U.S. Department of Transportation

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
WASHINGTON, D.C. 20590

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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Revised WRH/lg 12/8/83

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VOLUME I

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REFERENCE LIST

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LESSON OUTLINE
GENERAL AND HISTORICAL REVIEW OF PAVEMENTS

Instructional Objectives

1. To review the general and historical concepts of pavements to provide a common background of beginning for all members of the class.

2. To outline basic components and differences among flexible, rigid and composite pavement types.

Performance Objectives

1. The student should be able to outline the different components of a pavement structure.

2. The student shall understand the historical perspective of pavement design and performance.

Abbreviated Summary	Time Allocations, min.
1. Historical Background	10
2. Role of Pavements	10
3. Pavement Definitions and Terms	20
4. Types of Pavements	10
	50 minutes

Reading Assignment

1. Haas & Hudson - Chapter 1

2. Yoder & Witczak - Chapter 1, pages 1 to 10

3. Instructional Text

LESSON OUTLINE
PAVEMENT NOTES - GENERAL AND HISTORICAL

1.0 HISTORICAL BACKGROUND

The first real roads were built shortly after the discovery of the wheel, about 3500 B.C. The Romans were the first scientific road builders with the "Via Appia," or Appian Way, which was initiated in 312 B.C. The Appian Way was generally three to five feet thick and made up of three layers. The work involved hand placed stone, and this method became standard practice in the 19th century.

1.1 Pioneer Road Builders

1.1.1 Pierre Tresaguet (late 18th century). Introduced the idea that pavements should be well drained. He also recognized the need for continuous maintenance.

1.1.2 McAdam (1756-1836). McAdam is known as the father of modern pavement construction. His design was based on the principle that a drained and compacted subgrade should support the load applied to a pavement while the stone surfacing should act only as a wearing course.

1.1.3 Modern Roads. The first bituminous road was built in 1906; followed closely by the first Portland Cement Concrete pavement in 1909.

2.0 THE ROLE OF PAVEMENTS IN TODAY'S SYSTEM

Today's transport system includes marine highway rail, air and pipeline. Pavements represent approximately 50% of the total highway expenditure and this will increase as rehabilitation increases.

Of the above only marine and pipeline don't make use of a type of basic pavement structures.

2.1 Highways

The major structural elements of highways are pavements.

2.2 Air Travel

Pavements are required for runways, taxiways and parking areas in airports.

2.3 Railroads

Railroads operate on a form of pavement. In fact, now rails are often mounted on a properly designed continuous pavement.

3.0 PURPOSE OF THE PAVEMENT

To serve the applied traffic (which is often high-speed, high-volume and/or heavily loaded traffic) safely, comfortably and efficiently at minimum or at least reasonable overall cost is the purpose of pavement.

Although in the U. S. construction of new pavements will not continue at the fast pace seen since World War II, the existing investment must be protected through upgrading or remedial action.

4.0 PAVEMENT DEFINITIONS AND TERMS (VISUAL AID 1.1)

4.1 Subgrade (really subgrade material)

The natural material lying under the grade line material. Referred to variously as:

(a) subgrade material,

(b) subgrade,

(c) subgrade soil

(d) basement soil, and

(e) foundation soil.

4.2 Improved Subgrade

Improved subgrade usually involves compaction or mechanical stabilization and sometimes refers to chemical stabilization.

4.3 Subbase Material

Generally an improved or imported material is of better quality than the existing subgrade material. It is often granulous but lower quality than base material. It can be stabilized. Subgrade material is usually "pit run." It is usually well compacted, but of lower specification than base. There may be none, one or more subbases.

4.4 Base Material

The layer may be granular material such as crushed rock or gravel. It may be stabilized, and may even be a plant mix AC. It is always well compacted. Cement stabilized bases can also be mixed in a central plant stabilization: Chemical stabilization with cement or lime. Mechanical stabilization, asphalt, sulfur, and polymer. We do not really consider compaction as mechanical stabilization, but some refer to it that way.

4.5 Load (Visual Aid 1.2 and 1.3)

Whenever a design method is used, make sure what load is to be used.

4.5.1 Axle Load.

(a) Single axle — single tired 0——0
dual tired 00——00

(b) Tandem axle - need a load spreading device.

(c) Wheel load - in general is 1/2 the axle load or the half axle load - awkward for tandem.

4.5.2 Gears.

(a) Single tire 0——0

(b) Single tire with duals 00——00

(c) Nose (or tail) - generally not more than 10% of the load - make sure from the vehicle manufacturer's specs.

(d) Twin tandem 00——00
00——00

4.5.3 Load Equivalency. EWL equivalent wheel load can be based on:

(a) stress,

(b) deflection, and

(c) damage.

This concept came up during World War II. Could say equivalent loads have the same destructive effect on pavement. Best to look at damage, than stress or deflection in defining EWL.

18-k EWL generally used because it is the legal limit in many states and countries - can use any other load here.

It was derived at AASHO Road Test by studying different loads on similar pavement. The AASHO Road Test will be covered later.

5.0 TYPES OF PAVEMENTS

5.1 Pavement

Haas and Hudson (Ref 1) define a pavement as " .. the upper portion of the road, airport or parking lot structure and includes all the layers resting on the subgrade. Additionally, the pavement is considered to have a bound surface and includes the load carrying capacity of the subgrade."

5.1.1 Rigid Pavements. (Visual Aid 1.4 and 1.5)

(a) Rigid pavements include PCC pavements,

(b) Are considered to carry load in bending, and

(c) Method of analysis: slab or plate theory.

5.1.2 Flexible Pavements.

(a) Materials used in flexible pavement are asphaltic concrete or asphalt surface treatments and granular materials or base layers.

(b) They are considered to carry load in shear and compression - spread the load.

(c) Method of analysis is elastic or viscó elastic layered theory usually linear.

5.1.3 Composite Pavement. (Visual Aid 1.6)

(a) Sometimes this term is used for flexible pavements with one or more stabilized layers, usually a layer treated with portland cement.

(b) More commonly it refers to rigid pavements overlayed with asphaltic concrete.

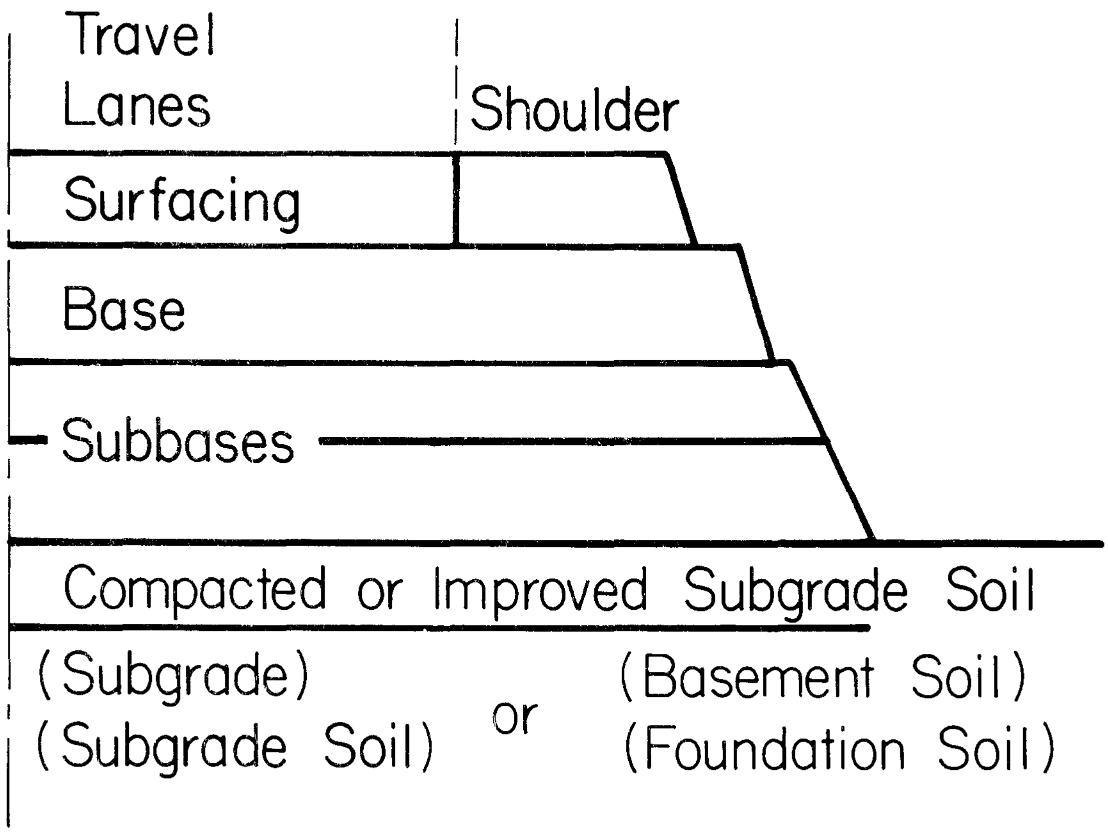
(c) The method of analysis usually requires special assumptions to use slab theory or layered theory.

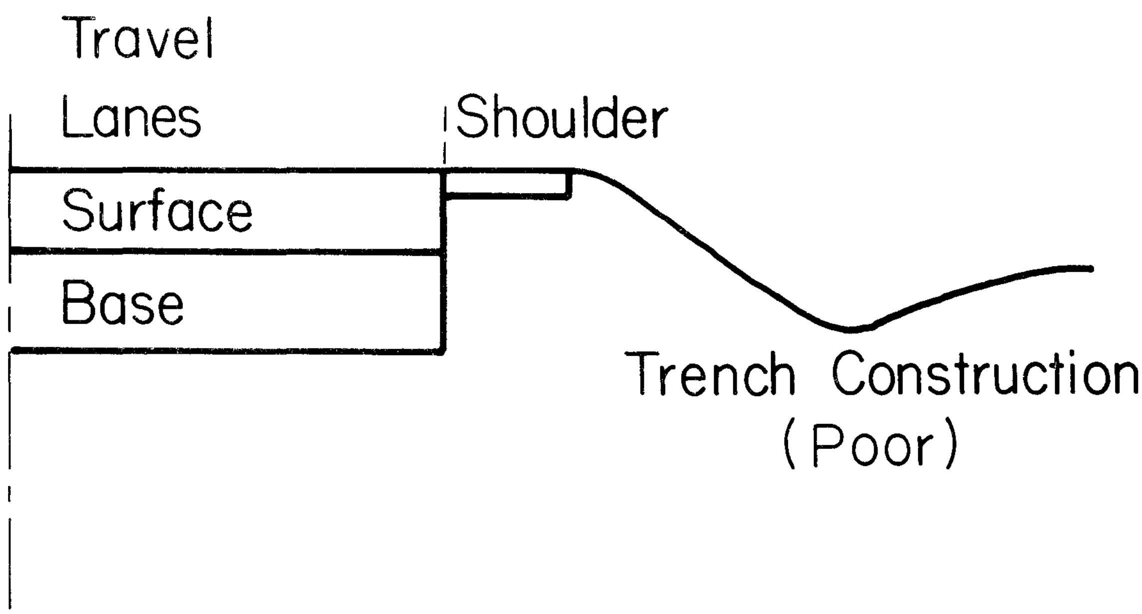
- Treat as a flexible and deal with the stabilized layer.
- Treat as a rigid pavement.

LESSON OUTLINE
GENERAL AND HISTORICAL REVIEW OF PAVEMENTS

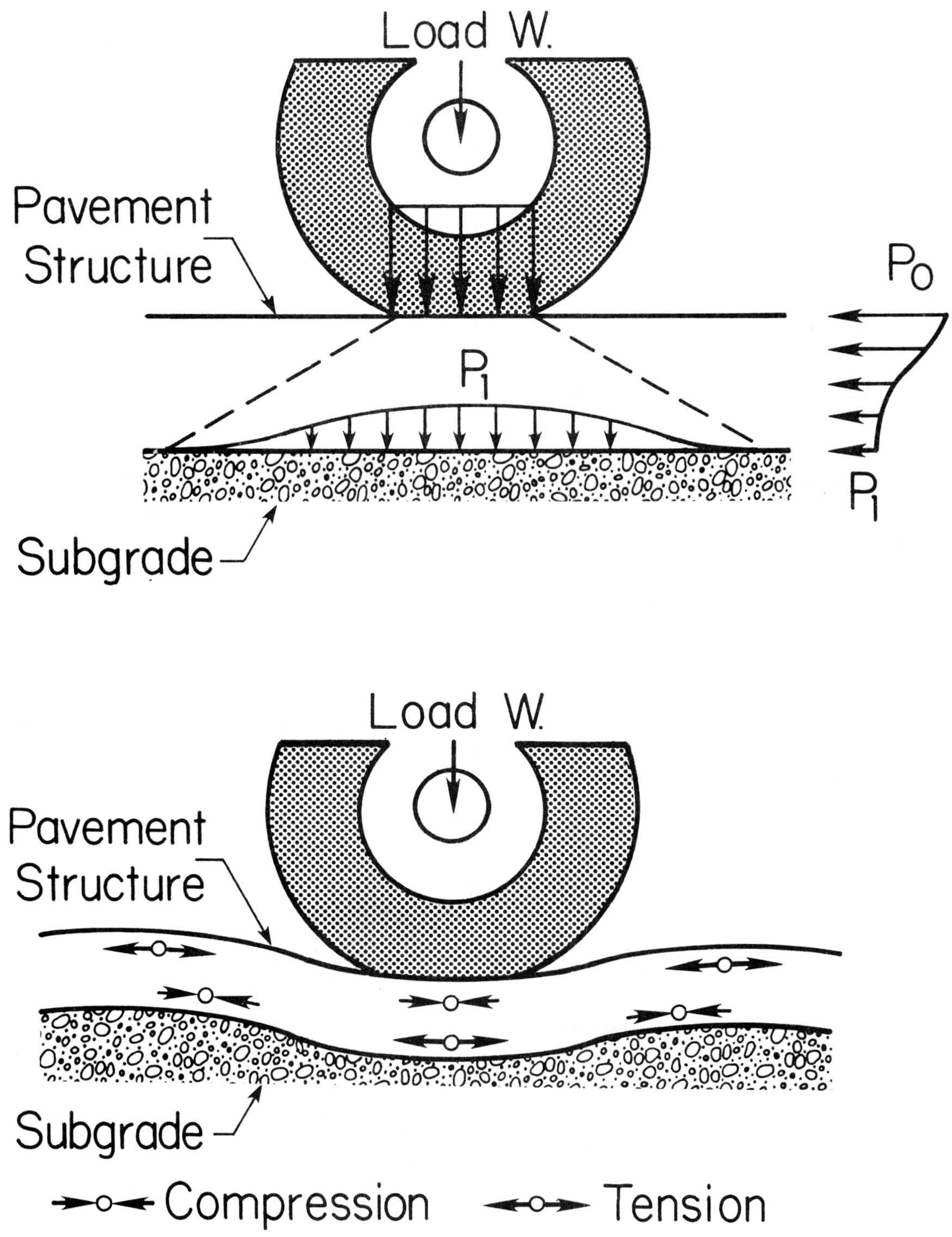
VISUAL AID	TITLE
Visual Aid 1.1.	Pavement cross section.
Visual Aid 1.2.	Pavement wheel loads.
Visual Aid 1.3.	Influence of multiple wheels on stresses.
Visual Aid 1.4.	Flexible and rigid pavement cross section.
Visual Aid 1.5.	Flexible pavement.
Visual Aid 1.6.	Composite pavement.

Visual Aid 1.1. Pavement Cross Section

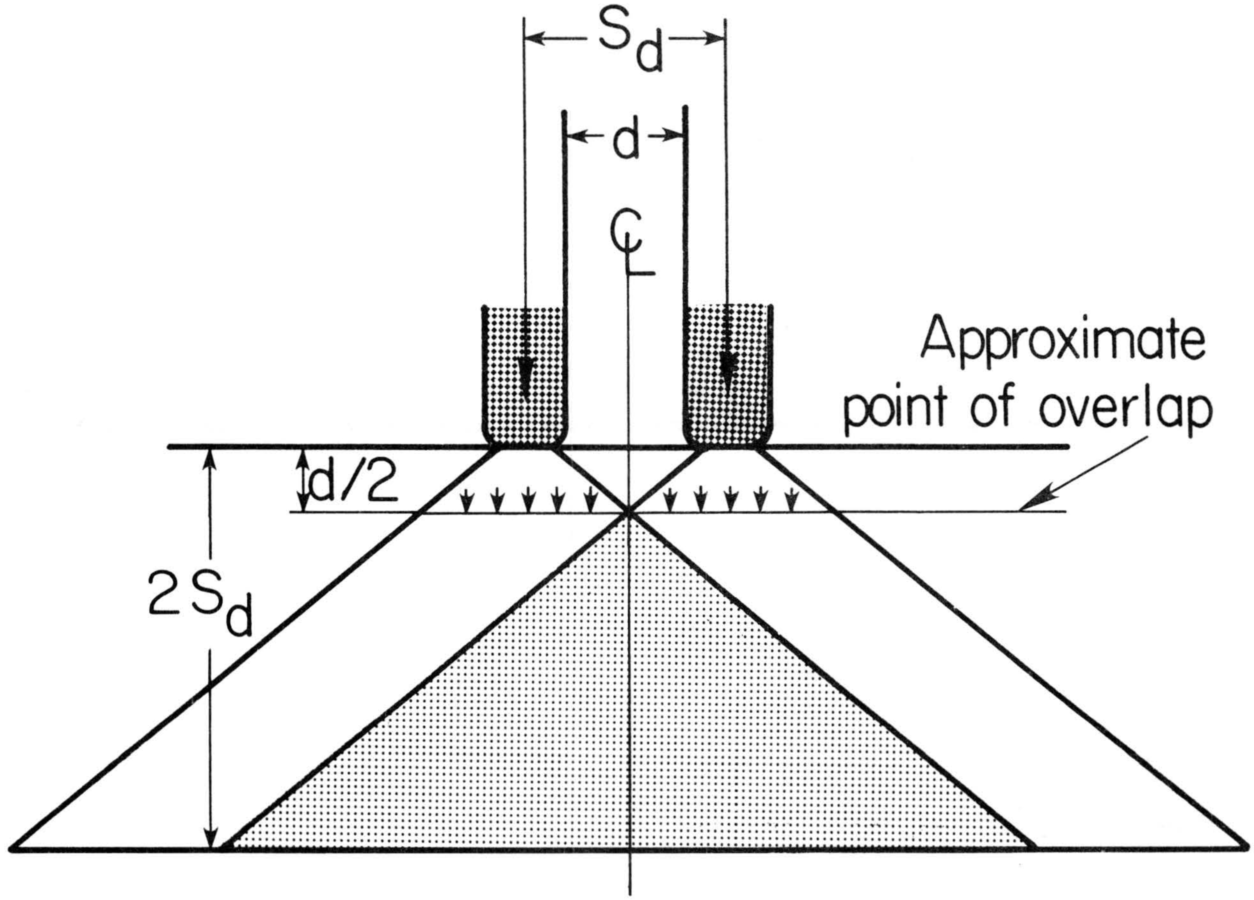




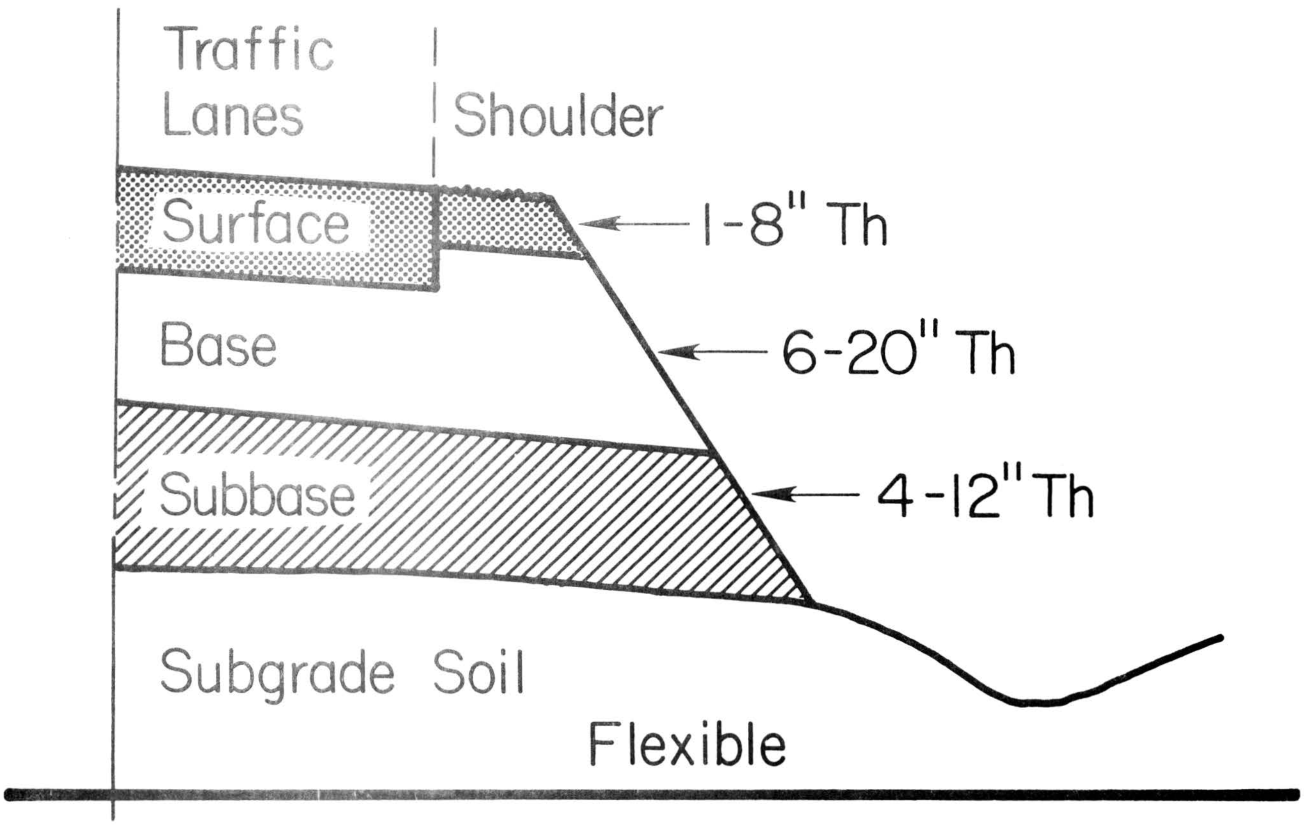
Visual Aid 1.2. Pavement wheel loads.

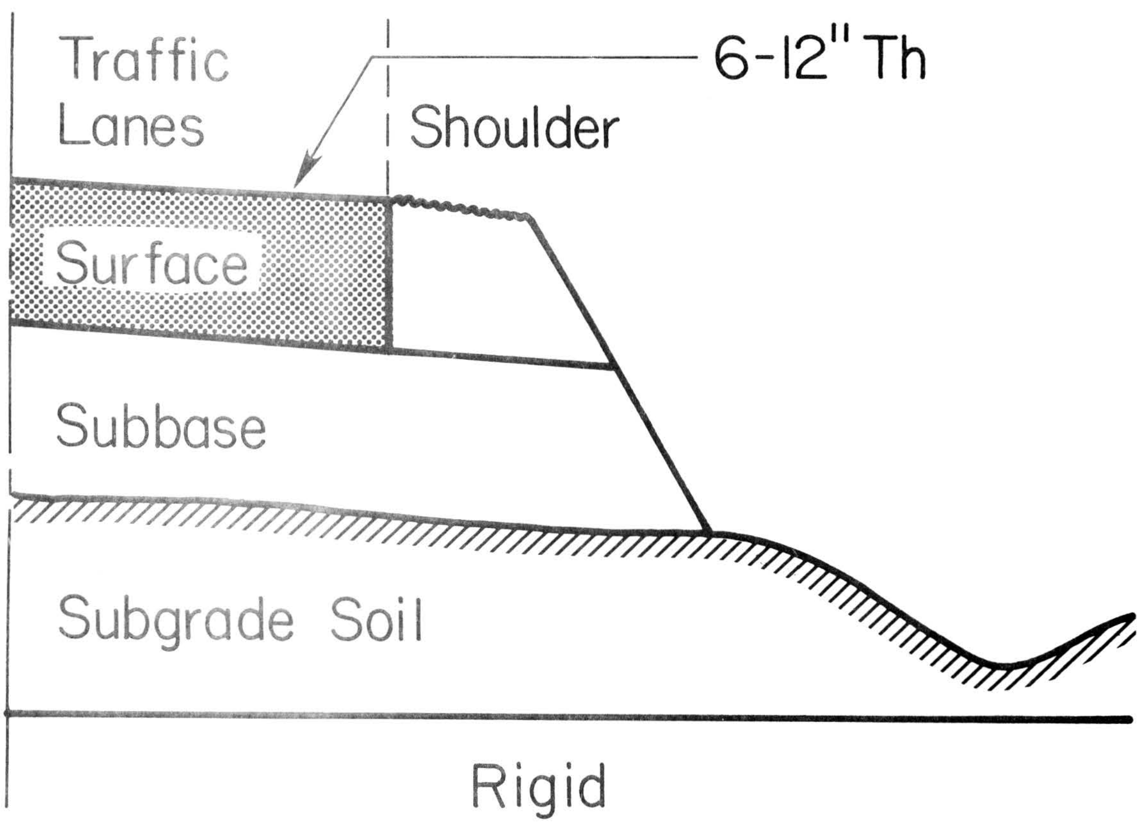


Visual Aid 1.3. Influence of multiple wheels on stresses.

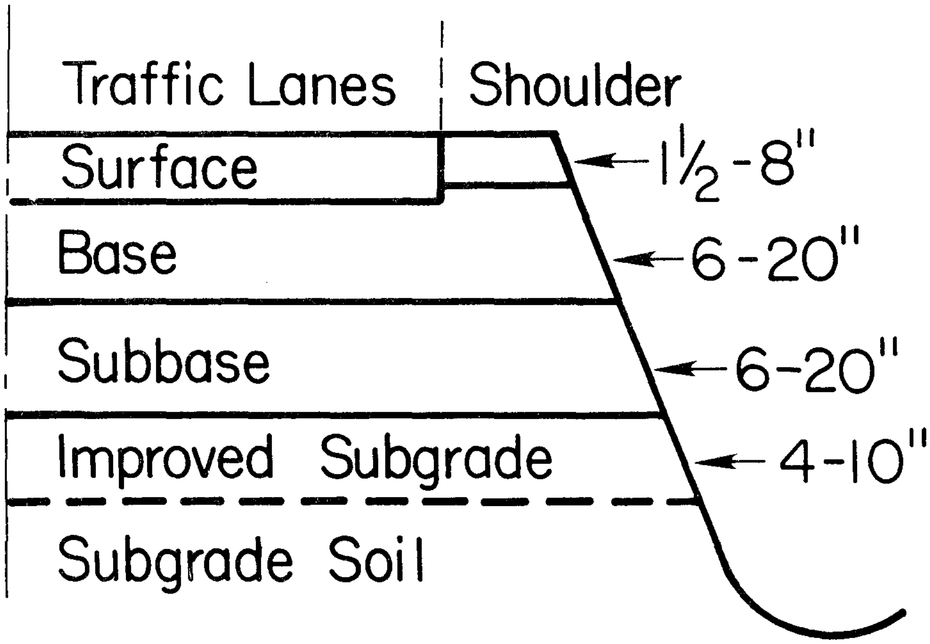


Visual Aid 1.4. Flexible and rigid pavement cross sections.

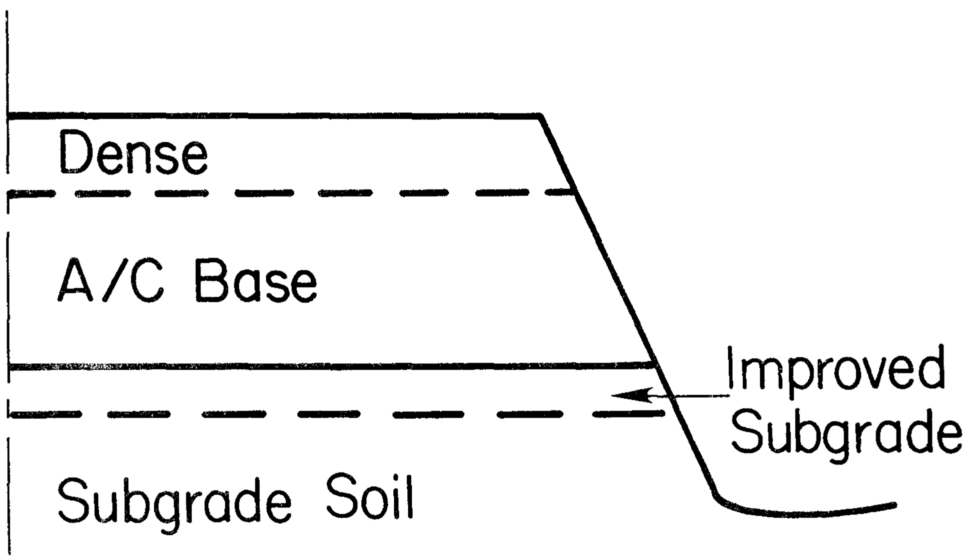




Visual Aid 1.5. Flexible pavement.



(a) Standard.



(b) Full depth.

Visual Aid 1.6. Composite pavement.

1. A PAVEMENT CONTAINING ONE OR MORE "RIGID" LAYERS

2. USUALLY HAS AN ASPHALT/CONCRETE SURFACE

3. TYPES:

   (A) OVERLAID PORTLAND CEMENT CONCRETE

   (B) CEMENT TREATED BASES

   (C) LIME FLY ASH STABILIZATION

INSTRUCTIONAL TEXT

PAVEMENT NOTES

GENERAL & HISTORICAL

REFERENCE 2

MODERATORS OPENING REMARKS

SESSION I

W. R. HUDSON

FOURTH INTERNATIONAL CONFERENCE

STRUCTURAL DESIGN

OF

ASPHALT PAVEMENTS

1977

SESSION I

Moderators W. RONALD HUDSON, Professor of Civil Engineering, The University of Texas at Austin, and International Director, Highway Cost Study, Texas Research and Development Foundation, Austin, Texas, U.S.A.

R. C. G. HAAS, Professor, Faculty of Engineering, Department of Civil Engineering, University of Waterloo, Waterloo, Ontario, Canada

MODERATORS' OPENING REMARKS BY W. R. HUDSON

Today we are considering complete design systems for asphalt pavements. What has led to these complete systems? How have we progressed?

Certainly history has played an important part. About 312 BC, Roman Engineers completed a remarkable feat of pavement design and construction, the Appian Way. They used layers of stone and mortar and knowingly or unknowingly provided the concept of spreading loads. Over 2,000 years later, some sections of these roads are still in use. This is perhaps why some people suggest we haven't learned much since that time. Is this fair, or has our past helped us to develop more complete design methods?

We might show, for example, that the Romans didn't understand drainage, as witnessed by their trench construction; they had cheap (slave) labor and so they could vastly over design; they also had much lighter loads, etc. While these Roman roads taught us something, they were no panacea.

Great individuals have made significant contributions. For example, Tresaguet, in France, presented his treatise on road construction to the Assembly of Bridges and Highways in 1777. He recognized the need for good drainage, crowning the road surface, providing good materials and subgrades, better economy by reducing the depth of stone and he basically laid the foundation for the great system of French roads later developed under Napoleon.

MacAdam in the early 19th century was another outstanding pioneer who recognized the importance of the wearing course, the stability provided by the interlocking of angular broken stones and the adequacy of well-drained earth subgrades when covered with proper surfaces.

Francis Hveem made many additional contributions, including more rational test methods such as the Hveem stabilometer.

In 1920, we see the Bates Experimental Road in Illinois providing us with some of the first basic data from a controlled experiment. The results showed that heavier loads require thicker pavements.

With this demonstrated relationship, pavement designers began to recognize the need for methods of measuring material strength, load carrying capacity of the subgrade, traffic and other parameters. As a result, in the late 1920's, we see the development of the CBR method.

Then World War II came along in 1939 and suddenly we found that CBR based pavement designs, with the relatively light traffic loads of the 1930's, were not sufficient for the new, heavy loads and aircraft. The design challenge was met by the Corps of Engineers' modification of the CBR method.

This modification or extension of an empirical method of design is a tribute to the ingenuity of engineers, but it also indicated a need for more fundamentally based methods of analysis and design in which different materials and different loads could be accounted for in a rational manner. Professor Burmister, in 1943, was one of the first to recognize this need in developing his theoretical, elastic layer analyses. His efforts plus the advent of the computer provided a basis for much of our current routine use of layer theory in design.

Following World War II, there was literally an explosion of road building. Traffic volumes, loads and speeds increased sharply and high-speed surfaces were required. Engineers saw the need for systematically obtained design data for such conditions from full-scale experiments. They responded by designing and building a number of test roads. The WASHO Test Road in Idaho (1955), for example, provided us with some basic data on pavement behavior and designs for heavy loads. In Great Britain, the Alconbury Hill experiment was constructed (1957) to provide similar design data. Then the famous AASHO Road Test was conducted and provided us with a most comprehensive data base on structural damage and load equivalencies. More important perhaps, Carey and Irick (1962) formally defined for the first time pavement deterioration and "failure" in terms of the user, through their serviceability - performance concept.

1962 also saw the First International Conference in Ann Arbor. We can recall that it was devoted to presenting basic theories for design and to examining the data from the AASHO Road Test.

The Second Conference in 1967, also in Ann Arbor, was largely concerned with "consolidating" design theory. It was felt by many present that we were in a position to more universally relate design theory, economics, etc. to performance of the structure. As stated by Bill Carey "...it does little good to develop precise equations to determine stresses in the elements of a pavement unless a means is sought simultaneously to relate stresses to pavement performance".

This is perhaps why, during those mid 1960's, that a small group of pavement engineers felt these objectives could be most effectively and comprehensively achieved through the application of systems principles. We started to use the terms pavement design and management system, but the world did not exactly beat a path to our door. In fact, the 1972 Conference in London, which was supposed to be devoted to translating theory to practice, had very few systems type papers. Some major reasons undoubtedly were the use of too much jargon, bewildering flow charts and the lack of proper communications.

Now, however, a few short years later, almost everyone uses the terms pavement design and management system, including many operating agencies. And we have as one of the principal themes of this 1977 Conference the discussion of complete systems for pavement design.

So to answer the question originally posed, we have learned a lot about pavement design, from the fundamental theories to the factors that affect pavement response and performance. We have accumulated considerable knowledge about the effects of loads, environmental factors, materials behavior, economics and so forth and we have learned to put much of this knowledge together in a systematic and efficient way. But while we may have some reasonably complete working systems, let us not delude ourselves into thinking that they are perfect. Our estimates of the various load, environmental and materials variables, and our predictions of performance, are still subject to considerable error. In addition we are faced with serious design challenges in responding to changing energy and materials problems.

So at the Eighth Conference in 1997 we might see headlines something to the effect "Professor Emeritus Carl Monismith of the University of California at Berkeley has summarized the deliberations of a meeting of world experts on pavement design by saying that the challenges posed by the new synthetic flexible pavements are being met in a comprehensive, efficient and systematic way."

What is a Complete Design System?

Trying to define the complete design system is something like trying to define the complete person. We could probably start by listing some of the key attributes that a complete person should have, such as a sense of humor, physical well being, honesty etc. But when it comes to the details, those characteristics that make a sense of humor function, that comprise physical well being, that make honesty work, we begin to encounter difficulties.

So it would be with a complete design method. We could also start, by listing some of the key features, such as being able to consider traffic, materials and environmental inputs, being able to estimate response of the structure in a rational manner, being able to reliably predict the performance of any alternative design... But again, when it comes to agreeing on the details of procedures, the models, the specific objectives, we are in difficulty. There are many different structural and performance models, and many different ways of fulfilling objectives.

Perhaps the problem is largely one of distinguishing between the general attributes of a complete design system, and the particular objectives, procedures models and so forth that apply to us as individuals or to our individual agencies. We might make better progress by firstly defining the generally complete design system and then directing our research efforts to developing new knowledge and better methods within this context.

So, let us begin by trying to define the key attributes or features of a complete design system; then, in a subsequent section, we will comment on the papers of this session and the Conference as a whole within this complete design system context. Future conferences will undoubtedly document progress towards the ultimate goal of being both complete and perfect.

The Attributes of a Complete Design System

The first brochure for this Conference contained a very simple diagram that defined the elements of a complete structural design system. Figure 1, which shows these same elements, was intended to outline the scope of the conference. It would be useful to expand the concepts underlying Figure 1, in terms of the key attributes and requirements associated with the various elements. We can then use these to compare with actual methods presented in this and other sessions.

Firstly, a complete system has a set of input information requirements

SESSION I

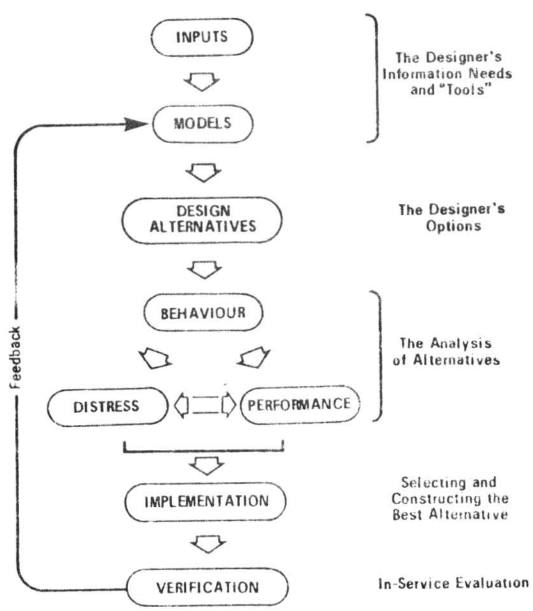


Fig. 1. Basic Elements of a Pavement Structural Design System.

and contains the necessary models for structural and economic analysis. This is what the designer starts with. He should know or specify what the design criteria and constraints are, what the costs, design period and discount rate are, what the materials, traffic, subgrade and environmental characteristics are, what sort of variability can be expected in construction and maintenance, and what the condition of the existing pavement is if a rehabilitation design is involved. Moreover, he should have available to him the necessary structural and economic analysis models to estimate the outcome of any design alternative, given this input information.

Figure 2, Part A, illustrates these key attributes or requirements for input information and models. This diagram is simply an elaboration of Figure 1, and it is intended to cover both new pavement design and rehabilitation design.

Secondly, a complete design system should be able to consider all the feasible design alternatives, shown as Part B of Figure 2. Such alternatives include the materials types and layer thicknesses and may include the expected construction and maintenance policies if they are thought to have different effects on different designs. Moreover, future rehabilitation alternatives wit' in the design period, comprise an ov rall alternative design strategy.

Thirdly, Part C of Figure 2 shows that we should be able to calculate the expected behavior or response of each design alternative to the inputs, in terms of stress, strain, deflection or deformation; then estimate the limiting behavior in terms of distress (i.e. fatigue cracking, distortion, etc.). As well, we should be able to predict the performance or serviceability age relationship for each alternative. Why should we be able to predict both behavior and performance? Perhaps the best answer is that we need the mechanistic predictions of behavior because it is cracking, distortion, disintegration, etc. that the engineer treats or corrects during the service life of the structure. However, it is performance, as related to the user, that we need for determining initial service life and rehabilitation service lives, and for working out the costs and benefits of a design alternative.

It would also be desirable to explicitly or quantitatively relate distress to performance. This need has been very actively endorsed by a number of people, but there are some who remain unconvinced. Again, perhaps one of the best answers is that the engineer takes corrective or maintenance action on distress, not on serviceability, until it reaches its minimum acceptable level. Yet, it is distress that leads to a subsequent loss of serviceability; if the engineer knew the relationship, he would be in a much better position to determine the type, amount and timing of his corrective action in order to get the maximum benefit.

Fourth, Part D of Figure 2, a complete design method would be able to apply the decision criteria which may include not only direct economic considerations but also such factors as energy implications and recyclability of the materials, and then select the best alternative for construction.

Finally, as given in Part E of Figure 2, a vital element of design is verification. Because a method may be complete does not mean it is perfect. Thus, pavement designers are faced with an equally important task of continuing verification to: a) update and improve their design models, and b) check their original design estimates. The means for such verification is usually periodic, in-service evaluation of structural capacity, distress, serviceability and safety. While the regular network of roads may provide most of the long term data, test roads have played a most important role in the verification and development of structural models.

In summary, a complete pavement design would contain certain key features

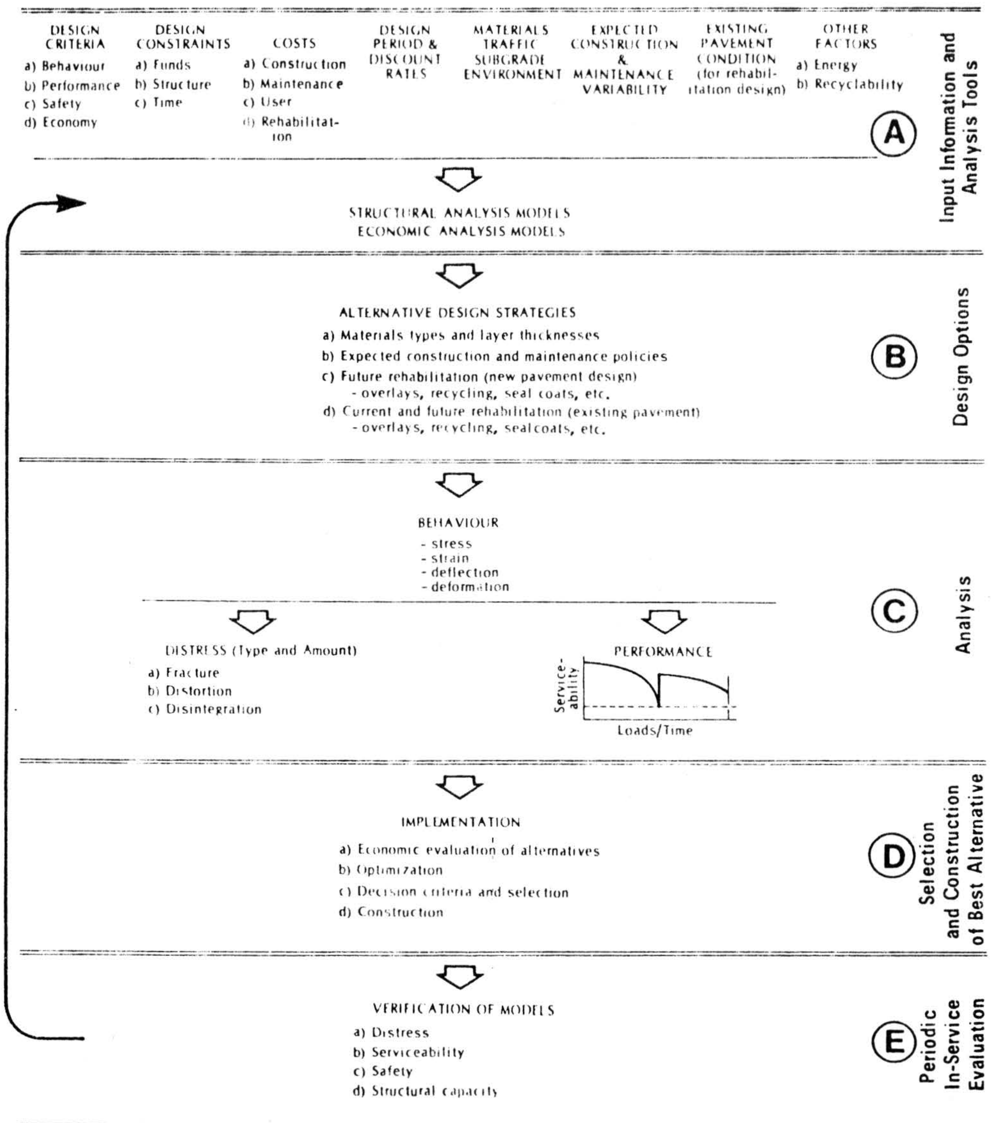


Fig. 2. Key Attributes of a Complete Pavement Design System.

and requirements as listed in Figures 1 and 2, and discussed in the preceding paragraphs. However, while these diagrams may be used to characterize "completeness", they don't tell us anything about "perfection". We all realize that pavement design technology is still amenable to considerable improvement.

LESSON OUTLINE
FHWA SLIDE - TAPE PRESENTATION "PAVEMENT MANAGEMENT"

Instructional Objectives

To introduce the pavement management concept. Familiarize the student with basic terms and aspects of a pavement management system.

Performance Objectives

1. The student should be able to answer three basic questions (what, why, how) about pavement management.

2. The student should be able to compare existing practices with those of an ideal pavement management program.

Abbreviated Summary	Time Allocations, min.
1. FHWA Slide-Tape Presentation	1 hour

Reading Assignment

1. Instructional Text

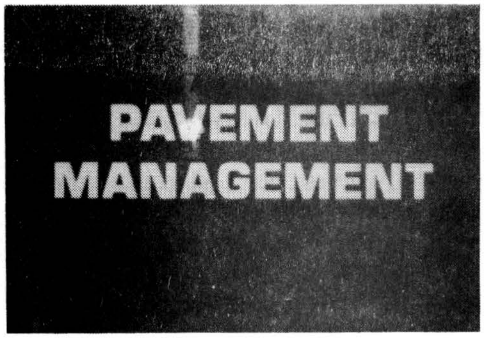
INSTRUCTIONAL TEXT

PAVEMENT MANAGEMENT

PRESENTATION BY

FHWA
(Implementation Division and
Highway Design Division)

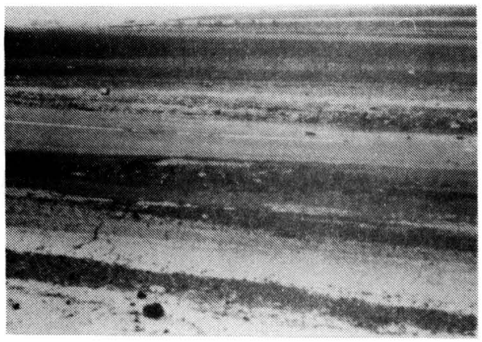
OCTOBER 1979



1.

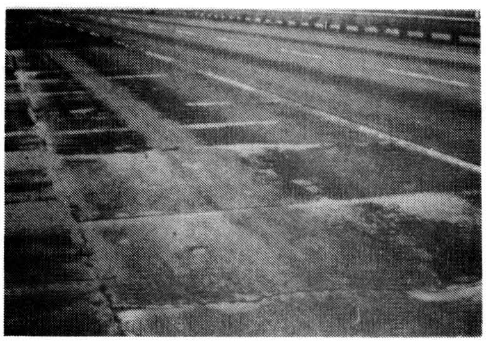
Title Slide

Note to projectionist: First pulse occurs before title slide.



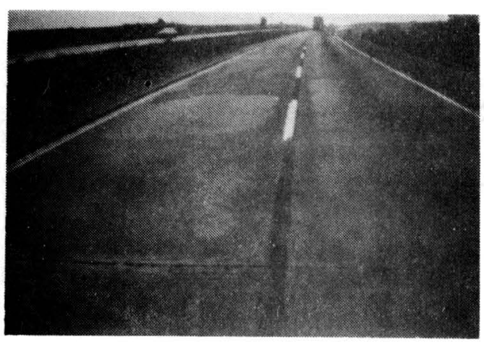
2.

Possibly the most critical problem facing highway administrators today is the deterioration of our nation's highways.



3.

During the past 20 years, we have witnessed a period of unprecedented road construction. But many of the pavements built during that period are now approaching the end of their lives and are in need of reconstruction or rehabilitation.



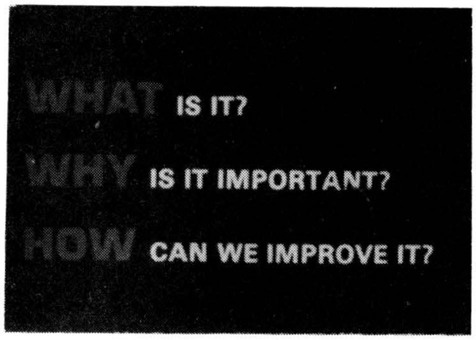
4.

Others are showing signs of serious distress much earlier - a warning sign to highway managers that something must be done - and soon.

5.

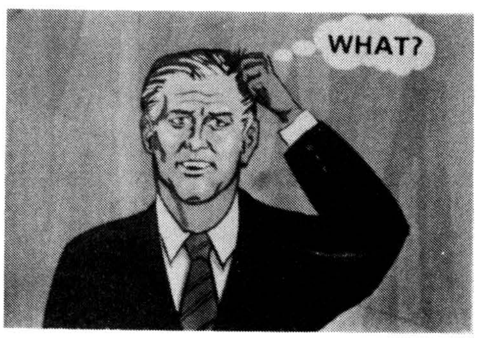
This realization has led some highway administrators to take a fresh look at the way they have programmed, designed, constructed and maintained pavements in the past. It has also led to the use of a new term - "Pavement Management."

6.



This slide presentation has been developed to answer three basic questions about pavement management. First, What is it? Second, Why is it important? And third, How can we manage pavements more effectively?

7.



First, let's look at what is meant by the term, "Pavement Management."

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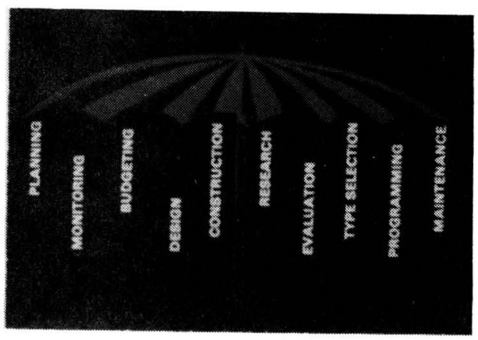
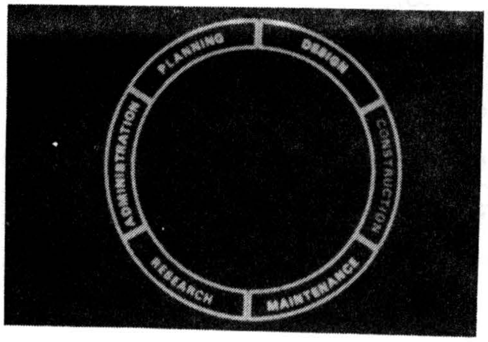
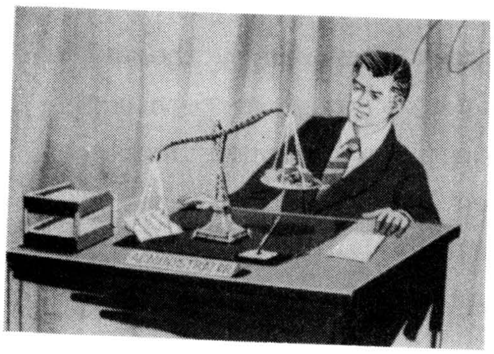


Pavement Management is an umbrella term, or a concept, which in its broadest sense encompasses many of the daily activities of every highway agency.



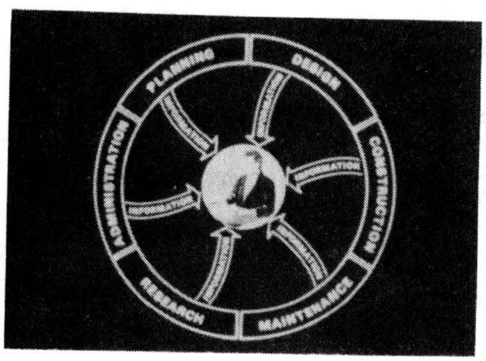
9.

For convenience, we have grouped these activities into six major categories: administration, planning, design, construction, maintenance, and research.



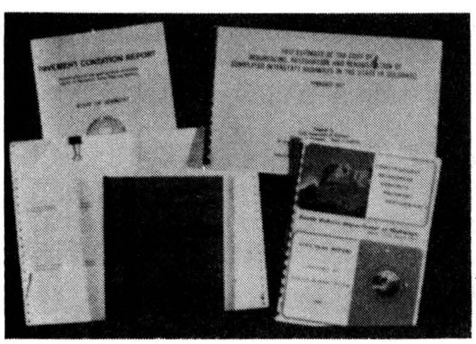
10.

Central to the pavement management picture is the highway manager, the person responsible for the execution and coordination of these activities, and for weighing the alternatives to achieve the best possible value for the available public funds.



11.

In order to manage pavements effectively, it is essential that the manager have good information upon which to base his decisions. This information takes many forms and comes from a variety of sources both within and outside of the highway agency.



12.

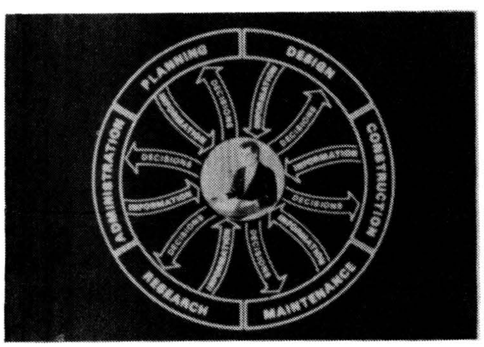
For example, information about a particular section of pavement, such as its physical characteristics; the number of loads it has sustained; its cost; and its performance over the years, is generated internally by the activities of the agency.

13.

Other types of information, such as new technological developments and economic indices, may come from sources external to the agency including the Federal Government, industry, academic institutions, and other highway organizations.

14.

Before any of this data can be used by the decisionmaker, or manager, it must be combined and analyzed. Depending upon the situation, this may be done subjectively in the manager's own head, by a few manual calculations, or by the use of various computer programs developed for that purpose.

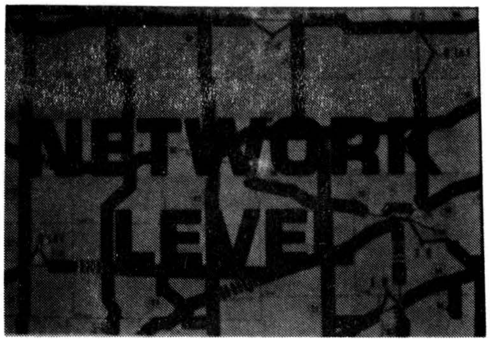


15.

The product of the analysis is, of course, a decision. Many decisions affecting pavements are made every day by highway managers throughout the organization.

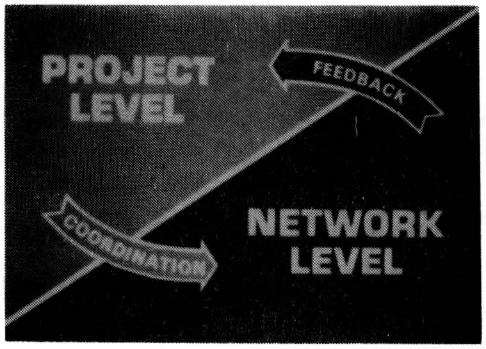
16.

The process of generating information, analyzing it, and making decisions takes place at two different levels. One is the project level, where decisions are made about specific projects or sections of pavement.



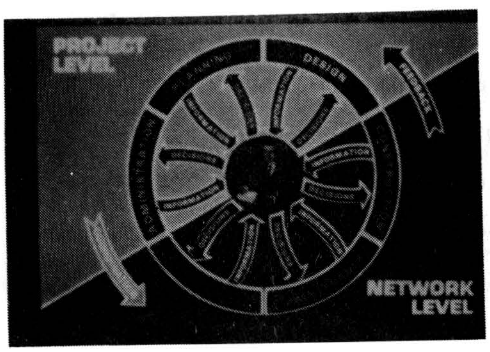
17.

The other is the network or program level, where decisions are made which affect the entire system of pavements or which involve trade offs between projects and activities.



18.

Good pavement management requires good coordination and feedback between these two levels as well as among the activities themselves.

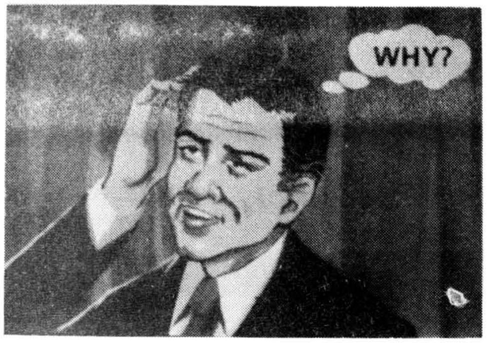


19.

When we look at all of these things together - the activities, the levels at which they take place, the data gathered, the analysis performed and the decisions that are made - we have a picture of the total pavement management process.

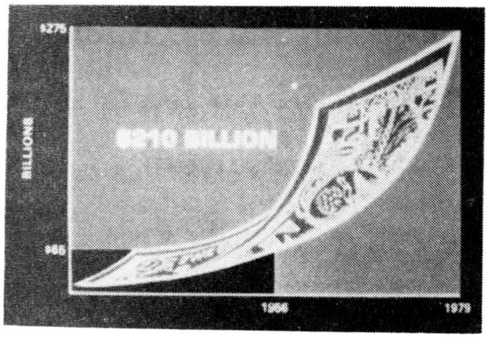
20.

It's complicated. There is no question about it. But it exists today in one form or another, in every highway agency. So when we speak of pavement management, we are not necessarily speaking about a new program. What we are speaking about is getting a better "handle" on the existing practices and making them more effective.



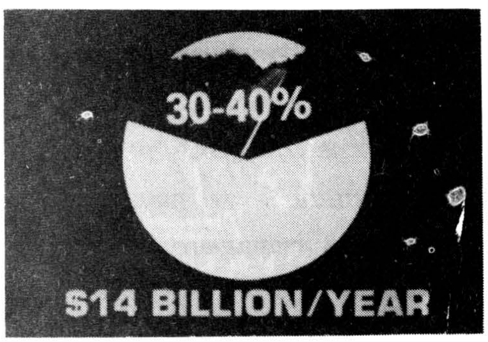
21.

This brings us to our second question - <u>Why</u> is pavement management so important today?



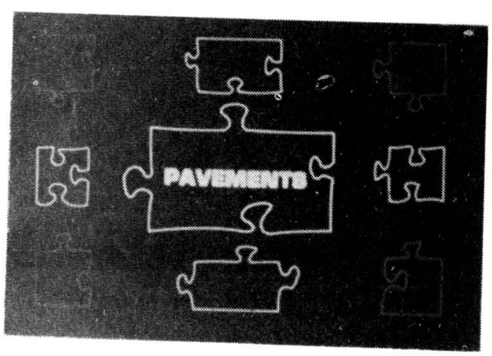
22.

The highway system in this country represents a total capital investment of about $275 billion. Over three quarters of that investment, or about $210 billion, has been made since the beginning of the interstate program in 1956.



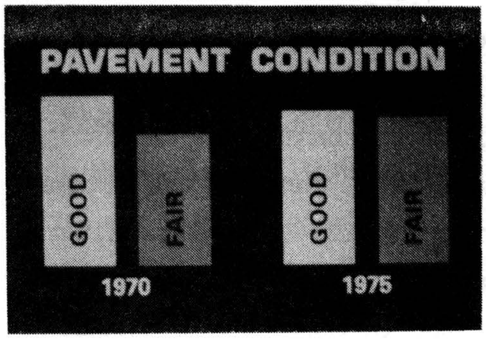
23.

The current rate of expenditure for captial improvements is about $14 billion annually, 30 to 40 percent of which goes for improvements in the pavement structure. And this figure doesn't include routine maintenance and operating costs.



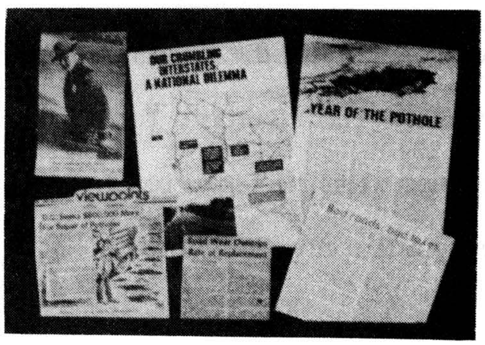
24.

Pavements in fact are the largest single piece in the overall highway picture. As the amount of new construction declines, pavements will assume an even larger role in the future.



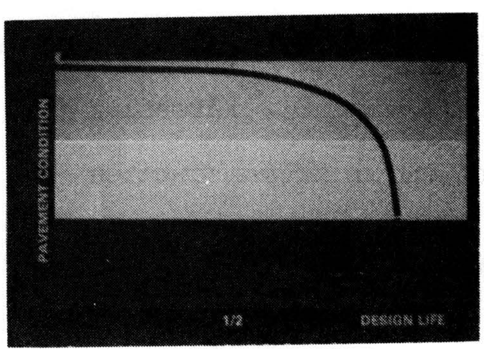
25.

Yet, even with all of these revenues going into pavements, the statistics show that our highway network is gradually deteriorating. Information submitted by the states, and used by the Secretary of Transportation in his 1977 report to Congress, showed that from 1970 to 1975 there was a small but significant shift in pavement condition from the "good" category to the "fair" category. This shift translates into many thousands of miles of pavements.



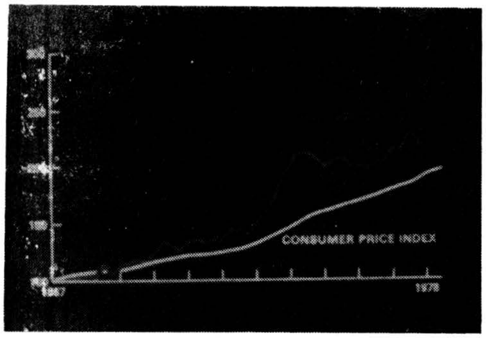
26.

Captions like these are becoming all too common in newspapers and magazines across the country. References to "gravel Interstates" and "multi-million dollar potholes" paint a grim, but in many cases true, picture of pavement conditions in some locations.



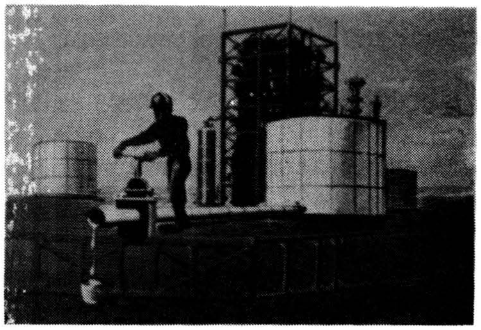
27.

What is perhaps the most alarming fact of all is that much of our Interstate System is now past the half way mark in its design life. As these pavements approach an age of 20 years, the rate of deterioration can be expected to increase significantly resulting in far more serious problems in the years ahead.



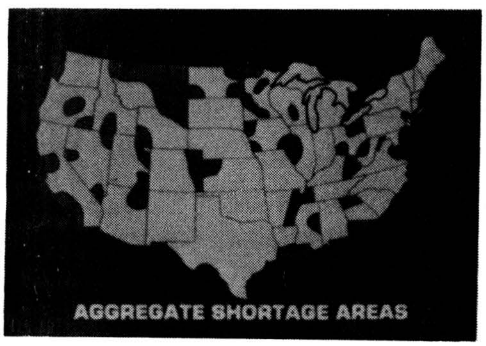
28.

We can all cite reasons for this trend in pavement condition. Inflation, the one cited most often, has hit the highway industry particularly hard. Between 1967 and 1978, the Federal Highway Administration's Contract Price Index rose from a base of 100 to a value of over 300. That's a rate of twice that of the Consumer Price Index.



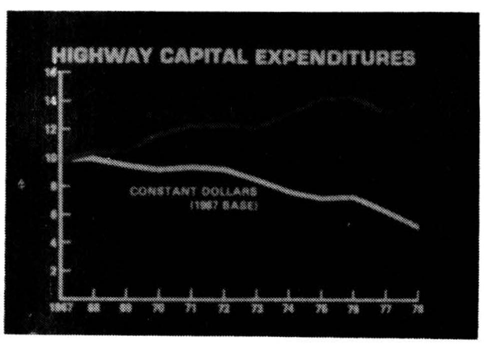
29.

Much of the difference between the two indices can be attributed to periods when the supply of crude oil was greatly reduced or threatened.



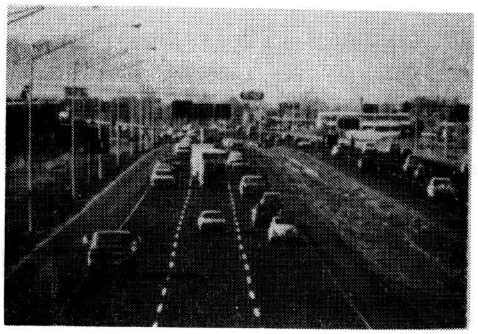
30.

But other factors have contributed as well. For instance, in some areas of the country, top quality paving aggregates must be imported, causing the price to double or even triple.



31.

Highway revenues have not risen nearly as fast as highway costs. As a result, the purchasing power of the highway dollar is just a fraction of what it was in the late 1960's.



32.

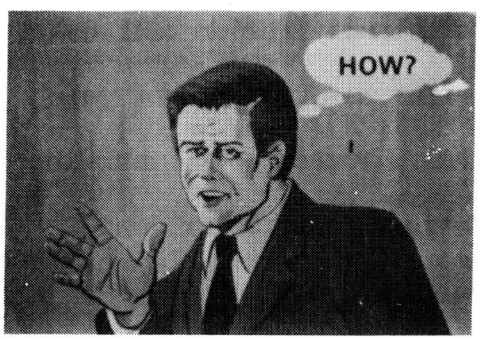
Traffic has also taken its toll. Increases in traffic volumes and accumulated loads far greater then those anticipated in design have caused many highway pavements to wear out long before their time.

33.

All of these factors have put the highway administrator in a difficult spot. He has had to cut some corners and, in many instances, defer much needed maintenance or rehabilitation work.

34.

What will the outcome of these actions be? It may be many years before we know for sure. But a good pavement management program would allow us to predict the consequences with reasonable accuracy and begin now to plan for the future.



35.

This brings us to our final question. How can we make the pavement management practices of our highway organizations more effective?

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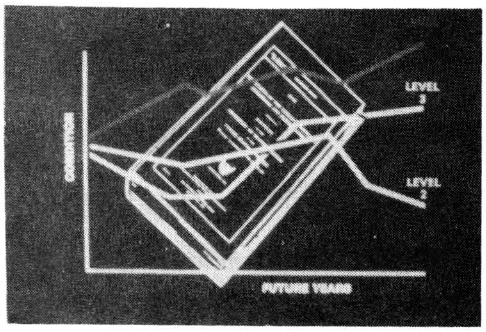
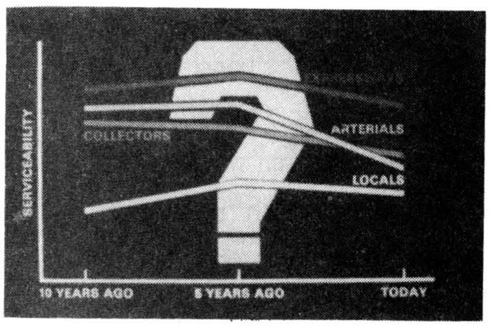
Before we start making changes, it is important that we first step back and take a good, hard look at the way we are doing things right now. In doing so, we must ask ourselves some difficult questions.

37.



Can we support our budget requests with solid facts and figures? Are we able to demonstrate to our legislatures and the public the consequences of a given funding level in terms of future pavement condition?

38.



Do we even know what the present condition of our system is? How does it compare with the condition last year? 5 years ago? or 10 years ago?

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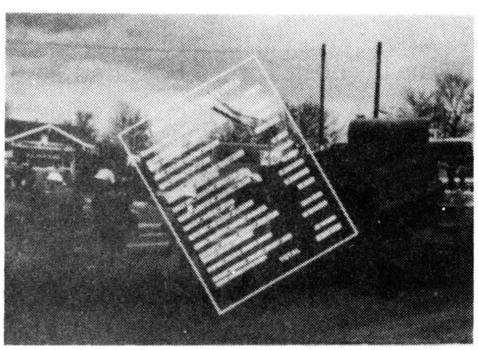
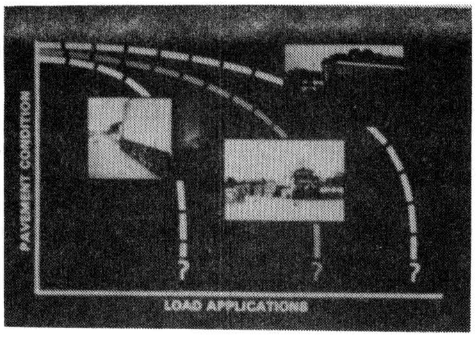
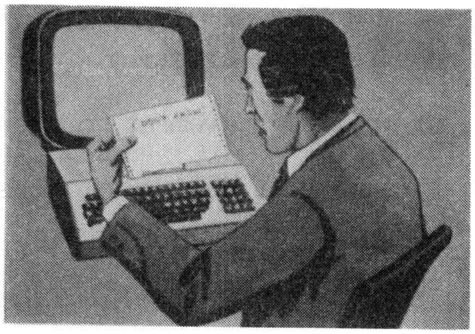


For any given section of highway, can we tell how much was spent for pavement maintenance last year? Do we know the cost of all pavement work, including construction, reconstruction, resurfacing, and maintenance, over its entire life time?



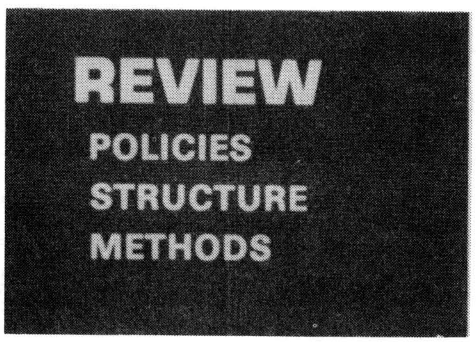
40.

Are our pavements giving us the service that was expected of them when they were designed? In terms of years? In terms of axle loads? What happens to that service if we change the legal load limit?



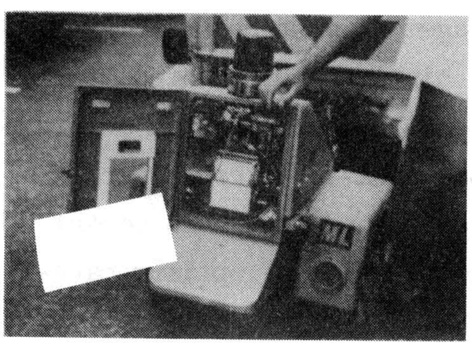
41.

If we are getting answers like this, our pavement management programs can probably stand substantial improvement.



42.

A good place to begin is with a review. Not a review of our pavements, but a review of our own organization -- its policies, its organizational structure, its methods of operation.



43.

We should also include in our review the various types of information about pavements that we have available and how that information is collected and stored.

44.

Finally, we should review the standards being used — standard specifications, standard plans, and standard procedures.

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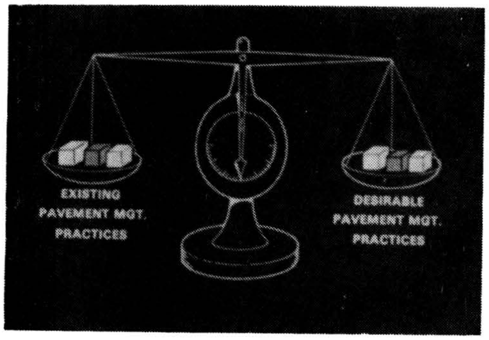
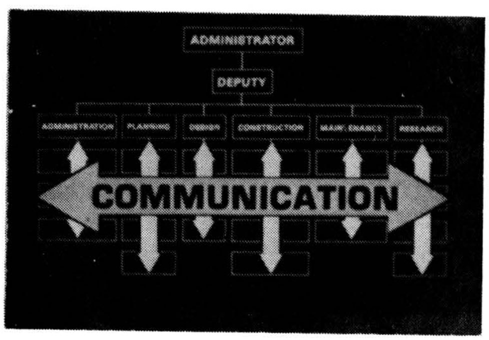


The next step is to compare these existing practices with those of an ideal pavement management program. To do this we must decide what the characteristics of an ideal program are.

46.



First, in order to have a good program, there must be good communication within the agency, not only vertical communication in the various operating units but horizontal communication among the units as well. Effective pavement management must be a cooperative effort and requires constant information sharing and feedback.

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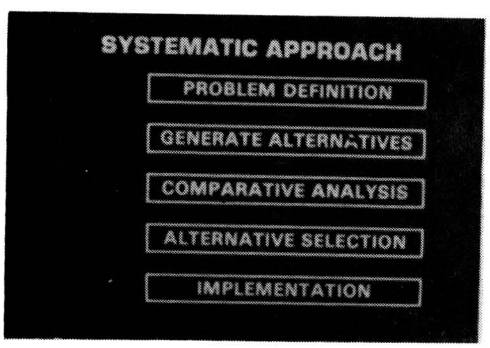
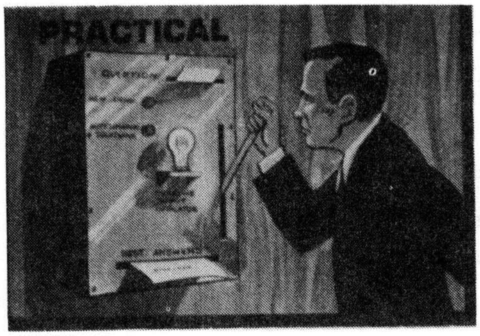


Second, pavement management must be systematic. That does not mean a highly sophisticated, fully computerized program. It does mean that an organized approach must be taken to be certain that all activities are considered and that each contributes to the overall objective of optimum pavement performance.



48.

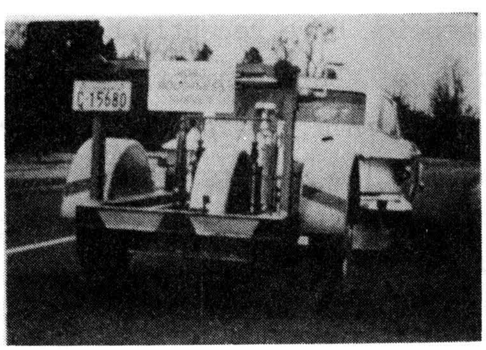
Third, it must be practical. Idealized or conceptional programs look fine on paper, but pavement management is an ongoing, real life activity and the program must reflect this by being completely workable and realistic.

49.

And finally, for a pavement management program to be fully effective, there must be acceptance by top management and a total committment to make it work.

50.

With these characteristics in mind, we can begin to analyze the practices of our own agencies and decide what improvements may be necessary. Let's look at a few examples.



51.

One of the most important activities in a pavement management program is the monitoring of pavement performance.

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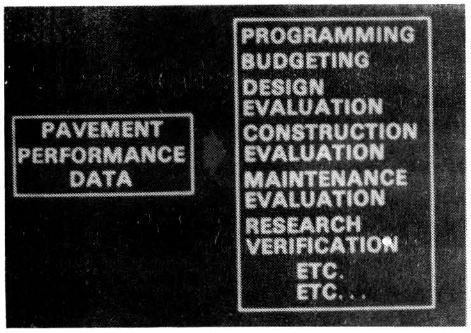

Pavement performance data, or more specifically the trends in pavement condition which are identified from year to year, serve as a basis for many other activities in the process including programming, budgeting, the evaluation of certain design, construction and maintenance practices, and field verification of research results.

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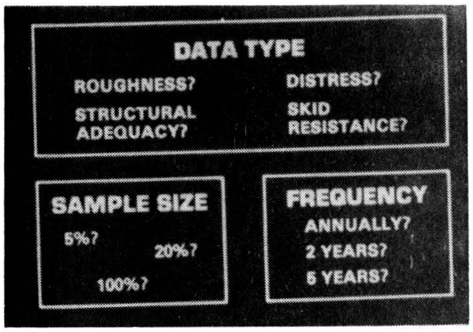
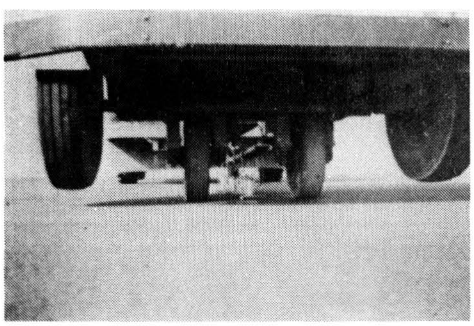


For this reason, our monitoring programs deserve close scrutiny. Our analysis should cover the types of data collected, the amount collected, and the frequency.

54.



Data collection is expensive and we cannot afford to collect it unless we also make good use of it. Many thousands of dollars can be wasted by improper selection of condition data, sample size, and sampling frequency.

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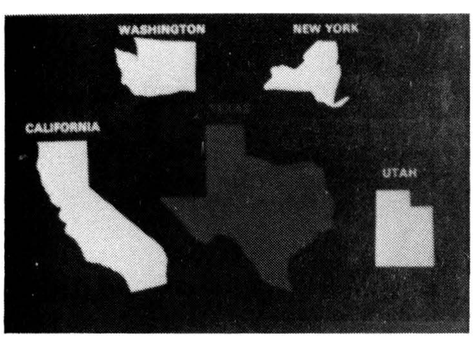
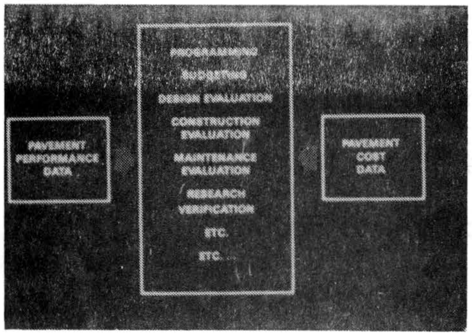
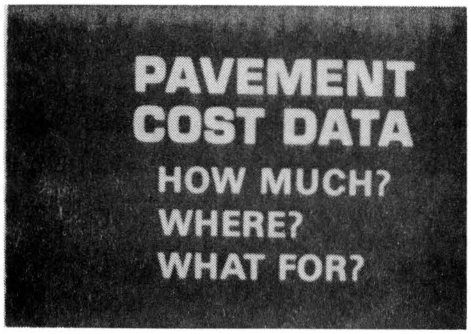


Some state highway agencies have recently completed thorough evaluations of their monitoring programs - Washington, California, Texas, Utah, and New York, to name a few. The results do not always have to mean additional data. A review can also identify unnecessary data which can be eliminated from current monitoring programs.



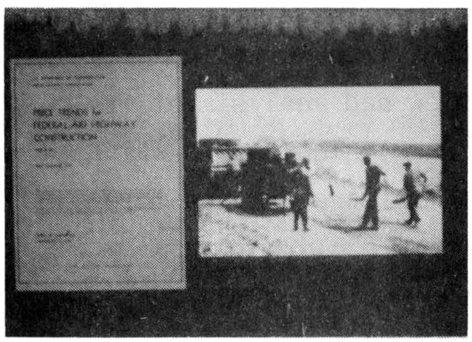
56.

Of equal importance is good cost data. Optimum pavement performance means the most cost-effective pavement possible; something we cannot hope to achieve with a good cost reporting system together with a good pavement monitoring program.



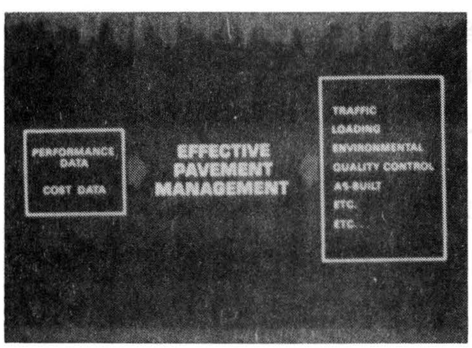
57.

The analysis should cover the type of cost data collected - not only how much was spent but where it was spent and exactly what it was spent for.



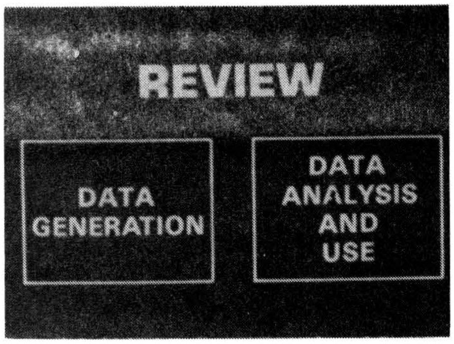
58.

Most agencies have good procedures for reporting unit bid price data on contract work. But it is often not as easy to identify the true costs for pavement maintenance items such as joint repair, crack sealing, and so on, and attribute those costs to a specific section of pavement.



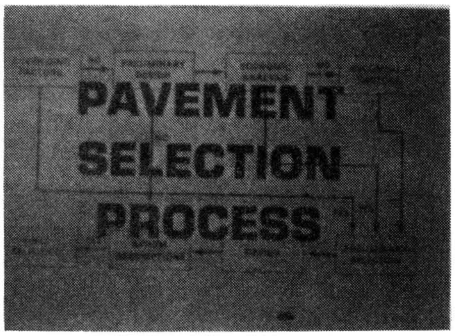
59.

Performance and cost data are but two of the types of information we need for pavement management. There are many others. Traffic and loading data, environmental data, quality control data, as-built measurements - all are important and the activities which produce them must also be carefully analyzed.



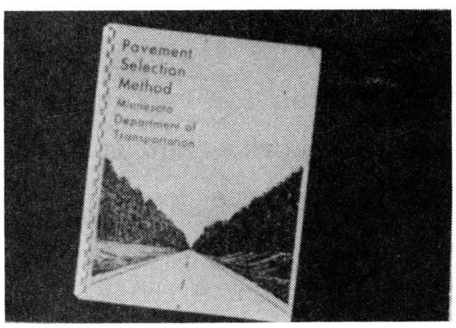
60.

While good data are essential, the evaluation and use of that data in other activities of the pavement management process are perhaps even more important and they must be analyzed too.



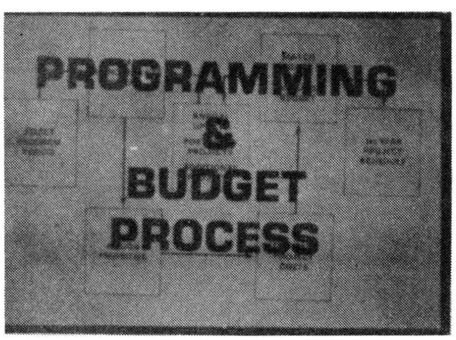
61.

The pavement selection process is one of those activities. To evaluate design options, they must be reduced to a common basis of comparison such as annual cost. All of the assumptions and methods used in the process must be periodically reanalyzed to be assured of their continued validity.



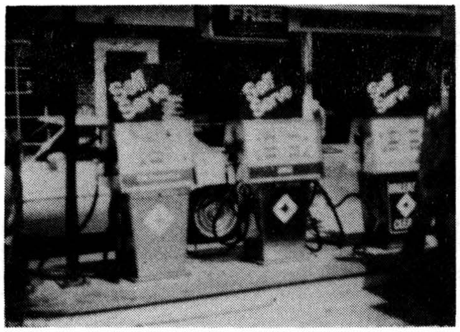
62.

The Minnesota Department of Transportation recently completed a review and analysis of its pavement selection procedure. The result was a procedure in which the administrators had more confidence in and which they could comfortably defend.



63.

The programming and budget process is another. The procedures used must result in accurate estimates of future needs and realistic priorities to allow us to plan and allocate available resources in an efficient manner.

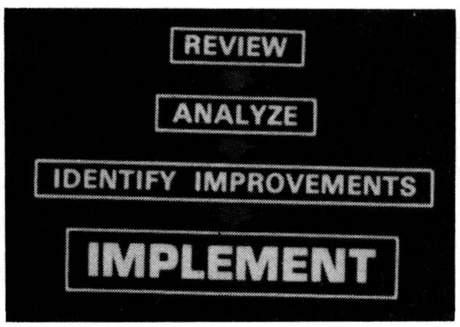


64.

The Utah Department of Transportation used its procedures to advantage and made a strong case before its legislature for additional funding. The result - a two cent per gallon increase in state gasoline tax.

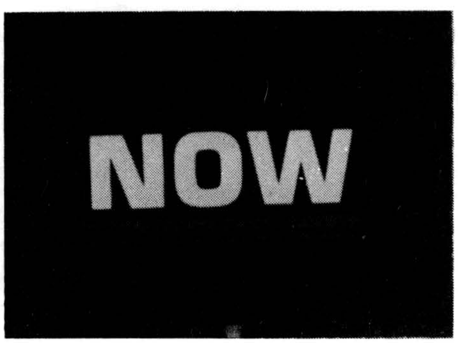
65.

These are just a few examples of good pavement management and the activities which must be reviewed and evaluated. A similar approach should be taken for each activity in the process which impacts on pavement performance.



66.

Once we have completed our reviews, analyzed our existing practices and identified improvements in the process, the only thing remaining is to make the necessary changes and implement them.



67.

Many can be implemented right now! - With available data and existing procedures. Good pavement management does not have to be a thing of the future.

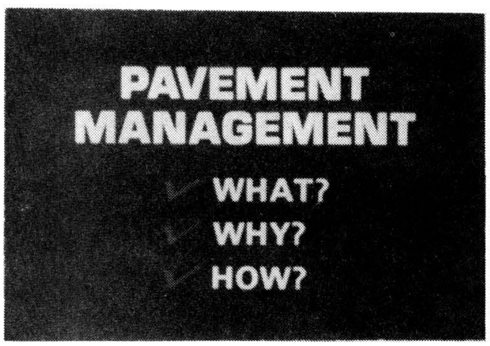
68.

For example, there are plenty of data around to evaluate the traffic and loading estimates we have used for past designs. If we have been accurate, let's verify it. If not, let's strengthen our estimating procedures where possible to make our designs more reliable.

69.

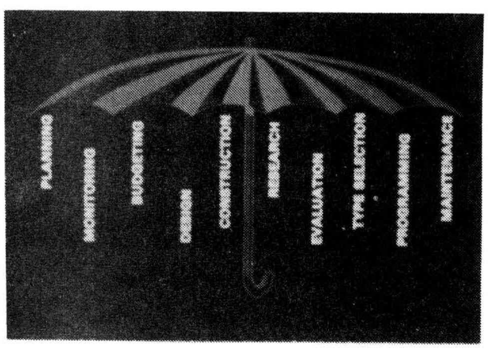
There will, however, be some things which we are not prepared to implement. In these cases, we must undertake the needed research and evaluation work as soon as possible to provide us with good data on which to base future decisions.

70.

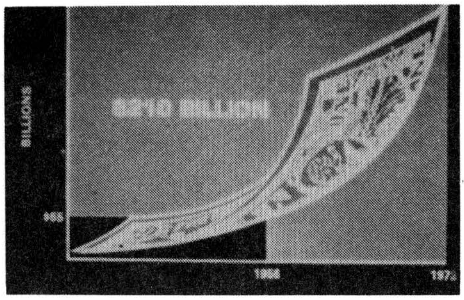


That's it. We've covered what pavement management is, why it's important, and how we can do something about it. Let's quickly summarize.

71.

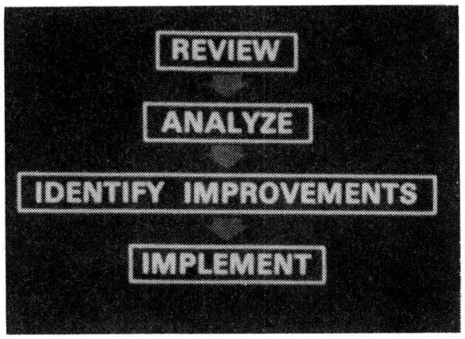


Pavement management is a term which includes all pavement related activities of a highway agency.



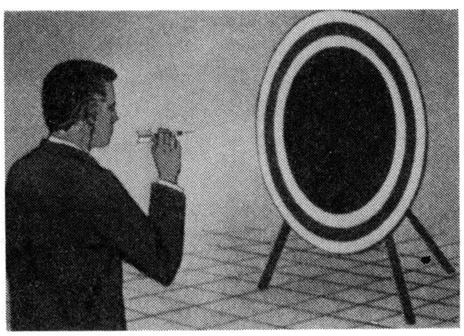
72.

It's important because of what is at stake, a tremendous investment that we may be beginning to lose.



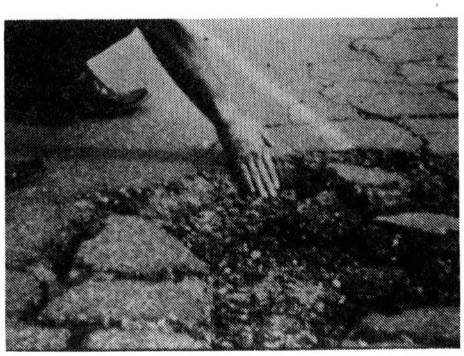
73.

But we can do something about it by reviewing our existing operations; by analyzing them to insure they are up-to-date, valid, and efficient; by identifying improvements that can be made; and by tailoring a program to correct any deficiencies that exist.



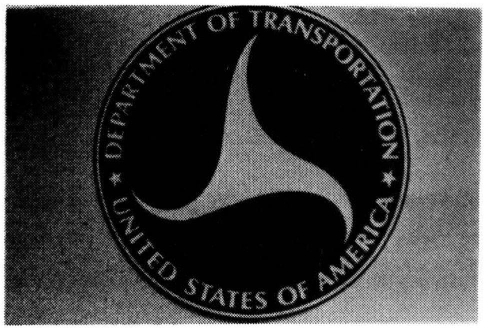
74.

A good pavement management program will not result in the correct decision every time. But it will greatly improve our chances by minimizing the possibility of error.



75.

It's an effort we can't afford to take lightly. We must meet the challenge head on and find ways to reverse the trend in pavement deterioration, safely, soundly, and economically through better pavement management.



76.

The end.

LESSON OUTLINE
INTRODUCTION TO THE PAVEMENT MANAGEMENT PROCESS

Instructional Objectives

1. To introduce the pavement management process and to define a pavement management system.

2. To outline the appliable levels and subsystems of a pavement management system and the basic features of each level.

Performance Objectives

1. The student should obtain a good foundation for the following detailed lectures on pavement management.

Abbreviated Summary	Time Allocations, min.
1. Background	20
2. Recommended Framework	20
3. Summary	10
	50 minutes

Reading Assignment

1. Haas and Hudson - Chapter 1

2. RTAC - Pavement Management Guide - Part 1

3. NCHRP 215

4. Instructional Text

LESSON OUTLINE
INTRODUCTION TO THE PAVEMENT MANAGEMENT PROCESS

1.0 BACKGROUND - ESSENTIAL PMS FUNCTIONS AND CONCEPTS

1.1 The Process of Pavement Management (Visual Aid 3.1)

This process has been developed primarily to manage a substantial investment in transportation.

1.1.1 Pavement Investment. A substantial investment exists in the present transportation network. Proper management of this investment is essential. (Visual Aid 3.2)

1.1.2 Maintenance Investment. Substantial annual expenditures are just to preserve and maintain this investment.

1.1.3 Limited Funds. Available funds for investments in pavements, and for maintenance of these investments are generally limited. Good management is, therefore, essential to obtain maximum value for limited funds.

1.2 Definition of a PMS

1.2.1 Coordinated Activities. A pavement management system consists of a comprehensive, coordinated set of activities associated with planning, design, construction, maintenance, evaluation, and research of pavements. (Visual Aid 3.3 and 3.4)

1.2.2 Optimum or Prioritized Strategies. Provides decision makers at all management levels with optimum or at least prioritized strategies.

1.2.3 Evaluate Alternatives. Provides an evaluation of alternate strategies over a specified analysis period.

1.2.4 Quantifiable Analysis. Based on predicted values of quantifiable pavement attributes, subject to predetermined criteria and constraints.

1.2.5 Dynamic Process. It is a dynamic process which incorporates feedback regarding the various attributes, criteria, and constraints involved in the optimization or prioritization procedure.

1.2.6 Applicability. The system is applicable for all types of decisions including those related to:

(a) information needs,

(b) projected network deficiencies,

(c) budgeting,

(d) programming,

(e) research,

(f) project design,

(g) construction,

(h) maintenance, and

(i) resource requirements.

2.0 RECOMMENDED FRAMEWORK FOR A PMS

2.1 Two Generalized Management Levels (Visual Aid 3.5)

An interface must exist between lower or detailed management levels and the pavement management level; as well as between a general highway or transportation system management level and the pavement management level.

2.1.1 Network and Project Levels of Activity. (Visual Aids 3.6, 3.7, 3.8, and 3.9) PMS involves primarily network and project activities but also research and special studies.

2.1.2 Feedback Loops. (Visual 3.10) Monitoring and evaluation of pavements on a periodic basis provides one of the primary sources of feedback at both the project and network level.

2.1.3 Data Base. (Visual Aid 3.11) A data base or information record is crucial for all pavement management activities both for input and outputs.

2.2. Rational Decision-Making (Visual Aid 3.12)

The similarity of the flow of information between the different activity areas (such as maintenance, design, and construction) forms the basis for a comprehensive basic pavement management framework.

2.2.1 Information. Pavement information is gathered.

2.2.2 Analysis. Consequences of the available choices are analyzed.

2.2.3 Decision. Based on this analysis and on other non-quantifiable considerations.

2.2.4 Implementation. The results of the decision are recorded on the data bank and passed on to other management levels.

3.0 SUMMARY (Visual Aids 3.13 and 3.14)

3.1 Three Basic Pavement Management Subsystems

These are identified as "Information," "Analysis," and "Implementation." The remaining steps are not considered to be components of the PMS. The pavement management system is directly involved in (a) the storage and retrieval of data, (b) the performance of technical and economic analysis, (c) the coordination and reporting of all activities, and (d) the associated updating of records.

3.2 What a PMS is Not

The PMS cannot directly consider non-quantifiable factors such as political factors; nor does it make decisions. These functions must be handled by the decision maker or administrator who uses the PMS output to assist him in making final decisions or recommendations.

LESSON OUTLINE
INTRODUCTION TO THE PAVEMENT MANAGEMENT PROCESS

VISUAL AID	TITLE
Visual Aid 3.1.	Why Pavement Management; What is it.
Visual Aid 3.2.	Transportation budget process.
Visual Aid 3.3.	Pavement management components and operational responsibility.
Visual Aid 3.4.	Basic principles of coordination.
Visual Aid 3.5.	Block diagram of pavement design system.
Visual Aid 3.6.	Project/Network Level design practices.
Visual Aid 3.7.	Project/Network Level construction practices.
Visual Aid 3.8.	Project/Network Level maintenance practices.
Visual Aid 3.9.	Project/Network Level rehabilitation practices.
Visual Aid 3.10.	Project/Network Level rehabilitation monitoring and evaluation practices.
Visual Aid 3.11.	Other considerations in good pavement management.
Visual Aid 3.12.	Relationships and activities of key components in the pavement management process.
Visual Aid 3.13.	Costs and benefits of pavement management.
Visual Aid 3.14.	Benefits to senior management.

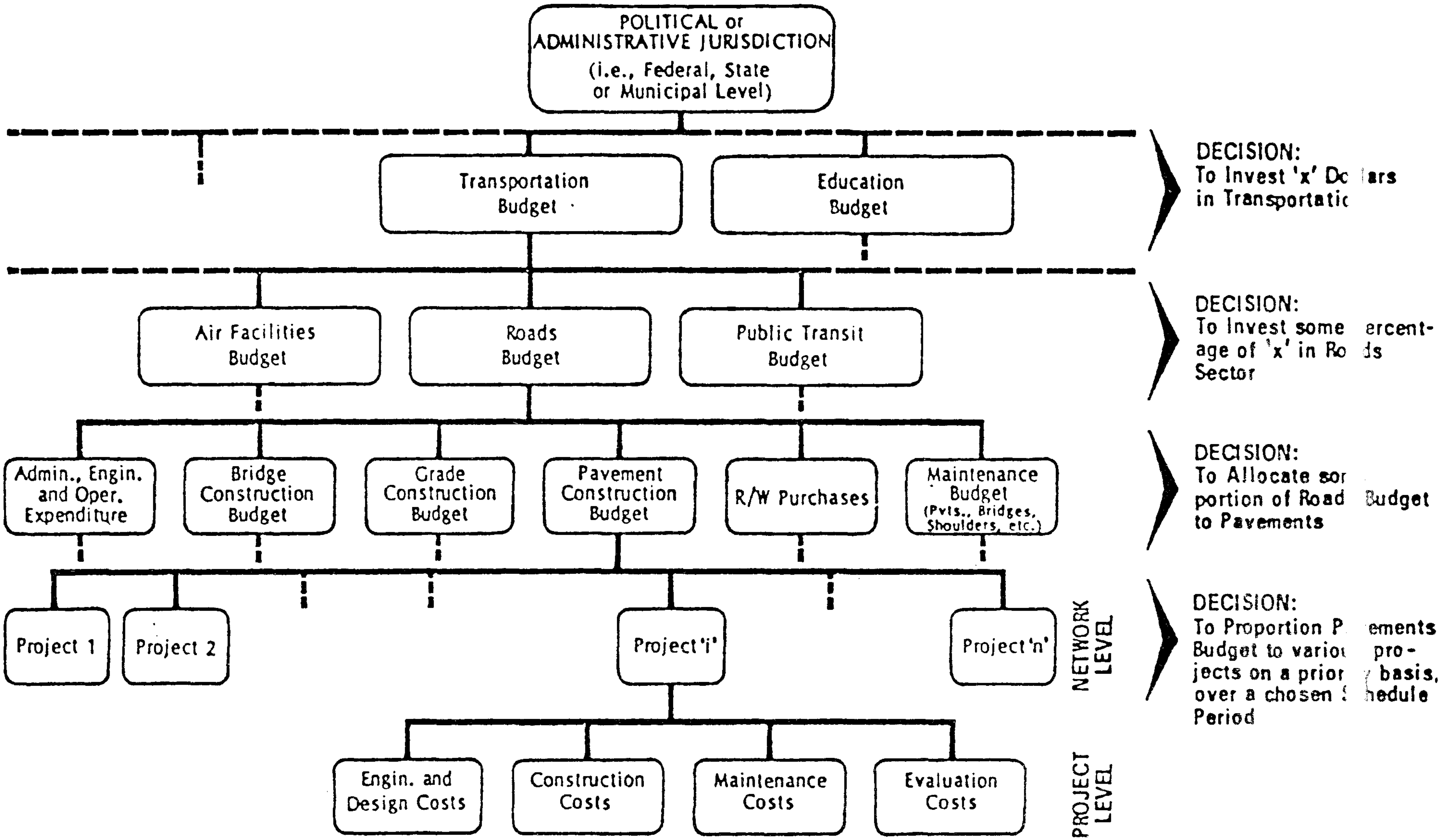
Visual Aid 3.1. Why Pavement Management; What is it.

