (RAMS) which determines optimal maintenance strategies for pavements. This is accomplished by maximizing the overall maintenance effectiveness for all highway segments considered. The program can use numerous maintenance strategies, resources, and feasibility constraints to obtain solutions. An example problem containing actual field data on fifteen highway segments located in one highway district in Texas was used to demonstrate typical program input and output. This example revealed that maintenance strategies selected by the computer program were essentially identical to those selected by district TSDHPT personnel for nine of the fifteen pavement segments studied. The differences between RAMS and TSDHPT selections are examined.

OPTIMIZATION OF PAVEMENT REHABILITATION AND MAINTENANCE USING INTEGER PROGRAMMING

INTRODUCTION

Optimization techniques are being applied to the problem of allocating highway rehabilitation and maintenance funds because of their established record in industry of saving around 10 to 25 percent of equipment maintenance budgets (1). If this kind of record can even be approached in the area of highway maintenance, very significant savings to the nation can be realized.

This paper describes the solution to such a problem by an operating computer program called RAMS (Rehabilitation and Maintenance Strategies). The program uses an integer programming technique that is based upon a mathematical model of the optimization process which was formulated by Lu and Lytton (2). The program is part of a methodology presently being developed by the Texas Transportation Institute for the Texas State Department of Highways and Public Transportation (TSDHPT).

The approach described here is different from what has been tried elsewhere. The University of California at Berkeley has developed an optimization computer program called CALMS 1 which uses a Markov process for describing the transition from one pavement condition state to another (3). Two kinds of pavement condition states are considered, roughness and cracking, and these are treated with three major alternative strategies: thin, medium, and heavy overlays. In a similar development, the Washington State Department of Highways has developed an optimization procedure for their highway system (4).

The present method recognizes that a large number of maintenance and rehabilitation strategies are in fact used by all transportation agencies ranging from seal coating, through patching and overlaying, to complete reconstruction of the pavement. The problem described in this paper recognized five types of distress and six maintenance and rehabilitation strategies. The program is written flexibly so that either more or fewer distress types and maintenance strategies may be used.

The purpose of this paper is to describe how optimal maintenance solutions for highway segments are obtained using the RAMS program and to show the results of the solution of an actual problem with a group of highway segments located in Texas, complete with a general description of the required inputs.

DESCRIPTION OF THE OPTIMIZATION METHODOLOGY

The mathematical model to maximize the overall effectiveness of maintenance activities as applied to highways may be formulated in terms of 0 - 1 integer programming which may be written as follows:

$$\text{Maximize} \sum_{i=1}^{N_H} \sum_{j=1}^{N_S} \sum_{k=1}^{N_D} \sum_{t=1}^{N_T} L_{1i} L_{2i} d_{ijk} P_{ijkt} x_{ij} \qquad (1)$$

subject to the following constraints:

Decision variable

$$\sum_{j=1}^{N_S} x_{ij} \leq 1 \qquad i = 1,2,\ldots,N_H \qquad (2)$$

Available supplies

$$\sum_{i=1}^{N_H} \sum_{j=1}^{N_S} s_{ijg} L_{1i} L_{2i} x_{ij} \leq S_g \qquad g = 1,2,\ldots,N_G \qquad (3)$$

Available equipment

$$\sum_{i=1}^{N_H} \sum_{j=1}^{N_S} e_{ijf} L_{1i} L_{2i} x_{ij} \leq E_f \qquad f = 1,2,\ldots,N_F \qquad (4)$$

Available manpower

$$\sum_{k=1}^{N_H} \sum_{j=1}^{N_S} h_{ijq} L_{1i} L_{2i} x_{ij} \leq H_q \qquad q = 1,2,\ldots,N_Q \qquad (5)$$

Available budget

$$\sum_{i=1}^{N_H} \sum_{j=1}^{N_S} C_{ij} L_{1i} L_{2i} x_{ij} \leq C \qquad (6)$$

Minimum rating for each distress type

$$r_{ik} + \sum_{j=1}^{N_S} d_{ijk} P_{ijkt} x_{ij} \geq R_{ikt} \qquad \begin{array}{l} i = 1,2,\ldots,N_H \\ k = 1,2,\ldots,N_D \\ t = 0,1,\ldots,N_T \end{array} \qquad (7)$$

Minimum overall pavement rating score

$$\sum_{k=1}^{N_D} \{r_{ik} + \sum_{j=1}^{N_S} d_{ijk} P_{ijkt} x_{ij}\} \geq W_{it} \qquad i = 1,2,\ldots,N_H \qquad (8)$$

where

N_H = number of highway segments in analysis;

N_S = number of maintenance strategies;

N_D = number of distress types;

N_T = number of years in analysis period;

L_{1i} = pavement length in miles of highway segment i;

L_{2i} = pavement width in feet of highway segment i;

d_{ijk} = potential gains of pavement rating of highway segment i, maintenance strategy j and distress type k;

P_{ijkt} = pavement survival probability of highway segment i, maintenance strategy j and distress type k, at time t;

x_{ij} = a decision variable which will be 1 if maintenance strategy j is selected for highway segment i, and 0 otherwise;

s_{ijg} = amount of material (or supply) type g per unit surface area (one mile long and one foot wide) required in highway segment i, if maintenance strategy j is selected;

S_g = total amount of material (or supply) type g available;

N_G = number of different material (or supply) types;

e_{ijf} = amount of equipment type f (in equipment-days per unit one mile long and one foot wide surface area) required in highway segment i, if maintenance strategy j is selected;

E_f = total amount of equipment type f (in equipment-days) available;

N_F = number of different equipment types;

h_{ijq} = amount of manpower type q (in man-days per unit one mile long and one foot wide surface area) required in highway segment i, if maintenance strategy j is selected;

H_q = total amount of manpower type q (in man-days) available;

N_Q = number of different manpower types;

C_{ij} = cost in dollars per unit one mile long and one foot wide surface area required in highway segment i, if maintenance strategy j is selected;

C = total budget available (in dollars);

r_{ik} = current pavement rating of highway segment i and distress type k;

R_{ikt} = minimum required pavement rating of highway segment i and distress type k at time t;

W_{it} = minimum required pavement rating of highway segment i of all distress types at time t.

Solution Procedure

Optimizing the maintenance strategies for a large number of highway segments with numerous strategies, resources, and feasibility constraints exceeds the capacity of current mathematical integer programming techniques to achieve exact optimal solutions. The problem, formulated by use of integer programming, is solved by Senju and Toyoda's (5) "effective gradient" method which achieves near optimal solutions.

Effective Gradient Method

Consider a simple example by using five highway segments. The data for these highway segments comes from a larger, more realistic problem which will be discussed later. The goal of this short example is to demonstrate by use of the effective gradient method how the five segments can be maintained optimally. For simplicity it is assumed that only one maintenance strategy and two resources are needed. The maintenance strategy chosen is reconstruction and the two resources are the amount of budget and materials available to accomplish the work. The RAMS problem presented later actually considers six maintenance strategies and the resources of materials, equipment, manpower, and budget.

Table 1 shows a listing of the five segments (designated H_1, H_2, ..., H_5) and the percentage of the total resources used for each. These segments correspond to the last five segments shown in Table 4. The maintenance strategy that is considered is reconstruction with a total available budget of $300,000. The cost to reconstruct each segment was obtained by multiplying the length and width by the cost per unit area. The percentage of materials required by each segment was assumed to approximate the

Table 1. Resource Requirements for Five Highway Segments

Highway Segment	Percent of Total Available Budget Resource Used	Percent of Total Available Material Resource Used	Maintenance Effectiveness
H_1	74	70	6507
H_2	46	45	4072
H_3	76	70	3863
H_4	42	40	78,109
H_5	47	50	78,355
Total Required	285	275	170,906
Total Available (Limit)	100	100	
Extra Resource Required	185	175	

Total Available Budget = $300,000

percentage of the budget consumed. The total required for each resource is shown and is the sum of the individual percentages for each highway segment. For the budget resource, the total required is larger than the available budget by a factor of 2.85. A similar situation occurs for the material resource.

Maintenance effectiveness is also shown in Table 1 and is computed from the objective function in Equation 1. Thus, the maintenance effectiveness is obtained by multiplying together the length, width, gain-of-rating for each distress, and the sum of the survival probabilities (gain-of-rating and survival probabilities will be discussed in more detail later in the paper). The maintenance effectiveness would be greater for highly distressed pavements as opposed to nondistressed pavements of equal length and width. Highway segment H_5 will be used to demonstrate how maintenance effectiveness is computed. For H_5:

1. Length = 7.444 mi (11.980 km)
2. Width = 20 ft (6.1 m)
3. Gain-of-rating points for reconstruction for distress types present on roadway:

Distress type		Maximum Points Available		Current Condition Rating		Gain-of-Rating
(a) Rutting	=	15	-	10	=	5
(b) Alligator cracking	=	25	-	10	=	15
(c) Longitudinal cracking	=	25	-	10	=	15
(d) Transverse cracking	=	20	-	8	=	12
(e) Failures/mile	=	40	-	20	=	20

4. Probability of survival for reconstruction summed over ten years for distress types present on roadway:

(a) Rutting = 7.97

(b) Alligator cracking = 6.86

(c) Longitudinal cracking = 9.25

(d) Transverse cracking = 9.25

(e) Failures/mile = 6.69

5. Maintenance effectiveness $= L_{15}L_{25} \sum_{i=1}^{1} \sum_{j=1}^{1} \sum_{k=1}^{5} \sum_{t=1}^{10} d_{ijk}P_{ijkt}$

$$= (7.444)(20)\{(5)(7.97) + (15)(6.86) + (15)(9.25) + (12)(9.25) + (20)(6.69)\} = 78{,}355$$

In Figure 1 the vectors $\overline{H}_1$, $\overline{H}_2$, ..., $\overline{H}_5$ are plotted as a function of the required resources for each highway segment, i.e., $\overline{H}_1$ denotes the amount of budget and materials required if reconstruction is done to this segment. The following vectors are defined:

Let $\overline{R}$ = resultant vector of all highway segments

$= \overline{H}_1 + \overline{H}_2 + \overline{H}_3 + \overline{H}_4 + \overline{H}_5$

$\overline{L}$ = limiting resources vector

= (100,100) in example

$\overline{E}$ = excess vector

$= \overline{R} - \overline{L} = (285,275) - (100,100) = (185,175)$

If enough resources are available to reconstruct all five highway segments, that is what should be done. Of course, this situation will rarely occur. Resources are generally scarce so maintenance cannot be applied to all the highway segments being considered. The maintenance should be applied to that combination of highway segments that maximize the overall maintenance effectiveness and satisfy the available resource restraints. Thus, some method must be used to determine which segments are dropped from consideration.