WHY : **PAVED HIGHWAYS AND STREETS REPRESENT A VERY LARGE INVESTMENT ⟶ DESERVES GOOD MANAGEMENT**

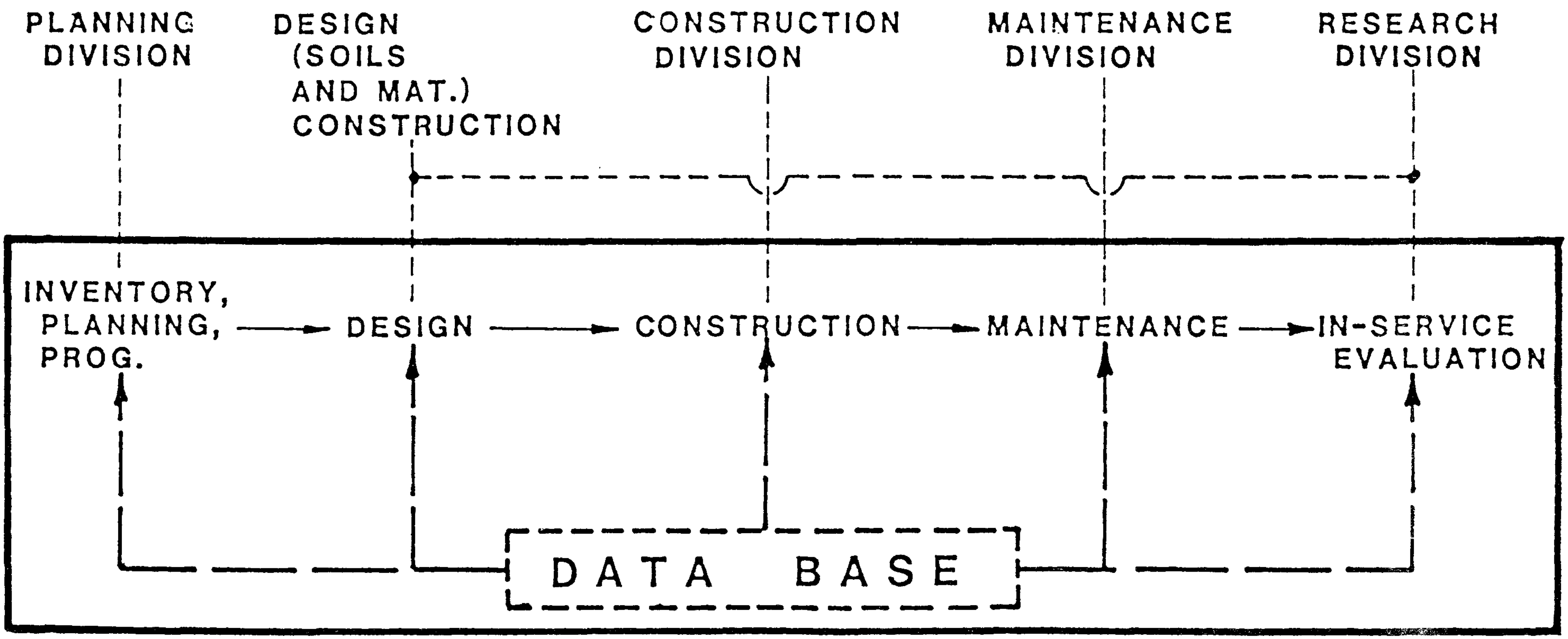
WHAT : **a) BASIC DEFINITION**

A COORDINATED, SYSTEMATIC WAY OF PROGRAMMING INVESTMENTS, DESIGN CONSTRUCTION, MAINTENANCE, IN-SERVICE EVALUATION AND RESEARCH FOR PAVEMENTS

Visual Aid 3.2. Transportation budget process.



Visual Aid 3.3. Pavement management components and operational responsibility.



Visual Aid 3.4. Basic principles of coordination.

BASIC PRINCIPLES

i) INFORMATION FROM IN-SERVICE EVALUATION

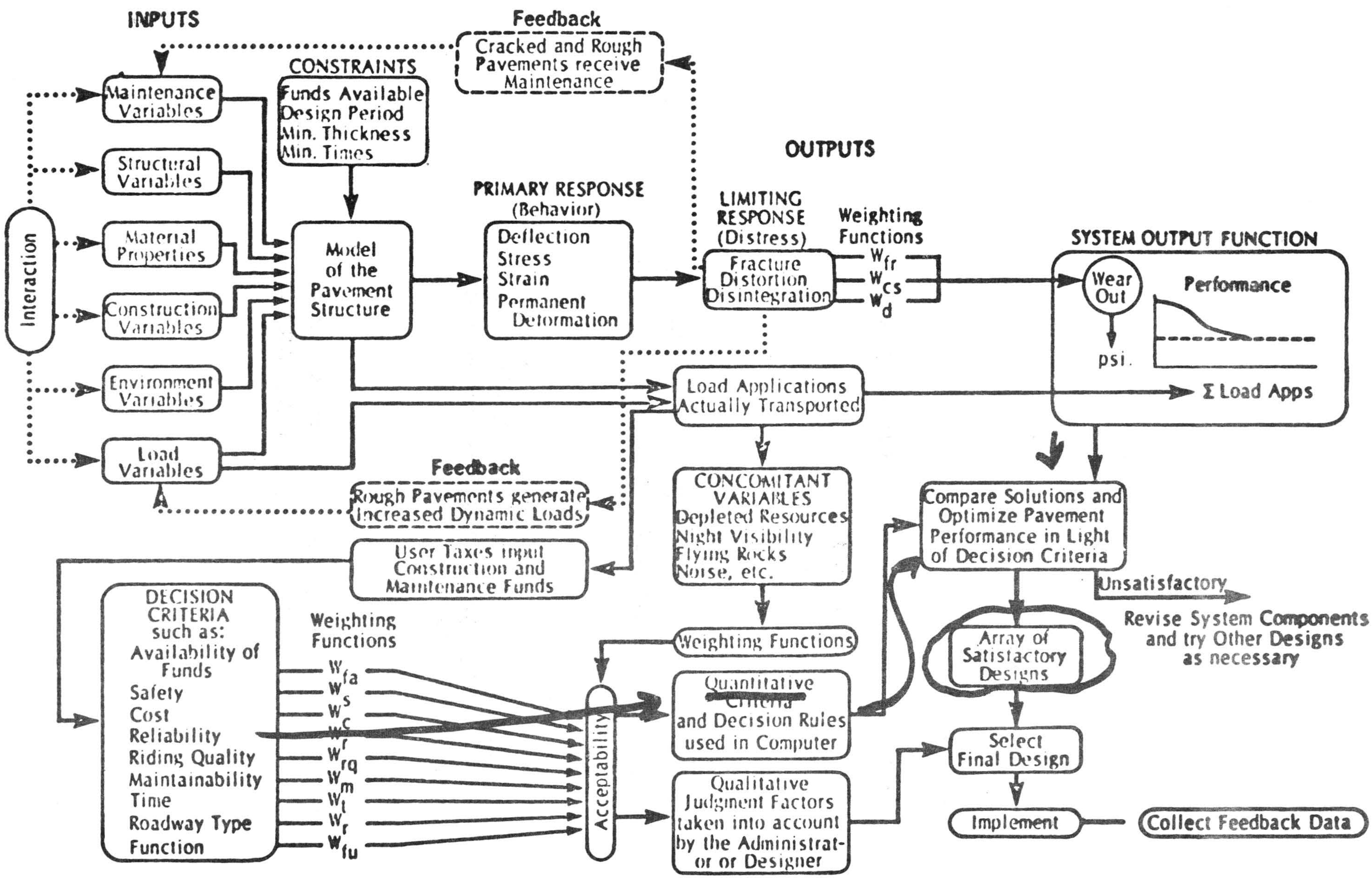
- IDENTIFY NEEDS
- PROGRAM $
- IMPROVE TECHNOLOGY

ii) DECISIONS OCCUR AT NETWORK AND PROJECT LEVELS; ALTERNATIVES ARE USUALLY AVAILABLE

iii) "SUCCESS" OF A DESIGN IS CLOSELY RELATED TO QUALITY OF CONSTRUCTION AND MAINTENANCE

iv) GOOD PAVEMENT MANAGEMENT MUST BE TAILORED TO THE HIGHWAY DEPARTMENT

Visual Aid 3.5. Block diagram of pavement design system.



Visual Aid 3.6. Project/Network Level design practices.

GOOD DESIGN PRACTICES: THE STARTING POINT

a) ASSIGNING RESPONSIBILITY

b) - TRAFFIC VOLUME AND LOAD DATA

- MATERIALS PROPERTIES , AVAILABILITY AND COSTS

- CLIMATIC DATA

c) DECIDE DESIGN OBJECTIVES, AND CONSTRAINTS

d) DECIDE WHAT ALTERNATIVES TO BE CONSIDERED

FLEXIBLE PAVEMENT DESIGN

- THICKNESS DESIGN (GOOD MODELS AVAILABLE)

- COST ESTIMATES

RIGID PAVEMENT DESIGN

- THICKNESS AND JOINT DESIGN (GOOD MODELS AVAILABLE)

- SUBBASE AND CONCRETE SHOULDER CONSIDERATIONS

- CRCP AS AN ALTERNATIVE

OTHER CONSIDERATIONS

- DRAINAGE AND FROST

- REGIONAL FACTORS

- FUTURE RECYCLABILITY

- CHECKING DESIGNS FOR DISTRESS (MODELS AVAILABLE)

Visual Aid 3.7. Project/Network Level construction practices.

CONSTRUCTION

SPECIFICATIONS AND CONTRACT AWARD

QUALITY CONTROL PROCEDURES

CONSTRUCTION PRACTICES AND MANAGEMENT

- ACCEPTANCE AND PENALTIES

AS BUILT DATA AND COST RECORDS

Visual Aid 3.8. Project/Network Level maintenance practices.

MAINTENANCE

MAINTENANCE MANAGEMENT

- NEEDS , BUDGETS, AND WORK SCHEDULES
- STANDARDS
- FIELD CONTROL AND REPORTING
- COSTS BY ACTIVITIES AND UNITS
- MAINTENANCE DATA RECORDS

Visual Aid 3.9. Project/Network Level rehabilitation practices.

REHABILITATION

- **EVALUATION**
- **OVERLAYS**
- **RECYCLING**
- **RRR PROGRAMS**

Visual Aid 3.10. Project/Network Level rehabilitation monitoring and evaluation practices.

MONITORING/EVALUATION

TYPES

a) STRUCTURAL EVALUATION (DEFLECTION)

b) SERVICEABILITY EVALUATION (ROUGHNESS)

c) DISTRESS EVALUATION (CONDITION SURVEYS)

d) SAFETY EVALUATION (SKID RESISTANCE)

FREQUENCY AND COSTS OF MONITORING , USES OF DATA

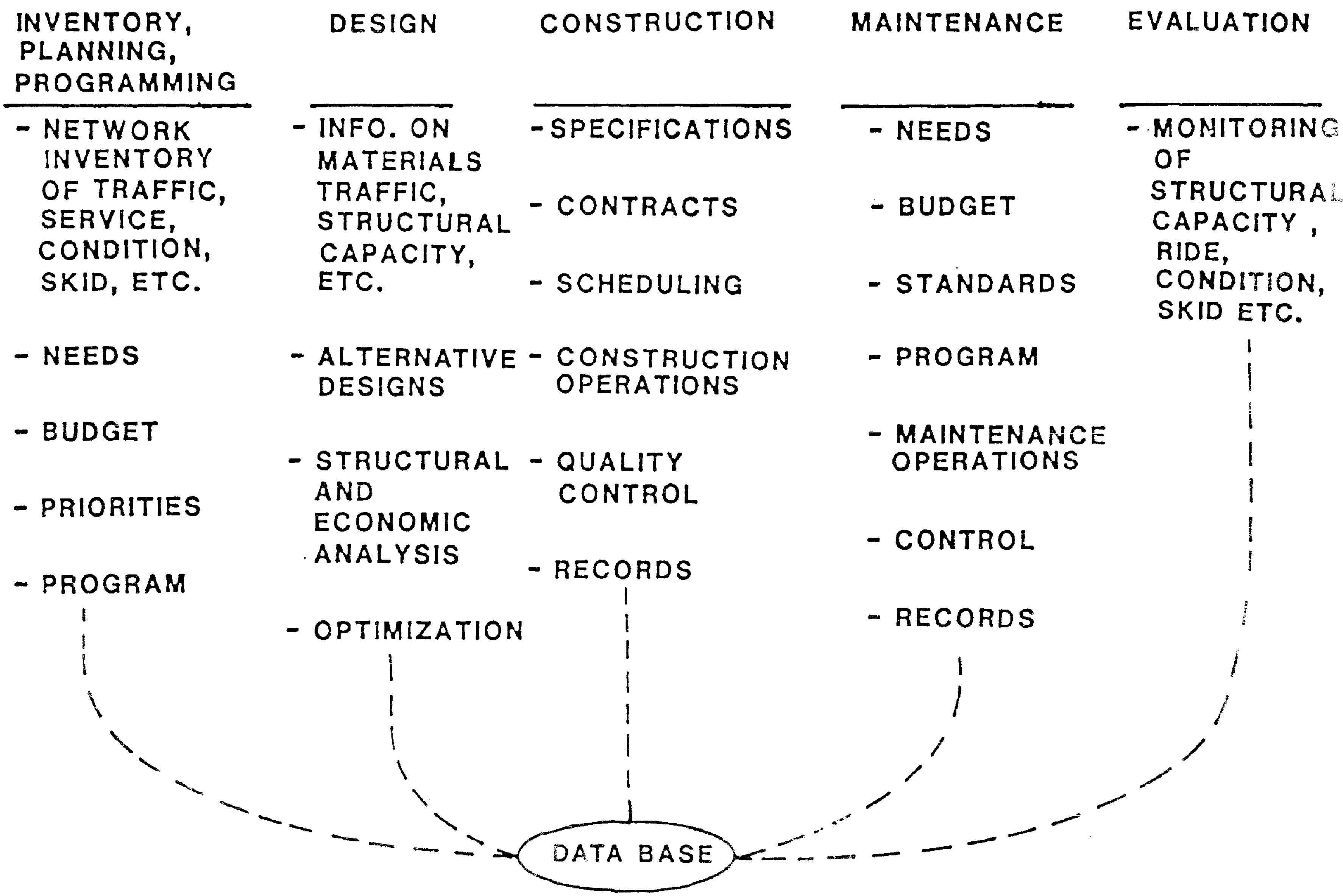
Visual Aid 3.11. Other considerations in good pavement management.

OTHER CONSIDERATIONS IN GOOD PAVEMENT MANAGEMENT

PRACTICES NOW

- **DATA BASE AS THE "FOUNDATION" FOR DESIGN, CONSTRUCTION MAINTENANCE AND REHABILITATION**
- **QUESTION OF PAVEMENT TYPE SELECTION**
- **NEW MATERIALS AND EXPERIMENTAL PROJECTS**
- **CHANGES IN LOAD LIMITS**
- **IMPLEMENTATION OF RESEARCH RESULTS, RESEARCH NEEDS AND PRIORITIES**
- **TRAINING OF PEOPLE**

Visual Aid 3.12. **RELATIONSHIPS AND ACTIVITIES OF KEY COMPONENTS IN THE PAVEMENT MANAGEMENT PROCESS**



Visual Aid 3.13. Costs and benefits of pavement management.

COSTS AND BENEFITS OF PAVEMENT MANAGEMENT

a) "COSTS":

- DESIGNATION OF CAPABLE , MOTIVATED PERSON(S)

- GETTING GOOD INVENTORY,PERIODIC EVALUATION, IMPLEMENTATION AND MAINTENANCE

b) "BENEFITS"

- BETTER CHANCE OF CORRECT DECISIONS: BETTER USE OF AVAILABLE FUNDS

- IMPROVED COORDINATION AND USE OF TECHNOLOGY.

- BETTER COMMUNICATION

Visual Aid 3.14. Benefits to senior management.

BENEFITS TO SENIOR MANAGEMENT

COMPREHENSIVE, COMPARATIVE ASSESSMENT OF CURRENT STATUS OF NETWORK

OBJECTIVELY BASED ANSWERS TO:

a) WHAT LEVEL OF FUNDING TO KEEP CURRENT STATUS , OR

b) IMPLICATIONS OF GREATER OR LESSER BUDGETS

ABLE TO BACK UP OR JUSTIFY CAPITAL AND MAINTENANCE PROGRAM TO LEGISLATURE

ASSURANCE THAT PROGRAM REPRESENTS BEST USE OF AVAILABLE DOLLARS

ABLE TO ASSIGN PRIORITIES ON OBJECTIVE BASIS UNDER LIMITED FUNDING

INSTRUCTIONAL TEXT

FHWA Pavement and Shoulders Notebook

Section 1.2

REVIEW GUIDELINES FOR PAVEMENT MANAGEMENT

Introduction

These guidelines are intended to identify and promote consideration of some of the more significant factors pertinent to the attainment of good pavement and shoulder performance at the minimum cost to the public. It is not intended or possible that these guidelines address, in depth, all of the factors having an impact on pavement management. The level of detail for the review of each aspect of the broad subject of pavement management should be tailored to best satisfy the most critical perceived needs.

ORGANIZATION

1. Is there adequate coordination within the highway agency (HA) to carry out an effective pavement management (PM) program? Although this question is listed first because of its obvious significance, it is recognized that it may not be possible to provide a meaningful answer to this question until other portions of the pavement management review have been completed. In answering this question, it is important that the coordination between all highway agency elements (i.e., planning, programming, budgeting, design, construction, maintenance, safety, materials, research, etc.) be addressed. Is there adequate feedback or communication between these elements to permit each one to function effectively? Is there duplication of efforts? How are the individual elements integrated into a pavement management process either formally or informally? Who is responsible for pavement management related functions within the elements? And on an overall basis?

PAVEMENT AND SHOULDER EVALUATION

2. Does the present process for evaluating the performance of pavements and shoulders provide the information needed to properly evaluate the adequacy of current design, planning, programming, construction and maintenance practices? It would, of course, be highly desirable if the evaluation process could provide pavement and shoulder performance data (in terms of both age and loading) and cost data (initial, 3R, and maintenance) in a form suitable for analysis. The quality of the highway agency's evaluation data, if any, should be reviewed considering sample size, use, equipment used, and repeatability of data.

and weighing procedures to insure reasonable data are collected upon which to base a projection; and finally, projection of trends for a future design period. Basing truck equivalents on trends projected into the future is more logical than using past values. A Utah study showed that consistent, logical trends could be developed if projection was done for each truck type individually (such as 5 axle semi-trailer). However, rational maximums, such as the 18 kip rate occurring with 100 percent loaded trucks should not be exceeded. A logical trend becomes asymptotic for projected values.

DESIGN

5. How and how often are project pavement designs checked in the division office?
   Enough checks should be made to draw the conclusion that pavement designs and the thickness can be supported. In general, the design procedures should take into account the same factors that the AASHTO procedures do (i.e., loading, soil strength, material strength, etc.); or there should be substantiated reasons for deviation, acceptable from an engineering basis. (Par. 3.a.(13) of FHPM 6-2-1-1.)

6. How does the highway agency determine (design) the structural section for shoulders? Does in-service shoulder performance and cost data support this procedure? What is the highway agency's criteria (warrants) for constructing stabilized, high type shoulders? Have the warrants been reviewed recently?

7. What expected life is assumed for new construction? For 3R construction? Does the actual service life of pavements or shoulders, in terms of both age and loading, support these assumed lives? Does the highway agency have a procedure for evaluating past designs or identifying design related performance problems?

8. When stage construction is utilized, is the second stage applied in a timely manner? Subsequent stages should be programmed and funded prior to the onset of significant structural deterioration of the initial stage.

DATA FOR FLEXIBLE PAVEMENT DESIGN

9. What soil testing procedure is used for pavement designs? What is the basis for its use?

Although FHWA is not in a position to dictate how an individual highway agency should collect or use performance evaluation data, the effective management of any product requires feedback on the performance and cost of the product. Pavement performance data can provide an important tool for decisionmakers who can use the data as input for:

a. Prioritizing pavement segments for reconstruction, 3R, or maintenance work.

b. The evaluation and selection of design, 3R, construction, programming and maintenance practices, and strategies.

c. The assessment of the present condition of the highway system, the projection of future funding needs, and the analysis of alternative strategies for optimizing the return on the expenditure of funds.

d. Allocation of funds for pavement related work among districts or sections.

TYPE SELECTION

3. Does the pavement type selection process currently used by the highway agency objectively evaluate alternative pavement sections? The highway agency's process should rely heavily on performance evaluation data to support expected service life estimates and estimates of future 3R and maintenance costs. Because of rapidly escalating construction costs and fluctuations in the availability of critical paving materials, it may often be desirable to reevaluate the pavement type selection for a project a few weeks prior to advertising for bids.

   Reviews of the highway agency's pavement and shoulder type selection procedures should be made periodically to insure their validity and that the factors listed in "An Informational Guide on Project Procedures, AASHTO, 1963," pages 49 to 54, have been adequately addressed (FHPM 6-2-1-1, paragraph 3.a(28)). It is not necessary that the type chosen be that which would be chosen by FHWA, but it is important that we be able to support the procedure as being a logical one. The highway agency should document the factors considered in the selection process.

LOADING

4. Is the present process of predicting traffic loading adequate for pavement design? In evaluating the process, it is desirable to look at: how well prior projections compare with actual loadings; vehicle classification

It is desirable that the procedure used by the highway agency be correlated with actual pavement performance within the State or area. In the event that it is necessary to use soil test-soil support relationships developed by other agencies, it is important to have a clear understanding of exactly how the other agency performs their soil tests. Minor variations in soil testing procedures can often have a significant impact on test results. The use of group index-soil support relationships is discouraged unless it is based on pavement performance data.

10. Does the designer have adequate information on the values and variability of soil strength to enable him to intelligently choose a value or values to be used in design? What is the frequency of testing? Are the values verified during construction? And changes made if necessary? Sufficient soil testing should be performed to insure proper identification of significant changes, and to insure representative values.

STRUCTURAL COEFFICIENTS

11. How are the values for structural coefficients, structural design strength, or gravel equivalents chosen? A comparison of predicted with actual performance is the best and most direct method of evaluation. It should be remembered that a significant variation in pavement performance may be expected.

12. Are material tests adequate to insure that the quality of materials meets assumed values for design? Durability, in addition to strength, must be taken into account.

REGIONAL FACTORS

13. Is the design procedure sensitive to regional differences in pavement performance that may result due to significant differences in climatic and environmental conditions? It is desirable that regional factors, if used, be based on actual regional differences in pavement performance. The regional factor at the AASHTO Road Test was developed from deflection measurements made throughout the year. There are other methods of accounting for seasonal or climatic effects such as adjustments to materials properties or soil strength, so that a regional factor is not necessarily required.

DATA FOR RIGID PAVEMENT DESIGN

14. What method is used to determine the modulus of subgrade reaction? The determination of the modulus value is not nearly as important to the structural requirements for a rigid pavement as the determination

of the soil support value is to the structural requirement of a flexible pavement. Therefore, conservative design assumptions may be appropriate. It is cautioned that subgrade and subbase support may decrease with time and that the selection of high subbase reaction values for design may be inappropriate unless the values can be supported by past experience.

15. How is the flexural strength of the concrete determined and how is a value selected for design? It is desirable to consider the variability in concrete strength that actually occurs in pavements when choosing a value for design. A more conservative value might be indicated for higher type highways. Field testing to determine values should be encouraged. The AASHTO Interim Guide contains recommendations concerning the relationship of flexural strength to working stress.

JOINT DESIGN

16. Are the joints performing as intended? What is the joint spacing? Do joint spacings conform to recommended practice? Is the shape factor and sealant type designed for the anticipated joint movement? Are noncorrosive dowels used? FHPM 6-2-4-4 (to be reissued as a TA) Recommended Procedures for PCC Pavement Joint Design covers this subject.

SUBBASE

17. What is the experience of the State with subbase design? The evolution of the presently used subbase design should be determined. The decision regarding the type currently used should desirably be supported by pavement performance evaluations.

CRCP

18. What method is used for CRCP thickness design? In the past, the typical approach called for a design solution for conventional PCC with a thickness reduction for CRCP. When local experience with CRCP indicates that added thickness is desirable, sections as thick as conventional PCC pavement designs may be considered favorably. The decision to use CRCP should be based on an engineering analysis that considers annual costs, and the other factors listed in the AASHTO Informational Guide on Project Procedures.

19. What are the design details and what is the percentage of steel used? Not less than 0.6 percent is recommended. There is some evidence that higher percentages should be used in colder climates. See FHPM 6-2-4-6, which will soon be converted to a Technical Advisory, for detailed recommendations. It is cautioned that past CRCP design practices--particularly those involving percentage of steel, concrete strength, and subbase type--that have provided good performance should not be changed without giving considerable thought to the possible consequences.

CONCRETE SHOULDERS

20 Are concrete shoulders considered for use with concrete pavements? (FHPM 6-2-4-5, to be reissued as a TA.) Have any been built? How have they performed? Has there been a comparison made of concrete shoulders versus asphalt concrete shoulders that attempts to analyze their annual costs (initial, maintenance, and 3R)? Better performance is attributed to PCC shoulders tied to the adjacent pavement because of reduced edge deflection, decreased water infiltration at the pavement/shoulder joint, and the lack of differential settlement between the shoulder and pavement.

DRAINAGE OF PAVEMENTS

21. During the design process, does the State attempt to analyze the hydraulics of water entering the pavement section? How? For what types of pavements?

22. Describe the subpavement drainage systems the State has constructed as part of both new and 3R pavement projects. What was the rational for their use? How have they performed? In comparison to control sections, or other pavements with similar characteristics? How is permeability evaluated? What is the basis for their design? Are permeable materials properly protected as per established filter criteria? Are the openings in underdrains compatible with backfill particle size?

FROST DESIGN

23. If frost is a consideration, how is it accounted for in design? A major cause of poor performance has been the assumption of frost free well draining materials, without adequate specifications or construction quality assurance to insure nonfrost susceptibility.

SKID RESISTANCE

24. How does the division assure itself that the PS&E's that it approves provide adequate skid resistance? (FHPM 6-2-4-3 - Skid Accident Reduction Program.)

25. Have the skid characteristics of the standard bituminous mixes and materials used in the State been evaluated so that skid values on future projects can be predicted? What are typical skid numbers for the various mixes? Have material sources been categorized based on their skid characteristics? The mixes should contain a high proportion of polish resistant coarse aggregate to provide an adequate texture. Maintenance mixes as well as construction mixes should be evaluated.

26. What texture procedure is specified for high speed (greater than 40 m.p.h.) rigid pavements? (FHWA Notice N 5080.59, soon to be reissued as a TA.) Are metal tines specified? What pattern is used? What is the basis for the pattern used?

27. Is a sharp, polish resistant sand specified in the PCC mixture? Fine texture, provided by the fine aggregate (and coarse aggregate when exposed), provides the adhesion component of skid resistance. Has the need for polish resistant coarse aggregates been evaluated?

28. Does the State have a skid trailer? Has it been calibrated? When? When is the next calibration scheduled? Skid numbers are sensitive to trailer repairs and operator knowledge. Therefore, they should be calibrated periodically.

29. What is the status of the skid inventory of selected sections, of accident locations, and of a sample of the highway system? What use is made of the data? Are seasonal variations recognized/considered in the use of the data? Reference FHPM 6-2-4-7.

30. How are skid overlay projects identified? Features other than skid number should play a significant role in the project selection process (FHPM 8-2-3). Is there a correlation between the skid resistance properties of pavement surfaces and accident experience? What life is expected with the typical surfaces used to correct skid prone locations (load life and age life)?

31. Does the State have a studded tire policy? Over the past several years, what is the trend in studded tire usage in the State? How is the durability of overlays or other skid corrective treatment impacted by studded tires? Are further efforts to secure a ban or restriction warranted?

RRR

32. How are overlay thicknesses and other RRR pavement strategies normally selected? Regardless of the design tools utilized, the design should desirably rely heavily on local experience. The performance of the RRR designs should be periodically evaluated.

TECHNOLOGY TRANSFER

33. How are technology transfer activities pertaining to pavements coordinated at the State? In the division? Cite examples of pavement or shoulder technology transfer that the State has recently implemented. (Of course, design and construction practices that have provided good performance should not be tampered with without giving a lot of thought to the possible consequences.)

34. What consideration is the State giving to energy and materials conservation, and the use of recycled materials? Describe performance of recycled pavements/shoulders, in comparison to virgin pavements? Is there some logical basis for determining whether a project should be recycled or overlayed?

35. What consideration is given for the use and evaluation of the following pavement materials or techniques?

    a. Fly ash or lime ash bases or subbases
    b. Emulsified asphalts
    c. Econocrete
    d. Thin bonded rigid overlays
    e. Sprinkle mix
    f. Sulfur extended asphalt mixes
    g. PCC shoulders
    h. Pavement/shoulder drainage systems
    i. Improved joint sealants

36 What additional effort is needed to improve the climate for technology transfer?

37. What pavement related research is underway in the State, and what is the status of that research? How well do the research and operations arms of the highway agency interact?

38. What pavement related experimental features has the State tried? How have they performed? Since a number of years are required before a pavement develops enough history on which to make a significant performance finding, experimental pavement features should be well thought out (work plan), and evaluated against control sections on a systematic basis.

CONSTRUCTION STANDARDS AND PRACTICES

39. Is the construction process providing the quality anticipated in design? (Initial serviceability index, initial skid resistance, long-term durability, densities, stabilities, thickness, strength.)

Are specifications evaluated to assure that the specified pavement construction methods and materials are in accordance with design assumptions?

40. Describe the bituminous mix design procedures. How are the following items evaluated in the process:

    a. Asphalt type and quality
    b. Mix resistance to water damage
    c. Aggregate quality, including skid resistance
    d. Prevention of bleeding
    e. Thermal cracking considerations

41. How are "D" cracking and deleterious aggregates identified in the concrete mix design process? How are wear and skid resistance qualities of the fine aggregate identified in the mix design process?

42. How are construction and maintenance personnel informed (trained) of the goals and assumptions used in the design, and the relative importance that construction and maintenance variables have on the service life of the pavement? Evaluate the "feedback" process between construction, planning, maintenance, design, etc.

MAINTENANCE STANDARDS AND APPLICATIONS

43. Describe the States' program, if any, for evaluating the effectiveness of their various maintenance practices.

44. Describe the process for the exchange of pavement information (feedback) between those responsible for highway maintenance with construction, design, and evaluation.

PLANNING AND PROGRAMMING

45. How is pavement related work (reconstruction, 3R, and maintenance) prioritized and programmed? What advance planning has been accomplished in this area?

LESSON OUTLINE
THE SYSTEMATIC APPROACH TO PAVEMENT MANAGEMENT

Instructional Objectives

1. The concepts presented here provide an overall framework for subsequent pavement management lectures.

2. The instructor should outline the basic elements of systems methodology as they apply to network and project level analysis.

Performance Objectives

1. The student should have a strong foundation in principles of a coordinated Pavement Management System.

2. The student should be able to explain the benefits gained by the systematization of the Pavement Management process.

Abbreviated Summary	Time Allocations, min.
1. Pavement Management Defined	5
2. What is Performance?	10
3. General Structure of Systematic Management	20
4. Monitoring and Evaluation	15
	50 minutes

Reading Assignment

1. Haas and Hudson - Chapter 2

2. NCHRP 215

3. Instructional Text

LESSON OUTLINE
THE SYSTEMATIC APPROACH TO PAVEMENT MANAGEMENT

1.0 PAVEMENT MANAGEMENT DEFINED (Slides 4.1 to 4.5)

There is a great deal of difference of opinion as the terminology, key factors and related items associated with Pavement Management Systems. Many people think the words "management" and "systems" are nothing more than "buzz words" coined to gain attention. At the other extreme, some people feel that a Pavement Management System (PMS) is a highly sophisticated computer based technology that is a panacea for all pavement problems. Both of these views are, of course, false.

1.1 <u>"System"</u>

The word "System" has been appropriated for many purposes, such as circulatory system, drainage system, sprinkler system, the highway system. The dictionary says that:

> a "system" is a regularly interacting or interdependent group of items forming a unified whole.

1.2 <u>"Management"</u>

The word "management" means many things to many people. To some it means "to administer". To others it is "to control", and still to others, it means "to coordinate the various elements of". The dictionary definition of management is "the act or art of managing", or less circularly, "the judicious use of means to accomplish an end".

1.3 <u>"Pavement Management"</u>

"Pavement Management" in its broadest sense encompasses all the activities involved in organizing and managing the pavement portion of a public works program, large or small. The objective of the management system is to use reliable information and decision criteria in an organized framework to produce a cost effective pavement program.

2.0 IS SYSTEMS ENGINEERING HELPFUL? (Slides 4.6 - 4.10)

The space program had many spinoffs. Your digital watch, your pocket calculator, your miniature radio, are examples. One other thing that resulted from the space program is "a set of improved mathematical tools for predicting behavior of complex physical entities and for analyzing effects".

2.1 Concept of Interaction

This is the effect of factors on each other to drastically change the overall effect of either factor alone.

2.2 Feedback

That is "using information and/or a physical reaction to adjust a process or subsequent activities".

2.3 Systems Methodology

Systems methodology comprises a body of knowledge that has been developed for the efficient planning, design and implementation of new systems and for structuring the state of knowledge on an existing system or modeling its operation. There are three main uses of system methodology from which we can draw:

(a) The system approach
(b) Systems analysis
(c) Systems engineering

3.0 WHAT IS THE SYSTEMATIC APPROACH (Slides 4.24 - 4.28)

The systematic approach is the framing or structuring of a problem or a body of knowledge.

3.1 Systems Analysis

Systems analysis is closely related in that it is the use of analytical tools for actually modeling and solving the problem as structured.

3.2 Systems Engineering

Systems engineering is a more complete manifestation of the systems method, with design, implementation and performance evaluation aspects getting strong attention.

4.0 WHAT IS PERFORMANCE?

The evaluation of pavement performance involves a study of the functional behavior of a length of pavement in its entirety. Performance can be explained as the sum total of service provided by the pavement, where serviceability is defined relative to the purpose for which the pavement was constructed.

4.1 Performance Studies (Slides 4.29 - 4.38)

During the early and mid 60's studies were conducted on the recently acquired AASHO Road Test data and concepts. Researchers at the University of Texas began a basic new look at pavement design using a systems approach. Somewhat independent efforts were being conducted at the same time in Canada by Phang, Haas, et al, to structure the overall pavement design and management program and several of its subsystems. A third concurrent keystone effort in this area was that of Scrivner and others of the Texas Transportation Institute. All of these studies pointed to broader needs in the pavement field, such as a need to qualitatively look at pavements. The results included the following findings.

(a) The studies pointed out the need for real, continuous observations of pavements in service and the need to record these observations in a data set.

(b) It was found that pavements generally do not last 20 to 25 years without heavy maintenance and/or overlays.

(c) It was generally found that equations or mathematical models are essential to predict pavement deterioration history as a function of time, traffic and environment. Such existing models were simply not adequate.

(d) It was found that there is a significant variability in most pavement factors, such as materials, construction and traffic. This variability requires that periodic updating be done of all predictions of plans, of maintenance programs, etc.

4.2 Details of Studies

The remainder of this couse will cover many of the details and advancements that resulted from these early works.

5.0 THE GENERAL STRUCTURE OF SYSTEMATIC MANAGEMENT INCLUDES 8 BASIC FACTORS (Slides 4.39 - 4.49)

As a part of the pavement management system development at the design level, the design process was structured and its components were identified more specifically.

5.1 Inputs

Inputs, objectives and criteria for good designs were established.

5.2 Models

A structural analysis of alternatives was identified. It became understood by most persons that a simple model of pavement design expressed, for example, as a "simple design chart" could not adequately treat the analysis of improved pavement materials.

5.3 Behavior - Distress

It was recognized that most pavement models predicted pavement behavior. Given the prediction of behavior, it was further recognized that behavior carried to its limit leads to distress. It became clear that better prediction models for pavement behavior, and thus for cracking and other pavement distress, were essential.

5.4 Performance - Output Function

Accumulated distress changes the pavement serviceability and the pavement serviceability history defines its performance.

5.5 Safety

It was also essential to provide evaluations of the inservice behavior with regards to safety as well structurally.

5.6 Costs

Economic analysis become recognized as a vital part of the pavement management process.

5.7 Decision Criteria

Closely tied to the economics were decisions on allowable costs versus the resulting benefits related to a particular pavement choice. These factors must be explicitly defined and considered in the analyses.

5.8 Compare - Optimize

Optimization became recognized as an important step in the process which must be applied in making pavement decisions rather than relying on gut reactions or engineering judgement totally.

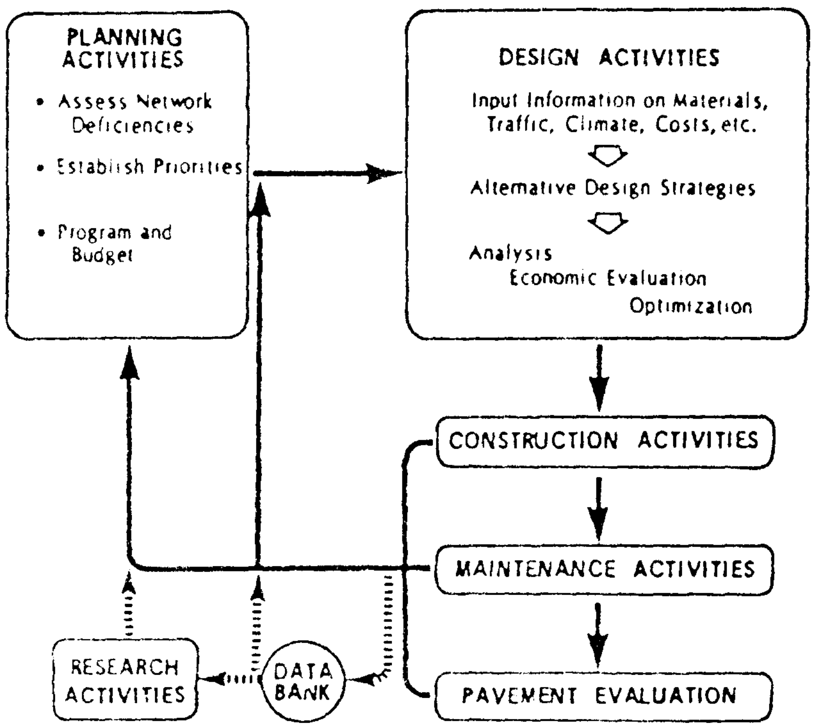
6.0 IMPLEMENTATION (Slides 4.50 - 4.52)

While there were significant developments in the pavement management system concepts, especially at the project level, there have been significant misunderstandings and delays in implementation in many cases. "We don't have enough money to do all the good projects we have under consideration, therefore, why do we need a PMS to help us make decisions?" Or, "We have adequate funds to take care of the projects we have and any time we can overlay our pavements or seal them we can be assured that the money will be wisely spent, thus we don't need a PMS." Systems of all kinds involve the re-evaluation of traditional organizational and operational methodologies and objectives. Because of this it is not easy to establish a pavement management system, and while there has been significant development in the pavement management field in the 1960s, there was not the expanded use of such management systems as expected.

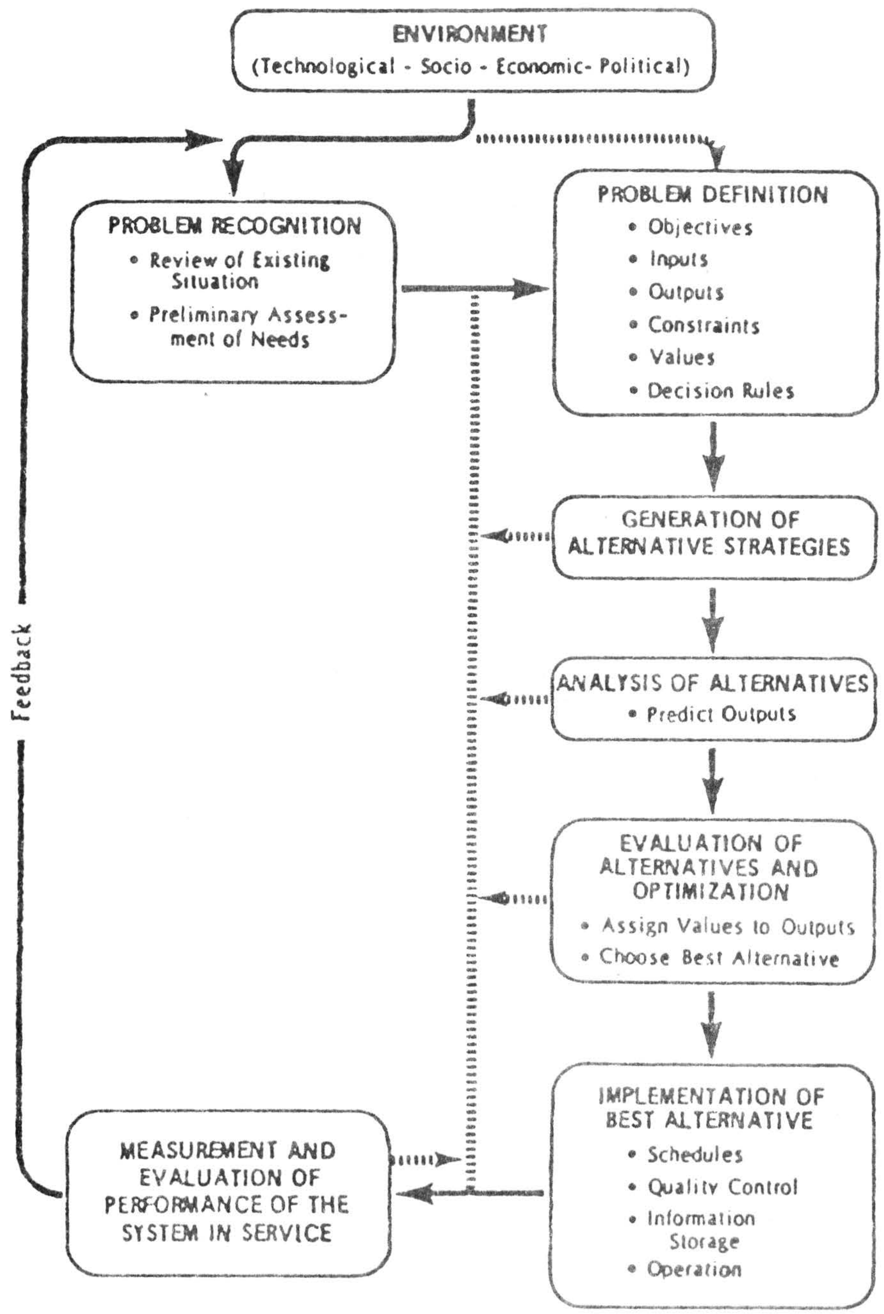
LESSON 4 - LESSON OUTLINE
THE SYSTEMATIC APPROACH TO PAVEMENT MANAGEMENT

VISUAL AID	TITLE
Visual 4.1	Major Classes of Activities in Pavement Management
Visual 4.2	Major Phases and Components of the Systems Methods
Visual 4.3	Simplified Block Diagram of the Major Components of Pavement Design
Visual 4.4	Simplified Predictive Portion of Pavement Design and Related Examples of Types of Periodic Evaluation Measurements

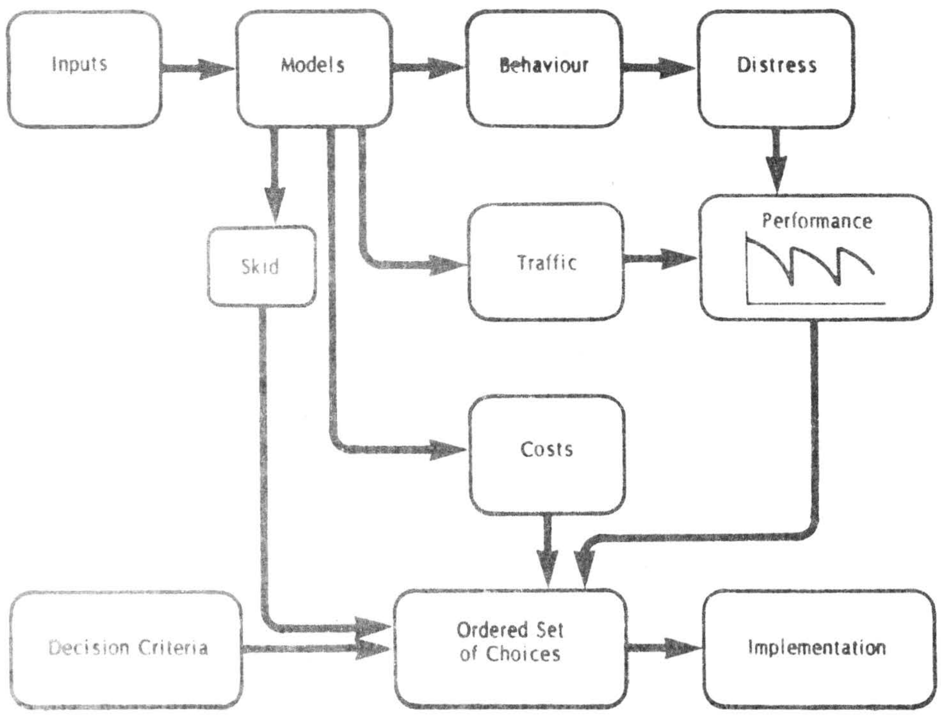
Visual Aid 4.1. Major classes of activities in pavement management.



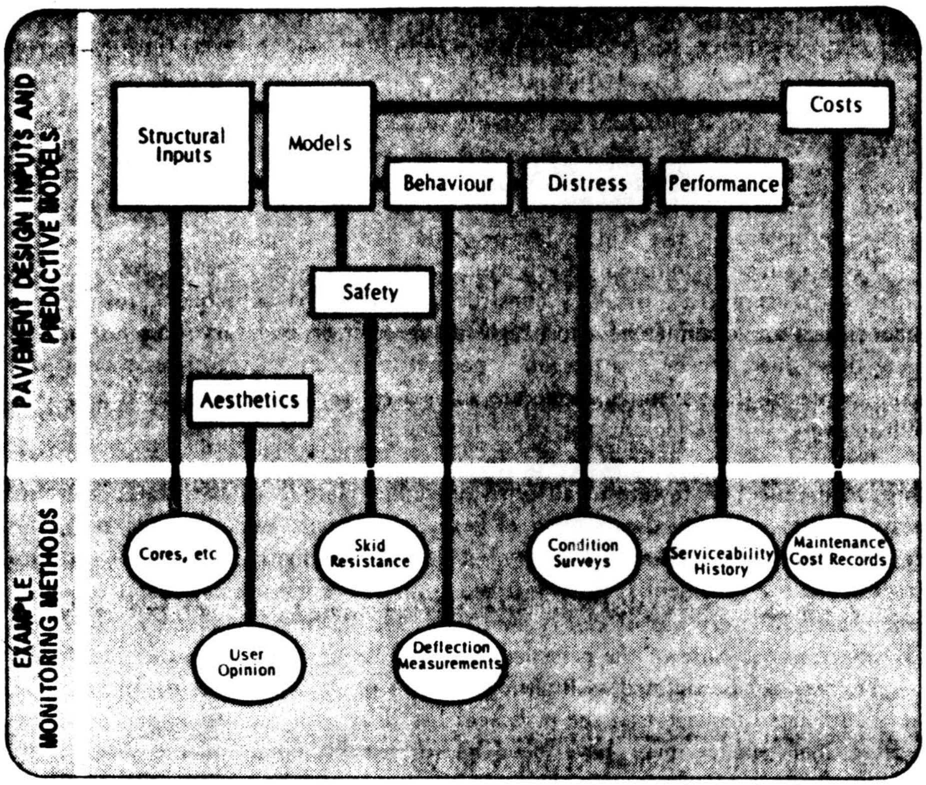
Visual Aid 4.2. Major phases and components of the systems methods.



Visual Aid 4.3. Simplified block diagram of the major components of pavement design.



Visual Aid 4.4 Simplified predictive portion of pavement design and related examples of types of periodic evaluation measurements.



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INSTRUCTIONAL TEXT

SYSTEMS APPROACH TO DESIGN, EVALUATION
AND MANAGEMENT OF PAVEMENTS

by

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Systems approach to Design, Evalution and Management of Pavements

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Acknowledgments

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Other major portions of the work were supported by the Texas Highway Department and the Federal Highway Administration.
Work of this nature is highly complex and a large team of researchers has been responsible for the findings given herein. Key contributors who deserve special recognition include Fred N. Finn, B. F. McCullough, B. A. Vallerga, Kesh Nair, Frank Scrivner, James L. Brown and Karl Pister.

Abstract

The complex nature of highway pavements and the demands placed on them by traffic and environment have resulted in a piecemeal and incomplete design methodology. It has become apparent from analysis of the problem that realistic analyses of pavement design and management problems can be obtained only by looking at the total pavement system, i.e., through systems analysis.
This report describes such an approach and presents some systems concepts. More than 50 physical inputs and constraints affect pavement design strategies from which the pavement designer or administrator may select his design. The systems approach gives him considerable scope and flexibility in exploring design options and a better chance of achieving the best possible design with no loss of the normal decision-making power.
The report discusses possible ways of establishing an overall system of pavement analysis and research implementation, i.e., a pavement management system. It is concluded that a systems approach to pavement design and

research is feasible and should be further pursued to develop more comprehensive pavement management techniques.

KEY WORDS: systems analysis, systems engineering, design, pavements, flexible pavements, pavement structure, optimization, pavement design, performance, analysis, research management, computer program.

1. Introduction

Highway pavements can be viewed as complex structural systems involving many variables, e.g., combinations of load, environment, performance, pavement structure, construction, maintenance, materials, and economics. In order to design, build, and maintain better pavements, it is important that most aspects of a pavement system be more completely understood and that design and research be conducted within a systems framework. Some people think of pavements as inexpensive parts of the highway system; but they are not. An investment of approximately 20 billion dollars will be made in pavements for the U.S. Interstate Highway System alone, and millions more will be spent annually on maintenance and upgrading. Thus, it may be concluded that pavements are an important and expensive part of the total transportation system and that improvements in designing them could result in substantial savings.

The Problem

In recent years considerable research has been conducted to investigate many specific problems concerning components of pavement design. Each of the 50 U.S. states has been involved in such projects and the Federal Highway Administration has sponsored a series of projects at the national level. Additional work has been supported by the National Cooperative Highway Research Program (NCHRP). Unfortunately, many of these efforts are fragmented and uncollated, and thus cannot be easily combined to improve design methods. As modern technology has developed and the complexity of the interaction of design factors has become better known, the need for a systematic approach to the overall problem of pavement design and management has become more evident. It is also evident that this approach should involve a team effort of interest research agencies and sponsors.

The AASHO Road Test illustrates the magnitude of the pavement design problem (Ref. 15). Though it was a 30 million dollar research project it answered only a few of the important design questions, and it seems that no single experiment is big enough to answer all the questions.
Likewise, no single mathematical equation or model can be used to describe pavement behavior completely. Instead, a coordinated, systematic approach

is needed; that is, a framework within which the multitude of physical and socio-economic variables involved can be sorted out and related in a meaningful way. Such an approach has been called the ,systems approach' (Ref. 2). Because this terminology has many definitions,[1] a brief write-up of the approach involved within this project is given in Appendix D.

A 1967 NCHRP project led to the first work in the applications of systems engineering to pavement design (Ref. 23). In a similar but independent effort, Hutchinson and Haas (Ref. 24), applied a systems approach to structuring the overall problem and several of the subsystem design problems. Simultaneously, the Texas Transportation Institute developed a working design model in connection with a cooperative research project with the Texas Highway Department. As a result of these studies, the Texas Highway Department, recognizing the need for a system for organizing and coordinating their pavement research program and updating their design system, initiated a project in cooperation with The University of Texas Center for Highway Research and the Texas Transportation Institute of Texas A&M University.

Pavement design guides based on the results of the AASHO Road Test have been developed by the American Association of State Highway Officials, but their use has shown major limitations and called attention to the lack of proven information which can be used in extrapolating the Road Test results to a variety of paving materials, methods of construction, environments and traffic characteristics. Attempts to use the AASHO Road Test results have also been made by other agencies, including individual state highway departments, and their experience has been similar. This problem does not suggest a weakness in the Road Test experiments, but does point out the need for a theoretical base from which to correlate and extrapolate the resulting information and data to other situations. These efforts to translate the AASHO Road Test information and other pavement research results have made it evident that an extension of the range of pavement design methods to incorporate (a) material properties, (b) methods of construction, (c) environment, and (d) traffic would require a more fundamental approach using modern technology.

In 1966 a project entitled ,Translating AASHO Road Test Findings Basic Properties of Pavement Components' was begun. The general objective of this project, as written in the project statement (1), was to provide the type

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[1]It is important to note that the ,system' being considered can be the actual pavement structure or some component of it; the broad management framework of some component used to provide and operate this structure, or it can be some combination of both. In effect, the work ,systems' has a broad meaning and its operational definition for a particular situation is determined by the manner in which the problem is structured.

of basic information required to adapt to local environments information such as that obtained on the AASHO Road Test. This was to be accomplished by carrying out the specific objectives described in the project statement:

1. development of descriptions of significant basic properties of materials used in roadway structures;
2. development of procedures for measuring these properties in a manner applicable to pavement design and evaluation;
3. development of procedures for pavement design utilizing the measured values of the basic properties which would be applicable to all locations, environments, and traffic loadings.

Summarized and briefly restated, the objective of the project was ,to formulate the overall pavement problem in broad theoretical terms', which would enable the solution of a variety of pavement problems which have long plagued engineers.

The initial effort of this undertaking was to gather together a group of experienced designers and solicit their assistance in preparing a list of significant basic material properties and their interaction. Early attempts did not prove fruitful and, in fact, it was the consensus that only by looking at the overall area of pavement behavior and performance and then formulating all ideas into a systems engineering approach could this task be accomplished. It also soon became apparent that the basic properties sought could be developed best by the realistic characterization of materials behavior, using the available principles of continued mechanics which have served other engineering disciplines so well. However, more realistic and complete materials characterizations are valuable only if they are significantly better than present empirical test methods. Furthermore, these characterizations are worthwhile only of they can be incorporated into analytical models or boundary value problems which will predict the required responses of the pavement systems. In relating basic materials properties to pavement performance, the following considerations play an important role:

1. Gaining an understanding of how materials truly behave and then developing methods for characterizing them in suitable physical and mathematical terms is an extremely difficult and complicated operation that cannot be separated from the use and function of the pavement, per se. Therefore, in order to approach the material problem in relationship to the total pavement design requirements, it became necessary to adopt systems engineering concepts for use in this study.
2. In turn, material properties cannot be meaningfully utilized unless they can be related to the performance of pavement structures constructed from them. Therefore, in order to incorporate these properties into the investigation, it became necessary to formulate and test various hypotheses on how material properties influence such performance.

In this connection, Chapter II of this paper introduces the systems engineering concept and presents the preliminary development of a pavement structural system description.

2. Applying systems engineering to pavement structural behavior

A pavement is a complex structure which is subjected to many diverse combinations of loading and which must perform under a variety of environments. Because the subjects of material characterization and pavement performance and their interrelationships are so complicated, a coordinated framework for solution of the overall problem of pavement design is needed. Examination of available techniques for analyzing such complex relationships revealed that the concepts of systems engineering (which have evolved in recent years in the electronics, communications, and aerospace industries) would be quite appropriate to the evaluation of pavement structures.

The use of systems engineering does not, per se, develop new and dramatic inputs to the solution of the pavement design problem, but it does provide a means of organizing the various segments of the total problem into an understandable framework. It proved to be necessary for this project, not only as an aid in the overall definition of the problem, but also for pointing out related studies which might ultimately provide needed input for the ultimate solution. To understand the systems engineering approach, it is probably better to talk about the ‚concepts of systems engineering' rather than systems engineering itself.

Ellis and Ludwig (Ref. 2) give a definition for a system which can be applied to highway and pavement structural systems:

> A system is something which accomplishes an operational process; that is, something is operated on in some way to produce something. That which is produced is called output; and that which is operated on is usually input, and the operating entity is called the system. The system is a device, procedure, or scheme which behaves according to some description, its function being to operate on information and/or energy and/or matter in a time reference to yield information and/or energy and/or matter and/or service.

Dommasche and Lauderman (Ref. 3) use the term ‚systems engineering' to describe an integrated approach to the synthesis of entire systems designed to perform various tasks in what is expected to be the most efficient manner. Thus, the term ‚systems engineering' is used to describe an approach which views an entire systems of components as an entity rather than simply as an assembly of individual parts, i.e., a system in which each component is designed to fit properly with the other components rather than to function by itself.

The systems approach emphasizes the ideas and factors which are common to the successful operation of relatively independent parts in an integrated whole. Furthermore, the successful operation of the whole is the primary objective of the system. Individual parts and equipment may not be operating most efficiently at a particular time. However, in the interest of the complete system, their action at the particular time must be compatible with overall systems requirements for the entire period of interest.

The design of a large-scale system is overwhelming if it is attacked all at once, but if the attack is made piecemeal, it is unlikely to be successful. It is necessary to subdivide the problem in a number of ways, both conceptually and organizationally, but in order to do this, it must be possible to formulate the problem as a whole. It is also important in systems engineering to divide the problem into subsystems for analysis and to develop appropriate models, mathematical or physical, for the overall system. Such models are inevitable simplifications of the very complex natural world, but successive iterations in the solution of the model will make it possible to increase the complexity and the acceptability of the model and its solutions. This iterative process is shown, for example, in Figure 3 (see page 118).

Any system has a number of characteristics which can be related to the objectives of the individual subfunctions within the system or which may be objectives of the whole system. These characteristics may be such things as simplicity, ease of maintenance, low cost, long life and/or good performance, all of which may be required either simultaneously or at different times (e.g., asphalt concrete must provide long life or durability at minimum cost). Under these conditions, some compromise is often required (e.g., an increase in asphalt content to increase durability may result in lower strength and lower skid resistance).

In some systems, such as a typical city freeway, emphasis is placed on low-maintenance performance, while cost is considered less significant.

Some other systems, such as farm-to-market roads, are extremely cost sensitive and are less responsive to reliability or other factors. Because of these differences in balance, it is necessary that each system be considered on its own basis and the relative merits of the different objectives be considered in order of importance. Establishing this order is the highway engineer's function.

Applications of the Systems Approach

The system can be considered as a black box. (Figure 1) equipped with a set of accessible terminals and obeying some physical law or set of laws. It is often convenient to separate the quantities that characterize the system into three categories:

1. excitation variables – the external stimuli that influence the systems behavior;

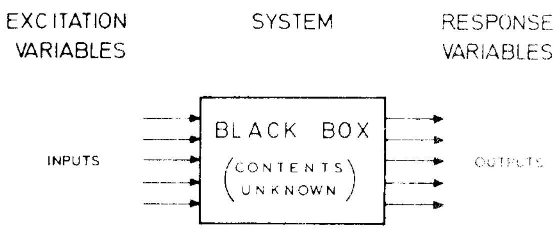

Fig. 1 Black Box System.

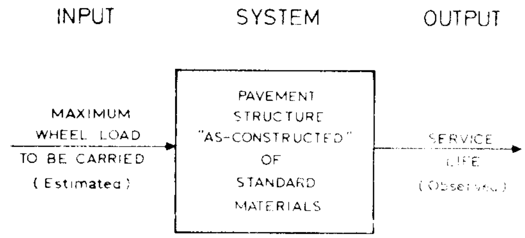

Fig. 2 System Diagram of Early Pavement Design Methods.

2. response variables – those aspects of systems behavior that are of interest to the investigator; and
3. intermediate variables – those which are neither excitation nor response variables.

Rather than referring to the system as a ‚black box', it can be described as a physical object which transforms the input variables or excitation variables to the response or output variables in some still undefined manner.

If a designer could define a pavement system well enough to predict outputs from a given set of inputs with a minimum of complexity, as illustrated in Figure 2, he would be satisfied from an operational point of view. Unfortunately, most of the systems problems facing civil engineers, particularly in transportation engineering, will not yield to solution without some understanding of what is going on inside the system or ‚black box'.

The scientific and engineering aspects of a systems problem usually span a broad spectrum of activities:

1. the use of physical observations to determine the laws governing its behavior;
2. the statement of mathematical models that approximate physical phenomena;
3. the design of a system for prescribed behavior using mathematical models; and
4. the physical realization of a mathematical design.

Thus, it is essential that systems engineers be able to formulate the system in terms of a mathematical or physical model, or failing this, the system must be simulated in some realistic way to observe the necessary outputs.

Another important systems engineerings precept is that a number of alternate methods or designs should be considered and that the method actually used be one that can be shown to meet most adequately the known needs of the system.

Systems Applied to Pavements

Having discussed the generalities of systems analysis, one may now turn to the development of a description of the pavement system. It is often convenient to regard the pavement system as the ,black box' in Figure 1, the contents of which are not completely discernible. The box accepts certain inputs in the form of traffic and environmental variables and responds by developing within its structure a mechanical state which, in the case of a successful design, sustains the input variables over a certain lifetime. The basic design process involves several distinct operations:

1. Appropriate input and response variables must be identified and described quantitatively.
2. Methods of selection of both construction materials and construction techniques must be adopted.
3. Response of the system to all classes of input expected to occur in service must be measured, wither directly in the system itself or in some type of simulated system.
4. Quality of the response or measure of the performance of the system must be judged by an approximate criterion.
5. Modification of the system must be permitted in order to attain as near an optimum condition as possible.

In order to treat quantitatively the ideas described above, it is necessary to define terms and operations more precisely. The input to the system consists of traffic, environment, and maintenance. The effect of traffic is to impress, through wheel loads, certain stresses on the pavement surface. The spatial distribution and time variations (both dynamic and cyclic) are ascribable functions. The environmental input consists of, among other things, diffusion of heat and moisture into the system. Once again, these inputs are characterized as functions of space and time. In certain instances a chemical input may occur, e.g., the use of de-icing salts. The response consists of the generation of a mechanical state identified by deformation and internal stress. For our purposes, the mechanical state is most readily described in terms of stress and strain.

The pavement system itself is characterized by properties of the individual constituents, their arrangement and, to some extent, the method by which the system is constructed. The systems function is defined as the operator which describes the manner in which the pavement accepts an input and converts

it to a response. The systems function is evidently an intrinsic property of the pavement system and may be affected by aging and by the input itself, particularly in the case of ,overloading' input; the environmental input may influence strongly the response to traffic input.
It is well to observe here that for a particular system, it is possible, though perhaps not practical, to look no further into the ,black box'. The alternative would be to carry out a series of experiments in which expected traffic and environmental inputs are fed into the system and the response measured. A number of alternative ,boxes' could be used and their responses compared, and based upon evaluation of these responses, a measure of the performance of the system could be set up. Performance is in some sense a measure of the quality of the response, e.g., whether or not breakdown (i.e., distress) of the system results during the response or whether excessive permanent deformation occurs and, furthermore, whether or not good performance is attained for reasonable cost, both initial and maintenance. Evidently, an objective measure of performance will involve concepts of mechanical and economic life of the system. In order to obtain an optimum systems design, it is necessary to alter the structure of the system until a maximum mechanical-economic life is achieved for a given range of inputs. It appears that some ,road tests' and ,satellite studies' fall into this class of black box experiment. The principal disadvantage of the type of experiment described above is that it is not predictive; that is, changes of input variables or changes in the systems function falling outside of the range covered in the experiment must be examined by extrapolation rather than interpolation. Furthermore, the large number of variables involved in the system (input, response, and systems function) magnifies the experimental task enormously. Consequently, it is highly desirable to place as much as possible of the system description on a rational basis so that simulation of the operation of the system can be effected, and design optimization studies can be carried out on these simulated systems prior to validation in the field. For this reason, system formulation is the next step.

Phase Development in a System

Any system develops in a series of phases, which repeat themselves as they succeed one another. In the first trial, the general outline of the system and one significant estimate of its performance can be drawn up or developed by engineers skilled in the state-of-the-art using rules of thumb for many of the input parameters and omitting many others. Figure 3 is an example of a simple system diagram of early pavement design methods.
The pavement engineer observes the performance of these pavements and repeats the construction of those which perform well. Those designs which perform poorly are either discontinued or modified for future use. In succes-

sive phases, the design is refined in greater detail with the evaluation of performance and the design of interconnections in the system being carried on with greater specificity. Such has certainly been the case in the development of the design of pavement structural systems.

Figure 3 shows a block diagram of the evolution of many existing pavement design techniques. These have evolved primarily through observations of pavement behavior and their use to modify materials specifications and testing procedures, as shown in the figure. The resulting methods are primarily empirical, although the designs themselves may be expressed as equations and the materials test values are sometimes related to a mathematical theory (e.g., Young's modulus of elasticity).

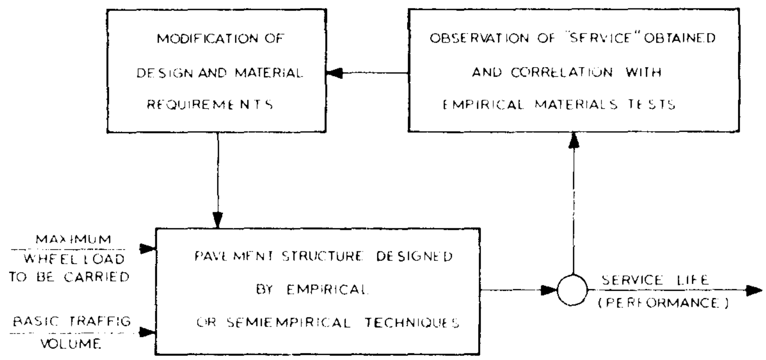

Fig. 3 Block Diagram of Some Current Pavement Design Techniques.

Formulation of the Pavement System

A great deal of work remains to be done before a truly realistic description of the pavement system can be formulated. More must be known about the relationships and interactions of various classes of input variables. It will be mandatory that some type of mathematical model or transfer function be developed to describe the relationships in the system, and yet it is possible through observations of pavement behavior and knowledge of theory to begin more realistic formulations of the pavement system, as shown in Figure 4. This chart is not intended to present an exhaustive development of the details of such a system; it is instead an attempt to interrelate many of the factors involved in the design of a pavement system.

The important aspects of the system description include its inputs, physical character, response, output, and decision criteria.

The inputs to the system include a variety of load, environmental, construction, and maintenance variables. These are not independent variables, but affect each other as indicated by the interactions shown. These variables are stochastic in nature and are difficult to specify and predict.

Physical characteristics of the system include among other things geometric measurements such as thickness and arrangements, and the basic properties which characterize the material behavior.

Systems response involves the behavior of the physical system when subjected to inputs such as load or temperature. These are usually measurable and involve the mechanical state, such as deflection, stress, and strain. When these so-called primary responses reach some limiting value, some type of distress occurs in the form of rupture, distortion, or disintegration. The output of the system is measured by the goods and people (the load applications) actually transported. Chapter III discusses the combination of these factors in a systems performance or output function which can be used as a measure of system adequacy.

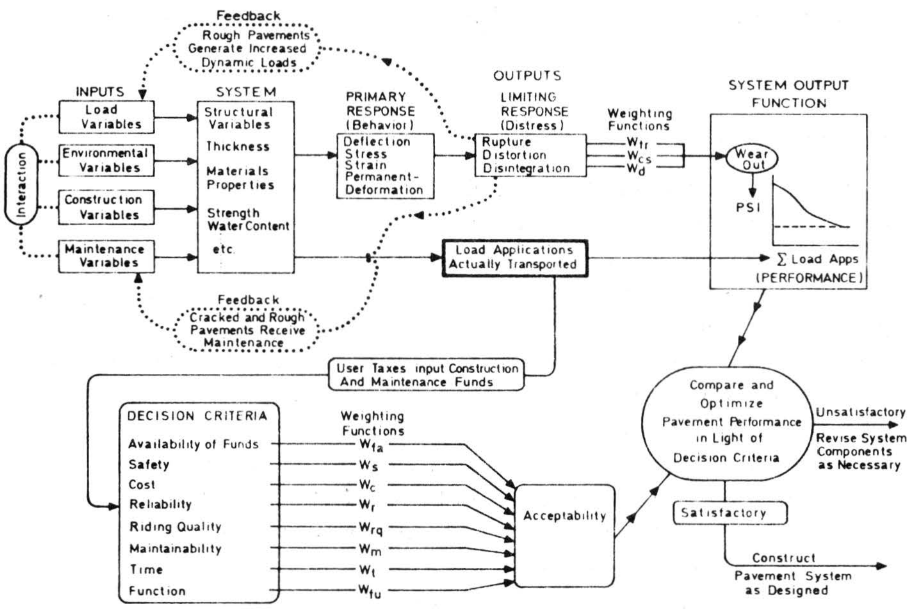

Fig. 4 Block Diagram of the Pavement System (After Ref. 23).

Decision criteria are also essential in systems formulation, involving a variety of factors such as funding, cost, reliability, and riding quality. These must be combined in an appropriate way to select the proper level of acceptability for a particular purpose. This level of acceptability then provides a basis for comparing and optimizing the system output or pavement performance. These factors are more completely discussed in Chapter III.

Feedback and interaction are important parts of this and any system, but they are hard to quantify and relate mathematically. Much remains to be done with these factors, but the systems approach provides the necessary framework. The illustrations in Figure 4 indicate, for example, that as the pavement deteriorates, it gets rough and generates increased maintenance costs and increased dynamic loads.

It is useful to show the interrelationships of the system graphically as in Figure 4. If proper progress is to be made toward an adequate solution of the problem, however, it is necessary to develop some type of mathematical model or transfer function to describe the relationships in the system. This would allow electronic computers to be used in making the decisions involved without bias. These models will be complex because they must ultimately be stochastic to provide some adequate simulation of the real pavement system. More specific decision criteria must also be developed for use in the process.

The need for these improved methods of pavement systems evaluation will be intensified as traffic demands grow, as costs increase, and as the complexity and variety of materials used in pavement construction continue to multiply.

3. Pavement behavior and performance

Examination of the pavement system diagrammed in Figure 4 illustrates the complex interrelationships which necessarily exist between the following:

1. materials comprising the system
2. manifestations of pavement behavior, and
3. pavement performance.

This chapter will define terms and establish concepts for relating these factors for use in the evaluation and design of pavement systems.

Pavement behavior will first be considered in terms of pavement performance and failure. These will then be discussed for the purpose of conceptually quantifying the factors included in the block diagram of the pavement system of Figure 3. The top part of the figure can be quantified in terms of a ‚distress index', and the lower part by a ‚decision criteria index'. The level of funding in research to date has not permitted development of specific working equations; hopefully, such equations will be forthcoming in subsequent work.

Since the output function is defined in terms of performance and since performance as well as distress mechanisms associated with it have a variety of connotations, a series of definitions are presented below to insure a uniform basis for the ensuing discussion. The definitions have been selected for clarity in this presentation and are generally based on concepts developed by Carey and Irick (Ref. 4) for evaluating the performance of the various pavements in the AASHO Road Test. Inherent in the definitions and the development of the equations for the system is the purpose of the highway facility, e.g., to provide a safe, comfortable, and economical method of transporting goods and people.

Definitions of Terms

1. Performance is a measure of the accumulated service provided by a facility; i.e., the adequacy with which a pavement fulfills its purpose. Performance is often specified with a performance index as suggested by Carey and Irick (Ref. 4). As such, it is a direct function of the present serviceability history of the pavement.
2. Present serviceability is the ability of a specific section of pavement to serve high-speed, high-volume, mixed (truck and automobile) traffic in its existing condition. (Note that the definition applies to the existing condition – that is, on the date of rating – not to the assumed condition the next day or at any future or past date).
3. Behavior is the reaction or response of a pavement to load, environment, and other inputs. Such response is usually a function of the mechanical state (i.e., the stress, strain, or deflection which occurs in response to the input.
4. Distress mechanisms are those responses which can lead to some form of distress when carried to a limit (e.g., deflection under load is a mechanism which can lead to fracture). Some behavioral responses may not provide distress mechanisms.
5. Distress manifestations are the visible consequences of various mechanisms of distress which usually lead to a reduction in serviceability.
6. Fracture is the state of being broken apart, a cleavage of the member or material including all types of cracking, spalling, and slippage.
7. Distortion is a change of the pavement or pavement component from its original shape or condition. Such changes are permanent or semi-permanent as opposed to transient, such as deflections.
8. Disintegration is the state of being decomposed or abraded into constitutive elements (i.e., stripping, raveling, scaling, etc.).

Pavement Behavior

It would be desirable to define or list the various manifestations of pavement distress which typically occur and to relate these manifestations through behavior to material properties. The rationale to such an approach would be for instance, to relate specific values of measurable material properties to the specific distress symptoms observed in such a way as to able to predict the potential for distress occurring. By design and specification, therefore, the distress can be minimized.

The factors affecting pavement structural behavior have been defined and characterized over the years in different ways by various individuals and groups (Refs. 4, 5, 6, 7, 8, 9, 10, 11, 12). While reasons for these characterizations may vary, it appears that the basic motive in all cases has been to provide guidelines for design or evaluation. Such descriptions of pavement structural behavior have usually been formulated by defining either factors which affect pavement performance or factors which affect failure of the pavement structure. A survey of the literature, however, indicates that there are no clear-cut and generally accepted definitions of failure which relate to some level of serviceability or performance; nor is there a complete set of well-defined and generally accepted failure mechanisms for the pavement components.

In this study an attempt has been made to associate material properties with modes of failure or distress through considerations of the various mechanisms and manifestations of distress. As seen in Figure 5, limiting response (i.e., distress) modes have been divided into three categories:

1. fracture,
2. distortion, and
3. disintegration.

With the exception of pavement slipperiness associated with the surface coefficient of friction, all forms of pavement distress can be related individually or collectively to these modes. Also shown in this table are the manifestations of each mode of distress, together with a listing of the mechanisms associated with each manifestation of failure. While the next logical step would be to list the pertinent material properties for each of the failure mechanisms noted, this has not been done herein because time and space do not allow.

4. Summary

On the basis of the findings resulting from a wide scope of investigations covering the broad question of developing a rational method of pavement design as well as narrow searches for approaches and solutions to specific problems, the following major conclusions are drawn:

1. The task of developing a systematic approach to the analysis of pavement structural systems is enormous in both magnitude and complexity. Only a concentrated, coordinated effort employing a *systems engineering* approach will provide the results needed to describe adequately the overall behavior of a pavement system.
2. The concepts of *systems engineering* must be applied not only to the total pavement system, but also to the various subsystems, including but not limited to:
    a. Materials characterization.

Distress Mode	Distress Manifestation	Examples of Distress Mechanism [1]
Fracture	Cracking	Excessive loading Repeated loading (i.e., fatigue) Thermal changes Moisture changes Slippage (horizontal forces) Shrinkage
	Spalling	Excessive loading Repeated loading (i.e., fatiugue) Thermal changes Moisture changes
Distortion	Permanent deformation	Excessive loading Time-dependent deformation (e.g., creep) Densification (i.e., compaction Consolidation Swelling
	Faulting	Excessive loading Densification (i.e., compaction) Consolidation Swelling
Disintegration	Stripping	Adhesion (i.e., loss of bond) Chemical reactivity Abrasion by traffic
	Raveling and scaling	Adhesion (i.e., loss of bond) Chemical reactivity Abrasion by traffic Degradation of aggregate Durability of binder

[1] Not intended to be a complete listing of all possible distress mechanisms.

Fig. 5 Categories of Pavement Distress.

b. Computation of the mechanical state (i.e., stress and strain) within the pavement in terms of load and environment (i.e., primary response).
c. Systems output function (i.e., performance).
d. Decision criteria for judging acceptability and optimization of design.

The problem of ‚designing' pavements is really more precisely a pavement management problem as outlined by Haas (Ref. 24) Scrivner, et al (Ref. 25) and Hudson, et al (Ref. 23, 27). While the information presented here does not provide a direct solution to the problem, it does provide the necessary framework for moving ahead as outlined in the following paper (Ref. 28).

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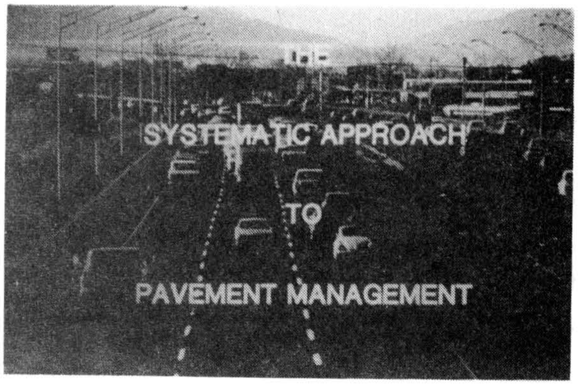
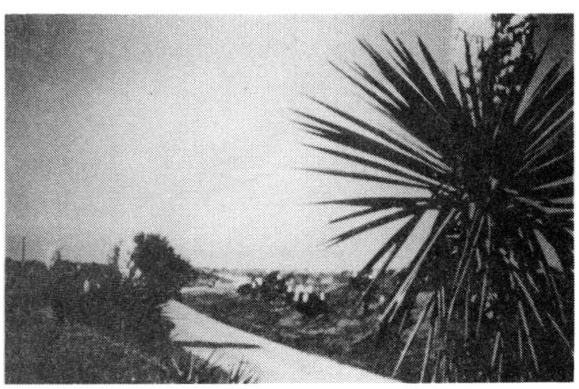
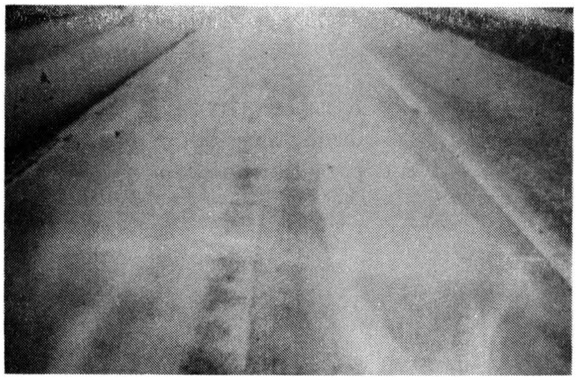


Slide 4.1. Systematic Approach to Pavement Management (Opening Slide).

Slide 4.2. A PMS can be applied to pavements constructed in cold regions.

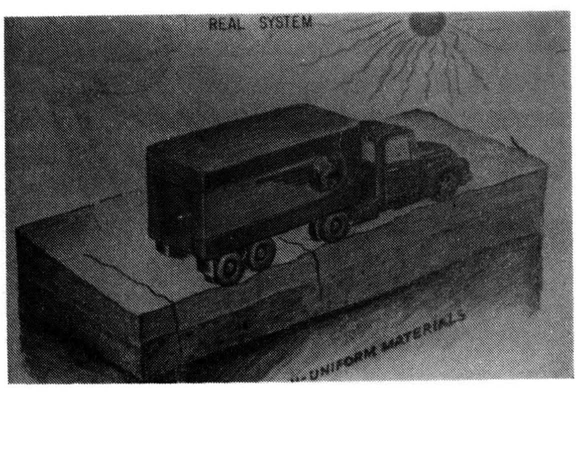


Slide 4.3. A PMS should also be capable of handling pavements constructed in tropical environments.

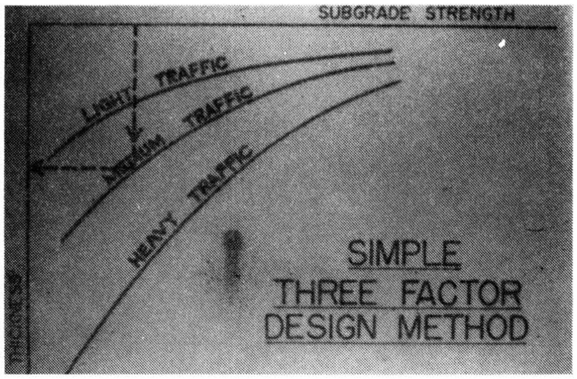


Slide 4.4. A PMS should include paved roads.

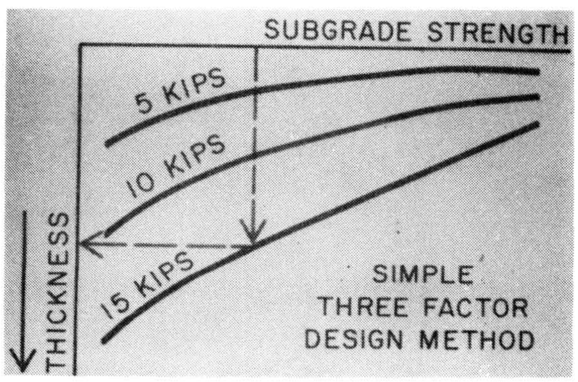
Slide 4.5. Unpaved roads should also be considered in a PMS.



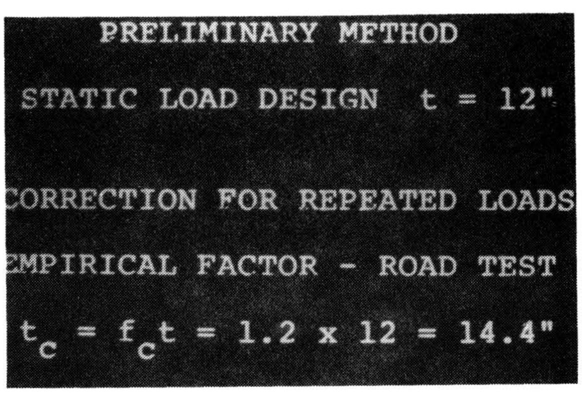
Slide 4.6. Effect of various factors on the behavior and performance of a pavement structure.



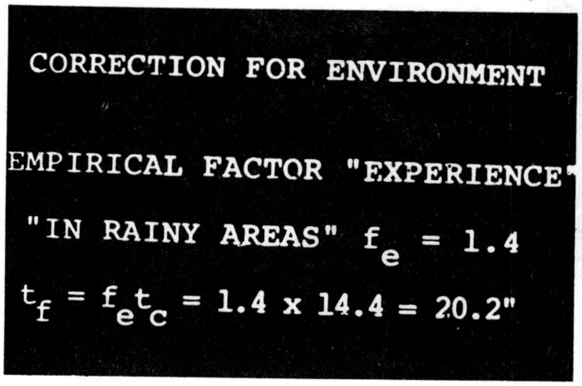
Slide 4.7. Simple three factor design method for pavements (an example).



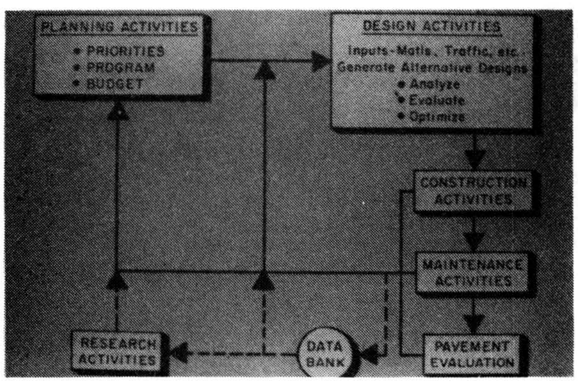
Slide 4.8. Simple three factor design method for pavements (an example).



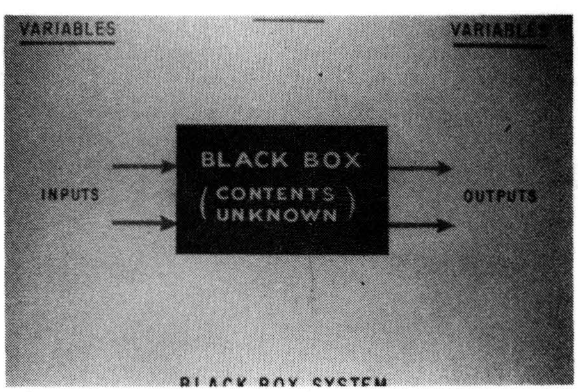
Slide 4.9. Pavement Design using correction for repeated load (AASHO Road Test).



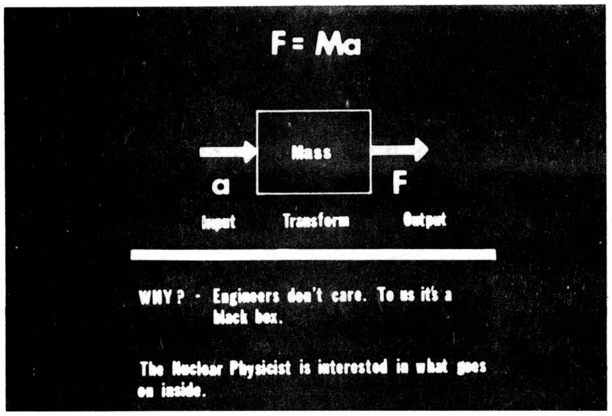
Slide 4.10. Use of correction factor for environment in designed thickness.



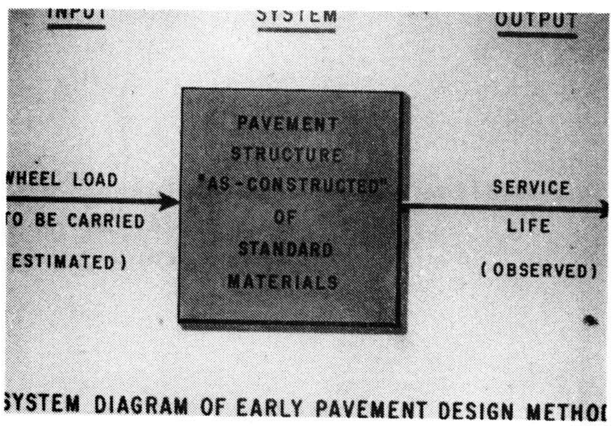
Slide 4.11. Major classes of activities in pavement management.



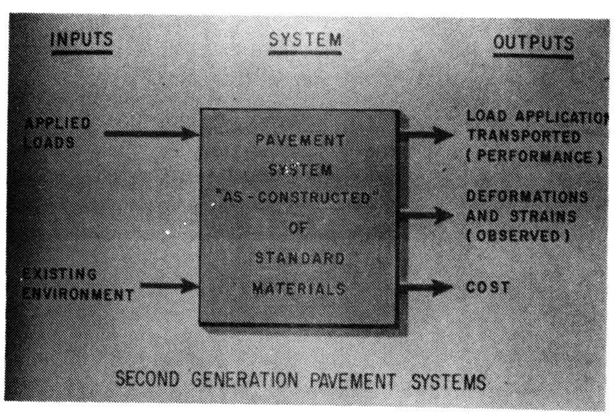
Slide 4.12. Use of system's approach in pavement design.



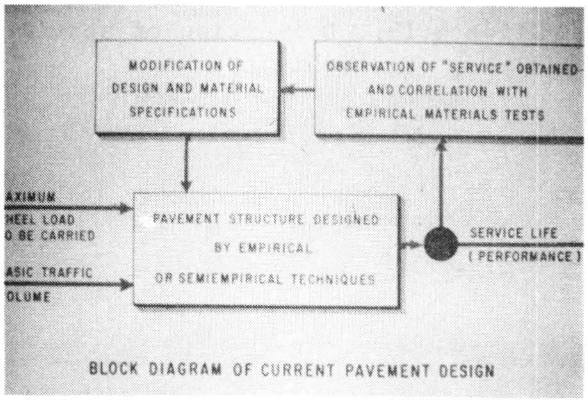
Slide 4.13. Example of the "black-box" concept.



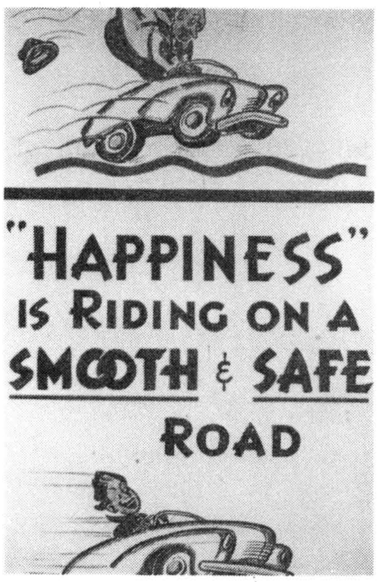
Slide 4.14. System diagram of early pavement design method.



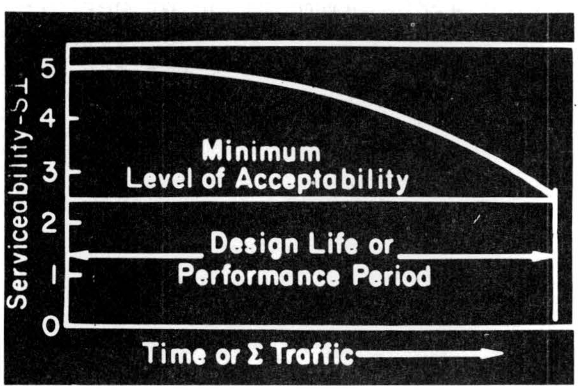
Slide 4.15. Second generation pavement system.



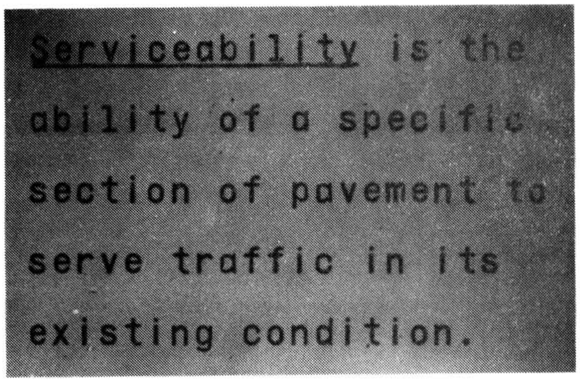
Slide 4.16. Block diagram of current pavement design.



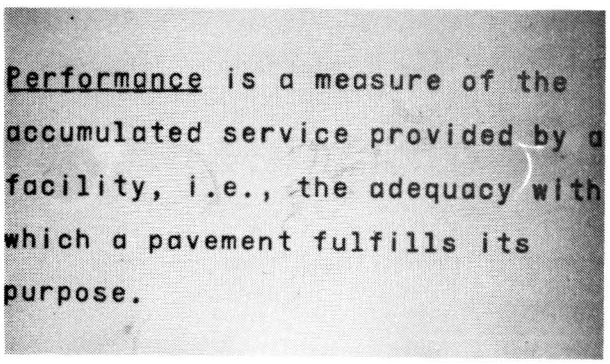
Slide 4.17. Importance of having smooth roads.



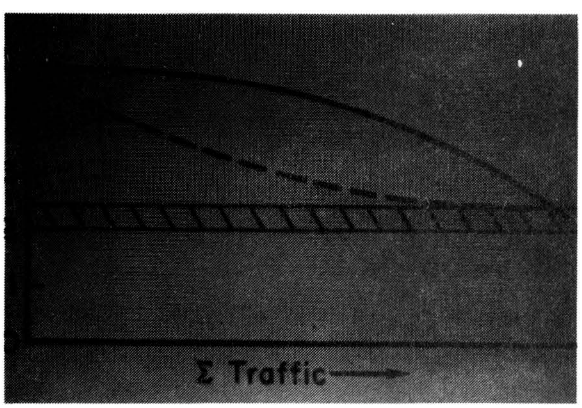
Slide 4.18. Plot of PSI versus time or load applications.



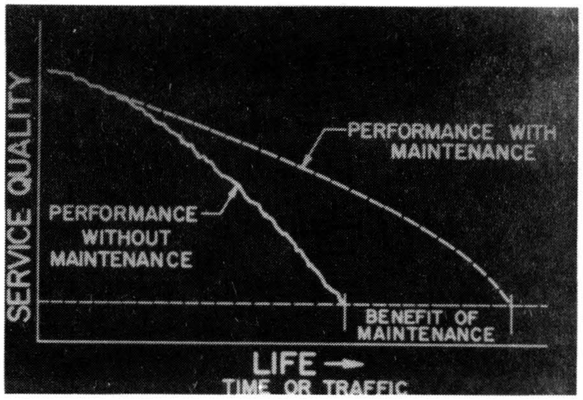
Slide 4.19. Definition of serviceability.



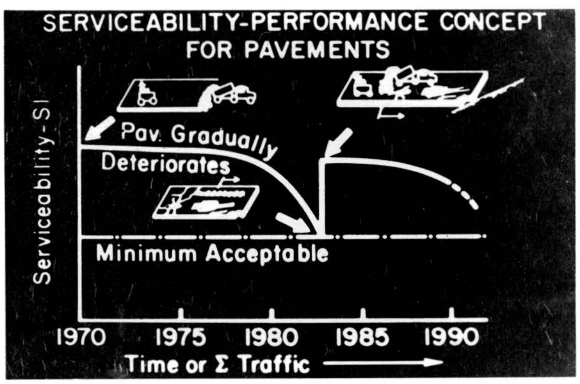
Slide 4.20. Definition of performance.



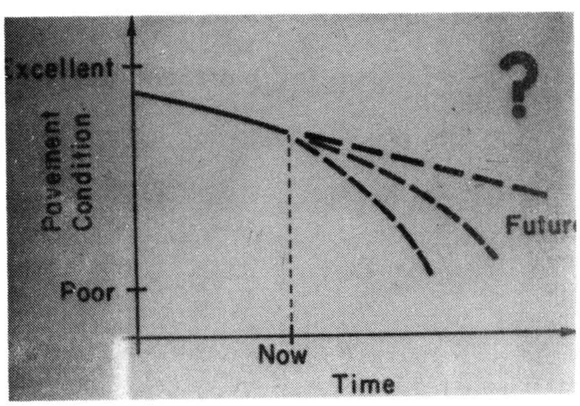
Slide 4.21. Comparison of performance of two different pavements for the same design life.



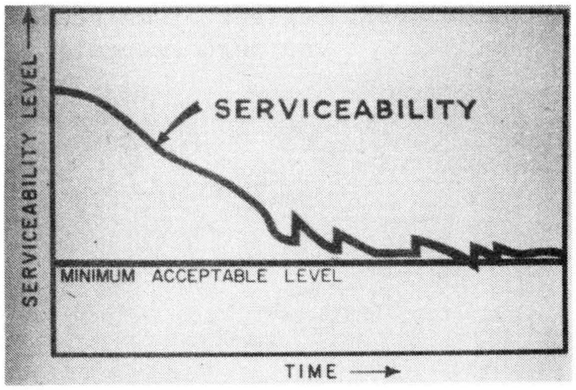
Slide 4.22. Effect of maintenance on performance.



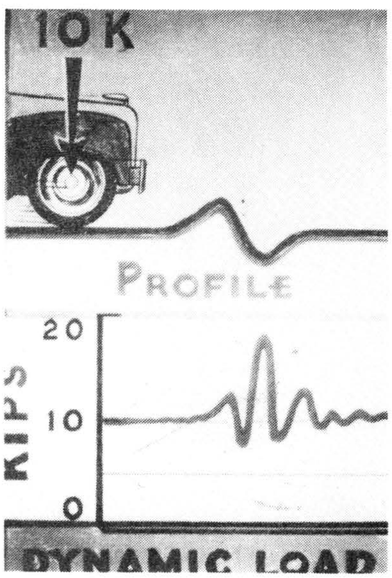
Slide 4.23. Effect of an overlay on PSI.



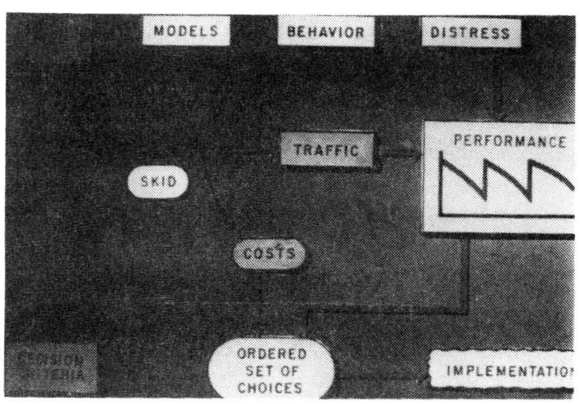
Slide 4.24. Determination of the best maintenance and rehabilitation strategy.



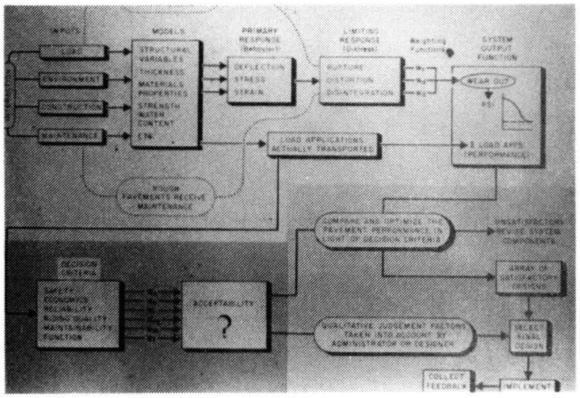
Slide 4.25. Example of an actual plot of PSI versus time.



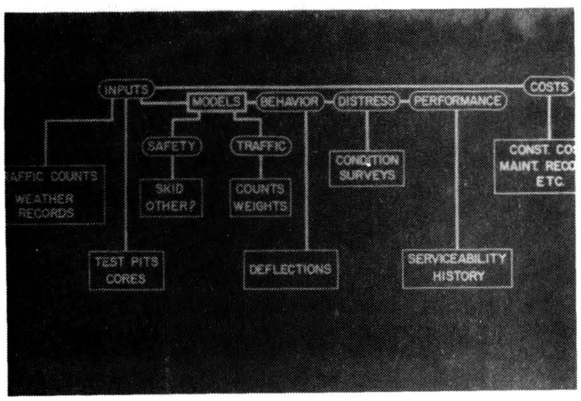
Slide 4.26. The dynamic force generated by a vehicle is related to the pavement profile.



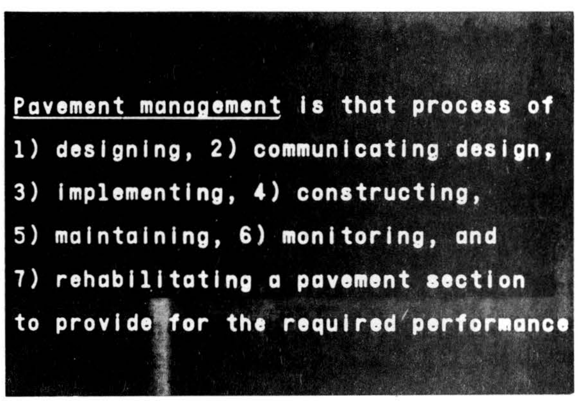
Slide 4.27. Simplified block diagram of the major components of pavement design.