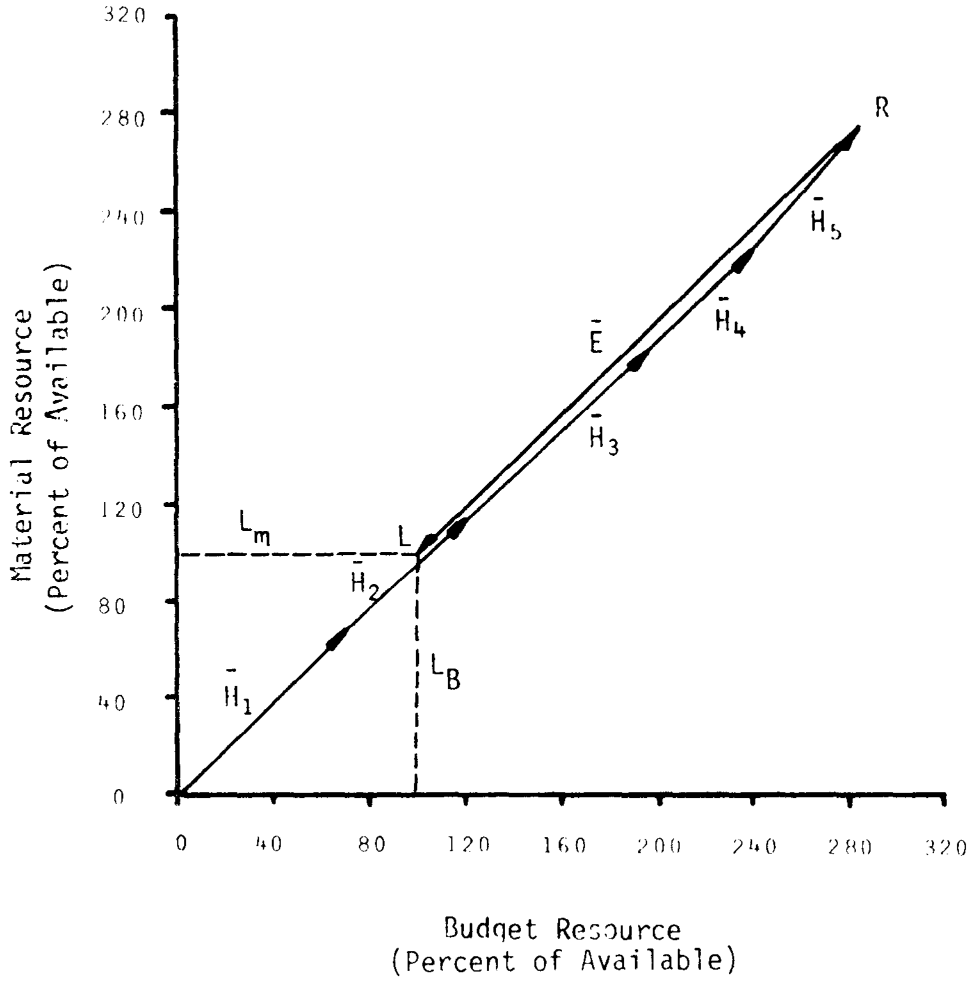

Figure 1. Vector Sum of Resource Requirements For Each Highway Segment

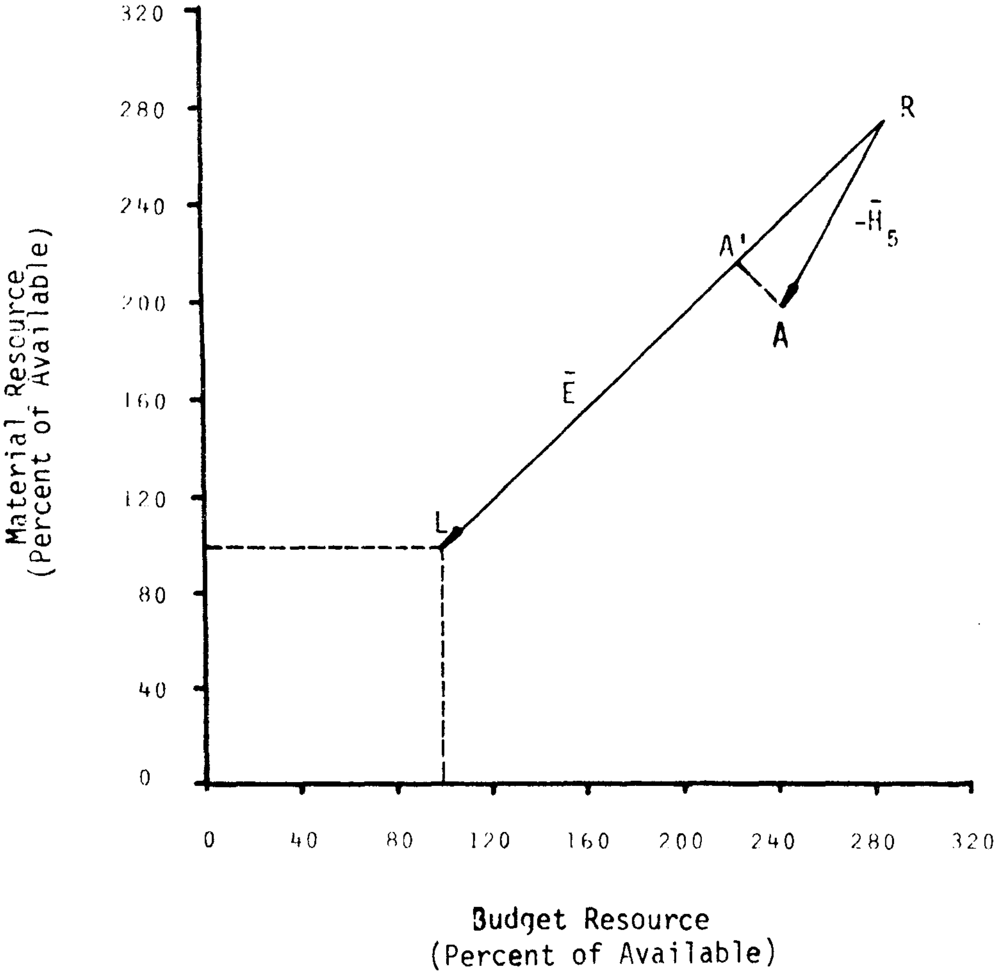

Figure 2. Effective Reduced Length For Highway Segment 5

Figure 2 shows highway segment H_5 being dropped. This caused the point R to move in the general direction of L and 78,355 units of maintenance effectiveness is lost. Highway segment H_5's contribution toward moving back toward L (to satisfy the resource availability constraint requirement) is expressed by the projected length of vector $\overline{H}_5$ on the excess vector $\overline{E}$ (denoted by $\overline{A'R}$). The decision to drop a highway segment should be based on a comparison of maintenance effectiveness with the projected length on the vector $\overline{E}$. This comparison determines the "effective gradient" and is taken as the ratio of maintenance effectiveness for a highway segment to the projected length $\overline{A'R}$ for that highway segment. Phrased another way, effective gradient indicates which highway segments have the greatest maintenance effectiveness for the smallest amount of resources. Highway segments with small effective gradients are less desirable to schedule for maintenance than segments with large effective gradients. Therefore, the effective gradient for each segment is calculated and those segments with the smallest gradients are dropped until the availability resource constraints are satisfied.

The effective gradient for each highway segment is shown in Table 2. The following will demonstrate how the effective gradient is calculated. Let $\overline{U}$ stand for a unit vector parallel to $\overline{E}$ and with the same sense.

$$\overline{U} = \overline{E}/|\overline{E}|$$

and from the example

$$\overline{U} = \{185/(185^2 + 175^2)^{1/2},\ 175/(185^2 + 175^2)^{1/2}\}.$$

Let U_5 = projection of vector $-\overline{H}_5$ on vector $-\overline{U}$ where U_5 is given by the scalar product of vectors $-\overline{H}_5$ and $-\overline{U}$

Table 2. Five Highway Seqments Ranked By Effective Gradient

Proposed Order	Effective Gradient
H_3	37
H_2	63
H_1	64
H_5	1144
H_4	1347

$U_5 = -\bar{H}_5 \cdot -\bar{U} = (47)(185/(185^2 + 175^2)^{1/2}) + (50)(175/(185^2 + 175^2)^{1/2})$

$= 68.5.$

Let G_5 = effective gradient of maintenance effectiveness

$$= \frac{\text{maintenance effectiveness}}{U_5}$$

$$= \frac{78,355}{68.5} = 1144$$

Similarly, the effective gradients for the other four highway segments were computed.

By using the ranked effective gradients, a choice of highway segments to be dropped can be made. The segments dropped are shown in Table 3. It can be seen that after dropping highway segments H_3, H_2, and H_1, 11 percent of the budget and 10 percent of the materials are not used. The overall result is that only segments H_4 and H_5 can be reconstructed and represent the optimal solution.

The problem of determining optimum maintenance strategies grows rapidly when additional strategies, resources, and distress considerations are added. The RAMS program treats this kind of problem.

Program Steps

The RAMS program considers the following steps in obtaining optimal maintenance solutions:

1. Finds the feasible maintenance strategies for each highway seg- according to the minimum rating for each distress constraint (Equation 7 and Table 10) and the overall pavement rating constraint (Equation 8 and Table 10).
2. Ranks the feasible strategies for each highway segment according to the ratio of maintenance effectiveness to resource requirement.

Table 3. Selection of Highway Segments by Dropping Least Effective

	Budget Resource	Material Resource
Initial Excess Resource Requirements	185	175
Subtract H_3 (76, 70)	109	105
Subtract H_2 (46, 45)	63	60
Subtract H_1 (74, 70)	-11	-10

The ranking criterion is computed as follows:

$$r_{ij} = \frac{M_{ij}}{\sum_{l=1}^{m} a_{ijl}}$$

where:

r_{ij} = ranking ratio for highway segment i and strategy j.

M_{ij} = maintenance effectiveness if strategy j is applied to highway segment i.

a_{ijl} = percent of l^{th} type of resource needed if strategy j is applied to highway segment i.

For each highway segment the feasible strategies are ranked according to the highest value of the ranking ratio.

3. Selects the best ranked feasible strategy for each highway segment and calculates the effective gradient.

4. Sorts the effective gradients for all highway segments.

5. Selects the highway segment with the smallest effective gradient and exchanges its currently considered strategy with the next best available. This highway segment with its exchanged strategy and the remaining highway segments with their current strategies are used to recalculate the effective gradients for all highway segments. The program then switches back to Step 4 unless all the available, feasible strategies for this highway segment are exhausted in which case the program goes to Step 6.

6. One of two possible decisions are made at this step. These two decisions are:

(a) If any of the constraints are exceeded, drop the highway segment from the solution and subtract the resources required for the segment from the excess resource vector. The effective gradients for the remaining highway segments with their current strategies are recalculated and the program then returns to Step 4.

(b) If all of the constraints are satisfied, there is no need to drop more highway segments. The program goes to Step 7.

7. The remaining highway segments together with their corresponding strategies constitute the optimal solution set. If additional or "slack" capacity is available in the resource constraints then additional highway segments may be added back to eliminate or reduce this capacity.

EXAMPLE PROBLEM USING THE RAMS PROGRAM

The purpose of this larger example problem was to compare the maintenance strategies that were selected by TSDHPT personnel and with those selected by the RAMS program. The problem was prepared using actual field data which was obtained from fifteen highway segments located in TSDHPT District 17. This district is located in eastern-cental Texas.

Eleven of the fifteen highway segments selected were scheduled for various kinds of contracted highway maintenance or rehabilitation within the next several months. The highway department has actually scheduled these segments for either a seal coat, asphalt concrete overlay, or reconstruction. Four additional highway segments were added to the initial eleven because they were considered to be in excellent condition and as such to require no significant maintenance. Although the intent of the methodology contained in RAMS was not to optimize maintenance on segments which require none, it was felt that adding the four segments would demonstrate that the program could distinguish a segment that needed rehabilitation from one that does not.

The following outline will be used in describing this example problem:

1. A description of the highway segments used.
2. Pavement condition determination for each segment.
3. The gain-of-rating matrices used.
4. The pavement survivor matrices used and how these matrices were obtained.
5. Resource information with emphasis on the budget.
6. A comparison of the TSDHPT selected maintenance strategies and those selected by the RAMS program.