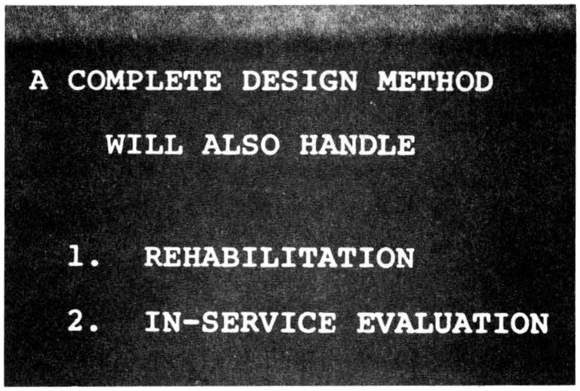
Slide 4.28. Block diagram of the PMS.



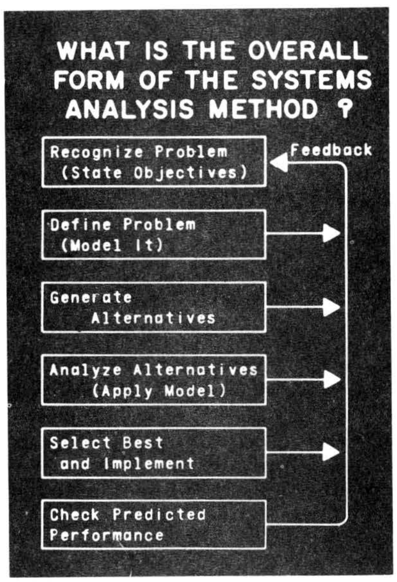
Slide 4.29. Simplified predictive portion of pavement design and related examples of types of periodic evaluation measurements.



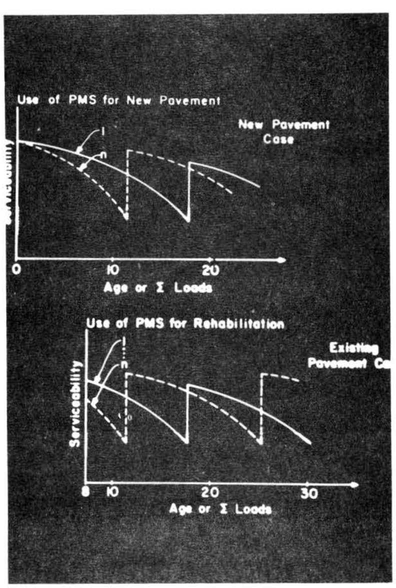
Slide 4.30. Definition of Pavement Management.



Slide 4.31. Extra requirements of a complete design method.

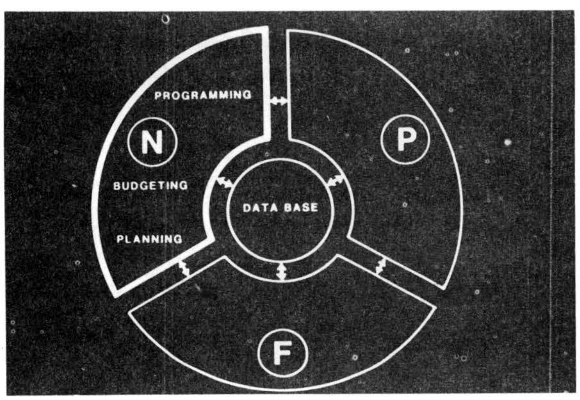


Slide 4.32. Overall form of the systems analysis method.

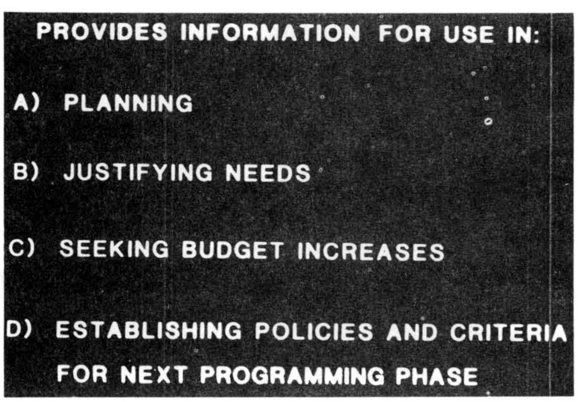


Slide 4.33. Use of a PMS for both a new and an existing pavement.

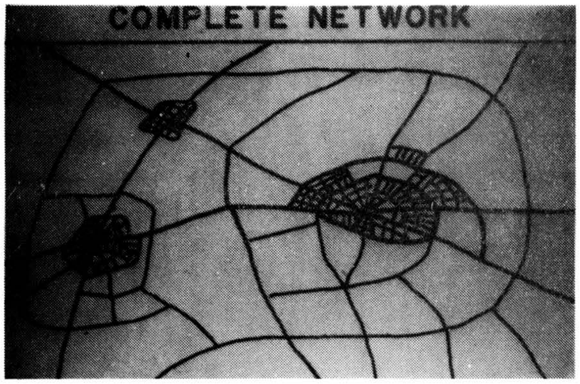
Slide 4.34. Location and identification of rehabilition needs in a pavement network.



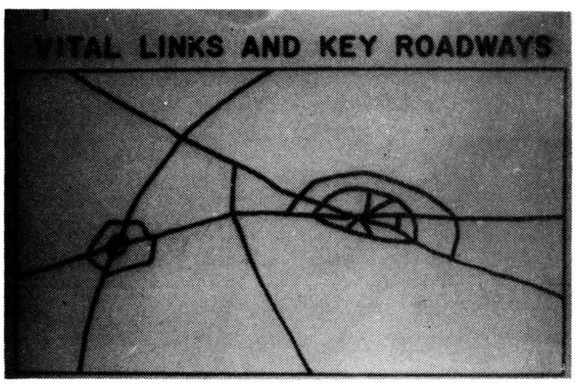
Slide 4.35. Major activities of a PMS.



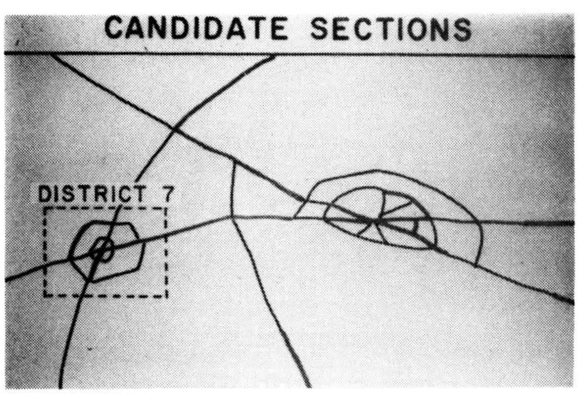
Slide 4.36. Uses of a PMS at the network level.



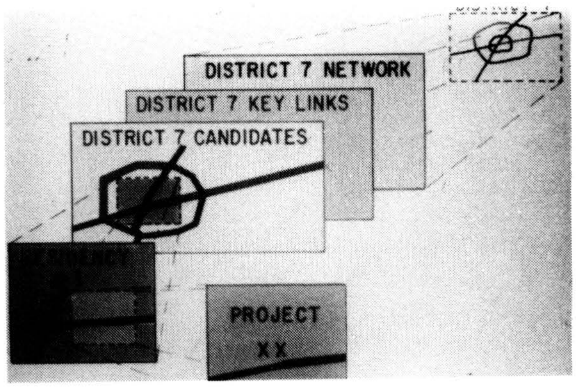
Slide 4.37. Example of a pavement network.



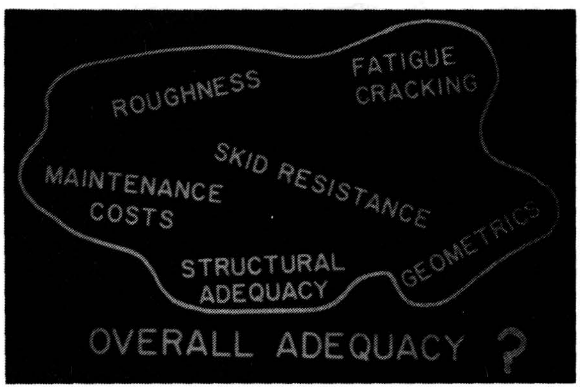
Slide 4.38. Vital links and key roadways.



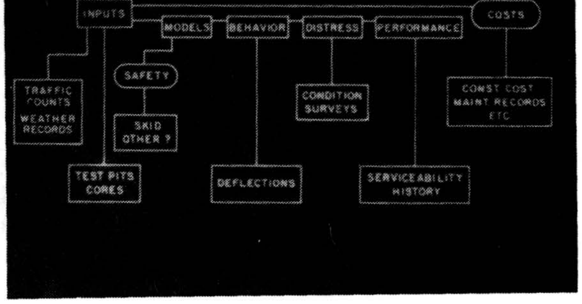
Slide 4.39. Selection of candidate sections.



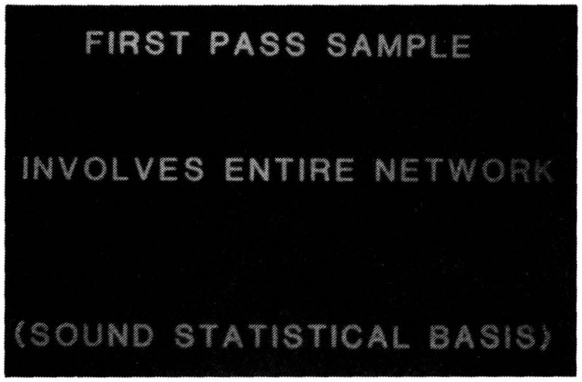
Slide 4.40. Prioritization of rehabilitation needs in a pavement network.



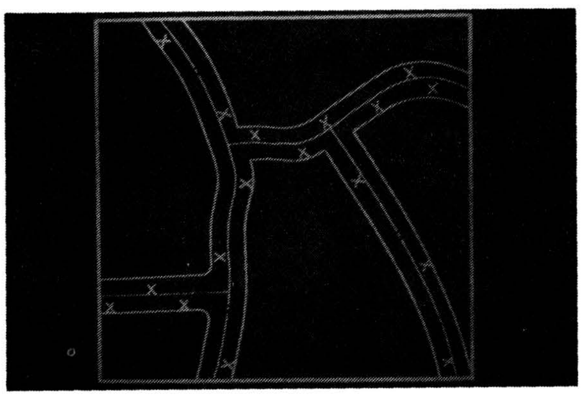
Slide 4.41. Factors considered in the prioritization process.



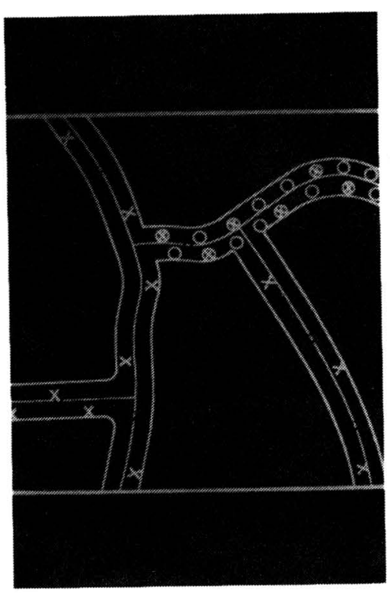
Slide 4.42. Simplified predictive portion of pavement design and related examples of types of periodic evaluation.



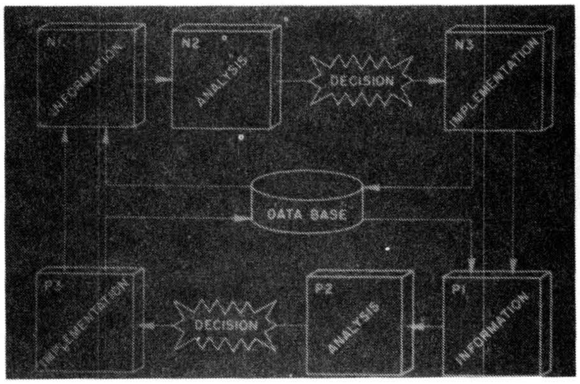
Slide 4.43. First stage of the prioritization process.



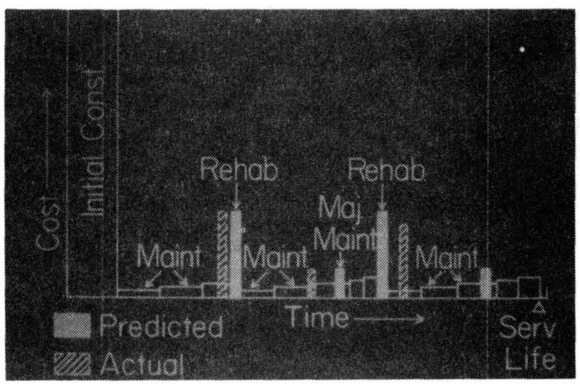
Slide 4.44. Preliminary collection of information on selected pavement sections.



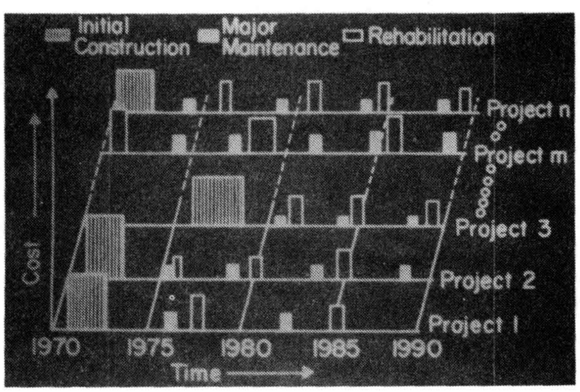
Slide 4.45. More information is gathered for those candidate sections selected after the first stage of the prioritization process.



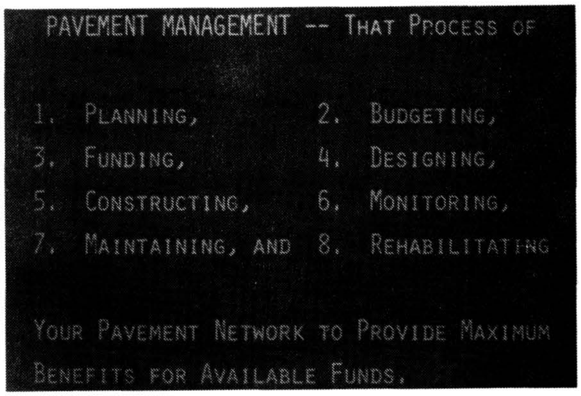
Slide 4.46. Different activities necessary to arrive at a decision as to the test maintenance and rehabilitation strategies in a PMS.



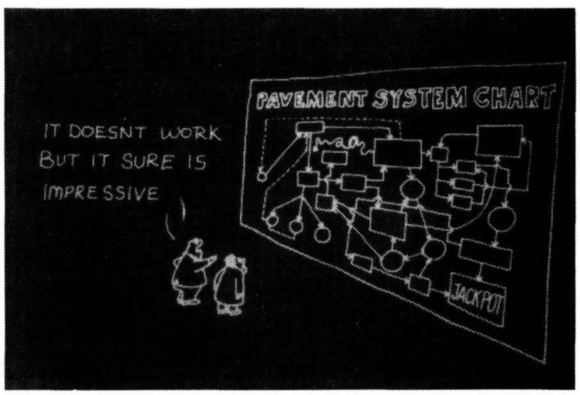
Slide 4.47. Example of predicted versus actual rehabilitation cost for a given pavement section.



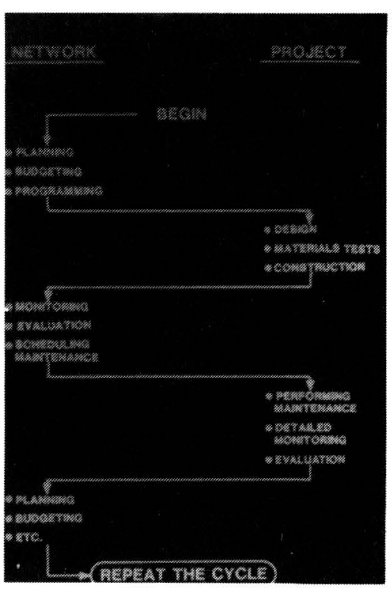
Slide 4.48. Cost information collected in a project basis for a given pavement network.



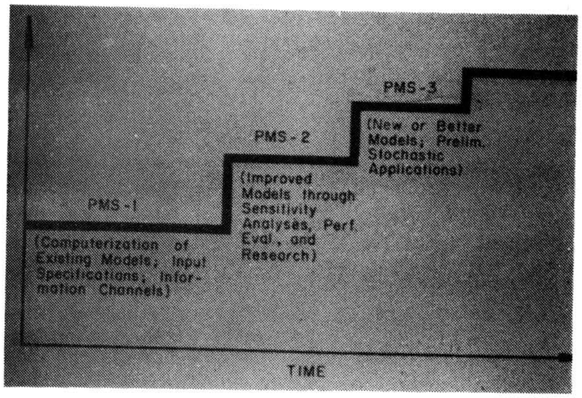
Slide 4.49. Definition of a PMS.



Slide 4.50. Complexity of a PMS.



Slide 4.51. Step-by-step improvements in the development of a PMS.



Slide 4.52. Relationship between the activities at the network level and those at the project level in a PMS.

LESSON OUTLINE
MONITORING AND EVALUATION PAVEMENT SERVICEABILITY - PERFORMANCE CONCEPTS

Instructional Objective

1. To provide the student with a basic concept of pavement performance.

2. To present various approaches for predicting pavement performance.

Performance Objectives

1. The student should be able to explain the pavement performance concept.

2. The student should be able to explain various approaches for predicting pavement performance.

Abbreviated Summary	Time Allocation, Min.
1. Pavement Performance Concepts	20
2. Evaluation of Pavement Performance	15
3. Prediction of Pavement Performance	15
	50 minutes

Reading Assignment

1. Haas & Hudson - Chapter 6 and 7

2. HRB Bulletin 250

LESSON OUTLINE
MONITORING AND EVALUATION PAVEMENT SERVICEABILITY - PERFORMANCE CONCEPTS

1.0 MONITORING AND EVALUATION - WHAT IS IT? HOW DO WE MEASURE IT? (Slides 5.1 - 5.12)

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 encompasses monitoring, but involves a judgment or determination of the meaning of the information collected.

2.0 DEFINITIONS (Slides 5.13 - 5.20)

2.1 Serviceability

The ability of a specific section of pavement to serve traffic in its existing condition.

2.2 Performance

A measure of the accumulated service provided by a facilty, i.e., the adequacy with which a pavement fulfills its purpose. 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.

2.3 Behavior

The immediate response of the pavement to load.

2.4 Distress

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 evidence of this maintenance in the form of patches and sealed areas should also be monitored.

3.0 MEASUREMENT

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; and
(5) provide information for updating network improvement programs.

3.1 Safety

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 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.

3.2 Structural Capacity

3.2.1 Direct Measurement. Physical structure and material strength can be monitored by physical testing and sampling; i.e., coring and laboratory testing.

3.2.2 Indirect Measurement. 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. This, 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.

3.3 Distress

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 are measured will vary from agency to agency.

3.4 Maintenance Costs

Costs 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.

3.5 Riding Comfort

The evaluation of riding quality is a complex problem, depending on three separate components:

(a) the pavement user;
(b) the vehicle and the pavement roughness, and
(c) interactions among the first two.

4.0 SERVICEABILITY (Slides 5.21 - 5.40)

The primary operating characteristic of a pavement is the level of service it provides to the users, both today and in the future. It is important to (1) measure or evaluate this level of service to establish the current status of a pavement, and (2) to predict the change of level of service in the future, for either an existing pavement or for a pavement to be constructed.

4.1 AASHO Road Test

Until a measure of pavement serviceability was developed in conjunction with the AASHO Road Test, little attention was paid to evaluation of pavement performance per se. A pavement was either satisfactory or unsatisfactory (i.e., in need of repair or replacement). The ideas of "relative" performance were not adequately developed.

4.2 Serviceability as a Design Function

Many popular design systems involve determination of the pavement thickness required to hold certain computed stresses or strains below some specified levels. It is clear that cracks will occur if the pavement is overstressed, but not much information was available prior to the time of the AASHO Road Test to relate such cracks to functional behavior.

4.3 User Relationship

Serviceability must be defined relative to the purpose for which the pavement is constructed, that is, to give a smooth, comfortable, and safe ride. In other words the measurement should relate explicitly to the user, who is influenced by several attributes of the pavement.

INSTRUCTIONAL TEXT A

THE PAVEMENT SERVICEABILITY - PERFORMANCE CONCEPT

W. N. Carey and P. E. Irick

HIGHWAY RESEARCH BOARD
Bulletin 250
1960
Washington, D. C.

THE PAVEMENT SERVICEABILITY—PERFORMANCE CONCEPT*

The relative performance of various pavements is a function of their relative ability to serve traffic over a period of time. There have been no widely accepted definitions of performance that could be used in the evaluation of various pavements or that could be considered in the design of pavements. In fact, design systems in general use in highway departments do not include consideration of the level of performance desired. Design engineers vary widely in their concepts of desirable performance. By way of example, two designers are given the task of designing a pavement of certain materials for certain traffic and environment for 20 years. The first might consider his job to be properly done if not a single crack occurred in 20 years while the second might be satisfied if the last truck that was able to get over the pavement made its trip 20 years from the date of construction. There is nothing in existing design manuals to suggest that either man is wrong. This is simply to demonstrate that any design system should include consideration of the level of serviceability to traffic that must be maintained over the life of the road. How long must it remain smooth and how smooth?

One popular design system involves the determination of the thickness of slab required in order to hold certain computed stresses below a certain level. It is clear that cracks will occur if a pavement is overstressed, but nowhere can be found any reference to the effect of such cracks on the serviceability of the pavement. Engineers will agree that cracks are undesirable, and that they require maintenance, but the degree of undesirability seems to have been left dimensionless. It may be apparent that one pavement has performed its function of serving traffic better than another, but a rational answer to the question, "How much better?" has not been available.

To provide dimensions for the term "performance" a system has been devised that is rational and free from the likelihood of bias due to the strong personal opinions of groups or individuals. It is easily conceivable that such a system could be adopted by all departments thus providing for the first time a national standard system for rating highways and pavements.

Before discussing the derivation and a particular application of the pavement serviceability-performance system, it is necessary to set down some fundamental assumptions upon which the system is based.

1. There is a statement attributed to D. C. Greer, State Highway Engineer of Texas: "Highways are for the comfort and convenience of the traveling public." A reasonable inference from this simple statement is that the only valid reason for any road or highway is to serve the highway users. Another opinion is that "a good highway is one that is safe and smooth."

2. The opinion of a user as to how he is being served by a highway is by-and-large subjective. There is no instrument that can be plugged into a highway to tell in objective units how well it is serving the users. The measurement of damage to goods attributed to rough roads may provide an exception to this rule but one of minor importance since a road rough enough to damage properly packed and properly suspended goods would be classed subjectively so low by all users that little could be gained by an objective measure.

3. There are, however, characteristics of highways that can be measured objectively which, when properly weighted and combined, are in fact related to the users subjective evaluation of the ability of the highway to serve him.

4. The serviceability of a given highway may be expressed by the mean evaluation given it by all highway users. There are honest differences of opinion even among experts making subjective evaluations of almost anything. Thus there are differences of opinion as to which automobile in a given price range is best, differences among judges of a beauty contest, and differences as to which bank, broker, grocery store, or bar to patronize. Opinion as to the serviceability of highways is no exception. Economic considerations alone cannot explain these differences.

Therefore, in order for normal differences of opinion to be allowed with the smallest average error for each individual highway user, serviceability, may be expressed in terms of the mean evaluation of all users.

5. Performance is assumed to be reflected by the serviceability trend of a pavement with increasing number of axle load applications. It is assumed that the performance of a pave-

* An adaption of a paper given at the 39th Annual Meeting of the Highway Research Board.

ment can be described if one can observe its serviceability from the time it was built to the time its performance evaluation is desired and can plot this serviceability record against the traffic the pavement has served. The traffic history must include the number of axle loads and their magnitude sustained by the pavement.

USE OF THE SERVICEABILITY-PERFORMANCE SYSTEM

A typical example of the system which has been in actual field use at the AASHO Road Test, is described in this section. Definitions and detailed steps in the development and use of a performance index for evaluation of the Road Test pavements are included. It is emphasized that this case is only one of many possible applications of the principles involved. It related to the performance of the pavements only, yet it would have been easy to extend the system to provide a measure of the sufficiency of the entire highway, including grade, alignment, access, condition of shoulders, and drainage, as well as characteristics of the pavement itself.

Purpose

The principal objective for the AASHO Road Test calls for significant relationships between performance under specified traffic and the design of the structure of certain pavements. To fulfill this objective an adequate and unambiguous definition of pavement performance was required. None was available.

Special Considerations

In addition to the four primary assumptions, certain special considerations relating to the specific requirements of the Road Test were included. Inasmuch as the project was designed to provide information relating to the pavement structure only, certain aspects of normal pavement serviceability were excluded from consideration. Among these were surface friction and condition of shoulders.

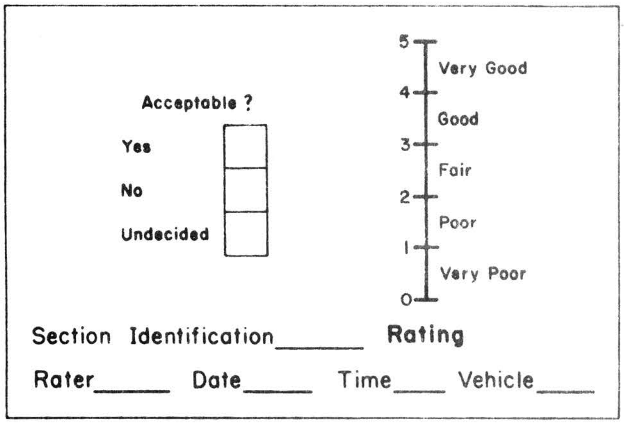


Figure 1–F. Individual present serviceability rating form.

Test sections at the Road Test were as short as 100 ft—too short for a satisfactory subjective evaluation of their ability to serve traffic (most highway users consider a high-speed ride over a pavement necessary before they will rate it). Thus, objective measurements that could be made on the short sections had to be selected and used in such a way that pavements only 100 ft long could be evaluated as though they were much longer.

Definitions

To fulfill the requirements of the Road Test rather ordinary terms were given specific definitions as follows:

Present Serviceability—the ability of a specific section of pavement to serve high-speed, high volume, mixed (truck and automobile) traffic in its existing condition. (The definition applies to the existing condition; that is, on the date of rating, not to the assumed condition the next day or at any future or past date.) Although this definition applies to the Road Test and may apply to any primary highway system, the system could easily be modified for use with city streets, farm roads, etc. Obviously, serviceability must be defined relative to the intended use of the road.

Individual Present Serviceability Rating—an independent rating by an individual of the present serviceability of a specific section of roadway made by marking the appropriate point on a scale on a special form (Fig. 1–F). This form also includes provision for the rater to indicate whether or not the pavement being rated is acceptable as a primary highway. For the Road Test application, the rater was instructed to exclude from consideration all features not related to the pavement itself, such as right-of-way width, grade, alignment, and shoulder and ditch condition.

Present Serviceability Rating (PSR) —the mean of the individual ratings made by the members of a specific panel of men selected for the purpose by the Highway Research Board. This panel was intended to represent all highway users. It included experienced men, long associated with highways, representing a wide variety of interests, such as highway administration, highway maintenance, a federal highway agency, highway materials supply (cement and asphalt), trucking, highway education, automotive manufacture, highway design, and highway research.

Present Serviceability Index (PSI) —a mathematical combination of values obtained from certain physical measurements of a large number of pavements so formulated as to predict the PSR for those pavements within prescribed limits.

Serviceability Trend.—a continuous graph of serviceability plotted against axle load applications.

Performance—the serviceability trend of a section of pavement with increasing number of axle load applications.

Formulation of a Present Serviceability Index

A minimum program for the establishment, derivation and validation of a PSI (or any similar index that may be considered for another purpose) is as follows:

1. Establishment of Definitions—There must be clear understanding and agreement among all those involved in rating and in formulation and use of the index as to the precise meanings of the terms used. Exactly what is to be rated, what should be included, and what excluded from consideration?

2. Establishment of Rating Panel—Because the system depends primarily on the subjective ratings of individuals, great care should be taken in the selection of the persons composing the rating group. Inasmuch as serviceability is defined as the mean opinion of this group, it is important that the raters represent highway users, and they should be selected from various segments with divergent views and attitudes.

3. Orientation and Training of Rating Panel—The members of the panel are instructed in the part they are to play; they must understand clearly the pertinent definitions and the rules of the game. It has been found worthwhile to conduct practice rating sessions where the raters can discuss their ratings among themselves. When they make their official ratings they must work independently with no opportunity for discussion of the ratings until the entire session has been completed.

4. Selection of Pavements for Rating—Because ratings are to be made of the serviceability of pavements, a wide range of serviceability should be represented among the pavements that are selected for rating. Moreover, there should be among the sections selected pavements containing all of the various types and degrees of pavement distress that are likely to influence the serviceability of highways. Before a field rating session, engineers study the highway network in the area under consideration (200 mi or less in diameter, for example) and pick sections of roadway so that a reasonable balance is obtained among obviously very good, good, fair, poor and obviously very poor sections. The Road Test system was based on four rating sessions in three different states; 138 sections of pavement were studied. About one-half were flexible pavement; the other half, rigid. The Road Test panel agreed that the minimum desirable length of a pavement to be rated was 1,200 ft; however, in a few cases shorter sections were included. This length was sufficient for the raters to ride over the section at high speed without being influenced by the condition of pavement at either end.

5. Field Rating—The members of the panel are taken in small groups to the sections that are to be rated. They are permitted to ride over each section in a vehicle of their choice (usually one with which they are familiar), to walk the pavement and to examine it at will. Each rater works independently—there is no discussion among the raters. When he is satisfied as to his rating, he marks his rating card and turns it in to a staff representative. The group then moves on to the next section. Each group takes a different route to reduce the possibility of bias over the day (raters may rate differently in the afternoon than in the morning, therefore, the groups are scheduled so that some sections are rated by one or two groups in the morning and the same sections by the other groups in the afternoon). It has been found that, near metropolitan areas, sections with satisfactorily different characteristics can be found close enough together so that the raters can travel routes containing about 20 sections per day. When rating present serviceability of a pavement, raters have found it helpful to ask themselves "How well would this road serve me if I were to drive my own car over roads just like it all day long today?" Here again, of course, serviceability is related to the intended use of the road, primary highway, city street, farm road, etc.

6. Replication—It is necessary to determine the ability of the panel to be consistent in its ratings. The Road Test panel rated many sections twice, first on one day and again on another day near enough to the first so that the section did not change physically, yet remote enough so that all extraneous influences on the raters would be in effect. In general, it might be expected that replicate ratings would differ more when separated by several months than when separated by only one day. For this reason, the replication differences observed in the Road Test rating sessions are perhaps to some degree an underestimate of replication differences in a larger time reference. The difference between repeated ratings on the same section is a criterion for the adequacy of a present serviceability index derived from measurements.

7. Validation of Rating Panel—Because the panel is intended to represent all highway users, it is necessary to test its ability to do so. To a limited extent such validation was obtained for the Road Test panel by selecting other groups of users and having them rate some of the same sections that had been rated by the panel. One such group consisted of two commercial truck drivers who made their ratings based on the rides they obtained when driv-

ing their own fully-loaded tractor-semitrailer vehicles. Another group was made up of ordinary automobile drivers not professionally associated with highways. For the sections involved, these studies indicated that the ratings given pavements by the Road Test panel were quite similar to those that were given by the other user groups. Of course, if a greater number of sample groups had been studied, more positive statements could be made as to how well the panel represented the universe of all users.

8. Physical Measurements—If it is practicable for the panel to rate all roads in the area often enough, no measurements need be taken. Analyses may be based on the PSR itself. Since it was not possible for the panel to rate the Road Test sections (ratings were desired every two weeks), it was necessary to establish a PSI or index that would predict the panel's ratings. To accomplish this, measurements of certain physical characteristics of the pavements were necessary. To determine which measurements might be most useful, the members of the panel were asked to indicate on rating cards which measurable features of the roadway influenced their ratings. It was apparent that present serviceability was a function primarily of longitudinal and tranverse profile with some likelihood that cracking, patching, and faulting would contribute. Therefore, all of these characteristics were measured at each of the 138 sections that were rated by the panel. Several other objective measurements could have been added to the list if other phenomena were permitted consideration by the established rules of the game. Skid resistance, noise under tires, and shoulder and ditch conditions might be in this category.

Measurements fall rather naturally into two categories: those that describe surface deformation and those that describe surface deterioration. Of course, phenomena in the second category may or may not influence measurements in the first category. Measures of surface deformation will reflect the nature of longitudinal and transverse profiles, or may represent the response of a vehicle to the profile, as does the BPR roughometer. Supplemental profile characteristics, such as faulting will ordinarily be measured. Present and past surface deterioration will be reflected through measures of cracking, spalling, potholing, patching, etc., and may include phenomena whose influence on present serviceability ratings range from negligible to appreciable.

9. Summaries of Measurements—There are many different ways to summarize longitudinal and transverse profiles. For example, longitudinal profile may be expressed as total deviation of the record from some base line in inches per mile, number of bumps greater than some minimum, some combination of both of these, or by any number of other summary statistics involving variance of the record, power spectral density analysis, etc. Transverse profile may be summarized by mean rut depth, variance of transverse profile, etc. The variance of rut depth along the wheel paths is also a useful statistic. Cracking occurs in different classes of severity as do other measures of surface deterioration. Measurements in any of these classes may be expressed in one unit or another.

10. Derivation of a Present Serviceability Index—After obtaining PSR's and measurement summaries for a selection of pavements, the final step is to combine the measurement variables into a formula that "gives back" or predicts the PSR's to a satisfactory approximation. Part of this procedure should consist in determining which of the measurement summaries have the most predictive value and which are negligible after the critical measurements are taken into account. The technique of multiple linear regression analysis may be used to arrive at the formula, or index, as well as to decide which measurements may be neglected. For example, a longitudinal profile summary may be sensitive to faulting so that faulting measurements need not appear in the index formula whenever this profile measure is included.

The decisions as to which terms should be in the serviceability formula and which terms should be neglected may be made by comparing the lack of success with which the formula gives back the ratings with a pre-selected criterion for closeness of fit, such as the Panel's replication error. There is no justification for a formula that can predict a particular set of ratings with greater precision than the demonstrated ability of the panel to give the same ratings to the same pavements twice. Therefore, the multiple linear regression analysis will yield a formula that will combine certain objective measurements to produce estimates of the panel's ratings to an average accuracy no greater than the panel's average ability to repeat itself.

Performance

The serviceability index is computed from a formula containing terms related to objective measurements that may be made on any section of highway at any time. At the AASHO Road Test, these measurements were made and the index computed for each test section every two weeks. Thus a serviceability-time history is available for each test section beginning at the time test traffic operation was started. The present serviceability values range in numerical value from 0 to 5 (Fig. 1–F).

To fulfill the first Road Test objective of finding relationships between performance and pavement structure design, some summarization of the serviceability-time history is implied. Performance may be said to be related to the ability of the pavement to serve traffic over a period of time. A pavement with a low serviceability during much of its life would not have performed its function of serving traffic as well as one that had high serviceability during most of its life even if both ultimately reached the same state of repair.

Performance, at the Road Test, was defined as the trend of serviceability with increasing load applications. Analysis of performance was based on mathematical models for expressing the serviceability trend in terms of design, load, and number of load applications. The procedures for analysis are discussed in Appendix G.

ROAD TEST INDEXES

The techniques previously described were used in the derivation of present serviceability indexes for the AASHO Road Test. This section includes tabulations of the actual data obtained in the field rating sessions by the Road Test Rating Panel and data obtained from the objective measurements of the pavements rated. Relationships among the ratings and various measurements are shown graphically and the results of the regression analyses in which the serviceability indexes were derived are given.

The matter of precision required of an index and precision attained in the Road Test indexes is discussed. Alternate measurement systems are mentioned for the benefit of agencies not able to equip themselves with elaborate instruments.

Ratings for Selected Pavements

After establishing concepts, ground rules, and rating forms for present serviceability ratings, the AASHO Road Test performance rating panel rated 19 pavement sections near Ottawa, Ill. on April 15-18, 1958, 40 sections near St. Paul-Minneapolis on August 14-16, 1958, 40 sections near Indianapolis on May 21-23, 1959, and 39 sections on and near the Road Test on January 20-22, 1960. Ten Illinois sections, 20 Minnesota sections, 20 Indiana sections and 24 sections on and near the Road Test were flexible pavements; all remaining sections were rigid pavements. Each section was 1,200 ft long except those on the Road Test which averaged 215 ft. With the cooperation of the respective state highway departments, sections were selected to represent a wide range of pavement conditions.

Coincident with the rating session, Road Test crews and instruments were used to obtain condition surveys and profile measurements for each section. Summaries for all evaluations of the 74 flexible pavement sections are given in Table 1–F, and corresponding evaluations for the first 49 rigid pavements are given in Table 2–F.

Although the panel members had indicated that rutting in flexible pavement must influence serviceability, the first three rating sessions did not include pavements with rutting severe enough to contribute significantly to the pavement serviceability. Since severe rutting occurred at the Road Test it was necessary to assemble the panel for a fourth session in which sections with severe rutting were rated. Reanalysis of the data from all four sessions then made it possible to determine the effect of rutting on serviceability. A second objective of the fourth session was to rate a small number of rigid pavements only for the purpose of checking present serviceability indexes derived from the first 49 sections. For these reasons, flexible pavements from all four sessions appear in Table 1–F; Table 2–F includes only rigid pavement sections from the first three sessions.

Present serviceability ratings shown in the third column of Tables 1 and 2 are mean values for individual ratings given by the Road Test panel. In general, each mean represents about ten individual ratings. For both pavement types, the PSR values range from about 1.0 to 4.5 with nearly the same number of sections in the poor, fair, good, and very good categories (Fig. 1–F). The grand mean PSR for all rated pavements was slightly less than 3.0 for both pavement types.

Over forty of the sections were revisited by the panel during the same rating session, and differences between first and second mean ratings are shown in the fourth columns of Tables 1 and 2. The replication differences ranged from 0 to 0.5; the mean difference was less than 0.2 for both flexible and rigid pavements. The fifth columns give the standard deviation of individual PSR values for each section. These standard deviations are of the order 0.5, an indication that only about two or three individual ratings (out of ten) were farther than 0.5 rating points from the panel mean PSR.

The mean ratings of the two truck drivers who rated certain Illinois sections are shown in the sixth columns. The seventh columns show mean ratings given to selected Illinois sections by a group of about 20 Canadian raters. The general agreement among the various rating groups is apparent.

The eighth and ninth columns represent summaries of the AASHO Panel response to the acceptability question (Fig. 1–F). The tables give what fraction of the panel decided the present state of a particular pavement section to be acceptable and what fraction decided the

TABLE 1-F

DATA FOR 74 SELECTED FLEXIBLE PAVEMENTS

Pvt. Loc.	Sect. Code	Present Serviceability Ratings: AASHO Panel: 1st PSR	AASHO Panel: Replic. diff. in PSR	AASHO Panel: Std. dev. of PSR among raters	Truck Dr'v'rs PSR	Canad. Raters PSR	Acceptability Opinions: AASHO Panel Fraction: Yes	Fraction: No	Longitudinal and Transverse Roughness: $\overline{SV}$ Mean Slope Var'nce in Wh'p'ths ($\times 10^6$)	$\overline{AR}$ Mean AASHO Ro'm't'r in Wh'p'ths in./mi. @10 mph	$\overline{RD}$ Mean Rut Depth, (in.)	$\overline{RDV}$ Mean Rut Depth Var'nce in$^2 \times 100$	Major Cracking: Class 2 + Class 3, ft^2/1000ft^2	Long. & Trans. ft./1000ft^2	Patching: P Ft^2 per 1000 ft^2	Transformations: Log $(1+\overline{SV})$	$\overline{RD}^2$	Sq. rt. C+P	PSI 121 Pres. Serv. Index	Resid. Diff. Betw'n PSR & PSI
Ill.	F 3	4.3		.1	4.5	4.3	1.0	.0	2.8		.10	.7	0	0	0	.57	.01	.0	3.9	.4
	F 4	2.4		.4		2.0	.0	.6	20.5		.22	9.2	343	0	0	1.33	.05	18.5	2.3	.1
	F 5	3.3		.7	3.5	2.6	.6	.2	9.2		.08	3.6	8	0	0	1.01	.01	2.8	3.1	.2
	F 6	4.4	.1	.2	3.5		1.0	.0	3.5		.08	.7	0	0	0	.65	.01	.0	3.8	.6
	F 7	3.8	.1	.6	2.5	3.6	.9	.0	15.5		.06	.4	0	0	0	1.22	.00	.0	2.7	1.1
	F 8	2.6	.2	.7	2.0	2.7	.3	.6	9.5		.08	5.7	64	0	0	1.02	.01	8.0	3.0	.4
	F 9	3.2	.2	.6	3.0	3.0	.6	.2	14.0		.15	3.4	2	0	3	1.18	.02	2.2	2.7	.5
	F10	2.4	.0	.5	3.0	2.2	.1	.6	16.8		.16	3.4	17	0	14	1.25	.03	5.6	2.6	.2
	F11	1.3	.3	.5	1.5		.0	1.0	42.8		.26	10.3	292	0	11	1.64	.07	17.4	1.6	.3
	F12	1.1	.3	.2	1.0	1.7	.0	1.0	56.0		.19	10.9	21	0	2	1.76	.04	4.8	1.6	.5
Minn.	101	3.8	.3	.4			1.0	.0	1.9		.04	.4	0	29	0	.46	.00	5.4	4.1	.3
	102	3.8		.6			1.0	.0	1.5		.09	.3	0	34	0	[illegible]	.00	5.8	4.2	.4
	103	3.8		.4			1.0	.0	1.7		.05	.2	0	14	0	.43	.00	3.7	4.2	.4
	104	3.8	.0	.4			1.0	.0	2.1		.04	.4	0	9	0	.49	.00	3.0	4.1	.3
	105	3.2		.4			.6	.1	7.0		.14	.5	0	0	10	.90	.02	3.2	3.3	.1
	106	1.3		.4			.0	1.0	58.5		.07	6.6	145	22	35	1.77	.00	14.2	1.5	.2
	107	1.3	.2	.4			.0	1.0	58.4		.08	2.9	75	12	55	1.77	.01	11.9	1.5	.2
	108	2.1		.6			.0	.9	17.6		.13	3.2	15	0	5	1.27	.03	4.5	2.5	.4
	109	1.5	.2	.2			.0	1.0	36.2		.37	5.6	30	2	76	1.57	.13	10.4	1.7	.2
	110	2.4	.0	.9			.0	.8	11.4		[illegible]	3.2	2	0	3	1.09	.00	2.2	2.9	.5
	111	4.2		.2			1.0	.0	1.7		.11	.2	0	0	0	.42	.01	.0	4.2	.0
	112	3.9		.4			1.0	.0	1.4		.09	.2	0	0	0	.38	.01	.0	4.3	.4
	113	3.1	.1	.6			.4	.4	7.8		.08	1.3	1	68	66	.94	.01	11.6	3.1	.0
	114	2.2		.7			.0	.9	27.8		.13	5.2	0	1	4	1.46	.02	2.2	2.2	.0
	115	1.5		.6			.0	1.0	33.4		.08	5.4	0	7	6	1.54	.01	3.6	2.0	.5
	116	2.9	.5	.5			.6	.1	6.0		.02	.7	0	180	0	.85	.00	13.4	3.3	.4
	117	1.6	.0	.6			.0	1.0	39.4		.12	4.6	0	0	0	1.61	.01	.0	1.9	.3
	118	4.0	.2	.3			1.0	.0	1.6		.04	.2	0	0	0	.41	.00	.0	4.2	.2
	119	4.2		.5			1.0	.0	1.3		.03	.2	0	0	0	.36	.00	.0	4.3	.1
	120	2.9	.1	.5			.3	.2	5.8		.01	1.4	0	74	0	.84	.00	8.6	3.3	.4
	301	4.1		.6			1.0	.0	4.6	101	.24	.4	44	43	0	.75	.06	9.3	3.4	.7
	302	4.0		.5			1.0	.0	5.4	123	.34	.5	204	75	0	.81	.12	16.7	3.2	.8

Ind.	303	3.2		.5			.6	.1	20.1	146	.12	.4	12	1	0	1.32	.00	3.0	2.5	.7
	304	2.4	.0	.4			.2	.5	29.2	134	.17	1.8	455	0	17	1.45	.03	21.7	2.0	.4
	305	2.9	.1	.3			.4	.3	9.1	129	[illegible]	1.2	292	0	32	[illegible]	.01	18.0	2.9	.0
	306	2.4	.2	.5			.3	.5	[illegible]	[illegible]	.12	2.4	816	0	0	1.33	.01	28.6	2.2	.2
	307	1.7	.3	.6			.1	.9	95.9	383	[illegible]	1.9	719	0	111	1.99	.00	28.8	1.0	.7
	308	1.0		.4			.0	1.0	51.8	296	.03	5.1	691	0	161	1.72	.00	29.2	1.5	.5
	309	1.3		.4			.0	1.0	41.2	233	.14	7.0	613	0	159	1.62	.02	27.8	1.6	.3
	310	3.2		.6			.7	.1	11.5	144	.18	2.0	17	0	0	1.11	.03	4.1	2.8	.4
	311	2.7		.4			.4	.4	15.0	162	.14	.8	45	0	0	1.2[illegible]	.02	6.7	2.6	.1
	312	1.6		.4			.0	1.0	49.8	217	.23	2.3	502	0	31	1.71	.05	23.1	1.5	.1
	313	1.4		.4			.0	1.0	42.0	182	.27	2.9	437	0	72	1.63	.07	22.6	1.7	.3
	314	2.6		.5			.3	.4	19.0	127	.24	1.2	10	64	2	1.31	.06	8.7	2.4	.2
	315	3.4		.7			.9	.0	6.9	107	.22	.2	183	46	0	[illegible]	.05	15.1	3.1	.3
	316	2.9		.5			.5	.4	11.3	140	.09	.8	177	0	4	1.09	.01	13.4	2.8	.1
	317	4.3		.2			1.0	.0	2.9	95	.01	.1	0	0	0	.59	.00	.0	3.9	.4
	318	4.3		.3			1.0	.0	3.3	92	.00	.0	1	0	0	.63	.00	1.0	3.8	.5
	319	4.2	.0	.3			1.0	.0	3.8	92	.12	.2	0	1	0	.68	.01	1.0	3.7	.5
	320	3.9	.3	.4			.9	.0	3.8	105	.16	.4	0	2	0	.62	.03	1.4	3.7	.2
Test Road Sect.	501	3.8		.3			1.0	.0	5.8	132	.08	.3	0	0	0	.83	.01	.0	3.4	.4
	502	3.4		.6			.8	.1	10.3	168	.20	2.2	51	0	0	1.05	.04	7.2	3.9	.5
	503	3.1	.0	.3			.7	.0	7.6	129	.11	.8	17	0	7	.93	.01	4.9	3.2	.1
	504	4.1		.2			1.0	.0	2.6	109	.03	.2	0	0	0	.56	.00	.0	4.0	.1
	505	3.4		.2			1.0	.0	3.8	89	.33	.9	14	0	0	.68	.11	3.7	3.5	.1
	506	3.4		.4			.9	.0	4.8	89	.24	.9	0	0	0	.76	.06	.0	3.5	.1
	507	2.8		.4			.6	.3	7.6	103	.43	1.4	5	0	25	.93	.18	5.4	2.9	.1
	508	3.5		.5			.9	.0	4.0	75	.46	.5	0	0	0	.70	.21	.0	3.4	.1
	509	3.3		.5			.8	.0	2.9	84	.39	.4	9	0	0	.59	.15	3.0	3.7	.4
	510	3.3		.4			.9	.0	5.0	90	.44	1.0	2	0	0	.78	.19	1.3	3.3	.0
	511	3.6		.5			1.0	.0	3.5	90	.47	.6	0	0	0	.65	.22	.0	3.5	.1
	512	3.2		.7			.8	.0	5.1	87	.53	1.3	0	0	0	.79	.28	.0	3.1	.1
	513	3.4		.5			.8	.0	2.6	75	.56	.5	0	0	0	.56	.31	.0	3.5	.1
	514	1.8		.5			.0	.9	11.1	122	.73	5.1	80	0	16	1.08	.53	9.8	2.1	.3
	515	3.3		.7			.9	.0	2.5	79	.38	.5	0	0	0	.54	.14	.0	3.8	.5
	516	2.6		.6			.4	.3	5.4	86	.55	.6	15	0	0	.81	.30	3.9	3.0	.4
	517	3.2		.6			.5	.1	5.4	83	.54	.7	1	0	0	.81	.29	1.2	3.1	.1
	518	1.7		.4			.0	.8	21.0	149	.92	2.8	222	0	0	1.34	.85	14.9	1.2	.5
	519	2.4		.5			.1	.4	6.5	99	.53	1.5	21	0	0	.88	.28	4.6	2.9	.5
	520	3.0	.2	.6			.6	.1	6.8	89	.46	1.1	0	0	0	.89	.21	.0	3.0	.0
Off Site Sect.	521	3.3		.5			.7	.1	4.3	118	.09	.4	0	0	0	.72	.01	.0	3.6	.3
	522	2.7		.4			.6	.1	13.7	185	.11	3.8	300	0	1	1.17	.01	17.4	2.6	.1
	523	2.4		.4			.2	.2	10.8	137	.22	1.3	496	0	52	1.07	.05	23.4	2.7	.3
	524	0.9		.5			.0	1.0	88.1	281	.25	6.2	392	0	60	1.95	.06	21.2	1.0	.1
Sum		215.4	3.9	34.2												75.19	5.59	565.7	215.4*	22.3
Mean		2.91	.16	.46												1.02	.076	7.64	2.91	.30
Sum of Squares		66.85														13.27	1.34	5255	56.42*	10.42*
Sum of Products with PSR																-26.69	-1.51	-369.3		
Sum of Products with $\overline{\log}(1+\overline{SV})$																	- .166	171.63		
Sum of Products with $\overline{RD}^2$																		-- 3.93		

*Obtained from Unrounded Calculations

$$PSI\ 121 = 5.03 - 1.91 \log(1+\overline{SV}) - 1.38\overline{RD}^2 - .01\sqrt{C+P}$$

TABLE 2–F

Data for 49 Selected Rigid Pavements

Pvt. Loc.	Sect. Code	Present Serviceability Ratings: AASHO Panel: 1st PSR	AASHO Panel: Replic. diff. in PSR	AASHO Panel: std. dev of PSR among raters	Truck Driv'rs PSR	Canad. Raters PSR	Acceptability Opinions: AASHO Panel Fraction Yes	AASHO Panel Fraction No	Longitudinal Roughness: $\overline{SV}$ Mean Slope Var'nce in wh'pth (x10⁶)	$\overline{AR}$ Mean AASHO R'om'tr @ 10 mph (in./mi.)	F Fault'g in Wh'pth in in/1000'	Cracking: C Class 2 and Sealed Cracks ft/1000 ft²	Spalling: ft.²/1000ft.² for areas > 3" Dia.	Patching: P Patch'd Area ft²/1000 ft.²	Transformations: Log $(1+\overline{SV})$	Log $\overline{AR}$	Sq. root C+P	PSI 211 Pres. Serv. Index	Resid. Diff. Betw'n PSR & PSI
	R1	2.0	.2	.6	1.5		.0	.8	52.0		2	53	4	8	1.72		7.8	1.7	.3
	R2	4.2		.3	4.5		1.0	.0	6.5		0	4	0	0	.88		2.0	3.7	.5
	R3	2.6	.3	.6	2.5		.2	.5	22.2		0	42	0	11	1.37		7.3	2.3	.3
	R4	2.3	.2	.3	2.5		.0	.5	26.2		7	46	0	7	1.44		7.3	2.2	.1
Ill.	R5	1.2		.4	1.5		.0	1.0	47.8		1	102	0	28	1.69		11.4	1.4	.2
	R6	2.8	.1	.6	2.5	3.0	.2	.1	25.5		3	15	2	1	1.42		4.0	2.5	.3
	R7	4.4	.0	.3	4.5	4.4	1.0	.0	3.2		0	0	0	0	.63		0	4.3	.1
	R8	1.1	.2	.4			.0	1.0	50.8		3	65	11	5	1.71		8.4	1.6	.5
	R9	0.9	.0	.3			.0	1.0	76.8		1	74	19	85	1.89		12.6	0.9	.0
	201	1.3	.1	.6			.0	1.0	43.3		1	40	60	59	1.65		10.0	1.6	.3
	202	1.8		.5			.0	1.0	24.2		0	23	4	66	1.40		9.4	2.1	.3
	203	2.1	.3	.6			.1	.9	24.7		0	47	1	41	1.41		9.4	2.1	.0
	204	4.1		.3			1.0	.0	2.4		0	4	0	0	.54		2.0	4.3	.2
	205	3.8	.3	.4			1.0	.0	4.0		0	2	0	0	.70		1.4	4.0	.2
	206	3.0	.0	.5			.6	.2	7.8		1	14	0	1	.95		3.9	3.4	.4
	207	3.0		.6			.4	.2	7.5		1	22	0	0	.93		4.7	3.3	.3
	208	2.9	.1	.6			.3	.4	9.7		0	14	0	0	1.03		3.7	3.2	.3
Minn.	209	2.5		.4			.1	.6	17.6		0	34	0	0	1.27		5.8	2.6	.1
	210	1.4		.5			.0	1.0	59.2		0	16	500	12	1.78		5.3	1.8	.4
	211	4.3		.2			1.0	0	3.0		0	0	0	0	.60		0	4.3	.0
	212	4.3	.0	.4			1.0	0	4.0		0	0	0	0	.70		0	4.1	.2
	213	3.7		.4			1.0	0	5.3		0	0	0	0	.80		0	4.0	.3
	214	3.6	.3	.5			1.0	0	4.4		0	0	0	0	.73		0	4.1	.5
	215	3.9		.4			1.0	0	5.3		0	0	0	0	.80		0	4.0	.1
	216	3.9	.0	.6			1.0	0	6.3		0	0	0	0	.87		0	3.8	.1
	217	1.3	.0	.4			.0	1.0	32.3		0	76	2	1	1.52		8.8	1.9	.6
	218	1.2		.4			.0	1.0	27.8		10	64	0	0	1.46		8.0	2.1	.9
	219	2.2		.6			.0	.9	25.6		4	97	0	1	1.42		9.9	2.0	.2
	220	4.4	.0	.3			1.0	.0	4.0		0	0	0	0	.70		0	4.1	.3
	401	4.0		.3			1.0	0	6.6	134	2	0	1	0	.88	2.13	0	3.8	.2
	402	3.8		.4			1.0	0	6.6	126	4	11	1	0	.88	2.10	3.3	3.5	.3
	403	3.6		.6			.9	0	6.8	113	1	2	4	0	.89	2.06	1.4	3.7	.1
	404	3.2		.6			.6	.2	9.8	131	4	1	1	2	1.03	2.12	1.7	3.4	.2
	405	2.6		.6			.3	.5	14.6	167	5	72	13	0	1.19	2.22	8.5	2.5	.1
	406	2.8		.6			.4	.3	10.4	151	5	70	10	1	1.06	2.18	8.4	2.8	.0
	407	1.8	.5	.6			.1	.8	49.4	268	1	41	4	29	1.70	2.43	8.4	1.6	.2
	408	1.8		.6			.1	.8	54.5	245	2	42	8	37	1.74	2.39	8.9	1.5	.3
Ind.	409	2.1		.6			.2	.8	36.6	276	1	50	7	29	1.58	2.44	8.9	1.8	.3
	410	2.2		.5			.2	.8	25.1	230	2	86	5	33	1.42	2.36	10.9	1.9	.3
	411	1.8		.5			.1	.8	45.4	286	0	40	6	65	1.67	2.46	10.2	1.5	.3
	412	2.7		.6			.4	.4	9.9	147	5	81	3	5	1.04	2.17	9.3	2.7	.0
	413	4.2		.4			1.0	.0	6.1	106	1	0	1	0	.85	2.03	0	3.9	.3
	414	4.3		.4			1.0	.0	5.2	112	1	0	0	0	.79	2.05	0	4.0	.3
	415	4.3		.4			1.0	.0	7.1	132	1	0	0	0	.91	2.12	0	3.8	.5
	416	1.2	.3	.6			.0	.9	81.9	338	8	54	1	219	1.92	2.53	16.5	0.5	.7
	417	2.2	.0	.6			.1	.7	32.2	252	18	36	1	0	1.52	2.40	6.0	2.2	.0
	418	4.3	.1	.3			1.0	.0	4.6	113	1	0	0	0	.75	2.06	0	4.1	.2
	419	2.8	.0	.7			.5	.3	12.6	126	2	5	2	13	1.13	2.10	4.2	3.0	.2
	420	2.7	.1	.4			.1	.3	17.8	137	2	5	7	16	1.27	2.14	4.6	2.7	.0
Sum		138.6	3.1												58.23		254.3	138.6*	12.5
Mean		2.83	.13												1.19		5.19	2.83*	[illegible]
Sum of Squares		57.92													7.55		905.70	53.08*	[illegible]
Sum of Products with PSR															-19.70		-206.53		
Sum of Products with Log $(1+\overline{SV})$																	71.71		

*Obtained from Unrounded Calculations

$$PSI\ 211 = 5.41 - 1.80 \log(1+\overline{SV}) - .09\sqrt{C + P}$$

pavement to be unacceptable. By implication the remaining fraction of the panel gave the undecided response.

Figures 2–F through 5–F show the connection between corresponding PSR values and acceptability opinions for the two types of pavement. Freehand curves have been drawn to indicate (Figs. 2–F and 3–F) that the 50th percentile for acceptability occurs when the PSR is in the neighborhood of 2.9; the 50th percentile for unacceptability corresponds roughly to a PSR of 2.5 (Figs. 4–F and 5–F).

Measurements for Selected Pavements

Following the acceptability opinion, Tables 1 and 2 give summary values for measurements that were made on the selected pavements. Measurements are shown in three categories: those that describe longitudinal and transverse roughness, those that summarize surface crack-

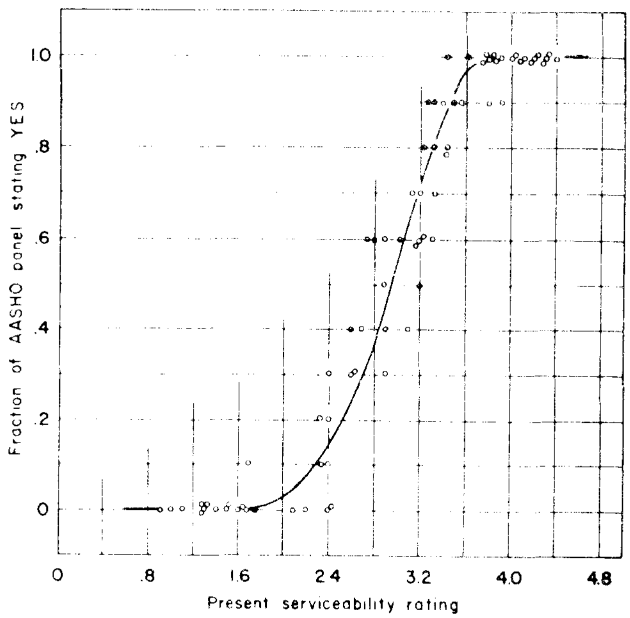


Figure 2–F. Acceptability vs present serviceability rating; 74 flexible pavements.

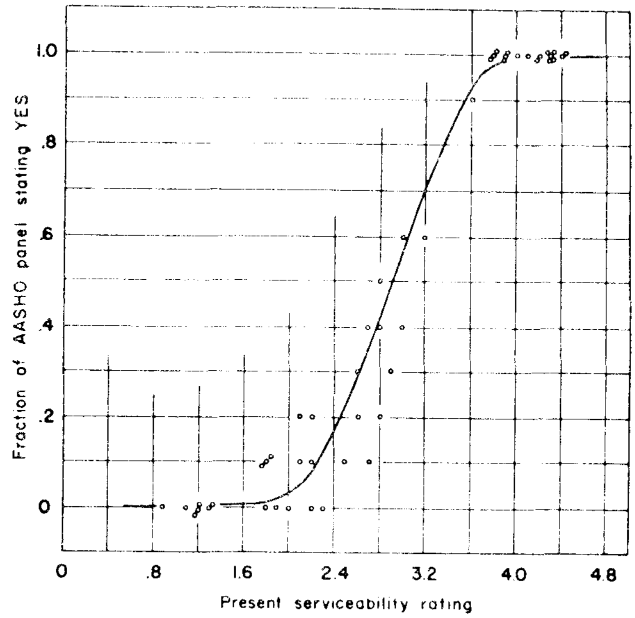


Figure 3–F. Acceptability vs present serviceability rating; 49 rigid pavements.

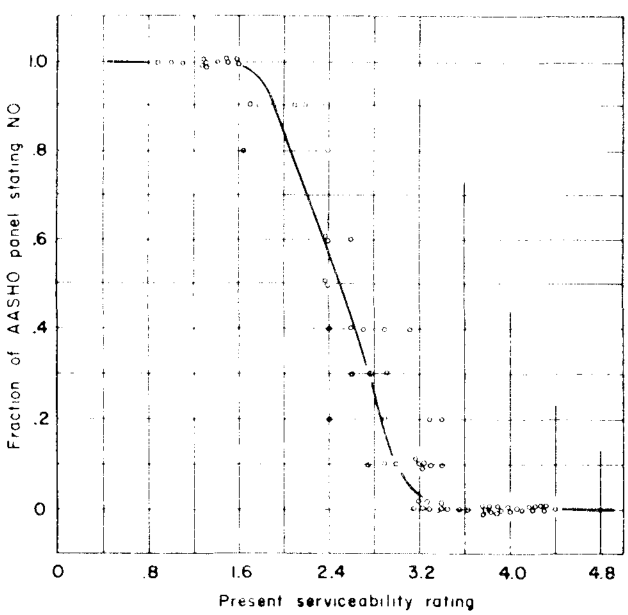


Figure 4–F. Unacceptability vs present serviceability rating; 74 flexible pavements.

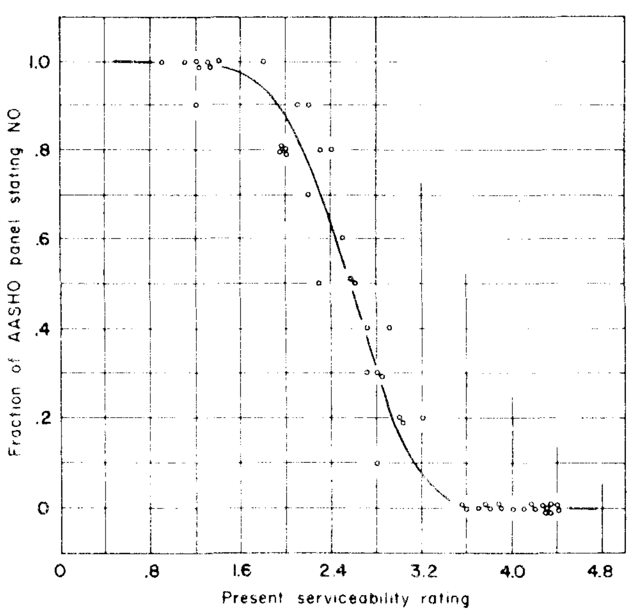


Figure 5–F. Unacceptability vs present serviceability rating; 49 rigid pavements.

ing, and finally a measurement of the patched area found in the section.

The symbol $\overline{SV}$ is used for the summary statistic of wheelpath roughness as measured by the Road Test longitudinal profilometer. For each wheelpath the profilometer produces a continuous record of the pavement slope between points 9 in. apart. For a particular wheelpath, the slopes are sampled, generally at 1-ft intervals, over the length of the record. A variance* is calculated for the sample slopes in each wheelpath, then the two wheelpath slope variances are averaged to give $\overline{SV}$.

A Bureau of Public Roads roughness indicator, or roughometer, was adapted for use at the AASHO Road Test, but this development was not made until just before the Indiana rating session and still more developmental work was done on the AASHO roughometer after the Indiana session. The AASHO roughometer has a modified output and was operated at 10 mph, so that roughometer values shown in Tables 1 and 2 are not the values that would be obtained with the BPR roughometer at 20 mph. Nevertheless, roughometer values in inches per mile are given; the roughometer values averaged for both wheelpaths, $\overline{AR}$, are correlated with the corresponding mean slope variances. Figures 6–F and 7–F show the extent of this correlation for the last two rating sessions.

One other instrument, a rut depth gage, was used to obtain profile characteristics of the flexible pavement sections. This gage is used to determine the differential elevation between the wheelpath and a line connecting two points each 2 ft away (transversely) from the center of the wheelpath. Rut depth measurements were obtained at 20 ft intervals in both wheelpaths. A erage rut depth values, $\overline{RD}$, for the flexible sections are given in Table 1–F; the values range from 0 to nearly 1 in. Variances were calculated for the rut depths in each wheelpath, then the two wheelpath variances were averaged to give the $\overline{RDV}$ values (Table 1–F). Figure 8–F shows the correlation between $\overline{SV}$ and $\overline{RDV}$ for the 74 flexible sections.

Profile information for rigid pavements included a measure of faulting in the wheelpaths. These measurements are given in Table 2–F expressed in total inches of faulting (in wheelpaths only) per 1,000 ft of wheelpath.

The remaining measurements for flexible pavement sections are given in Table 1 in terms of area affected by class 2 and class 3 cracking, length of transverse and longitudinal cracks, and patched area, where areas and lengths are expressed per 1,000 square feet of pavement area. Corresponding measurements for rigid pavements are shown in Table 2–F in terms of length of class 2 and sealed cracks, spalled area, and patched area. Lengths for rigid pavement cracks were determined by projecting the cracks both transversely and longitudinally, choosing the larger projection, then expressing the accumulated result in feet per 1,000 sq ft of pavement area. Only spalled areas having more than 3-in. diameters were considered, and both spalling and patching are expressed in square feet per 1,000 sq ft of pave-

* The variance of a set of N sample values, Y_1, Y_2, ..., Y_N is defined to be the sum of all N squared deviations from the mean divided by $N - 1$. Thus the variance of Y is $\Sigma\ (Y - \overline{Y})^2/(N - 1)$, where $\overline{Y} = \Sigma\ Y/N$ is the sample mean.

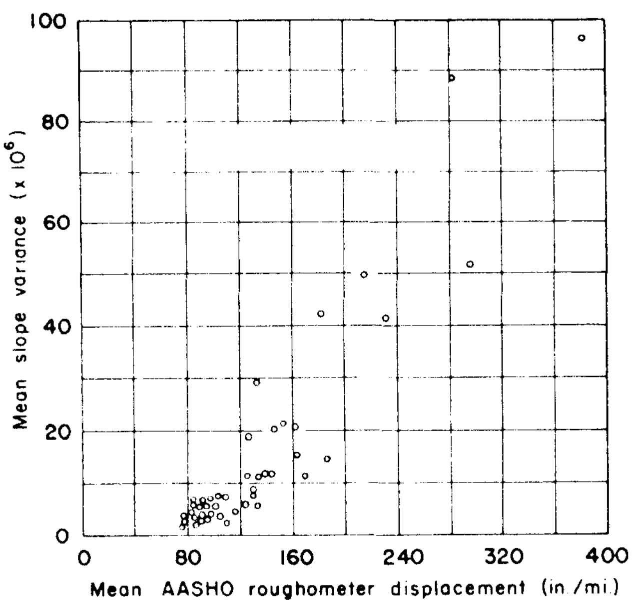


Figure 6–F. Slope variance vs AASHO roughometer displacement; 44 flexible pavements.

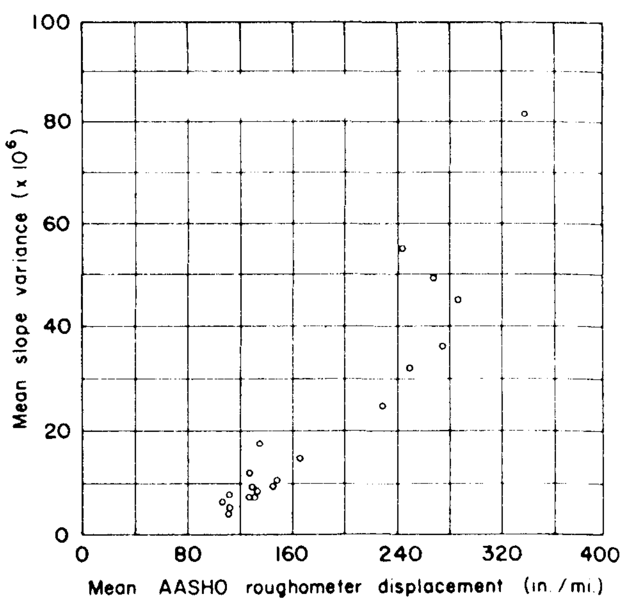


Figure 7–F. Slope variance vs AASHO roughometer displacement; 20 rigid pavements.

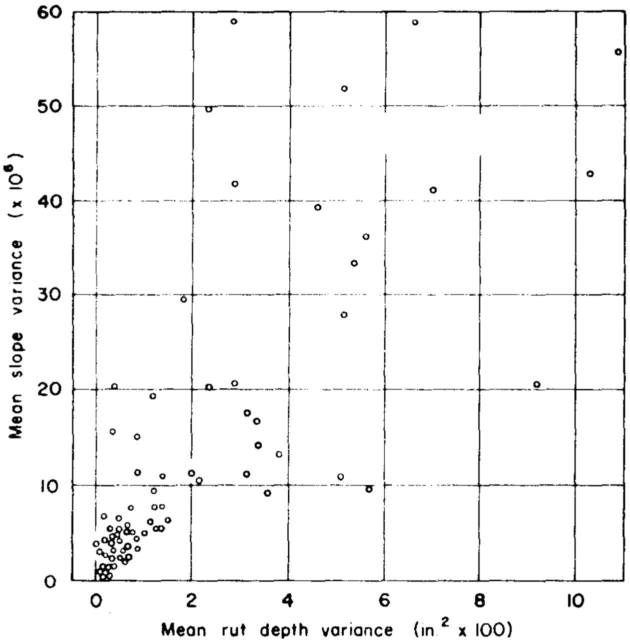


Figure 8–F. Rut depth variance vs slope variance; 74 flexible pavements.

ment area. Virtually any pair of measurements are intercorrelated to some degree, some more highly than others. Figures 9–F and 10–F indicate the degree to which $\overline{SV}$ is correlated with the sum of cracking and patching values. A stronger correlation is shown in Figure 10–F than in Figure 9–F. If either correlation were perfect, one or the other of the plotted variables would be redundant in an index of present serviceability.

Hypothesis and Assumptions for Present Serviceability Index

One requirement for an index of present serviceability is that when pavement measurements are substituted into the index formula, the resulting values should be satisfactorily close to the corresponding present serviceability ratings. There are also advantages if the index formula is relatively simple in form and if it depends on relatively few pavement characteristics that are readily measured.

Guided by the discussion of the AASHO rating panel as well as by results from early rating sessions, the general mathematical form of the present serviceability index was assumed to be

$$\text{PSI} = C + (A_1R_1 + A_2R_2 + \ldots) + (B_1D_1 + B_2D_2 + \ldots) \qquad (1\text{–F})$$

where R_1, R_2, . . . are functions of profile roughness and where D_1, D_2, . . . are functions of surface deterioration. The coefficients C, A_1, A_2, . . ., B_1, B_2, . . . may then be determined by a least squares regression analysis. It is expected, of course, that A_1, A_2, . . . B_1, B_2 . . . will have negative signs. To perform the analysis, the PSR for the j^{th} of a set of sections is represented by

$$\text{PSR}_j = \text{PSI}_j + E_j \qquad (2\text{–F})$$

in which E_j is a residual not explained by the functions used in the index. Minimizing the sum of squared residuals for all sections in the analysis leads to a set of simultaneous equations whose solutions are the required coefficients. The respective effect of adding or

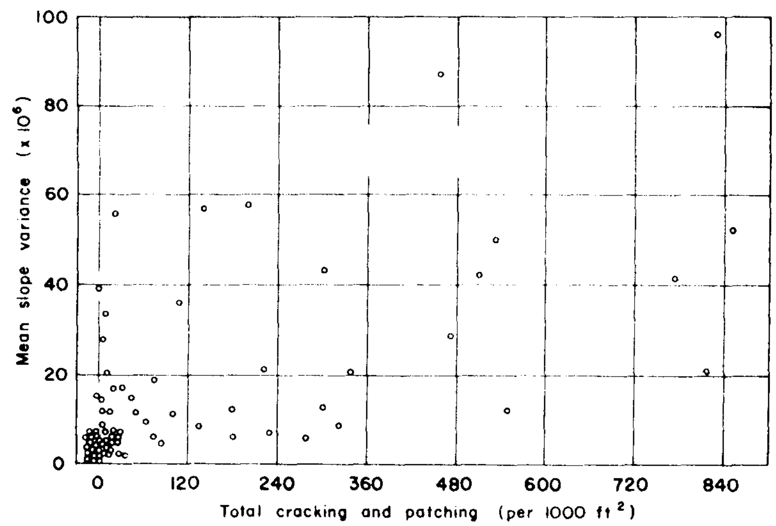


Figure 9–F. Mean slope variance vs cracking and patching; 74 flexible pavements.

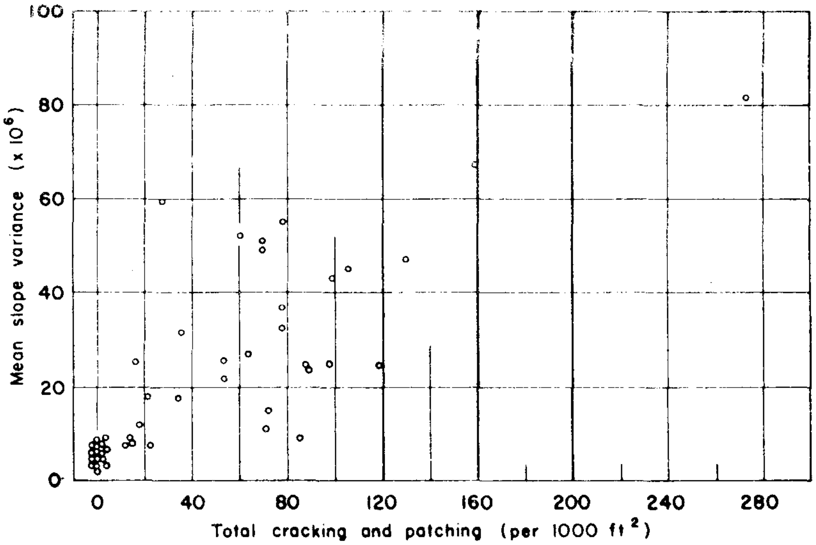


Figure 10–F. Mean slope variance vs cracking and patching; 49 rigid pavements.

deleting terms in Eq. 1–F will be to decrease or increase the sum of squared residuals. The change in residual sum of squares can be used to deduce the significance of adding or dropping terms from the index formula.

The model for PSI is linear in that if all functions save one are given a numerical value, then PSI versus the remaining function represents a straightline relationship. For this reason it is desirable to choose functions R_1, $R_2, \ldots$, D_1, $D_2, \ldots$, that have linear graphs when plotted with PSR values. For example, logarithms and powers of the original measurements may be used as linearizing transformations.

A present serviceability index developed from observed ratings and measurements can only reflect the characteristics that were actually present in the observed pavements. For any particular characteristic, the index can only reflect the observed range of values for the characteristic. For example, if the selected pavements had no potholes, there is no objective way to infer how potholing would affect the present serviceability ratings, and the index cannot contain a function of potholing. As another example, if faulting in the selected pavements ranged from 0 to 10, there would be no way to infer the effect on PSR of pavements whose faulting was in the range 50 to 100.* This same argument applies to the present serviceability ratings themselves. If PSR's for the selected pavements range only from 2.0 to 4.0, there is no way to infer what pavement characteristics must be like in order to produce a value of 1.0 or 5.0, except to extrapolate the index on the assumption that linearity holds over the full range of pavement characteristics.

* It was for this reason that it was not possible to determine the effect of rutting in flexible pavements after the first three rating sessions which included pavements with rutting ranging from 0 to only 0.37 in. Thus the fourth rating session was necessary to determine the effect of ruts in the range of 0.5 to 1.0 in. deep.

For these reasons it has been stated that selected pavements should show all phenomena of interest, the complete range of interest for each phenomenon, and should be associated with PSR values that span the full range of interest. Therefore, pavement selection amounts to the assumption that all interesting phenomena and ranges have been encompassed by the selections. Extrapolations of the index to measured values outside the range of those found in the selected pavements amounts to the assumption that the index formula remains linear in the region of extrapolation.

Choice of Functions for the Present Serviceability Index

Measurements from the Illinois and Minnesota sections were plotted in succession against corresponding PSR values to determine which measurements were essentially uncorrelated with PSR and to deduce the need for linearizing transformations. It was indicated that the mean wheelpath slope variance, $\overline{SV}$ was highly correlated with PSR, though curvilinearly. Figures 11–F and 12–F show the nature of this correlation for all selected pavements. From several alternatives, the transformation

$$R_1 = \log (1 + \overline{SV}) \qquad (3\text{–F})$$

was selected as the first function of profile roughness to appear in the PSI model for both flexible and rigid pavements. The result of this transformation is shown in Figures 13–F and

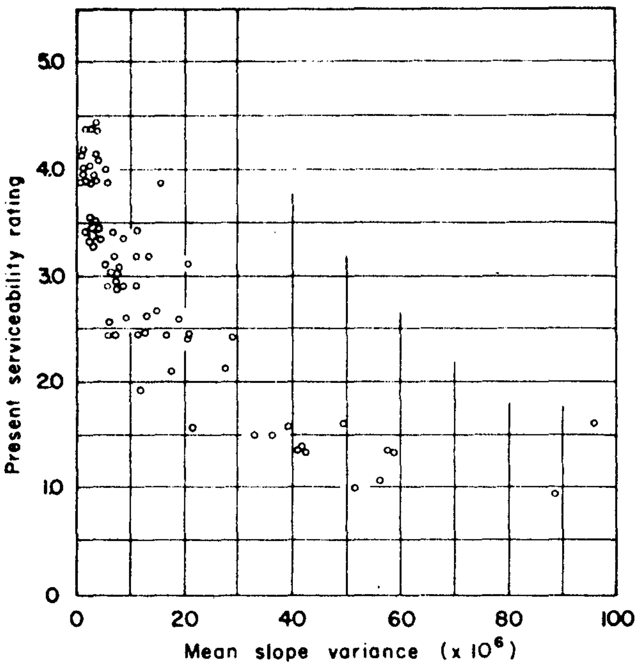


Figure 11–F. Present serviceability rating vs slope variance; 74 flexible pavements.

14–F where PSR values are plotted against R_1 for flexible and rigid pavements, respectively.

For the flexible pavements, mean wheelpath rut depth, $\overline{RD}$, was included as a second profile measurement to appear in the PSI equation. The selected function of rut depth was

$$R_2 = \overline{RD}^2 \qquad (4\text{–F})$$

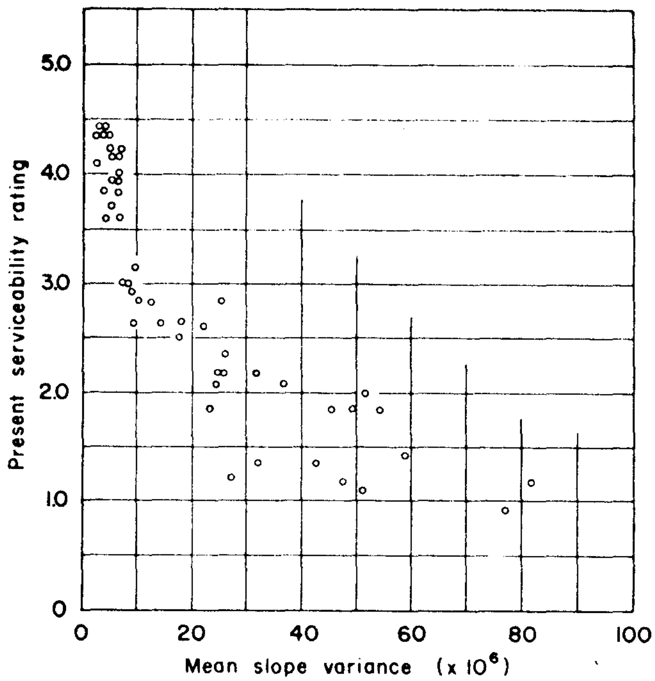


Figure 12–F. Present serviceability rating vs slope variance; 49 rigid pavements.

The scatter diagram of PSR vs $\overline{RD}^2$ is shown in Figure 15–F.

Although preliminary analyses considered the possibility of several functions of surface deterioration, for example, one function for each of the measured manifestations, it was ap-

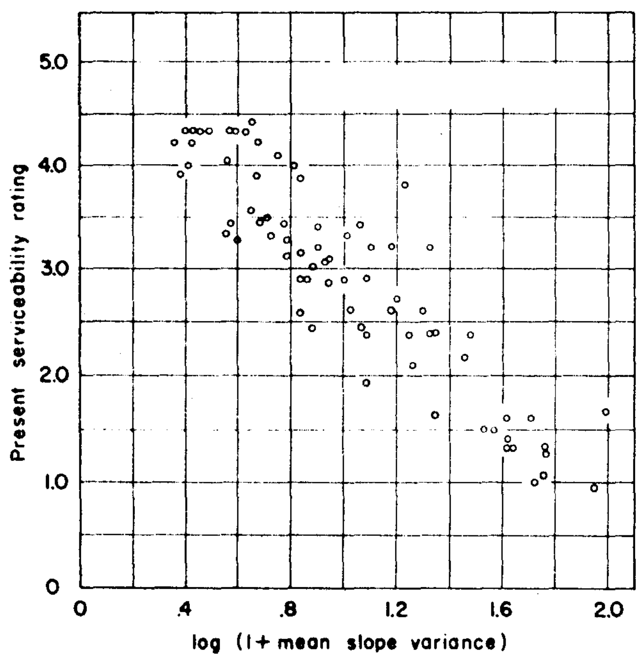


Figure 13–F. Present serviceability rating vs log (1 + mean slope variance); 74 flexible pavements.

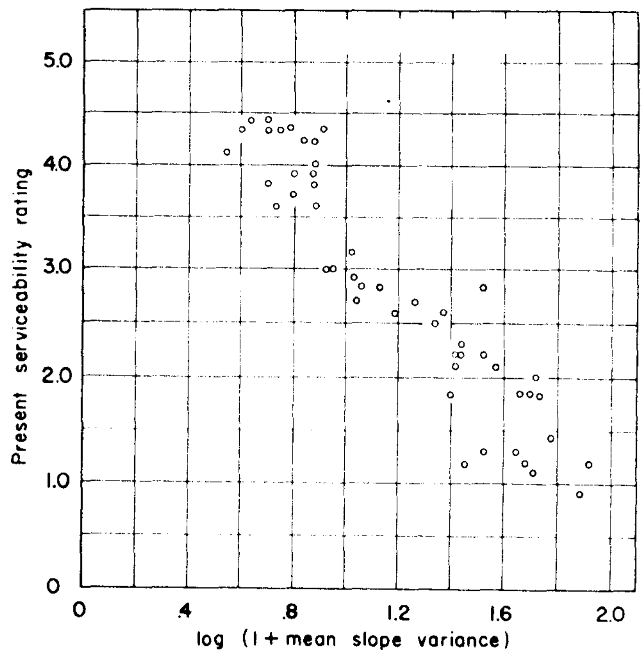


Figure 14–F. Present serviceability rating vs log (1 + mean slope variance); 49 rigid pavements.

parent that no loss would be incurred by lumping all major cracking and patching into a single number to represent surface deteriorations. Values for $C + P$ are not shown in Tables 1–F and 2–F, but may be obtained from the cracking and patching measurements. Scatter diagrams for the PSR versus $C + P$ are shown in Figures 15–F and 16–F.

For whatever reasons, it is apparent that there is little correlation between PSR and $C + P$ for the flexible pavements, but that a fair degree of correlation exists between these variables for the rigid pavements. For both flexible and rigid pavements the transformation

$$D_1 = \sqrt{C + P} \tag{5–F}$$

was selected as a linearizing transformation for $C + P$ (Figs. 17–F and 18–F).

Thus the present serviceability index models to be used are

For flexible pavements:

$$\text{PSI} = A_0 + A_1 R_1 + A_2 R_2 + B_1 D_1 = A_0 + A_1 \log (1 + \overline{SV}) + A_2 \overline{RD}^2 + B_1 \sqrt{C + P} \tag{6–F}$$

For rigid pavements:

$$\text{PSI} = A_0 + A_1 R_1 + B_1 D_1 = A_0 + A_1 \log (1 + \overline{SV}) + B_1 \sqrt{C + P} \tag{7–F}$$

It is not expected that the coefficients A_0, A_1, and B_1 have the same values for both equations.

There are many other possibilities for Eqs. 6–F and 7–F—other instruments might be used to detect deformation and deterioration, and summary values other than $\overline{SV}$, $C + P$ and $\overline{RD}$ might be used. Moreover, different functions of $\overline{SV}$, $C + P$ and $\overline{RD}$ could be chosen, or more functions of pavement measurements could be included.

One of the most important elements of pavement serviceability is its longitudinal profile in the wheelpaths. The profile of the road coupled with the appropriate characteristics of the vehicle (mass, tires, springs, shock absorbers,

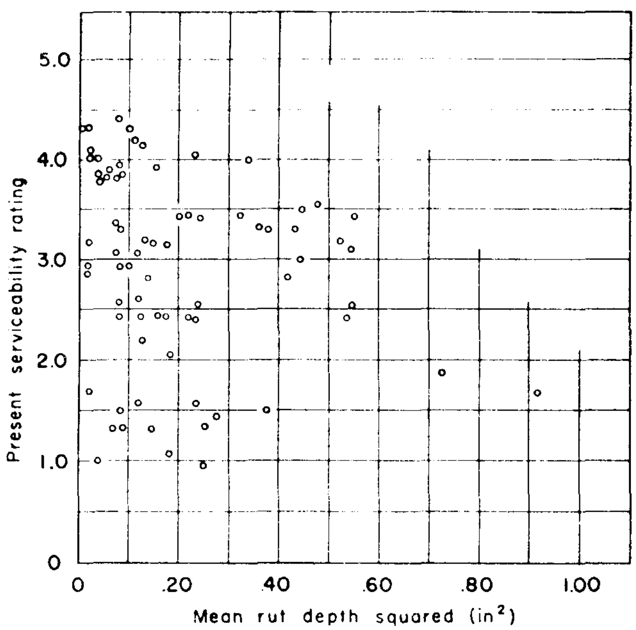


Figure 15–F. Present serviceability rating vs mean depth squared; 74 flexible pavements.

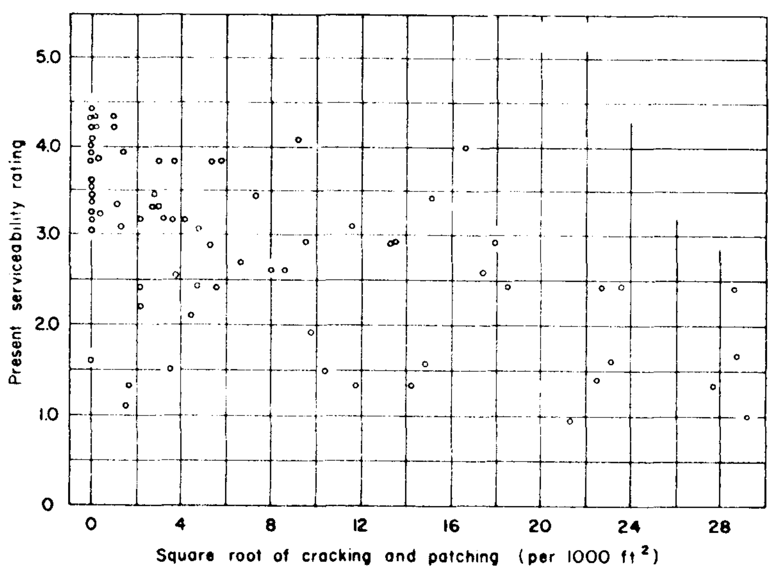


Figure 16–F. Present serviceability rating vs square root cracking and patching; 74 flexible pavements.

speed, etc.) produce the "ride" attained in that vehicle over that road. The actual profile of the wheelpath as though taken with rod and level at very close spacing is called the displacement profile, p. The first derivative of the displacement profile is the profile of the slope, p'. A plot of the slope profile has the same abscissa (distance along the road) as the displacement profile and its ordinate represents the rate of change of displacement, or slope of the road at any point. The second derivative of the displacement profile is the "acceleration" profile, p'', and represents the rate of change of slope, and the third derivative is the "jerk" profile, p''', the rate of change of acceleration. It has been suggested that jerk may be more highly

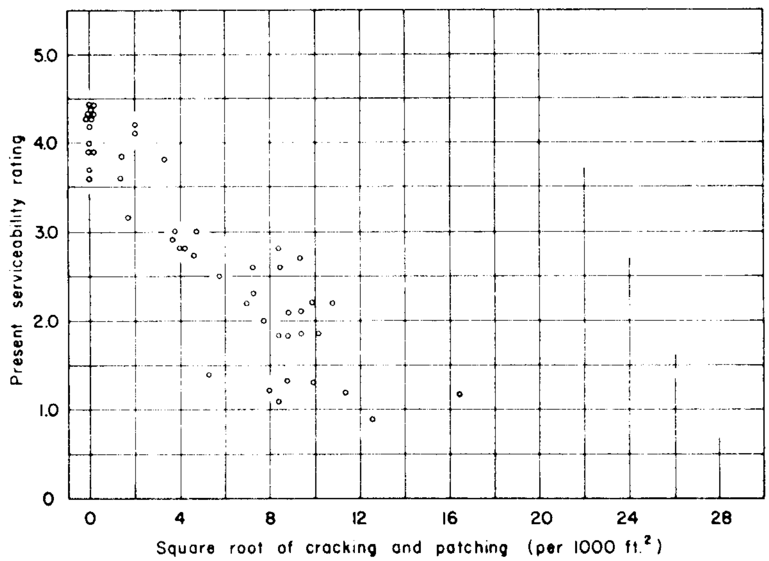


Figure 17–F. Present serviceability rating vs square root cracking and patching; 49 rigid pavements.

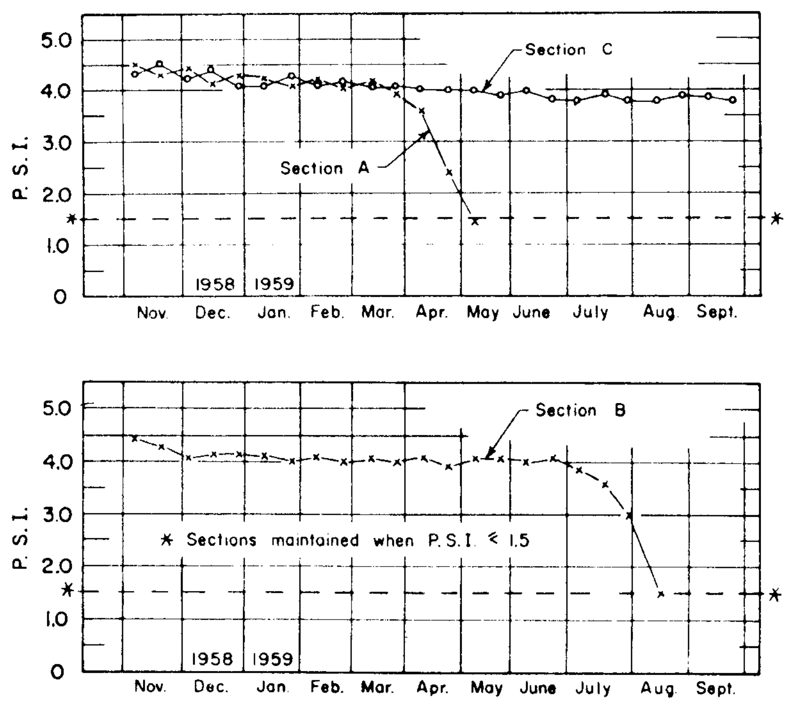


Figure 18–F. Present serviceability history of three selected test sections on the AASHO Road Test.

correlated with a rider's opinion of his ride than any of the other representations. Perhaps this is true if one is seeking to define "ride"—but the efforts at the Road Test were directed towards a definition of the "smoothness of a road" independent of the vehicle that might use it. Considerable effort was spent in studying correlations of the variances of various profile derivatives with the present serviceability ratings, but there was no evidence that elevation variance, acceleration variance, or jerk variance has higher correlation with PSR than the slope variance. On the other hand, when a number of the slope profiles were subjected to generalized harmonic analysis to determine how variance was associated with the wavelength spectrum, there was some indication that slope variance in certain regions of the wavelength spectrum is more highly correlated with PSR than is the total slope variance.

Coefficients for the Present Serviceability Index

Substitution of Eq. 6–F into Eq. 2–F gives for flexible pavements

$$\text{PSR}_j = A_0 + A_1R_{1j} + A_2R_{2j} + B_1D_{1j} + E_j \quad (8\text{–F})$$

in which

$R_{1j} = \log(1 + \overline{SV}_j)$, $R_{2j} = RD_j^2$ and $D_{1j} = \sqrt{C_j + P_j}$ for the j^{th} pavement.

Least squares estimates for A_0, A_1, A_2 and B_1 are found by minimizing the sum of squared residuals, E_j, through solving four simultaneous equations for A_0, A_1, A_2 and B_1. The solution of these equations gives the index

$$\text{PSI} = 5.03 - 1.91 \log(1 + \overline{SV}) - 1.38\,\overline{RD}^2 - 0.01\sqrt{C + P} \quad (9\text{–F})$$

Because the model for rigid pavement (Eq. 7–F) has only three undetermined coefficients, only three simultaneous equations need be solved. Their solution gives the index

$$\text{PSI} = 5.41 - 1.78 \log(1 + \overline{SV}) - 0.09\sqrt{C + P} \quad (10\text{–F})$$

The multiple squared correlation coefficients for these derivations are $r^2 = 0.844$ for the flexible pavements, and $r^2 = 0.916$ for the rigid pavements.

Therefore, the PSI formulas account for 84.4 percent and 91.6 percent of the variation in PSR for flexible and rigid pavements, respectively. The respective root mean square residuals are about 0.38 and 0.32, respectively.

The last columns of Tables 1–F and 2–F show calculated values for the present serviceability indexes as well as for residuals. At the bottom of the last column, the mean residual was 0.30 for flexible pavements and 0.26 for rigid pavements. In both cases, the mean residual is about twice the mean difference between replicate ratings given by the AASHO rating panel.

From the residual columns, six flexible and three rigid pavement residuals exceeded 0.5, the largest replication difference given by the panel. However, the index formulas span ratings made more than a year apart whereas all replicate ratings were made on successive days. As stated before, it is quite possible that replicate PSR's would be more different when made over longer intervals of time.

When the 15 rigid pavement PSR values from the fourth rating session were compared with PSI values given by Eq. 10–F, the sum of the algebraic deviations was practically 0 whereas mean discrepancy was 0.3. Since only two of the deviations exceeded 0.5, it was inferred that Eq. 10–F fitted the new PSR values to about the same degree as it predicted those from which it was derived.

INSTRUCTIONAL TEXT B

THE PAVEMENT SERVICEABILITY - PERFORMANCE CONCEPT

The Importance of Pavement Serviceability

The primary operating characteristic of a pavement is the level of service it provides to the users, both today and in the future. It is important to: a) measure or evaluate this level of service to establish the current status of a pavement, and b) to predict the change of level of service in the future, for either an existing pavement or for a pavement to be constructed.

This level of service, or more simply, serviceability, can change slowly or relatively quickly with time, depending on such factors as traffic, type and thickness of structure, surface distress, original construction quality, climatic factors, type and degree of maintenance performed, etc. When the change of serviceability over time is considered, we refer to it as performance.

Development of the Serviceability Concept

The evaluation of pavement performance involves a study of the functional behavior of a length of pavement in its entirety. For a functional behavior or performance analysis, information is needed on the history of the riding quality of the pavement section for a period of time and the associated traffic during that time. This can be determined by periodic observations and measurements of the pavement riding quality coupled with records of traffic history and time. It is this history of deterioration of the riding quality or function of the pavement that defines pavement performance as shown in Fig. 4.1.