Description of Highway Segments

Table 4 contains general information for each highway segment used. It includes a general description of each segment and the TSDHPT scheduled maintenance strategies. Additionally, the average Serviceability Index (SI) for each segment is shown and was obtained by use of the Mays Ride Meter. As can be seen in the table, a mixture of US, State and Farm-to-Market highways were used. The pavement length and width for each highway were direct inputs into the computer program.

Pavement Condition For Each Highway Segment

The pavement condition rating system used is the one currently being implemented in Texas (6, 7) with slight modifications. This system is based on evaluating the quantity and severity of nine different distress manifestations. Due to reasons which will be explained later, only five distress types were used in this example problem.

Each distress type is assigned a certain amount of "points" up to a maximum amount. The "points" determine the current pavement rating of

Table 4. General Description of Highway Segments Used In Example Problem

Segment Number	Highway	County	Segment Length mi (km)	Segment Width ft (m)	Avg. SI	TSDHPT Scheduled Maintenance
1	US 79	Milam	4.525 (7.282)	26 (7.9)	2.7	2.5 cm HMAC Overlay + Extensive Patching
2	US 77	Milam	12.316 (19.821)	28 (8.5)	2.5	2.5 cm HMAC Overlay
3	US 190	Milam	3.617 (5.821)	26 (7.9)	2.1	3.8 cm HMAC Level-up Overlay
4	SH OSR	Madison	7.000 (11.265)	20 (6.1)	2.3	Seal Coat
5	SH OSR	Madison	2.257 (3.632)	22 (6.7)	1.9	Seal Coat
6	FM 1696	Walker	13.804 (22.215)	20 (6.1)	1.9	Seal Coat
7	FM 1791	Walker	12.374 (19.914)	22 (6.7)	0.8	Seal Coat
8	FM 2821	Walker	3.337 (5.370)	24 (7.3)	2.1	Seal Coat
9	SH 30	Walker	7.385 (11.885)	26 (7.9)	3.4	Seal Coat
10	SH 36	Burleson	12.021 (19.346)	26 (7.9)	3.9	None
11	US 290	Washington	9.019 (14.515)	26 (7.9)	3.9	None
12	US 79	Milam	5.644 (9.083)	26 (7.9)	4.5	None
13	SH 36	Burleson	9.321 (15.001)	26 (7.9)	4.7	None
14	SH OSR	Brazos	6.667 (10.729)	20 (6.1)	0.9	Recondition Base and Surfacing
15	FM 908	Milam	7.444 (11.980)	20 (6.1)	1.5	Recondition Base and Surfacing

Table 5. Current Pavement Condition Rating Information For Highway Segments

Distress Type	Highway Segment Number															Maximum Points Available
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	
Rutting	10	10	10	10	10	10	8	10	15	15	15	15	13	8	10	15
Alligator Cracking	5	15	10	20	25	25	0	15	25	25	25	25	25	5	10	25
Longitudinal Cracking	20	25	15	20	25	25	10	25	5	25	25	25	25	0	10	25
Transverse Cracking	17	20	13	20	20	20	20	20	5	20	17	17	20	17	8	20
Failures/Mile	20	40	40	40	40	40	10	20	40	40	40	40	40	20	20	40
Total Points (Overall Rating)	72	110	88	110	120	120	48	90	90	125	122	122	123	50	58	125
Percent of Total	58	88	70	88	96	96	38	72	72	100	98	98	98	40	46	100

highway segment i and distress type k. The more points assigned to a certain highway segment and distress type, the less distress is present. The summation of available points for the individual distress types for a given highway segment will determine the overall rating. Table 5 shows the current condition rating information which was used as input to the computer program. Note that the maximum overall rating score taken over the five distress types is 125, not 100 as used in many other rating systems (8). The "Percent of Total" is taken as the ratio of the overall rating to the maximum rating and is equivalent to a pavement score based on a 0 to 100 scale.

Gain-of-Raing Matrix

The gain-of-rating matrix represents the d_{ijk} input for the RAMS program. The gain-of-rating "points" are the same kind of points as used in determining the pavement condition for the highway segments.

There are three kinds of ratings (points) which are used to generate the gain-of-rating matrix. These are: Maximum points available (Table 5) for a given type of distress, Maximum gain-of-rating points (Table 6) for a given maintenance strategy and distress type and current pavement rating (Table 5) for a given highway segment and distress type. The maximum points available for a distress type indicates what magnitude of points constitute a perfect rating (no distress condition). The maximum gain-of-rating points indicate the maximum gain which can be expected by using a given kind of maintenance strategy to treat a specific distress. Current pavement rating was previously discussed.

The three ratings are used by the RAMS program to generate the gain-of-rating points (d_{ijk}) for each highway segment (i), maintenance strategy (j) and distress type (k) by one of two possible procedures. If the

Table 6. Maximum Gain-of-Rating Matrix for All Highway Segments

Maintenance Strategy	Distress Type				
	Rutting	Alligator Cracking	Longitudinal Cracking	Transverse Cracking	Failures Mile
Seal Coat	0	15	15	15	10
Thin Overlay (3.8cm or less)	13	20	20	20	25
Moderate Overlay (>3.8 to 7.6cm)	15	25	25	20	30
Thick Overlay (>7.6cm)	15	25	25	20	35
Reconstruction (Light-Duty)	15	25	25	20	40
Reconstruction (Heavy-Duty)	15	25	25	20	40

maximum gain-of-rating and the current pavement rating points sum to less than the maximum points available for a given highway segment and distress type, then the maximum gain-of-rating points is used as the d_{ijk} input. If the above sum of points is greater than the maximum points available, then the difference between the maximum points available and current pavement rating points is used as the d_{ijk} input. For example, if a moderate overlay, thick overlay, or reconstruction maintenance strategy is used, the maximum gain-of-rating points for rutting is 15. This indicates for a highway segment with a rutting distress rating of 0 (which is the severest rutting condition), application of one of these three strategies would completely eliminate the distress manifestation immediately after the required work was performed. Some maintenance strategies may have negative gain-of-rating points for some types of distress indicating that they have accentuated the distress.

The six maintenance strategies used in this example problem are considered to be typical of the maintenance performed on TSDHPT District 17 pavements. The only maintenance strategies which require additional description are light-duty and heavy-duty reconstruction. Light-duty reconstruction is generally used on low traffic highways and consists of scarifying the existing surface and base, recompacting, and then applying of a one course surface treatment. Heavy-duty reconstruction is generally used on higher traffic highways and consists of scarifying the existing surface and base, adding additional flexible base (unstabilized), recompacting and applying a thin (3.8 cm or less) asphalt concrete surface.

The maximum gain-of-rating points associated with each maintenance strategy and distress type were obtained from subjective ratings by TTI personnel and are expected to change slightly as TSDHPT personnel begin to use the computer program.

Pavement Survivor Matrices

Pavement survivor matrices were developed for each distress type and maintenance strategy combination. An example of this is Table 7 which shows the probability of survival for the six maintenance strategies obtained for transverse cracking conditions. The determination of the probabilities for each of the five distress types used in this example problem will be described in detail below. The maintenance strategies considered are: (1) seal coat, (2) thin overlay, (3) moderate overlay, (4) thick overlay, (5) reconstruction (light-duty), (6) reconstruction (heavy-duty).

To determine the probability of survival for a given maintenance strategy, failure must first be defined. Sivazlian and Stanfel (9) define it as ". . .. an event associated with a shift in the operating characteristics of a system from its permissible limits". Thus pavement failure may be when the Serviceability Index for a given highway type reaches or goes below a preselected lower limit. Failure could also be defined as when the highway develops a certain amount of a particular distress manifestation. But, for this problem, the time to failure for a given maintenance strategy will be taken as that time when some type of maintenance strategy must be accomplished which supersedes the previously applied maintenance.

The pavement survival matrices are currently based on subjective "failure analysis" data obtained from TSDHPT district maintenance management personnel. This data was obtained from a diagnostic examination of pavement segments located in four separate areas within the state. The district personnel evaluated these highway segments for future maintenance

Table 7. Pavement Survival Matrix For Transverse Cracking

Maintenance Strategy	Time After Maintenance (Yrs)									
	1	2	3	4	5	6	7	8	9	10
Seal Coat	1.00	0.92	0.86	0.85	0.67	0.38	0.33	0.18	0.09	0.06
Thin Overlay (3.8 cm or less)	1.00	1.00	0.94	0.94	0.43	0.18	0.18	0.14	0.06	0.01
Moderate Overlay (>3.8 to 7.6 cm)	1.00	1.00	1.00	1.00	1.00	0.63	0.26	0.22	0.11	0.04
Thick Overlay (>7.6 cm)	1.00	1.00	1.00	1.00	1.00	0.33	0.33	0.28	0.17	0.17
Reconstruction (Light-Duty)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.65	0.60
Reconstruction (Heavy-Duty)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.65	0.60

and rehabilitation needs based on their visual observations of the pavement and objectively measured data which was provided to them. This data included traffic, skid, deflection, ride, and construction histories.

From such information, time to failure was calculated for each maintenance strategy considered. For seal coats, a time to failure is determined when any of the six maintenance strategies considered were rescheduled for application. For the three overlays and reconstruction, a time to failure is determined only when one of these five maintenance strategies are rescheduled for application i.e., seal coats were not considered as superseding any of these five.

The time to failure data obtained for each maintenance strategy was arranged into histograms. These histograms approximate the failure density distribution curve discussed in reliability theory (9, 10). Failure density distributions are similar to normal distributions of data in that the area under the curve is equal to one.

From these histograms or failure density distributions, the failure density function can be defined by $f(x)$ taken over $0 < x < \infty$ where x defines a time scale. The probability that a maintenance strategy will fail within a time interval $(x,x + dx)$ is given by $f(x)dx$.

The corresponding cumulative density function can be defined by $F(x)$ also taken over the interval $0 < x < \infty$ and is the probability that a given maintenance strategy will fail on or before some time t. This can be expressed as follows:

$$\text{Probability of failure on or before } t = F(t) = \int_0^t f(x)dx$$

The above expression assumes that a maintenance strategy will survive past time t is given by $R(t)$ and is expressed as:

$$R(t) = 1-F(t) = \int_{t}^{\infty} f(x)dx$$

This expression can be adequately approximated for a given maintenance strategy by a cumulative frequency distribution which may be plotted from a histogram of time to failure data. The result is a survival curve, a generalized form of which is shown as Figure 3. Data from such curves are entered into the RAMS program in matrix form as is demonstrated by the use of Table 7.

The pavement survival matrices currently being used will be updated in the near future. This will be accomplished by combining the subjectively obtained data just described with objective data from a pavement data base assembled for Texas pavements. It is planned to use Bayesian techniques to accomplish this task.

Budget Resource

There are four types of resource constraints used in the program: (1) material and supply, (2) equipment, (3) manpower, and (4) cost. Each resource constraint has two major inputs: requirements and availability. The requirement input indicates how much of a given resource will be used by a maintenance strategy and availability indicates how much of a given resource is available to be used. Of the four types of resource constraints, budget is the most significant in this example problem.