Until a measure of pavement serviceability was developed in conjunc-

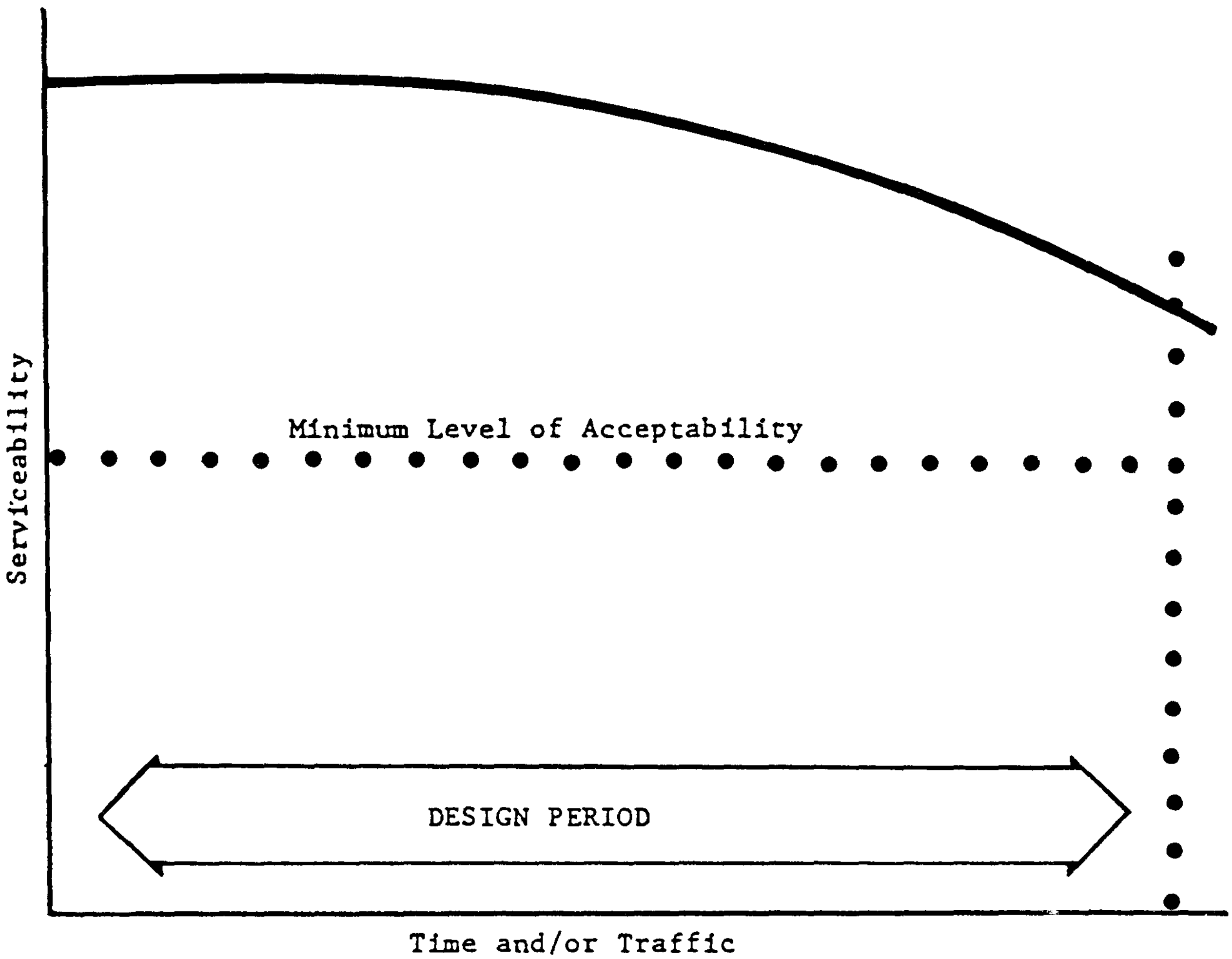


Figure 4.1 The serviceability-performance concept as developed by Carey and Irick (1).

tion with the AASHO Road Test (10), little attention was paid to evaluation of pavement performance per se. A pavement was either satisfactory or unsatisfactory (i.e., in need of repair or replacement). The ideas of "relative" performance were not adequately developed. Most pavement design concepts in general use did not consider the level of performance desired. Design engineers as a group have varied widely in their concepts of desirable performance. As an example, suppose that two engineers are asked to design a pavement for a certain expected traffic history for 20 years. The first might consider the job properly done only if not a single crack occurred during the 20 years, whereas the second designer might be satisfied if the last predicted application was able to pass safely over the pavement before total collapse at the end of the twentieth year of life.

Many popular design systems involve determination of the pavement thickness required to hold certain computed stresses or strains below some specified levels. It is clear that cracks will occur if the pavement is overstressed, but not much information was available prior to the time of the AASHO Road Test to relate such cracks to functional behavior. Thus a method of performance evaluation was badly needed for use in the pavement field at the time of the AASHO Road Test, and it was fulfilled with the "serviceability-performance concept," developed by Carey and Irick (1). This concept, first used at the AASHO Road Test, is a well-defined technique for evaluating pavement performance, as subsequently discussed in more detail.

Serviceability must be defined relative to the purpose for which the pavement is constructed, that is, to give a smooth, comfortable, and safe ride. In other words the measurement should relate explicitly to the user, who is influenced by several attributes of the pavement, including the following:

1. Response to motion as characterized by the particular pavement-vehicle-human interaction for a particular speed

2. Response to appearance, as characterized by such factors as cracking and patching, color, shoulder condition, etc.

Concept of Ratings

A rating procedure requires the construction of some type of arbitrary scale to be used in the rating. Teachers often rate students on a scale of 0 to 100 percent; amateur golfers are rated by an arbitrary system called a handicap, which is derived as a percentage of their average score over par for a period of time. A large number of such arbitrary scales are in use today and could be cited as examples. For many years, the "roughness index" was used as a rating scale for pavements. This roughness index is rather arbitrary, and a "good" value depends largely on the particular piece of equipment used in the evaluation.

If some absolute roughness standard were available, this problem would be minimized. It is not likely, however, that such an absolute standard will ever be developed. As a result, "scaling factors" have been developed to provide a basis for comparing ratings from many sources.

Hutchinson (3) has presented some of the basic considerations associated with subjective ratings. Care must be taken in the development of such rating schemes, and improved rating scales can no doubt be developed if additional attention is given to this subject.

The evaluation of riding quality is a complex problem, depending on three separate components: the pavement user, the vehicle and the pavement roughness, plus interactions among them. Hutchinson has described the problems associated with analyzing the subjective experience of highway users in deriving an absolute measure of riding quality. These require: (1) the development of a suitable mathematical model to characterize pavement roughness, (2) the development of a suitable mathematical model to describe the suspension characteristics of highway vehicles that may be used along with the roughness model to predict the dynamic response of vehicles, and (3) a quantitative knowledge of the response of human beings to motion.

In order to improve our subjective rating systems it will be necessary to evaluate objectively human sensibilities including the effect of motion sickness and its causes. These will involve studies of frequency, wavelength, and amplitude of roughness input parameters.

Development of a Serviceability Index

The WASHO Test Road in the early 1950's proved to be especially difficult with respect to establishing a failure condition for the pavement sections subjected to the test traffic. As a result of these difficulties, the idea of subjectively established average pavement ratings to measure serviceability was developed by Carey and Irick (1). They stated that there are five fundamental assumptions associated with the pavement serviceability concept, which may be summarized as follows:

1. Highways are for the comfort and convenience of the traveling public.

Stated another way, a good highway is one that is safe and smooth.

2. Users' opinions as to how they are being served by highways is by-and-large subjective.

3. There are, however, characteristics of highways that can be measured objectively and that, when properly weighed and combined, are in fact related to users' subjective evaluation of the ability of the highway to serve them.

4. The serviceability of a given highway may be expressed by the mean evaluation given by all highway users. Honest differences of opinion preclude the use of a single opinion in establishing

serviceability ratings. The mean evaluation of all users, however, should be a good measure of highway serviceability.

5. Performance is assumed to be an overall appraisal of the serviceability history of a pavement. Thus it is assumed that the performance of a pavement can be described if one can observe its serviceability from the time it was built up until the time its performance evaluation is desired.

Based on these fundamental assumptions, Carey and Irick developed the Present Serviceability Index (PSI) measure used at the AASHO Road Test. They showed that pavement roughness can be closely related to ratings of serviceability. Furthermore, the AASHO Road Test (10) showed that pavement performance, in terms of the history of the serviceability index, can be correlated with certain pavement design factors. A similar well-known technique was developed in the studies conducted by the Pavement Design and Evaluation Committee of the Canadian Good Roads Association (which became the Roads and Transportation Association of Canada in 1971) in the late 1950's, and early 1960's (6-9).

These serviceability measures are supposed to simulate users' opinions or evaluations, which are subjective, of the riding quality provided by the pavement. In the AASHO and Canadian studies, procedures for obtaining user-simulated opinions were developed by constituting rating panels and having the members of these panels drive over a number of pavement sections. Certain "ground rules" were established for these ratings sessions, as described in Refs. (1, 10, 12). Each panel member records his or her independent, subjective opinion on the type of form shown in Fig. 4.2. The AASHO terminology for each such rating is Individual Present Serviceability Rating, with the mean of the individual ratings termed as Present Serviceability Rating (PSR). The Canadian equivalent was originally termed Present Performance Rating but was changed in 1968 to Riding Comfort Index (RCI) to denote more explicitly the evaluation of pavement riding quality only (12).

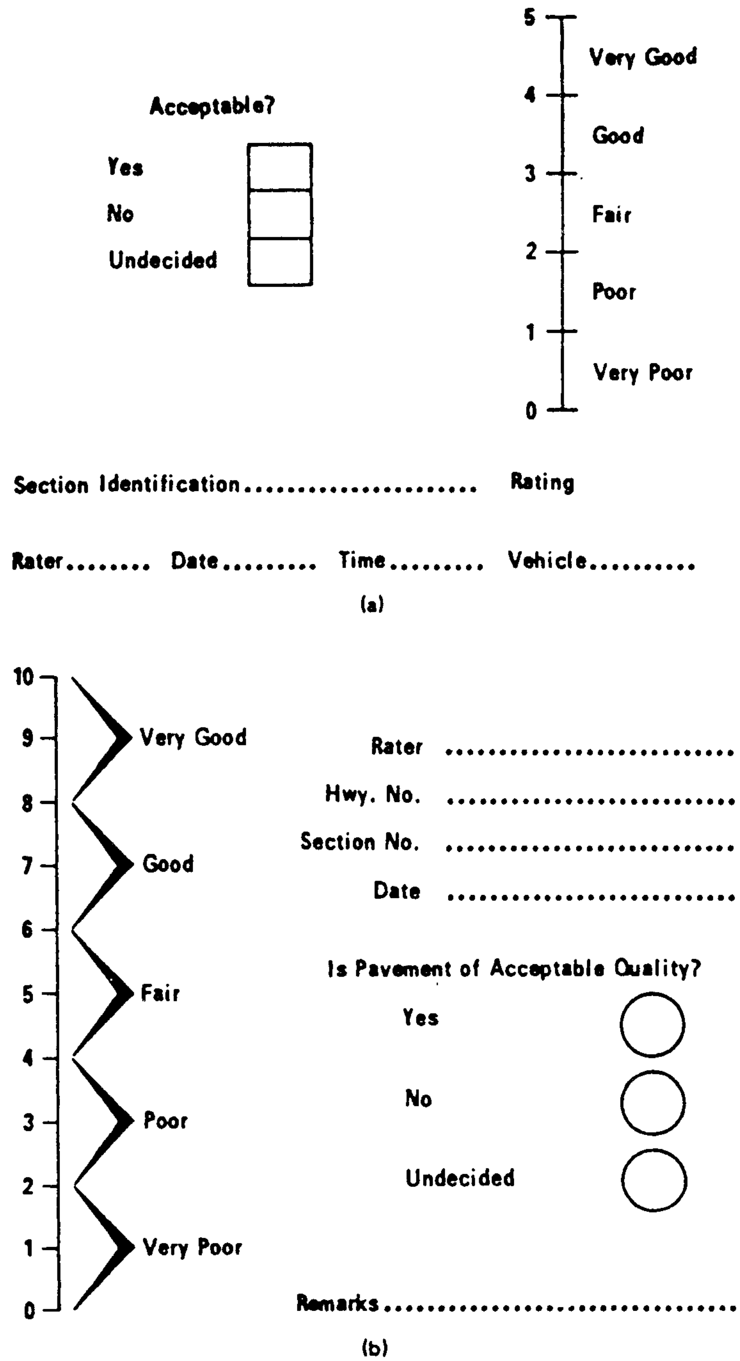


Figure 4.2 Evaluation forms for individual, subjective pavement ratings. (a) Individual Present Serviceability Rating (PSR) form used at AASHO Road Test. (b) Present Performance Rating (now Riding Comfort Index) form developed by the Canadian Good Roads Association (now the Roads and Transportation Association of Canada).

The major difference between the two approaches, as shown by comparing Fig. 4.2a and Fig. 4.2b is in the construction of the scales. There are five descriptive cues in each; however, the construction of the RCI scale means that it has 10 categories instead of 5. Both methods emphasize that only the descriptive words are to be given attention by the rater in judging a particular section and that an exact numerical rating will be scaled off later.

It is obviously impractical and expensive to evaluate serviceability on anything but a very limited basis using the rating panel method. Consequently, considerable effort has gone into correlating various mechanical measurements with these subjective ratings. The purpose of such efforts is to develop efficient, repeatable objective methods for estimating serviceability. Figures 4.3 through 4.7 illustrate various concepts regarding pavement performance and serviceability. These will be more fully discussed during the presentation of this session.

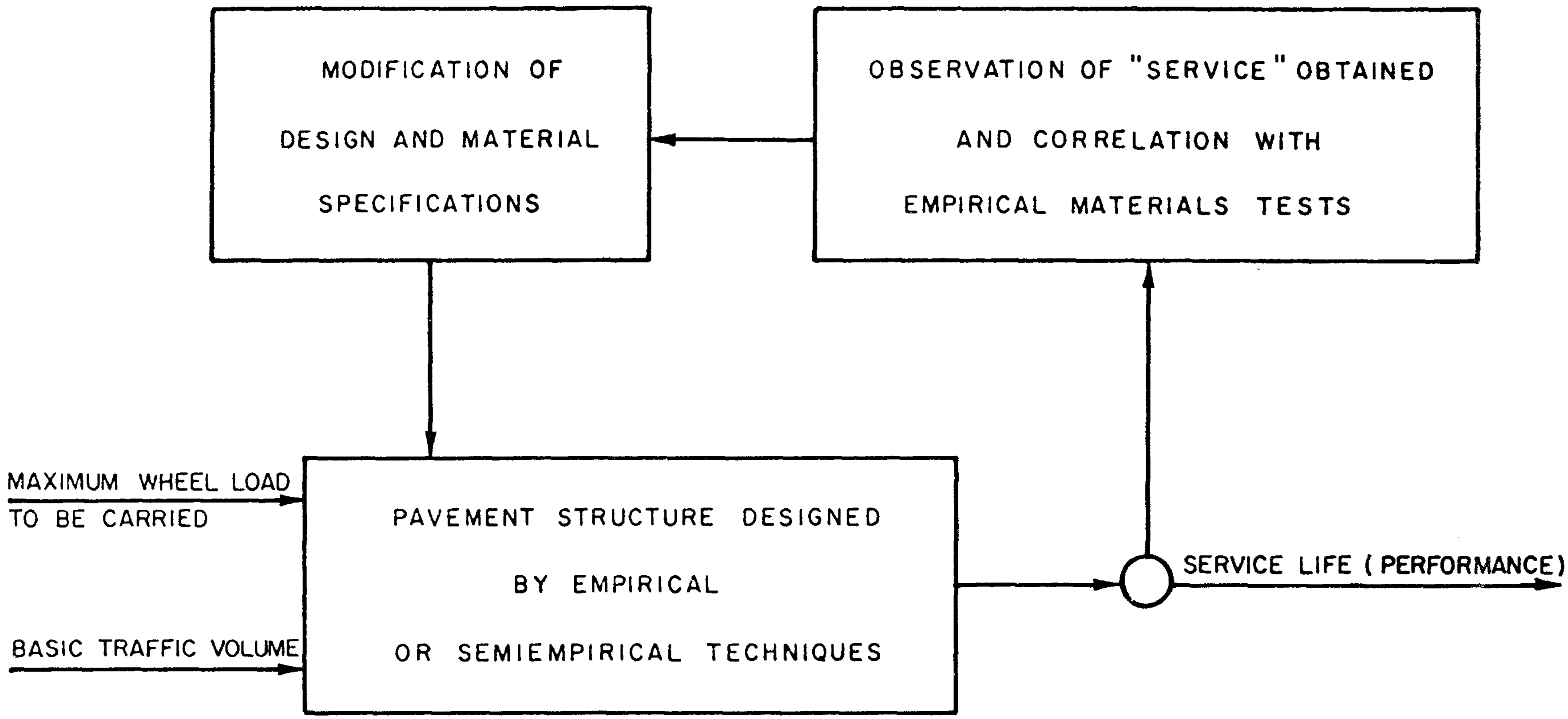
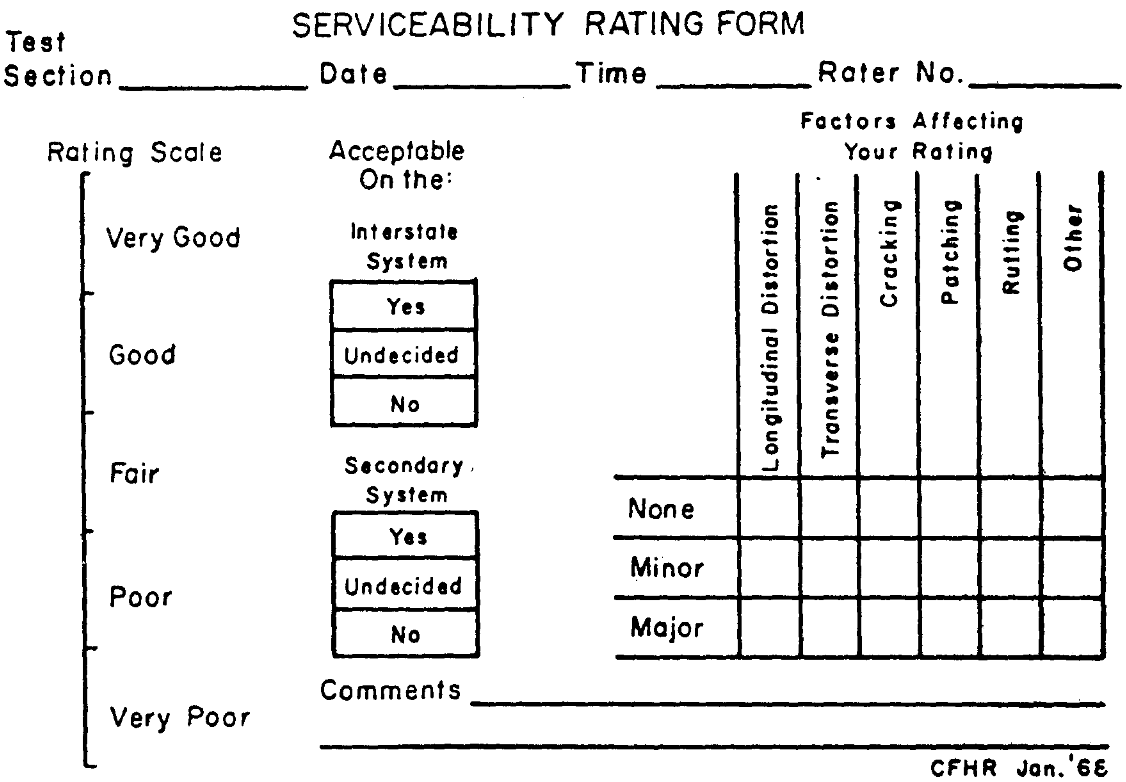


Figure 4.3 **Block diagram of current pavement design techniques**
Showing the Need for Performance Evaluation **(Slide S-1)**



SERVICEABILITY RATING FORM

Test Section __________ Date __________ Time __________ Rater No. __________

Rating Scale

- Very Good
- Good
- Fair
- Poor
- Very Poor

Acceptable On the:

Interstate System

Yes
Undecided
No

Secondary System

Yes
Undecided
No

Factors Affecting Your Rating

	Longitudinal Distortion	Transverse Distortion	Cracking	Patching	Rutting	Other
None						
Minor						
Major						

Comments ______________________________

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Figure 4.4 Rating form used by Texas Highway Department 1965-1968 (Slide S-21)

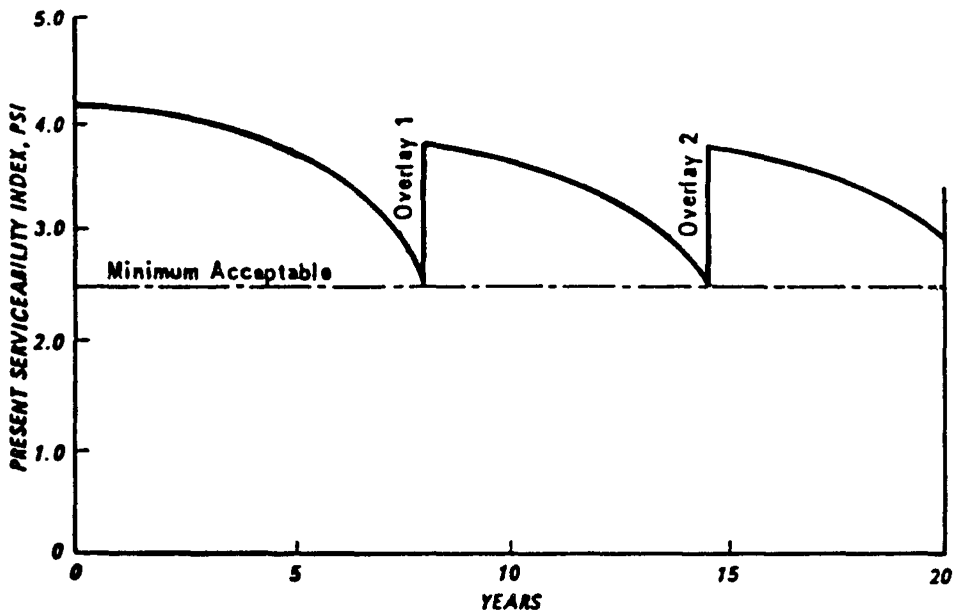

Figure 4.5 PSI history can easily account for overlays. (Slide S-27)

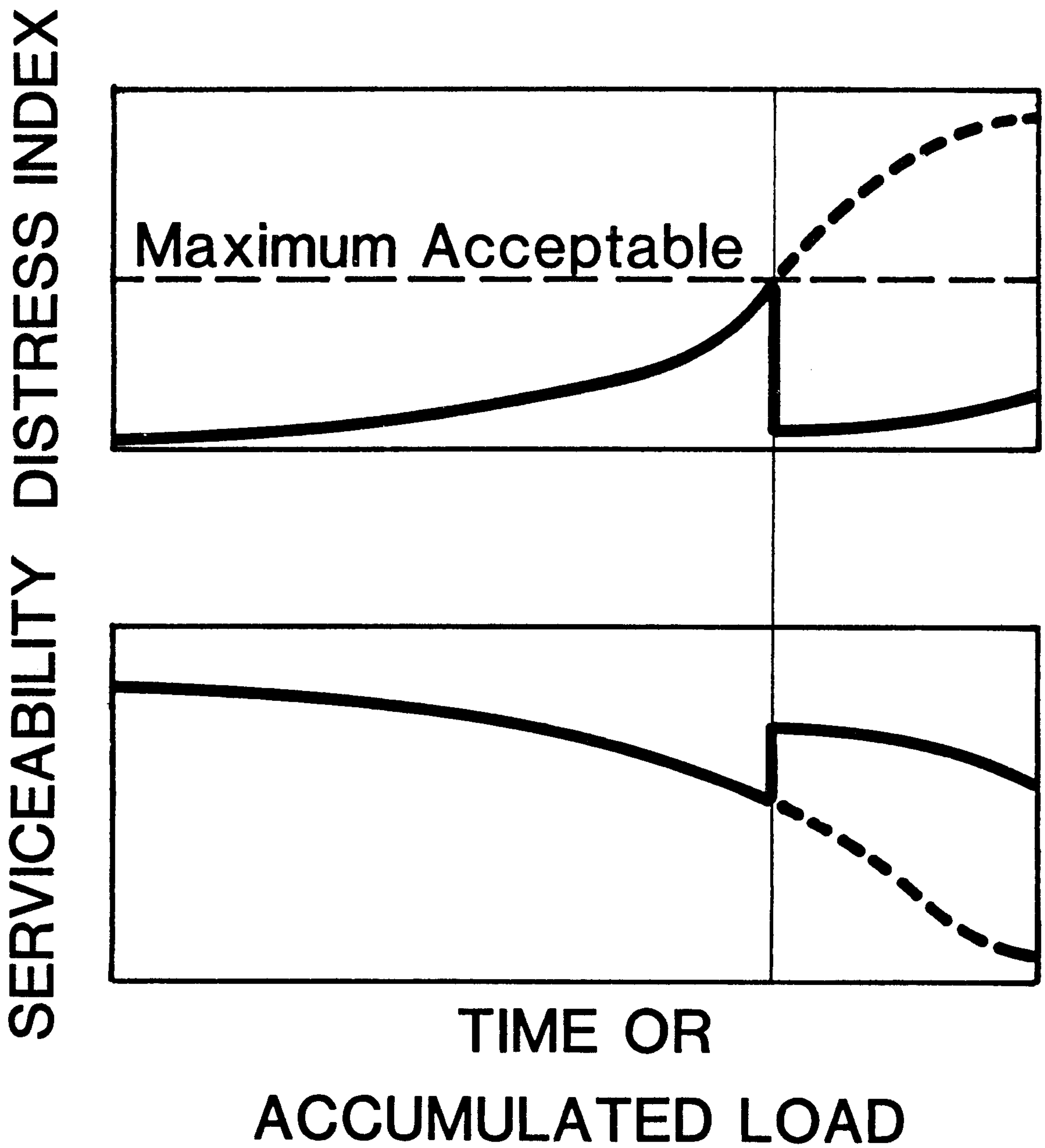

Figure 4.6 Serviceability does not replace distress evaluation (Slide S-47)

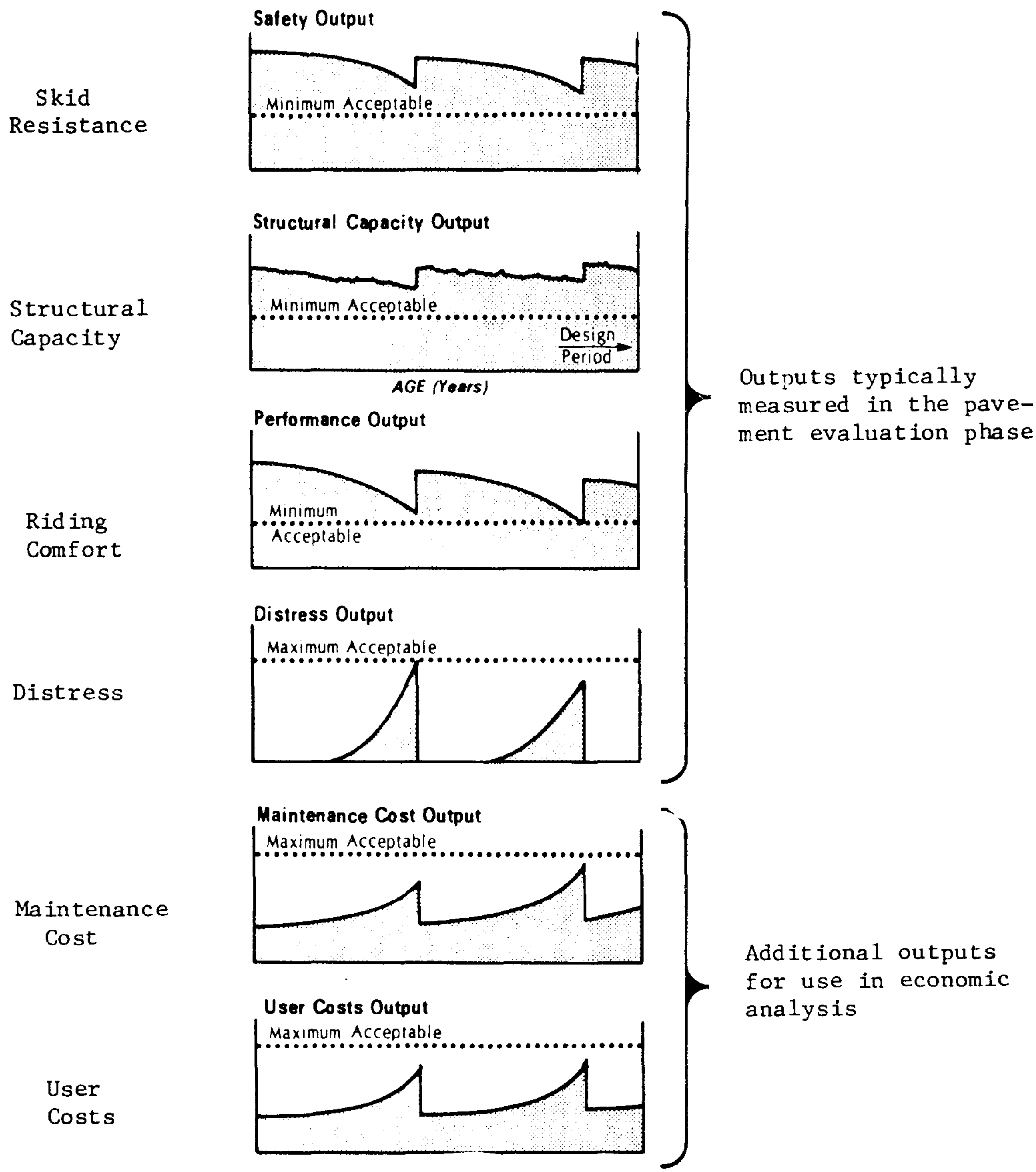

Figure 4.7 There are many evaluation factors in addition to PSI (Slide S-53)

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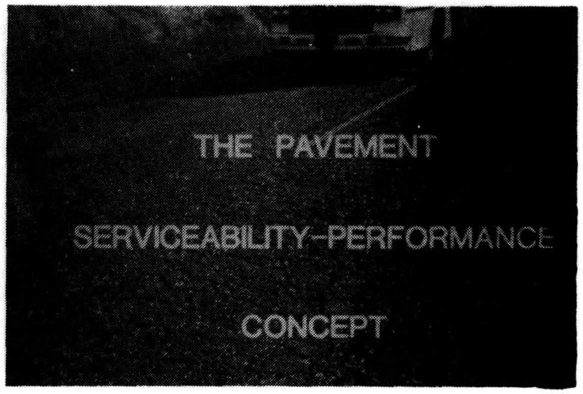
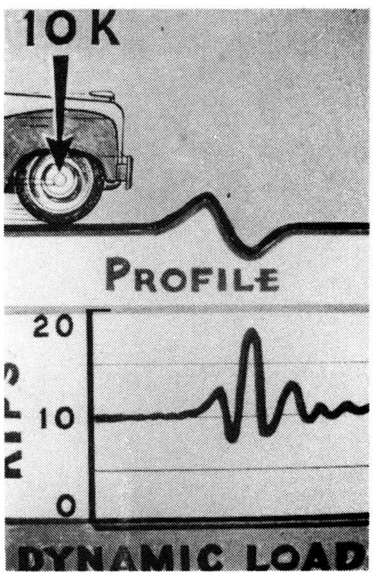
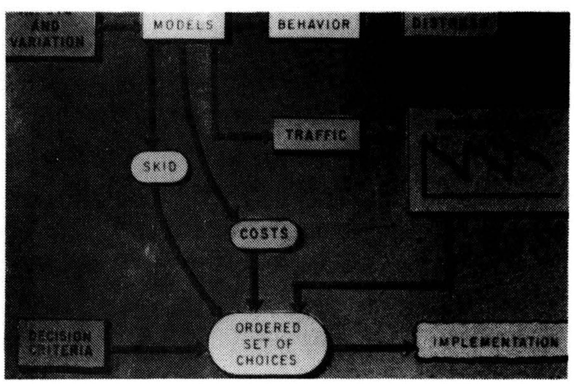


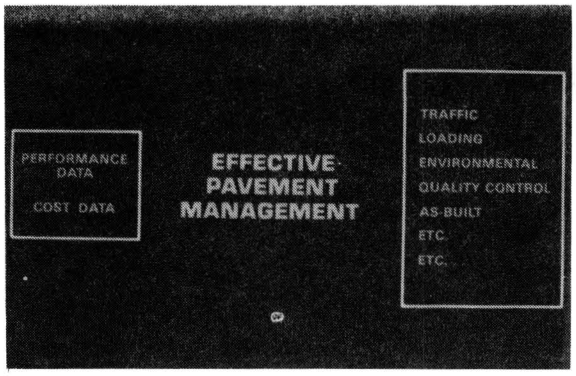
Slide 5.1. The Pavement - Service-**ability** Performance **Concept.**



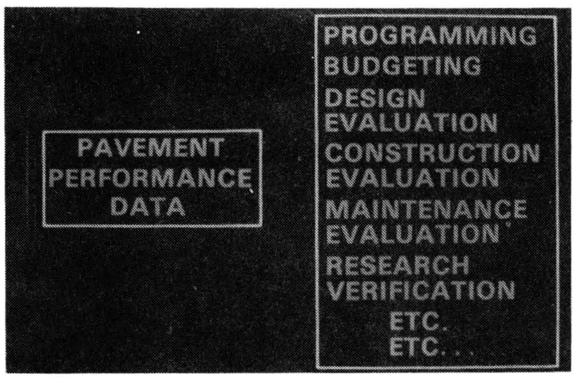
Slide 5.2. Effect of dynamic load **on pavement.**



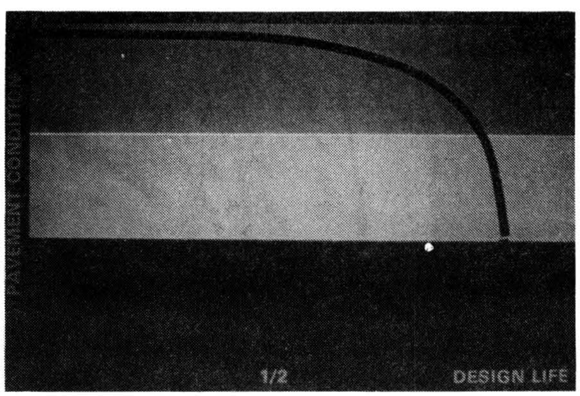
Slide 5.3. Performance evaluation.



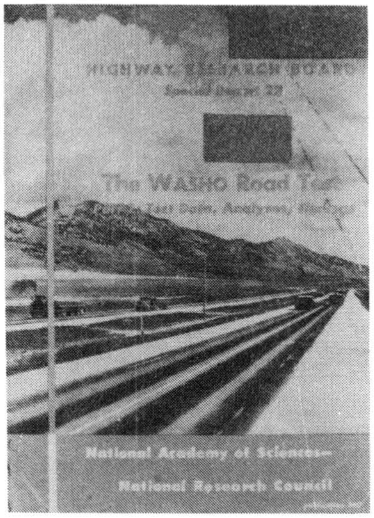
Slide 5.4. Important considerations for pavement management.



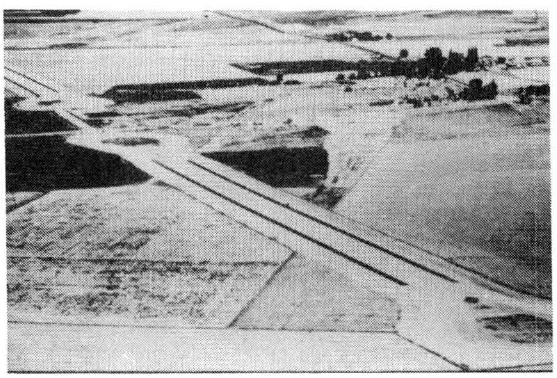
Slide 5.5. Pavement performance data.



Slide 5.6. Pavement condition versus age.

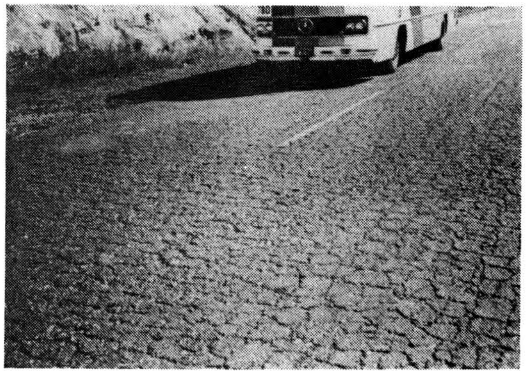


Slide 5.7. WASHO Road Test Report.



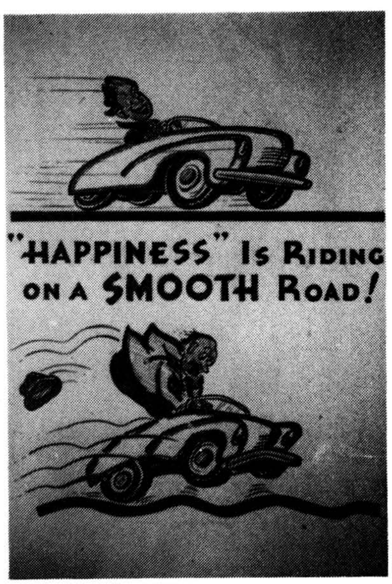
Slide 5.8. View of WASHO Road Test site.

Slide 5.9. Distress in pavement.

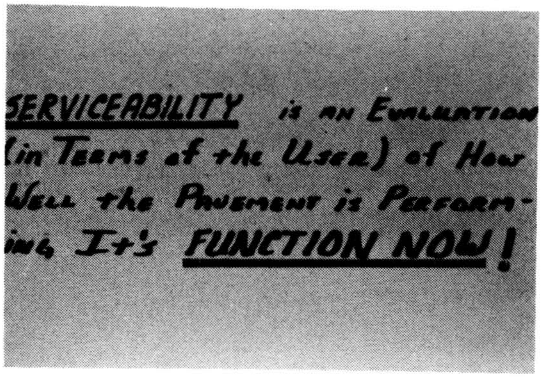


Slide 5.10. Example of distress in flexible pavement.

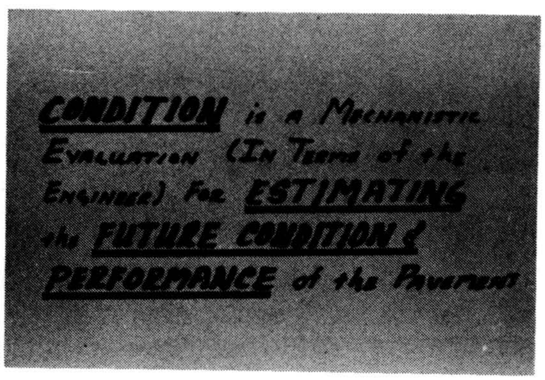
Slide 5.11. Example of distress in rigid pavement.



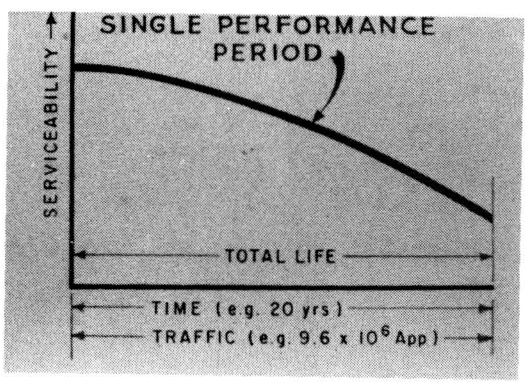
Slide 5.12. User's concept of riding comfort.



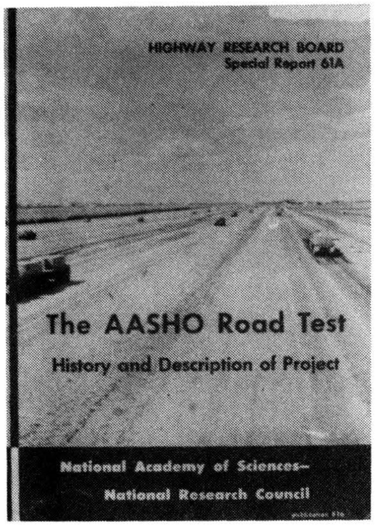
Slide 5.13. Concept of serviceability.



Slide 5.14. Definition of CONDIITON.

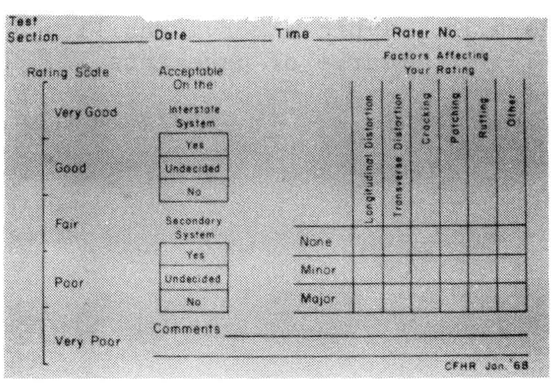


Slide 5.15. Typical serviceability curve of pavements.

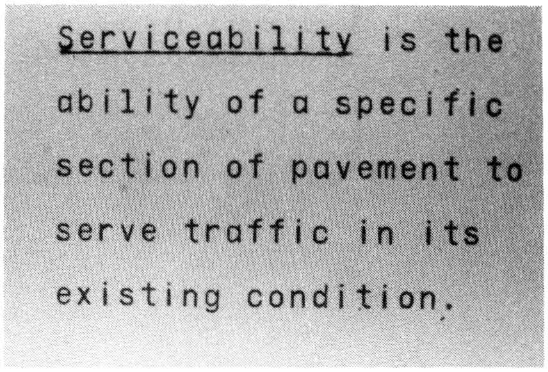


Slide 5.16. AASHO Road Test Report.

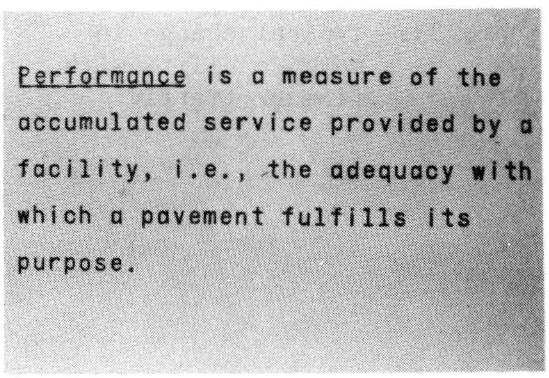
Slide 5.17, AASHO Road Test site.



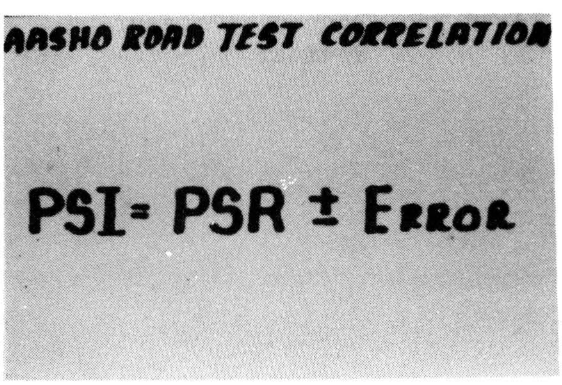
Slide 5.18, Serviceability rating form.



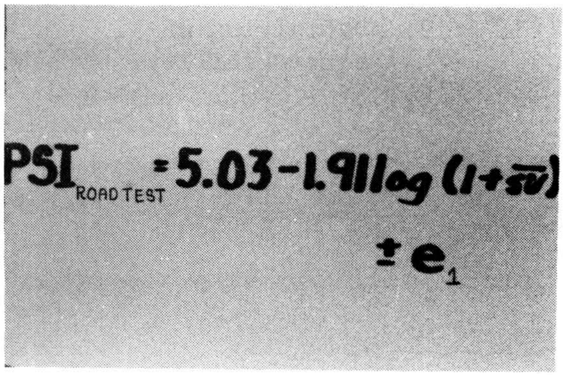
Slide 5.19. Definition of serviceability.



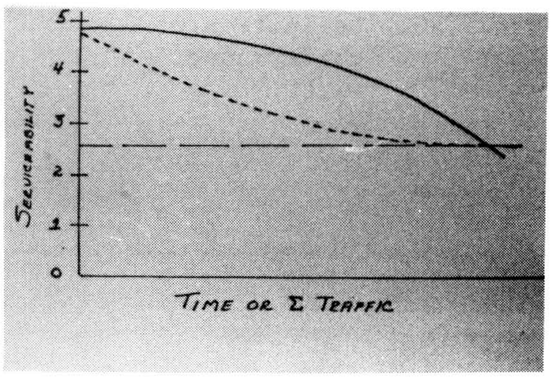
Slide 5.20. Definition of performance.



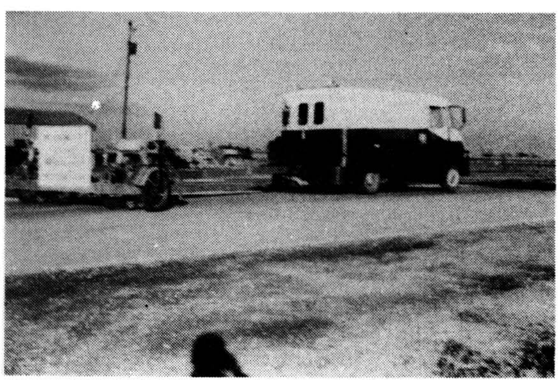
Slide 5.21. Serviceability - General model.



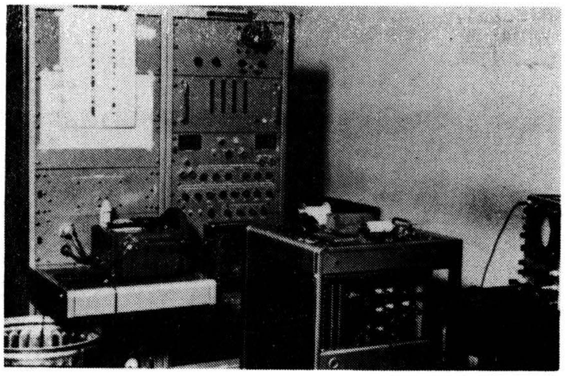
Slide 5.22. Serviceability - AASHO Road Test model.



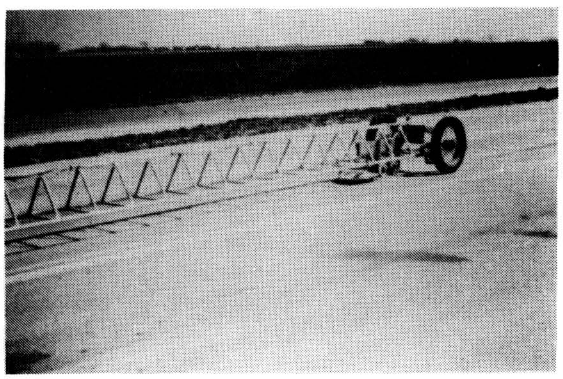
Slide 5.23. Typical change in serviceability with time or traffic.



Slide 5.24. Roughness measuring device.

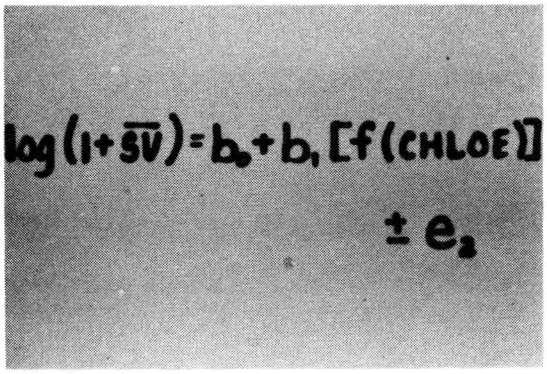


Slide 5.25. Data processing facility.

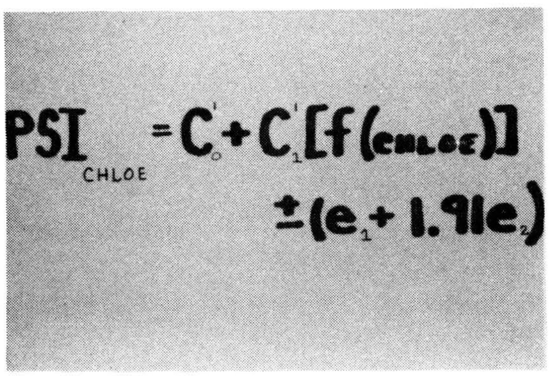


Slide 5.26. Roughness measuring device.

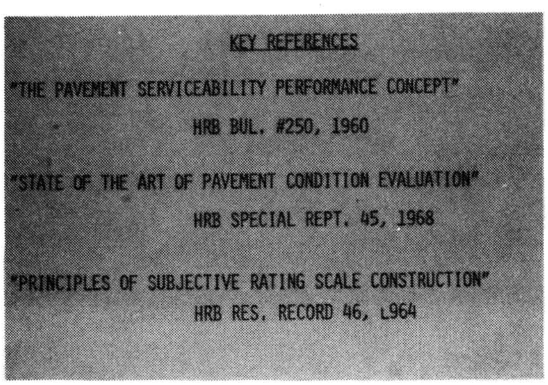
Slide 5.27. Roughness measuring device - close up view.



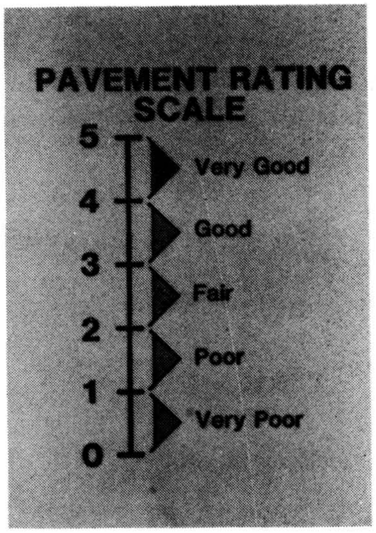
Slide 5.28. Correlation of slope variance with CHLOE profilometer.



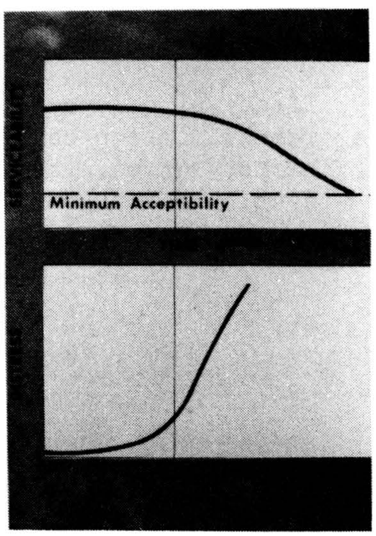
Slide 5.29. Serviceability and CHLOE measurement relationship.



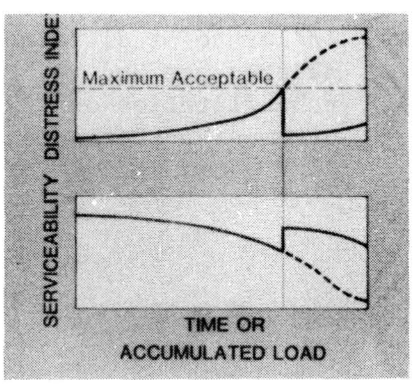
Slide 5.30. KEY references related to serviceability - performance concept.



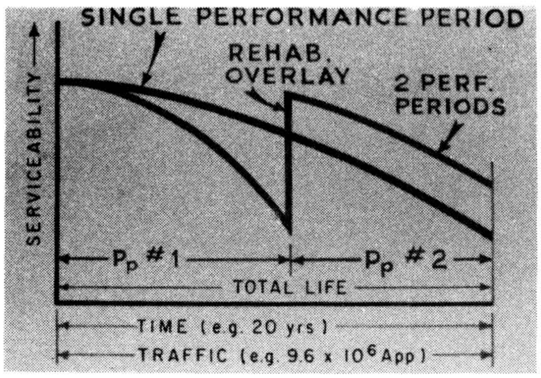
Slide 5.31. 5 point scale for pavement rating.



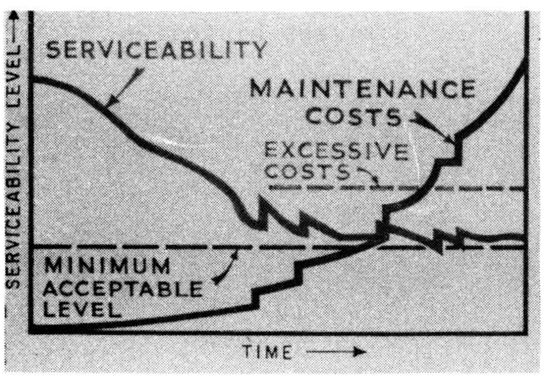
Slide 5.32. Serviceability and distress histories.



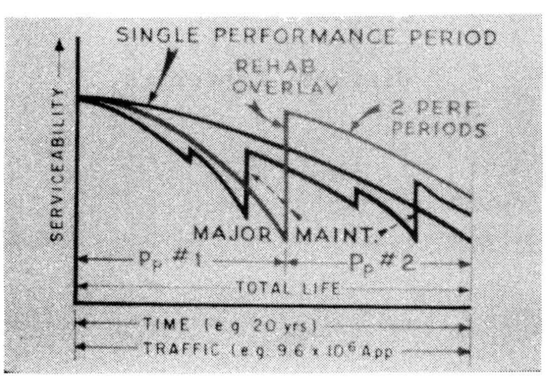
Slide 5.33. Effect of major rehabilitation on serviceability and distress histories.



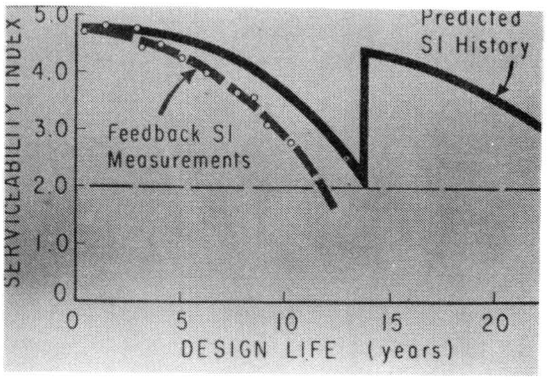
Slide 5.34. Influence of rehabilitation on performance.



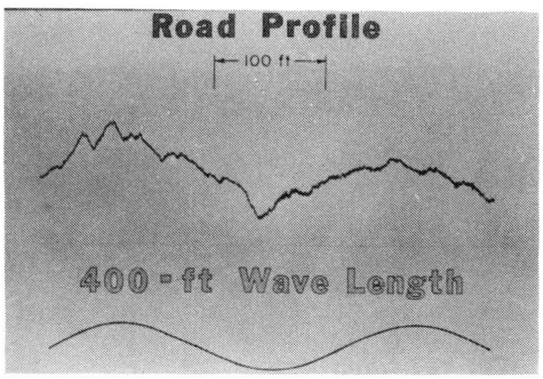
Slide 5.35. Effect of delayed rehabilitation on history of maintenance cost.



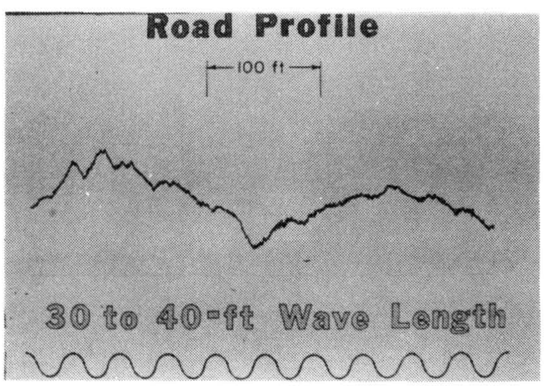
Slide 5.36. Influence of different maintenance and rehabilitation on pavement performance.



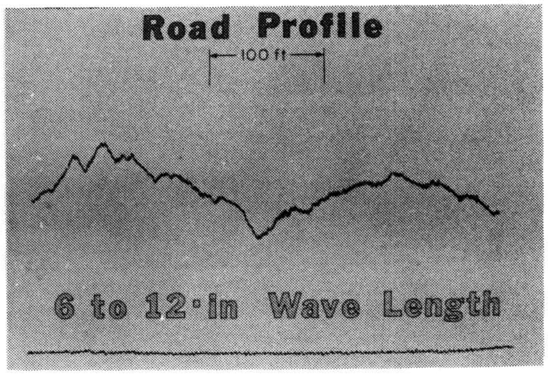
Slide 5.37. Predicted versus measured serviceability.



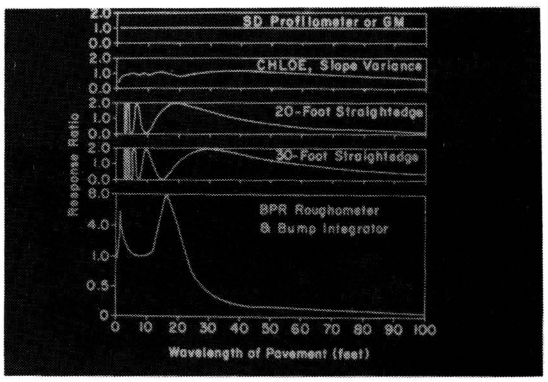
Slide 5.38. Illustration of long wave length.



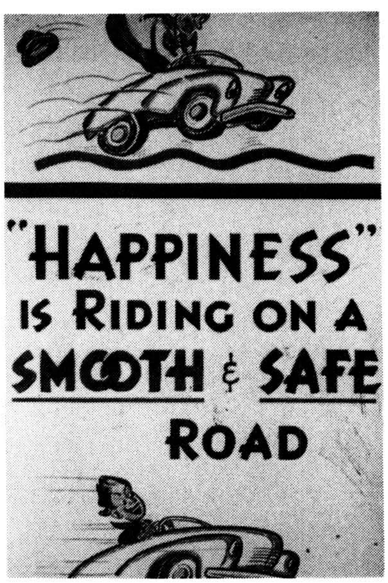
Slide 5.39. Illustration of medium wave length.



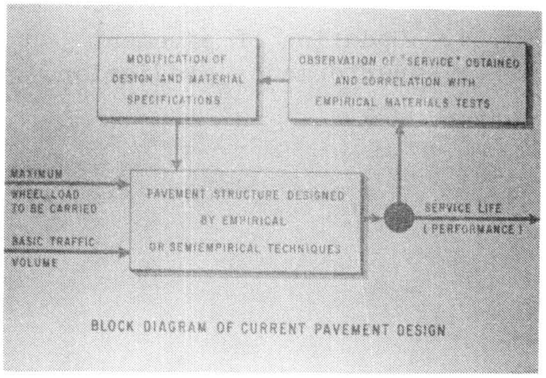
Slide 5.40. Illustration of short wave length.



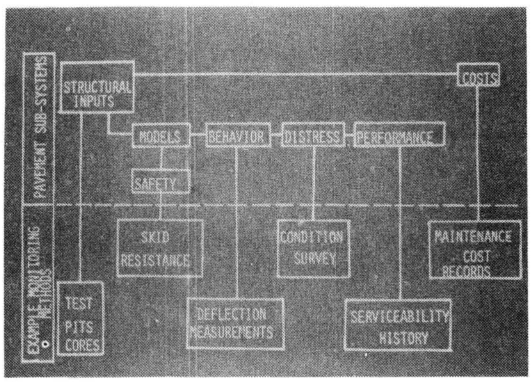
Slide 5.41. Response ratio versus wave length for different roughness measuring equipments.



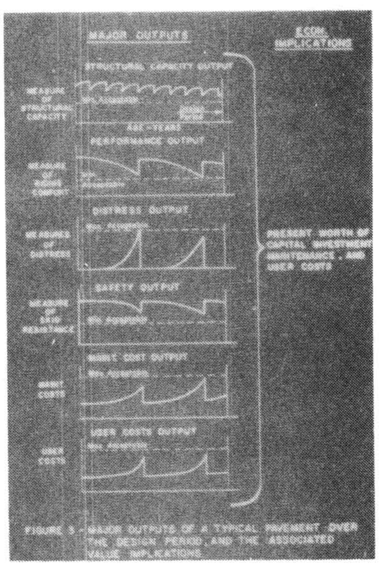
Slide 5.42. User's concept of pavement performance.



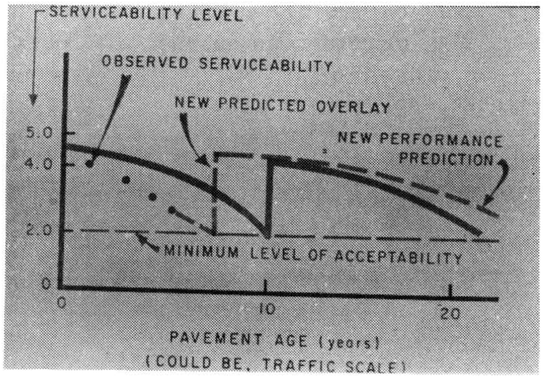
Slide 5.43. Block diagram of current pavement design.



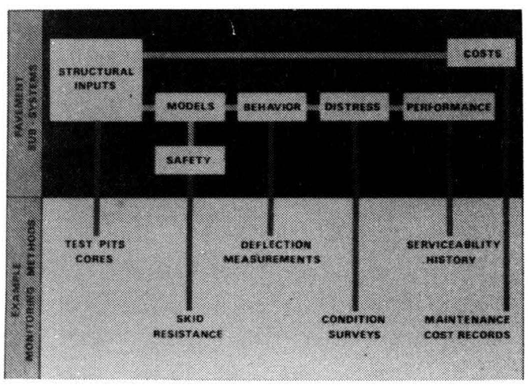
Slide 5.44. Monitoring of pavement subsystems.



Slide 5.45. Major outputs of a typical pavement over the design period.



Slide 5.46. Updating of pavement performance prediction.



Slide 5.47. Pavement monitoring.

LESSON OUTLINE
PAVEMENT LOAD CARRYING CONCEPTS

Lecture Objectives

1. To introduce the two basic definitions of the "rigid" and "flexible" pavements and to distinguish between the theoretical principles of each.

2. To explain the role of the base and subbase in the distribution of stresses and other functions in the pavement system.

Performance Objectives

1. The student should be able to explain the differences in the rigid and flexible pavement method of stress distribution.

2. The student should be able to explain the use of the base and subbase courses in both the rigid and flexible pavement systems.

Abbreviated Summary	Time Allocations, min.
1. Pavement Definitions	10
2. Subgrade Loading/Base and Subbase Course	15
3. Load Carrying Concept	15
4. Effect of Tire Pressure and Total Load	10
	50 minutes

Reading Assignment

1. Yoder & Witczak - Chapter 1 and Chapter 2, pages 72-77

2. Haas & Hudson - Chapter 13, pages 137-150

LESSON OUTLINE
PAVEMENT LOAD CARRYING CONCEPTS

1.0 INTRODUCTION

The primary function of a pavement is to serve the user in a safe, comfortable, and economic manner. In order to satisfy this function, the pavement must have adequate structural capacity under the influence of traffic loads and environmental factors.

1.1 FLexible Pavements

Classified as a pavement structure having a relatively thin asphalt wearing course with layers of granular base and subbase being used to protect the subgrade from being overstressed. This type of pavement design was primarily based upon empiricism or experience, with theory playing only a subordinate role in the procedure.

1.2 Rigid Pavements

Rigid pavements or Portland cement concrete pavement design has long been based primarily upon a theoretically related analysis involving some empirical modifications to the classical Westergaard approach.

1.3 Arbitrary Definitions

It should be obvious that the definitions "flexible" and "rigid" are arbitrary and were established to distinguish between asphalt and Portland cement concrete pavements.

1.3.1 Thick Asphalt. Asphalt pavements may possess as much stiffness as PCC pavements, by using stabilized pavement layers or thick asphalt layers.

1.3.2 Flexible Design. In the case of "rigid" asphalt pavements the classical methods of designing flexible pavements no longer apply.

1.4 Load Distribution Over Subgrade

The essential difference between the two types of pavements, flexible and rigid, is the manner in which they distribute the load over the subgrade.

1.4.1 Rigid Pavement. The rigid pavement, because of its rigidity and high modulus of elasticity, tends to distribute the load over a relatively wide area of the soil; thus, a major portion of the structure capacity is supplied by the slab itself. The major factor considered in the design of rigid pavements is the structural strength of the concrete. For this reason, minor variations in subgrade strength have little influence upon the structural capacity of the pavement.

1.4.2 Flexible Pavements. The load carrying capacity of a truly flexible pavement is brought about by load distributing characteristics of the layered systems. Flexible pavements consist of a series of layers with the highest quality materials at or near the surface. Hence, the strength of a flexible pavement is the result of building up of thick layers and, thereby distributing the load over the subgrade, rather than by the bending action of the slab. The thickness design of the pavement is influenced by the strength of the subgrade.

2.0 BASE COURSE

The function of the base course varies according to the type of pavement. In general they provide additional structural support, drainage and protection against frost action, (when necessary).

2.1 Rigid Pavements

2.1.1 Control of Pumping. To prevent pumping, a base course must either be free draining or it must be highly resistant to erosion action of water.

2.1.2 Protection Against Frost Action. The base course needs to be designed for free drainage and be non-frost susceptable.

2.1.3 Drainage. The base may or may not be a well graded material, but it should contain little or no fines.

2.1.4 Protection of Volume Change of The Subgrade. This may require stabilization with cement or asphalt.

2.1.5 Increased Structural Capacity.

2.2 Flexible Pavements

Primarily used to increase the load-supporting capacity of the pavement by preventing added stiffness and resistance to fatigue as well as building up relatively thick layers to distribute the load through a finite thickness of pavement.

2.3 Base Construction (Visual Aid 6.1)

Base courses are constructed some distance beyond the edge of the wearing surface. This is done to make certain that loads applied at the edge of the pavement will be supported by the underlying layers.

2.3.1 Little or No Fines*. This aggregate gains its stability by grain-to-grain contact. Usually exhibits:

(a) low density,

(b) pervious,

(c) non-frost susceptible, and

(d) difficult to handle during construction.

* The term "fines" for this discussion indicates the portion of the mix which will pass a No. 20° mesh sieve.

2.3.2 Sufficient Fines. This aggregate gains its strength from grain contact but with increased resistance.

2.3.3 A Great Amount of Shear Fines. The aggregate "floats" in the soil due to the loss of grain-to-grain contact.

(a) low density,

(b) impervious,

(c) frost susceptible,

(d) stability affected by moisture, and

(e) compacts readily.

3.0 SUB-BASE

Sub-base may consist of select materials, such as natural grouts, that are stable but have characteristics that make them not completely suitable as base courses. They may also be of stabilized soil or select borrow.

3.1 Purpose

The purpose of the sub-base is to permit the building of relatively thick pavements at low cost. Thus, the quality of subbases can vary within wide limits, as long as the thickness design criteria are fulfilled.

3.2 Lab Tests

Density and moisture requirements are determined from the results of laboratory or field design tests.

4.0 LOAD CARRYING CONCEPT

4.1 General Response of a Pavement

4.1.1 Traffic Load in a Single Position. A pavement that carries a traffic load will be stressed in the general manner shown in Visual Aid 6.2. Maximum stresses occur under the center of the load shown in Visual Aid 6.2a. Visual aid 6.2b and Visual Aid 6.2c show these stresses 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 (i.e., below the neutral axis, Visual Aid 6.2c). The distribution of temperature, as illustrated in Visual Aid 6.2d, will also affect the magnitude of the stresses.

4.1.2 The Moving Load. In reality, the load is moving. The stresses shown in Visual Aid 6.2b and 6.2c can be considered as peak values, which occur when the load is directly over the vertical dotted line shown in Visual Aid 6.2a. When the load is approaching, or leaving, smaller vertical and horizontal stresses will occur along that line. This situation can be represented by Visual Aid 6.3 for an approaching load.

Consider an element in the pavement, as shown in Visual Aid 6.3. It is simultaneously subjected to a buildup in both major principal stress, σ_1, and a minor principal stress, σ_3, as the load approaches. In addition, as the stress build up (i.e., when the load approaches position B from Position A), a rotation of the axis for these principal stresses occurs.

5.0 EFFECT OF TIRE PRESSURE AND TOTAL LOAD

5.1 Variation of Vertical Stress With Depth (Visual Aid 6.4)

The magnitude of vertical stress at a point due to a load at the surface on a pavement will depend on the applied pressure as well as the magnitude of the total load. Visual Aid 6.4 and 6.5 represent Boussinesq vertical pressures in an ideal soil mass due to various combinations of tire pressure and total load. In Visual Aid 6.4 one curve is for a tire pressure of 100 psi and single load of 80,000 pounds. Also presented is that for an identical gross wheel load but for the tire pressure of 200 psi. As seen on th- curves the effect of the high tire pressure is pronounced in the upper layers of the pavement, whereas at a depth of about 36 inches the stresses are about equal for both cases.

High tire pressures, thus, necessitate high-quality materials in the upper layers of the pavement, but the required total depth of pavement is not affected appreciably by tire pressures. On the other hand, for a constant tire pressure an increase in total load increases the vertical stress for all depths.

5.2 Effect of Number of Wheels (Viusal Aid 6.5)

Visual Aid 6.5 shows the effect of dual wheels on stresses for constant tire pressure. Calculated stresses at the surface are not affected by the wheel configuration and are equal to the applied tire pressure. Dual wheels, however, result in increased stresses at greater depths as do tandem axles when the pressure bulbs of the tires overlap. Notes for Visual Aid 6.5 are as follows:

(a) All tires have 100 psi inflation.

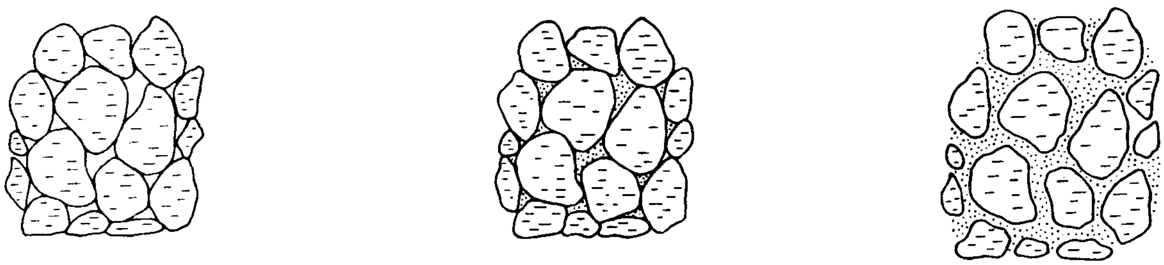
(b) Depth at which interaction of dual wheels is significant is about equal to one-half the clear distance between tires.

(c) Depth at which dual tires will act as a single tire is about two times the c-c spacing of the tires.

LESSON OUTLINE
PAVEMENT LOAD CARRYING CONCEPTS

VISUAL AID	TITLE
Visual Aid 6.1.	Physical states of soil-aggregate mixtures.
Visual Aid 6.2.	Typical stress and temperature distributions under a wheel load.
Visual Aid 6.3.	Rotation of principal stress axis of an element as a vehicle moves over the surface.
Visual Aid 6.4.	Variation of vertical stress with depth.
Visual Aid 6.5.	Effect of number of wheels on vertical stress.

Visual Aid 6.1. Physical states of soil-aggregate mixtures.

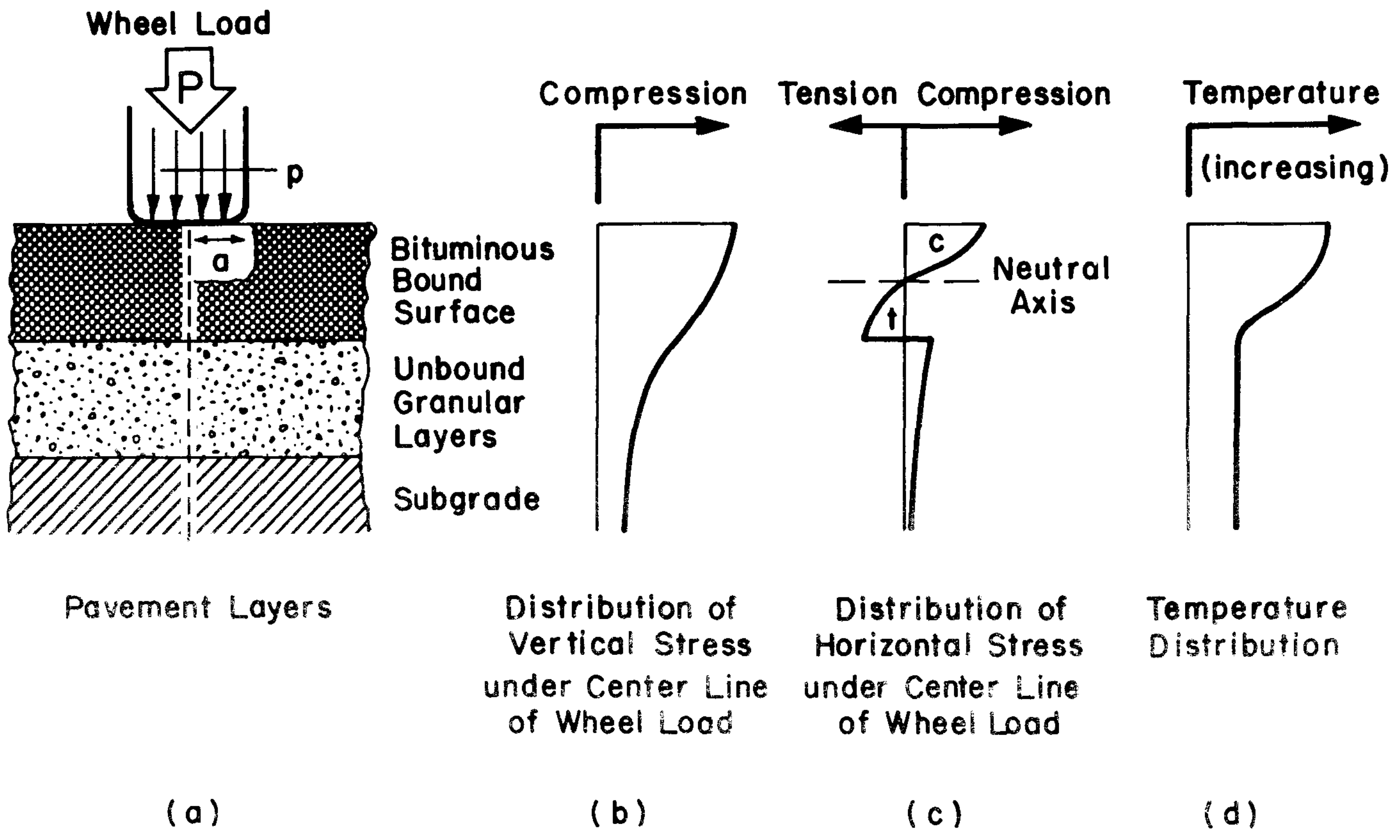


(a) Little or No Fines

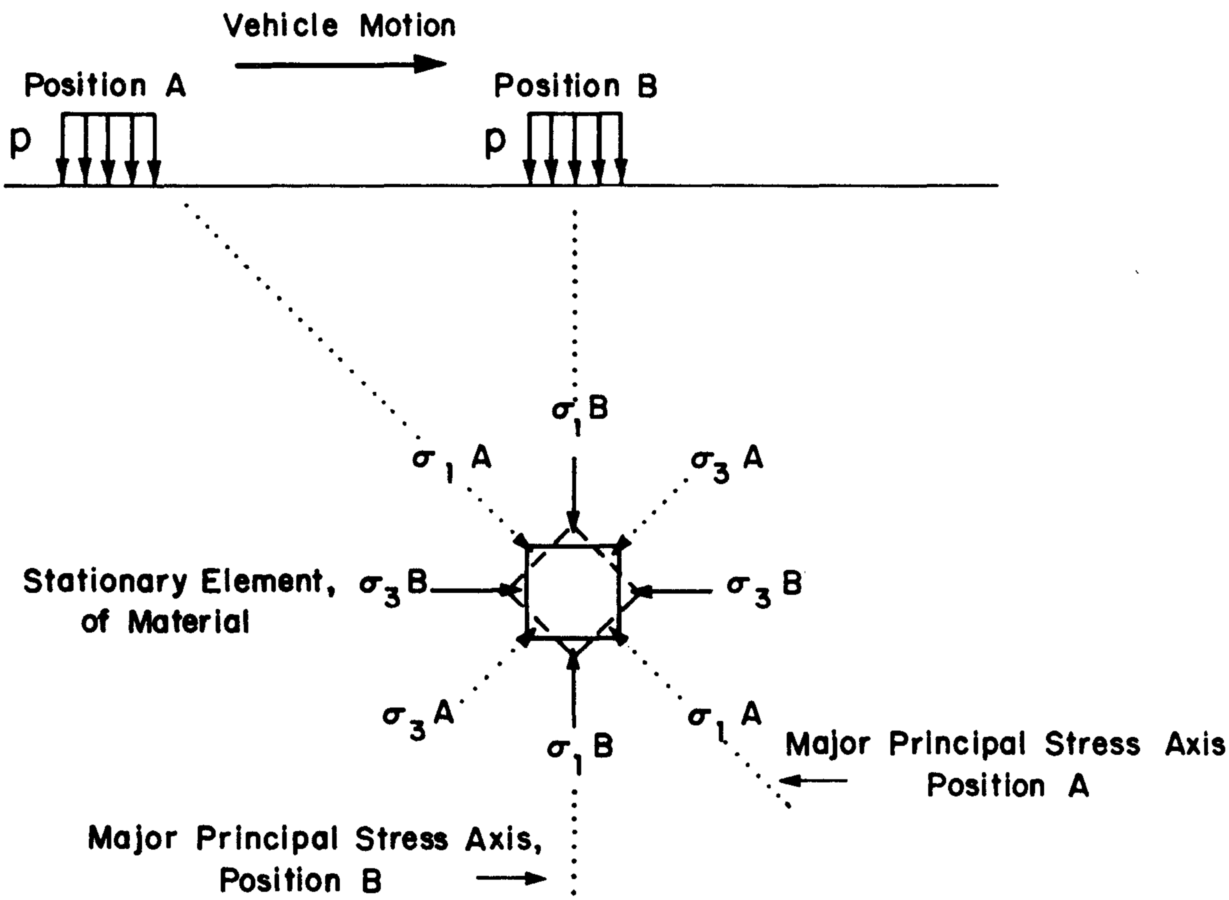
(b) Sufficient Fines

(c) A Great Amount of Fines

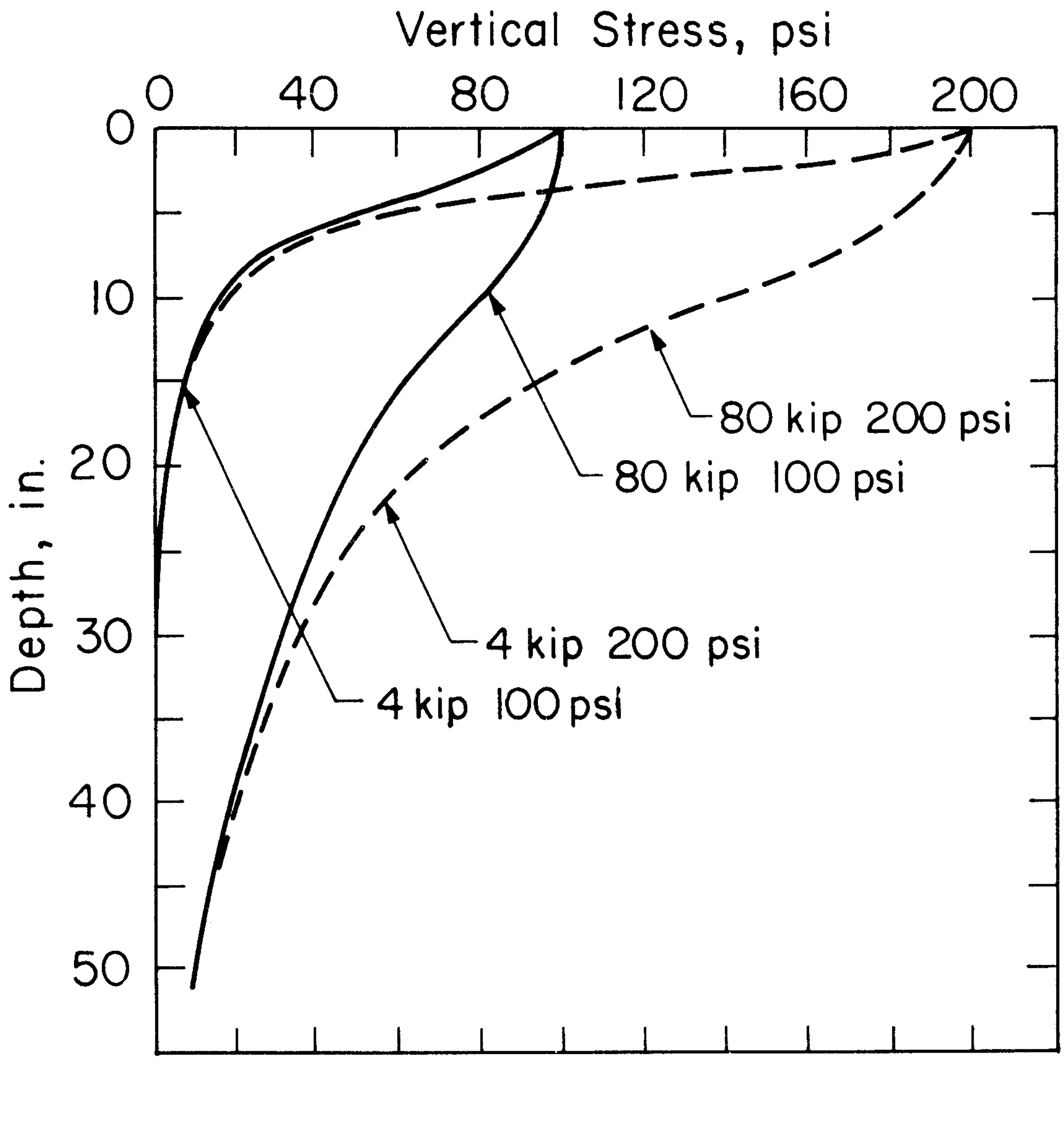
Visual Aid 6.2. Typical stress and temperature distributions under a wheel load.



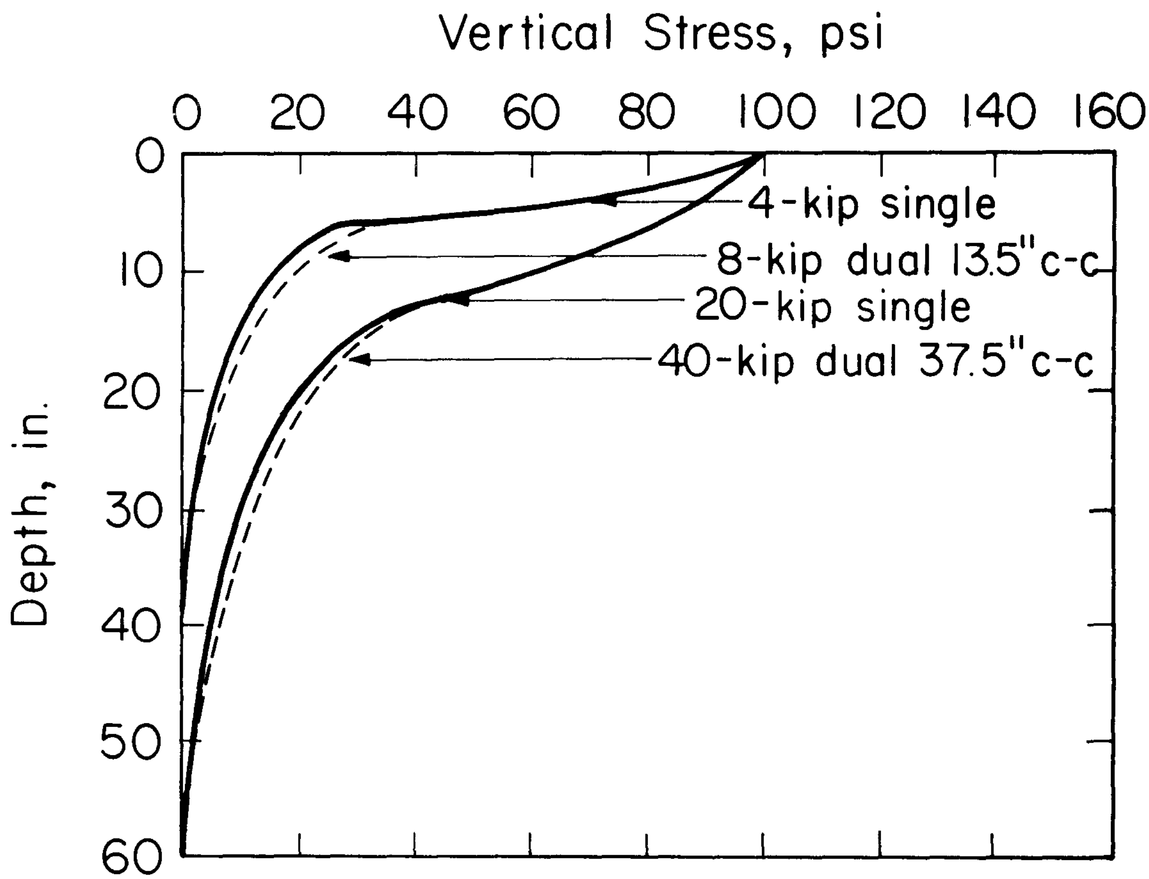
Visual Aid 6.3. Rotation of principal stress axis of an element as a vehicle moves over the surface.



Visual Aid 6.4. Variation of vertical stress with depth.



Visual Aid 6.5. Effect of number of wheels on vertical stress.



LESSON OUTLINE
PAVEMENT DESIGN VARIABLES

Instructional Objectives

1. To demonstrate that the most difficult aspect of solving complex problems (i.e., engineering problems) is often properly defining the problem itself. The number of variables in a seemingly simple problem can often be large once the problem is properly defined.

2. The instructor should emphasize in his summary that engineering judgment is the key ingredient in determining which variables should be included in the analysis or ignored.

Performance Objectives

1. The student should understand the complexity of pavement design problems and the simplifications and assumptions that are in applying the various design theories.

Abbreviated Summary	Time Allocations min.
1. Both Pavement Types	20
2. Flexible Pavement	15
3. Rigid Pavement	15
	50 minutes

Reading Assignment

1. AASHTO Interim Guide - Introduction, Chapter 1

2. Haas & Hudson - Chapter 12

3. Yoder & Witczak - Chapter 1

LESSON OUTLINE
PAVEMENT DESIGN VARIABLES

1.0 PAVEMENT OBJECTIVES

Before we can design pavements we need to clearly see the objectives of a pavement (Visual Aid 7.1).

(a) Maximum or reasonable economy (in terms of agency costs and user costs).
(b) Maximum or adequate safety.
(c) Maximum or reasonable pavement serviceability over the design period.
(d) Maximum or adequate load-carrying capacity (magnitude and repetitions).
(e) Minimum or limited physical deterioration due to environmental and traffic influences.
(f) Minimum or limited noise and air pollution during construction.
(g) Minimum or limited disruption of adjoining land use.
(h) Maximum or good aesthetics.

2.0 DESIGN OBJECTIVES

It is also necessary to fully understand what the objectives of "design" are (Visual Aid 7.2).

(a) Development of a design strategy of maximum (or "reasonable") economy, safety, and serviceability.
(b) Consideration of all possible design alternatives.
(c) Recognition of the variational nature of the design factors.
(d) Maximization of the accuracy of prediction of serviceability, safety, and physical deterioration for the alternatives considered.
(e) Maximization of the accuracy of estimating costs and benefits.
(f) Minimization of the costs of design (materials, testing, computer time, personnel time, etc.).
(g) Maximization of information transfer and exchange between construction and maintenance people.
(h) Maximum use of local materials and labor materials and labor in the design strategies considered.

3.0 DESIGN CONSTRAINTS

Design constraints must also be recognized within any good design approach (Visual Aid 7.3).

(a) Availability of time and funds (for construction, and for conducting the design itself).
(b) Minimum level of serviceability allowed for the pavement before rehabilitation of materials.

(c) Availability of materials.
(d) Minimum or maximum layer thickness.
(e) Minimum time between overlays or seal coats.
(f) Capabilities of construction and maintenance processes.
(g) Testing capabilities.
(h) Capabilities of the structural and economic models available.
(i) "Quality" and extent of the design information available.

4.0 TRAFFIC AND LOAD VARIABLES

4.1 Load Factors

(a) magnitude,
(b) repetitions, and
(c) sequence.

4.2 Placement

(a) distribution and
(b) coverage.

4.3 Representative Contact Area

(a) Configuration of area

(1) shape and
(2) proximity of loaded area.

(b) Contact pressure-tires to pavement.

4.4 Type of Load Application with Respect to Rate and Duration

(a) static and
(b) dynamic (repeated, impact, vibratory).

4.5 Tangential Forces

(a) accelerating,
(b) braking, and
(c) cornering.

5.0 SUBGRADE EVALUATION

5.1 Strength - Stress - Deformation Characteristics

(a) with respect to loading
(b) considering properties influencing

(1) density with respect to time,
(2) moisture content with respect to time,

(3) texture of soil,
(4) structure,
(5) gradation,
(6) porosity, and
(7) permeability.

5.2 Volume Change

Volume change is dependent on soil classification and degree of confinement of particular interest to the pavement engineer are:

(a) swelling characteristics,
(b) shrinkage characteristics, and
(c) consolidation.

6.0 CLIMATE - WEATHERING EFFECTS

6.1 Rainfall

(a) frequency,
(b) duration, and
(c) intensity.

6.2 Temperature and Humidity

(a) extremes,
(b) frequency and duration of cycle, and
(c) rate of change.

7.0 LOCATION

The same soil will behave differently depending on other factors often encountered or created by the pavement engineer such as:

(a) cut and fill,
(b) proximity of sea water or chemical action,
(c) water table,
(d) deep, soft deposits (organic),
(e) earth movements

(1) landslides and
(2) mudflows.

8.0 THICKNESS AND QUALITY OF PAVEMENT STRUCTURE (Visual Aid 7.5)

Determination of thickness and quality of the pavement materials with respect to vertical position is dependent upon the soil strength and expected volumetric changes , degree of confinement, soil classification, and the stress-strain characteristics.

8.1 Evaluation of Wearing Surface

(a) stability,
(b) durability, and
(c) deformation characteristics compatible with the underlying layers.

8.2 Strength of Wearing Surface

(a) flexural,
(b) compressive, and
(c) tensile.

LESSON OUTLINE
PAVEMENT DESIGN VARIABLES

VISUAL AID	TITLE
Visual Aid 7.1.	Pavement objectives.
Visual Aid 7.2.	Design objectives.
Visual Aid 7.3.	Design constraints.
Visual Aid 7.4.	Traffic and load variables.
Visual Aid 7.5.	Major pavement dsign components.

VISUAL AID 7.1. PAVEMENT OBJECTIVES

1. MAXIMUM OR REASONABLE ECONOMY
   (IN TERMS OF AGENCY COSTS AND USER COSTS)
2. MAXIMUM OR ADEQUATE SAFETY
3. MAXIMUM OR REASONABLE PAVEMENT
   SERVICEABILITY OVER THE DESIGN PERIOD
4. MAXIMUM OR ADEQUATE LOAD-CARRYING CAPACITY
   (MAGNITUDE AND REPETITIONS)
5. MINIMUM OR LIMITED PHYSICAL DETERIORATION
   DUE TO ENVIRONMENTAL AND TRAFFIC INFLUENCES
6. MINIMUM OR LIMITED NOISE AND AIR POLLUTION
   DURING CONSTRUCTION
7. MINIMUM OR LIMITED DISRUPTION OF ADJOINING
   LAND USE
8. MAXIMUM OR GOOD AESTHETICS

VISUAL AID 7.2. DESIGN OBJECTIVES

1. DEVELOPMENT OF A DESIGN STRATEGY OF MAXIMUM (OR "REASONABLE") ECONOMY, SAFETY, AND SERVICEABILITY

2. CONSIDERATION OF ALL POSSIBLE DESIGN ALTERNATIVES.

3. RECOGNITION OF THE VARIATIONAL NATURE OF THE DESIGN FACTORS

4. MAXIMIZATION OF THE ACCURACY OF PREDICTION OF SERVICEABILITY, SAFETY, AND PHYSICAL DETERIORATION FOR THE ALTERNATIVES CONSIDERED

5. MAXIMIZATION OF THE ACCURACY OF ESTIMATING COSTS AND BENEFITS

6. MINIMIZATION OF THE COSTS OF DESIGN (MATERIALS, TESTING, COMPUTER TIME, PERSONNEL TIME, ETC.)

7. MAXIMIZATION OF INFORMATION TRANSFER AND EXCHANGE BETWEEN CONSTRUCTION AND MAINTENANCE PEOPLE

8. MAXIMUM USE OF LOCAL MATERIALS AND LABOR IN THE DESIGN STRATEGIES CONSIDERED

VISUAL AID 7.3. DESIGN CONSTRAINTS

1. AVAILABILITY OF TIME AND FUNDS
   (FOR CONSTRUCTION, AND FOR CONDUCTING THE DESIGN ITSELF)

2. MINIMUM LEVEL OF SERVICEABILITY ALLOWED FOR THE PAVEMENT BEFORE REHABILITATION

3. AVAILABILITY OF MATERIALS

4. MINIMUM OR MAXIMUM LAYER THICKNESSES

5. MINIMUM TIME BETWEEN OVERLAYS OR SEAL COATS

6. CAPABILITIES OF CONSTRUCTION AND MAINTENANCE PROCESSES

7. TESTING CAPABILITIES

8. CAPABILITIES OF THE STRUCTURAL AND ECONOMIC MODELS AVAILABLE

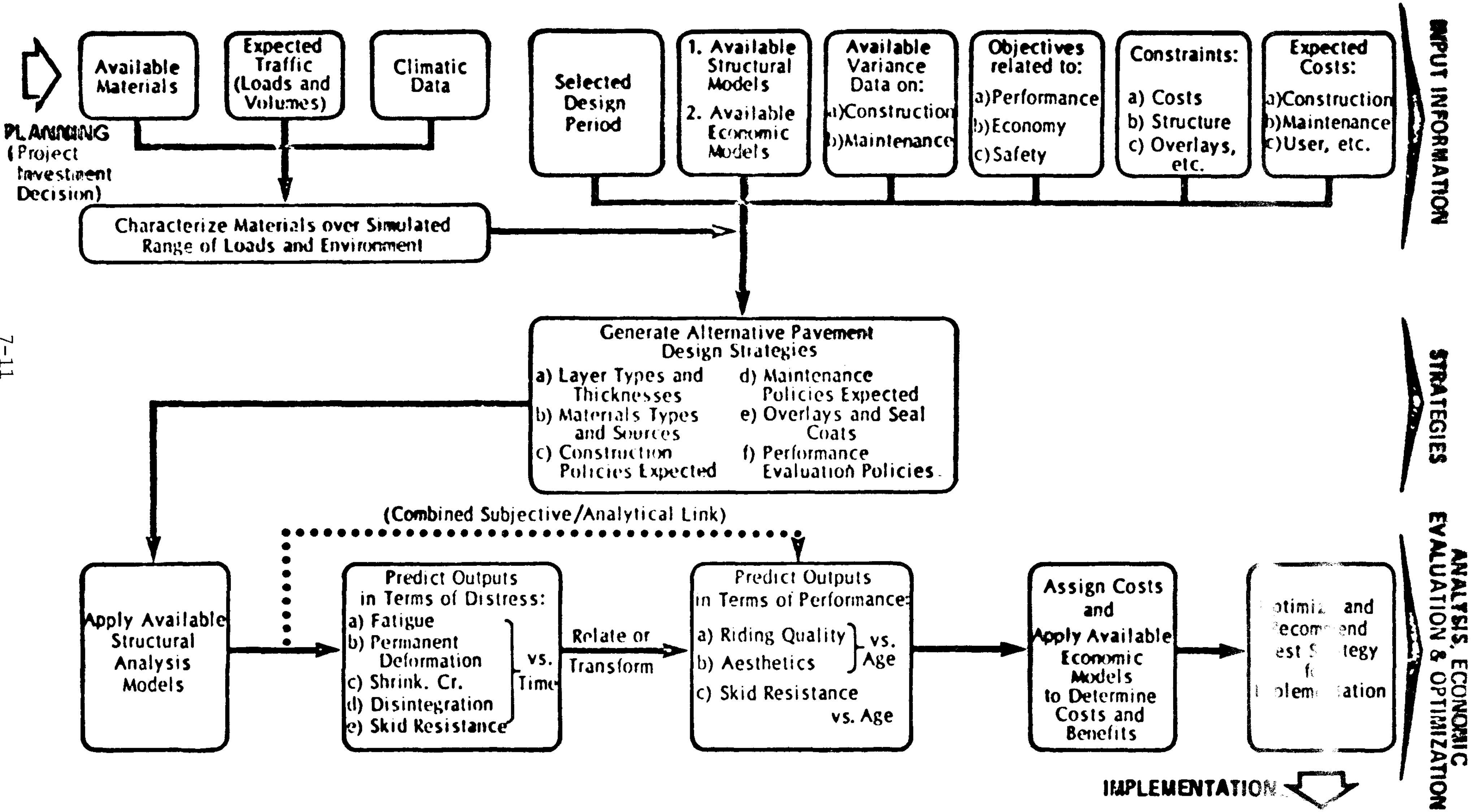
9. "QUALITY" AND EXTENT OF THE DESIGN INFORMATION AVAILABLE

10. ADMINISTRATIVE AND POLITICAL CONTROL

VISUAL AID 7.4. TRAFFIC AND LOAD VARIABLES

1. WHEEL LOAD, AXLE LOAD, AND TOTAL VEHICLE LOAD
2. NUMBER OF LOAD APPLICATIONS, AND THEIR SEQUENCE
3. VEHICLE SPEED
4. LATERAL AND LANE DISTRIBUTION OF LOADS
5. TIRE PRESSURES
6. WHEEL OR GEAR CONFIGURATIONS

VISUAL 7.5. MAJOR PAVEMENT DESIGN COMPONENTS



LESSON OUTLINE
INTRODUCTION TO THE AASHO ROAD TEST
A MOVIE

Instructional Objectives

1. To illustrate the complexities involved in pavement testing and to introduce the Road Test as the foundation for the Serviceability Performance concept and the AASHTO Interim Design Guides.

Performance Objectives

1. The student will obtain background information pertaining to the history of the AASHO Road Test as well as an understanding of the magnitude of the problems associated with the scale of experimental testing.

Abbreviated Summary	Time Allocation, min.
1. Introduction	15
2. AASHO Movie	35
	50

Reading Assignment

1. Instructional Text

LESSON OUTLINE
INTRODUCTION TO THE AASHO ROAD TEST
A MOVIE

1.0 INTRODUCTION

1.1. History of Road Tests

The AASHO Road Test was the third full-scale test of pavement behavior under controlled truck traffic to be administered by the Highway Research Board of the National Academy of Sciences--National Research Council. The first such project, Road Test One-MD, was conducted on an existing portland cement concrete pavement in Maryland. A complete report on this project, Special Report 4, was published by the Board in 1952. The second such project, the WASHO Road Test, was conducted on two specially-built test loops of asphaltic concrete pavement in Idaho. Two reports on this project were published by the Board as Special Report 18 (1954) and Special Report 22 (1955).

1.2. AASHO Road Test

The AASHO Road Test was conceived and sponsored by the American Association of State Highway Officials as a study of the performance of highway pavement structures of known thickness under moving loads of known magnitude and frequency.

2.0 TEST DESCRIPTION

The project was considerably larger and more comprehensive than the previous studies, and the design of the experiment contained features not incorporated in the other two tests. Both portland cement and asphaltic concrete pavements, as well as certain types of bridges, were included in the test facility.

3.0 AASHO REPORTS

The AASHO Road Test was completed in 1960 and the results comprise five major reports. A subsequent large volume of special papers was presented at a conference in St. Louis, Missouri and published as TRB Special Report 73. Since that time, the AASHO Road Test data has been used hundreds of times to test theories, to develop pavement performance methodology. As covered elsewhere in this course, the AASHO Road Test forms the basis for the AASHTO Interim Design Guides, the most widely used pavement design manual in the world.

4.0 AASHO ROAD TEST MOVIE

The AASHO Road Test Movie was produced by the then Bureau of Public Roads and released widely in the division offices and regional office of the BPR throughout the United States. Approximately 20 or 30 copies were shown hundreds of times. However, since 1970, there has been little use of the movie. The movie demonstrates the massive nature and complicated aspects of the road test better than any report alone can do.

INSTRUCTIONAL TEXT
INTRODUCTION TO THE AASHO ROAD TEST
A MOVIE

THE AASHO ROAD TEST
Report 5
Pavement Research

Highway Research Board
Special Report 61E

National Academy of Sciences
National Research Council
Publication No. 954
Washington, D.C.

1962

THE AASHO ROAD TEST
Report 5
Pavement Research

Chapter 1

General Information

1.1 BACKGROUND AND OBJECTIVES

1.1.1 History

The events leading to the three most recent large-scale highway research projects, Road Test 1–MD, the WASHO Road Test and the AASHO Road Test, are described in detail in AASHO Road Test Report 1, "History and Description of the Project" (HRB Special Report 61A). The following is a summary of these events and the activities of the AASHO Road Test.

For many years the member states of the American Association of State Highway Officials had been confronted with the dual problem of constructing pavements to carry a growing traffic load and establishing an equitable policy for vehicle sizes and weights. The Association recognized the common need for factual data for use in resolving the problem. Therefore, in September 1948, it set up a procedure for initiating and administering research projects to be jointly financed by two or more states.

In December of the following year a meeting was held at Columbus, at the request of the Governor of Ohio, to consider the problem of vehicle weight and its effect upon existing and future pavements. The conference was attended by representatives of the Council of State Governments and highway officials of 14 eastern and midwestern states. The need for more factual data concerning the effect of axle loads of various magnitudes on pavements was confirmed.

As a result, Road Test 1–MD was conducted in 1950. An existing concrete pavement in Maryland was tested under repeated application of two single- and two tandem-axle loads. The Highway Research Board administered the test and published the results as HRB Special Report 4.

Concurrently, the Committee on Highway Transport of the American Association of State Highway Officials recommended that additional road tests be initiated by the regional members of the Association. As a result, the Western Association of State Highway Officials sponsored the WASHO Road Test, consisting of a number of specially-built flexible pavements in Idaho tested in 1953-54 under the same loads used in the Maryland test. The results of this test, also conducted by the Highway Research Board, were published as Special Reports 18 and 22.

In March 1951, the Mississippi Valley Conference of State Highway Engineers had started planning a third regional project. However, the idea of another regional project of limited extent was abandoned in favor of a more comprehensive road test to be sponsored by the entire Association. In October, complying with a request by the Association, a Highway Research Board task committee submitted a report, "Proposal for Road Tests," after which the Association appointed a working committee to prepare a prospectus on the project. By December it had been decided to include bridges in the research.

In June 1952, the Working Committee produced a report, "AASHO Road Test Project Statement." In July it selected a site for the project near Ottawa, Ill. In January 1953, it submitted a second report, "AASHO Road Test Project Program," and in August 1954, a third entitled "Project Program Supplement." In May 1955, this committee produced its fourth and final report "Statement of Fundamental Principles, Project Elements and Specific Directions."

Meanwhile, in March 1953, AASHO had formulated a plan for prorating the cost of the project among its member departments and, later, had received assurances of participation from the States, the Automobile Manufacturers Association, the Bureau of Public Roads and the American Petroleum Institute, while the Department of Defense had agreed to furnish military personnel for driving the vehicles.

On February 22, 1955, the Highway Research Board with the approval of its parent organization, the National Academy of Sciences—National Research Council, accepted from the Association the responsibility to administer and direct the new project. The Board opened a field office at Ottawa, Ill., in July 1955; and in August a task force of the Illinois Division of Highways moved to the site to undertake the preparation of plans and to prepare for the construction of the test facilities.

In March 1956, the Board appointed the National Advisory Committee as its senior advisory group and in April selected a project director.

In June 1956, the National Advisory Committee passed a resolution recommending that the Executive Committee of the Highway Research Board consider the inclusion in the facility of a fifth test loop to be subjected to light axle loads. This resolution, recommended by the Bureau of Public Roads, was based on the pending enactment of the Federal Aid Highway Act of 1956. In July, the Executive Committee of the Board approved this change and made additional changes involving special studies areas. The final layout of the test facilities is described in Section 1.2.2.

Construction of the test facilities began in August 1956, and test traffic was inaugurated on October 15, 1958. Test traffic was operated until November 30, 1960, at which time 1,114,000 axle loads had been applied to the pavement and the bridges.

A special studies program was conducted in the spring and early summer of 1961 over some of the remaining test sections. Strains, deflections and pressures were measured in these studies under a wide variety of vehicle types, load suspensions, tires and tire pressures. Special military vehicles, included at the request of the Army, as well as highway construction equipment, were included in these tests. The results of the studies are presented in Road Test Report 6.

During 1961, the research staff concentrated on analysis of the test data and the preparation of reports. Each of the major reports was approved by a review subcommittee of the National Advisory Committee and later submitted to the entire National Advisory Committee and the Regional Advisory Committees prior to its publication by the Highway Research Board. All reports were completed by the project staff, reviewed by the various committees, and submitted to the Board.

The field office for the project was closed in January 1962. However, the Highway Research Board agreed to continue certain studies associated with the Road Test pavement performance analyses in its Washington office. The results of these studies will be reported by the Highway Research Board.

1.1.2 Intent of the AASHO Road Test

The following formal statement of the intent of the Road Test was approved by the Executive Committee of the Highway Research Board January 13, 1961:

> The AASHO Road Test plays a role in the total engineering and economic process of providing highways for the nation. It is important that this role be understood.
>
> The Road Test is composed of separate major experiments, one relating to asphalt concrete pavement, one relating to portland cement concrete pavement, and one to short span bridges. There are numerous secondary experiments. In each of the major experiments, the objective is to relate design to performance under controlled loading conditions.
>
> In the asphalt concrete and portland cement concrete experiments some of the pavement test sections are underdesigned and others overdesigned. Each experiment requires separate analysis. Eventually the collection and analysis of additional engineering and economic data for a local environment are necessary in order to develop final and meaningful relations between pavement types.
>
> All of the short span bridges are underdesigned. Each is a separate case study.
>
> Failures and distress of the pavement test sections and the beams of the short span bridges are important to the success of each of the experiments.
>
> The Highway Research Board of the National Academy of Sciences—National Research Council has the responsibility of administering the project for the sponsor, the American Association of State Highway Officials, within the bounds of the objectives of the test. The Board is also responsible for collecting engineering data, developing methods of analysis and presentation of data, preparing comprehensive reports describing the tests, and drawing valid findings and conclusions. It is here that the role of the Highway Research Board ends.
>
> As the total engineering and economic process of providing highways for the nation is developed, engineering data from the AASHO Road Test and engineering and economic data from many other sources will flow to the sponsor and its member departments. It is here that studies will be made and final conclusions drawn that will be helpful to the executive and legislative branches of our several levels of government and to the highway administrator and engineer.

1.1.3 Objectives

The objectives of the AASHO Road Test as stated by the National Advisory Committee were as follows:

> 1. To determine the significant relationships between the number of repetitions of specified axle loads of different magnitude and arrangement and the performance of different thick-

nesses of uniformly designed and constructed asphaltic concrete, plain portland cement concrete, and reinforced portland cement concrete surfaces on different thicknesses of bases and subbases when on a basement soil of known characteristics.

2. To determine the significant effects of specified vehicle axle loads and gross vehicle loads when applied at known frequency on bridges of known design and characteristics.

3. To make special studies dealing with such subjects as paved shoulders, base types, pavement fatigue, tire size and pressures, and heavy military vehicles, and to correlate the findings of these special studies with the results of the basic research.

4. To provide a record of the type and extent of effort and materials required to keep each of the test sections or portions thereof in a satisfactory condition until discontinued for test purposes.

5. To develop instrumentation, test procedures, data, charts, graphs, and formulas, which will reflect the capabilities of the various test sections; and which will be helpful in future highway design, in the evaluation of the load-carrying capabilities of existing highways and in determining the most promising areas for further highway research.

This report deals primarily with work done in connection with Objectives 1 and 5 and with some of the special studies mentioned in Objective 3. Material relating to Objective 2 will be found in Road Test Report 4 and Objective 4 is discussed in Report 3. Other special studies suggested in Objective 3 are discussed in Report 6.

1.1.4 *Objectivity of Findings*

Discussion of the results given in this report has generally been limited to specific relationships derived from the data. Restraint has been exercised in expressing opinions, conjectures, and speculations. Conclusions have been drawn only when supported by data acquired during the tests.

At the request of the National Academy of Sciences a panel of statisticians was appointed in 1955 so that professional advice was available for both the designs of the Road Test experiments and for the procedures by which the experimental data would be analyzed. It was not the function of this group to select variables nor levels for variables to be included in the Road Test. This was the responsibility of the National Advisory Committee, acting upon the recommendations of the original AASHO Transport Committee's Working Committee. The Statistical Panel played an important role in influencing the experimental layout through its recommendations for complete factorial designs, randomization, and replication. Its recommendations, accepted by the Advisory Committee, made possible effective studies of the relationships sought by the objectives.

Within the space, time and funds available, only a few variables could be studied thoroughly. The experiment was designed and the test facilities built specifically for the study of these variables. In general, mathematical models were used to represent associations among experimental variables, then statistical methods were employed to determine constants for the models as well as to describe the reliability of the evaluated models. Thus experimental designs and analytical procedures were developed in order to obtain unbiased estimates of the effects (and the statistical significance of many of the effects) of controlled experimental factors. The designs and procedures did not, however, make it possible to obtain effects for other factors that were either held constant or that varied in an uncontrolled fashion, for example, embankment soil, strength of materials, and environmental conditions. Although estimates were obtained for the effects of axle load and axle configuration, it was not possible to determine the statistical significance of these effects because replication of load or configuration was not provided. Nevertheless, particularily in the cases of load effect on both pavement types and axle configuration effect on rigid pavement the differences observed were so great as to leave practically no doubt that the effects were significantly greater than zero.

Basic data will be made available to other groups equipped to perform independent analyses. Further analyses are to be encouraged by the Highway Research Board in the expectation that the over-all usefulness of the project will be enhanced.

1.1.5 *Applicability of Findings*

The findings of the AASHO Road Test, as stated in the relationships shown by formulas, graphs, and tables throughout this report, relate specifically to the physical environment of the project, to the materials used in the pavements, to the range of thicknesses and loads and number of load applications included in the experiments, to the construction techniques employed, to the specific times and rates of application of test traffic, and to the climatic cycles that occurred during construction and testing of the experimental pavements. More specific limitations on certain of the findings are given in the discussion of results in various sections of this report. *Generalizations and extrapolations of these findings to conditions other than those that existed at the Road Test should be based upon experimental or other evidence of the effects on pavement performance of variations in climate, soil type, materials, construction practices and traffic.*

1.2 FACILITIES AND OPERATIONS

1.2.1 *Site Location*

The location of the AASHO Road Test was near Ottawa, Ill., in LaSalle County, about 80 mi southwest of Chicago (Fig. 1). The test

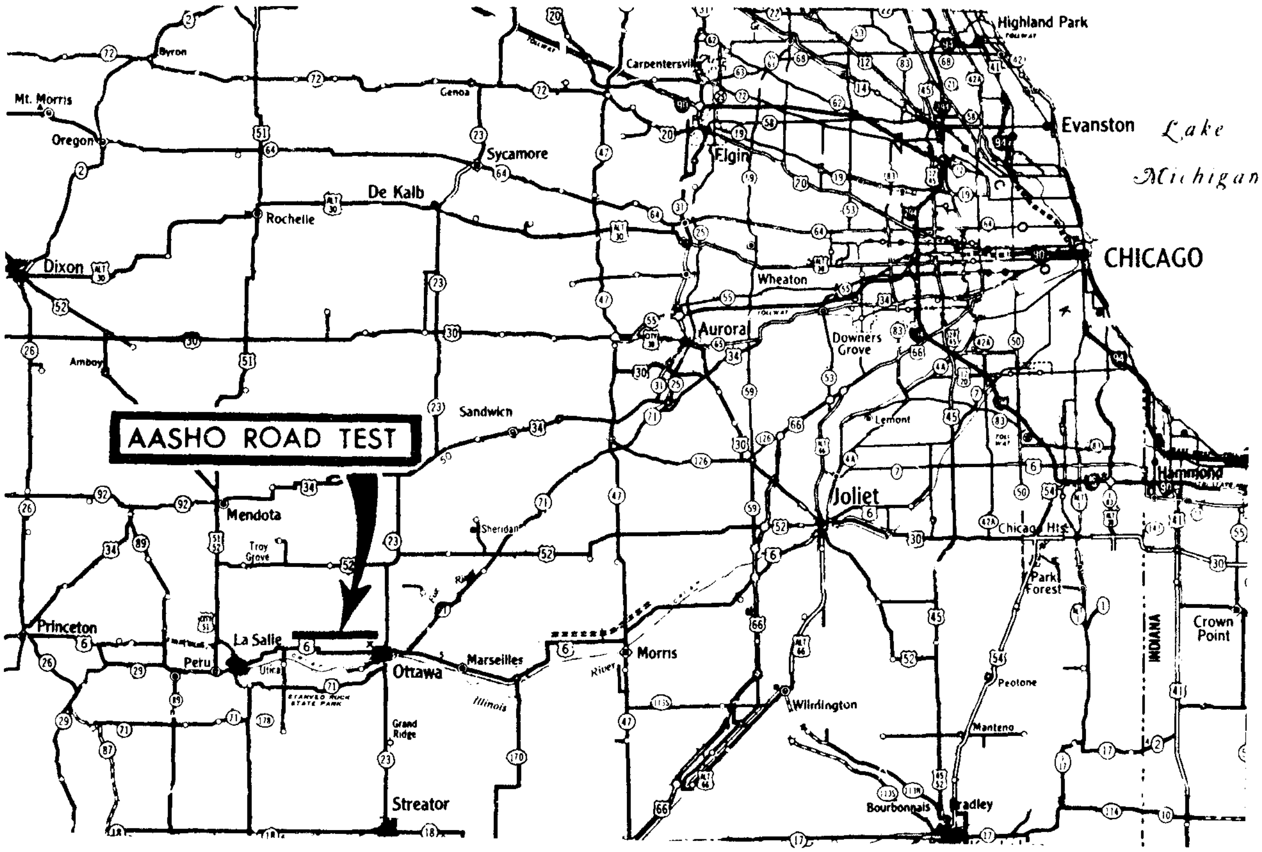


Figure 1. Site location.

facility was constructed along the alignment of Interstate Route 80. The site was chosen because the soil within the area was uniform and of a type representative of that found in large areas of the country, because the climate was typical of that found throughout much of the northern United States, and because much of the earthwork and pavement construction could ultimately be utilized in the construction of a section of the National System of Interstate and Defense Highways.

1.2.2 Test Facilities

The test facilities consisted of four large loops, numbered 3 through 6, and two smaller loops. 1 and 2. Test bridges were at four locations in two of the large loops. The layout of the six test loops, the administration area and the Army barracks is shown in Figure 2.

Each loop was a segment of a four-lane divided highway whose parallel roadways, or tangents, were connected by a turnaround at each end. Tangent lengths were 6,800 ft in Loops 3 through 6, 4,400 ft in Loop 2 and 2,000 ft in Loop 1. Turnarounds in the major loops had 200-ft radii and were superelevated so that the traffic could operate over them at 25 mph with little or no side thrust. Loop 2 had superelevated turnarounds with 42-ft radii. Centerlines divided the pavements into inner and outer lanes, called lane 1 and lane 2 respectively.

All vehicles assigned to any one traffic lane of Loops 2 through 6 had the same axle arrangement-axle load combinations. No traffic operated over Loop 1. In all loops, the north tangents were surfaced with asphaltic concrete and south tangents with portland cement concrete. All variables for pavement studies were concerned with pavement designs and loads within each of the 12 tangents. Each tangent was constructed as a succession of pavement sections called structural sections. Pavement designs, as a rule, varied from section to section. The minimum length of a section was 100 ft in Loops 2 through 6, and 15 ft in Loop 1. Sections were separated by short transition pavements. Each structural section was separated into two pavement test sections by the centerline of the pavement. Figure 3 shows the layout of two typical test loops and locations of the test bridges.

Details of the experiment designs are given in Report 1 and are summarized in Sections 2.1.1 and 3.1.1 of this report. Details concerning all features of bridge research are given in Road Test Report 4.

GENERAL INFORMATION

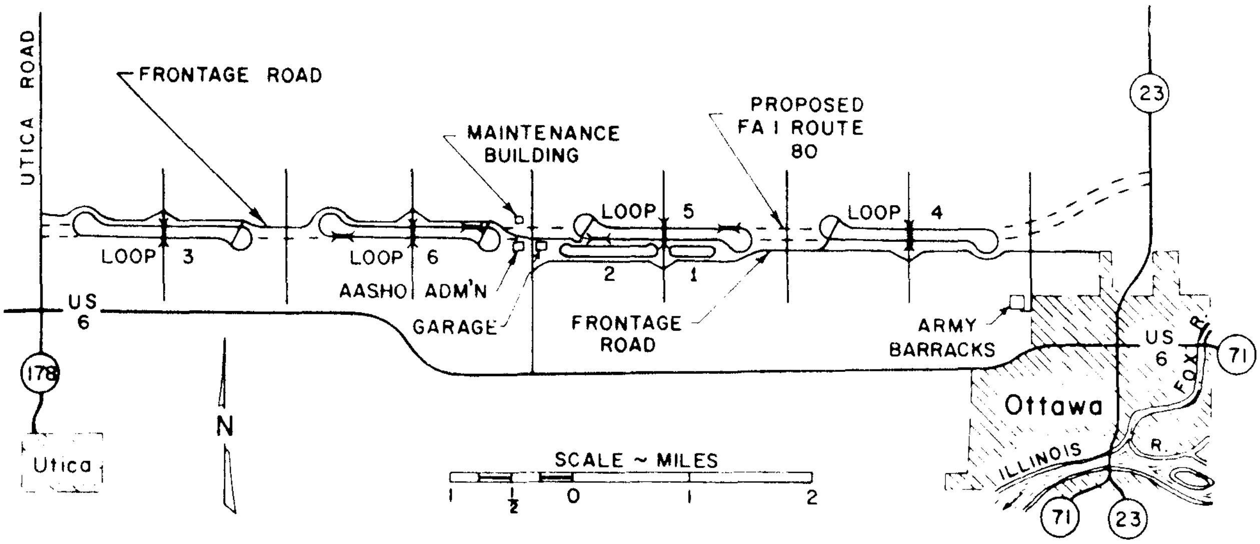


Figure 2. Layout of AASHO Road Test.

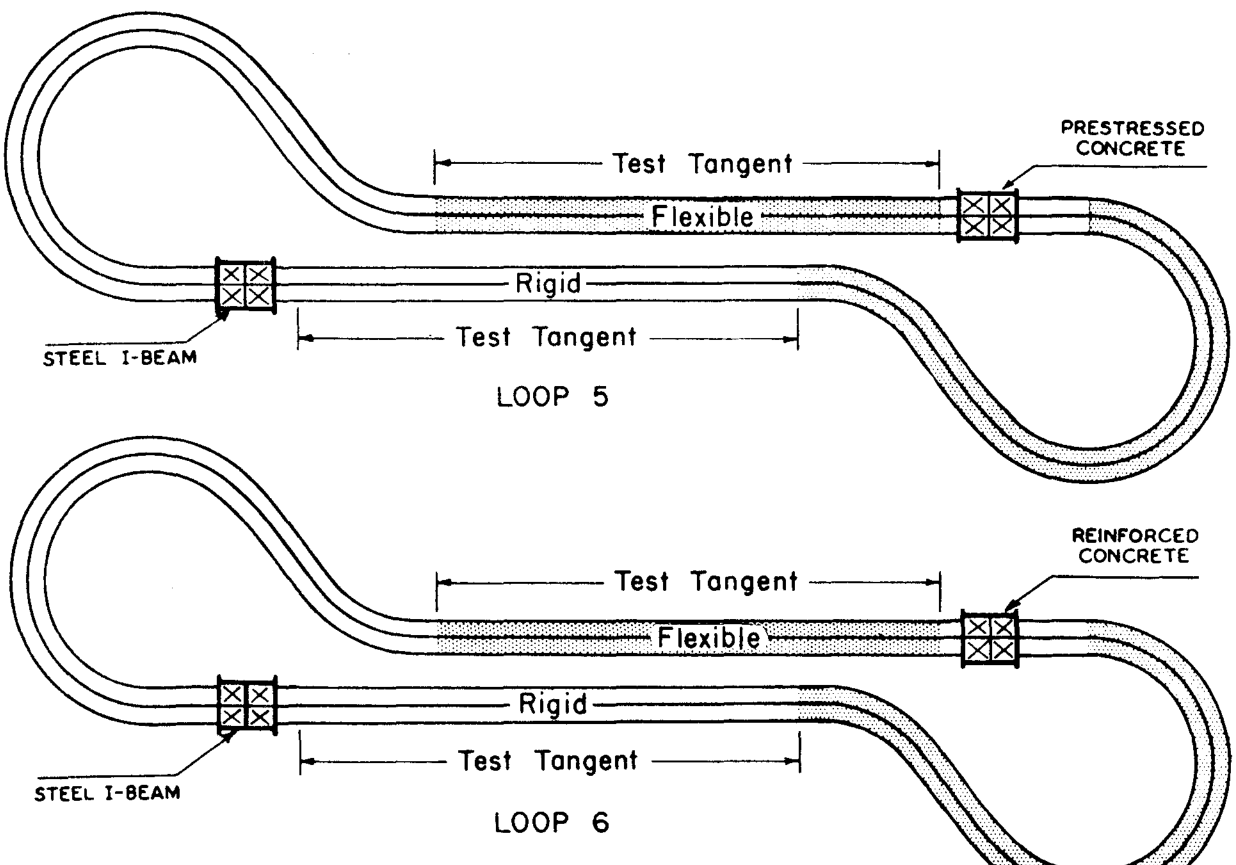


Figure 3. Location of test bridges.

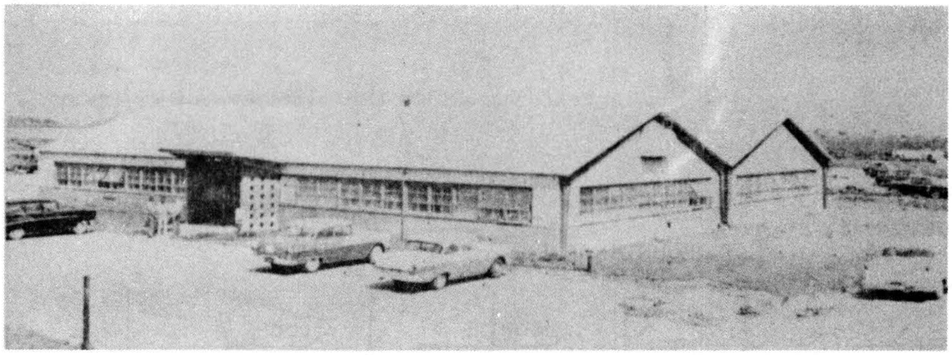


Figure 4. Administration building.

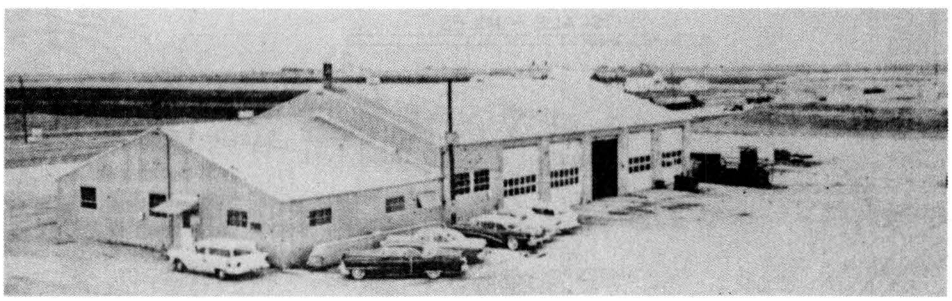


Figure 5. Vehicle maintenance garage.

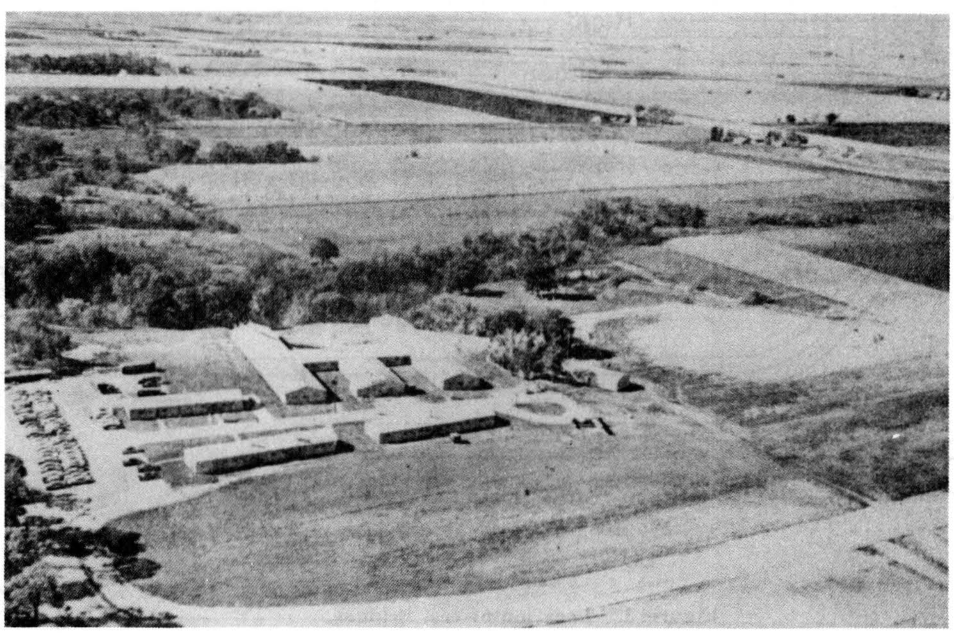


Figure 6. Army driver quarters (Wallace Barracks).

An administrative area was located at the center of the project. Laboratories and offices were located in the building shown in Figure 4. Shop facilities for vehicle maintenance were provided in the building shown in Figure 5. A military installation called Wallace Barracks (Fig. 6) was provided by the National Academy of Sciences to house the Army Transportation Corps Road Test Support Activity.

1.2.3 Construction

A comprehensive description of the construction of the AASHO Road Test facilities is given in Road Test Report 2. Construction was supervised by the task force of the Illinois Division of Highways. On-site materials control and testing were provided by the Highway Research Board Staff on the project. Conventional techniques for construction were used except that extraordinary effort was put forth to insure uniformity of all pavement components. For example, no construction equipment other than that necessary for compaction was permitted to operate in the center 24-ft width of the roadway, and all turning operations on the grade were limited to specially designated transition areas. Specifications for density of compacted embankment soil, subbase and base materials included stipulations of maximum densities as well as the conventional minimums.

Construction was performed under contracts negotiated through normal Illinois contractual channels. It was started in late summer 1956 and completed in time for test traffic to begin in the fall of 1958. S. J. Groves and Sons was the principal contractor in a joint venture with Arcole Midwest, Inc., in the embankment construction and with Rock Roads, Inc., as a subcontractor for asphaltic concrete surfacing. Valley Builders, Inc., built the bridges.

1.2.4 Test Traffic

A detailed description of the operation of the test traffic is presented in Road Test Report 3. As previously stated, Loop 1 was not subjected to test traffic. One lane of this loop was used for subsurface and special load studies, the other for observing the effect of environment on pavements not subjected to traffic. The remaining five loops, 2 through 6, were subjected to traffic for slightly more than two years. Every vehicle in any one of the ten traffic lanes had the same axle load and axle configuration. The assignment of axle loads and vehicle types to the various lanes is shown in Figure 7.

The vehicles were loaded with concrete blocks that were anchored down with steel bands and chains. Although the traffic phase was inaugurated on October 15, 1958, early operation indicated the need to readjust the test loads. This delayed full-scale traffic until November 5, 1958. From November 1958 to January 1960 controlled test traffic consisted of six vehicles in each lane of Loops 3 through 6, four vehicles

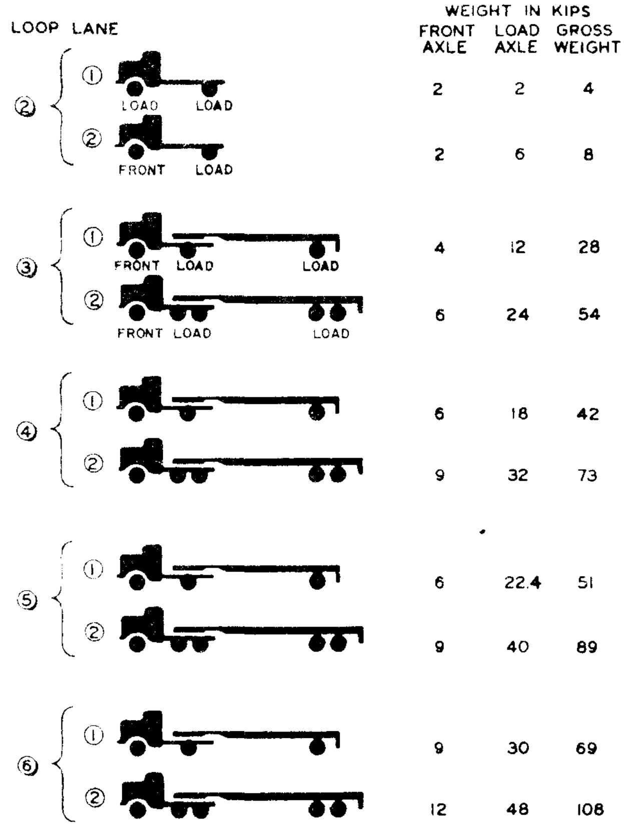


Figure 7. Typical test vehicle axle loadings.

in lane 1 of Loop 2 and eight vehicles in lane 2 of Loop 2. In January 1960, the traffic was increased to ten vehicles in each lane of Loops 3 through 6, six in lane 1 and 12 in lane 2 of Loop 2. These vehicle distributions were selected in order that axle load applications could be accumulated at the same rate in each of the ten traffic lanes.

All lanes had identical specifications for transverse placement, speed, and rate of axle load accumulation. Tire pressure and steering axle loads were representative of normal practice. Some of the vehicles were gasoline and others diesel powered. Further information concerning the vehicles is contained in Road Test Reports 1 and 3.

Whenever possible, traffic was operated at 35 mph on the test tangents. Traffic was scheduled to operate over an 18-hr, 40-min period each day, 6 days a week, except that during the first 6 months of 1960 the schedule was extended to 7 days a week. The schedule was maintained except when pavement distress, truck breakdowns, bad weather and certain other causes made it impossible. A total accumulation of 1,114,000 axle load applications was attained during the 25-month traffic testing period. To accomplish this, soldiers of the U. S. Army Transportation Corps Road Test Support Activity drove more than 17 million miles.

1.2.5 Measurement Programs

Each measurement program was designed to accomplish one or more of the following purposes: (1) to furnish information at regular and frequent intervals concerning the roughness and visible deterioration of the surfacing of each section; (2) to record early in the life of each section transient load effects that might be directly correlated with the ultimate performance of the section; and (3) to furnish a limited amount of additional information which might contribute to a better understanding of pavement mechanics.

Programs falling in the first category were concerned with measurements of permanent changes in the pavement profile along and across the wheelpaths, as well as the extent of cracking and patching of the surfacing. These measurements were given major emphasis since they were used to define the performance of each section as required by the first Road Test objective.

Programs falling in the second category included the measurement of strains and deflections which became the basis for estimating pavement capability, as required by the fifth objective.

Finally, programs of the third category encompassed such measurements as the severity of pumping of rigid pavements, changes in layer thickness in flexible pavements, pavement temperatures, subsurface conditions, and numerous other measurements.

In general, measurements were restricted to those variables that had been demonstrated by previous research to be related significantly to pavement performance. A further restriction, applying especially to subsurface studies, was imposed by the overriding necessity to keep the test traffic moving.

In spite of these restrictions, a formidable amount of data was accumulated, and special electronic systems were evolved to facilitate the storage and initial processing of the data. For example, in the case of some programs, means were provided to record automatically in the field the desired information directly on perforated paper tape, thus eliminating the task of the manual reading of analog records. In another case, an electronic device was used to read field analog records and to punch the information on paper tape for immediate transference to an electronic computer. In general, automatic data handling was used wherever possible and the majority of the data were stored on IBM cards.

Data from the various measurement systems were classified into data systems, and a particular system was identified by a four digit code. Appendix I lists major Road Test data systems concerned with pavement research and notes how the systems may be obtained from the Highway Research Board. Major data systems from the bridge research are listed in Appendix A, Road Test Report 4.

The text of this report contains many references to data systems whose contents are pertinent to the discussion. These references are explained in Appendix I. For example, a reference to Data System 5121, or simply DS 5121, is explained in Appendix I as containing all routine Benkelman beam deflection data for flexible pavement sections on the traffic loops with an IBM printout of the data available on request.

Specific measurement programs are described in the appropriate sections of Parts 2 and 3.

1.2.6 Pavement Maintenance

Detailed descriptions of maintenance criteria and procedures are given in Road Test Report 3. Complete maintenance histories of each test section are available in DS 6300.

The objectives of the Road Test were concerned with the performance of the test sections as constructed. Consequently, maintenance operations were held to a minimum in any section that was still considered under study. When the "present serviceability" (see Section 1.3) of any section dropped to a specified level the section was considered to be out of test and maintenance or reconstruction was performed as needed.

Since the prime objective of the maintenance work was to keep test traffic operating as much as possible, minor repairs were made when required regardless of weather or time of day. The use of pierced steel landing mats permitted traffic to operate through a complete driving period so that more conventional repairs could be made during the daily 5-hr, 20-min traffic break.

All repairs were made with flexible-type pavement material. Deep patches and reconstruction consisted of compacted crushed stone base material surfaced with hot-mixed asphaltic concrete. Overlays consisted of asphaltic concrete. Thin patches were made either with hot-mix or cold-mix materials. Crushed stone base material and cold-mix surfacing were stockpiled at several locations on the project, and hot-mix asphaltic concrete was generally purchased from a nearby contractor.

As a general rule, pavement maintenance was done by project forces with project-owned equipment. However, in the critical spring periods of 1959 and 1960, it was necessary to augment the project maintenance forces with additional men and equipment.

1.2.7 Environmental Conditions

The topography of the Road Test area is level to gently undulating with elevations varying from 605 to 635 ft. Drainage is provided by several small creeks which are tributaries of the Illinois River. Surface drainage, how-

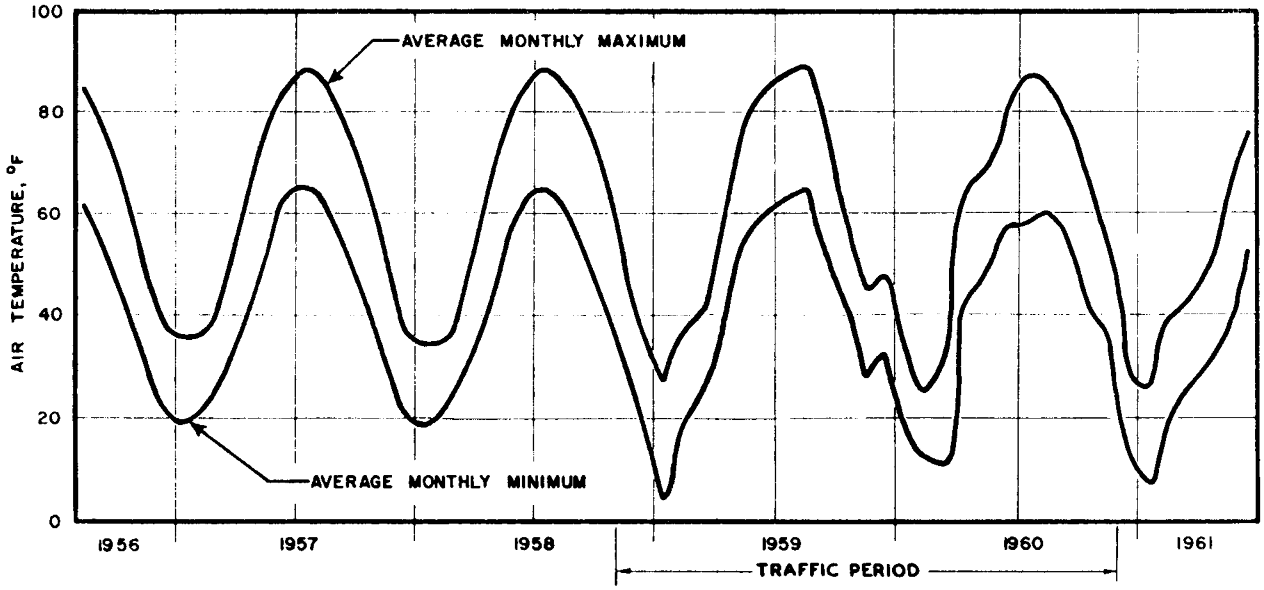


Figure 8. Average monthly air temperature at project.

ever, is generally slow. Geologic information indicates that the area was covered by ice during several glacial periods and that the subsurface soils were deposited or modified during these periods. Surface soils were subsequently derived from a thin mantle of loess deposited during a post-glacial period and were reasonably uniform in the area of the project. Soil drainage is generally poor. Bed rock is found 10 to 30 ft below the surface.

The upper layer of soil was from 1 to 2 ft thick and consisted generally of A–6 or A–7–6 soil with similar characteristics. The adjacent underlying stratum was usually from 1 to 2 ft thick and most of this material was fairly plastic A–7–6 soil. Substratum layers were usually represented by samples exhibiting A–6 characteristics.

In the interest of uniformity, soil making up the top 3 ft of embankment directly under the test pavements was taken from borrow areas near the project. This soil, underlying the surface stratum, was shown by tests to have a plasticity index from 11 to 15, a liquid limit from 27 to 32, and a grain size distribution of 80 to 85 percent finer than the 200 mesh sieve, 58-70 percent finer than 0.02 mm and 34-40 percent finer than 0.005 mm. Maximum dry densities were in the range 114 to 118 lb per cu ft and optimum moisture contents in the range of 14 to 16 percent when compacted in accordance with standard procedure, AASHO T99–49.

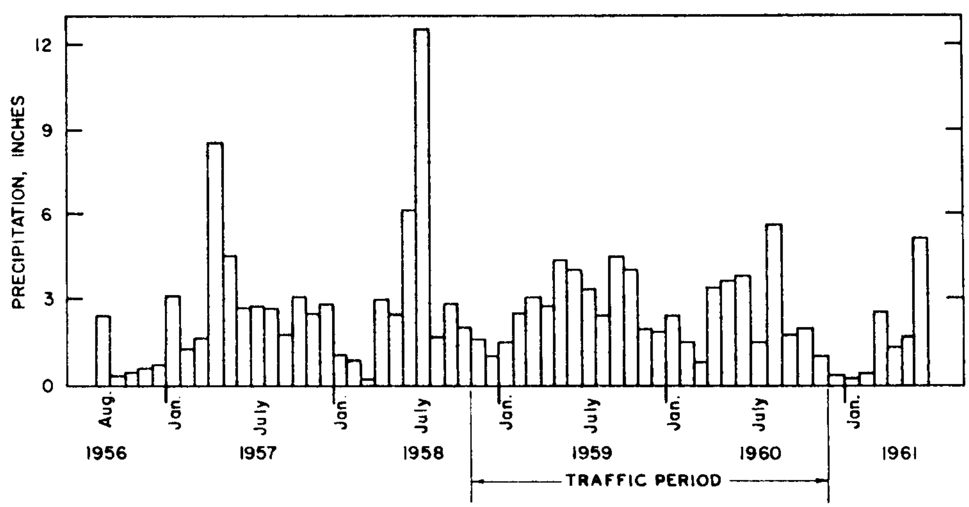


Figure 9. Precipitation at project.

The climate of the Road Test area is temperate with an average annual precipitation of about 34 in. of which about 2.5 in. occurs as 25 in. of snow. The average mean summer temperature is 76 F and the average mean winter temperature is 27 F. The soil usually remains frozen during the winter with alternate thawing and freezing of the immediate surface. Normally the average depth of frost penetration in the area is about 28 in.

Summaries of climatological data observed at weather stations on the project are given in Figures 8 through 10 and frost depth information in Figure 11. Depth of frost under the test pavements was obtained by means of special instrumentation involving the measurement of electrical resistance of the soil as described in *Highway Research Abstracts*, Vol. 27, No. 4. More detailed climatological and frost information is available in the form of IBM listings in Data Systems 3300, 3301, 3140 and 3240. Figure 12 summarizes the observations made at the project on the elevation of the water table under the test pavements and adjacent natural ground.

1.3 PAVEMENT SERVICEABILITY AND PERFORMANCE

1.3.1 Relation to Objectives

The first objective of the Road Test (see Section 1.1.3) asks for relationships between the performance of the pavement and the pavement design variables for various loads. In order to define performance, a new concept was evolved founded on the principle that the prime function of a pavement is to serve the traveling public. Briefly, it was considered that a pavement which maintained a high level of ability to serve traffic over a period of time was superior in performance to one whose riding qualities and general condition deteriorated at a more rapid rate under the same traffic. The term "present serviceability" was adopted to represent the momentary ability of a pavement to serve traffic, and the performance of the pavement was represented by its serviceability history in conjunction with its load application history.

Though the serviceability of a pavement is patently a matter to be determined subjectively, a method for converting it to a quantity based on objective measurements is given in the next two sections. Since the Road Test was concerned only with the structural features of the pavement, such items as grade, alignment, access, condition of shoulders, slipperiness and glare were excluded from consideration in arriving at a value for pavement serviceability.

The serviceability of each test section was determined every two weeks during the traffic testing phase, and performance analyses were based on the trend of serviceability with increasing number of load applications. The serviceability-performance concept is described in detail in Appendix F.

1.3.2 Rating of Pavements in Service

Serviceability was found to be influenced by longitudinal and transverse profile as well as the extent of cracking and patching. The amount of weight to assign to each element in

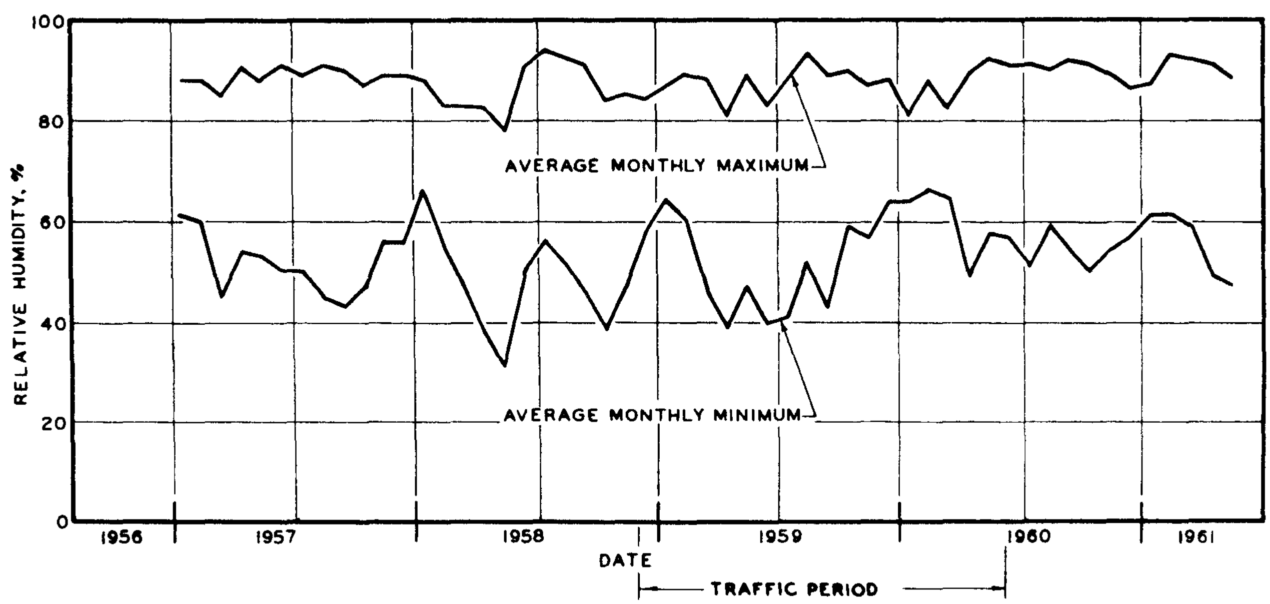


Figure 10. Relative humidity, weather station at Peoria, Ill.

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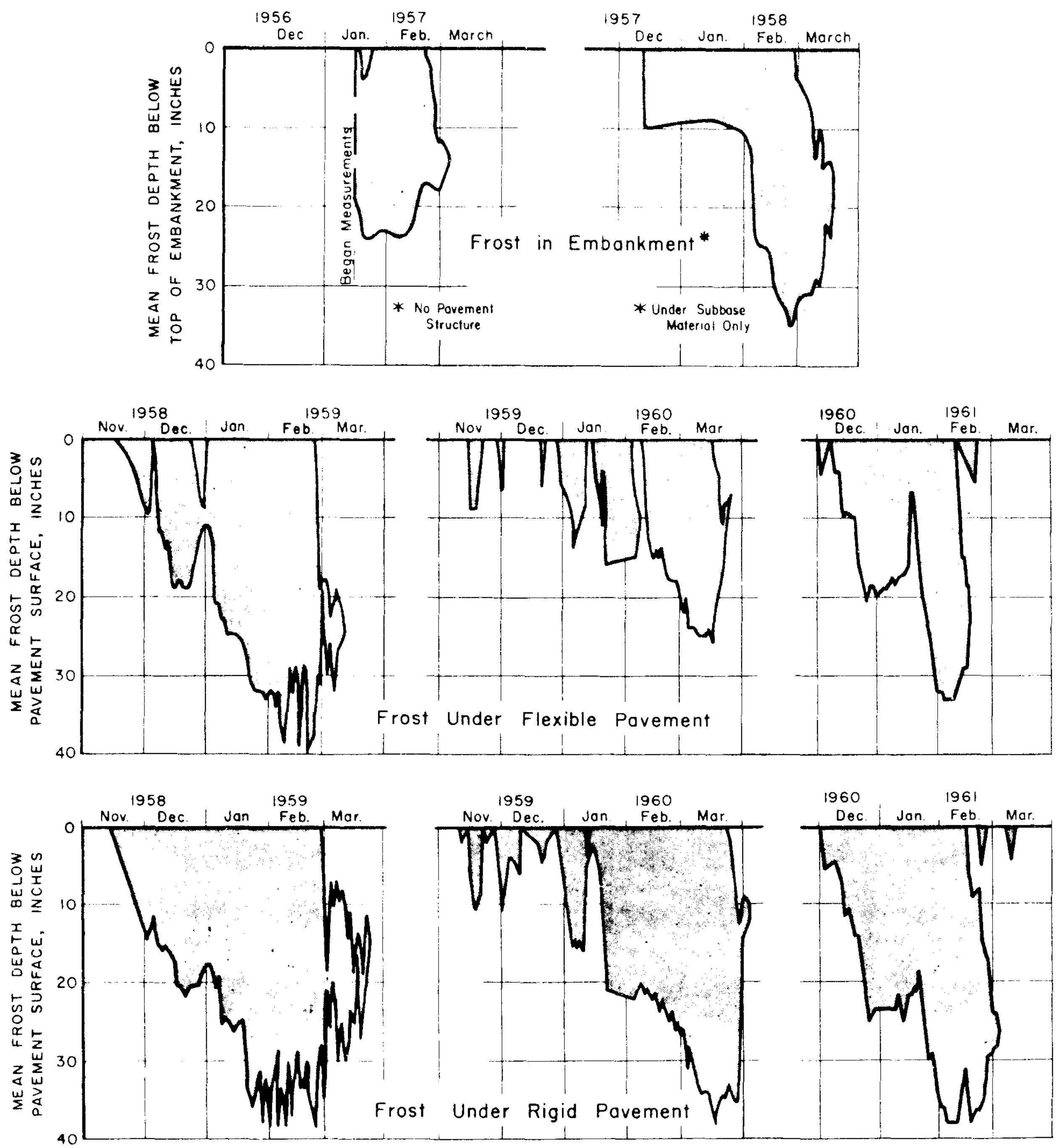

Figure 11. Frost depth.

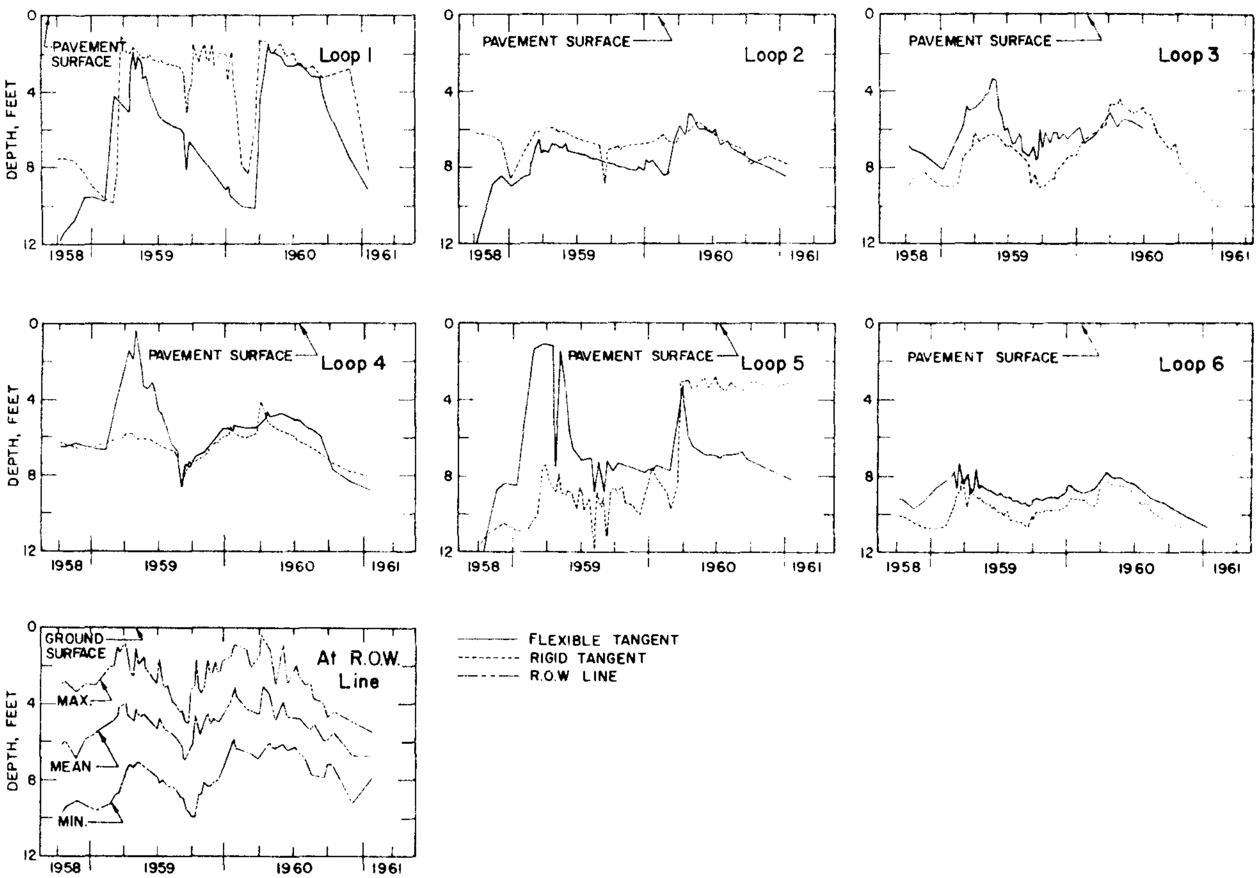

Figure 12. Water table data.

the determination of the over-all serviceability is a matter of subjective opinion. Furthermore, the degree of serviceability loss to be associated with a given change in any one of these elements depends on subjective judgment. To obtain a good estimate of the opinion of the traveling public in these subjective matters a Pavement Serviceability Rating Panel was appointed. This panel included highway designers, highway maintenance men, highway administrators, men with materials interests, trucking interests, automobile manufacturing interests and others. These men made independent ratings of the ability of 138 sections of pavement, located in three states, to serve high speed, mixed truck and passenger traffic. Both rigid and flexible pavements were included, and certain sections were selected for rating in each of five categories ranging from very poor to very good. The members were instructed to use whatever system they wished in rating each pavement and to indicate their opinions of the ability of the pavement to serve traffic at the time of rating on a scale ranging from 0 to 5 with adjective designations of very poor (0–1), poor (1–2), fair (2–3), good (3–4), and very good (4–5). For each section the mean of the independent ratings of the individual panel members was taken as the section's present serviceability rating. Some of the sections were rated more than once in order to determine the ability of the panel to repeat itself. Road Test field crews then measured variations in longitudinal and transverse profiles, as well as the amount of cracking and patching of each section.

1.3.3 Present Serviceability Index

Through a conventional statistical procedure (multiple regression analysis) it was possible to correlate the present serviceability rating with the objective measurements of longitudinal profile variations, the amount of cracking and patching and, in the case of flexible pavements, transverse profile variations (rutting). For either type of pavement this analysis resulted in a formula that used pavement measurements to compute a "present serviceability index" which closely approximated the mean rating of the panel.* The necessary measurements and serviceability index compu-

* A detailed discussion of the work of the Rating Panel, including the ratings, the data obtained in the measurements of the sections that were rated, and the derivation of the present serviceability indexes is presented in Appendix F.

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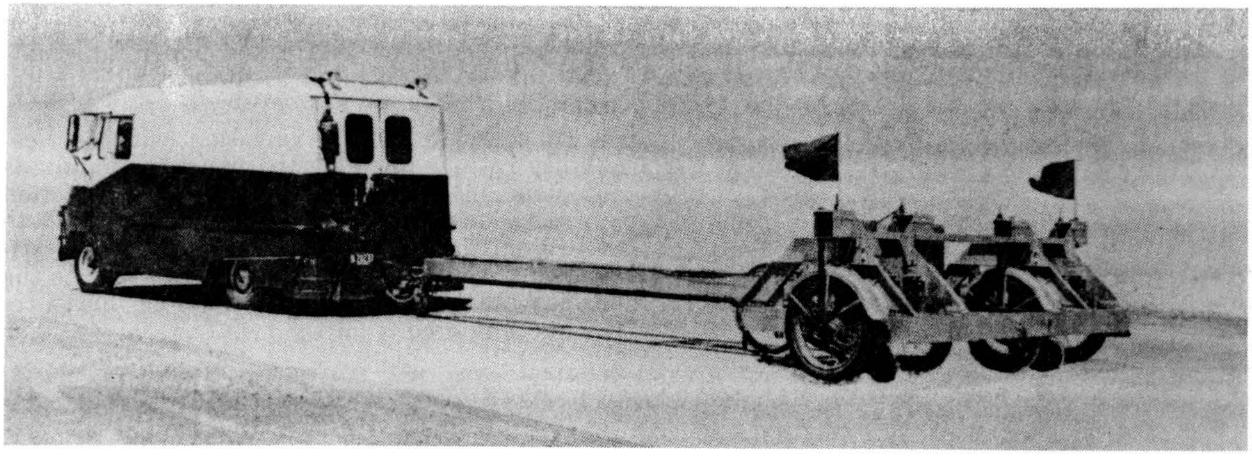


Figure 13. Longitudinal profilometer.

tations were made for each Road Test section at two-week intervals throughout the traffic phase.

Formulas for the present serviceability index, together with descriptions of the measurements entering into them, will be found in Chapters 2 and 3 for flexible and rigid pavement, respectively. The method of measuring longitudinal profile variations was the same for both pavement types and is described below.

The instrument used for recording longitudinal profile variations was the longitudinal profilometer pictured in Figure 13 and shown schematically in Figure 14. This instrument, moving at a speed of 5 mph, recorded continuously the angle, A, formed by the line of the support wheels G and H, and the line CD that connects the centers of two small (8-in. diameter) hard-rubber tired wheels, E, arranged in tandem. One pair of these wheels traveled in the center of each wheelpath.

Since the distance between the centers of the wheels, E, was small (9 in.) the line, CD, was assumed to be approximately parallel to the tangent to the road surface at the point, F, midway between the wheels.

The distance between the supports, G and H, of the tongue being relatively large (25.5 ft), the line GH was regarded as being approximately parallel to the pavement surface had it been perfectly smooth. Thus, the angle, A, between CD and GH represents a departure from a smooth pavement surface and variations in A represent variations in the longitudinal profile. It was this angle that the instrument was designed to measure. The effect of vibration of the tires and springs at G and H was held to a low level by restricting the operating speed and by electrically filtering out high frequencies so that they did not appear on the record.

It was recognized that line GH was not a stable reference and that as a consequence the

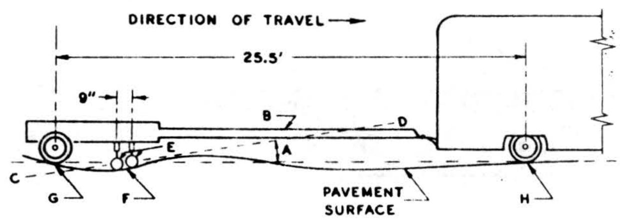
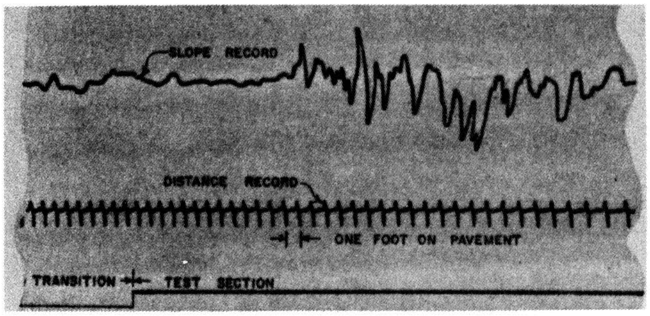


Figure 14. Schematic of longitudinal profilometer.

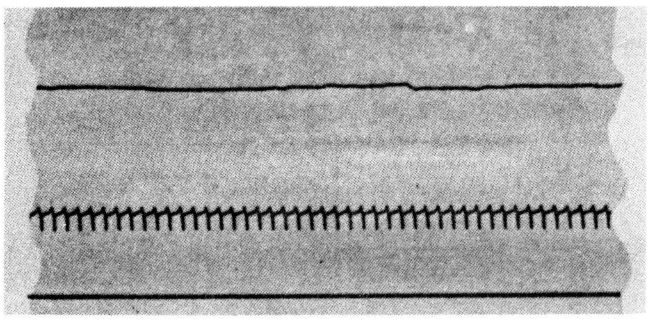
instrument could not respond correctly to gradual changes in the true pavement slope occurring over relatively long distances. Therefore, considerable effort was expended to develop a means to detect and correct for rotations of the line GH with respect to a horizontal reference. An inertial reference system was devised that would accomplish this purpose for short runs (that is, 2,000 ft). But tests of the effectiveness of the instrument with and without the reference indicated that the inconvenience of operation with the reference far outweighed the small increases in the over-all system effectiveness. Consequently, the inertial reference was abandoned.

The angle A rarely exceeded 3 deg even on rough pavements. Within the range of ±3 deg, the tangent of an angle is virtually equal to the radian measure of the angle, and thus the record of angle A could be interpreted as the slope of the pavement. In this report the profilometer output will be referred to as the pavement slope.

The instrument output on paper tape was a continuous analog of the slope of the pavement in each wheelpath, together with 1-ft distance marks along the margin of the tape (Fig. 15). The tapes were fed into an automatic electronic chart reader (Fig. 16) which measured the ordinate of the chart at intervals equivalent to



For Rough Pavement



For Smooth Pavement

Figure 15. Typical longitudinal profilometer record.

1 ft on the pavement, digitized this information and punched it on perforated paper tape suitable for use as an input to the project's digital computer.

To correlate profile variation with serviceability ratings made by the panel the hundreds of slope measurements taken in each section were reduced to a single statistic intended to represent the roughness of the section. Investigation of several alternative statistics led to the choice of the variance of the slope measurements computed from:

$$SV = \frac{\sum_{i=1}^{n} X_i^2 - \frac{1}{n}\left(\sum_{i=1}^{n} X_i\right)^2}{n-1} \qquad (1)$$

in which

SV = slope variance;
X_i = the i^{th} slope measurement; and
n = total number of measurements.

The slope variance for each section was calculated by the digital computer directly from the tape output of the chart reader. For use by other agencies, the Road Test staff has developed a simplified profilometer (Fig. 17), designated the CHLOE Profilometer, whose

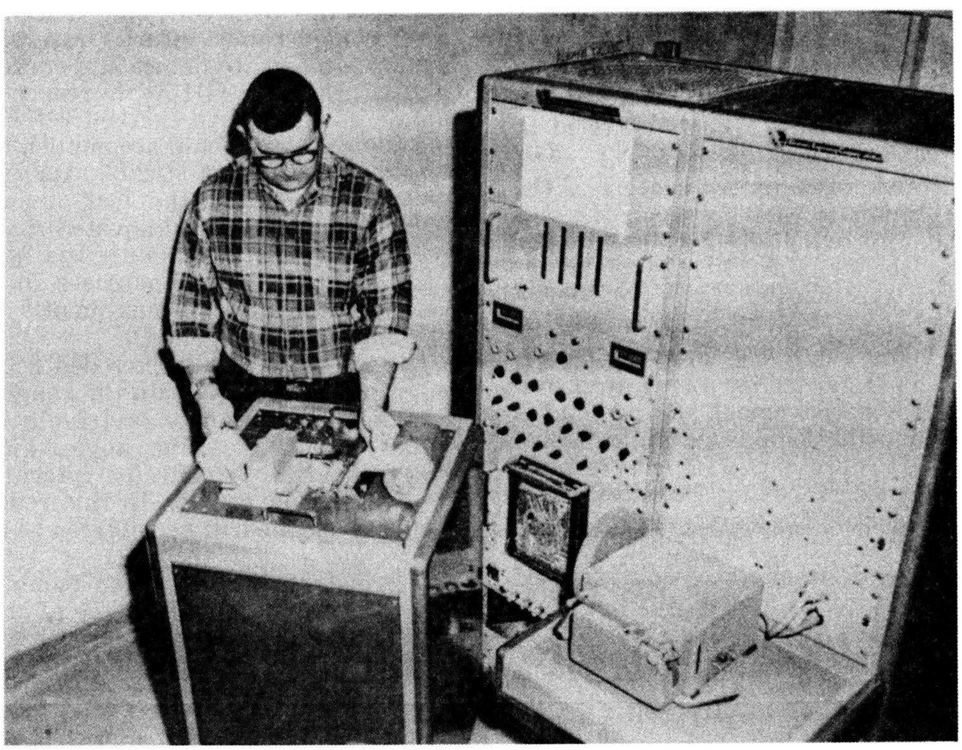


Figure 16. Electronic analog chart reader.

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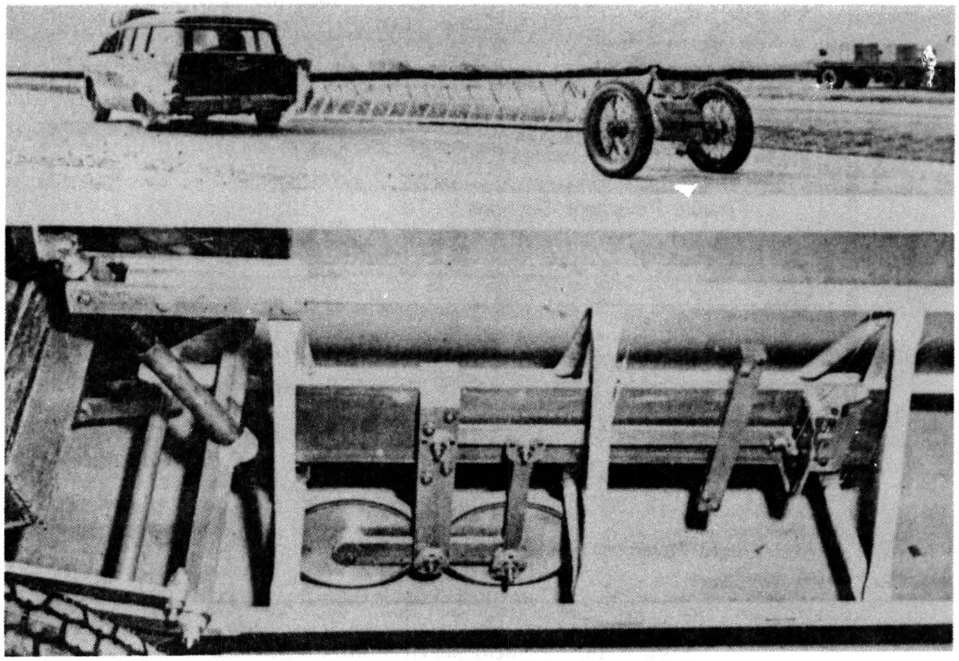


Figure 17. CHLOE profilometer.

output is slope variance. Thus, neither a chart reader nor a digital computer is required when the CHLOE Profilometer is used.

It was found that of the several types of measurements used in the serviceability index formulas, longitudinal profile variation of a section of pavement when represented by the logarithm of the slope variance correlated most highly with the rating of that section by the panel.

1.3.4 Pavement Performance Data

As stated in Section 1.3.1, pavement performance analyses were based on the trend of the serviceability index (determined at intervals of two weeks, or more often when required) with increasing axle applications. Prior to use in the analyses, performance data were identified and processed.

Each 2-week period was termed an "index period", and the last day of each period was called an "index day". Index days were numbered sequentially from 1 to 55, the first occurring on November 3, 1958, and the fifty-fifth on November 30, 1960. Because all sections had been subjected to almost the same number of applications of axle loads on any given date, the pairing of an index value with an index day was equivalent to specifying the serviceability index corresponding to a given number of axle applications. The symbol p_t' was used to represent the serviceability index of any section as determined by measurements made on the t^{th} index day, and the plot of p_t' versus time was termed the "serviceability history" of a section. (Usually the last three days of an index period were required to make the measurements on all sections for determining p_t'.)

The serviceability history of each section was converted to a "smoothed serviceability history" by a moving average that included at least three (generally five) successive index values except that the end values for the history were sometimes taken as end values for the smoothed history. Typical serviceability data and smoothed serviceability histories are shown in Figure 18.

The number of axle applications applied during the t^{th} index period, averaged over the ten traffic lanes, was represented by n_t, and the total number accumulated through that period by N_t; thus,

$$N_t = n_1 + n_2 + \ldots + n_t \qquad (2)$$

It was observed early in the traffic phase of the Road Test, confirming experience elsewhere, that for sections of insufficient design relative to load, the rate at which pavement damage accumulated with applications of load was affected by seasonal changes, especially in the case of flexible pavements. The design of the Road Test experiment did not permit a clearcut comparison of the damage rate in the various seasons since sections which failed in one season were not available for observation during subsequent seasons. Nevertheless Table 1, giving the percentage of failures occurring in each season for each type of pavement, suggests that the damage rate was relatively low in winter for both types of pavement and relatively high in spring for flexible pavements.

Changes in the effect of load with seasons

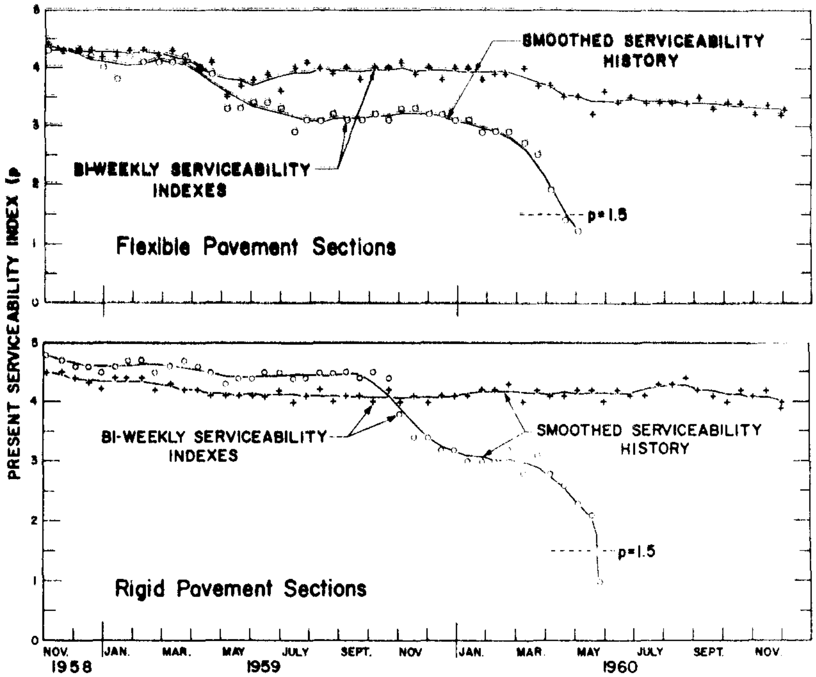


Figure 18. Typical serviceability histories.

TABLE 1
PAVEMENT FAILURE, BY SEASONS

Season	Axle Load Applications (× 10³)	Seasonal Distribution Section Failure[1] (%) Rigid	Flexible
Fall			
1958 Oct., Nov.	9	0	3
1959 Sept., Oct., Nov.	109	28	1
1960 Sept., Oct., Nov.	173	12	1
All	291	40	5
Winter			
1958-59 Dec., Jan., Feb.	64	0	4
1959-60 Dec., Jan., Feb.	167	11	5
All	231	11	9
Spring			
1959 March, April, May	59	0	57
1960 March, April, May	215	22	23
All	274	22	80
Summer			
1959 June, July, Aug.	109	3	3
1960 June, July, Aug.	209	24	3
All	318	27	6
Total	1,114	100	100

[1] A section was considered to have failed when its serviceability index dropped to 1.5. Table includes only factorial sections (first replicates) in Design 1.

suggested the use of a "seasonal weighting function," q_t, to be multiplied by the number of load applications made during each index period, with the value of q_t depending on some measurement designed to reflect the general variation above and below a "normal" value in the strength of the test sections. The function q_t presumably would take on values greater than unity during periods when the pavement was weaker than normal, and between 0 and 1 when stronger than normal. The product, $q_t n_t$, would then yield "weighted applications," w_t, corresponding to the actual application, n_t, made on each test section during an index period. The total number of weighted applications, W_t, would be given by

$$W_t = q_1 n_1 + q_2 n_2 + \ldots + q_t n_t \qquad (3)$$

Weighted application, W_t, could then be substituted for actual applications, N_t, in the performance analyses. (Hereafter W will be used to represent either weighted or unweighted axle applications, the meaning of the symbol being specified wherever used.)

A seasonal weighting function, dependent on the periodic measurement of flexible pavement deflections in Loop 1, was developed and used in an analysis of flexible pavement performance described in Section 2.2. In the case of rigid pavements, although all rigid pavement distress was associated with pumping and although pumping must be associated with periods of high rainfall, the seasonal variations in damage rate were less pronounced, and no effective function was developed.

For the analyses of pavement performance it was assumed that the trend of serviceability, p, with increasing axle application, W, could be satisfactorily represented by five pairs of coordinates. For sections that failed during the test period, simultaneous values of p and W were taken at $p = 3.5$, 3.0, 2.5, 2.0 and 1.5. For sections that survived the traffic testing period, the coordinates were chosen from the smoothed serviceability history at 11, 22, 33, 44 and 55 index days. Sets of coordinates from the serviceability trend, that is, performance data, for each Road Test section are given in Appendix A.

1.3.5 Procedures for Analysis

The analyses of performance resulted in empirical formulas wherein performance was associated with load and pavement design variables. To use mathematical procedures for the analyses it was necessary to assume some analytical form or model for these associations. In addition to the experimental variables the models include constants whose values were either to be specified or to be estimated from the data. Thus the analytical procedures were for the estimation of constants whose values were unspecified in the model—constants that indicate the effects of design and load variables upon performance. The procedures also included methods for estimating the precision with which the data fit the assumed model. The procedures used in the Road Test analyses are set forth in detail in Appendix G.

There are many different mathematical forms that could be used as models for serviceability trends, and several of these may fit the data with more or less the same precision. Different models were tested for goodness of fit to the Road Test performance data. Preference for one model over another was governed mainly by relative goodness of fit, but consideration was also given to relative agreement with highway design practice and experience for traffic conditions beyond the Road Test.

The mathematical model ultimately chosen for both the flexible and rigid pavement analyses is of the form

$$p = c_0 - (c_0 - c_1)\left(\frac{W}{\rho}\right)^{\beta} \qquad (4)$$

in which

$c_1 \leqq p \leqq c_0$;

p = the serviceability trend value;

c_0 = the initial serviceability trend value (for the Road Test $c_0 = 4.5$ for rigid pavements, and 4.2 for flexible pavements—these values were the means of the initial serviceability of test sections);

c_1 = the serviceability level at which a test section was considered out of test and no longer observed (for the Road Test $c_1 = 1.5$);

W = the accumulated axle load applications at the time when p is to be observed and may represent either weighted or unweighted applications.

ρ and β are functions of design and load to be discussed later. Rearranging Eq. 4 in logarithmic form, and defining G, a function of serviceability loss, as $\log\ (c_0 - p)/(c_0 - c_1)$ gives

$$G = \beta\ (\log W - \log \rho) \qquad (5)$$

Plotting G against $\log W$ for Eq. 5 gives a straight line whose slope is β and whose intercept on the $\log W$ axis is $\log \rho$. For each Road Test section the performance data given in Appendix A were converted into values for G and $\log W$ and a straight line was fitted to the G, $\log W$ points. From these straight lines, estimates of β and $\log \rho$ were obtained for each test section. For the cases where the serviceability loss was very small over the traffic testing period β may be nearly zero and $\log \rho$ extremely large. Special rules were applied for these cases in order to obtain logical values of β and $\log \rho$ (see Appendix G).

The assumed relationship between β and the design and load variables was

$$\beta = \beta_0 + \frac{B_0\ (L_1 + L_2)^{B_2}}{(a_1D_1 + a_2D_2 + a_3D_3 + a_4)^{B_1}\ L_2^{B_3}} \qquad (6)$$

in which

β_0 = a minimum value assigned to β;

L_1 = the nominal load axle weight in kips (*e.g.*, for 18,000-lb single axle load, $L_1 = 18$; for 32,000-lb tandem axle load, $L_1 = 32$);

L_2 = 1 for single axle vehicles, 2 for tandem axle vehicles;

D_1, D_2 and D_3 = the three pavement design factors surfacing, base and subbase thickness for flexible pavement and reinforcement, slab thickness and subbase thickness for rigid pavement.

The remaining symbols of Eq. 6 are positive constants whose values were either to be assigned as was done for β_0 or to be estimated by means of the analysis.

Equations in this same form were determined from analysis of the rigid pavement data and the flexible pavement data, respectively.

The analysis rationale assumes that estimates for β from the equation are better than estimates based only on the individual section performance data. Consequently, the values of β estimated from the equation were used in conjunction with the data to obtain new estimates of log ρ for every test section.

The algebraic form assumed for the association of ρ with the design and load variables is

$$\rho = \frac{A_0 (D + a_4)^{A_1} L_2^{A_3}}{(L_1 + L_2)^{A_2}} \tag{7}$$

where D ($= a_1 D_1 + a_2 D_2 + a_3 D_3$) represents a "thickness index" of the pavement, L_1 and L_2 are as defined for Eq. 6, and the remaining symbols are constants whose values are either to be assumed or to be estimated from the analysis.

Evaluation of the constants in Eqs. 6 and 7 is reported in Section 2.2.2 for flexible and 3.2.2 for rigid pavements.

Eqs. 6 and 7 when evaluated and used in conjunction with Eq. 5 thus represent the first goal of the Road Test—to associate performance with design and load variables.

At various stages in the development of the equations, tests were made for the significance of pavement design factors, and statistics were computed to express the degree of correlation between observations and corresponding predictions from the equations. Finally, average residuals were used to indicate the extent to which observations were scattered from the corresponding calculated values of p and log W. Average residuals, correlation indexes, and inferences from the significance tests are summarized after presentation of derived equations in Sections 2.2.2 and 3.2.2.

Many different models and fitting procedures were studied and one selected from which the performance equations fit the Road Test data with satisfactory precision. In time, other models may be found that also fit the data satisfactorily and which may prove equally or more useful.

1.4 NEEDED RESEARCH—GENERAL

1.4.1 Modification of Performance Relationships

Any further effort by the Highway Research Board to fit a mathematical model to the Road Test performance data will likely involve modifications either in the basic models for p, β, and ρ, or in the fitting procedures, or in both. It is the purpose of this section to mention several possibilities for both types of modification that are contemplated in further work with the performance data.

Even if no changes are made in Eq. 4, it is possible to modify the formulas for β and ρ. For example, it might be assumed that β is a constant,

$$\beta = b_0 \tag{8}$$

or that β is a simple function of ρ, for example,

$$\beta = b_0 + \frac{b_1}{\rho^{b_2}} \tag{9}$$

The concept of a thickness index for flexible pavements might be generalized after further research to a "structural index," S, where S would account for all pavement layers (their thicknesses and strengths) as well as the embankment soil. A single index for vehicle load, L, might be introduced so that L could account for all axle loads (including steering axles) and their spacing. Then it might be assumed that

$$\rho = \left(\frac{S}{\sqrt{L}}\right)^{4} \tag{10}$$

so that the structural index is squared relative to the load index. It may be noted that the ratio of A_1 to A_2 in Eqs. 18 and 21 (see Section 2.2) is already of the order two to one, so that Eq. 10 appears to be a reasonable assumption at least for flexible pavements.

As is explained in Appendix G, performance equations developed for the present report result from a step-by-step fitting procedure where the results of one step are used as input for the next step. Modification of the fitting procedures will likely take the form of an over-all procedure that determines all unassigned constants simultaneously as a particular residual criterion is minimized. Once the over-all fitting procedure is developed, the residual criterion can include both residuals from log W estimates and residuals from p estimates. Moreover, performance data from experiments that have been analyzed separately in this report may be combined in an effort to obtain a more general analysis.

Although it was not possible to investigate modifications of the type just described in time for inclusion in this report, the Highway Research Board will undertake these studies. It is hoped that further effort will produce modified equations that can represent all the Road Test performance data with at least the same precision as given in this report and that simplifications can be introduced with little sacrifice in precision over the equations reported herein.

1.4.2 Generalization and Extension of Relationships

Discussion in the preceding subsection relates to the need for additional study of the data obtained in the Road Test. A larger area for future research involves the extension of the performance equations to include parameters that were not varied in the project. It is important to know, for example, the effects on pavement performance of variations in the characteristics of the soil and the materials used in the pavement structure. The effects of environment need study. Not only the differences in performance associated with the existence of heavy rainfall, desert conditions, frost, etc., must be considered, but also the differences that may be associated with different rates of traffic application and distribution of axle loads in the traffic stream. (For example, at the Road Test a million axle loads of one weight were applied in two years to each section. What would have been the situation had these loads, accompanied by several million lighter loads, been applied in 20 years?)

Studies designed to fill these gaps may fall in four categories: (1) theoretical studies, (2) major satellite studies, (3) field tests, and (4) laboratory tests.

There should be continuing encouragement of research into the mechanical and physical laws involved in pavement performance. Only through such theoretical work will there be developed rational mathematical models by which performance can be related to the fundamental properties of materials and to the dynamic characteristics of the loading.

Since the completion of such theoretical work appears to be years away, immediate attention should also be given to means for extending the empirical models developed at the Road Test to include additional important parameters. A most effective device for this purpose is the so-called satellite study. These studies have been described* as relatively small road tests in different parts of the country (and other countries) involving consideration of variables most of which were not included in the AASHO Road Test. A very important finding of the Road Test was that, within the range of precision of measurements systems and estimation techniques available, no significant interactions were found among the design variables. Therefore, in the design of satellite experiments where the variables are like those in the Road Test (structure thickness, base type, etc.) balance in the experiment can be attained through the use of partial rather than full factorials.** This means that to test a given number of variables any satellite experiment will require only a small fraction of the test sections that would have been required had the AASHO Road Test shown that significant interactions existed.

Such satellite experiments are also different from the Road Test in that traffic is not a variable. The test sections would be constructed as part of the regular highway system and their serviceability trends observed under the normal traffic using the facility. A careful record of the number and magnitudes of axle loads over the test sections would be required.

These experiments would provide for verification of the coefficients in the Road Test performance equations and for the inclusion of terms in the equations relating to variables that were not under study in the AASHO Road Test. More specific areas for study in the satellite experiments are discussed at the ends of Chapters 2 and 3.

Field tests would be simple pavement performance experiments, with 2 or 3 test sections each, constructed as part of normal highway construction in a large number of locations where only one or two variations from normal pavement design would be observed along with the normal design. These studies would prove very useful to engineers who must use judgment in the application of Road Test findings and in their attempts to evaluate new designs and new materials. However, the field tests would not be designed in such a way as to permit analyses that would result in important modification of the Road Test equations themselves. Many states have constructed test pavements in the field test category in the past. If traffic records are available, further study of these pavements would be extremely useful.

Laboratory tests are those needed in the study of materials characteristics as they might affect pavement performance. Here again more detailed recommendations are given at the ends of Chapters 2 and 3.

1.4.3 Serviceability of Pavements

It is believed that the serviceability-performance concept developed at the Road Test has added a new technique of value in the design and maintenance of highway pavement. It is emphasized, however, that the specific serviceability indexes developed for the Road Test, were based on very small samples of the American highway network by a very small group of highway engineers. There is no reason to think that more extensive sampling will result in major modification of these indexes, but if the system is to receive widespread use, it is imperative that other groups, working under the same rules as the Road Test Rating Panel, make subjective ratings of many sections of pavement over the entire country containing many types of distress leading to loss of serviceability. Accompanying these rating sessions should be objective measurements of those elements that may be involved in serviceability such as, slope variance (roughness), rut depth, cracking, faulting, patching, and slipperiness. Regression analyses of the ratings in terms of the objective measurement data will produce new more generally applicable serviceability indexes.

* "Extending the Findings of the AASHO Road Test" before the Design Committee, AASHO, at the AASHO meeting in Denver, Colo., October 1961.

** See Hain, R. C., and Irick, P. E., "Fractional Factorial Analysis," HRB Road Test Conference, May 1962.

LESSON OUTLINE

TRAFFIC AND TRUCK LOADING DATA

Instructional Objectives

1. To outline the basic type and uses of traffic data and its relation to highway engineering.

2. To familiarize the student with the weigh-in-motion equipment and capabilities.

Performance Objectives

1. The student should develop a feel for the various types of traffic data that can be collected and the relative importance of each variable in the different highway engineering phases.

2. The student should understand the use of weigh-in-motion equipment and how to incorporate this equipment in his or her agencies data collection scheme.

Abbreviated Summary	Time Allocation, min.
1. Vehicle and Traffic Considerations	15
2. Traffic Variables	15
4. Traffic Data Collection	20
	50

Reading Assignment

1. Haas and Hudson - Chapter 13.1, 13.3, 14.1, 14.2

2. RTAC - Canadian Guide - Part 4

3. Yoder & Witczak, Chapter 4

Additional Reading

1. Lin, Han-Jei, Clyde E. Lee, and Randy Machemehl, "Texas Traffic Data Acquisition Program," Research Report 245-1F, Center for Transportation Research, The University of Texas at Austin, February 1980.

LESSON OUTLINE
TRAFFIC AND TRUCK LOADING DATA

1.0 VEHICLE AND TRAFFIC CONSIDERATIONS

Traffic data is essential for investment programming, structural and geometric design, certain aspects of construction, and maintenance functions. Volumes, loads, and vehicle classification needs to be known for:

1.1 Investment Programming

Traffic data is used in benefit-cost analysis and project economic evaluation.

1.2 Structural and Geometric Design

The prediction of performance is directly linked to vehicle type, traffic volume, and mode of operation of the vehicle.

1.2.1 Vehicle Types. To insure adequate structural and geometric design , all types of vehicles expected to be encountered in design life must be considered.

1.2.2 Vehicle Movements. The volume of specific movements per vehicle type including lateral and/or longitudinal variation of vehicular load must be counted or approximated as accurately as possible.

1.3 Maintenance

Traffic volume is of special concern when maintenance work must be carried out during peak demand times. The type of loads is again a critical item in determining the optimum maintenance strategy from a structural standpoint.

2.0 TRAFFIC VARIABLES

Traffic loading and variation comprises one of the most difficult classes of variables confronting the pavement engineer. Actual values can vary markedly from design estimates and thus result in actual performance that may be significantly different from that originally predicted. There are several potentially important traffic variables, including the following (Ref 1):

(a) Wheel load, axle load, and total vehicle load.
(b) Number of load applications, and their sequence.
(c) Vehicle speed.
(d) Lateral and lane distribution of loads.
(e) Tire pressures.
(f) Wheel and gear configurations
(g) Environmental factors.

For design purposes, the variation in axle loads is usually handled through reducing to an "equivalent axle load" basis, as discussed more in a later lesson.

2.1 Allowable Wheel Loads and Configurations

When calculating the effect of wheel loads to the pavement structure, particular concern must be placed on the duration of the load, how the load is distributed to the pavement, and the magnitude of the load (wheel/axle/gross) proximity of wheel load, and the number of repetitions of load.

2.1.1 Allowable Loads in the Interstate System. The legal limits for axle loads, and gross vehicle weight in most states are as follows:

Maximum load (gross)	=	80,000 lbs
Single Axle load	=	20,000 lbs
Tandem Axle load	=	34,000 lbs

2.1.2 Wheel Configuration. The common arrangement of axles is on either a single or a tandem basis. A typical example of the wheel configuration is shown in the Visual Aid 9.1.

2.1.3 Proximity of Wheel Load. By distributing the load to tandem wheels the area of the overlap from the two loads creates less stress than the maximum allowable single load (Visual Aid 9.2).

2.1.4 Bridge Formula (Visual Aid 9.3). The maximum weight that can be carried on a group of two or more axles without overstressing highway bridges can be obtained from the equation below:

$$W = 500\left(\frac{LN}{N - 1 + 12N + 36}\right)$$

where

L = spacing in feet between the outer axles of any two or more consecutive axles

N = number of axles being considered

Spreding the load according to this formula also has beneficial effects in preventing overloading of highway pavements.

4.0 TRAFFIC DATA COLLECTION

4.1. Common Methods

Most agencies have well established procedures for obtaining traffic volumes and vehicle classification counts and for measuring axle loads at selected sites.

4.1.1 Speed. Usually obtained by radar at selected locations. Automatic speed monitoring devices using loop detectors are also used.

4.1.2 Vehicle Type and Lane Distribution. Many states utilize human observers at roadside stations in conjunctions with automatic traffic counters to get percentage truck figures and lanewise truck volume counts.

4.1.3 Vehicle Weight (Truck Weight).

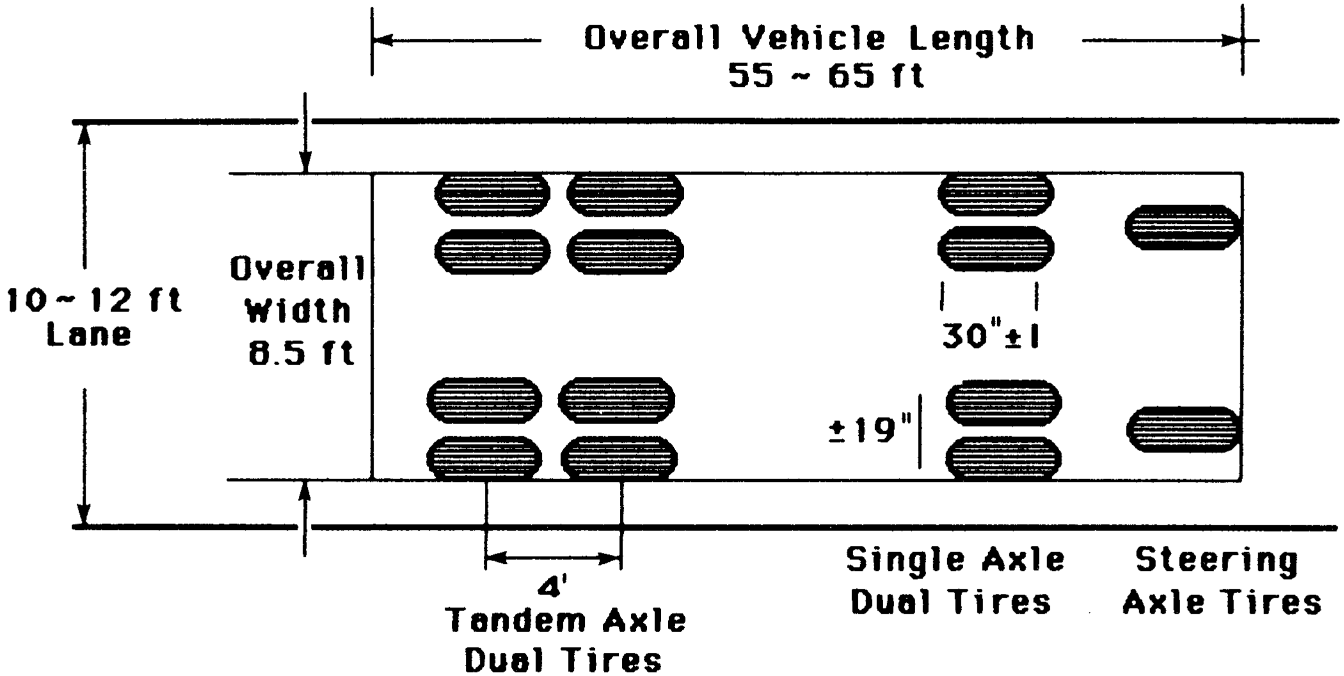
(a) Conventional Weigh Station. The conventional weigh station utilized a full time crew, say six employees, for static weight detrminations and for vehicle size measurements. A typical site consists of a paved roadway section parallel to the existing traffic lane on both sides of the highway. A level weighing area is located adjacent to a small recessed metal-lined pit in which a static wheel-load weigher is placed during survey operations. Surveying is done at select locations throughout the state on a routine schedule.

(b) In Motion Vehicle Weighing. This system has the capability of measuring vehicle wheel weights while vehicles move in a normal traffic lane at highway speeds. The system determines and records dynamic wheel forces in each wheel path of the traffic lane, axle spacings, vehicle speed, number of axles per vehicle, and time of day. From these measurements, summary statistics including axle weights, gross vehicle weight, and wheel base are automatically computed. More advanced systems can measure up to four lanes at one time, give immediate computation of wheel weight, axle weight, gross weight, axle spacing, vehicle length, vehicle type, and speed. Suspected violations of weight limits, including, bridge formula, can be indicated automatically and tabulated in statistical summaries.

(c) WIM Experience (Visual Aid 9.4). The first successful WIM system in the US was developed in Texas between 1964 and 1969 for collecting statistical data. The commercial version of this system was marketed by UNITECH, Inc. of Austin, Texas until RADIAN Corporation, also of Austin, Texas, bought UNITECH and began marketing under their name. Visual Aid 9.4 shows the application of Radian systems in statistical data collection, enforcement-aid, and research applications. Other manufacturers including PAT (Siemens-Allis) from Germany, Golden River Corporation from Great Britain, CMI-Dearborn with a Canadian design, Streeter Amet, and Bridge Weighing Systems now offer commercial weigh-in-motion systems of various types. Their experience like Radian's is diverse and is continually changing as WIM is recognized as a feasible technique for obtaining truck weight information.

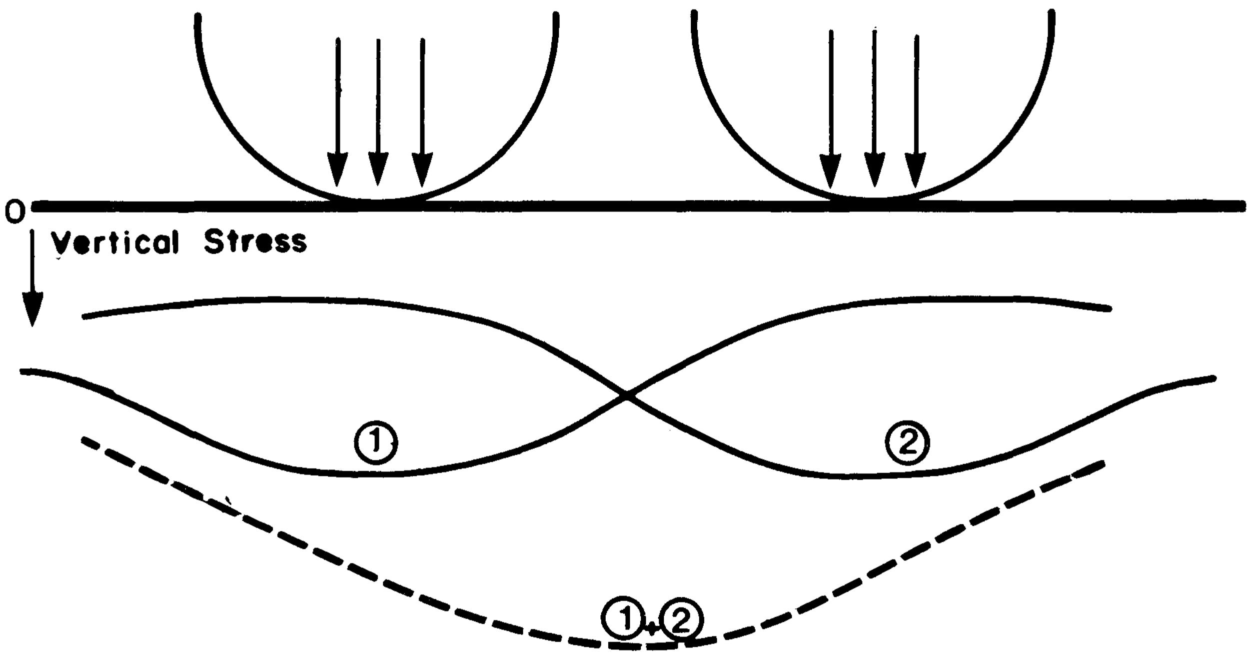
LESSON OUTLINE
TRAFFIC AND TRUCK LOADING DATA

VISUAL AID	TITLE
Visual Aid 9.1.	Wheel configuration.
Visual Aid 9.2.	Proximity of wheel load.
Visual Aid 9.3.	Bridge formula.
Visual Aid 9.4.	Radian WIM experience.



Visual Aid 9.1. Wheel configuration.

Visual Aid 9.2. Proximity of wheel load.



Visual Aid 9.3. Bridge Formula.

Permissible gross loads for vehicles in regular operation

Based on weight formula $W = 500 \left(\frac{LN}{N - 1 + 12N + 36} \right)$ modified

	Distance in feet between the extremes of any group of 2 or more consecutive axles	2 axles	3 axles	4 axles
	4	34,000		
Tandem Axle (by definition)	5	34,000		
	6	34,000		
	7	34,000		
	8	34,000	34,000	
	9	39,000	42,500	
	10	40,000	43,500	
	11		44,000	
	12		45,000	50,000
	13		45,500	50,500
	14		46,500	51,500
	15		47,000	52,000
	16		48,000	52,500
	17		48,500	53,500
	18		49,500	54,000
	19		50,000	54,500
	20		51,000	55,500
	21		51,500	56,000
	22		52,500	56,500
	23		53,000	57,500
	24		54,000	58,000
	25		54,500	58,500

The permissible loads are computed to the mearest 500 pounds. The modification consists in limiting the maximum load on any single axle to 20,000 pounds.

W = the maximum weight in pounds that can be carried on a group of two or more axles to the nearest 500 pounds
L = spacing in feet between the outer axles of any two or more consecutive axles
N = number of axles being considered

Visual Aid 9.4. Radian WIM Experience, Circa 1983.

Customer	Weighing Sites	Type	Since	Application
Domestic				
Texas Highway Department	3	P	71	Planning
Florida Dept. of Transportation	18	P	73	Planning
Louisiana Dept. of Highways	1	P	74	Research
New Mexico Highway Department	12	P	74	Planning
Nevada Highway Department	15	P	78	Enforcement/Planning
Georgia Dept. of Transportation	8	F	78	Enforcement
Alabama Highway Department	11	P	79	Enforcement/Planning
Idaho Dept. of Law Enforcement	4	F	80	Enforcement
Virginia Dept. of Transportation	2	F	80	Enforcement
Mississippi Dept. of Transportation	2	F	81	Enforcement
Wyoming State Highway Dept.	2	F	82	Enforcement
Total Sites	**78**			
Foreign				
Brazil-UNDP	1		75	Research
Argentina-DNV	1		79	Enforcement

P = Portable
F = Fixed

Revised CL/lg 1/27/84
Lesson 9

INSTRUCTIONAL TEXT

CONCEPTS OF WEIGH-IN-MOTION SYSTEMS

by

Clyde E. Lee
Phil M. Ferguson Professor in Civil Engineering
The University of Texas at Austin

NATIONAL WEIGH-IN-MOTION CONFERENCE

Denver, Colorado, July 11-15, 1983

WIM Technology Session, Wednesday, July 13, 1983

CONCEPTS OF WEIGH-IN-MOTION SYSTEMS

by

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In order to understand the complex technical requirements for a highway vehicle in-motion weighing system, it will be instructive to review some basic principles of physics and to define a few terms that are used in engineering mechanics to describe the static and dynamic behavior of objects which exist in the earth's gravitational field.