The available budget used as input was essentially the same amount as the contract funds allocated for the TSDHPT selected maintenance strategies. This is an important constraint because it forced the computer program to consider maintenance decisions within approximately the same financial framework used by TSDHPT personnel. This value is shown in Table 8 as the

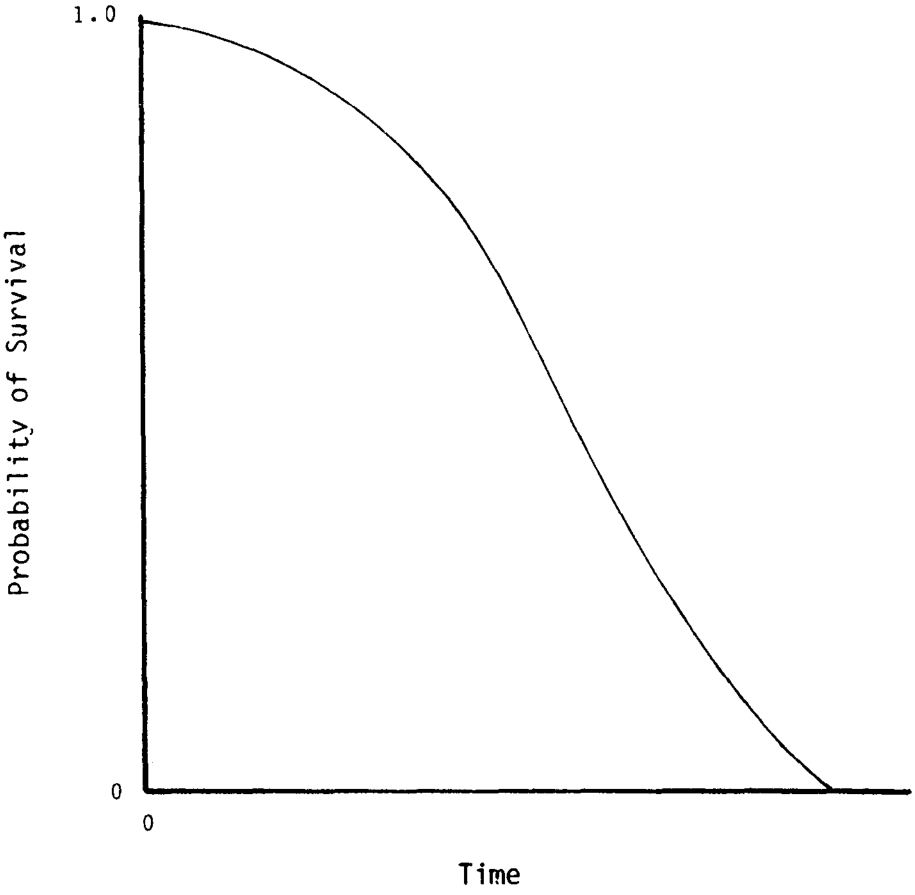

Figure 3. Generalized Form of a Survival Curve For A Maintenance Strategy

total available funds.

The budget requirement matrix indicates the required cost per unit area for each maintenance strategy and is shown in Table 8. The costs generally increase as the maintenance strategies become more extensive. The notable exceptions to this are the two kinds of reconstruction.

Comparison of TSDHPT and RAMS Selected Maintenance Strategies

Comparisons of the TSDHPT and RAMS selected maintenance strategies for the fifteen highway segments in the example are shown in Table 9. First, the TSDHPT and RAMS (Case 1) selected strategies are shown and both use the same original TSDHPT budget amount. Another RAMS solution (Case 2) is also shown and was obtained by increasing the TSDHPT budget by approximately six percent. To facilitate discussion of the comparisons, those highway segments which reveal little or no difference between the TSDHPT and RAMS (Cases 1 and 2) selected maintenance strategies will not be examined.

A combination of highway types were used in this example and the RAMS program treated all with equal priority except in applying the two kinds of reconstruction. For low traffic segments (Segment Numbers 4, 5, 6, 7, 8, 14 and 15), the program was restricted to applying only the light-duty type of reconstruction (if required) and for the remaining higher traffic segments only the heavy-duty type of reconstruction could be used. Traffic and climate indices can also be used as input to account for differences in highway types. Additionally, groupings of similar highway types can be assembled and processed together if desired.

Table 9 shows that the selected strategies for Segment Number 2 differ. The TSDHPT selected strategy is a thin overlay and the RAMS program (Cases 1 and 2) selected a seal coat. The pavement distress manifestations for this segment are comprised of alligator cracking and extensive flushing

Table 8. Cost Requirements Per Unit Area for Each Maintenance Strategy and Total Available Funds

Maintenance Strategy	Cost Per Unit Area	
	$/ft-mi	($/m-km)
Seal Coat	214	(436)
Thin Overlay	925	(1886)
Moderate Overlay	2000	(4078)
Thick Overlay	3549	(7234)
Reconstruction (Light-Duty)	944	(1925)
Reconstruction (Heavy-Duty)	2600	(5301)

Total Available Funds = $ 1,130,000

Table 9. Comparison of TSDHPT and RAMS Selected Maintenance Strategies

Segment Number	Highway	*Overall Pavement Rating	Percent of Total	**Serviceability Index (SI)	TSDHPT Selected Maintenance Strategies	RAMS Computer Program Selected Maintenance Strategies Using TSDHPT Budget (Case 1)	RAMS Computer Program Selected Maintenance Strategies Using TSDH Budget + 6.3% (Case 2)
1	US 79	72	58	2.7	2.5cm HMAC Overlay +Extensive Patching	Moderate HMAC Overlay	Moderate HMAC Overlay
2	US 77	110	88	2.5	2.5cm HMAC Overlay	Seal Coat	Seal Coat
3	US 190	88	70	2.1	3.8cm HMAC Level-up Overlay	Thin HMAC Overlay	Thin HMAC Overlay
4	SH OSR	110	88	2.3	Seal Coat	Seal Coat	Seal Coat
5	SH OSR	120	96	1.9	Seal Coat	None	None
6	FM 1696	120	96	1.9	Seal Coat	None	None
7	FM 1791	48	38	0.8	Seal Coat	Light Duty Reconstruction	Light Duty Reconstruction
8	FM 2821	90	72	2.1	Seal Coat	Thin HMAC Overlay	Thin HMAC Overlay
9	SH 30	90	72	3.4	Seal Coat	None	Thin HMAC Overlay
10	SH 36	125	100	3.9	None	None	None
11	US 290	122	98	3.9	None	(Seal Coat)	None
12	US 79	122	98	4.5	None	(Seal Coat)	None
13	SH 36	123	98	4.7	None	None	None
14	SH OSR	50	40	0.9	Recondition Base and Surfacing	Light Duty Reconstruction	Light Duty Reconstruction
15	FM 908	58	46	1.5	Recondition Base and Surfacing	Light Duty Reconstruction	Light Duty Reconstruction