Weight is the force with which an object is attracted toward the earth by gravitation; it is equal to the product of the mass of the object and the local value of gravitational acceleration. For practical purposes in weighing highway vehicles, gravitational acceleration can be considered constant at 32.2 ft/sec^2 for all locations.

Mass is the measure of the resistance of an object to acceleration, or its inertia. Mass is commonly taken as a measure of the amount of material which makes up an object and causes it to have weight in a gravitational field.

Acceleration is the time rate of change of velocity.

Velocity is the time rate of change of displacement.

Force is that which changes, or tends to change, the state of motion of an object.

Newton's Laws are applicable in defining the state of motion of a highway vehicle at any given instant of time.

1. There is no change in the motion of an object unless an unbalanced force acts upon it.
2. Whenever an unbalanced force acts on an object, it produces an acceleration in the direction of the force; an acceleration that is directly proportional to the force and inversely proportional to the mass of the object.

These concepts can be applied to weighing highway vehicles and interpreted first for the static (no-motion) case and then for the dynamic (in-motion) case. A highway vehicle is made up of several interconnected components, each with its own mass. The connectors, which also have mass, can be viewed as springs, hinges, and motion dampers. A force applied to any component will be transferred to the others through the connectors. (See Fig 1)

STATIC WEIGHING

To weigh a vehicle, a total upward force exactly equal to the downward force of gravity is applied through the motionless (in the vertical direction) tires of the vehicle and measured simultaneously by scales (force transducers) or a balance. This is known as static, single-draft weighing and is the most accurate means of determining gross vehicle weight.

Gross weight can also be determined accurately by successively measuring the downward force on the tires with all the vehicle components motionless and in exactly the same relative position to each other throughout the entire weighing sequence. This condition of juxtaposition can be approximated in practice, but rarely achieved. The center of oscillation of the composite vehicle mass usually changes when the vehicle is moved; therefore, the distribution of the total downward force among the tires changes. Some sacrifice in weighing accuracy can thus be expected if the vehicle is moved between successive tire force measurements as is the case when using axle load or wheel load weighers. This is especially true when the vehicle is moved several times and the weighing surface of the scales is not in the same horizontal plane as the surrounding surfaces supporting the tires which are not being weighed at the time.

A typical spring rate for a rear truck wheel suspension is about 3,500 to 4,000 pounds per inch of displacement and each tire also has a rate of about 4,000 pounds per inch. The front suspension generally has a spring rate of about 500 pounds per inch. Thus, if one wheel of a vehicle is raised or lowered with respect to the others during weighing, the wheel force on the scale, or weigher, will be considerably different than when the wheel is not displaced. Particular attention must be given to this concept when weighing the wheels of tandem or triple axles if reasonable accuracy is to be achieved with wheel load weighers. The same principles also apply to weighing axles and axle groups

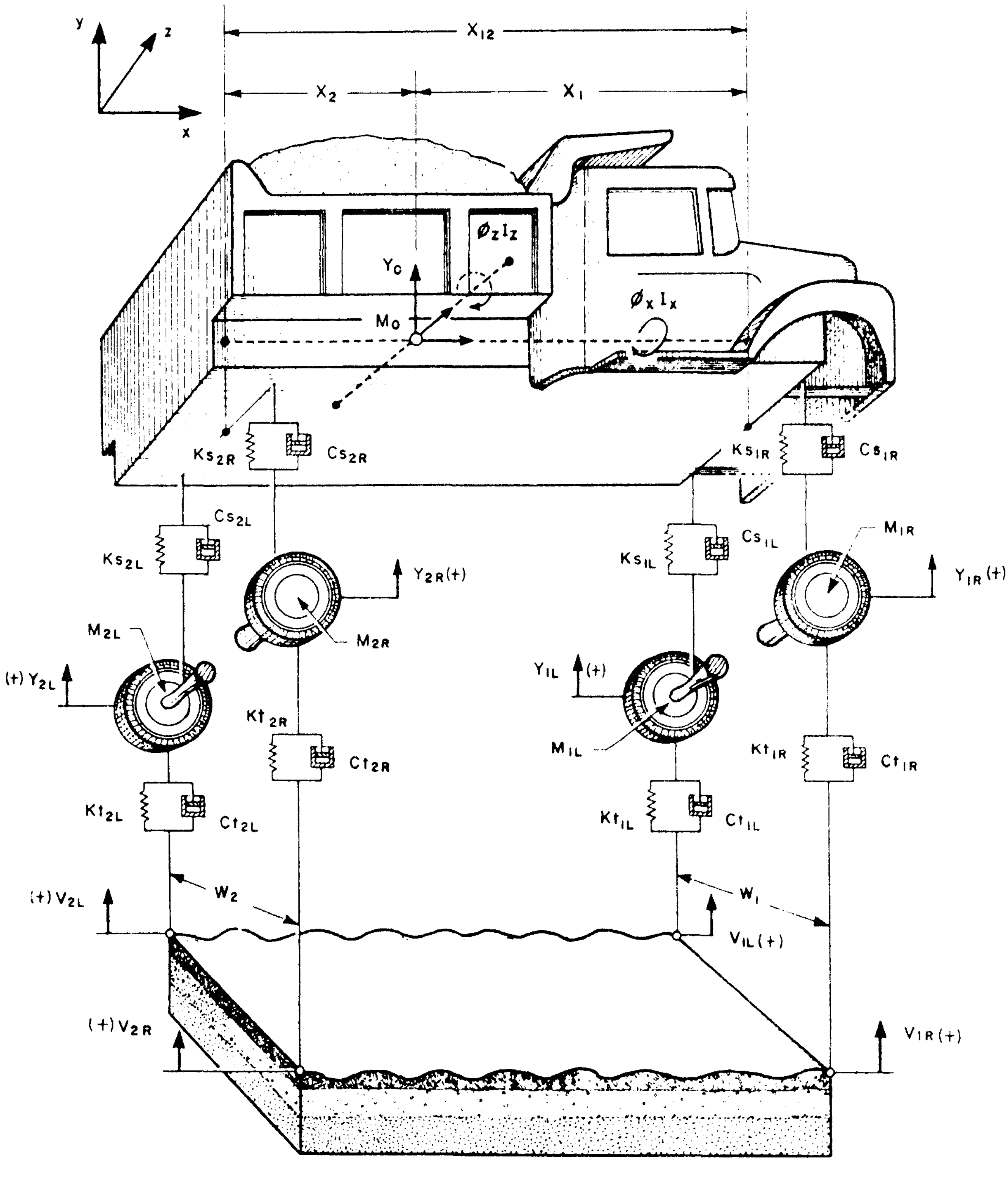

y
z
x
X12
X2
X1
ØzIz
Yo
Mo
ØxIx
Ks2R
Cs2R
Ks1R
Cs1R
Cs2L
Ks2L
Y2R(+)
Cs1L
Ks1L
M1R
Y1R(+)
M2L
M2R
(+) Y2L
Y1L
(+)
Kt2R
Ct2R
M1L
Kt1R
Ct1R
Kt2L
Ct2L
Kt1L
Ct1L
(+) V2L
W2
W1
V1L(+)
(+) V2R
V1R(+)

Figure 1

with sets of wheel load weighers or with axle scales. The only way to weigh a highway vehicle accurately by successive positioning of wheels on a scale, or a series of scales, is to maintain all wheels of the vehicle in a horizontal plane (a smooth level surface) and have no redistribution of weight during the weighing process. This means that the deflection of the scale itself must be considered and that the friction in the vehicle suspension, drive, and braking systems must be accounted for. A considerable amount of weight transfer among axles occurs during acceleration and stopping of a vehicle, and the weight distribution at the time of weighing depends on the frictional forces in the suspension system at that time. In practice, efforts must be made to minimize the effects of weight transfer during successive weighings in order to make measurements within acceptable tolerances.

IN-MOTION WEIGHING

By definition, and by common usage, the term weight means that only gravitational force is acting on an object at rest. In-motion weighing of a highway vehicle attempts to approximate the weight of the vehicle, a wheel, an axle, or a group of axles on the vehicle by measuring instantaneously, or during a short period of time, the vertical component of dynamic (continually changing) force that is applied to a smooth, level road surface by the tires of the moving vehicle. The weight of the vehicle does not change when it moves over the road, but the dynamic force applied to the roadway surface by a rolling tire of the vehicle varies from more than double its static weight when it runs up on a bump, thereby exerting a large unbalanced force on the wheel mass, to zero when the tire bounces off the road. Figure 2 illustrates the pattern and magnitude of variability in dynamic wheel force for the left rear wheel (dual tires) of an empty dump truck driven at 30 mph over the relatively smooth road profile shown in the figure. A sheet of 3/8-inch thick plywood was placed on the first pair of the nine wheel force transducers that were arrayed in the road surface as shown in Fig 3 for experimental measurements. Measured wheel forces for three successive runs of the truck are plotted in Fig 2 along with output from a vehicle simulation model called DYMOL. Fig 4 is a similar graph for the loaded vehicle. Several important concepts of dynamic vehicular behavior are illustrated by these figures. First, the pattern of wheel force for a given vehicle traveling over the same

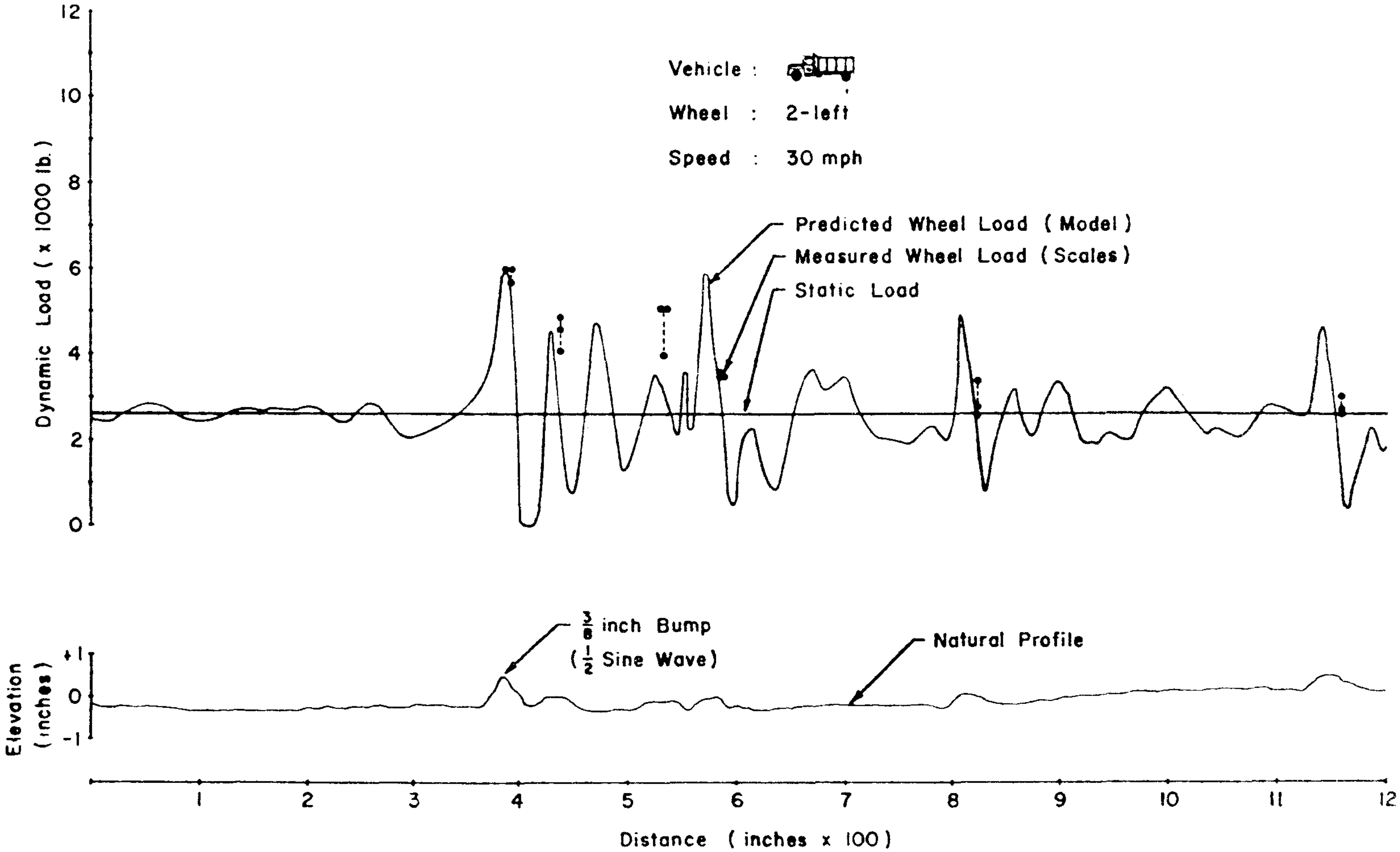

Vehicle :
Wheel : 2-left
Speed : 30 mph
Predicted Wheel Load (Model)
Measured Wheel Load (Scales)
Static Load
Dynamic Load (x 1000 lb.)
3/8 inch Bump
(1/2 Sine Wave)
Natural Profile
Elevation (inches)
Distance (inches x 100)

Figure 2

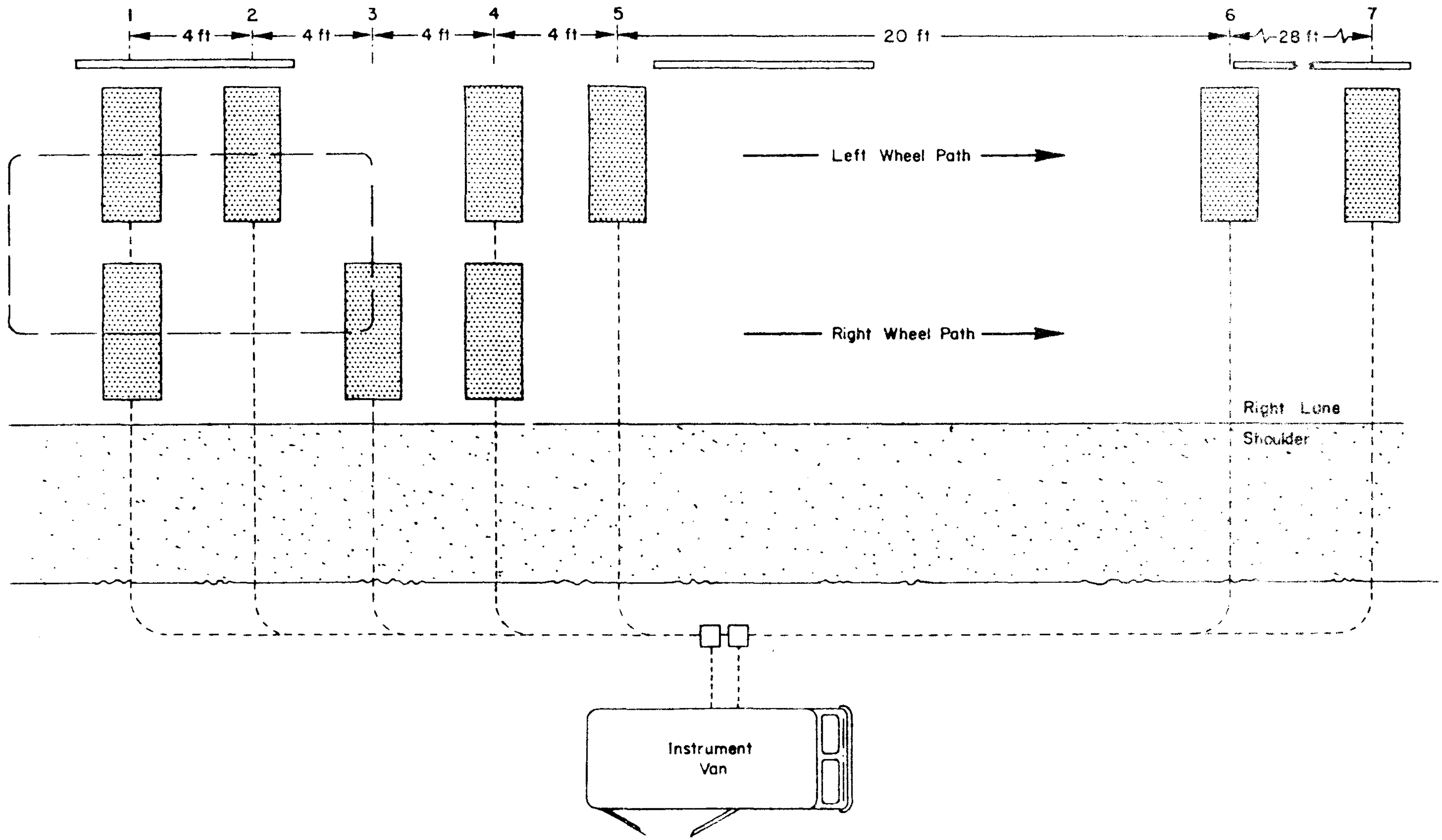
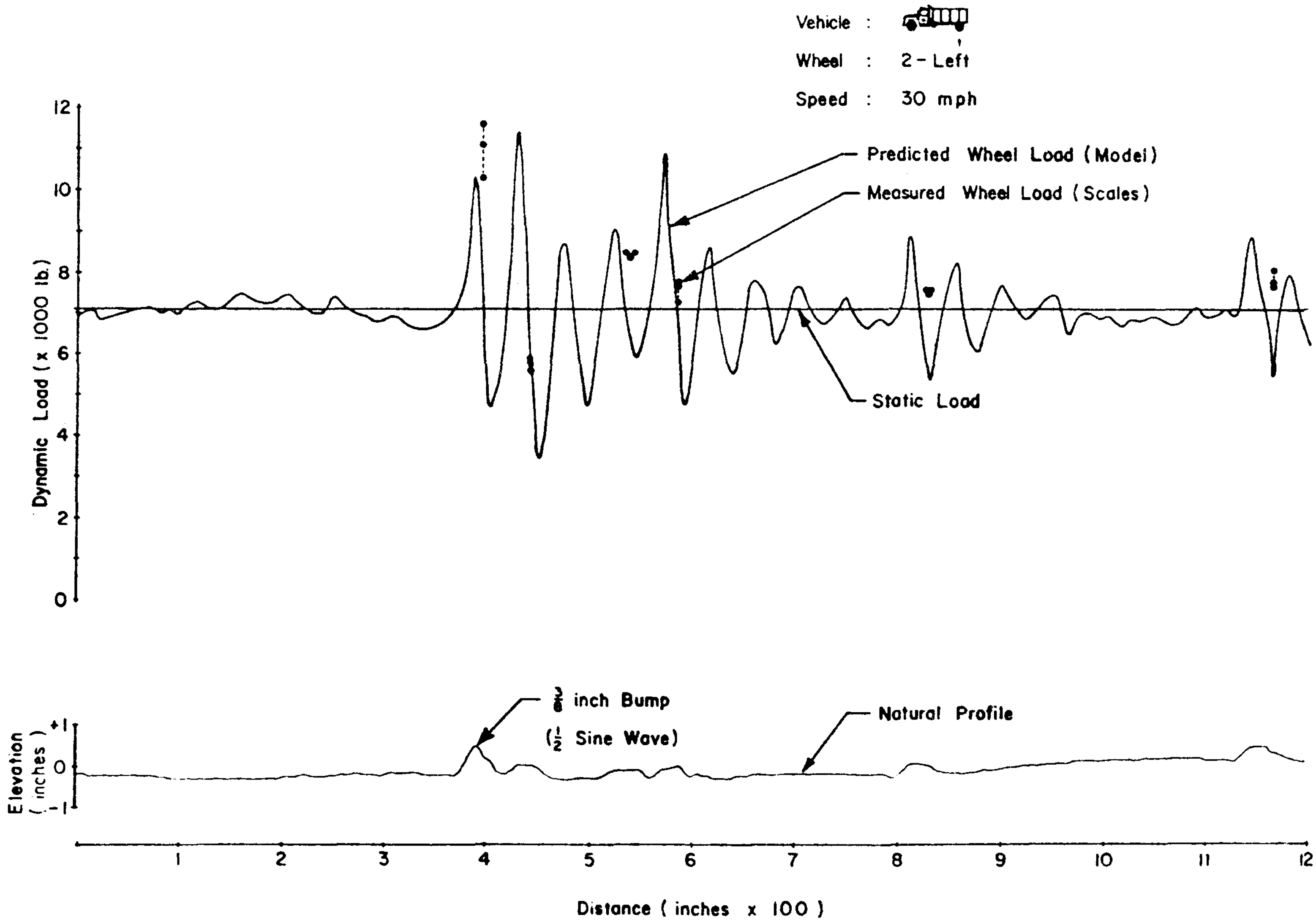

1
2
3
4
5
6
7
4 ft
4 ft
4 ft
4 ft
20 ft
28 ft
Left Wheel Path
Right Wheel Path
Right Lane
Shoulder
Instrument Van

Figure 3



Vehicle :
Wheel : 2 - Left
Speed : 30 mph
Predicted Wheel Load (Model)
Measured Wheel Load (Scales)
Static Load
Dynamic Load (x 1000 lb.)
3/8 inch Bump
(1/2 Sine Wave)
Natural Profile
Elevation (inches)
Distance (inches x 100)

Figure 4

roadway surface at the same speed is consistent as seen from the small scatter in the experimental measurements. Next, the mass of the vehicle components affects the magnitude and frequency of dynamic wheel forces and their variation from static weight as illustrated for the loaded and unloaded vehicle. Different vehicles will react differently to road roughness. The wheels (unsprung masses) oscillate typically in the range of about 8 to 12 Hz when displaced suddenly, and oscillations damp rather quickly. Finally, the dynamic wheel force is sometimes less than static weight, and sometimes greater. A characteristic behavior of trucks that is not illustrated by these figures, but which is known from actual observation and from computer simulation, is that the sprung mass (body and payload) typically oscillates at about 0.5 to 3 or 4 Hz depending on many factors which include mass. An out-of-round or out-of-balance tire or wheel can also apply vertical forces to the rotating mass and cause large variations in dynamic wheel force.

Accurate in-motion vehicle weighing is possible only when the vertical acceleration of all vehicle components is zero. The sum of the vertical forces exerted on a smooth, level surface by the perfectly round and dynamically balanced, rolling wheels of a vehicle at constant speed in a vacuum are equal to the weight of the vehicle. None of the vehicle components will be accelerating vertically under these ideal conditions. But, such conditions never exist in practice. Some of the factors which affect the tire forces of a moving vehicle are shown in the table below.

FACTORS THAT AFFECT WHEEL LOADS OF A MOVING VEHICLE		
Roadway Factors	Vehicle Factors	Environment Factors
• Longitudinal Profile • Transverse Profile • Grade • Cross Slope • Curvature	• Speed, Acceleration • Axle Configuration • Body Type • Suspension System • Tires • Load, Load Shift • Aerodynamic Characteristics • Center of Gravity	• Wind • Temperature • Ice

No road surface is perfectly smooth and level, no vehicle is perfect, and the existence of the atmosphere cannot be ignored. The nearer actual conditions approach ideal conditions, the better the approximation of vehicle weight that can be made by measuring the vertical forces applied to the roadway surface by the tires of a moving vehicle.

In practice, the adverse effects of the roadway factors can be made quite small by careful site selection and proper installation and maintenance of in-motion weighing equipment. Undesirable environmental effects can be recognized or perhaps avoided by scheduling weighing operations. The vehicle factors, except for possibly speed and acceleration, are largely uncontrollable at a weighing location. Legal and safety regulations restrict the range within which certain other vehicle factors occur, and economic considerations influence the vehicle operating conditions that drivers and owners are willing to tolerate. Perhaps the most significant uncontrolled vehicle factor that affects in-motion weighing is tire condition. Unbalanced or out-of-round tires rotating at high speed can cause large variations in the vertical component of force acting on the wheel mass and can therefore produce vertical acceleration of this mass. Tire inflation pressure also contributes significantly to the dynamic behavior of the tire and wheel mass. Even though the tire-condition variable cannot be controlled in in-motion weighing, observation and experience indicate that the tires on most over-the-road vehicles are maintained in reasonably good condition; therefore, the results of this potentially adverse effect might also fall within tolerable limits for most vehicles and for certain types of in-motion weighing operations. Several years of experience have demonstrated that in-motion weighing is practicable. Properly designed and maintained equipment is a basic requirement. Appropriate use of the equipment and interpretation of the measurements is equally important if satisfactory results are to be achieved with the techniques.

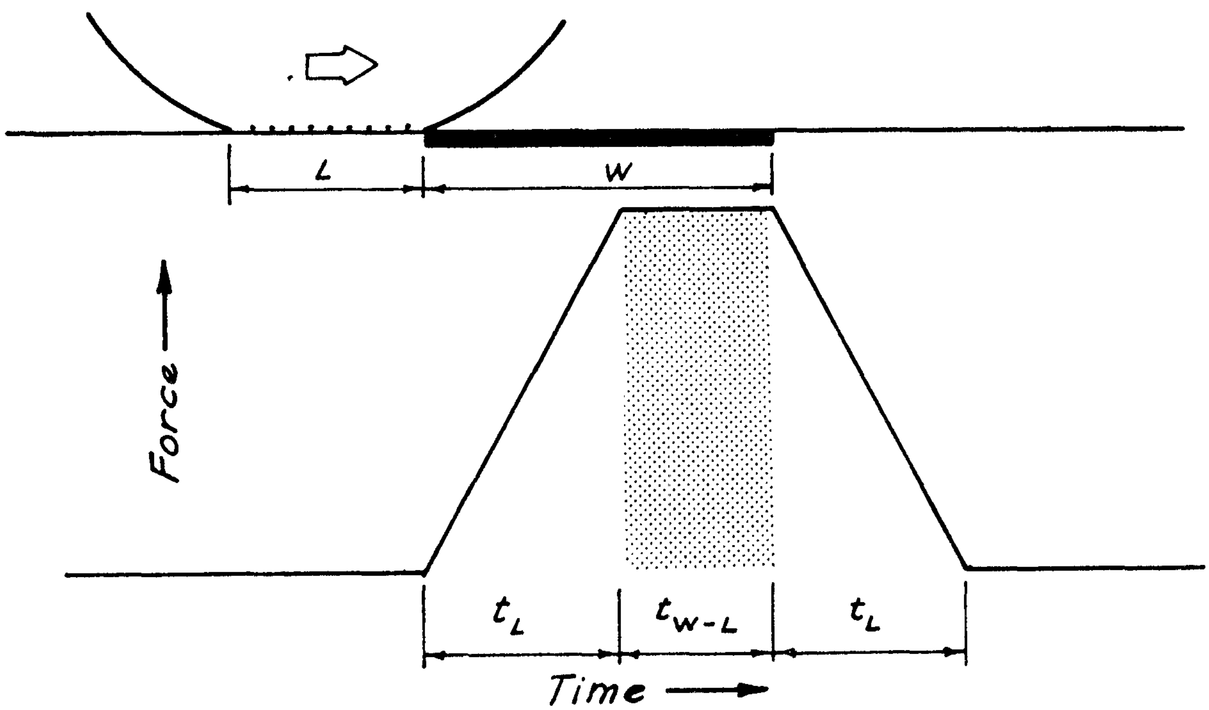
WIM SYSTEMS

A basic in-motion vehicle weighing system consists of one or more wheel force transducers plus the associated signal processing instruments. Supplementary vehicle presence sensors (e.g., inductance loop detectors) or axle passage detectors may also accompany the weighing system to measure speed, axle spacing, overall vehicle length, and lateral placement as the vehicle passes over the system.

Wheel Force Transducer

The key component of any WIM system is the wheel force transducer, which converts the vertical component of force applied to its surface through the tires of a moving vehicle into a proportional signal that can be measured and recorded. In order to measure the total vertical force imposed on the transducer by a selected tire, or by a group of tires, on a vehicle, the full tire contact area/s of interest must be supported completely and simultaneously by the transducer. The transducer must then produce a signal which is exactly proportional to the vertical force applied. This signal must not be affected by (1) tire contact area, stiffness, inflation pressure, nor position on the sensing surface of the transducer, (2) tractive forces, (3) temperature, nor (4) moisture.

An ideal force vs. time signal from a wheel force transducer is shown in the sketch below.



As the tire contact length, L , moves onto the transducer, force increases until the full tire contact area is supported by the transducer. Force does not change (assuming no vertical movement of the vehicle components) during the time t_{W-L} while the tire contact patch continues to be supported only by the transducer. This is the time when wheel force measurements are possible. Typically, t_{W-L} is about 0.006 seconds for a loaded truck traveling at 60 mph over a transducer 1.5 feet long with a tire contact length of 1.0 foot.

The surface of the wheel force transducer must be exactly even with the surface of the level roadway into which it is installed in order not to create an unbalanced force on the wheel/tire mass as the tire passes over the transducer. This unbalanced force will act upward to displace the mass if the transducer stands above the road surface, or it will cause the spring force of the vehicle suspension and the pneumatic tire to act downward on the mass as an unbalanced force if the transducer is below the surface. The inertia of the wheel/tire mass will affect the wheel force and thus the force measurements made by the transducer under either of these conditions. It is not possible to calibrate the signal from the transducer to compensate exactly for differences in elevation of the force-sensing transducer surface with respect to the surrounding road surface as each vehicle will respond differently to the surface irregularity. Such factors as speed, tire stiffness and inflation pressure, and mass of the various unsprung vehicle components are particularly affected, even by small surface irregularities.

Ideally, the transducer should deflect under load the same amount as the road surface. If the transducer is very stiff as compared with the pavement, the net effect upon force measurements will be like that of the wheel running up on a bump. Similarly, if the transducer deflects more than the road surface under load, the wheel will be affected as if it runs into a shallow hole. The transducer should deflect a small amount under load in order to behave like the surrounding road surface.

The mass of the transducer should be small in relation to the dynamic forces that are to be measured. In principle, a force transducer usually measures the displacement in an elastic body that is subjected to an applied force. This displacement is a function of the magnitude and duration of the force as well as the mass of the displaced body. To illustrate, think of your hand as an elastic spring supporting a mass and your nerves as a displacement measuring system. Place your palm upwards on the table and set a 10-pound steel block in your hand. Close your eyes. Have a friend strike the weight a sharp blow with a hammer. Gage and remember your sensation to the force applied by the hammer. Open your eyes and replace the steel block with a penny and close your eyes again. Have your friend strike the penny a similar sharp blow with the hammer. Was the applied force the same? Yes, the applied force <u>was</u> the same, but the <u>displacement</u> in your hand was much more in the second experiment leading you to an erroneous conclusion (if you really let your "friend" hit the penny). The mass of a wheel force transducer must be relatively small if dynamic forces of a few thousand pounds applied for a few milliseconds are to be measured accurately by sensing the displacement of the elastic element of the transducer. The inertia of the transducer mass affects its displacement with respect to time under an applied unbalanced force.

Closely associated with the mass of the transducer is its resonant, or natural, frequency of oscillation. The elastic transducer mass that is displaced downwards by an applied force will rebound when the force is removed and move upwards under the spring force of the elastic body until a restraining force in the opposite direction (gravity) reverses the movement. This pattern of unbalanced forces acting on the transducer mass will cause it to oscillate until some form of damping dissipates the energy stored in the elastic system. The period of oscillation is a function of mass. Generally, the greater the mass, the slower the period of oscillation and the more the energy required for damping.

A wheel force transducer measures the relative displacement of an elastic mass in response to the applied forces. If the transducer mass is being displaced from its reference position by an unbalanced force at the time a wheel force is applied, the net displacement under the wheel will result from the algebraic sum of the unbalanced force associated with the initial displacement plus the unbalanced force from the applied wheel force. If the transducer mass happens to be moving downward due to a previously applied unbalanced force when the tire applies an additional downward force, the mass will move further downward due to the sum of the two unbalanced forces both acting downwards. If, on the other hand, the transducer mass is moving upwards due to a previously applied unbalanced force, the final displacement of the transducer mass with respect to its rest position will result from a force equal to the difference in the upward inertial force and the downward wheel force.

An effective wheel force transducer must be at rest when the wheel force to be measured is applied. A low mass transducer tends to oscillate at high frequency and damp to a rest position relatively quickly; therefore, a low mass, critically damped transducer is generally preferred. Since the time between tandem axles (approximately 4 feet apart) on a vehicle moving at 60 mph is about 4/88 = 0.045 seconds, the transducer should cease oscillation within this short time in order to be ready to measure the wheel forces of such closely spaced axles. To assure that the transducer mass is at rest when an unknown tire force is to be measured, an oscilloscope should be used to examine the signal with respect to time. The force vs. time trace shown in the previous sketch should be closely approximated, particularly in the time just before the tire goes onto the transducer. The transducer should indicate no force except that of gravity when it is not loaded externally.

A wheel force transducer must be designed and constructed with adequate capacity to handle the wheel loads that will occur in practice. Legal axle load limits and possible overloads must be considered. Also, the fact that dynamic wheel force can sometimes be double the static wheel weight should be allowed for. The general relationship between fatigue life of the transducer elements and the expected number of repetitions of various stress levels should also be recognized. Wheel force transducers operate in an extremely hostile environment of impact loading, vibration, climatic extremes, and sometimes intentional abuse. Wear and tear are expected; therefore, good design must be complemented by proper inspection and maintenance if satisfactory service is to be realized from wheel force transducers.

A partial check list of wheel force transducer features is shown in the table below. This might be useful for assessing the adequacy of the transducer design and the potential performance of this important part of a WIM system.

WHEEL FORCE TRANSDUCER FEATURES	
Feature	O.K.
• Insensitive to: Tire contact area (single/dual)	
Tire stiffness	
Tire inflation pressure	
Tire position (edge-to-edge)	
Temperature	
Moisture	
• Installed even with roadway surface	
• Signal directly proportional to applied vertical force	
• Small deflection under load	
• Low mass / High compliance	
• High natural frequency / Critical damping	
• Capacity	
• Durability / Maintainability	

WIM Signal Processing Instruments

Analog signals from the wheel force transducers must be interpreted and recorded by appropriate electronic instruments to yield samples of dynamic wheel forces which serve as estimates of wheel, axle, and vehicle weight. Analog-to-digital conversion of signals is now routine; therefore, most WIM systems are based around digital data processors. The wheel force signal sketched in the previous section is digitized at a typical rate of about 1,000 Hz. The resulting digital array is evaluated rapidly and effectively to isolate the pertinent information and display a measured wheel force in appropriate units. This information is stored for further use in computing estimated axle weights and gross vehicle weights. All data, or only selected items, can be recorded for subsequent recovery and further processing. Proper software must be provided to utilize the hardware capabilities of any WIM system. There are few limitations today on the availability of quality WIM instrumentation systems. Almost any reasonable signal processing specification can be met by qualified and experienced vendors of such services.

ACCURACY OF WIM SYSTEMS

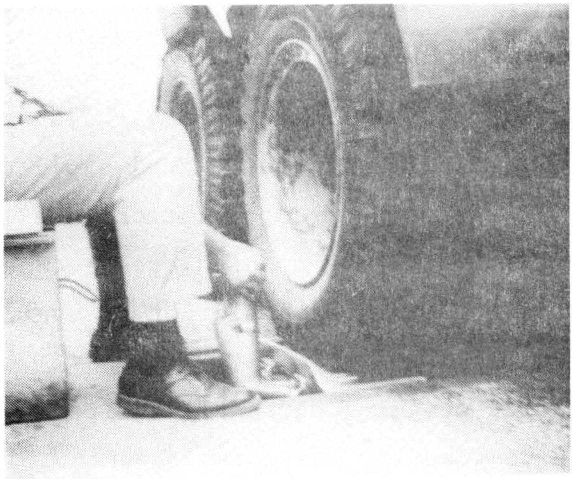
Highway vehicles are normally weighed for one or more of the following purposes: (1) commerce (buying and selling by weight); (2) statistical data (needed for planning, financing, designing, constructing, operating, and maintaining the road system); or (3) enforcement (assuring that design loading is not abused). The need for accurate (correct; without error; deviating only slightly or within acceptable limits from a standard) weighing varies somewhat for each of these purposes because the consequences of using inaccurate weight information involve different degrees of risk to the users. Tolerances, or permitted variations from a correct value, can be set to reflect the relative importance of accuracy in view of both the use of the information and the cost and feasibility of obtaining it. Setting of such tolerances involves the specification of the magnitude of allowable variations as well as the probability that any given measurement will lie within the stated limits. Considerable judgment must be exercised in developing these specifications, and the need for nationwide uniformity must be recognized.

From the previous discussion of in-motion weighing, it should be apparent that the dynamic interaction of an imperfect vehicle with an imperfect road surface in the earth's atmosphere makes highly accurate estimates of vehicle weight impossible by this technique. But the practical question remains, can samples of dynamic wheel force be used to estimate vehicle, axle, and wheel weights within acceptable tolerances for specific purposes? The demonstrated answer to this question is yes. The state-of-the-art in in-motion weighing now permits efficient, safe, economical measurements of vehicle weights and dimensions to be made for statistical data purposes. Properly designed, installed, and maintained WIM equipment is capable of making unbiased measurements of dynamic wheel forces that represent adequately the loading patterns to which our roads and bridges are being subjected. The fact that some of the sampled forces are greater than the true static weight and some areless is important; this is what the road surface actually experiences. As long as our structural design procedures and materials testing procedures are based on static loading, an estimate of the static loading pattern is a useful statistic. When these procedures can utilize a more sophisticated description of dynamic loading, the WIM technique can be adapted for providing such information. For now, however, a WIM system which can measure and record the applied wheel force within about 1 percent tolerance can be installed and used with confidence to collect large samples of data for statistical applications.

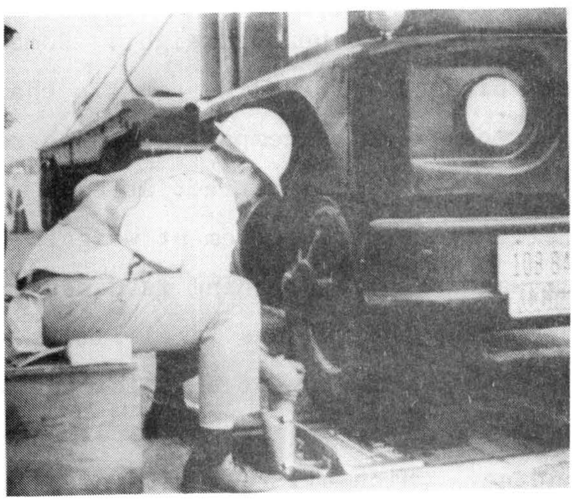
Tolerance needed for commercial vehicle weighing applications are long established and well recognized. In-motion weighing at high speeds cannot now satisfy these small tolerances, but this application should not be overlooked in future WIM development.

Enforcement applications of in-motion weighing currently utilize the technique mostly as a screening device to identify suspected weight violators for subsequent checking on static scales certified to the required tolerances. Weight threshold limits on the WIM system can be adjusted to allow for expected differences in dynamic force measurements and static weight and thereby select only those vehicles that are quite likely to be overweight. Some compensation can be made in the WIM system thresholds for site-specific characteristics such as local surface roughness or grade by comparing WIM measurements with actual static weights, but this will not be perfect as each vehicle will behave differently. The overall efficiency of enforcement weighing is considerably enhanced by the WIM technique as static weighing is necessary only for the vehicles which approach or exceed the various legal limits.

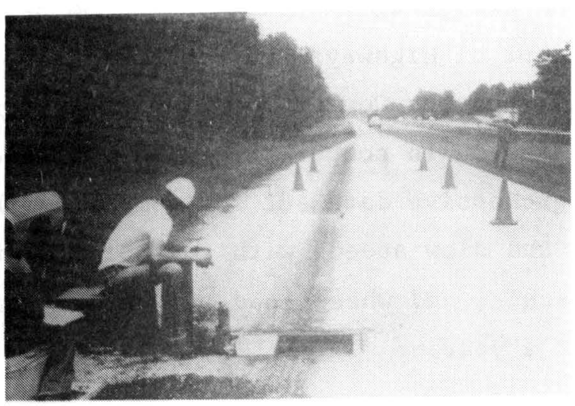
It is recognized that axle-by-axle static weighing of a vehicle on an axle-load scale that is certified to small tolerances (e.g., 0.2 percent) does not necessarily yield vehicle or axle weights which all fall within these tolerances. The probability is high that in-motion weighing of successive axles at slow speeds can give very good estimates of such weights; perhaps as good as static axle-by-axle weighing. A series of experiments is now being conducted in Texas by the State Department of Highways and Public Transportation, the Department of Public Safety, and the Center for Transportation Research at The University of Texas at Austin in cooperation with the Federal Highway Administration to obtain a comprehensive data set for comparing the accuracy of WIM at high, intermediate, and slow speeds with that of a static axle-load scale, a semi-portable axle scale, and wheel-load weighers. This experiment should be completed in about a year.



Slide 9.1. Conventional weighing of truck.

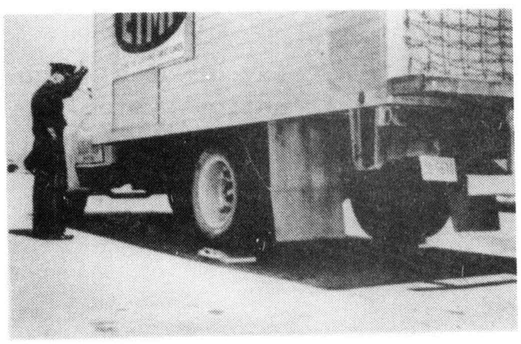


Slide 9.2. Conventional weighing of truck (continued).

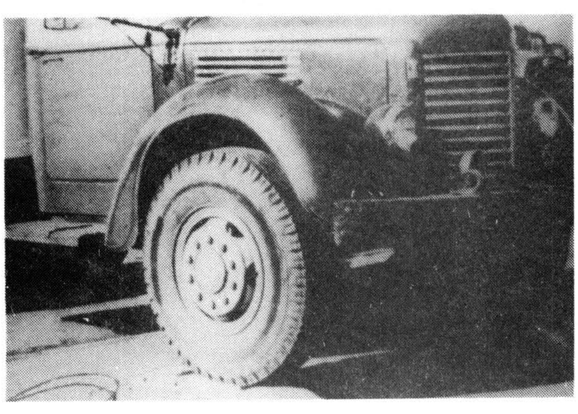


Slide 9.3. Arrangement of traffic control.

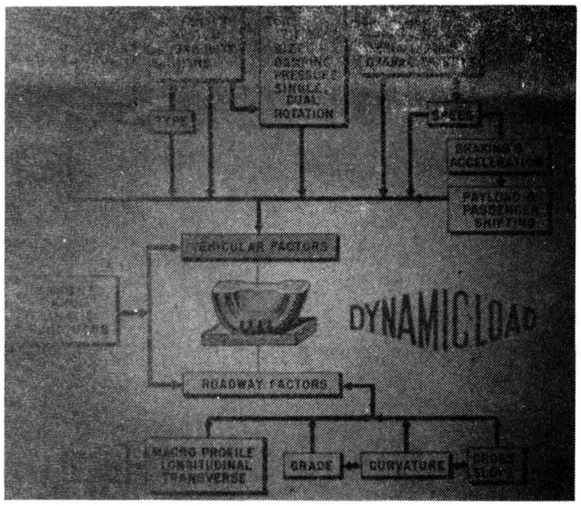
Slide 9.4. Utilization of a large crew in conventional weighing.



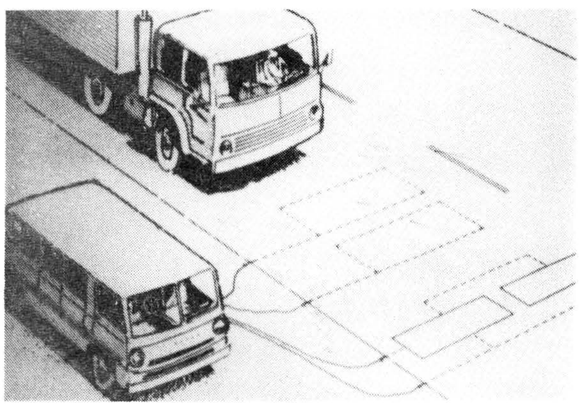
Slide 9.5. Use of two scales and an officer for conventional weighing.



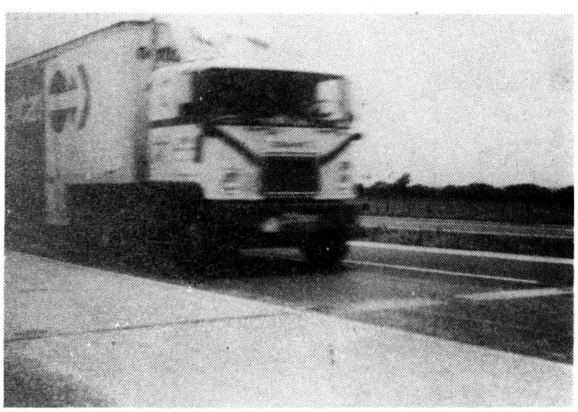
Slide 9.6. Conventional weighing (continued).



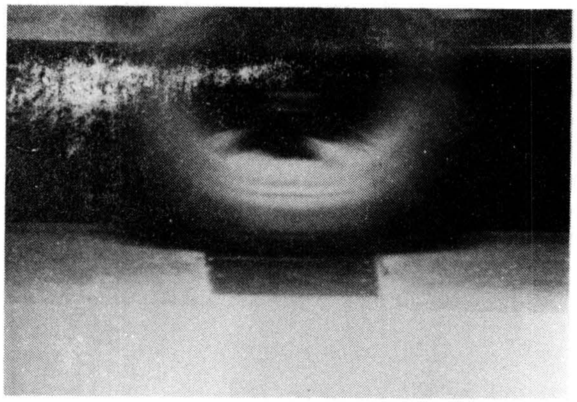
Slide 9.7. Factors considered in a vehicle weighing system.



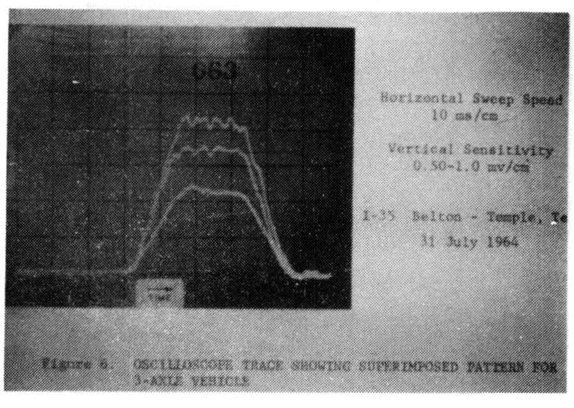
Slide 9.8. Layout of a weigh-in-motion system.



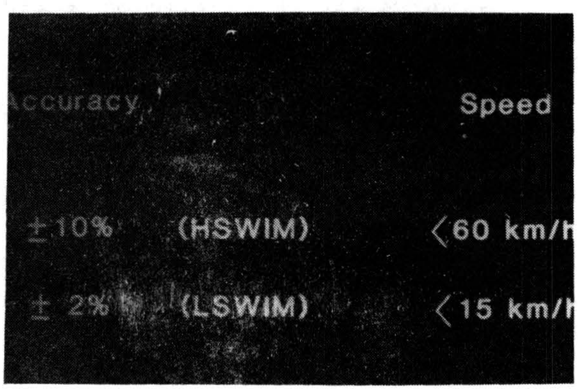
Slide 9.9. Truck approaching at a normal speed.



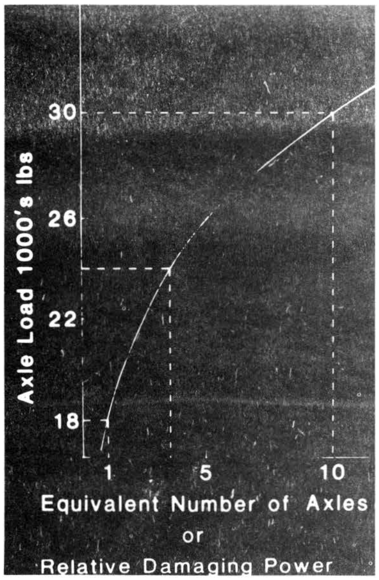
Slide 9.10. Truck passing at normal speed.



Slide 9.11. Oscilloscope trace of a 3-axle vehicle.

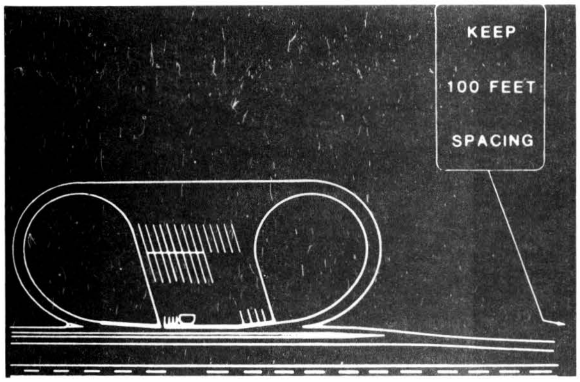


Slide 9.12. Accuracy in high speed and low speed weigh-in-motion.

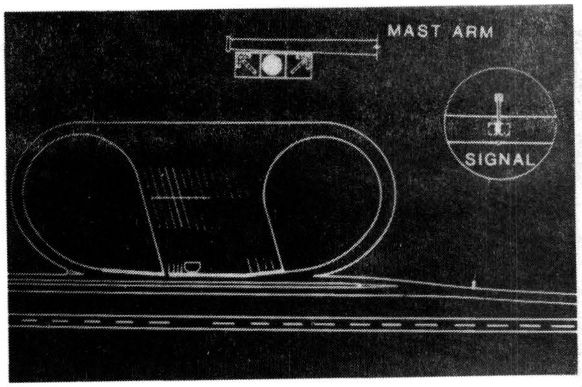


Slide 9.13. Axle load versus equivalent number of axles.

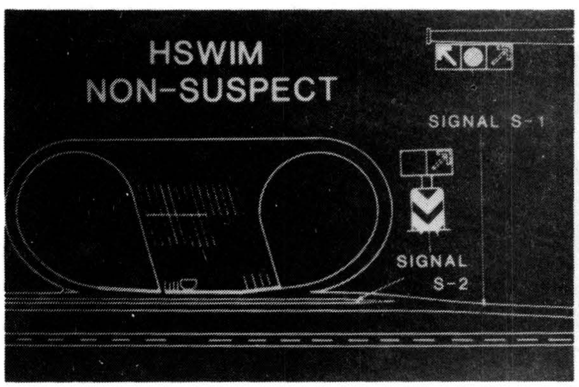
Slide 9.14. Installation of WIM system.



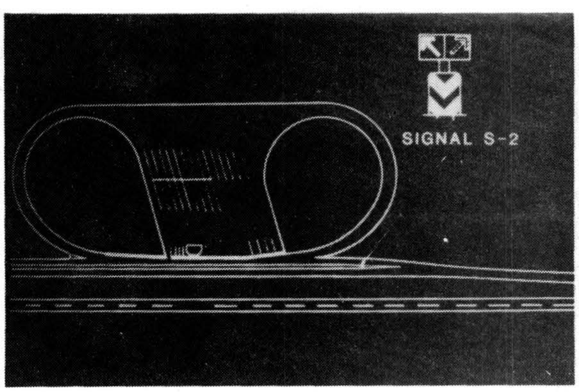
Slide 9.15. Schematic of a site where WIM system is installed.



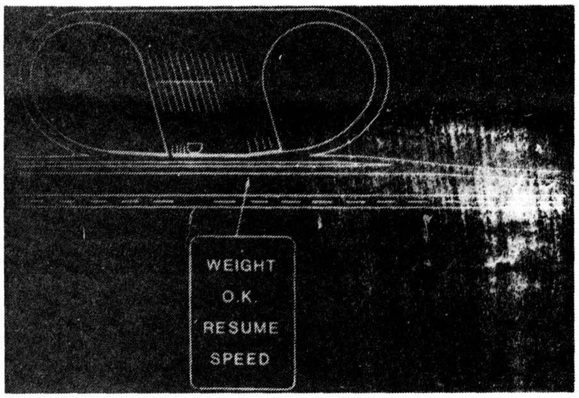
Slide 9.16. Schematic of WIM site (continued).



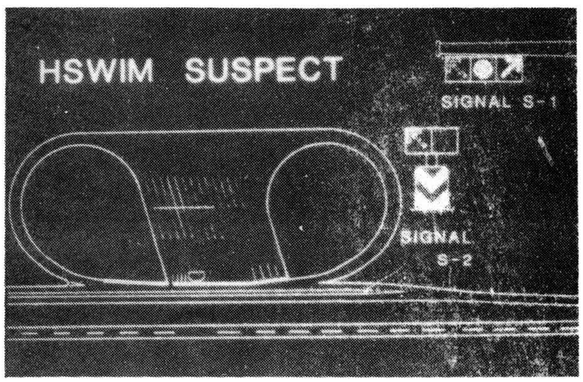
Slide 9.17. HSWIM non-suspect System.



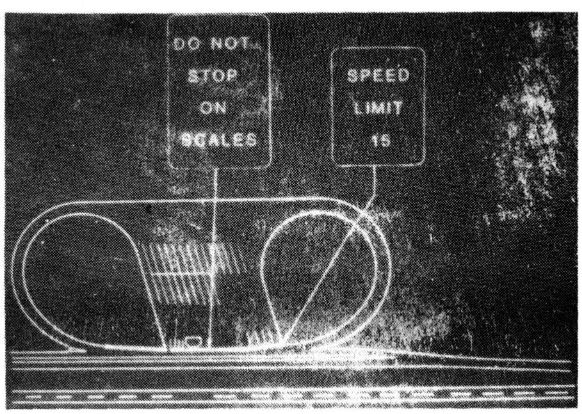
Slide 9.18. Illustration of signal S-2 (to resume speed).



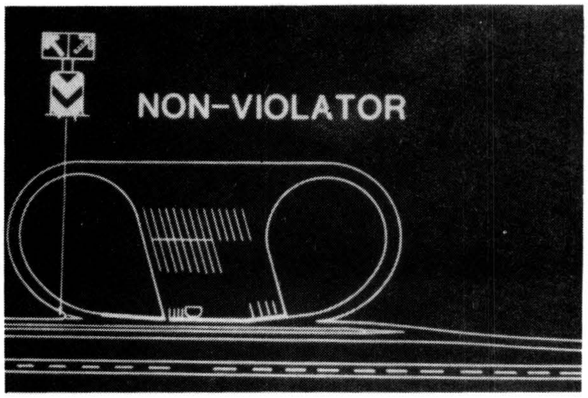
Slide 9.19. Sign to resume speed.



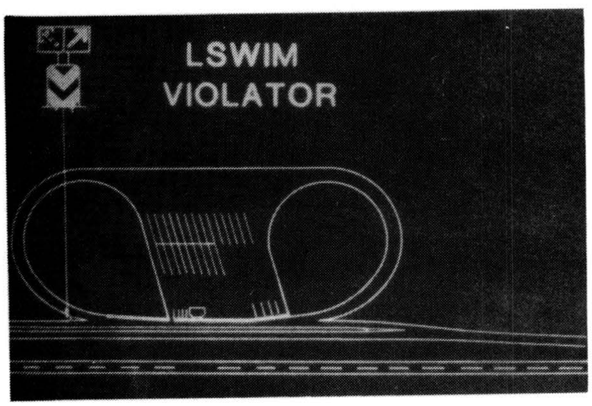
Slide 9.20. Signals for WSWIM suspect.



Slide 9.21. Sign to reduce speed for HSWIM suspect.



Slide 9.22. Sign for non-vilator.



Slide 9.23. Signal for LSWIM violator.

LESSON OUTLINE
AASHTO LOAD EQUIVALENCIES

Instructional Objectives

1. To provide the students with a basic method recommended by AASHTO for converting mixed traffic loads to equivalent 18-kip single-axle loads.

Performance Objectives

1. The student should be able to identify the data needed to convert mixed traffic into equivalent 18-kip single-axle loads.

2. The student should be able to perform the conversion using the AASHTO method.

Abbreviated Summary	Time Allocations, min.
1. Introduction	10
2. Method for Flexible Pavements	20
3. Method for Rigid Pavements	20
	50 minutes

Reading Assignment

1. Yoder & Witczak - Chapter 4, pages 162-172

2. AASHTO Interim Guide - pages 62-69 and 107-110

LESSON OUTLINE
AASHTO LOAD EQUIVALENCIES

1.0 INTRODUCTION

1.1 Procedures for Conversion

1.1.1 WASHO Road Test and Maryland Road Test.(Visual Aid 10.1) Both the WASHO and the Maryland road tests used similar vehicles. These consisted of single unit trucks with dual and tandem wheel configurations. The WASHO Road Test involved a wider load range than the Maryland test.

1.1.2 AASHO Road Test. (Visual Aid 10.2) The AASHO Road Test included three truck types. These included 2 axle, 3 axle (code 2-51) and 5 axle trucks (code 3-52) configurations. The 2-51 and 3-52 are tractor-trailer combinations. All trucks were fully loaded during the tests.

1.1.3 Asphalt Institute. For pavement design purpose, the cumulative effects of the number of vehicles and the weight on each wheel are reduced to a common denominator of equivalent 18,000 lbs single axle loads (Visual Aid 10.3). Visual Aid 10.4 is based on the analysis of extensive loadometer studies.

1.2 Flexible versus Rigid Pavement

1.2.1 Flexible based on:

(a) terminal serviceability,
(b) structural number, and
(c) number of axles.

1.2.2 Rigid based on:

(a) terminal serviceability,
(b) slab thickness, and
(c) number of axles.

2.0 METHOD FOR FLEXIBLE PAVEMENTS

2.1 Derivation of Equivalence Load Factors (Visual Aids 10.5, 10.6, 10.7 and 10.8)

$$\log W_t = 5.93 + 9.36 \log (SN + 1) - 4.79 \log (L_1 + L_2) + 4.331 \log L_2 + G_t/\beta$$

where

W_t = axle load applications of end of time t

SN = structural number

L_1 = load on one single or one tandem axle set, kips

L_2 = axle code (1 for single and 2 for tandem axle)

G_t = a function (the logarithm) of the ratio of loss in serviceability at time "t" to the potential loss taken to a point where p_t = 1.5.

β = a function of design and load variables that influence the shape of the p-versus-W serviceability curve.

Rearrange terms and substitute appropriate values for L_1 and L_2: (Visual Aid 10.9)

$$e_i = \left[\frac{(L_i + n)^{4.79}}{(18 + 1)^{4.79}}\right]\left[\frac{10^{G_i/\beta_{18}}}{\left(10^{G_i/\beta_i}\right)\left(n^{4.33}\right)}\right]$$

e_i = traffic equivalence factor for load group i

L_i = axle load, kips

n = number of axles

$$\beta_i = 0.4 + \frac{0.081\ (L_i + n)^{3.23}}{(SN + 1)^{5.19}\, n^{3.23}}$$

$$G_i = \log\left(\frac{4.2 - p_t}{4.2 - 1.5}\right)$$

(e.g., single axle, $L_i + n$ = 20 + 1;

tandem axle, $L_i + n$ = 20 + 2)

tabulated values of e_i are shown in Visual Aids 10.10 and 10.11.

2.2 Conversion of Mixed Traffic to Equivalent Traffic

$$W_{t_{18}} = N_t \sum_{i=1}^{n} P_i \, e_i$$

where

$W_{t_{18}}$ = equivalent number of 18-kip single axle loads for flexible pavement

N_t = total number of axles

P_i = percent of axles in load group i

2.3 Lane Distribution Considerations

The number of equivalent axle loads derived represents the total for all lanes and both direction of travel. Lane distribution considerations are:

(a) usually assign 50 percent of $W_{t_{18}}$ to each direction,

(b) usually assign 100 percent of traffic in each direction to the design lane, and

(c) possibly use lane distribution factors. (Visual Aid 10.12)

3.0 METHOD FOR RIGID PAVEMENT

3.1 Derivation of Equivalence Load Factors (Visual Aids 10.13, 10.14 and 10.15)

$$\log W_{t_{18}} = 5.85 + 7.35 (\log D + 1) - 4.62 \log (L_1 + L_2)$$

$$3.28 \log L_2 + G_t/\beta$$

where D = thickness of slab, inches.

Rearrange terms and substitute appropriate values for L_1 and L_2: (Visual Aid 10.9)

$$e_i = \frac{(L_i + n)^{4.62}}{(18 + 1)^{4.62}} \quad \frac{10^{G_i/\beta}}{10^{G_i/\beta} \quad n^{3.28}}$$

$$\beta_i = 1.00 + \frac{3.63\ (L_1 + L_2)^{5.20}}{(D + 1)^{8.46} L_2^{3.52}}$$

$$G_i = \log \frac{4.5 - p_t}{4\text{-}5 - 1.5}$$

Tabulated values of e_i are shown in Visual Aid 10.16.

3.2 Conversion of Mixed Traffic to Equivalent Traffic

$$W_{t_{18}} = N_t \sum_{i=1}^{n} P_i\ e_i$$

where

$W_{t_{18}}$ = equivalent number of 18-kip single axle loads for rigid pavement

N_t = total number of axles

P_i = percent of axles in load group i.

4.0 EXAMPLE - LOADOMETER STATION DATA (Visual Aid 10.17)

The example in Appendix C of the AASHTO Interim Guide will be covered in class or a lab session.

LESSON OUTLINE
AASHTO LOAD EQUIVALENCIES

VISUAL AID	TITLE
Visual Aid 10.1.	WASHO and Maryland Road Tests.
Visual Aid 10.2.	The Asphalt Institute's equivalency factors.
Visual Aid 10.3.	Nomograph method for estimating EAL_{18} for flexible pavements.
Visual Aid 10.4.	Main factorial experiment, relationship between design and axle load applicaitons at p = 1.5 (from Road Test equations).
Visual Aid 10.5.	Derivation of equivalency load factors - flexible.
Visual Aid 10.6.	Derivation of equivalency load factors - flexible.
Visual Aid 10.7.	Traffic equivalency factors.
Visual Aid 10.8.	Fatigue/Damage and load relationships.
Visual Aid 10.9.	Traffic equivalency factors.
Visual Aid 10.10.	Flexible pavement traffic equivalency factors (p_t = 2.0).
Visual Aid 10.11.	Flexible pavement traffic equivalency factors (p_t = 2.5).
Visual Aid 10.12.	Lane distribution factors on multilane roads.
Visual Aid 10.13.	Derivation of equivalency load factors - rigid.
Visual Aid 10.14.	Derivation of equivalency load factors - rigid.
Visual Aid 10.15.	Derivation of equivalency load factors - rigid.
Visual Aid 10.16.	Rigid pavement traffic equivalency factors (p_t = 2.5).
Visual Aid 10.17.	Example of determination of equivalent 18-kip (80kN) single axle loads from Loadometer station data.

Visual Aid 10.1. WASHO and Maryland Road tests.

(a) WASHO Road Test, Equivalent Axle Loads, Flexible Pavement*

Surfacing	Single-Axle Load (kip)	Equivalent Tandem-Axle Load: Based on Deflection (kip)	Equivalent Tandem-Axle Load: Based on Distress (kip)
2-inch asphaltic concrete	18.0	35.0	28.3
	22.4	44.0	36.4
4-inch asphaltic concrete	18.0	30.5	28.0
	22.4	40.9	33.6

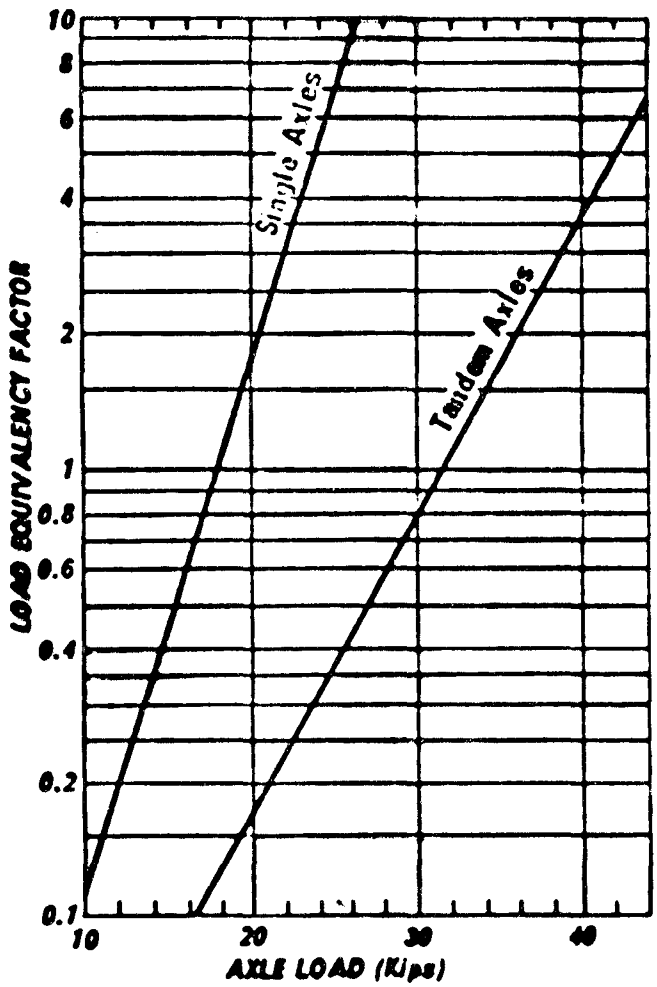
*From Highway Research Board Special Report 22.

(b) Maryland Test Road, Tandem-axle Loads Equivalent to 18,000 Pound Single-Axle Rigid Pavement*

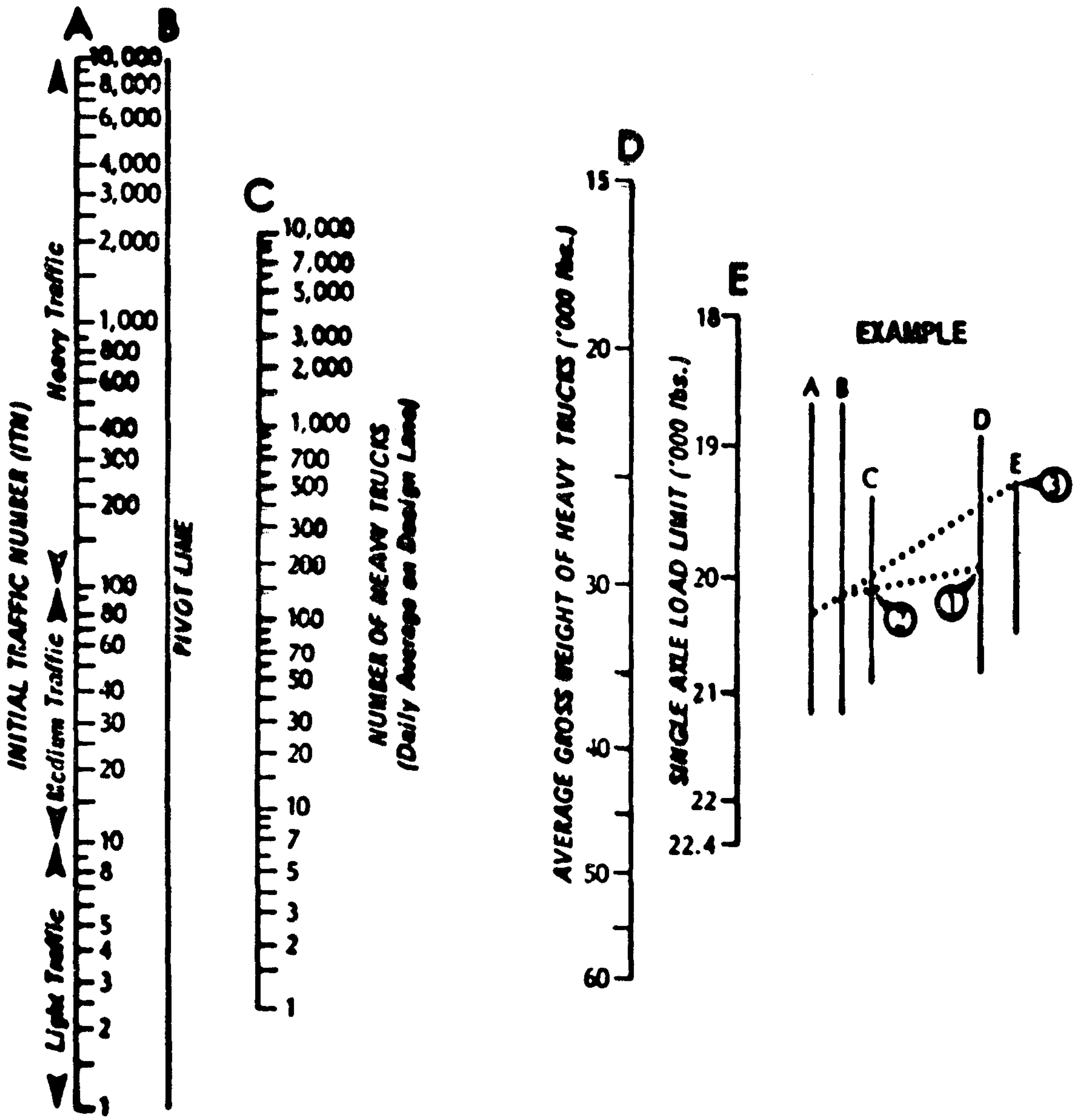
Subgrade Type	Equivalent Tandem Axle: Corner Stress (kip)	Equivalent Tandem Axle: Corner Deflection (kip)	Equivalent Tandem Axle: Free Edge Deflection (kip)
Average of four tests on gravel	31.4	29.3	24.3
Clay (pumping)	27.4	24.4	19.4
	27.7	24.1	18.9
	23.0	26.2	
	30.9	27.0	
Average Clay	29.75	25.4	19.2

*From Highway Research Board Special Report 4.

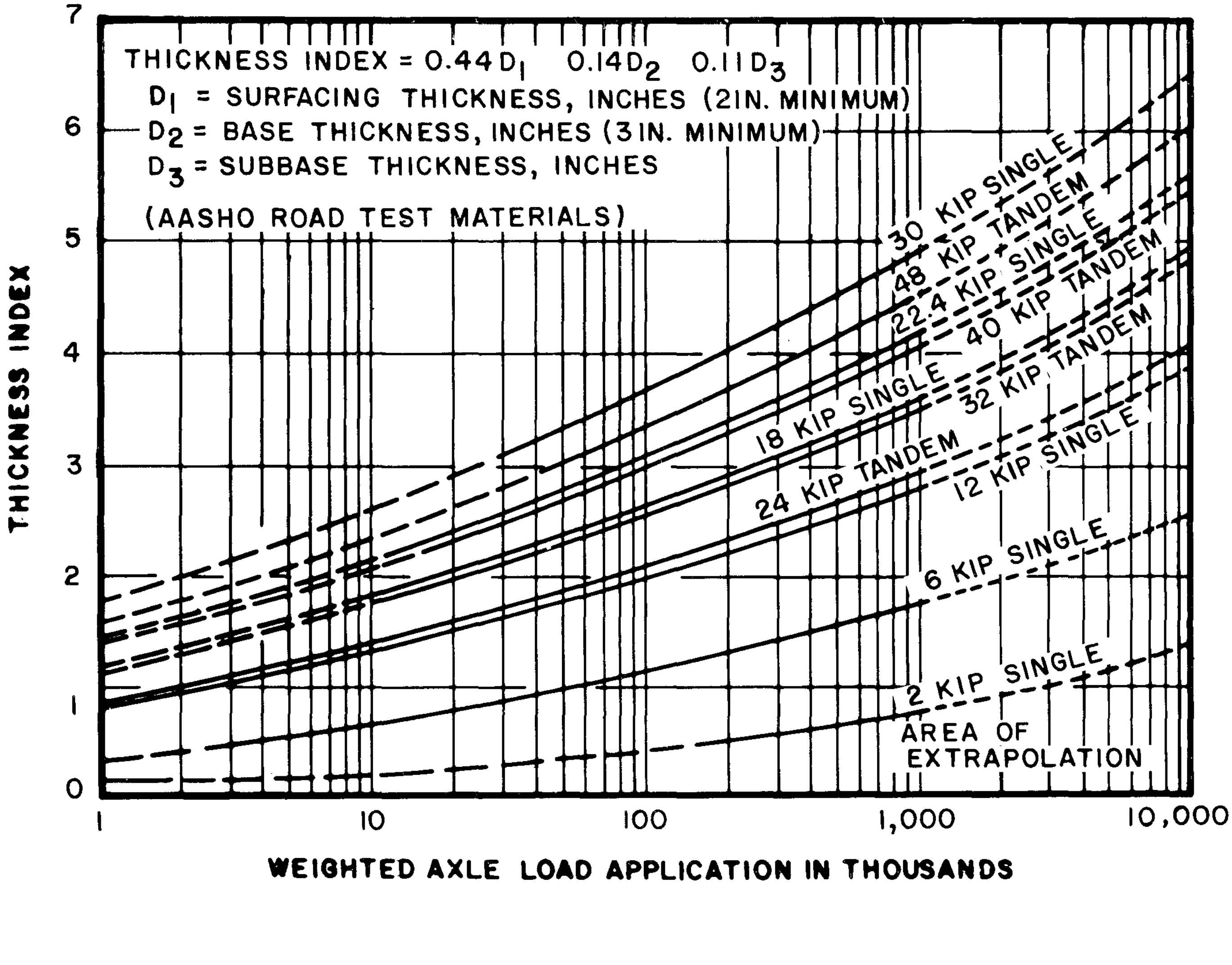
Visual Aid 10.2. The Asphalt Institute's load equivalency factors.



Visual Aid 10.3. Nomograph method for estimating EAL_{18} for flexible pavements.



Visual Aid 10.4. Main factorial experiment, relationship between design and axle load applications at p = 1.5 (from road test equations).



Visual Aid 10.5. Derivation of equivalency load factors - flexible.

$$\text{Log } W_t = 5.93 + 9.36 \text{ Log } (SN + 1) - 4.79 \text{ log } (L_1 + L_2)$$

$$+ 4.331 \text{ Log } L_2 + G_t / \beta$$

Where:

W_t = axle load applications at end of time t

SN = structural number

L_1 = load on one single or one tandem axle set, kips

L_2 = axle code (1 for single and 2 for tandem axle)

G_t = a function (the logarithm of the ratio of loss in serviceability at time t to the potential loss taken to a point where $p_t = 1.5$.

β = a function of design and load variables that influence the shape of the (p versus w) serviceability curve.

Visual Aid 10.6. Derivation of equivalency load factors - flexible.

If L_1 equals 18 kips, and L_2 equals 1 for single axles,

$$\text{Log } W_{t_{18}} = 5.93 + 9.36 \text{ Log } (SN + 1) - 4.79 \text{ Log}(18 + 1) + G_t/\beta_{18}$$

For any other axle load L_1, equal to X,

$$\text{Log } W_{t_{18}} = 5.93 + 9.36\text{Log}(SN + 1) - 4.79 \text{ Log}(L_x + L_2) + 4.33\text{Log } L_2 + G_t/\beta_x$$

Subtracting:

$$\text{Log } W_{t_x}/W_{t_{18}} = 4.79 \text{ Log}(18 + 1) - 4.79 \text{ Log}(L_x + L_2) + 4.33 \text{ Log } L_2 + G_t/\beta_x - G_t/\beta_{18}$$

Visual Aid 10.7. Traffic equivalency factors.

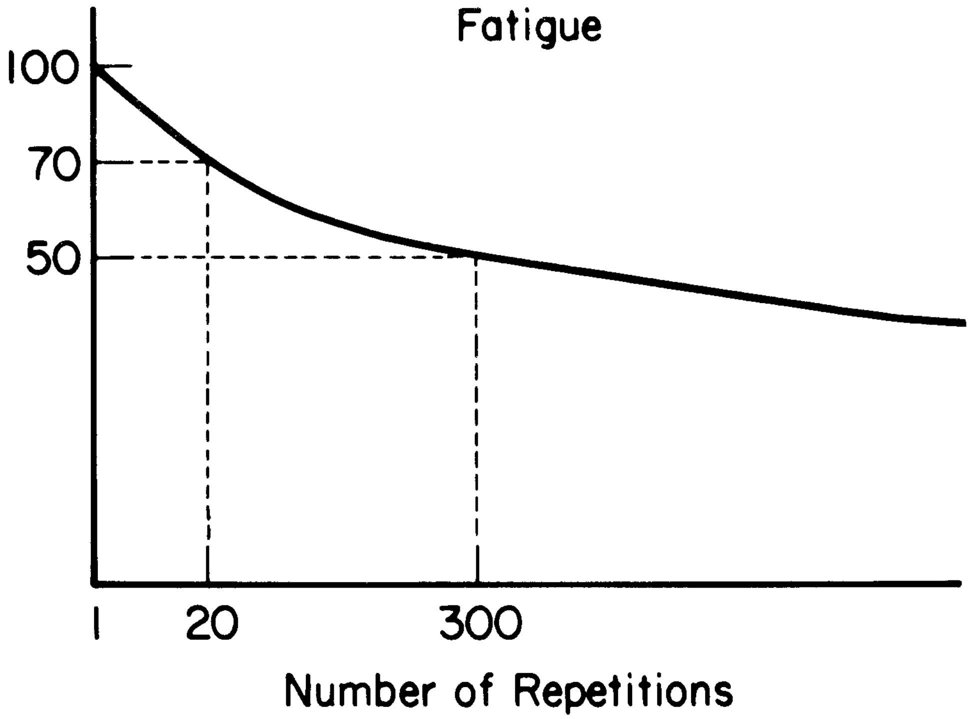
For single axles ($L_2 = 1$)

$$\text{Log } W_{t_x}/W_{t_{18}} = 4.79 \text{ Log}(18 + 1) - 4.79 \text{ Log } (L_x + 1) + G_t/\beta_x - G_t/\beta_{18}$$

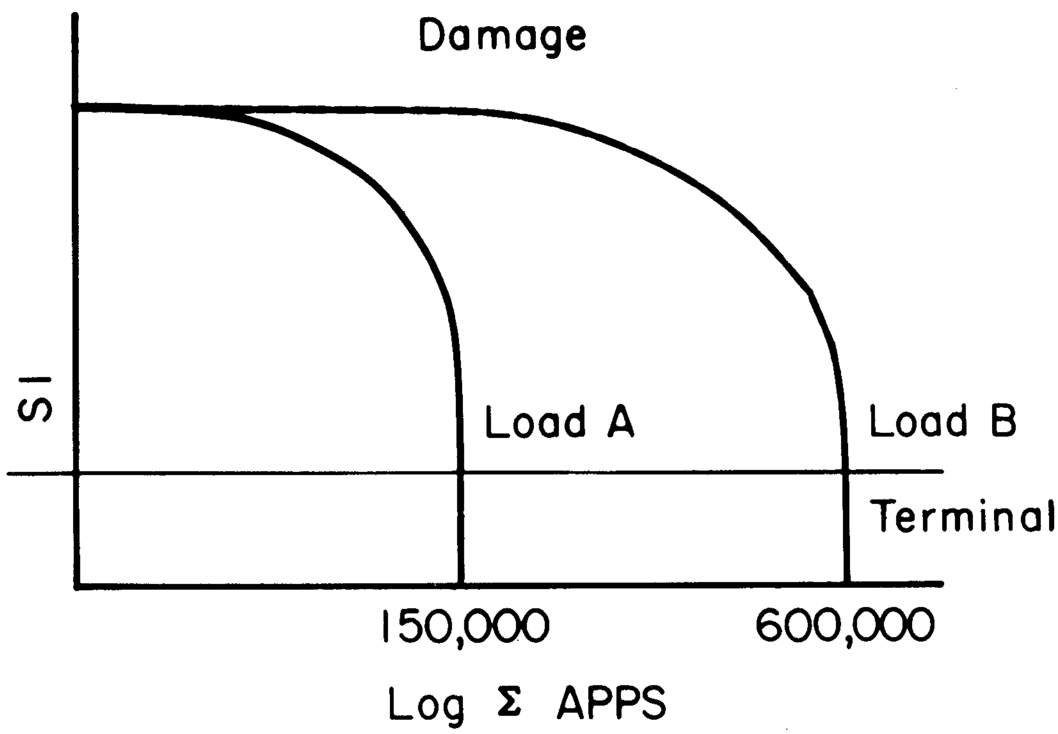
or, for tandem axles, ($L_2 = 2$), to:

$$\text{Log } W_{t_x}/W_{t_{18}} = 4.79 \text{ Log } (18 + 1) - 4.79 \text{ Log } (L_x + 2) + 4.33 \text{ Log } 2 + G_t/\beta_x - G_t/\beta_{18}$$

Visual Aid 10.8. Fatigue/Damage and load relationships.



$$300\ R_{50} = 20\ R_{70} = 1 R_{100}$$



150 A = 600 B

A = 4 B

Visual Aid 10.9. Traffic equivalency factors.

$$W_1 = N_1 \cdot e_1 = N_t \cdot P_1 \cdot e_1$$

$$W_2 = N_2 \cdot e_2 = N_t \cdot P_2 \cdot e_2$$

$$W_i = N_i \cdot e_i = N_t \cdot P_i \cdot e_i$$

$$W_n = N_n \cdot e_n = N_n \cdot P_n \cdot e_n$$

where:

W_1 = equivalent 18-kip (80kN) single-axle loads for load group i.

N_i = number of axles expected for load group i.

N_t = total number of axles.

P_i = percent of axles in load group i.

e_i = traffic equivalence factor for load group i.

Visual Aid 10.10. Flexible pavement traffic equivalency factors (p_t = 2.0).

Traffic Equivalence Factors, Flexible Pavement

Single Axles, p_t = 2.0

Axle Load		Structural Number, SN					
Kips	kN	1	2	3	4	5	6
2	8.9	0.0002	0.0002	0.0002	0.0002	0.0002	0.0002
4	17.8	0.002	0.003	0.002	0.002	0.002	0.002
6	26.7	0.01	0.01	0.01	0.01	0.01	0.01
8	35.6	0.03	0.04	0.04	0.03	0.03	0.03
10	44.5	0.08	0.08	0.09	0.08	0.08	0.08
12	53.4	0.16	0.18	0.19	0.18	0.17	0.17
14	62.3	0.32	0.34	0.35	0.35	0.34	0.33
16	71.2	0.59	0.60	0.61	0.61	0.60	0.60
18	80.1	1.00	1.00	1.00	1.00	1.00	1.00
20	89.1	1.61	1.59	1.56	1.55	1.57	1.60
22	97.9	2.49	2.44	2.35	2.31	2.35	2.41
24	106.8	3.71	3.62	3.43	3.33	3.40	3.51
26	115.7	5.36	5.21	4.88	4.68	4.77	4.96
28	124.6	7.54	7.31	6.78	6.42	6.52	6.83
30	133.4	10.38	10.03	9.24	8.65	8.73	9.17
32	142.3	14.00	13.51	12.37	11.46	11.48	12.07
34	151.2	18.55	17.87	16.30	14.97	14.87	15.63
36	160.1	24.20	23.30	21.16	19.28	19.02	19.93
38	169.0	31.14	29.95	27.12	24.55	24.03	25.10
40	177.9	39.57	38.02	34.34	30.92	30.04	31.25

Traffic Equivalence Factors, Flexible Pavement

Tandem Axles, p_t = 2.0

Axle Load		Structural Number, SN					
Kips	kN	1	2	3	4	5	6
10	44.5	0.01	0.01	0.01	0.01	0.01	0.01
12	53.4	0.01	0.02	0.02	0.01	0.01	0.01
14	62.3	0.02	0.03	0.03	0.03	0.02	0.02
16	71.2	0.04	0.05	0.05	0.05	0.04	0.04
18	80.1	0.07	0.08	0.08	0.08	0.07	0.07
20	89.0	0.10	0.12	0.12	0.12	0.11	0.10
22	97.9	0.16	0.17	0.18	0.17	0.16	0.16
24	106.8	0.23	0.24	0.26	0.25	0.24	0.23
26	115.7	0.32	0.34	0.36	0.35	0.34	0.33
28	124.6	0.45	0.46	0.49	0.48	0.47	0.46
30	133.4	0.61	0.62	0.65	0.64	0.63	0.62
32	142.3	0.81	0.82	0.84	0.84	0.83	0.82
34	151.2	1.06	1.07	1.08	1.08	1.08	1.07
36	160.1	1.38	1.38	1.38	1.38	1.38	1.38
38	169.0	1.76	1.75	1.73	1.72	1.73	1.74
40	177.9	2.22	2.19	2.15	2.13	2.16	2.18
42	186.8	2.77	2.73	2.64	2.62	2.66	2.70
44	195.7	3.42	3.36	3.23	3.18	3.24	3.31
46	204.6	4.20	4.11	3.92	3.83	3.91	4.02
48	213.5	5.10	4.98	4.72	4.58	4.68	4.83

Visual Aid 10.11. Flexible pavement traffic equivalence factors (p_t = 2.5).

Traffic Equivalence Factors, Flexible Pavement

Single Axles, p_t = 2.5

Axle Load		Structural Number, SN					
Kips	kN	1	2	3	4	5	6
2	8.9	0.0004	0.0004	0.0003	0.0002	0.0002	0.0002
4	17.8	0.003	0.004	0.004	0.003	0.003	0.002
6	26.7	0.01	0.02	0.02	0.01	0.01	0.01
8	35.6	0.03	0.05	0.05	0.04	0.03	0.03
10	44.5	0.08	0.10	0.12	0.10	0.09	0.08
12	53.4	0.17	0.20	0.23	0.21	0.19	0.18
14	62.3	0.33	0.36	0.40	0.39	0.36	0.34
16	71.2	0.59	0.61	0.65	0.65	0.62	0.61
18	80.1	1.00	1.00	1.00	1.00	1.00	1.00
20	89.0	1.61	1.57	1.49	1.47	1.51	1.55
22	97.9	2.48	2.38	2.17	2.09	2.18	2.30
24	106.8	3.69	3.49	3.09	2.89	3.03	3.27
26	115.7	5.33	4.99	4.31	3.91	4.09	4.48
28	124.6	7.49	6.98	5.90	5.21	5.39	5.98
30	133.4	10.31	9.55	7.94	6.83	6.97	7.79
32	142.3	13.90	12.82	10.52	8.85	8.88	9.95
34	151.2	18.41	16.94	13.74	11.34	11.18	12.51
36	160.1	24.02	22.04	17.73	14.38	13.93	15.50
38	169.0	30.90	28.30	22.61	18.06	17.20	18.98
40	177.9	39.26	35.89	28.51	22.50	21.08	23.04

Traffic Equivalence Factors, Flexible Pavement

Tandem Axles, p_t = 2.5

Axle Load		Structural Number, SN					
Kips	kN	1	2	3	4	5	6
10	44.5	0.01	0.01	0.01	0.01	0.01	0.01
12	53.4	0.02	0.02	0.02	0.02	0.01	0.01
14	62.3	0.03	0.04	0.04	0.03	0.03	0.02
16	71.2	0.04	0.07	0.07	0.06	0.05	0.04
18	80.1	0.07	0.10	0.11	0.09	0.08	0.07
20	89.0	0.11	0.14	0.16	0.14	0.12	0.11
22	97.9	0.16	0.20	0.23	0.21	0.18	0.17
24	106.8	0.23	0.27	0.31	0.29	0.26	0.24
26	115.7	0.33	0.37	0.42	0.40	0.36	0.34
28	124.6	0.45	0.49	0.55	0.53	0.50	0.47
30	133.4	0.61	0.65	0.70	0.70	0.66	0.63
32	142.3	0.81	0.84	0.89	0.89	0.86	0.83
34	151.2	1.06	1.08	1.11	1.11	1.09	1.08
36	160.1	1.38	1.38	1.38	1.38	1.38	1.38
38	169.0	1.75	1.73	1.69	1.68	1.70	1.73
40	177.9	2.21	2.16	2.06	2.03	2.08	2.14
42	186.8	2.76	2.67	2.49	2.43	2.51	2.61
44	195.7	3.41	3.27	2.99	2.88	3.00	3.16
46	204.6	4.18	3.98	3.58	3.40	3.55	3.79
48	213.5	5.08	4.80	4.25	3.98	4.17	4.49

Visual Aid 10.12. Lane distribution factors on multilane roads.

Number of Lanes in Both Directions	Percent of $W_{t_{18}}$ in Design Lane
2	100
4	80 - 100
6	60 - 80

Visual Aid 10.13. Derivation of equivalency load factors - rigid.

$$\text{Log } W_t = 5.85 + 7.35 (\text{Log } D + 1) - 4.62 (\text{Log } L_1 + L_2) + 3.28 \text{ Log } L_2 \, G_t/\beta$$

Visual Aid 10.14. Derivation of equivalency load factors - rigid.

If L_1 equals 18 kips and L_2 equals 1, for single axles,

$$\text{Log } W_{t_{18}} = 5.85 + 7.35 \text{ Log}(D + 1) - 462 \text{ Log } (18 + 1) + G_t/\beta_{18}$$

For any other axle load L_1 equal to X,

$$\text{Log } W_{t_x} = 5.85 + 7.35 \text{ Log } (D + 1) - 4.62 \text{ Log } (L_x + L_2) + 3.28 \text{ Log } L_2 + G_t/\beta_x$$

Subtracting:

$$\text{Log } W_{t_x}/W_{t_{18}} = 4.62 \text{ Log } (18 + 1) - 4.62 \text{ Log } (L_x + L_2) + 3.28 \text{ Log } L_2 + G_t/\beta_x - G_t/\beta_x$$

Visual Aid 10.15. Derivation of equivalency load factors - rigid.

For single axles ($L_2 = 1$),

$$\text{Log } W_{t_x}/W_{t_{18}} = 4.62 \text{ Log } (18 + 1) - 4.62 \text{ Log}(L_x + 1) + G_t/\beta_x - G_t/\beta_{18}$$

or, for tandem axles ($L_2 = 2$), to:

$$\text{Log } W_{t_x}/W_{t_{18}} = 4.62 \text{ Log } (18 + 1) - 4.62 \text{ Log } (L_x + 2) + 3.28 \text{ Log } 2 + G_t/\beta_x - G_t/\beta_{18}$$

Visual Aid 10.16. Rigid pavement traffic equivalence factors (p_t = 2.5).

Traffic Equivalence Factors, Rigid Pavement

Single Axles, p_t = 2.5

Axle Load		D - Slab Thickness - inches						
Kips	kN	6	7	8	9	10	11	12
2	8.9	0.0002	0.0002	0.0002	0.0002	0.0002	0.0002	0.0002
4	17.8	0.003	0.002	0.002	0.002	0.002	0.002	0.002
6	26.7	0.01	0.01	0.01	0.01	0.01	0.01	0.01
8	35.6	0.04	0.04	0.03	0.03	0.03	0.03	0.03
10	44.5	0.10	0.09	0.08	0.08	0.08	0.08	0.08
12	53.4	0.20	0.19	0.18	0.18	0.18	0.17	0.17
14	62.3	0.38	0.36	0.35	0.34	0.34	0.34	0.34
16	71.2	0.63	0.62	0.61	0.60	0.60	0.60	0.60
18	80.1	1.00	1.00	1.00	1.00	1.00	1.00	1.00
20	89.0	1.51	1.52	1.55	1.57	1.58	1.58	1.59
22	97.9	2.21	2.20	2.28	2.34	2.38	2.40	2.41
24	106.8	3.16	3.10	3.23	3.36	3.45	3.50	3.53
26	115.7	4.41	4.26	4.42	4.67	4.85	4.95	5.01
28	124.6	6.05	5.76	5.92	6.29	6.61	6.81	6.92
30	133.4	8.16	7.67	7.79	8.28	8.79	9.14	9.34
32	142.3	10.81	10.06	10.10	10.70	11.43	11.99	12.35
34	151.2	14.12	13.04	12.34	13.62	14.59	15.43	16.01
36	160.1	18.20	16.69	16.41	17.12	18.33	19.52	20.39
38	169.0	23.15	21.14	20.61	21.31	22.74	24.31	25.58
40	177.9	29.11	26.49	25.65	26.29	27.91	29.90	31.64

Table D.2-2

Traffic Equivalence Factors, Rigid Pavement

Tandem Axles, p_t = 2.5

Axle Load		D - Slab Thickness - inches						
Kips	kN	6	7	8	9	10	11	12
10	44.5	0.01	0.01	0.01	0.01	0.01	0.01	0.01
12	53.4	0.03	0.03	0.03	0.03	0.03	0.03	0.03
14	62.3	0.06	0.05	0.05	0.05	0.05	0.05	0.05
16	71.2	0.10	0.09	0.08	0.08	0.08	0.08	0.08
18	80.1	0.16	0.14	0.14	0.13	0.13	0.13	0.13
20	89.0	0.23	0.22	0.21	0.21	0.20	0.20	0.20
22	97.9	0.34	0.32	0.31	0.31	0.30	0.30	0.30
24	106.8	0.48	0.46	0.45	0.44	0.44	0.44	0.44
26	115.7	0.64	0.64	0.63	0.62	0.62	0.62	0.62
28	124.6	0.85	0.85	0.85	0.85	0.85	0.85	0.85
30	133.4	1.11	1.12	1.13	1.14	1.14	1.14	1.14
32	142.3	1.43	1.44	1.47	1.49	1.50	1.51	1.51
34	151.2	1.82	1.82	1.87	1.92	1.95	1.96	1.97
36	160.1	2.29	2.27	2.35	2.43	2.48	2.51	2.52
38	169.0	2.85	2.80	2.91	3.04	3.12	3.16	3.18
40	177.9	3.52	3.42	3.55	3.74	3.87	3.94	3.98
42	186.8	4.32	4.16	4.30	4.55	4.74	4.86	4.91
44	195.7	5.26	5.01	5.16	5.48	5.75	5.92	6.01
46	204.6	6.36	6.01	6.14	6.53	6.90	7.14	7.28
48	213.5	7.64	7.16	7.27	7.73	8.21	8.55	8.75

Visual Aid 10.17. Example of determination of equivalent 18-kip (80kN) single axle loads from Loadometer station data.

Axle Load Groups, lbs	Representative Axle Load, lbs	Equiv. Factor[1]	No. of Axles[2]	Equiv. 18-kip Single Axles
Single Axles				
Under 3,000	2,000	0.0003	512	0.2
3,000-6,999	5,000	0.012	536	6.4
7,000-7,999	7,500	0.0425	239	10.2
8,000-11,999	10,000	0.12	1,453	174.4
12,000-15,999	14,000	0.40	279	111.6
16,000-18,000	17,000	0.825	106	87.5
18,001-20,000	19,000	1.245	43	53.5
20,001-21,999	21,000	1.83	4	7.3
22,000-23,999	23,000	2.63	3	7.9
24,000 and over			0	
			Subtotal	459.0
Tandem Axles				
Under 6,000	4,000	0.01	9	–
6,000-11,999	9,000	0.008	337	2.7
12,000-17,999	15,000	0.055	396	21.8
18,000-23,999	21,000	0.195	457	89.1
24,000-29,999	27,000	0.485	815	395.3
30,000-32,000	31,000	0.795	342	271.9
32,001-33,999	33,000	1.00	243	243.0
34,000-35,999	35,000	1.245	173	215.4
36,000-37,999	37,000	1.535	71	109.0
38,000-39,999	39,000	1.875	9	16.9
40,000-41,999	41,000	2.275	0	–
42,000-43,999	43,000	2.74	1	2.7
44,000 and over	–	–	0	–
			Subtotal	1,367.8
			Total	1,826.8

Total, all trucks = 3,146

[1] For $p_t = 2.5$, and SN = 3.0

[2] Loadometer station data for 3,146 trucks

LESSON OUTLINE
RIGID PAVEMENT THEORY - STRESSES AND DEFLECTIONS

Instructional Objectives

1. To introduce the student to rigid pavement theory and the assumptions that are made in its analysis.

2. To familiarize the student with the stresses that are developed in rigid pavement and factors affecting the stresses.

Performance Objectives

1. The student should understand the concepts and complexity of rigid pavement analysis.

2. The student should be able to calculate the stresses covered and explain under what conditions a particular stress phenomenon can lead to failure.

Abbreviated Outline	Time, minutes
Introduction	10
Assumptions	10
Stresses due to bonding	25
Relative stiffness of slabs	20
Stresses due to warping	25
Stresses due to friction	20
	110 minutes

Reading Assignment

1. Instructional Text

2. Yoder & Witczak - Chapter 3, pp. 81-110

3. RTAC - Part 5.4

LESSON OUTLINE

RIGID PAVEMENT THEORY - STRESSES AND DEFLECTIONS

1.0 INTRODUCTION

A rigid pavement consists of a relatively thin slab placed upon subgrade foundation or base course. Since the modulus of elasticity of the concrete slab is much greater than that of the foundation material, a major portion of the load carrying capacity is derived from the bending in the slab itself. This has often been referred to as "beam or slab" action. (Ref. 1)

1.1 Causes of Stresses

Stresses result from a variety of causes, including wheel loads, cyclic changes in temperature (warping and shrinkage or expansion), changes in moisture, and volumetric changes in the subgrade or base course.

1.2 Magnitude of Stresses

The magnitude of the stresses depends upon continuity of the subgrade support. Complete continuity can be destroyed by pumping or plastic deformation of the subgrade. In addition, slab deformation itself, causes stresses of widely varying intensity.

2.0 ASSUMPTIONS

It is evident that the stress inducing factors are extremely varied and complex; in some cases they cannot be evaluated except by making certain simplifying assumptions. Thus, in the mathematical analysis certain assumptions are made regarding continuity and elasticity.

2.1 Conditions not Handled

- Permanent deformation of the supporting medium
- Badly cracked slabs (cannot resist bending)

3.0 STRESSES DUE TO BENDING (Visual Aid 11.1)

Consider a beam fully supported on an elastic foundation. Assume the reactive pressure is proportional to the deflection, that is:

$$p = kw$$

where, k is the modulus
p is the pressure
w is the vertical deflection

3.2 Radius of Curvature (R)

For beams:

$1/R = M/EI$
M = Bending Movement
E = Modulus of Elasticity
I = Moment of Inertia

This implies that stiffer subgrades result in higher stresses in concrete pavements, than do those of lesser stiffness.

3.3 Deflection - Curvature Relationships

$$EI\frac{d^4w}{dx^4} = -\frac{d^2M}{dx^2} = -k_z + q$$

where,
x = horizontal distance
w = vertical deflection
Others are previously defined

This is the differential equation defining deflection curvature of a beam supported on an elastic medium.

4.0 RELATIVE STIFFNESS OF SLABS

Slab deformations are dependent on the position, magnitude and area of "contact" of the load on the pavement surface. The resistance to deformation depends upon the stiffness of the supporting medium, as well as upon the flexural stiffness of the slab.

$$M = EI\frac{d^2w}{dx^2}$$

$$M_x = \frac{Eh^3}{12\ (1-\mu^2)}\ \frac{d^2w}{dx^2}$$

If Slab Stiffness $D = \dfrac{Eh^3}{12\ (1-\mu^2)}$

Then,

$$M_x = D\frac{d^2w}{dx^2}$$

The relative stiffness of the slab and subgrade according to Westergaard is: (Westergaard will be covered in subsequent sections.)

$$\ell = \sqrt[4]{\frac{Eh^3}{12\ (1 - \mu^2)\ k}}$$

where

ℓ = radius of relative stiffness (in.)
E = modulus of elasticity of the pavement (psi)
h = thickness of the pavement (in.)
μ = Poisson's ratio of the pavement
k = modulus of subgrade reaction (pci)

Visual Aid 11.2 shows calculated values of " ℓ "

5.0 STRESSES DUE TO WARPING (Visual Aid 11.3)

If a pavement is subjected to a temperature gradient through its depth, the surface will tend to warp. The tendency to warp is restrained by the weight of the slab itself. The analysis of stresses in rigid slabs is based upon work done by Westergaard and others.

$$-\frac{\partial^2 W}{\partial x^2} = \frac{12}{Eh^3}\ (M_x - \mu My) + \frac{\varepsilon_t\ \Delta t}{h}$$

$$-\frac{\partial^2 W}{\partial y^2} = \frac{12}{Eh^3}\ (My - \mu M_x) + \frac{\varepsilon_t\ \Delta t}{h}$$

$$-\frac{\partial^2 W}{\partial x \partial y} = \frac{12}{Eh^3}\ (1 + \mu)\ M_{xy}$$

where new variables are

M_x = movement in x direction

M_y = movement in y direction

M_{xy} = torsional movement

E_t = coefficient of expansion

Δt = temperature differential

∂^2 = 2nd partial differential

5.1 Westergaard Equations

Considering warping stresses caused by temperature differential through the slab, Dr. Westergaard developed equations for three cases. For case 1, the slab is assumed to be infinite in both "x" and "y" directions, for case 2 the slab is assumed to be infinite in the plus "y" and the plus and or minus "x" direction, and for case 3 the slab is assumed to be infinite in both the plus "x" and the plus "y" direction. Stresses for cases 2 and 3 are expressed in terms of the results for case 1. The derivation of the three cases is in Yoder and Witczak p. 85-87.

5.2 Bradbury Coefficients (Visual Aid 11.4)

Bradbury used Westergaard's concepts to develop coefficients for solution of the problem. The coefficient C_1 is in the desired direction, whereas C_2 is for the direction perpendicular to this direction (e.g. C_x and C_y) L_x and L_y are the free length and width respectively.

5.2.1 Edge Stresses.

$$\sigma = \frac{CE_t \Delta t}{2}$$

5.2.2 Interior Stresses.

$$\sigma = \frac{E_{\varepsilon t} \Delta t}{2} \left(\frac{C_1 + \mu C_2}{1 - \mu^2} \right)$$

5.2.3 Example Problem (Yoder & Witczak, p 88).

Determine the warping stress for a 10-inch concrete pavement with 40-foot transverse joints, width of lane is 12 foot. The modulus of subgrade reaction is 100 pci, assume temperature differential for day conditions to be 3°F per inch.

Longitudinal Edge Stresses:

$$\frac{1.05\ (4{,}000{,}000)\ (0.000005)\ (30)}{2} = 315 \text{ psi}$$

Interior Stresses:

$$\frac{4{,}000{,}000\ (0.000005)\ (30)}{2}\left[\frac{1.05 + 0.15\ (0.25)}{1 - (0.15)^2}\right] = 365 \text{ psi}$$

6.0 STRESSES DUE TO FRICTION

Stresses can also be set up in rigid pavements as a result of uniform temperature changes that cause the slab to contract or expand. If a slab cools uniformly, a crack will generally occur at about the center of the slab. Shrinkage of the concrete also causes cracks to form. Excessive expansion may cause "blowups" to occur.

6.1 Friction Between Slab and Subgrade (Visual Aid 11.5)

For equilibrium conditions, the summation of the friction forces from the center of the slab to the free end must be equal to the total tension in the concrete.

6.1.1 Displacement. Friction forces imply movement. It has been shown that the minimum amount of displacement required for friction to be fully developed is 0.06 in.

6.1.2 Distribution of Stress. A contracting shrinking slab will move more at its free end than in the center, with the result that frictional resistance varies along the slab from the center to the free edge.

6.2 Shearing Resistance of Soil or Base Course

If a concrete slab is poured on subgrade or base course, the bottom face of the slab is rough and in intimate contact with the subgrade. As contraction takes place, shearing stresses are transmitted down through the subgrade until they are dissipated at some depth. Thus, it is seen that rough concrete sliding over soil will have a coefficient of resistance which is dependent, in part upon the shearing resistance of the soil or base course.

6.2.1 Balanced Forces.

$$\sigma_c = \frac{W\ L\ f}{24h}$$

where

σ_c = unit stress in the concrete (psi)

W = weight of slab (psf)

L = length of slab (ft)

f = average coefficient of subgrade resistance

h = depth of slab (in.)

6.2.2 Average Subgrade Coefficient (Visual Aid 11.5). For "x" less than 1/2 L

$$f_a = f_m \left(1 - \frac{x^2}{3L}\right)$$

For "x" greater than 1/2 L

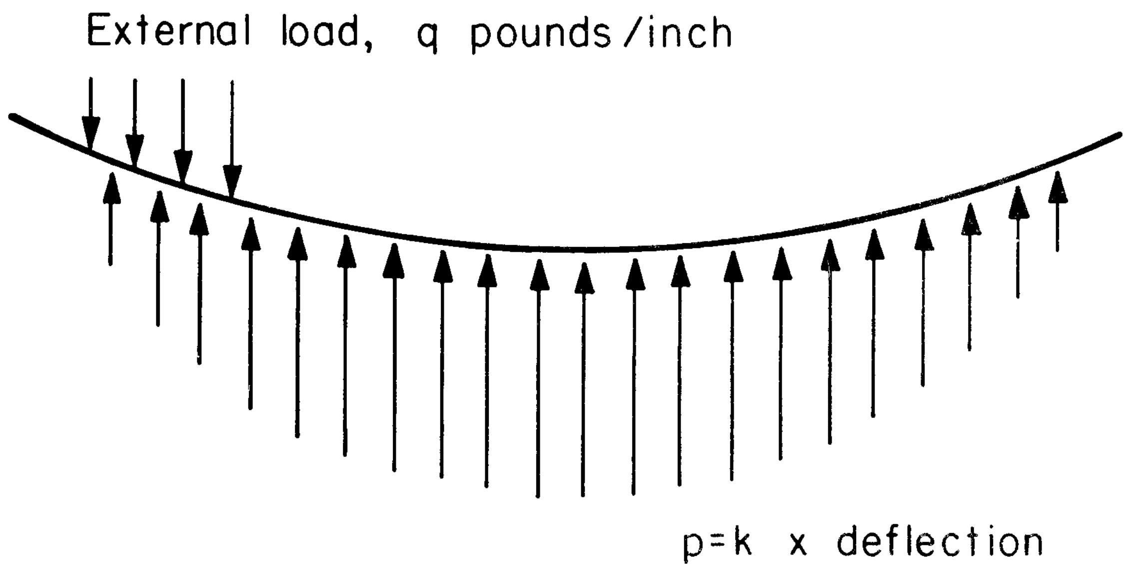
$$f_a = 2/3\ f_m \sqrt{\frac{L}{2x}}$$

The value of f is generally taken to be 1.5.

LESSON OUTLINE
RIGID PAVEMENT THEORY - STRESSES AND DEFLECTIONS

VISUAL AID	TITLE
Visual Aid 11.1.	Deflected beam on elastic foundation.
Visual Aid 11.2.	Radius of relative stiffness.
Visual Aid 11.3.	Curvature of elastic surface due to temperature warping.
Visual Aid 11.4.	Warping stress coefficients.
Visual Aid 11.5.	Stress resulting from contraction.

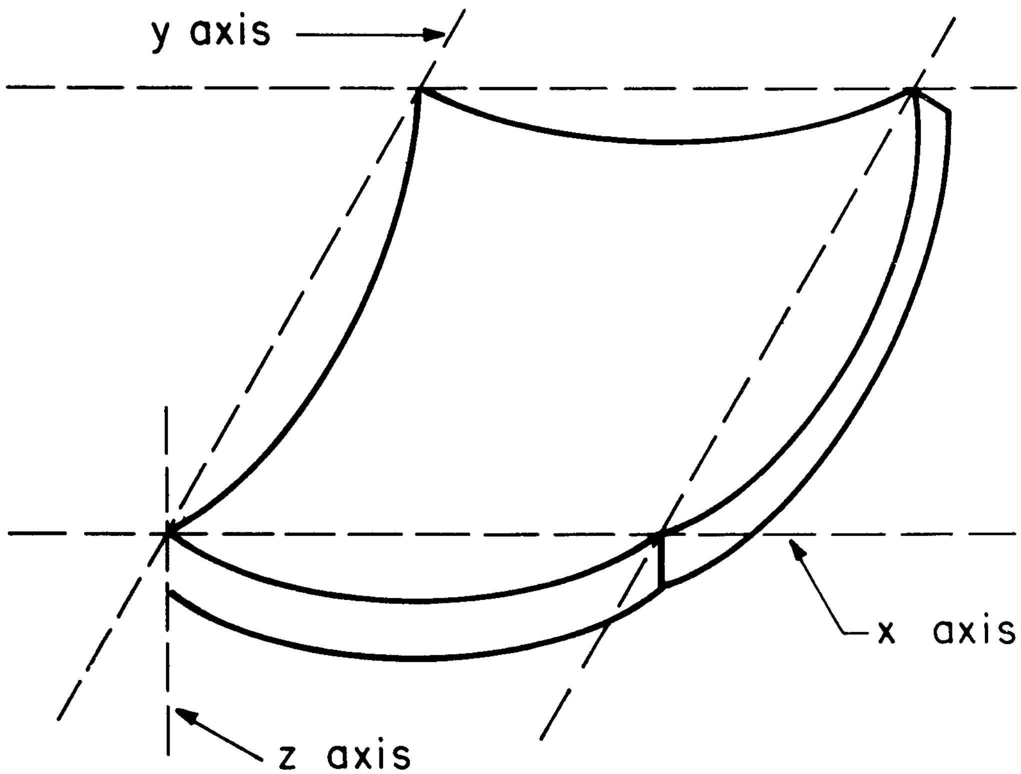
Visual Aid 11.1. Deflected beam on elastic foundation.



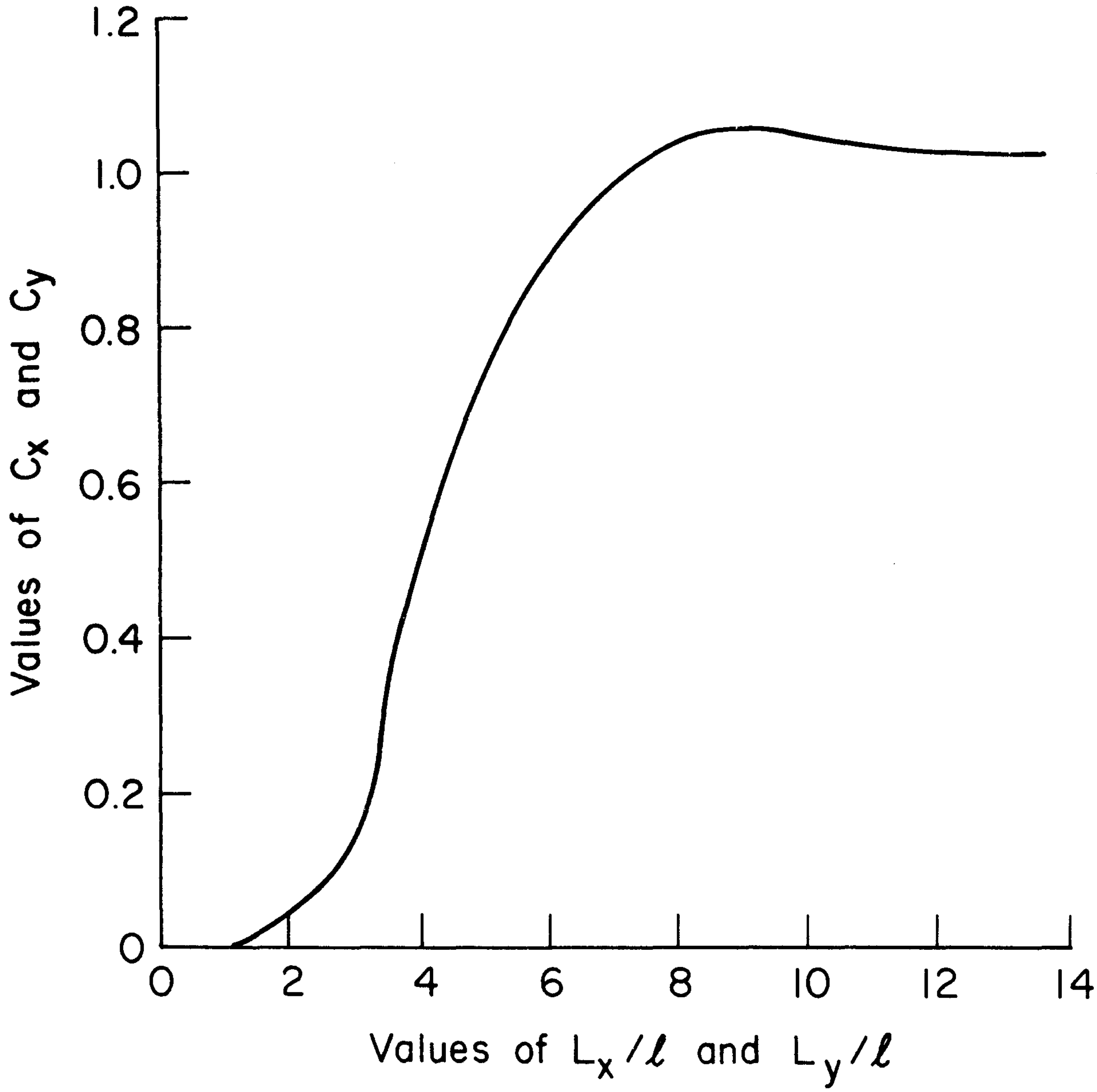
Visual Aid 11.2. Radius of relative stiffness.

h(in.)	K = 50	K = 100	k = 200	K = 300	K = 400	K = 500
9.0	47.22	39.71	33.39	30.17	28.08	26.55
9.5	49.17	41.35	34.77	31.42	29.24	27.65
10.0	51.10	42.97	36.14	32.65	30.39	28.74
10.5	53.01	44.57	37.48	33.87	31.52	29.81
11.0	54.89	46.16	38.81	35.07	32.64	30.87
11.5	56.75	47.72	40.13	36.26	33.74	31.91
12.0	58.59	49.27	41.43	37.44	34.84	32.95
12.5	60.41	50.80	42.72	38.60	35.92	33.97
13.0	62.22	52.32	43.99	39.75	36.99	34.99
14.0	65.77	55.31	46.51	42.02	39.11	36.99
15.0	69.27	58.25	48.98	44.26	41.19	38.95
16.0	72.70	61.13	51.41	46.45	43.23	40.88
17.0	76.08	63.98	53.80	48.61	45.24	42.78
18.0	79.41	66.78	56.16	50.74	47.22	44.66
19.0	82.70	69.54	58.48	52.84	49.17	46.51
20.0	85.95	72.27	60.77	54.92	51.10	48.33
21.0	89.15	74.97	63.04	56.96	53.01	50.13
22.0	92.31	77.63	65.28	58.98	54.89	51.91
23.0	95.44	80.26	67.49	60.98	56.75	53.67
24.0	98.54	82.86	69.68	62.96	58.59	55.41

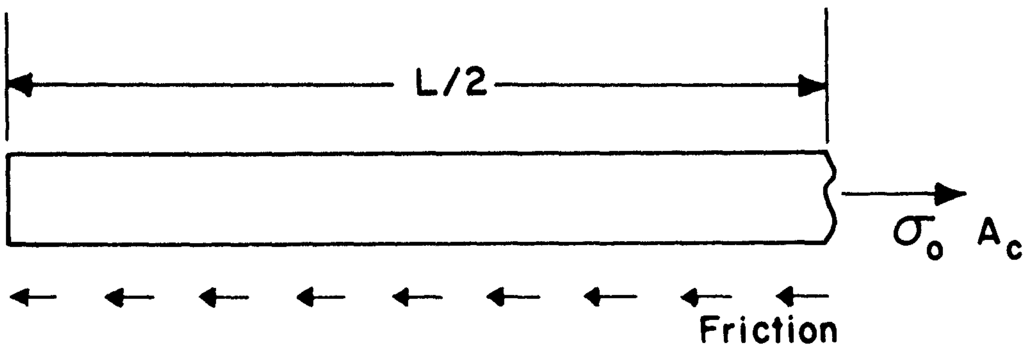
Visual Aid 11.3. Curvature of elastic surface due to temperature warping.

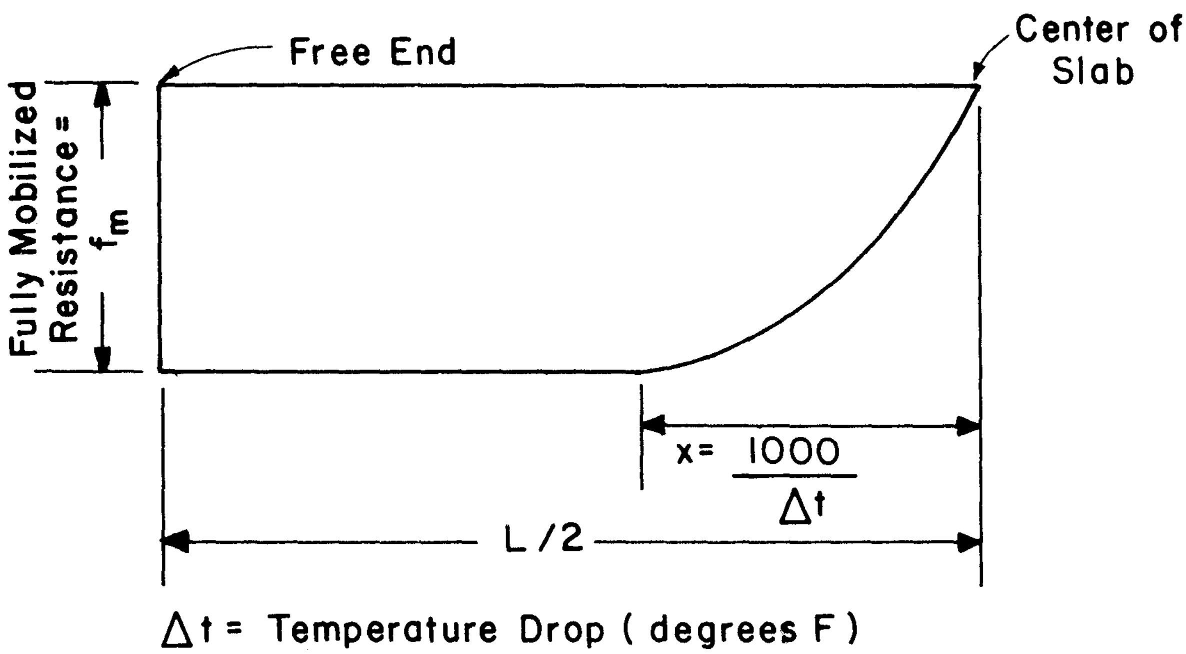


Visual Aid 11.4. Warping stress coefficients.



Visual Aid 11.5. Stresses resulting from contraction.





INSTRUCTIONAL TEXT

CURLING OF RIGID PAVEMENT SLABS DUE TO TEMPERATURE DIFFERENTIAL

By

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Master Thesis in Civil Engineering
The University of Texas at Austin
June 1967

CHAPTER I: INTRODUCTION

The Problem

Pavement designers must know the stress conditions in rigid slabs in order to rationally design structural concrete pavements. Such stresses are caused either by the external loads imposed upon the slab or by volume changes inherent in the concrete. External loads are those such as traffic loads, shrinkage and swelling of the subgrade, subgrade friction, and other restraining forces acting externally on the slab. Volume changes in the slab are caused by the shrinkage during hardening and changes are often referred to as "internal stresses" or "secondary stresses."

Design equations and charts have been developed experimentally through the years for several conditions of loading. Best known are the equations developed by Dr. H. M. Westergaard in 1926 for analyzing stresses in slabs of uniform thickness for three conditions of loading: corner, edge, and interior. (Ref. 31) Since that time, others have made important contributions to the development of design of concrete pavements--especially design of external loading conditions. The extent of those stresses caused by volume change in slabs, however, are more difficult to define and a great deal of research is still needed in this area.

Of particular interest in this thesis are those volume changes caused by differential temperature and moisture. Differential temperature occurs when the temperature of the top surface of a slab is different from the temperature at the bottom surface, the magnitude being the difference. Differential moisture is when the moisture content between the top and the bottom surface is different, the magnitude also being the difference. This condition is often referred to as "curling" and the resulting stresses are usually accounted for in the design procedure by using a factor of safety.

Curling and Warping Defined

The words "curling" and "warping" are often used interchangeably. The meaning of the two words, however, should not be confused. Curling is defined as "the distortion of a pavement slab from its proper plane caused by differential expansion or contraction resulting from a difference in moisture content or in temperature between the top and bottom of the slab." (4) Warping is defined as "the distortion or displacement of a pavement slab from its proper plane caused by external forces, other than loads." (4, 21) For example, the volumetric changes in the subgrade cause warping, while a differential gradient of moisture or of temperature within the slab causes curling.

Description of the Curling Phenomenon

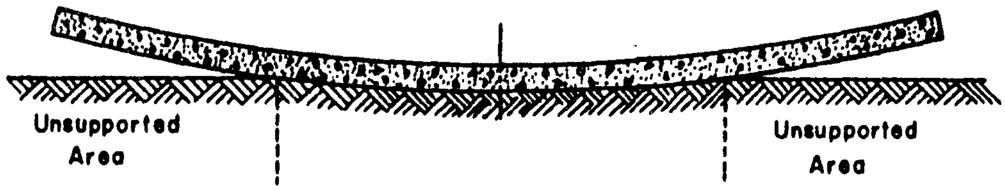
Temperatures in a pavement slab are seldom uniform. AASHO Road Test[1] curling studies showed that points on the upper surface of pavement slabs were usually in continuous vertical motion during periods of changing air

1. The AASHO Road Test was a comprehensive highway research study of the performance of pavement and bridge structures of known characteristics under moving loads of known magnitude and frequency. It was administered by the Highway Research Board of the National Academy of Sciences--National Research Council.

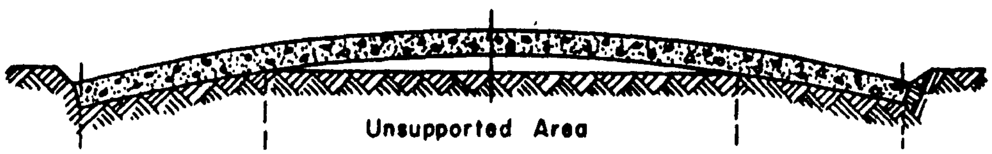
temperature. (2, 3) This phenomenon occurred because concrete is a relatively slow conductor of heat, and a temperature differential was created by the lag in time required for heat to transfer through the slab.

In a typical daily cycle a pavement slab will curl both upward and downward, especially in the spring and fall when there are greater ranges in daily temperatures. At night, when the surface of the slab is cooling, the surface length decreases and the slab curls upward; this tends to lift the corners and edges off the base. The corners and edges normally reach their maximum elevation early in the morning. The reverse occurs in the daytime when the surface is heated by the warmth of the day and the sun's rays. The surface then expands and curls the slab downward. Usually in late afternoon the lowest elevation is reached at the corners and edges while the center of the slab has risen to its maximum elevation. These extremes during the daily curling cycle are illustrated in Figure I-1, page 4. The magnitude of distortion due to temperature curling in the daily cycle is not large in relation to other slab dimensions. For example, as early as 1922 Older reported maximum vertical movements at the corner of slabs on the Bates Test Road of 0.25" for slabs 18' wide. (20) More recently at the AASHO Road Test corner displacements in a range from 0.09" to 0.15" were reported for slabs 12' wide. (2)

Curling due to moisture differential between the upper and lower surfaces of a slab occurs slowly and is not detectable in a daily cycle like that resulting from temperature differential. Moisture curling is more apparent from seasonal changes. Most pavement slabs in service are wet on the bottom surface and probably never dry out or lose appreciable moisture under normal conditions. This keeps the bottom surface of the slab saturated or nearly saturated and in an expanded condition almost



A. Slab Curled-up. (Typical of early morning, about dawn.)



B. Slab Curled-down. (Typical of late afternoon.)

Figure I-1. Illustration of daily curling extremes and changing support conditions due to temperature differential. (Scale is exaggerated.)

constantly. On the other hand, the upper surface is usually drier and in a contracted state relative to the bottom surface. This vertical differential of moisture thus tends to curl the slab upward and add to any upward curling that is due to temperature differential. However, curling due to moisture differential would compensate downward curling due to temperature differential. In the spring, when the subgrade is the wettest and the temperature differential is the greatest, the most critical combination of these two types of curling is probably reached. The magnitude of the distortion from moisture curling alone is more difficult to measure, especially on slabs in service.(1) In a controlled experiment at Purdue University, Hatt reported corner curling of 0.20" after soaking the bottom of the slab for 110 days. (11) This indicates that curling due to moisture differential may have effects on the slab of equal magnitude to those from temperature differential.

Effects of Curling

The effects of curling on the performance of rigid pavements are probably much greater than most highway designers suspect. Probably the most important effect of curling is that it alters the condition of support. As a slab curls in its daily temperature cycle and in its seasonal moisture cycle, some portion of the pavement slab is lifted off the base. Not only does this affect the magnitude of the stress that will be produced by wheel loads, but it tends to invalidate any design assumptions that the slab is uniformly supported. Even in the absence of wheel loads, there are significant curling stresses in the slab caused by the weight of the slab. It is also obvious that curling places additional stress in load transfer devices, adds to the difficulty of keeping joints sealed, and contributes to the erosion of the base or subgrade. All these effects allegedly lead to other pavement deficiencies such as surface roughness, joint faulting, cracking, and pumping. (5, 14, 15)

The effects of curling obviously place important limitations on the accuracy of present design procedures. A critical combination of stresses from both load and curling can easily exceed the designed modulus of rupture and eventually cause pavement failure. (5) In other instances the factor of safety used in design may be more than is needed for the critical environmental conditions. Any information that would help define the effects of curling in such a way as to lead to a more sophisticated design procedure would certainly be of value to this field of study.

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LESSON OUTLINE
RIGID PAVEMENT BEHAVIOR - BIHARMONIC EQUATION

Instructional Objectives

1. To introduce the student to the physical and theoretical assumptions that are included in the derivation of the biharmonic equation.

2. To briefly explain the biharmonic equation as an introduction to the various solutions that will be presented in upcoming lectures.

Performance Objectives

1. The student should be able to relay the basic assumptions that are incorporated in the analyses that are founded on the biharmonic equation. In particular the student should be able to recognize field situations where the assumptions do not apply.

Abbreviated Outline	Time Allocations, min.
1. Introduction	5
2. Biharmonic Equation	20
3. Generalized Hookes Law	20
4. Summary	5
	50 minutes

Reading Assignment

1. Yoder & Witczak - Pages 81 to 92

LESSON OUTLINE
RIGID PAVEMENT BEHAVIOR - BIHARMONIC EQUATION

1.0 INTRODUCTION

1.1 Categories of Theories

Pavement slabs on a foundation can be considered to be plates with various support conditions. Three kinds of plates are:

(a) thin plates - small deflection,

(b) thin plates - large deflection, and

(c) thick plates.

1.2 Small Deflections

1.2.1 Basic Assumptions. A satisfactory approximate bending theory can be obtained by assuming:

(a) there is no deformation in the center of plate (the middle plane of the slab),

(b) planes of the slab initially lying normal to the middle plane of the slab remain normal after bending. In other words, there is no slippage between planes.

(c) Normal stress in the direction transverse to the slab can be disregarded. It means that there is no vertical deformation.

2.0 BIHARMONIC EQUATION (VISUAL AID 12.1)

$$\frac{d^2w}{dx^2} = M$$

$$\frac{dM}{dx} = \tau$$

$$\frac{d\tau}{dx} = q \qquad q = \frac{d^2M}{dx^2}$$

The above equations describe the basic relationships between load and deflection etc. Structural plates and pavement slabs are normally subjected to loads applied perpendicular to their surface, i.e., lateral loads. Timoshenko and others have derived a differential equation which describes the deflection surface of such plates, the Biharmonic Equation (Equation 1).

$$\frac{dMx}{dx^2} - \frac{d^2Myx}{dxdy} + \frac{d^2My}{dy^2} - \frac{d^2Mxy}{dxdy} = q \qquad (1)$$

Where:

Mx = bending moment in x direction

My = bending moment in y direction

Mxy = twisting moment about x axis

Myx = twisting moment about y axis

q = lateral load

(Visual Aid 12.2)

By observing that Myx = Mxy, this allows reduction to

$$\frac{d^2Mx}{dx^2} + \frac{d^2My}{dx^2} - 2\,\frac{d^2Mxy}{dxdy} = q \qquad (2)$$

In the case of isotropy,

$$\left.\begin{aligned} Mx &= D\left(\frac{d^2w}{dx^2} + \mu\,\frac{d^2w}{dy^2}\right) \\ My &= D\left(\frac{d^2w}{dy^2} + \mu\,\frac{d^2w}{dx^2}\right) \\ Mxy &= Myx = D\,(1 - \mu)\,\frac{d^2w}{dxdy} \end{aligned}\right\} \qquad (3)$$

Where:

D = bending stiffness of the plate

μ = poisson's ratio

Substituting Eq. 3 in Eq. 2, we obtain

$$\frac{d^4w}{dx^4} + 2\,\frac{d^2w}{dx^2dy^2} + \frac{d^4w}{dy^4} = \frac{q}{D} \qquad (4)$$

It is seen that the problem of bending of plates by a lateral load q reduces to the integration of Eq. 4.

3.0 HOOKE'S LAW

3.1 Concepts

Elasticity: If the external forces producing deformation do not exceed a certain limit, and the bodies undergoing the action of external forces are assumed to be elastic, then they resume their initial form completely after removal of the forces. Linear relations between the components of stress and the components of strain are known generally as (Visual Aid 12.3).

3.2 Three Planes of Symmetry

$$\varepsilon_x = \frac{\sigma_x}{E}$$

This extention of the element in the x direction is accompanied by lateral strain components (contractions).

$$\varepsilon_y = -\mu \frac{\sigma_x}{E} \qquad \varepsilon_z = -\mu \frac{\sigma_x}{E}$$

If we superpose the strain components produced by each of three stresses, we obtain the equations.

$$\varepsilon_x = \frac{1}{E}\left[\sigma_x - \mu\,(\sigma_y + \sigma_z)\right]$$

$$\varepsilon_y = \frac{1}{E}\left[\sigma_y - \mu\,(\sigma_x + \sigma_z)\right]$$

$$\varepsilon_z = \frac{1}{E}\left[\sigma_z - \mu\,(\sigma_x + \sigma_y)\right]$$

If shearing stresses act on all the sides of an element, as shown in Visual Aid 12.4 the distortion of the angle between any two intersecting sides depends only on the corresponding shear - stress component. We have,

$$\gamma_{xy} = \frac{1}{G}\tau_{xy}$$

$$\gamma_{yz} = \frac{1}{G}\tau_{yz}$$

$$\gamma_{zx} = \frac{1}{G}\tau_{zx}$$

Where:

G = Modulus of Elasticity in Shear $G = \dfrac{E}{2\ (1 + \mu)}$

These distortions are independent of the elongations. By assuming there is no vertical deformation, and the material is isotropic, we obtain

$(\sigma_z = 0,\ \varepsilon_z = 0)$

$$\sigma_x = \frac{E}{1 - \mu^2}\ (\varepsilon_x - \mu\varepsilon_y)$$

$$\sigma_y = \frac{E}{1 - \mu^2}\ (\varepsilon_y - \mu\varepsilon_x)$$

$$\tau_{xy} = G\ \gamma_{xy}$$

For isotropic plates, only two independent elastic constants are required. (E, G)

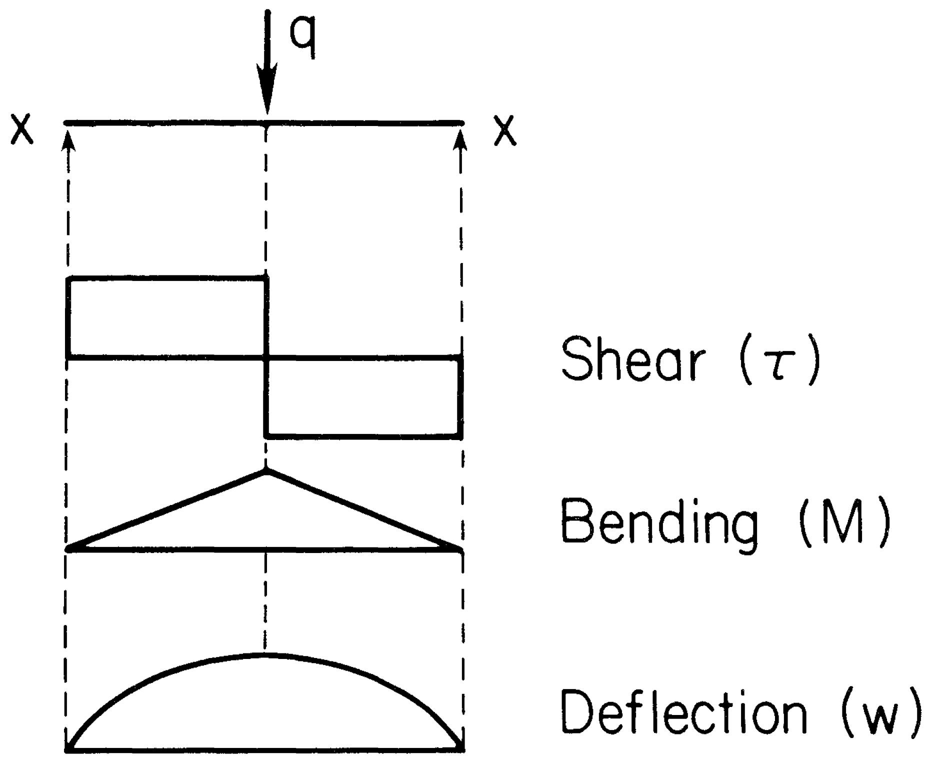
4.0 SUMMARY

Solutions of pavement slabs, or slab-on-foundation, are of particular interest in here. There are two basic theories concerning the behavior of such slabs. The first assumes that the intensity of the reaction of the foundation on the slab is proportional to the deflection (w) of the slab. This intensity is then given by the expression kw. Where the constant k, expressed in pounds per square inch per inch of deflection, is called the "support modulus of the foundation." The second theory considers the foundation of the slab as a semi-infinite elastic half-space. Although a great deal of work has been done on the pavement slab problem, probably the most significant work to date was accomplished by Westergaard, particularly with reference to the design problems encountered in concrete pavement. The Westergaard solutions to the biharmonic equation will be discussed in the next lecture.

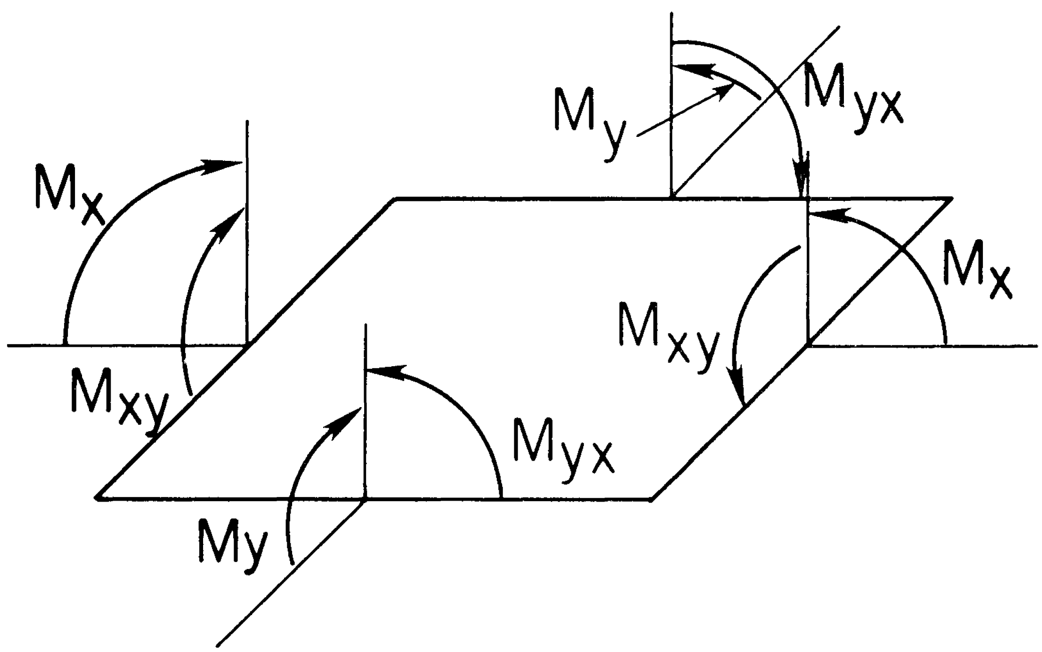
LESSON OUTLINE
RIGID PAVEMENT BEHAVIOR - BIHARMONIC EQUATION

VISUAL AID	TITLE
Visual Aid 12.1.	Simple beam configuration.
Visual Aid 12.2.	Bending moments and twisting moments on plate.
Visual Aid 12.3.	Elasticity.
Visual Aid 12.4.	Normal and shearing stresses on a cubic element.

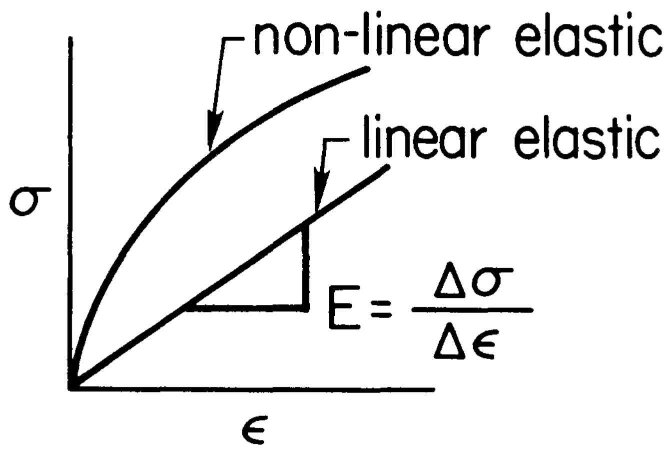
Visual Aid 12.1. Simple beam configuration.

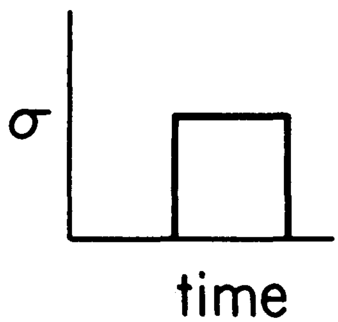


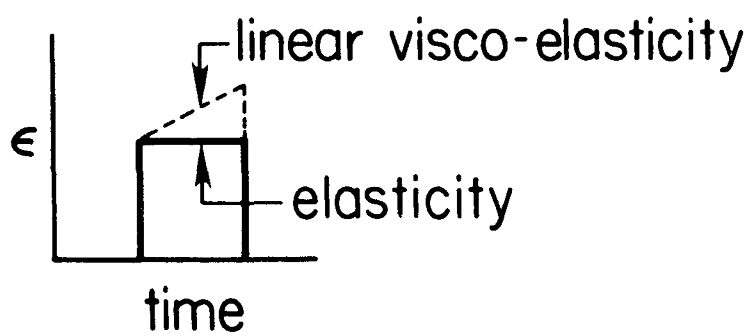
Visual Aid 12.2. Bending moments and twisting moments on plate.



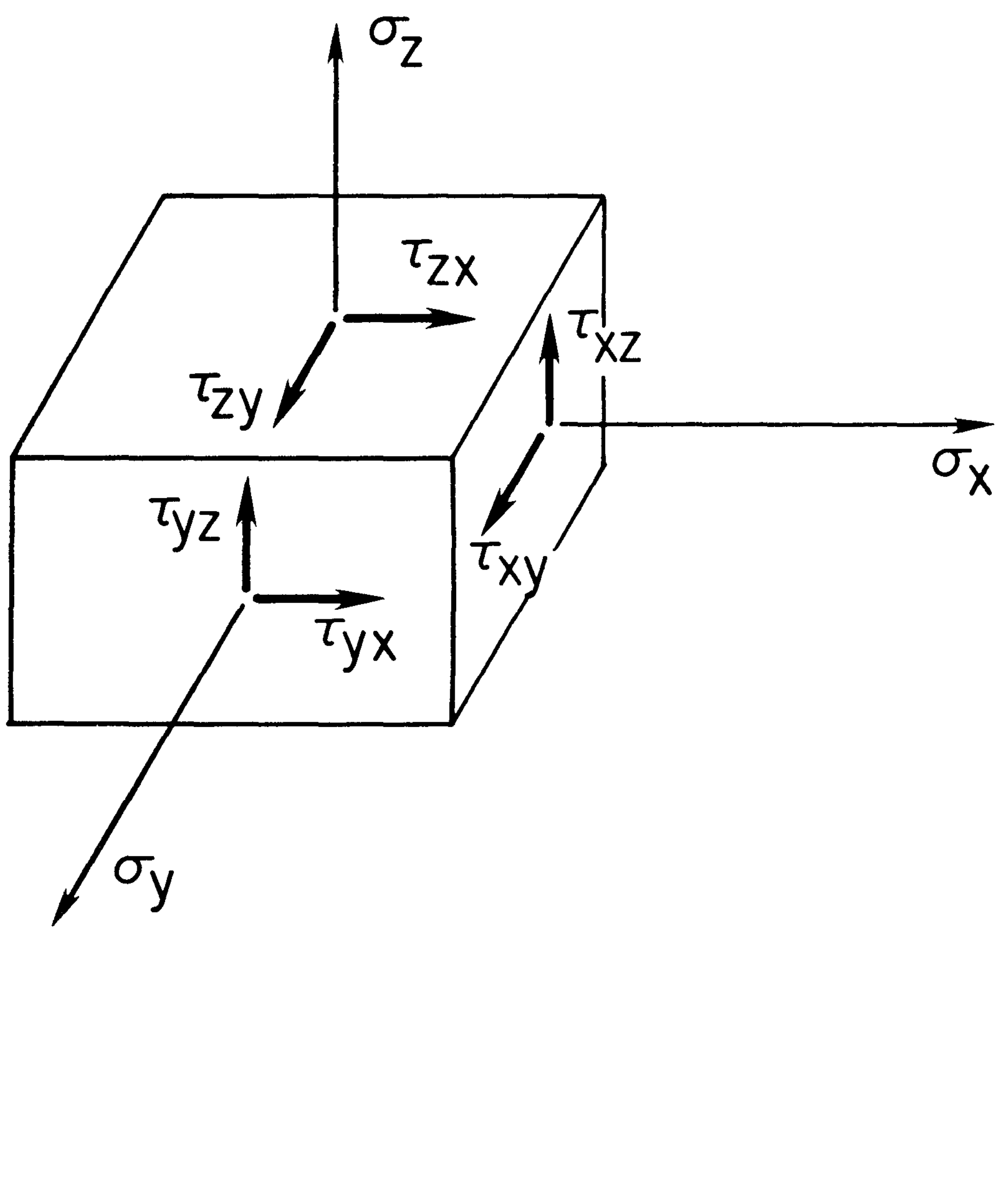
Visual Aid 12.3. Elasticity.







Visual Aid 12.4. Normal and shearing stresses on a cubic element.



LESSON OUTLINE

RIGID PAVEMENT - WESTERGAARD SOLUTIONS

Instructional Objectives

1. To establish the Westergaard solutions to the biharmonic equation for rigid slabs.

2. To introduce the three loading cases and to compare the results in the light of slab design.

Performance Objectives

1. The student should be aware of the complexity of rigid slab stress calculations and the assumptions that must be made for solution to be practical using this theoretical approach.

2. The student should be able to calculate deflections and stresses for all three loading cases and relate how to use these answers in a design analysis.

Abbreviated Summary	Time Allocations, min.
1. Background	10
2. Solution Development	20
3. Comparison of the Three Cases	10
4. Deflections	10
	50 minutes

Reading Assignment

1. Yoder and Witczak - pp. 110 to 121

2. Instructional Text

LESSON OUTLINE
RIGID PAVEMENT - WESTERGAARD SOLUTIONS

1.0 BACKGROUND

In 1926 Dr. H. M. Westergaard presented formulas for computing the stresses in plain Portland cement concrete pavements. He was an Associate Professor of Theoretical and Applied Mechanics at the University of Illinois when his analysis was presented before the Highway Research Board.

1.1 Loading Cases (Visual Aid 13.1)

1.1.1 Corner. Load applied near the corner of the rectangular slab.

1.1.2 Edge. Load applied near the edge of the slab, but at a considerable distance from the corner.

1.1.3 Interior. Load applied at the interior of a large slab at considerable distance from any edge.

1.2 Assumptions

In developing the formulas Westergaard made the following important assumptions. Using the assumptions, he developed solutions for the deflected shape of the pavement slab, then, the maximum moments and stresses.

1.2.1 Concrete Slab. The concrete slab acts as a homogeneous, isotropic, elastic solid in equilibrium.

1.2.2 Vertical Reactions of the Subgrade. The reactions of the subgrade are vertical only, and they are proportional to the deflections of the slab.

1.2.3 Reaction to Subgrade Equal to Modulus of Support Multiplied By the Deflection at that Point. The reaction to the subgrade is equal to the modulus of support multiplied by the deflection at that point. K is assumed to be constant at every point, independent of the deflection, and to be the same at all points within the area of consideration.

1.2.4 Thickness. The thickness of the slab is uniform.

1.2.5 Load at the Interior and Near Corner of Slab. For the cases of load at the interior and near the corner of the slab, the load is distributed uniformly over a circular area of contact. For the corner load, the circumference of this circular area is tangent to two edges of the slab.

1.2.6 Load at Edge of Slab. The load at the edge of the slab is distributed uniformly over a semi-circular area of contact; the diameter of the semi-circle occurs at the edge of the slab.

1.2.7 Slab is Infinite. The slab is infinite in all directions away from the load.

2.0 SOLUTION DEVELOPMENT

2.1 Radius of Relative Stiffness (ℓ)

$$\ell \propto \frac{\text{Slab Stiffness (E)}}{\text{Support Stiffness (K)}}$$

$$\ell \propto \frac{E_{\text{conc.}},\ \text{Thickness}}{K_{\text{Soil}}}$$

$$\ell = \sqrt{\frac{Eh^3}{12\ (1 - \mu^2)\ K}} \quad : 16 \text{ in.} < \ell < 55 \text{ in.}$$

The stronger the support (K value increases), the smaller ℓ, the load P spread less. Generally, $\sigma_{slab} \propto \ell$; the higher the relative stiffness, the higher stress in slab. For example:

σ_A : glass on mattress

σ_B : rug on mattress

$\sigma_A > \sigma_B$; glass has higher stiffness than rug.

2.2 Corner Equation

2.2.1 Goldbeck and Older Equation. (Visual Aid 13.2) Assumes that:

(a) load is concentrated at the corner, and

(b) there is no support.

$$M = P\ x \qquad (1)$$

$$\sigma = \frac{M\ c}{I} \qquad (2)$$

$$I = \frac{Bh^3}{12} = \frac{2\ x\ h^3}{12} \qquad (3)$$

Substituting Eq. 1 and Eq. 3 in Eq. 2, we obtain:

$$\sigma = \frac{P \times h/2}{2 \times h^3/12} = \frac{3P}{h^2}$$

2.2.2 Westergaard Equation (Corner). (Visual Aid 13.3 , 13.4 and 13.5)

$$M \text{ unit length} = \frac{M}{2x}$$

Max value occurred at $X_1 = 2\sqrt{a_1 \ell}$ and equal to

$$M = \frac{P}{Z}\left[1 - \left(\frac{a_1}{\ell}\right)^{0.6}\right]$$

I unit length (Moment of Inertia) = $h^3/12$

$$\sigma_c = \frac{M c}{I} = \frac{3P}{h^2}\left[1 - \left(\frac{a_1}{\ell}\right)^{0.6}\right]$$

Review:

a_1	increase	σc	decrease
P	increase	σc	increase
h	increase	σc	decrease
E	increase	σc	increase

In 1926, Arlington Road Test led Kelly to change the power from 0.6 to 1.2; as a result, stress increases by 25% to 50%.

2.2.3 Influence of Variables for Corner Load.

σ_c based on P = 10,000 lbs.
E = 3,000,000 psi
μ = 0.15

From Table:

Variation of a is appreciable
Variation of h is appreciable
Variation of K is not appreciable

2.3 Interior Load (Visual Aid 13.6)

2.3.1 Westergaard Equation.

$$\sigma_i = 0.31625 \frac{P}{h^2}\left[4 \log (\ell/b) + 1.0693\right]$$

for $a < 1.724h$ $b = \sqrt{1.6a^2 + h^2} - 0.675h$

for $a > 1.724h$ $b = a$

Under the same load, thickness, and contact area, the stress of interior is smaller than corner.

2.3.2 Influence of Variables for Interior Load. (Visual Aid 13.7)

From Table:

Variation of a is appreciable
Variation of h is appreciable
Variation of K is not appreciable

2.4 Edge Load (Visual Aid 13.8)

2.4.1 Westergaard Equation.

$$\sigma_e = 0.57185 \frac{P}{h^2} \left[4 \log (\ell/b) + 0.3593\right]$$

Contact area of edge case is the smallest in three cases, therefore, the stress of slab is the largest. (Use for conservative design).

2.4.2 Influence of Variables for Edge Load. (Visual Aid 13.9)

From Table:

Variation of a is appreciable
Variation of h is appreciable
Variation of K is not appreciable

3.0 COMPARISON OF THE THREE CASES

From the three tables for P = 10,000 lbs.

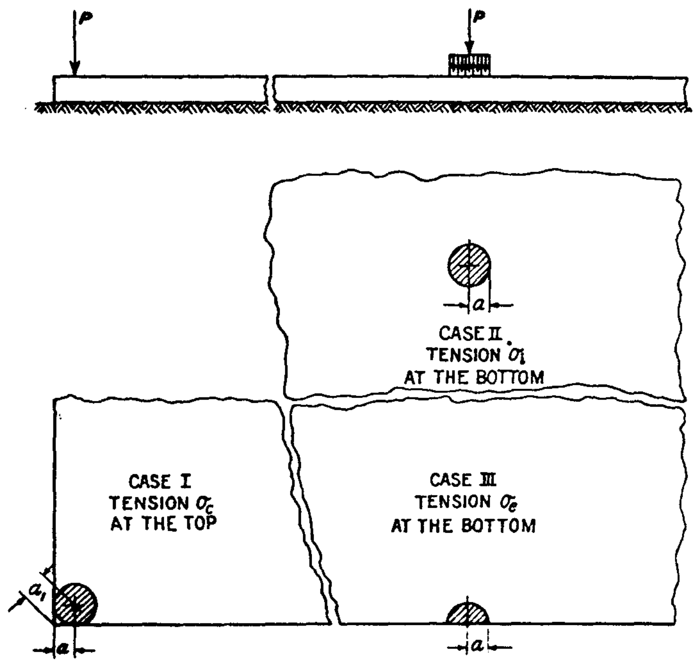
$K = 50$ lb/in.^{3} and $a = 4$ in.

$\sigma_c = 262$ psi $\sigma_i = 319$ psi $\sigma_e = 312$ psi

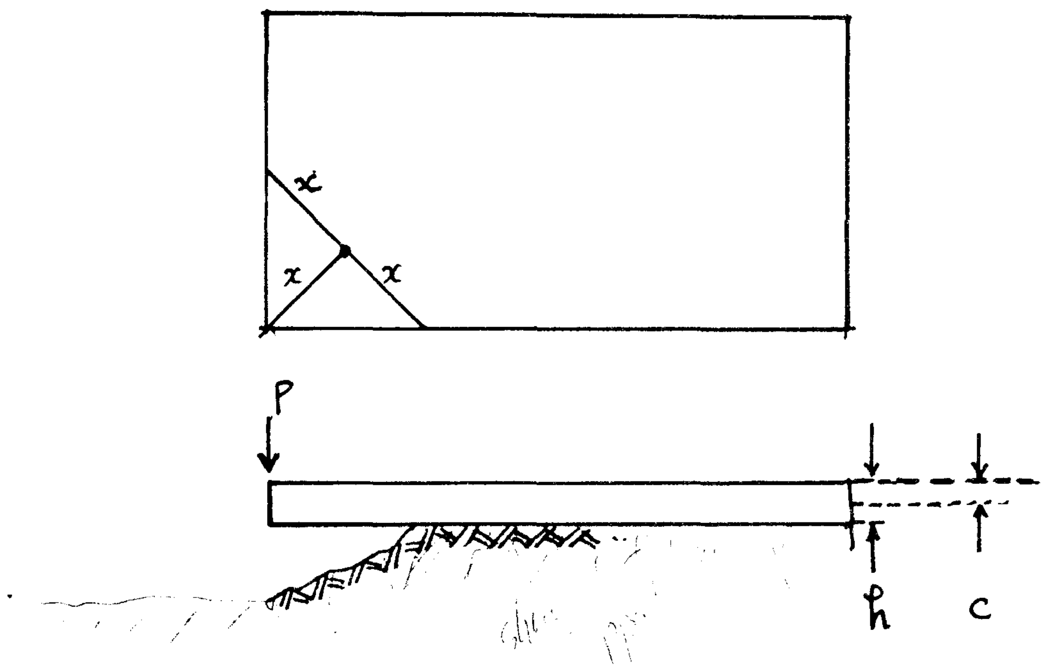
This example considers a pavement with the thickness of 7 inches in the interior portion, and 9 inches at the edges and corners. In comparing three stresses, their different characteristics should be considered. The stress, σ_c, at the corner acts presumably throughout the width of a whole cross section, whereas σ_i and σ_e are localized within smaller regions. With equal tendency to rupture at the three places, σ_c then, should be, probably, somewhat smaller then σ_i and σ_e.

LESSON OUTLINE
RIGID PAVEMENT - WESTERGAARD SOLUTIONS

Visual Aid 13.1. Three cases of loading.



Visual Aid 13.2. Load at the corner of slab.



Visual Aid 13.3. Modifications of corner equation.

Bradbury (1934):

$$\sigma_c = \frac{3P}{h^2}\left[1 - (a_1/\sqrt{2\ell})^{0.6}\right]$$

Kelly (1939):

$$\sigma_c = \frac{3P}{h^2}\left[1 - \frac{a_1}{\ell}^{1.2}\right]$$

Spangler (1942):

$$\sigma_c = \frac{3.2P}{h^2}\left[1 - \frac{a_1}{\ell}\right]$$

Pickett (1946):

Protected

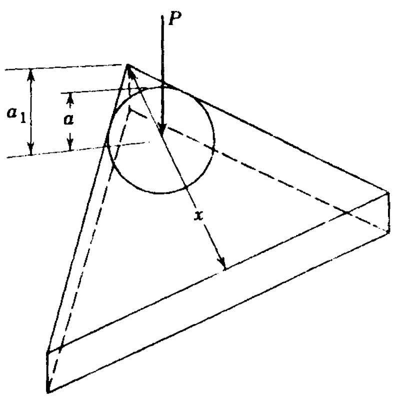
$$\sigma_c = \frac{3.36P}{h^2} \quad 1 - \left[\frac{\sqrt{a/\ell}}{0.925 + 0.22\ a/\ell}\right]$$

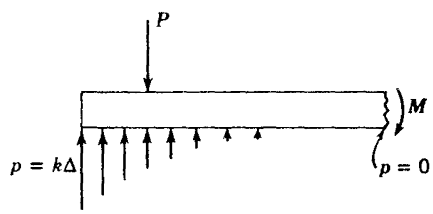
Unprotected

$$\sigma_c = \frac{4.2P}{h^2}\left[\qquad\qquad\right]$$

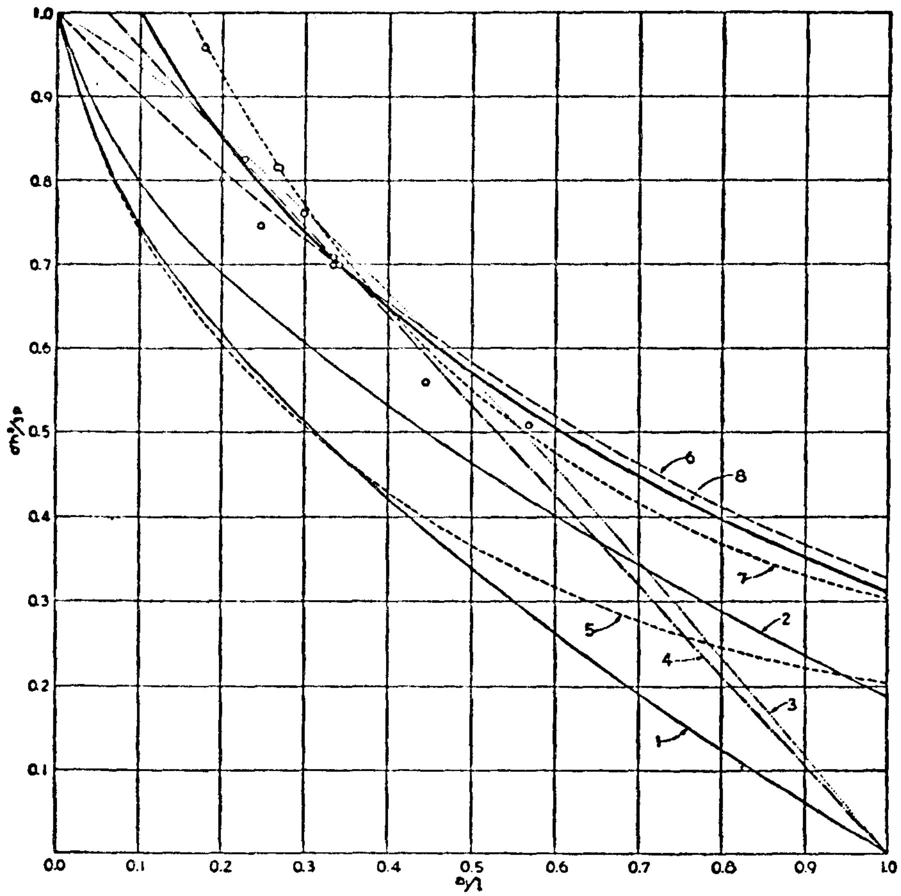
Visual Aid 13.4. Stresses acting under corner load.

INFLUENCE CHARTS





Visual Aid 13.5. Concrete pavement design.



LEGEND

$$\sigma = \frac{3P}{h^2}\left[1 - \left(\frac{a_1}{l}\right)^{0.6}\right] \quad (1)$$

$$\sigma = \frac{3P}{h^2}\left[1 - \left(\frac{a_1}{\sqrt{2}\,l}\right)^{0.6}\right] \quad (2)$$

$$\sigma = \frac{3P}{h^2}\left[1 - \left(\frac{a_1}{l}\right)^{1.2}\right] \quad (3)$$

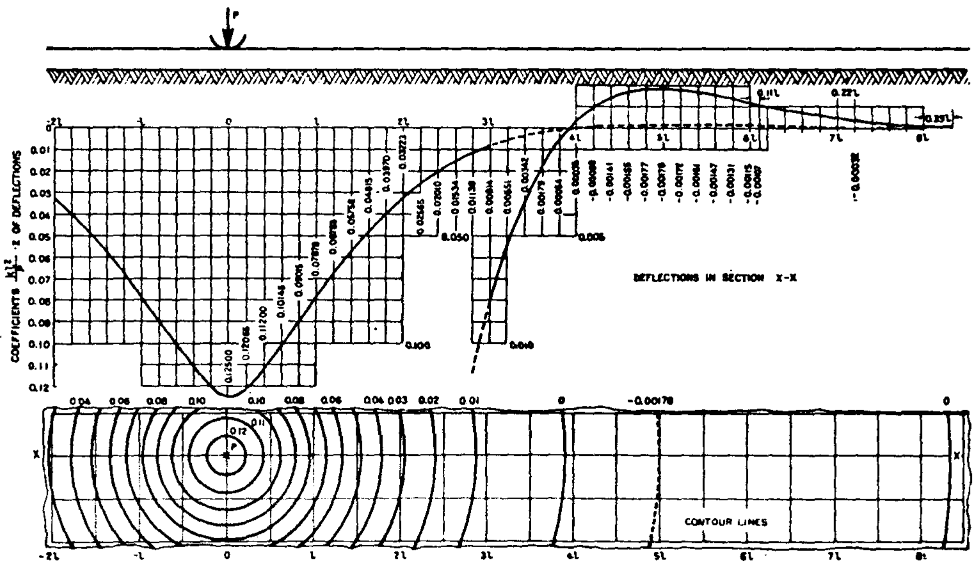
$$\sigma = \frac{3.2P}{h^2}\left[1 - \frac{a_1}{l}\right] \quad (4)$$

Visual Aid 13.6. Influence of variables for corner load.

P = 10,000 pounds, E = 3,000,000 pounds per square inch, μ = 0.15

Thick-ness of slab, h	Modulus of sub-grade reaction, k	Stress in slab			
		a = 0	a= 2 inches	a = 4 inches	a = 6 inches
Inches	Lb./in.^{3}	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.
6	50	833	641	541	461
	100	833	619	509	420
	200	833	596	474	375
7	50	612	480	412	357
	100	612	466	390	329
	200	612	450	366	298
8	50	469	373	325	285
	100	469	363	309	265
	200	469	352	291	242
9	50	370	299	262	233
	100	370	291	250	217
	200	370	282	237	201
10	50	300	245	216	193
	100	300	239	207	182
	200	300	232	197	169
11	50	248	204	182	164
	100	248	200	175	154
	200	248	194	167	144
12	50	208	173	155	140
	100	208	169	149	133
	200	208	165	143	124

Visual Aid 13.7. Deflections produced by a concentrated load at the interior.

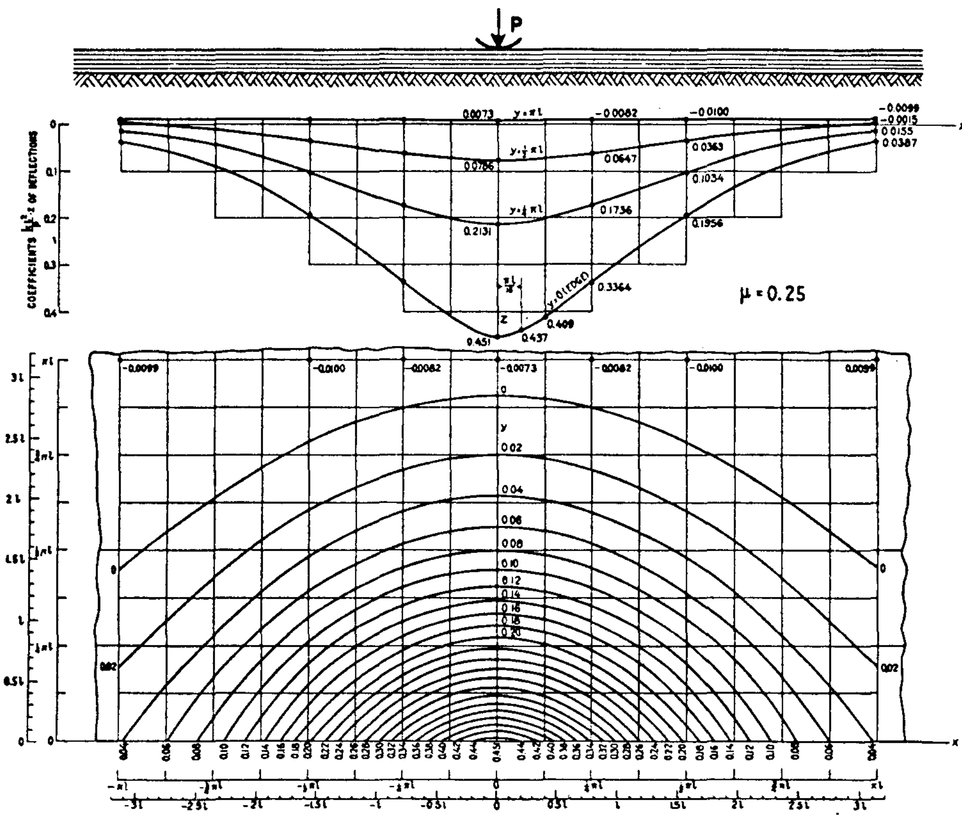


Visual Aid 13.8. Influence of variables for interior load.

P = 10,000 pounds, E = 3,000,000 pounds per square inch, μ = 0.15

Thickness of slab, h	Modulus of subgrade reaction, k	Stress in slab				
		a = 0	a= 2 inches	a = 4 inches	a = 6 inches	a = 8 inches
Inches	Lb./in.^{3}	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.
4	50	1,231	1,058	848	693	588
	600	998	845	634	480	367
	1500	919	766	556	401	288
5	50	763	694	580	487	415
	600	626	557	443	350	279
	1500	576	507	393	300	228
6	50	523	487	421	361	313
	600	428	393	326	266	218
	1500	393	358	291	232	183
7	50	380	360	319	279	245
	600	310	290	249	209	175
	1500	285	265	224	184	150
8	50	288	276	250	222	197
	600	235	223	196	168	144
	1500	215	203	177	149	124
9	50	226	218	200	180	162
	600	183	176	158	139	120
	1500	168	160	143	123	104
10	50	181	176	164	149	136
	600	147	142	130	115	101
	1500	135	129	117	103	89

Visual Aid 13.9. Deflections produced by a concentrated load at the edge.



Visual Aid 13.10. Influence of variables for edge load.

P = 10,000 pounds, E = 3,000,000 pounds per square inch, μ = 0.15

Thick-ness of slab, h	Modulus of sub-grade reaction, k	Stress in slab				
		a = 0	a = 2 inches	a = 4 inches	a = 6 inches	a = 8 inches
Inches	$Lb./in.^3$	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.
6	50	833	769	649	541	453
	600	661	597	477	369	282
	1500	598	534	414	306	219
7	50	604	568	494	422	360
	600	478	442	368	296	234
	1500	432	396	322	249	188
8	50	457	436	388	337	293
	600	361	339	292	241	196
	1500	325	304	256	205	161
9	50	358	344	312	276	243
	600	282	268	236	200	167
	1500	253	240	208	172	138
10	50	287	278	256	230	204
	600	225	216	194	168	143
	1500	203	193	171	145	120
11	50	235	229	213	194	174
	600	184	178	162	143	123
	1500	166	159	143	124	104
12	50	196	192	180	165	150
	600	153	149	137	123	107
	1500	138	133	122	107	92

INSTRUCTIONAL TEXT

PROCEEDINGS OF FIFTH ANNUAL MEETING
HRB

COMPUTATION OF STRESSES IN CONCRETE ROADS

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One may obtain a computation of stresses in concrete roads by assuming the slab to act as a homogeneous isotropic elastic solid in equilibrium, and by assuming the reactions of the subgrade to be vertical only and to be proportional to the deflections of the slab. With these assumptions introduced, the analysis is reduced to a problem of mathematical theory of elasticity.

The reaction of the subgrade per unit of area at any given point will be expressed as a coefficient k times the deflection z at the point. This coefficient is a measure of the stiffness of the subgrade, and may be stated in pounds per square inch of area per inch of deflection, that is, in lb./in^3. The coefficient k will be called the *modulus of subgrade reaction*. It corresponds to the "modulus of elasticity of rail support" which has been used in recent investigations of stresses in railroad track.[1] The modulus k is assumed to be constant at each point, independent of the deflections, and to be the same at all points within the area which is under consideration. It is true that tests of bearing pressures on soils have indicated a modulus k which varies considerably depending upon the area over which the pressure is distributed.[2]

[1] Progress report of the special committee to report on stresses in railroad track, Am. Soc. Civil Engineers, Trans., v. 82, 1918, p. 1191.

[2] Tests dealing with this question have been reported by A. T. Goldbeck, Researches on the structural design of highways by the United States Bureau of Public Roads, Am. Soc. Civil Engineers, Trans., v. 88, 1925, p. 261, especially p. 271; by A. T. Goldbeck and M. J. Bussard, The supporting value of soil as influenced by the bearing area, Public Roads, Jan. 1925; and by A. Bijls, in Génie Civil, v. 82, 1923, p. 490. According to these tests, in the case of a pressure which is distributed uniformly over an area, the modulus k would be approximately inversely proportional to the square-root of the area. This result is supported by theoretical considerations.

Yet, so long as the loads are limited to a particular type, that of wheel loads on top of the pavement, it is reasonable to assume that some constant value of the modulus k, determined empirically, will lead to a sufficiently accurate analysis of the deflections and the stresses. One finds an argument in favor of the assumption of a constant modulus k for a given stretch of road by examining the tables which are given below: they show that an increase of k from 50 lb./in^3. to 200 lb./in^3., that is, an increase of the stiffness of the subgrade in the ratio of four to one, causes only minor changes of the important stresses. Minor variations of k, therefore, can be of no great consequence, and an approximate single value of k should be sufficient for a quite accurate determination of the important stresses within a given stretch of the road. The modulus k enters in the formula for the deflections of the pavements, and may be determined empirically, accordingly, for a given type of subgrade, by comparing the deflections found by tests of full-sized slabs with the deflections given by the formulas.

It will be assumed for the time being that the thickness of the slab is uniform and is equal to h.

A certain quantity which is a measure of the stiffness of the slab relative to that of the subgrade occurs repeatedly in the analysis. It is of the nature of a linear dimension, like, for example, the radius of gyration. It will be called the *radius of relative stiffness.* It is denoted by l, and is expressed by the formula

$$l = \sqrt[4]{\frac{Eh^3}{12\,(1 - \mu^2)\,k}} \quad\ldots\ldots\ldots\ldots\ldots\ldots\ldots(1)$$

where E is the modulus of elasticity of the concrete, and μ is Poisson's ratio of lateral expansion to longitudinal shortening. The stiffer the slab, and the less stiff the subgrade, the greater is l. One may observe that l remains constant when E and k are multiplied by the same ratio. Table I contains values of l for three different values of k and for different thickensses of the slab. In computing this table as well as the three tables following, Poisson's ratio μ was assumed to be 0.15; this value agrees satisfactorily with the results of tests by A. N. Johnson.[1] The values of l given in the table lie between 16 inches and 55 inches; about 36 inches may be considered to be a typical average.

THREE CASES OF LOADING INVESTIGATED

Figure 1 shows three cases in which it is of particular interest to be able to compute the critical stresses. In case I, a wheel load acts close to a rectangular corner of a large panel of the slab. This load tends toward producing a corner break. The critical stress is a tension at the top of the slab. The resultant pressure is assumed to be on the

[1] A. N. Johnson, Direct measurement of Poisson's ratio for concrete, Am. Soc. for Testing Materials, Proc., v. 24, Part II, 1924, p. 1024.

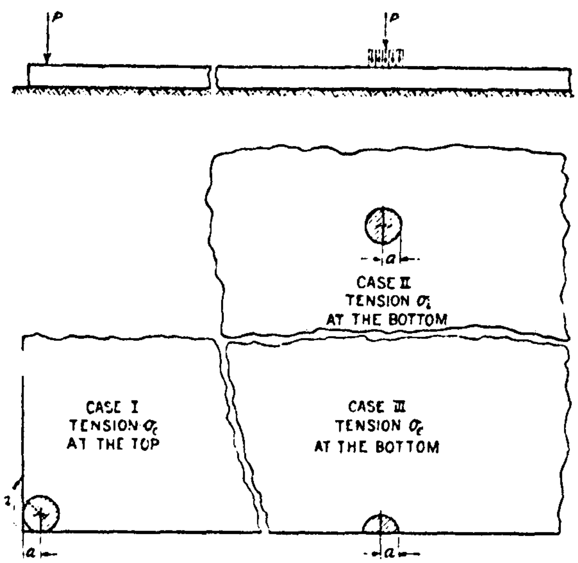


Figure 1—Three cases of loading. Corresponding greatest stresses are given in Tables II, III, and IV

bisector of the right angle of the corner, at the small distance a from each of the two intersecting edges; the distance from the corner, accordingly, is $a_1 = a\sqrt{2}$. In case *II*, the wheel load is at a considerable

TABLE I

Values of the radius of relative stiffness, l, for different values of the slab thickness, h, and of the modulus of subgrade reaction, k, computed from equation (1)

E = 3,000,000 pounds per square inch. μ = 0.15

Thickness of slab in inches h	Radius of relative stiffness, l, in inches		
	k = 50 lb./in.^{3}	k = 100 lb./in.^{3}	k = 200 lb./in.^{3}
4	23.91	20.11	16.92
5	28.28	23.78	20.00
6	32.40	27.26	22.92
7	36.40	30.60	25.73
8	40.23	33.83	28.44
9	43.94	36.95	31.07
10	47.55	40.00	33.62
11	51.08	42.94	36.11
12	54.52	45.84	38.56

distance from the edges. The pressure is assumed to be distributed uniformly over the area of a small circle with radius a. The critical tension occurs at the bottom of the slab under the center of the circle. In case *III*, the wheel load is at the edge, but at a considerable distance from any corner. The pressure is assumed to be distributed uniformly over the area of a small semicircle with the center at the edge and with radius a. The critical stress is a tension at the bottom under the center of the circle. In each of the three cases the load mentioned is assumed for the time being to be the only load acting.

For case *I* a computation which may be looked upon as a first approximation was proposed by A. T. Goldbeck. Further emphasis was given to this method by Clifford Older.[1] The load is treated as a force concentrated at the corner itself, that is, one assumes $a = a_1 = 0$. At small distances from the corner the influence of the reactions of the subgrade upon the stresses will be small compared with that due to the load. The corner portion may be considered, therefore, to act as a cantilever of uniform strength. At the distance x, measured diagonally from the corner along the bisector of the right angle of the corner, the bending moment is $-Px$. This bending moment may be assumed to be distributed uniformly over the cross-section, the width of which is $2x$. Thus one finds the bending moment per unit of width of cross-section equal to $-\frac{P}{2}$, and the tensile stress at the top equal to

$$\sigma = \frac{3P}{h^2} \qquad (2)$$

Since the wheel load is distributed over the area of contact between the tire and the pavement, the distances a and a_1 can not be zero. The greatest stress occurs, then, at some distances from the load. This distance will be sufficiently large to make the reactions of the subgrade outside the critical section contribute a noticeable reduction of the numerical value of the bending moment.

An improved approximation has been obtained in the following manner. The origin of the horizontal rectangular coordinates x and y is taken at the corner, the axis of x bisecting the right angle of the corner. By use of Ritz's method of successive approximation, which is based on the principle of minimum of energy,[1] the following approximate expression was found for the deflections in the neighborhood of the corner:

$$z = \frac{P}{kl^2}\left(1.1\, e^{-\frac{x}{l}} - \frac{a_1}{l} 0.88\, e^{-\frac{2x}{l}}\right) \qquad (3)$$

Then the reactions of the subgrade will be expressed with sufficient exactness in terms of this function as kz. One may compute, then, the total bending moment M^1 in the section $x = x_1$ due to the combined influence of the applied load and the reactions of the subgrade. When x_1 is not too large, this bending moment will be approximately uniformly

[1] Clifford Older, Highway research in Illinois, Am. Soc. Civil Engineers, Trans., v. 87, 1924, p. 1180, especially p. 1206.

[1] W. Ritz, Crelle's Journal, v. 135, 1909, p. 1.

distributed over the width $2x_1$ of the cross-section. That is, the bending moment per unit of width becomes $M = \frac{M^1}{2x_1}$. The numerically greatest value of M was found, in this manner, to occur approximately at the distance

$$x_1 = 2\sqrt{a_1 l} \dots \dots \dots \dots \dots (4)$$

and to be, approximately,

$$M = -\frac{P}{2}\left[1 - \left(\frac{a_1}{l}\right)^{0.6}\right] \dots \dots \dots \dots \dots (5)$$

Division by the section modulus per unit of width, $h^2/6$, leads to the corresponding greatest tensile stress

$$\sigma_c = \frac{3P}{h^2}\left[1 - \left(\frac{a_1}{l}\right)^{0.6}\right] \dots \dots \dots \dots \dots (6)$$

This stress may be stated also in the following form which is derived by substituting the value of l from equation (1):

$$\sigma_c = \frac{3P}{h^2}\left[1 - \left(\frac{E h^3}{12(1-\mu^2)k}\right)^{-0.15} \cdot a_1^{0.6}\right] \dots \dots \dots (7)$$

With $a_1 = 0$, the last two equations assume the simpler form of equation (2).

STRESS NOT GREATLY AFFECTED BY SUBGRADE CONDITION

Table II contains numerical values of the critical stress σ_c for $P =$ 10,000 lb., $E = 3{,}000{,}000$ lb. per sq. in., and $\mu = 0.15$. The table shows the influence of three variables: the thickness h, the modulus k of subgrade reaction, and the distance a from the edges to the center of the load.

An inspection of the table shows the influence of the variation of the distance a to be appreciable, amounting easily to a reduction of more than 30 per cent as compared with the value found by the first approximation, with $a = 0$. The influence of the variation of the modulus k from 50 to 200 lb./in^3, on the other hand, is not particularly large.

In case *II*, that of a wheel-load at a point of the interior, complications arise due to the fact that the load is concentrated within a rather small area. The theory of elasticity offers two types of theory of slabs: one theory may be called "ordinary theory of slabs," the other "special theory." The difference may be explained by an analogy with beams. In analysis of beams it is assumed ordinarily that a plane cross-section remains plane and perpendicular to the neutral surface during the bending. For beams of ordinary proportions, this assumption leads to satisfactory results, unless one is concerned with the local stresses in the immediate neighborhood of a concentrated load. In the latter case the assumption of the plane cross-section must be abandoned, and a special theory, which takes into account the deformations due to the vertical stresses, is required. In the ordinary theory of slabs it is assumed, correspondingly, that a straight line drawn through the slab perpendicular to the slab remains straight and perpendicular to the neutral surface. With slabs of proportions as found in pavements, the theory based on these assumptions leads to a satisfactory determination of stresses at all points except in the immediate neighborhood of a concentrated load, and leads to a satisfactory determination of the deflections at all points. At the point of application of a concentrated force this ordinary theory leads to a peak in the diagrams of bending moments, with infinite values at the point of the load itself (as indicated in Figures

TABLE II

Stresses in pounds per square inch computed from equation (7) for load condition as in Case I, Figure 1, for different values of h, k, and a

P = 10,000 pounds, E = 3,000,000 pounds per square inch, μ = 0.15

Thickness of slab, h	Modulus of subgrade reaction, k	Stress in slab			
		a = 0	a = 2 in.	a = 4 in.	a = 6 in.
Inches	*Lb./in.*^{3}	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*
6	50	833	641	541	461
	100	833	619	509	420
	200	833	596	474	375
7	50	612	480	412	357
	100	612	466	390	329
	200	612	450	366	298
8	50	469	373	325	285
	100	469	363	309	265
	200	469	352	291	242
9	50	370	299	262	233
	100	370	291	250	217
	200	370	282	237	201
10	50	300	245	216	193
	100	300	239	207	182
	200	300	232	197	169
11	50	248	204	182	164
	100	248	200	175	154
	200	248	194	167	144
12	50	208	173	155	140
	100	208	169	149	133
	200	208	165	143	124

5, 10, and 11). When the force is applied at the top of the slab, the tensile stresses at the bottom are not, in fact, infinite. One may say then that the effect of the thickness of the slab is equivalent to a rounding off of the peak in the diagrams of moments. In order to find out to what extent the diagrams are rounded off, it is necessary to abandon the assumption of the straight lines drawn through the slab remaining straight, as applying to the immediate neighborhood of the load, and a special theory is required. This special theory rests on only two assumptions: one is that Hooke's law applies, the constants being the modulus of elasticity E and Poisson's ratio μ; the other is that the material keeps its geometrical continuity at all points. As in the case of beams, the ordinary theory is much simpler than the special theory, and is used, therefore, except in particular cases like the present one, which deals with local effects around a concentrated load.

It is expedient to express the results of the special theory in terms of the ordinary theory in the following manner. Let the load P be distributed uniformly over the area of the small circle with radius a. The tensile stress produced by this load at the bottom of the slab under the center of the circle is denoted by σi. This stress is the critical stress

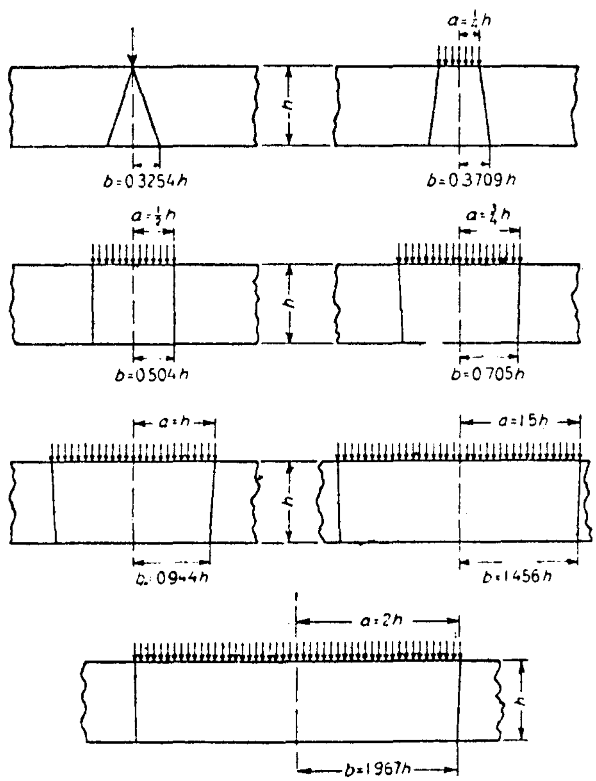


Figure 2—Cones of equivalent distribution of pressure

except when the radius a is so small that some of the vertical stresses near the top become more important; the latter exception need not be considered, however, in case of a wheel load which is applied through a rubber tire. By use of the ordinary theory one may find the same stress at the same place by assuming the load to be distributed over the area of a circle with the same center, but with the radius b. One finds that this equivalent radius b can be expressed with satisfactory approximation in terms of the true radius a and the thickness h only.

In order to find the relation between h, a, and b, numerical computations were made in accordance with an analysis which is due to A. Nadai.[1] The center of the load P is assumed for the time being to be at the center of a circular slab. The slab is supported at the edge in such a manner that the sum of the radial and tangential bending moments is zero at every point of the edge. Computations according to Nádai's analysis, with the radius of the slab equal to $5h$ gave the results which are represented in Figure 2 in the manner of "cones of equivalent distribution" and in Figure 3 by a curve with coordinates a and b. Approximately the same cones and the same curve are obtained for other radii of the slab; and the results may be applied generally to slabs of proportions such as are found in concrete pavements, with any kind of support which is not concentrated within a small area close to the load.

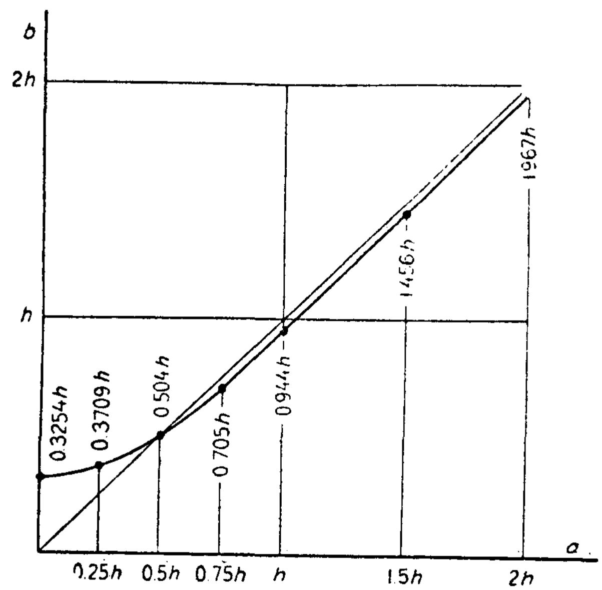


Figure 3—Relation between the true radius, a, the equivalent radius, b, and the thickness, h

[1] A. Nádai, Die Biegungsbeanspruchung von Platten durch Einzelkräfte, Schweizerische Bauzeitung, v. 76, 1920, p. 257; and his book, Die elastischen Platten, (Berlin) 1925, p. 308.

One may notice that when a increases gradually from zero, b is at first larger than a; but when a passes a certain limit, b becomes smaller than a. For the larger values of a, the ratio b/a converges toward unity, and the ordinary theory of slabs, accordingly, gives nearly the same results as the special theory.

The curve in Figure 3 is found to lie close to a hyperbola, the equation of which may be written in the following form, which is suitable for numerical computations, and which may be used for values of a less than $1.724h$:

$$b=\sqrt{1.6\,a^2 \times h^2}-0.675h \quad \ldots\ldots\ldots\ldots\ldots\ldots (8)$$

For larger values of a, one may use $b=a$, that is, the ordinary theory may be used without corrections.

By the ordinary theory one finds the following approximate expression for the critical stress:

$$\sigma_i = \frac{3\,(1+\mu)\,P}{2\,\pi\,h^2}\left(\log_e \frac{l}{a} + 0.6159\right) \ldots\ldots\ldots\ldots (9)$$

With $E=3{,}000{,}000$ lb. per sq. in. and $\mu=0.15$, and with l substituted from equation (1), this formula takes the form:

$$\sigma_i = 0.3162\frac{P}{h^2}\left(\log_{10}(h^3) - 4\log_{10} a - \log_{10} k + 6.478\right) \ldots (10)$$

The correction to be made in this formula in order to make it agree with the special theory is merely to replace the true radius a by the equivalent radius b. Thus one finds the following formula, which replaces equation (10) when a is less than $1.724h$:

$$\alpha_i = 0.3162\frac{P}{h^2}\left(\log_{10}(h^3) - 4\log_{10}\left(\sqrt{1.6\,a^2+h^2} - 0.675\,h\right) - \log_{10} k + 6.478\right) \ldots\ldots\ldots\ldots (11)$$

The stresses given in Table III have been computed in accordance with this formula for $P=10{,}000$ pounds. Like Table II, this table shows the influence of three variables: the thickness h, the modulus k of subgrade reaction, and a. In Table III, as in Table II, one may notice the relatively greater influence of the variation of a as compared with the influence of the variation of k.

In dealing with case *III*, that of a wheel load at the edge, it was assumed that an equivalent radius b may be introduced in the place of the true radius a in the same manner as in the preceding case, and by the same formula, that of equation (8). This assumption may be justified on the ground of the similarity in the two cases in the distribution of the energy due to vertical shearing stresses. By introducing the equivalent radius b in the place of a in the formula for the tensile stress

σ_e along the bottom of the edge under the center of the circle, as obtained by the ordinary theory, one finds the following expression which, like the analogous equation (11), is based on $E = 3{,}000{,}000$ lb. per sq. in. and $\mu = 0.15$:

TABLE III

Stresses in pounds per square inch computed from equation (11) for load condition as in Case II, Figure 1, for different values of h, k, and a

$P = 10{,}000$ pounds, $E = 3{,}000{,}000$ pounds per square inch, $\mu = 0.15$

Thickness of slab h	Modulus of subgrade reaction k	Stress in slab				
		$a = 0$	$a = 2$ in.	$a = 4$ in.	$a = 6$ in.	$a = 8$ in.
Inches	*Lb./in.*^{3}	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*
4	50	1,231	1,058	848	693	588
	100	1,172	998	788	634	528
	200	1,112	939	729	574	469
5	50	763	694	580	487	415
	100	725	656	542	449	377
	200	687	617	504	411	339
6	50	523	487	421	361	313
	100	497	461	395	335	287
	200	470	435	368	308	260
7	50	380	360	319	279	245
	100	361	341	300	260	226
	200	341	321	280	240	206
8	50	288	276	250	222	197
	100	273	261	235	207	182
	200	258	246	220	192	167
9	50	226	218	200	180	162
	100	214	206	188	169	150
	200	202	194	177	157	138
10	50	181	176	164	149	136
	100	172	167	154	140	126
	200	162	157	145	130	116

$$\sigma_e = 0.572 \frac{P}{h^2}\left[\log_{10}(h^3) - 4 \log_{10}\left(\sqrt{1.6\, a^2 + h^2} - 0.675\, h\right) - \log_{10} k + 5.767\right] \qquad (12)$$

Stresses computed according to this formula are given in Table IV, again for $P = 10{,}000$ pounds. The influence of the three variables h, k, and a is shown in the same manner as in the two preceding tables, and is seen to be of the same nature, the variation of a being of greater importance than that of k.

TABLE IV

Stresses in pounds per square inch computed from equation (12) for load condition as in Case III, Figure 1, for different values of h, k, and a

$P = 10{,}000$ pounds, $E = 3{,}000{,}000$ pounds per squrae inch, $\mu = 0.15$

Thickness of slab h	Modulus of subgrade reaction k	Stress in slab				
		$a = 0$	$a = 2$ in.	$a = 4$ in.	$a = 6$ in.	$a = 8$ in.
Inches	*Lb./in.*^{3}	*Lbs. per sq. in.*	*Lbs. per sq. in*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*
6	50	833	769	649	541	453
	100	785	721	601	493	406
	200	738	673	553	445	358
7	50	604	568	494	422	360
	100	569	533	459	386	325
	200	534	498	424	351	290
8	50	457	436	388	337	293
	100	430	409	361	311	266
	200	404	382	334	284	239
9	50	358	344	312	276	243
	100	337	323	291	255	222
	200	315	301	269	233	200
10	50	287	278	256	230	204
	100	270	261	239	212	187
	200	253	244	221	195	170
11	50	235	229	213	194	174
	100	221	215	199	180	160
	200	207	201	185	165	146
12	50	196	192	180	165	150
	100	184	180	168	153	138
	200	172	168	156	142	126

BALANCED DESIGNS TESTED BY USE OF TABLES

From the three tables, for cases *I*, *II*, and *III*, one may obtain suggestions on the question of balanced design. Consider, for example, a pavement with the thicknesses 7 inches in the interior portion, and 9 inches at the edges. It may be assumed for the time being that the outer portions behave as a large slab with uniform thickness 9 inches. With the thickness diminishing slowly toward the interior, the stresses σ_c and σ_e would be somewhat larger than with constant thickness of 9 inches, but the correction needed for this reason is probably only small. For the time being only the one wheel load which is considered in each of the three tables will be taken into account. The influence of other wheel loads acting on the same panel, but at some distance, will be considered later; in any case it is found to be relatively small. With $P = 10{,}000$ pounds, $k = 50$ lb./in.^{3}, and $a = 4$ inches, the three tables give the following value:

$$\sigma_c = 262, \qquad \sigma_i = 319, \qquad \sigma_e = 312 \text{ lb. per sq. in.}$$

In comparing these stresses, their different characters should be considered. The stress σ_c at the corner acts presumably throughout the width of a whole cross-section, whereas σ_i and σ_e are localized within smaller regions. With equal tendency to rupture at the three places, σ_c, then, should be, probably, somewhat smaller than σ_i and σ_e. The stress σ_e is produced under the influence of a load which is distributed over an area only one-half of that assumed for σ_i. While the situation represented by the smaller area may occur when a wheel moves in over the edge of the pavement, it is reasonable, for the purpose of a comparative study of the tendency to rupture, to assume a larger radius of the semi-circle at the edge than for the full circle in the interior portion. With $a = 6$ in., for example, at the edge, one finds the stress

$$\sigma_e = 276 \text{ lb. per sq. in.}$$

In comparing this stress with σ_i, it should be observed that σ_i represents a state of equal stresses in all horizontal directions at the points, whereas σ_e is a one-directional stress. The elongations per unit of length are in the two cases $\sigma_i\,(1-\mu)/E$ and σ_e/E. It appears to be reasonable, therefore, for the purpose of comparison, to replace σ_i by an equivalent one-directional stress; if in this case the elongation is a direct measure of the tendency to rupture, this equivalent stress should be

$$\sigma'_i = \sigma_i(1-\mu) = 319(1-0.15) = 271 \text{ lb. per sq. in.}$$

The three values 262, 271, and 276 lb. per sq. in. point toward the conclusion that the assumed design is suitably balanced.

The suggestion has been made already that one may determine suitable values of k by comparing the deflections found by tests of full-s'zed slabs with those given by the formulas. The following formulas lend themselves to this purpose; they refer to the three cases shown in Fig. 1; in each case the load P is the only one acting:

Case I. Equation (3) gives the deflection at the corner:

$$z_c = \left(1.1 - 0.88\frac{a^1}{l}\right)\frac{P}{k\,l^2} \dots\dots\dots\dots\dots\dots (13)$$

Case II. The deflection under the center of the load differs only slightly from the following value which is accurate when $a=0$:

$$z_i = \frac{P}{8\,k\,l^2} \dots\dots\dots\dots\dots\dots\dots\dots (14)$$

Case III. The deflection at the point of application of a concentrated force P at the edge is approximately equal to

$$z_e = \frac{1}{\sqrt{6}}\left(1 + 0.4\,\mu\right)\frac{P}{k\,l^2} \dots\dots\dots\dots\dots\dots (15)$$

that is, for $\mu = 0.15$

$$z_e = 0.433\,\frac{P}{k\,l^2} \dots\dots\dots\dots\dots\dots\dots (16)$$

The quantity kl^2 occurring in each of these formulas may be expressed, according to equation (1), as

$$k\,l^2 = \sqrt{\frac{E\,h^3\,k}{12\,(1-\mu^2)}} \dots\dots\dots\dots\dots\dots (17)$$

When experimental values of the deflections are at hand, one may determine the corresponding values of kl^2 by means of equations (13) to (16). Then equation (17) gives the value of k as

$$k = \frac{12\,(1-\mu^2)\,(k\,l^2)^2}{E\,h^3} \dots\dots\dots\dots\dots\dots (18)$$

Figures 4 to 11 are diagrams of deflections and moments. The titles of these figures explain the nature of the diagrams. The deflections and bending moments have been computed by means of the ordinary theory of slabs. The diagrams, therefore, give information concerning deflections in general, and concerning bending moments except in the immediate neighborhood of the concentrated load which produces the bending moments.

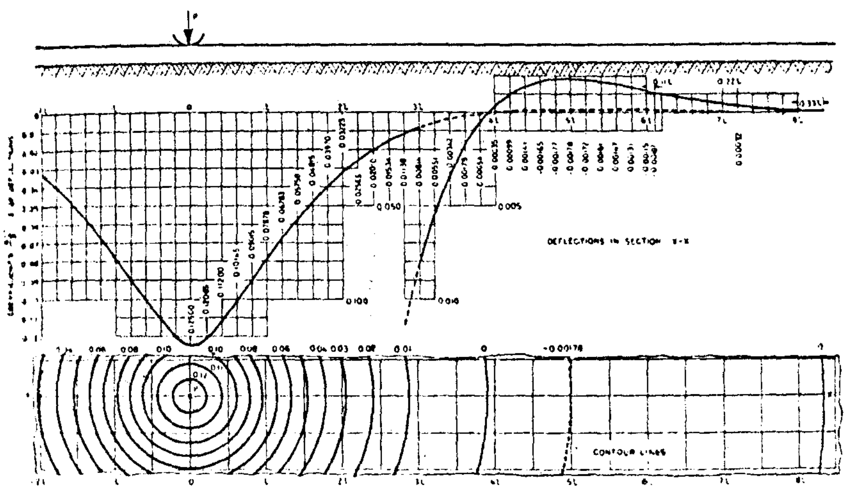


Figure 4—Deflections produced by a concentrated load which acts at a point of the interior at a considerable distance from the edges

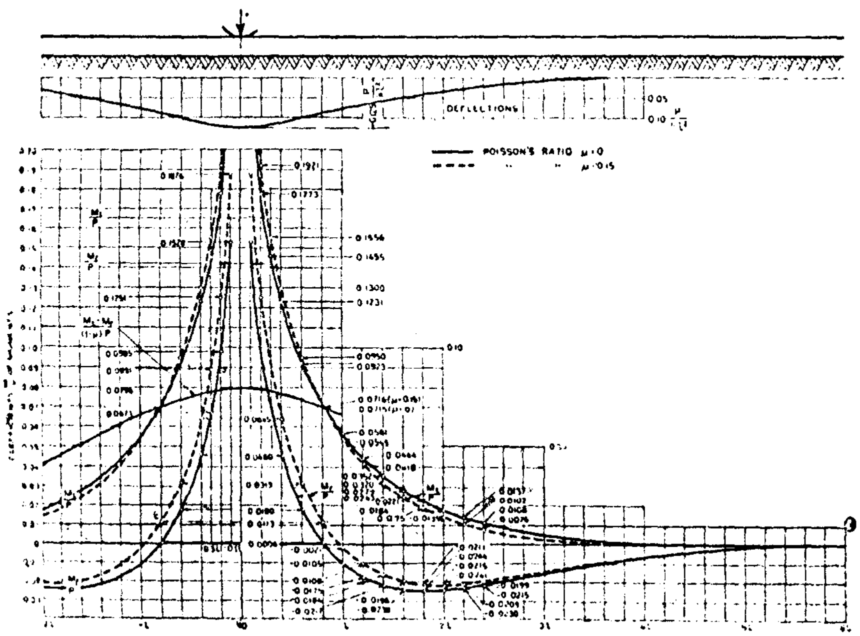


Figure 5—Tangential bending moments, M_t, and radial bending moment, M_r, produced by a concentrated load which acts at a point of the interior at a considerable distance from the edges.