*Perfect OPR = 125
Poorest OPR = 0

**Smoothest SI = 5.0
Roughest SI = 0.0

Budget Used = 100%
($1,130,000)

Budget Used = 97.8%
($1,105,140)

Budget Used = 106.3%
($1,201,520)

(flushing is not considered in the RAMS program). All maintenance strategies are feasible as determined by the minimum and overall rating constraints, the results of which are shown in Table 10, thus allowing the RAMS program to evaluate the appropriateness of five maintenance strategies (seal coat, thin overlay, moderate overlay, thick overlay and heavy-duty reconstruction). For this segment the maintenance effectiveness computed for a seal coat is about one-half that calculated for a thin overlay but the cost for a thin overlay is four times as great. It can be seen in a subjective way that a seal coat is an attractive maintenance strategy. The TSDHPT decision to use a thin overlay may have been additionally based on the rough ride and flushing present on this highway.

Segment Numbers 5 and 6 were scheduled for seal coats by the TSDHPT and no strategies were scheduled by the RAMS program. An examination of the Table 5 shows that no distress manifestations, with the exception of minor rutting, were present on these pavements. But, in fact, flushing was present (not shown in Table 5) and may have been a consideration in the TSDHPT decision.

Segment Number 7, which has numerous and extensive distress manifestations, is scheduled for a seal coat by the TSDHPT and a ligh-duty reconstruction strategy by RAMS. The feasible strategies allowed by the minimum and overall rating constraints shown in Table 10 indicate that only a thick overlay strategy or greater is allowable. A similar situation occurs with Segment Number 8.

For Segment Number 9, the TSDHPT scheduled a seal coat but the RAMS program (Case 1) scheduled no maintenance. This occurred because there was not enough budget to allow application of a thin overlay or greater to this segment. The inexpensive seal coat alternative was

Table 10. Feasible Maintenance Strategies Allowable by the Minimum Distress Rating and Overall Rating Constraints

Highway Segment	Maintenance Strategy: Feasible = 1 Infeasible = 0					
	Seal Coat	Thin Overlay	Moderate Overlay	Thick Overlay	Reconstruction (Light Duty)	Reconstruction (Heavy Duty)
1	0	0	1	1	1	1
2	1	1	1	1	1	1
3	0	1	1	1	1	1
4	1	1	1	1	1	1
5	1	1	1	1	1	1
6	1	1	1	1	1	1
7	0	0	0	1	1	1
8	0	1	1	1	1	1
9	0	1	1	1	1	1
10	1	1	1	1	1	1
11	1	1	1	1	1	1
12	1	1	1	1	1	1
13	1	1	1	1	1	1
14	0	0	0	0	1	1
15	0	0	1	1	1	1

eliminated by the minimum and overall rating constraints. For the RAMS (Case 2) selection, the original TSDHPT budget was increased by approximately six percent. This small budget change allowed the segment to be scheduled for a suitable, cost effective maintenance strategy (thin overlay).

As shown by use of Segment Number 9, the RAMS program can also be used to help estimate required maintenance budgets. This can be accomplished by inputting all data as previously discussed but varying the budget amount. The budget could be selected where adequate maintenance is scheduled for all necessary segments.

Segment Numbers 11 and 12 are in excellent condition with both having only minor transverse cracking. The RAMS program in Case 1 scheduled seal coats for these segments since some benefit could be obtained by using this strategy. This occurred because the program maximizes the maintenance effectiveness for the amount of budget available. In Case 2, the funds were more adequately used by slightly increasing the available budget with one result being that these two seal coats were eliminated.

A comparison of overall maintenance effectiveness resulting from the TSDHPT, RAMS Case 1 and Case 2 maintenance strategy selections provides an indication of the optimality of the computer solutions. The maintenance effectiveness obtained by use of Equation 1 for the three maintenance programs are:

TSDHPT:	359,412
RAMS - Case 1:	425,106
RAMS - Case 2:	451,318

Comparing the TSDHPT and RAMS Case 1 selections shows that use of the computer program increased the maintenance effectiveness by 18 percent and resulted in a two percent budget savings. But, Case 1 selections did exclude one pavement segment which needed maintenance. Case 2 selections filled this need and resulted in an increase in maintenance effectiveness of 26 percent over TSDHPT selections. The RAMS program accomplished this by using a budget approximately six percent larger than used by the TSDHPT.

SUMMARY

This paper has examined an operating computer program which uses integer programming to determine optimal maintenance strategies for pavements. The program uses the current pavement condition, potential gain-of-rating, and survivor matrices as input to maximize the overall maintenance effectiveness for any group of highway segments. The program can use numerous maintenance strategies, resources, and feasibility constraints in determining optimal solutions. The required inputs can be expanded or reduced as necessary.

An example problem with fifteen highway segments located in one highway district in Texas was used to demonstrate the program. Based on this actual field data a comparison of the computer program and TSDHPT selected maintenance strategies revealed similar selections with notable exceptions. It was shown that by using the RAMS program with the same budget the maintenance effectiveness of the selected maintenance strategies could be increased by 18 percent over TSDHPT selections. The maintenance effectiveness was increased by 26 percent with a six percent increase in the available budget. Although the example problem represented maintenance strategies planned for accomplishment by contract, the computer program also has the capability to optimize in-house district maintenance efforts.

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