DETERMINATION OF DEFLECTIONS DUE TO MORE THAN ONE WHEEL

The diagrams in Figures 4 and 5 have been obtained by an analysis which rests essentially on that given by the physicist Hertz[1] in 1884.

The diagrams in Figures 4 and 5 may be used in the following way, for the purpose of finding the resultant deflections and stresses due to the combined influence of two or four wheel loads each acting at a considerable distance from the edges of the slab.

Let each load be 10,000 pounds, and let the horizontal rectangular coordinates of the centers of the four loads be as follows:

Coordinate	Load No. 1	Load No. 2	Load No. 3	Load No. 4
$x =$	0	66 in.	0	66 in.
$y =$	0	0	66 in.	66 in.

Loads 1 and 2 alone may represent the two rear wheels of a four-wheel truck, and the four loads combined may represent the four rear wheels of a six-wheel truck.

With $h = 7$ in., $E = 3{,}000{,}000$ lb. per sq. in., $\mu = 0.15$, and $k = 50$ lb./in.^{3}, one finds by equations (1) and (17) or by Table I:

$$l = 36.40 \text{ in.}; \qquad kl^2 = 66{,}200 \text{ lb./in.};$$

distances 1-2 and 1-3: 66 in. $= 1.813l$; distance 1-4: $66\sqrt{2} = 2.564l$.

[1]H. Hertz, Über das Gleichgewicht schwimmender elastischer Platten, Wiedemann's Annalen der Physik und Chemie, v. 22, 1884, pp. 449–455; also in his Gesammelte Werke, v. 1, pp. 288–294. Hertz dealt with the problem of a large swimming slab, for example, of ice, loaded by a single force. A. Föppl in his Technische Mechanik, v. 5, 1907, pp. 112–130, presented Hertz's theory in a modified, and in some ways simplified form, and he called attention to the applicability of this analysis to the problem of the slab on elastic support. Hertz made use of Bessel functions in his analysis. Since his analysis was published, the number of published numerical tables of Bessel functions has been increased. Among the newer tables those representing Hankel's Bessel functions

$$H_0^{(1)}(x\sqrt{i}) \text{ and } H_1^{(1)}(x\sqrt{i})$$

are of especial interest for the present problem. Tables of these functions may be found in the book of tables by E. Jahnke and F. Emde, Funktionentafeln mit Formeln und Kurven, 1909, pp. 139 and 140. By means of these tables the numerical values given in Figures 4 and 5 were obtained by simple computations. After these diagrams had been prepared, two papers have appeared in which the same functions are used for the purpose of analysis of slabs on elastic support. One is by J. J. Koch, Berekening van vlakke platen, ondersteund in de hoekpunten van een willekeurig rooster, De Ingenieur, 1925, No. 6; the other is by Ferdinand Schleicher, Über Kreisplatten auf elastischer Unterlage, Festschrift zur Hundertjahrfeier der Technischen Hochschule Karlsruhe, 1925.

Thus one finds the stresses in the directions of x and y:

$$\sigma_x = -\frac{211}{8.167} = -26 \text{ lb. per sq. in.}$$

and

$$\sigma_y = \frac{181}{8.167} = 22 \text{ lb. per sq. in.}$$

These stresses are principal stresses, that is, one is the maximum, the other the minimum stress, and there are no shearing stresses in the directions of x and y.

For the case of the four-wheel truck, one finds, then, by super-position the following principal stresses due to the two rear wheels, loads No. 1 and No. 2; these principal stresses are in the directions of x and y:

$$\sigma_x = 279 - 26 = 253 \text{ lb. per sq. in.,}$$
$$\sigma_y = 279 + 22 = 301 \text{ lb. per sq. in.}$$

STRESSES DUE TO SIX-WHEEL TRUCK

In the case of the six-wheel truck the effects of loads No. 3 and No. 4 must be included. Load No. 3 contributes the same stresses at point 1 as does load No. 2, only the indices x and y are to be interchanged. Consequently the resultant stresses in the directions of x and y due to the combined influence of loads 1, 2, and 3 become

$$\sigma_x = \sigma_y = 279 - 26 + 22 = 275 \text{ lb. per sq. in.}$$

These stresses, again, are principal stresses. Since they are equal, the horizontal stresses will be the same in all directions, each stress being a principal stress

Let x^1, y^1 be a new system of horizontal rectangular coordinates with the axis of x^1 along the diagonal line from point 1 to point 4. Load No. 4 produces a radial bending moment in the direction of x^1 and a tangential bending moment in the direction of y^1. According to Fig. 5 these bending moments are

$$M_x{}^1 = -0.0186P = -186 \text{ lb. and } M_y{}^1 = 0.0058P = 58 \text{ lb.,}$$

respectively. The corresponding stresses are found, again, by dividing the bending moments by the section modulus per unit of width, that is, by 8.167 in.^{2}, and they are

$$\sigma_x{}^1 = -23 \text{ lb. per sq. in., and } \sigma_y{}^1 = 7 \text{ lb. per sq. in.}$$

These stresses are principal stresses. The resultant principal stresses due to all four loads combined, therefore, are in the directions of x^1 and y^1, and have the values

$$\sigma_x{}^1 = 275 - 23 = 252 \text{ lb. per sq. in.,}$$
$$\sigma_y{}^1 = 275 + 7 = 282 \text{ lb. per sq. in.}$$

Then equation (14) as well as Fig. 4, gives the following value of the deflection at point 1 due to load No. 1:

$$z_{1,\,1} = \frac{P}{8\,k\,l^2} = \frac{10{,}000}{8 \times 66{,}200} = 0.0189 \text{ in.}$$

Furthermore, Fig. 4 leads to the following value of the deflection at point 1 due to load No. 2 alone:

$$z_{1,\,2} = 0.03921\,\frac{P}{k\,l^2} = 0.03921\,\frac{10{,}000}{66{,}200} = 0.0059 \text{ in.}$$

Then, by superposition of the two deflections, one finds the deflection at point 1 due to the combined influence of the two rear wheels 1 and 2.

$$z_{1,\,(1,2)} = z_{1,\,1} + z_{1,\,2} = 0.0248 \text{ in.}$$

The deflection at point 1 due to load No. 3 alone is

$$z_{1,\,3} = z_{1,\,2} = 0.0059 \text{ in.}$$

The deflection at point 1 due to load No. 4 alone is, according to Fig. 4,

$$z_{1,\,4} = 0.01620\,\frac{P}{k\,l^2} = 0.0024 \text{ in.}$$

By superposition of the four deflections due to each separate load, one finds the resultant deflection due to the four loads:

$$z_{1,\,(1,\,2,\,3,\,4)} = 0.0331 \text{ in.}$$

For the purpose of computing the state of stresses at the bottom of the slab under the center of load No. 1 it will be assumed that load No. 1 is distributed uniformly over the area of a circle with a radius $a = 6$ inches. The stresses due to load No. 1 will be the same in all directions, and they are, according to Table 3:

$$\sigma_x = \sigma_y = 279 \text{ lb. per sq. in.}$$

According to Fig. 5, load No. 2 produces a radial bending moment M_r, in this case in the direction of x, equal to

$$M_x = -0.0211P = -211 \text{ in. lb. per in. (or } -211 \text{ lb.)},$$

and a tangential bending moment M_t, in this case in the direction of y, equal to

$$M_y = 0.0181P = 181 \text{ lb.}$$

The corresponding stresses are found by dividing these bending moments by the section modulus per unit of width, that is, by $\frac{1}{6}h^2 = 8.167 \text{ in}^2$.

One may draw the conclusion that the main part of the state of stresses at a given point is due to a wheel load right over the point. In the case examined, the contribution due to the three additional rear wheels of the six-wheel truck is of less importance than that due to the one additional rear wheel of the four-wheel truck.

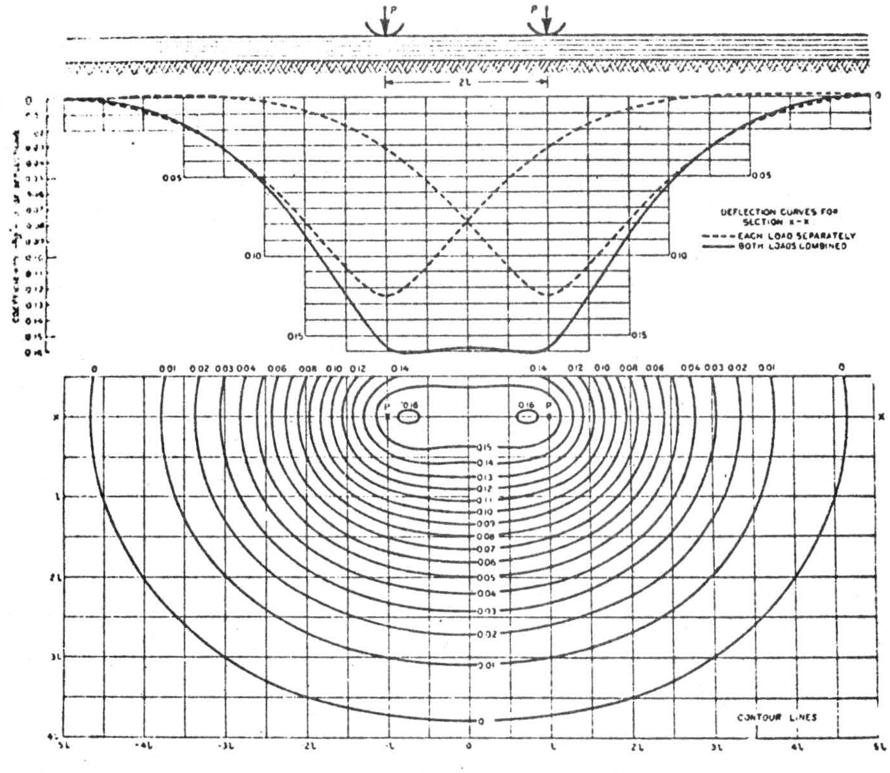


Figure 6—Deflections produced by two equal loads like the load in Figure 4, separated by a distance of 2l. The deflections are found by superposition of two diagrams of the kind shown in Figure 4

Figures 6 and 7 show deflections due to two wheel loads combined. Each of these diagrams was obtained by superposition of two diagrams such as shown in Figure 4.

Figures 8 to 11 show effects of loads at the edge, but at a considerable distance from any corner.[1]

By virtue of Maxwell's theorem of reciprocal deflections, the deflection at a point B of any slab due to a load P at the point A is the same as the deflection at A due to a load P at point B. Figures 8 and 9 may

[1]The theory by which these diagrams were obtained may be found in a paper by the writer: Om Beregning af Plader paa elastisk Underlag med særligt Henblik paa Spørgsmaalet om Spændinger i Betonveje, Ingeniören (Copenhagen), v. 32, 1923, pp. 513–524. See also, A. Nádai, Die elastischen Platten, (Berlin) 1925, p. 186.

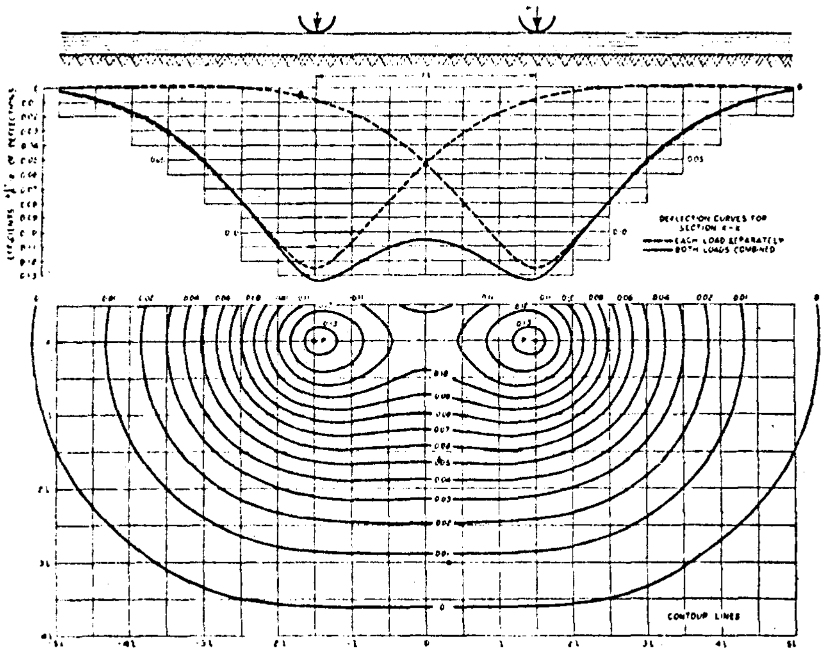


Figure 7—Deflections produced by two equal loads like the loads in Figure 4, separated by a distance of 3l

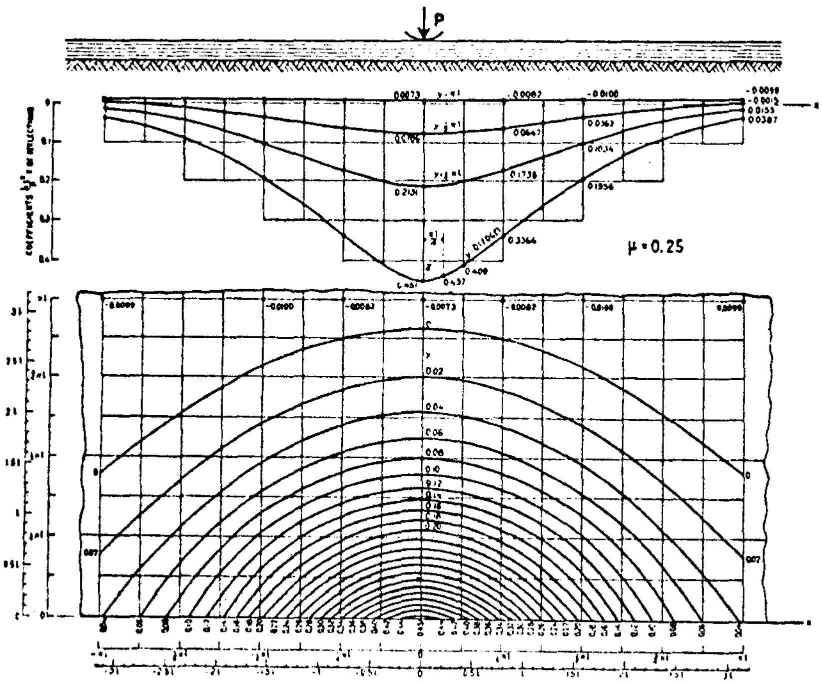


Figure 8—Deflections produced by a concentrated load at the edge at a considerable distance from any corner for $\mu = 0.25$

be interpreted, therefore, in a double manner: first, as diagrams of deflections at any point B due to a load P at the particular point A at the edge; secondly, as influence diagrams, showing the deflection at the particular point A at the edge due to a load P at any point.

From this reciprocity of deflections one may draw a further conclusion which may be applied to Figures 8 and 9, and which concerns the curve of deflections or elastic curve which is obtained by intersection of the deflected middle surface by a vertical plane. Two lines L_A and L_B are drawn parallel to two opposite parallel edges of a slab. Two equal loads are considered, one acting at a point A of the line L_A, the

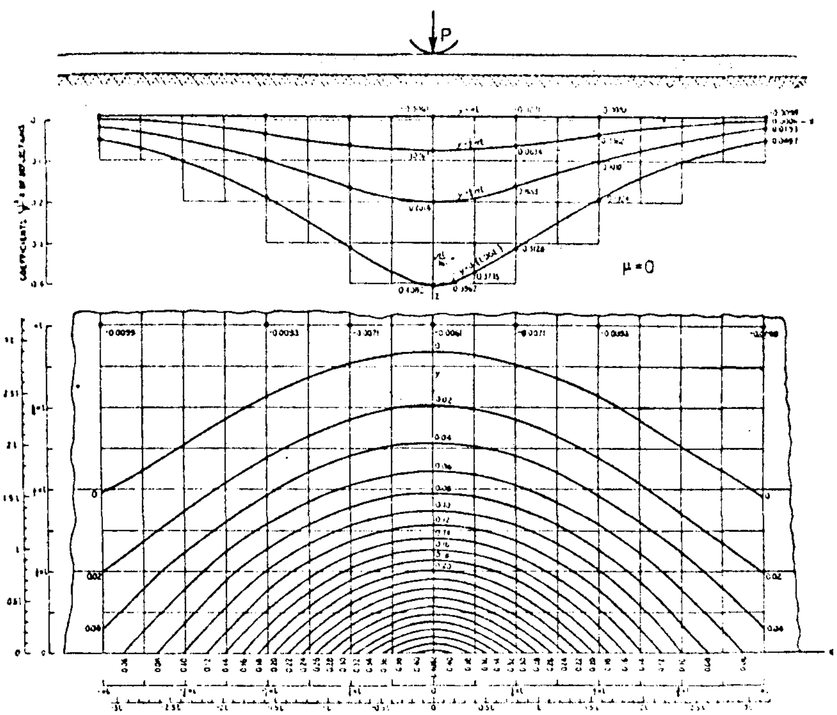


Figure 9—Deflections produced by a concentrated load at a considerable distance from any corner for $\mu = 0$

other acting at a point B of the line L_B. The points A and B are assumed to be sufficiently far from the remaining two edges of the slab to permit the assumption of zero deformations at these edges. Then one may conclude that the elastic curve produced along the line L_B under the influence of the load P at A has exactly the same shape as the elastic curve produced along the line L_A under the influence of the load P at point B. In applying this conclusion to Figure 8 or Figure 9, let the line L_A be the edge shown in the drawing, and let the line L_B be at some distance from the edge. By the direct use of the diagrams one obtains the elastic curve at any line L_B parallel to the edge, due to a load at the edge. But one may interpret this curve as the elastic curve for the edge produced under the influence of a load at a point of the line L_B. The curvature of the deflected middle surface at point A of the edge in the direction of the edge, produced by the load P at any point B at some distance from the edge, is the same, accordingly, as the

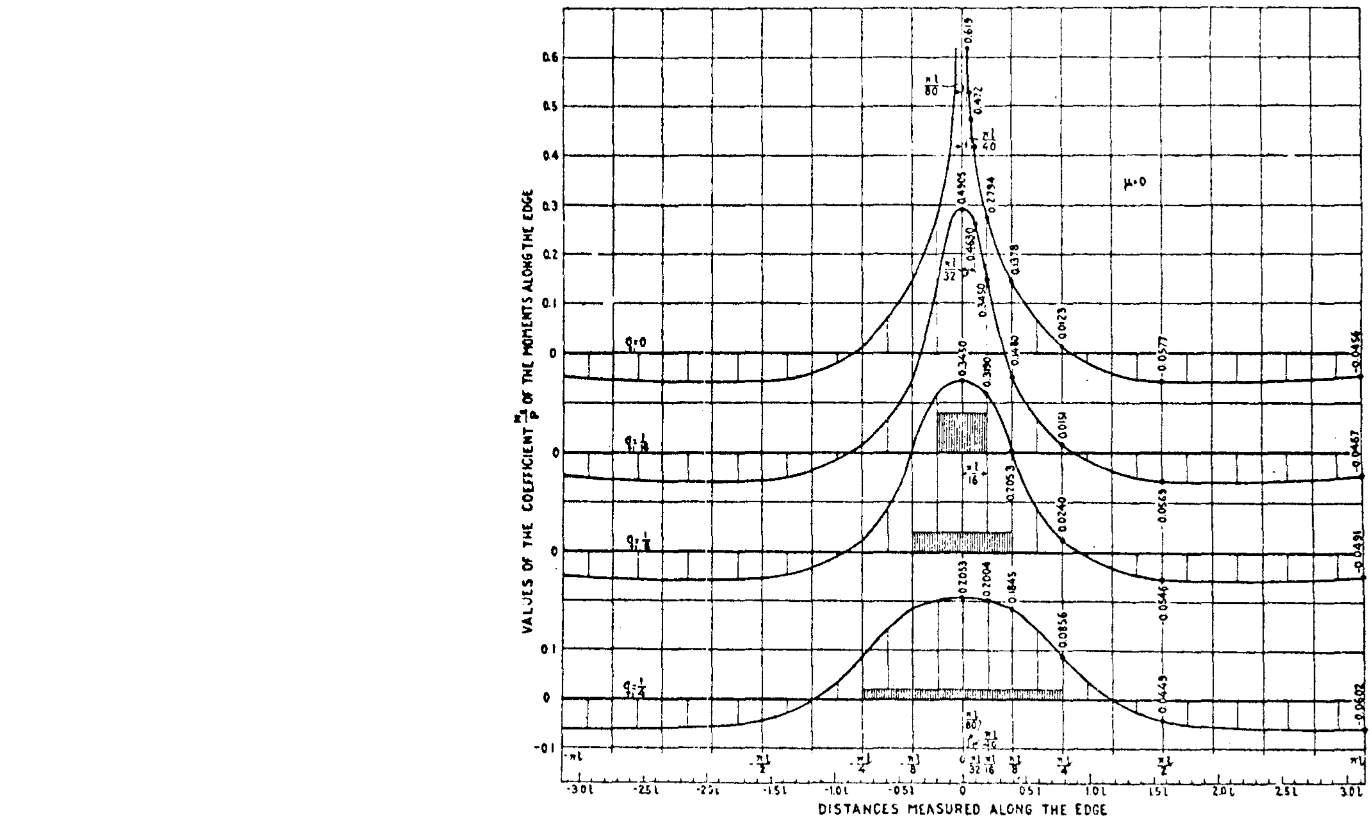


Figure 10—Bending moments along the edge for a load concentrated at a point of the edge (top diagram), and for loads distributed uniformly over lines of three lengths at the edge (lower three diagrams), $\mu = 0$

curvature of the deflected middle surface at point B in a direction parallel to the edge, as obtained in Figure 8 or Figure 9, due to the load P at the point A of the edge.

Thus Figures 8 and 9 may be used in studying the stresses produced along the edge by a wheel load at some distance from the edge.

The following use of the tables and diagrams is suggested. Let it be assumed that a certain pavement has been proved by tests and experience to be satisfactory for a given type of traffic. By the tables and diagrams one may compute, then, the corresponding critical stresses. These stresses may be adopted for the time being as allowable working stresses. With the stresses given, the tables and diagrams, through computations of the kind which has been shown, furnish answers to two questions: what additional thicknesses are required if the wheel pressures are increased in a given manner; and, what may be saved in the thicknesses by eliminating some of the heaviest vehicles.

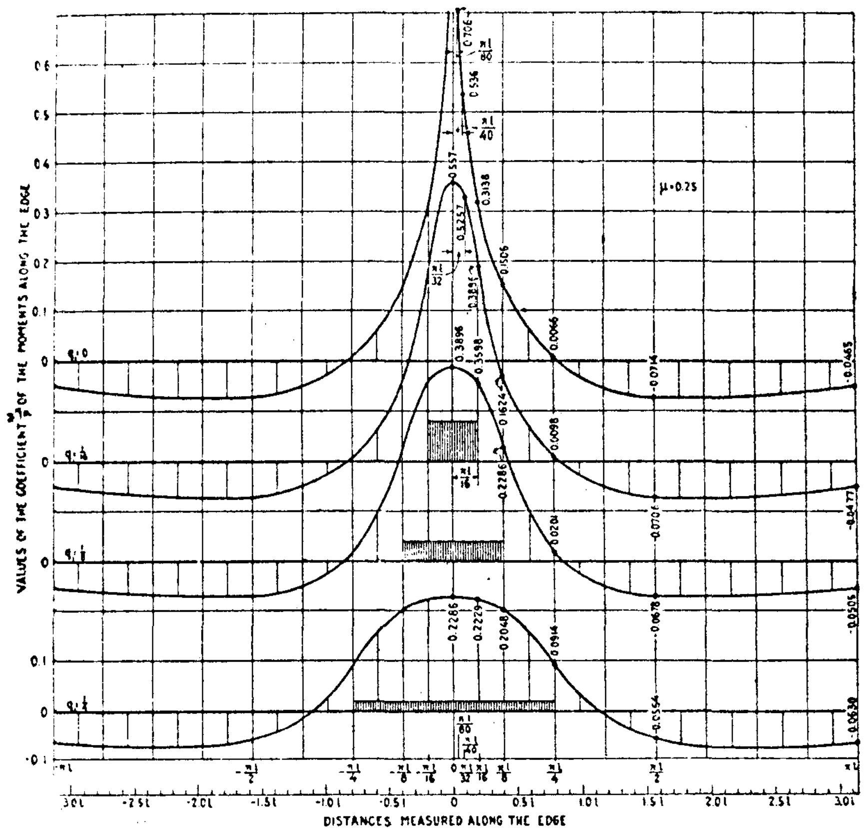


Figure 11—Bending moments along the edge as in Figure 10, but for $\mu = 0.25$

Professor T. R. Agg has called attention to the importance of having an answer to the latter question, when one attempts to apportion the cost of the pavement to the various kinds of traffic for which it is used.

In using the tables and diagrams it should be kept in mind that the analysis is based on those assumptions which were stated at the beginning of this discussion. By the nature of these assumptions certain influences were left out of consideration, especially the following: (1) variations of temperature, and other causes for tendency to change of volume; (2) the gradual diminishing of the thickness from the edge toward the interior; (3) local soft or hard spots in the subgrade; (4) horizontal components of the reactions of the subgrade; and (5) the dynamic effect, expressed in terms of the inertia of the pavement and subgrade. The horizontal components of the reactions of the subgrade, which are due to friction, may have a strengthening influence, especially at some distance from the edges, by causing a dome action in the pavement. As to the dynamic effects, with known values of the maximum pressure developed between the tire and the pavement, the effect of the inertia of the pavement may possibly be expressed approximately in terms of an increased value of the modulus k. These additional influences are suitable subjects for further analysis.

REPORT ON EXPERIMENTS ON EXTENSIBILITY OF CONCRETE

W. K. Hatt
Purdue University, Lafayette, Indiana

Two properties of materials are important—strength and toughness. Available data are few resulting from measurements of the ability of concrete to withstand extension without the appearance of fissures. These may range in magnitude: (a) from those in the order of 0.0004 inch width seen only with a microscope or appearing as "water veins" or "water marks," as Feret termed them, when a skin-dried surface breaks and capillary moisture comes from the interior through the fissures; to (b) larger fissures in the order of 0.0015-inch width, seen by the unaided eye; and (c) in the extreme to those large open cracks that occur when the elastic limit of reinforcing steel is exceeded. In the class of microscopic fissures are those crazes that mar the appearance of architectural concrete or other concrete products. Such crazes are not always evident to the unaided eye, but may be developed by a coating of light oil.

The various fissures may be produced by load or by the action of temperature or moisture changes.

The preservation of the integrity of the surface of exposed concrete is important. In many cases surface cracks are the first indication of subsequent failure in concretes that have been made of defective materials, either cement or aggregate.

We are increasingly required to compute expansions and contractions of structures; these movements are limited by extensibility.

As has been said, the active agents may be tensions due to loads, or due to the working back and forth of the surface under temperature and moisture changes. The latter express themselves most markedly when the surface of the concrete is of a richer composition than the interior, or when the surface is contracted by careless drying against a moist core. Indeed, the falling off in strength of cement briquettes

LESSON OUTLINE
SUBGRADE CHARACTERIZATION

Instructional Objectives

1. To provide the student with a basic knowledge of the tests most commonly used to characterize subgrade materials.

2. To acquaint the student with the limitations of each test in its role in pavement design.

Performance Objectives

1. The student should be able to sketch a simple representation of each test apparatus and the appropriate results.

2. The student should be able to state the advantages and disadvantages of each test in terms of the pavement design process.

Abbreviated Summary	Time Allocations, min.
1. Introduction to Materials Characterization	10
2. Plate Tests	5
3. California Bearing Ratio (CBR)	10
4. Triaxial Test	10
5. Resilient Modulus Test	5
6. R-Value Test	5
7. Group Index	5
	50 minutes

Reading Assignment

1. Yoder and Witczak - Chapter 8, pages 243-265
2. Instructional Text

Additional Reading

1. TRB Special Report 162, "Test Procedures for Characterizing Dynamic Stress-Strain Properties of Pavement Materials.

LESSON OUTLINE
SUBGRADE CHARACTERIZATION

1.0 INTRODUCTION TO MATERIALS CHARACTERIZATION

1.1 Renewed Interest

In recent years there has been a renewed interest in materials characterization due to the increasing use of marginal materials. We usually try to extrapolate old design procedures using quality materials to design procedures using marginal materials.

1.2 Variability of Materials

Variability due to the inherent nature of the material (non-homogeneous), moisture, temperature, particle shape, and particle surface texture among others.

1.3 Types of Tests

(a) Routine Tests. Such as plate loading, triaxial and CBR test.

(b) Layered Input Parameter. Such as resilient modulus test.

(c) Fundamental Distress. Such as fatigue and permanent deformation test.

1.4 Past - Static or Low Strain Rate

1.5 Present - Dynamic

Theoretical design procedures.

1.6 Future

Need to develop tests which recognize the variability of materials, relate performance, and relate actual engineering properties to use with elastic layered theory to develop design procedures.

1.7 Factors Used to Determine Test to Use

Such as E and μ for layered theory.

(a) Ease of Testing. If test is complicated it is not apt to be used on a routine basis. Complicated tests cost more due to equipment and trained personnel.

(b) Reproduceability of Results. Variability of materials, equipment and operators affect results.

(c) Size of Project and Within Project Variation. The larger the project, the more testing is typically justified. Variability along roadway becomes a consideration.

(d) Measurement of Fundamental Properties. Empirical tests are usually only good for empirical design procedures. Estimating fundamental properties from empirical tests should be avoided.

2.0 PLATE TESTS

2.1 Purpose

Measures the supporting power of materials.

2.2 Test Apparatus (Visual Aid 14.1)

2.3 Effect of Plate Size (Visual Aid 14.2)

Use 30" plate for rigid pavements and wheel area plate for flexible pavements.

2.4 Modulus of Subgrade Reaction (k)

$K_u = \frac{P}{\Delta}$ (Visual Aid 14.3)

P = unit load on plate, psi

Δ = deflection of plate, in. (sometimes P is taken as that pressure corresponding to a deflection of 0.05 in.)

(a) Correction for Service Condition.

$$K = \frac{d}{d_s} K_u$$

d = laboratory deformation under field conditions

d_s = laboratory deformation under saturated conditions

(b) Correction for Plate Bending. As shown in Visual Aid 14.4.

2.5 Test Method

(a) Load increment less than 10 percent of maximum wheel load.

(b) Maintain each increment until settlement is less than 0.002 in./min.

(c) Load until maximum load is reached.

(d) Unload using same increments.

2.6 Variations of Test

(a) Cyclic load (Visual Aid 14.5)

(b) Repeated load (Visual Aid 14.6)

2.7 Advantages

(a) Test performed on actual in-place material

(b) Experience (used by many agencies)

2.8 Disadvantages

(a) What to jack against

(b) Bending of reaction beam

(c) Gauges outside influence area

(d) Limited number of tests

3.0 CALIFORNIA BEARING RATIO (CBR)

3.1 Purpose

Measures the resistance to penetration of a subgrade.

3.2 Test Apparatus (Visual Aid 14.7)

3.3 Soil Factors Affecting Test

(a) Soil Texture.

(1) Granular soils - not affected by swelling during soaking period; therefore surcharge weight not important during soaking.

(2) Clayey soils - greatly affected by swelling pressures and thus CBR value highly dependent upon surcharge weight during soaking.

3.3.2 Soil Moisture and Density.

(1) Granular soils - compacted at optimum moisture content with three levels of compactive effort.

(2) Clayey soils - compacted at varying moisture and density conditions.

3.4 Test Method

(a) Compaction depending on soil type

(b) Soaking (swell reading)

(c) Perform penetration

3.5 Test Results (Visual Aid 14.8)

$$CBR = \frac{\text{unit load at .1 in}}{\text{unit load of standard}^{*}} = \frac{\text{unit load}}{\text{1000 psi}}$$

* standard is a high quality crushed stone that has a unit load of 1000 psi at .1 in. deflection.

note: Usually CBR decreases as penetration increases; however, if $CBR_{.2} > CBR_{.1}$ then use $CBR_{.2}$

3.6 Variations of Test

(a) Field CBR Tests. Correlations with laboratory results may be erratic particularly for granular soils.

(b) Undisturbed Samples.

3.7 Advantages

(a) Fast

(b) Experience

3.8 Disadvantages

(a) Information not very useful

(b) Doesn't simulate actual load conditions

(c) Affected by piece of aggregate under rear

(d) Doesn't simulate shearing forces present in flexible pavement

4.0 TRIAXIAL TESTS

4.1 Purpose

Measures the shear strength of a subgrade material under lateral pressure. Attempts to simulate stress conditions existing in field.

4.2 Test Apparatus (Visual Aid 14.9)

4.3 Test Theory

(a) Coulomb Equation. (Visual Aid 14.10)

$$S = C + \sigma \tan \phi$$

Where: S = internal stability

C = cohesion

σ = applied stress

ϕ = angle of internal friction

for clays - $S = C$ (Visual Aid 14.11)

for sands - $S = \sigma \tan \phi$ (Visual Aid 14.12)

(b) Assumption. Internal resistance is dependent upon shearing resistance due to internal friction and cohesion which may be expressed as a single shearing stress component.

5.0 RESILIENT MODULUS TEST

5.1 Purpose (Visual Aid 14.13 a-e)

Measures the modulus of subgrade materials in terms of the recoverable deformation response to a dynamic load.

5.2 Test Apparatus (Visual Aid 14.14)

5.3 General Equation

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

Where: M_R = modulus of resilient deformation

σ_d = repeated deviator stress (stress difference)

ε_a = repeated recoverable strain

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

5.4 Effect of Material Type

(a) Cohesive soils (Visual Aid 14.15)

(b) Granular materials (Visual Aid 14.16)

6.0 R - VALUE TEST

6.1 Purpose

Measures the tendencies of subgrade soils to resist deformation when loaded in a triaxial state.

6.2 Test Apparatus (Visual Aid 14.17)

6.3 R - Value Calculations

$$R = 100 - \frac{100}{\frac{2.5}{D}\left(\frac{P_v}{P_h} - 1\right) + 1}$$

Where: R = resistance value

P_v = vertical pressure (160 psi)

D = turns displacement reading (about 2-5)

P_h = horizontal pressure at P_v = 160 psi

* Note R (fluid) = where $P_v = P_h$

R (rigid solid) = 100 where $P_h = 0$

6.4 Test Method

(a) Apply load at 0.05 in./min.

(b) Road P_h at P_v = 160 psi

(c) Remove half of vertical load

(d) Reduce P_h to 5 psi

(e) Count number of turns to get P_h = 100 psi

7.0 GROUP INDEX

7.1 Purpose

Relate quality of soil for highway uses to gradation and plastic limits.

7.2 Group Index Formula

GI = 0.2a + 0.005ac + 0.01bd

Where: GI = group index

a = that portion of the percentage passing No. 200 sieve greater than 35 percent and not exceeding 75 percent, expressed as a positive whole number (0 to 40).

b = that portion of the percentage passing No. 200 sieve greater than 15 percent and not exceeding 55 percent, expressed as a positive whole number (0 to 40).

c = that portion of the numerical liquid limit greater than 40 and not exceeding 60, expressed as a positive whole number (0 to 20).

d = that portion of the numerical plasticity index greater than 10 and not exceeding 30, expressed as a positive whole number (0 to 20).

7.3 Based on AASHTO Classification System (Visual Aid 14.18)

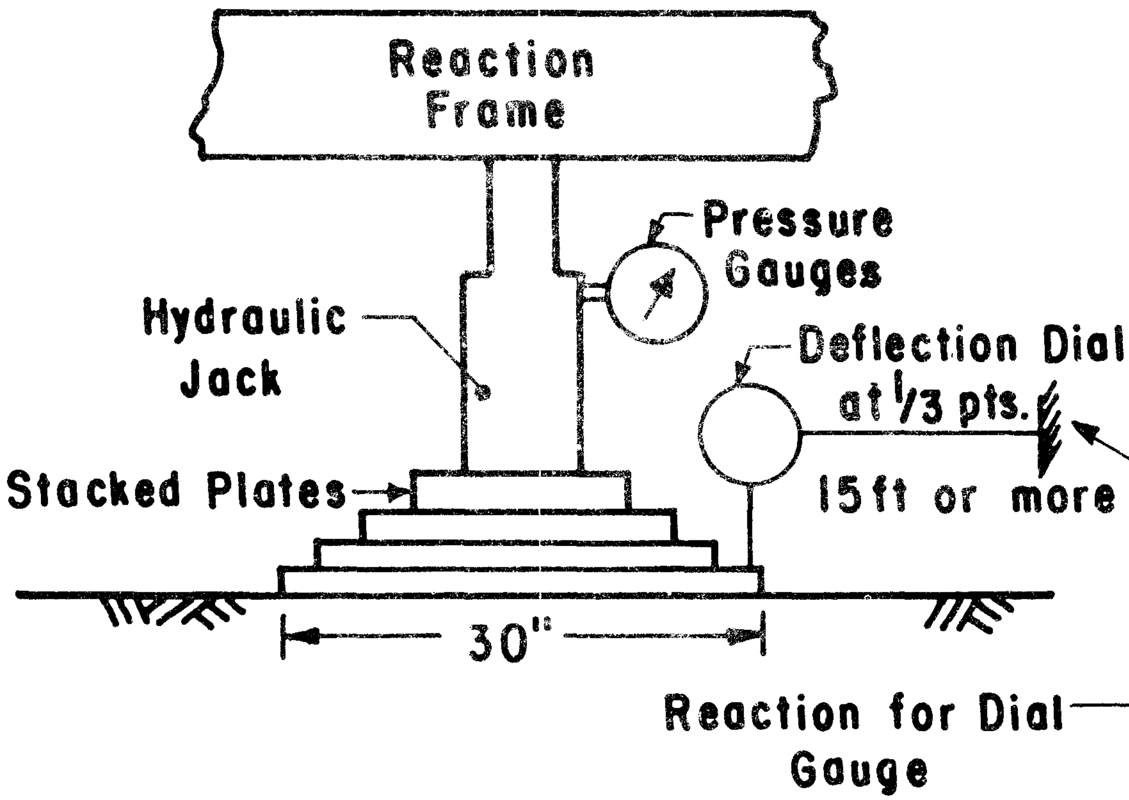
LESSON OUTLINE
SUBGRADE CHARACTERIZATION

VISUAL AID	TITLE
Visual Aid 14.1.	Plate load test apparatus.
Visual Aid 14.2.	Effect of plate size.
Visual Aid 14.3.	Determination of modulus of subgrade reaction.
Visual Aid 14.4.	Correction for plate bending (Corps of Engineers).
Visual Aid 14.5.	Cyclic plate load test results.
Visual Aid 14.6.	Repeated load plate test results.
Visual Aid 14.7.	CBR test apparatus.
Visual Aid 14.8.	CBR test results.
Visual Aid 14.9.	Schematic diagram of a triaxial cell.
Visual Aid 14.10.	Internal stability of soil represented by Coulomb equation.
Visual Aid 14.11.	Internal stability for cohesive soils.
Visual Aid 14.12.	Internal stability for cohesive soils.
Visual Aid 14.13(a).	Behavior of sample under all-around compression.
Visual Aid 14.13(b).	Volume change as function of time when drainage is permitted.
Visual Aid 14.13(c).	Volume change as function of time when drainage is permitted.
Visual Aid 14.13(d).	Porewater pressure as function of applied pressure if drainage is prevented.
Visual Aid 14.13(e).	Volume change of undrained sample as function of time for different initial degrees of saturation.
Visual Aid 14.14.	Apparatus for resilient testing of subgrade materials.
Visual Aid 14.15.	Typical resilient modulus response for cohesive soils.
Visual Aid 14.16.	Typical resilient modulus response for granular soils.

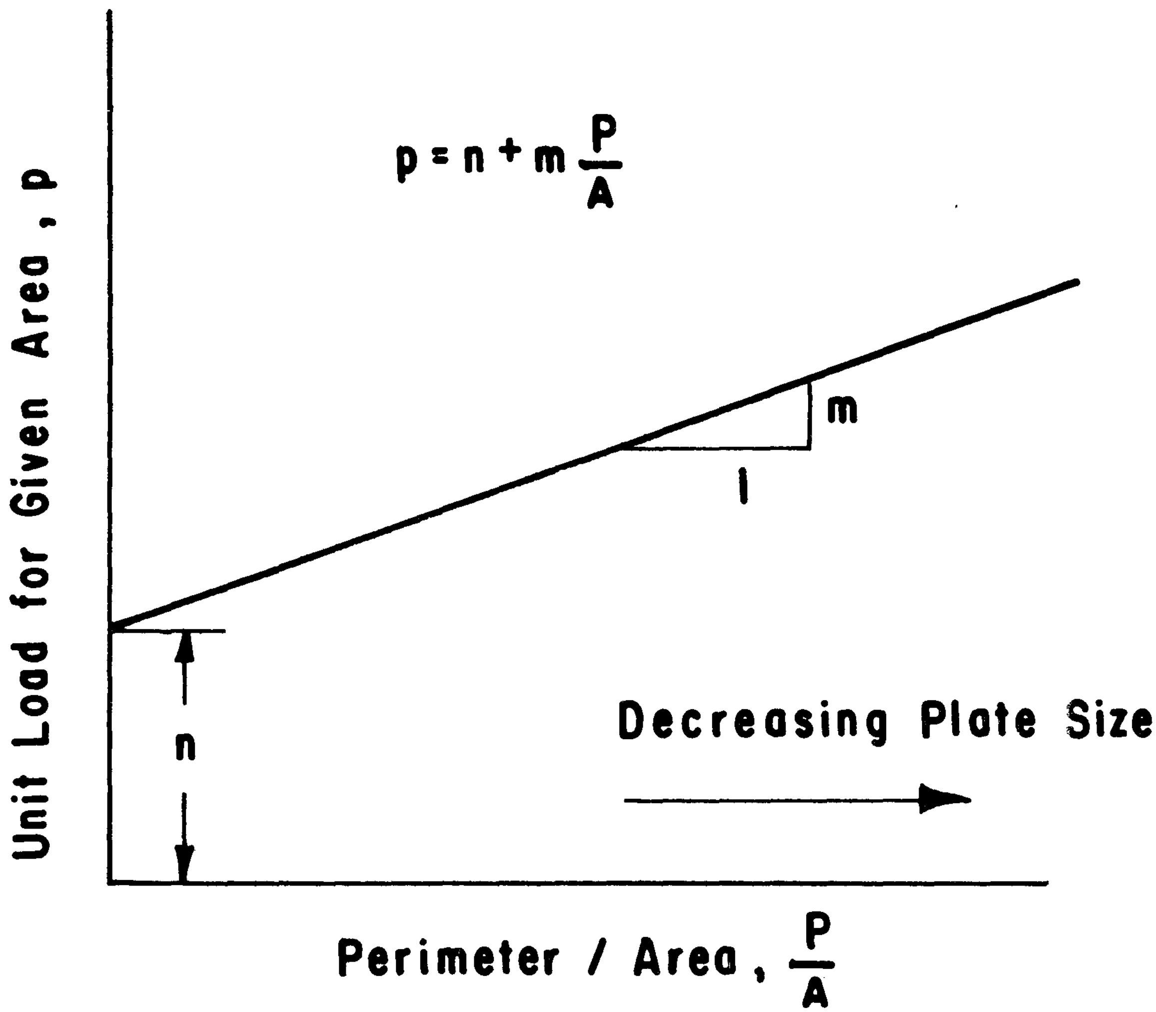
LESSON OUTLINE
SUBGRADE CHARACTERIZATION

VISUAL AID	TITLE
Visual Aid 14.17.	Stabilometer for determinating R-value.
Visual Aid 14.18.	AASHO soil classification.

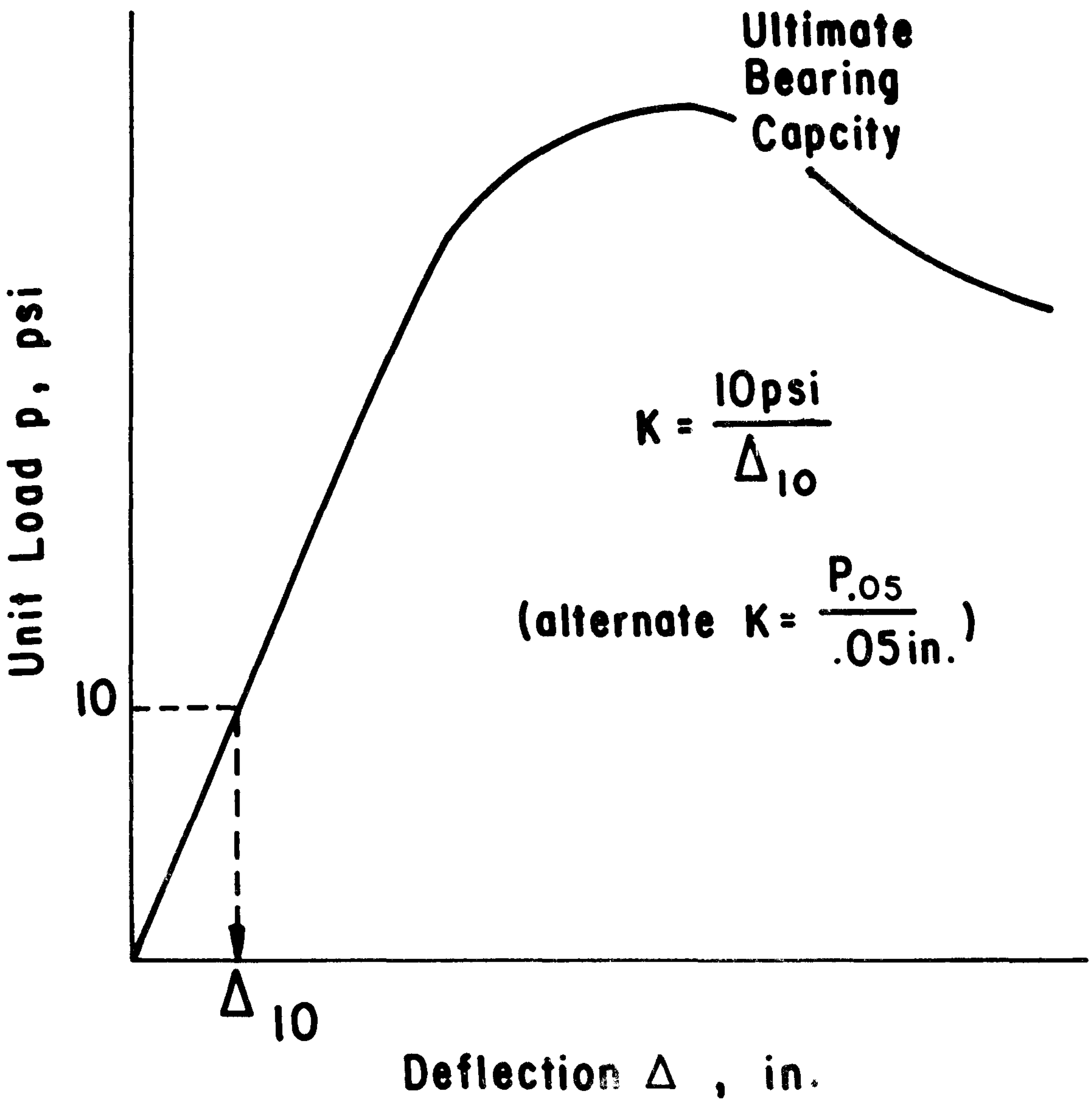
Visual Aid 14.1. Plate load test apparatus.



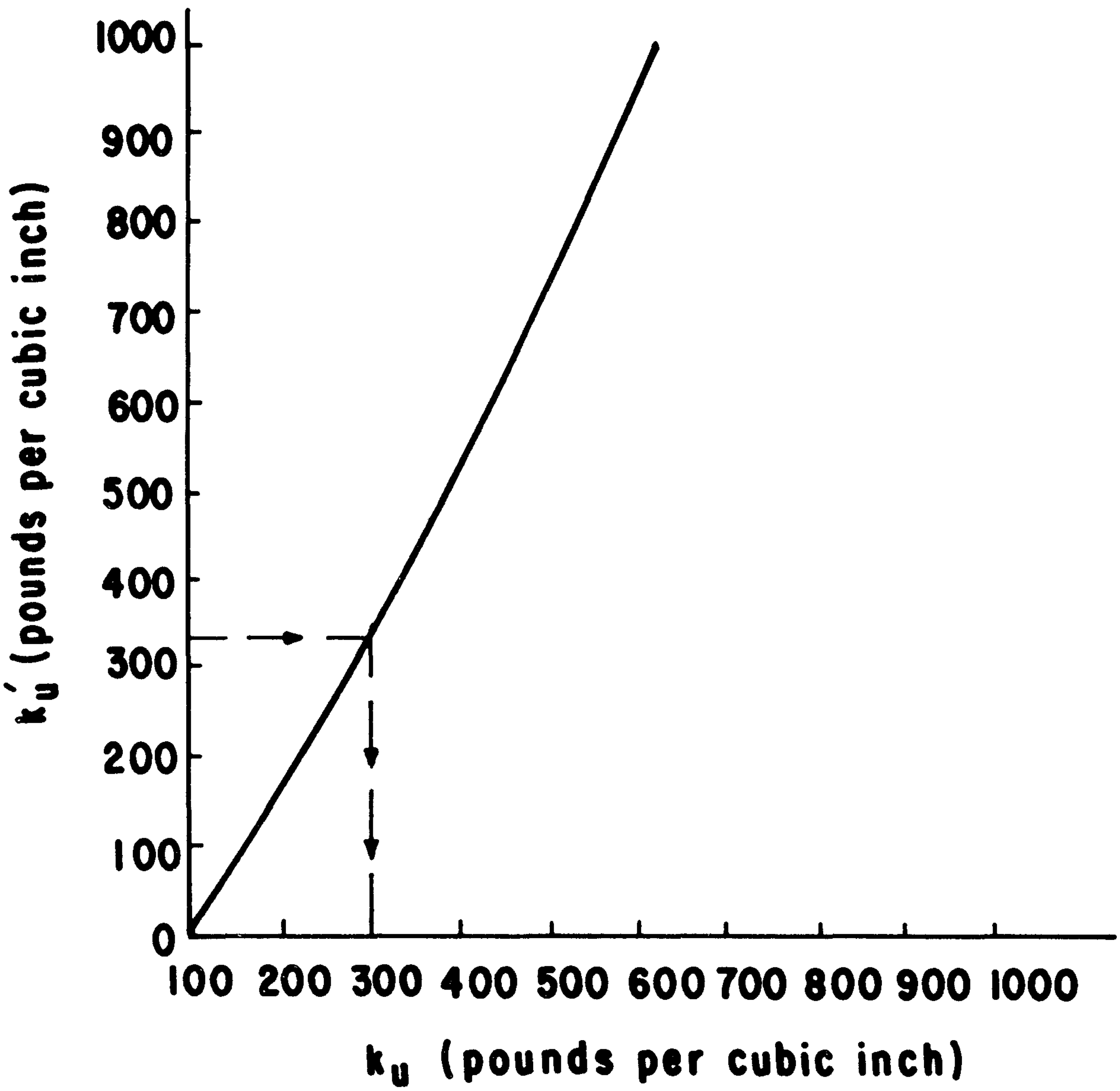
Visual Aid 14.2. Effect of plate size.



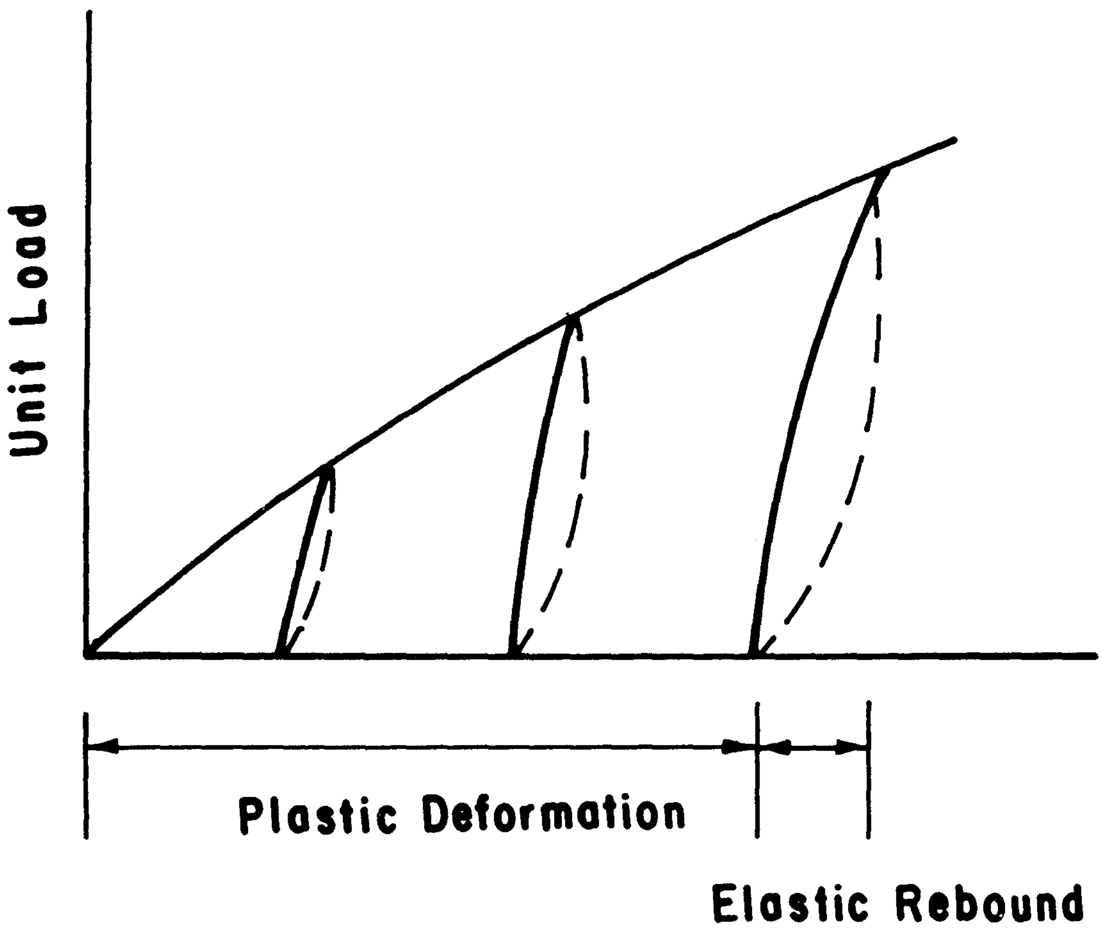
Visual Aid 14.3. Determination of modulus of subgrade reaction.



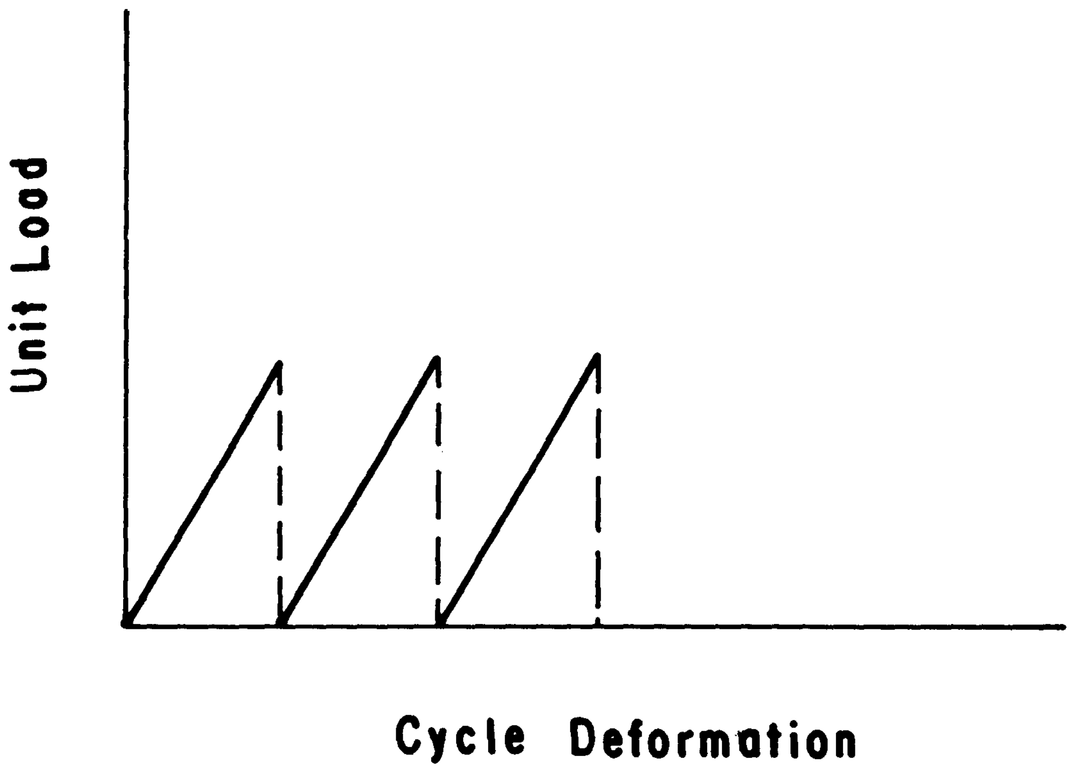
Visual Aid 14.4. Correction for plate bending (Corps of Engineers).



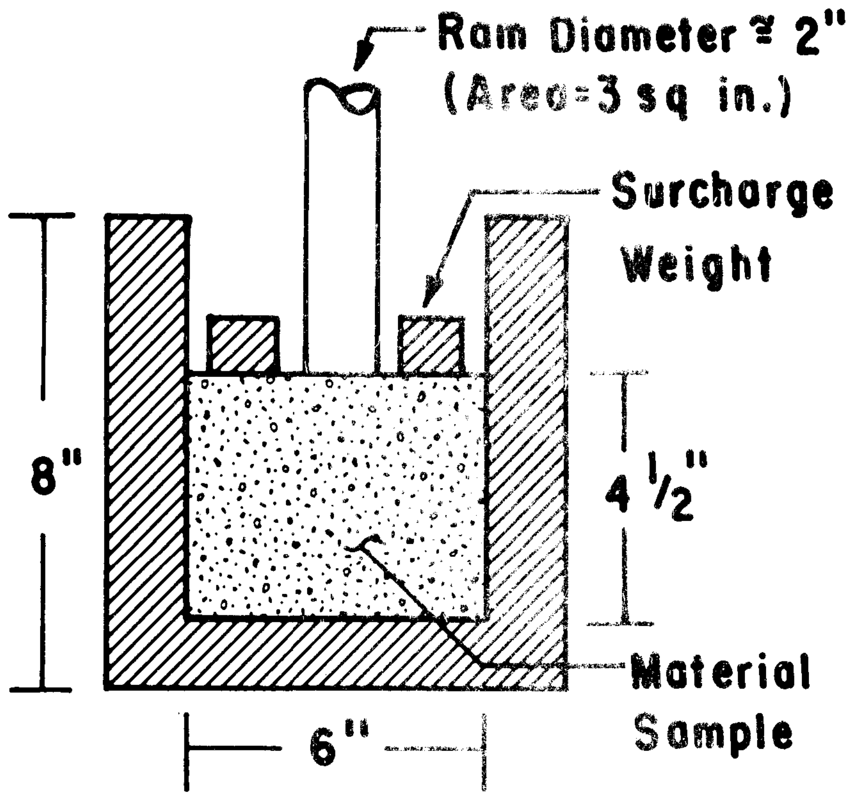
Visual Aid 14.5. Cyclic plate load test results.



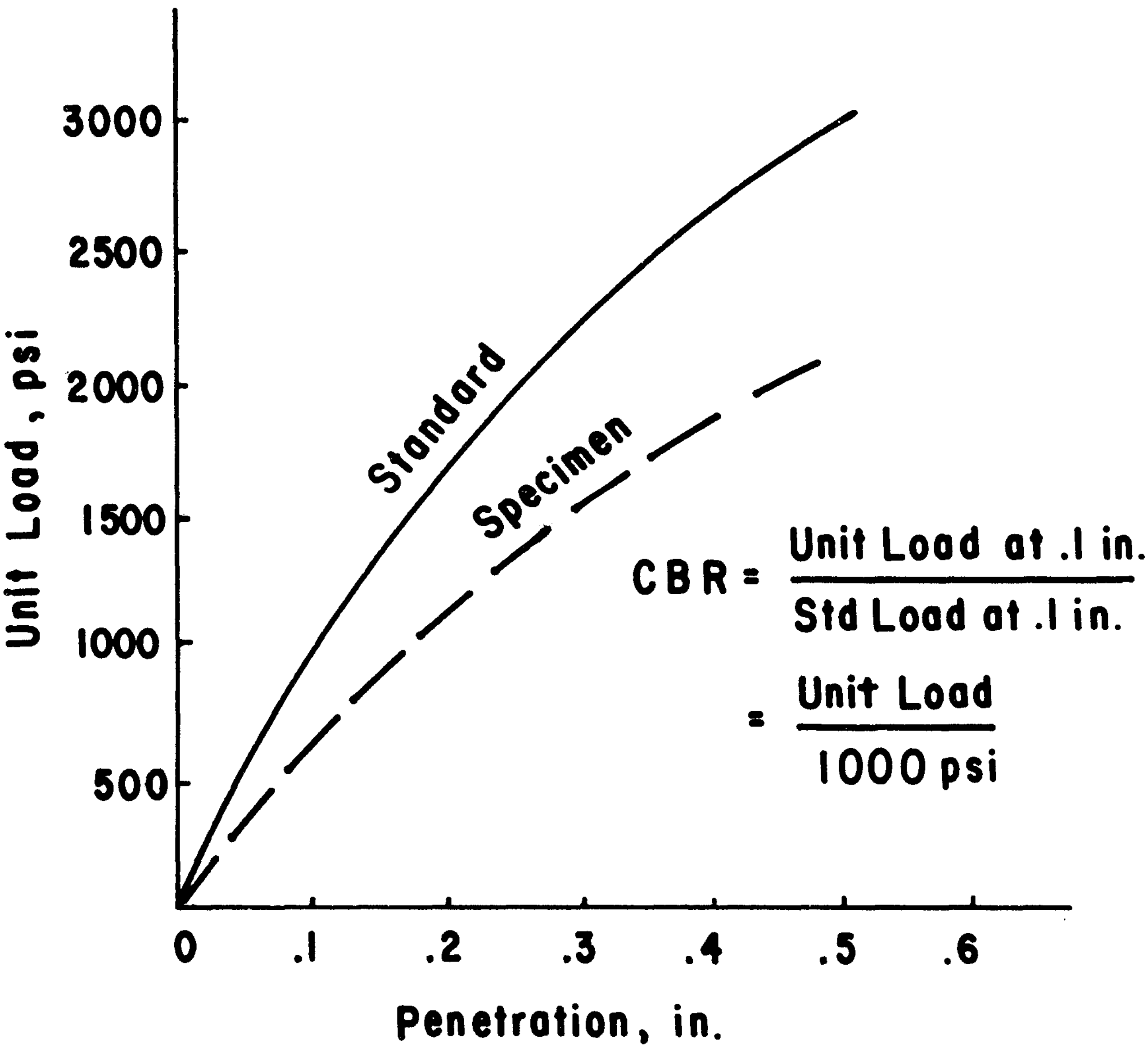
Visual Aid 14.6. Repeated load plate test results.



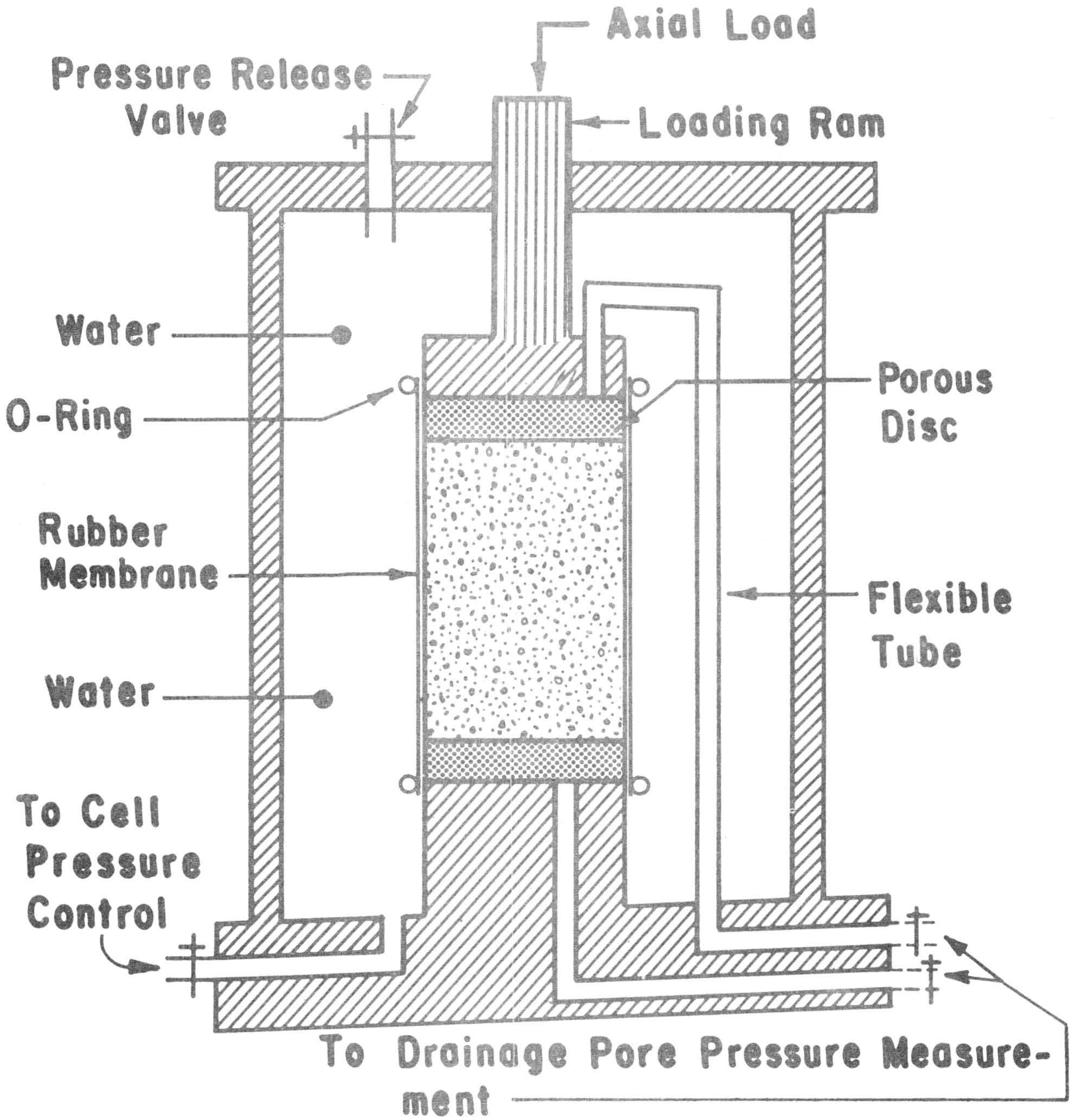
Visual Aid 14.7. CBR Test Apparatus



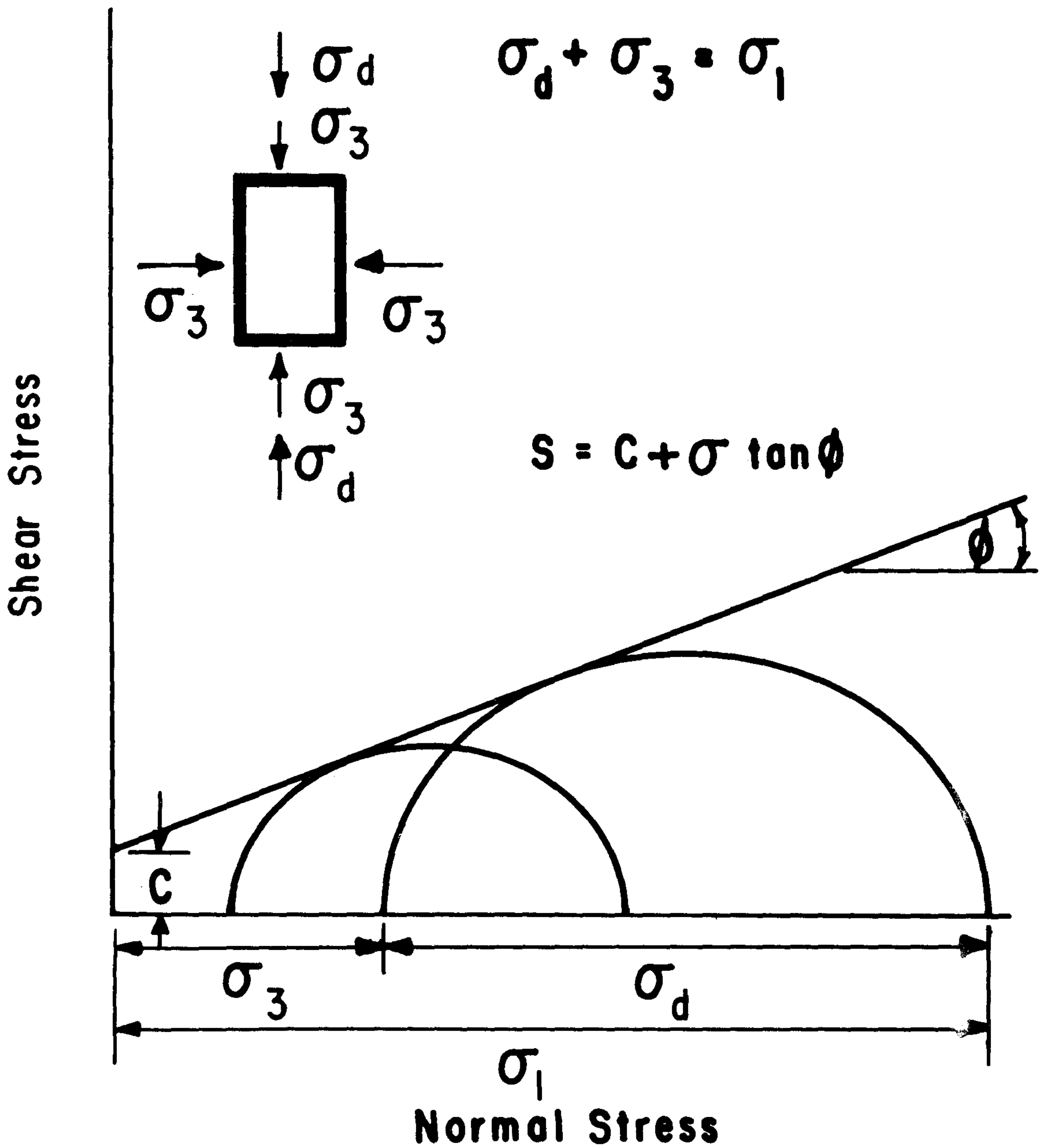
Visual Aid 14.8. CBR test results.



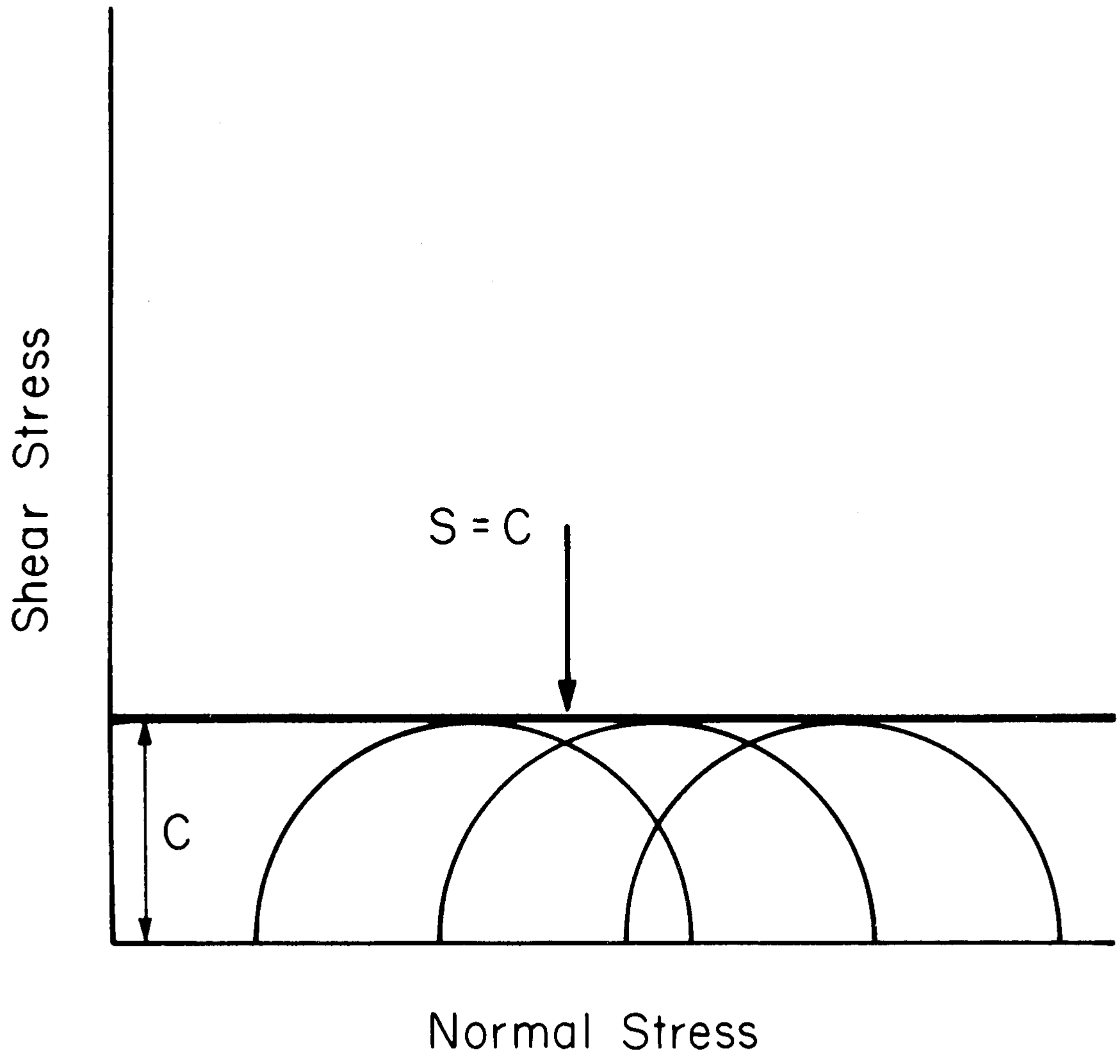
Visual Aid 14.9. Schematic diagram of a triaxial cell.



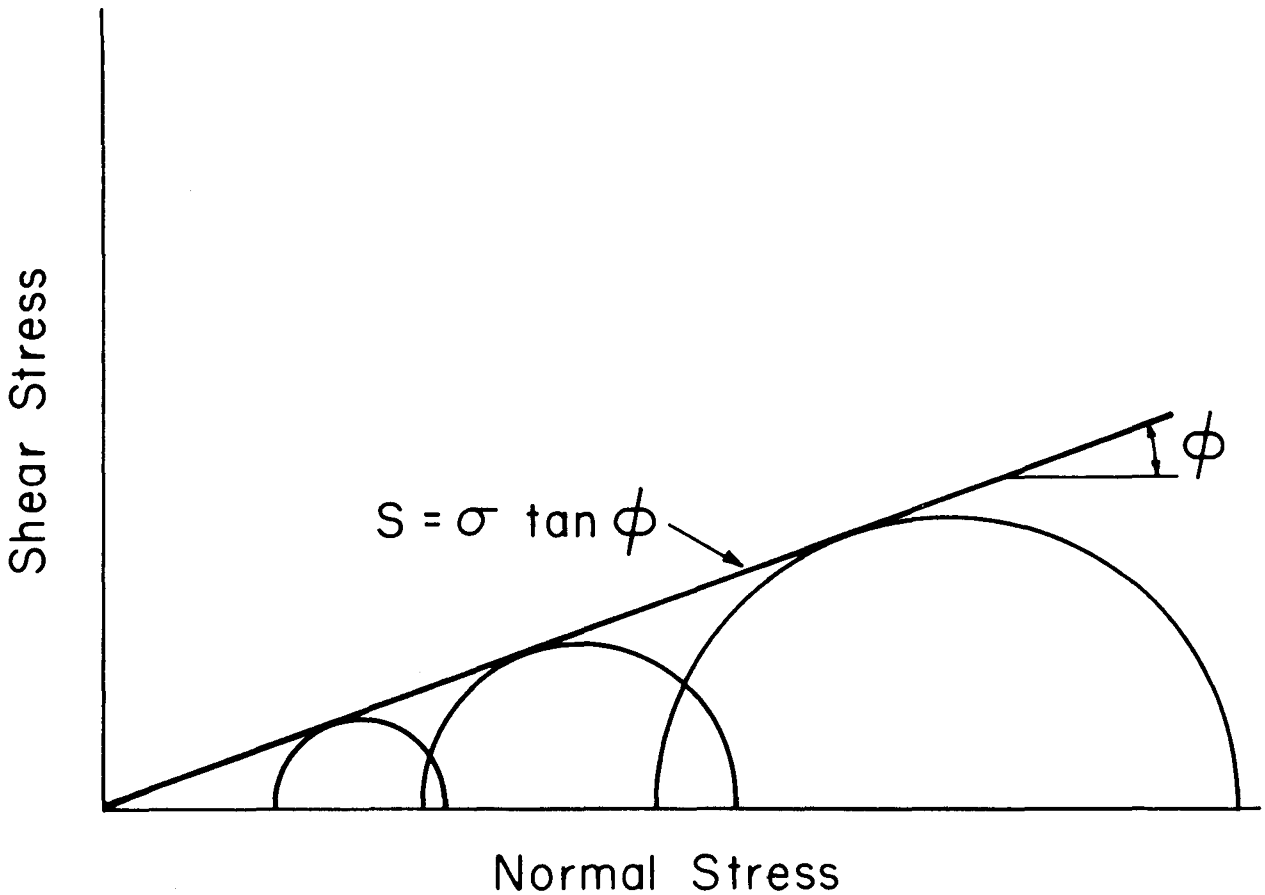
Visual Aid 14.10. Internal stability of soil represented by Coulomb equation.



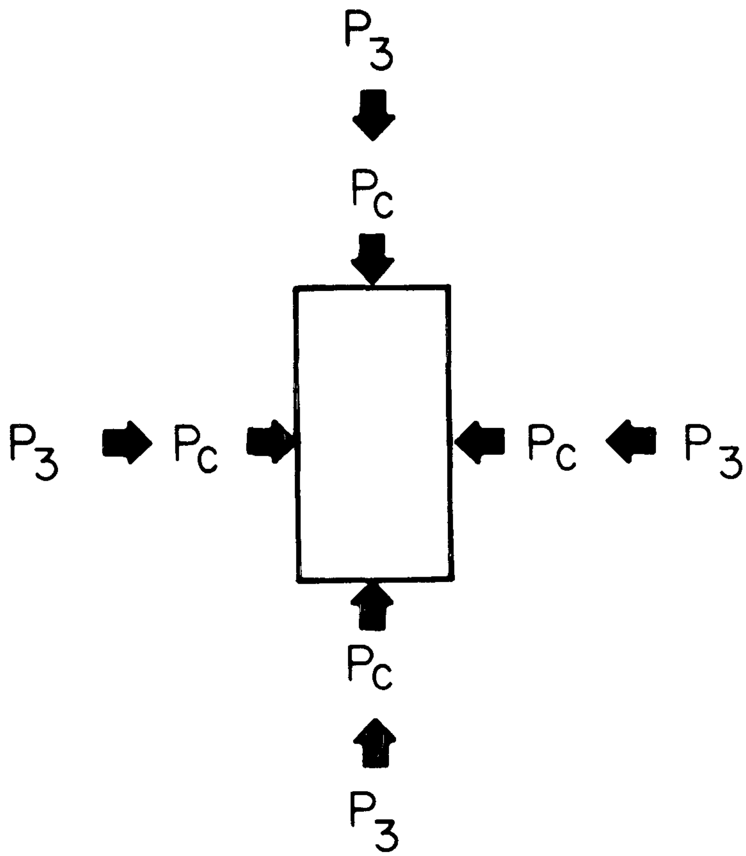
Visual Aid 14.11. Internal stability for cohesive soils.



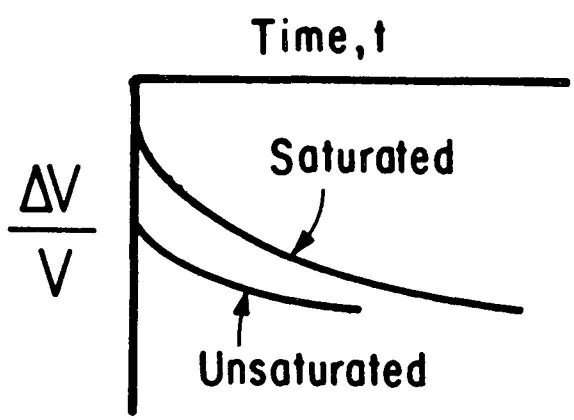
Visual Aid 14.12. Internal stability for cohesive soils.



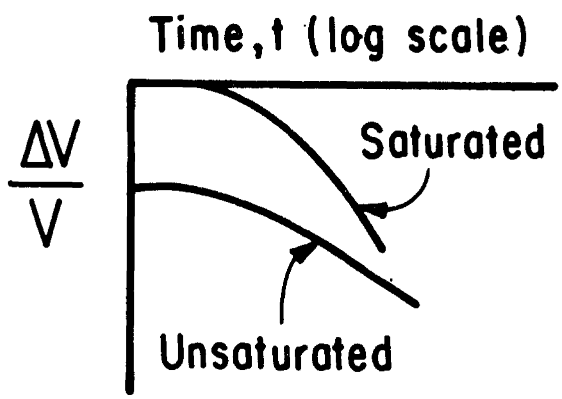
Visual Aid 14.13(a). Behavior of sample under all-around compression.



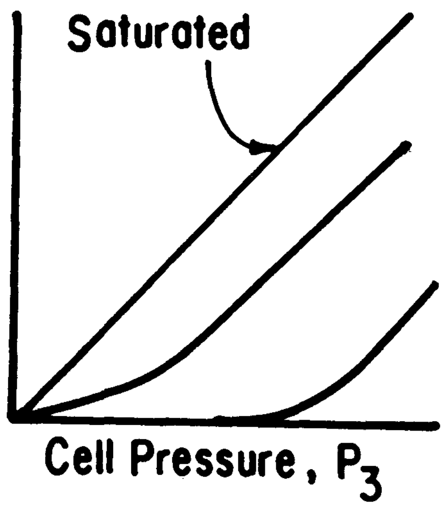
Visual Aid 14.13 (b). Volume change as function of time when drainage is permitted.



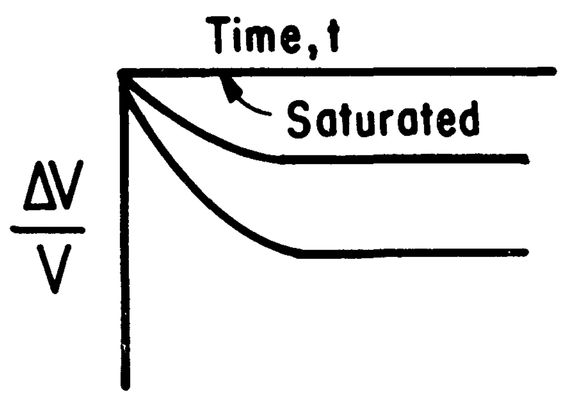
Visual Aid 14.13 (c). Volume change as function of time when drainage is permitted.



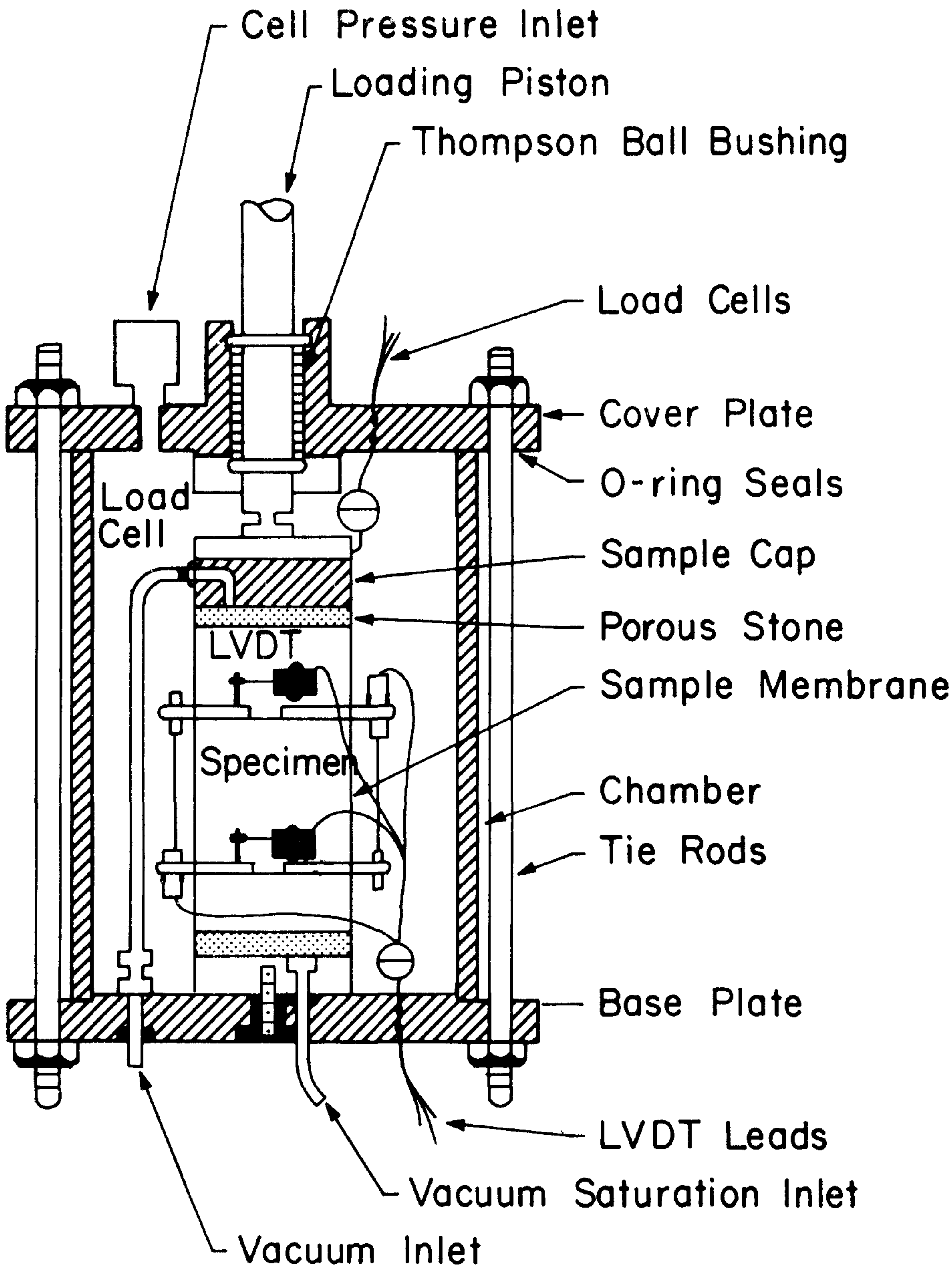
Visual Aid 14.13 (d). Porewater pressure as function of applied pressure if drainage is prevented.



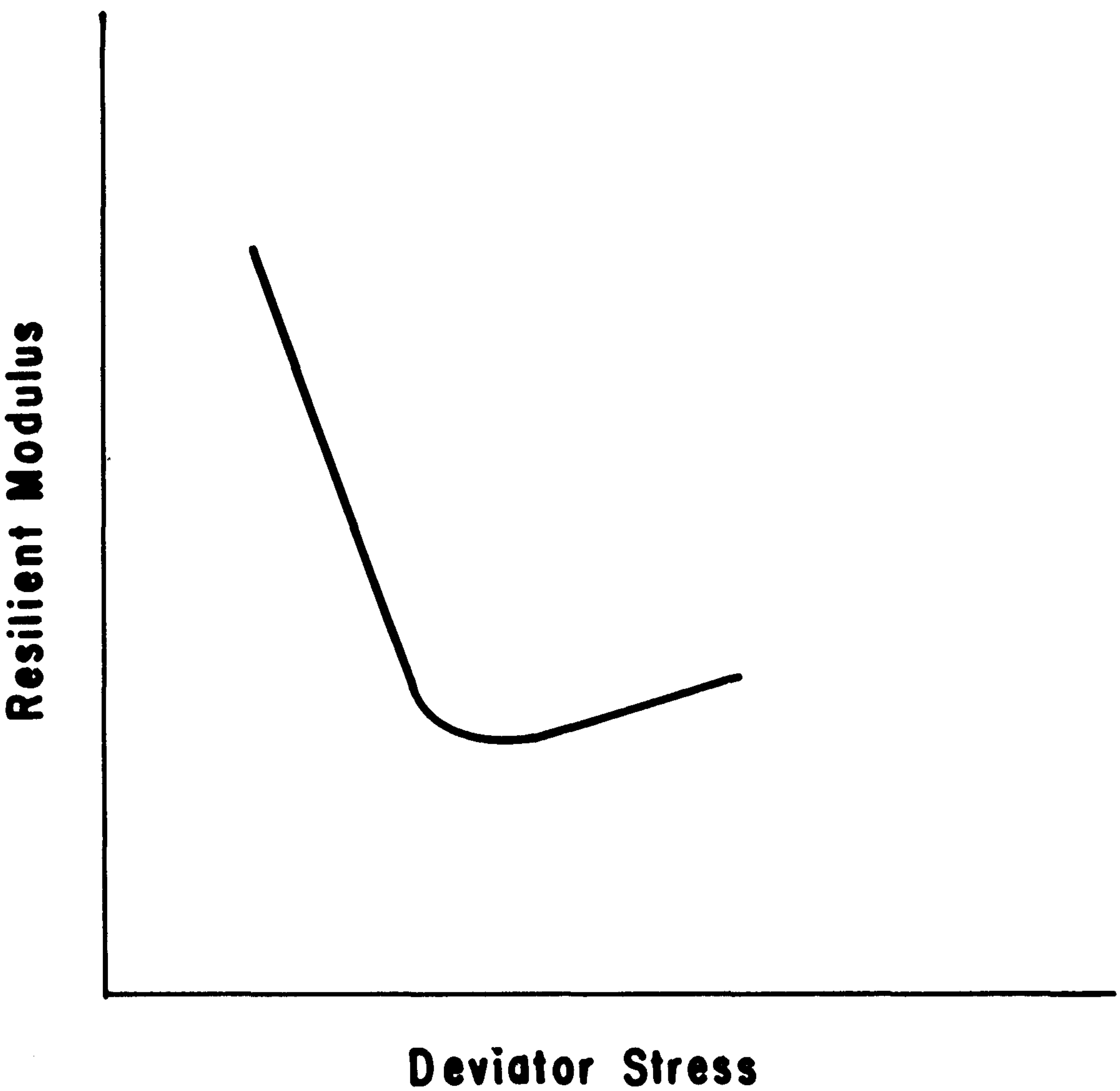
Visual Aid 14.13 (e). Volume change of undrained sample as function of time for different initial degrees of saturation.



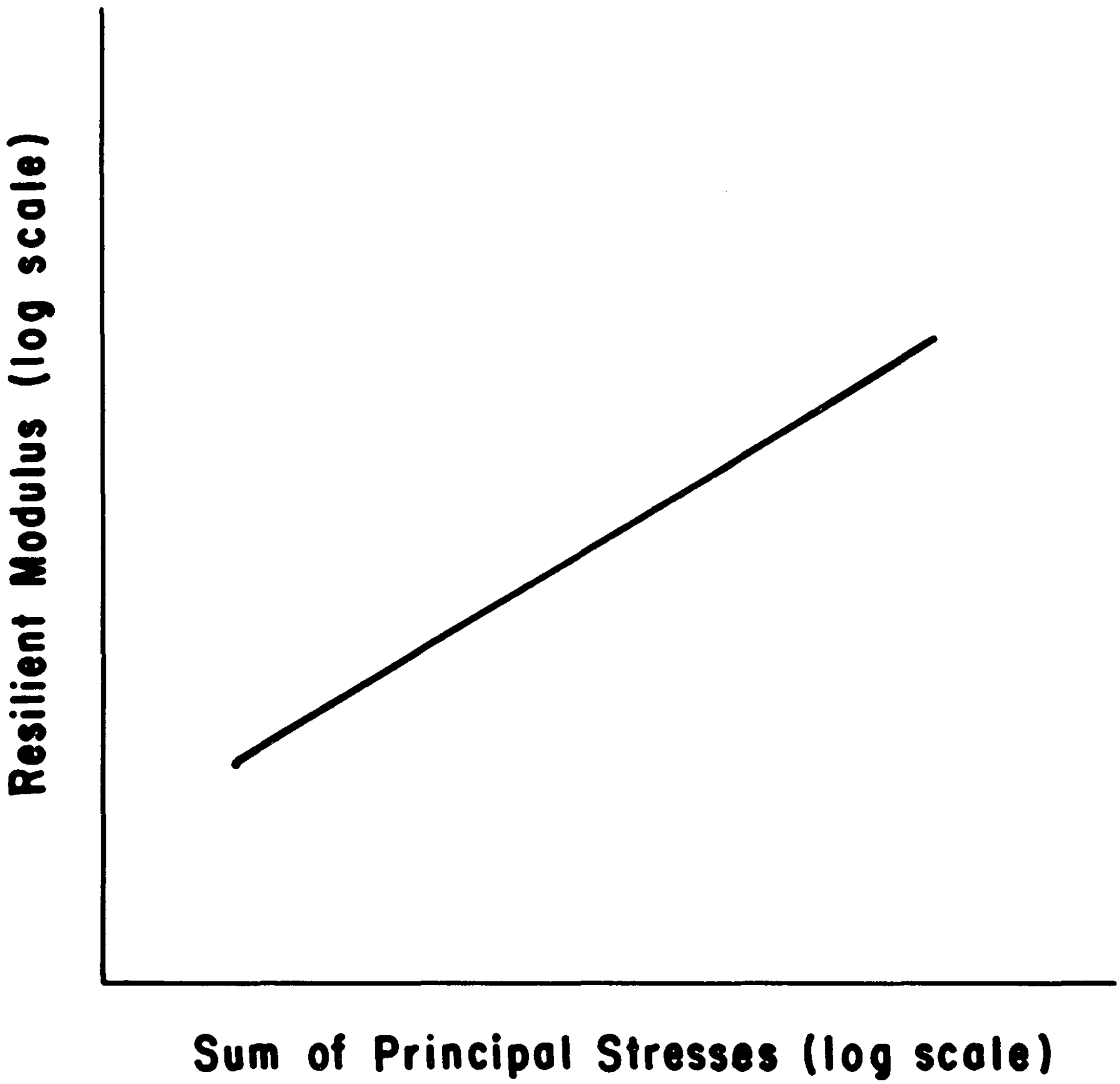
Visual Aid 14.14. Apparatus for resilient testing of subgrade materials.



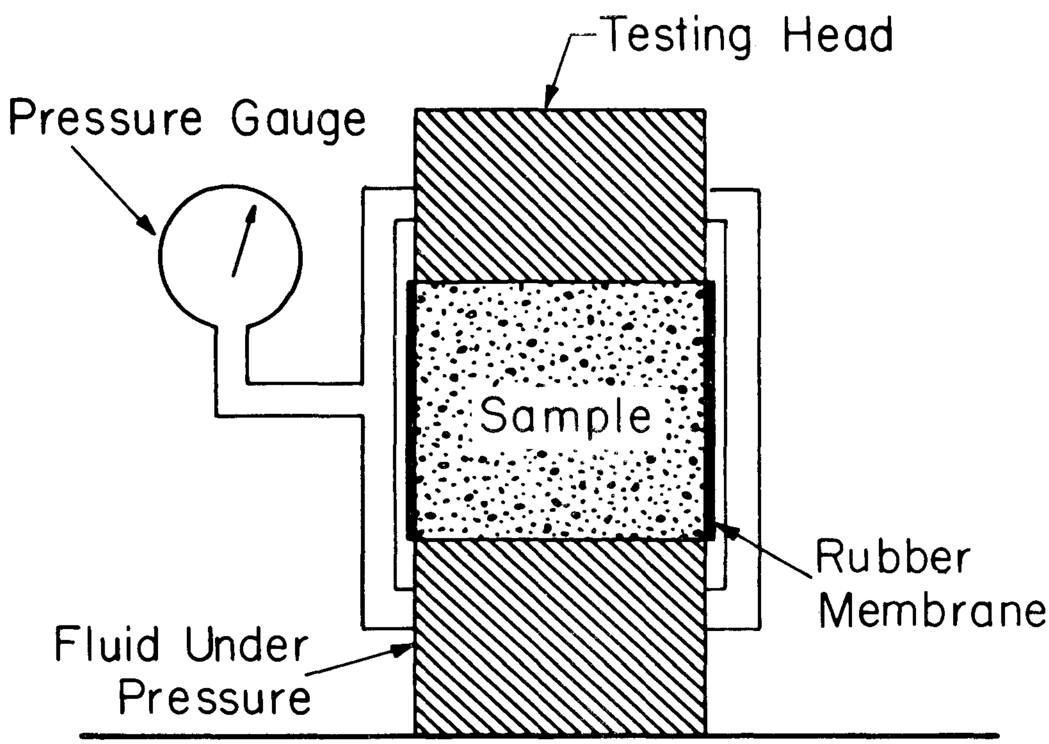
Visual Aid 14.15. Typical resilient modulus response for cohesive soils.



Visual Aid 14.16. Typical resilient modulus response for granular soils.



Visual Aid 14.17. Stabilometer for determining R-value.



Visual Aid 14.18. AASHO Soil Classification

(Classification of Highway Subgrade Materials)

General classification	Granular materials (35% or less passing No. 200)			Silt-clay materials (More than 35% passing No. 200)			
Group classification	A-1	A-3	A-2	A-4	A-5	A-6	A-7
Sieve analysis, per cent passing							
No. 10							
No. 40	50 max	51 min					
No. 200	26 max	10 max	35 max	36 min	36 min	36 min	36 min
Characteristics of fraction passing No. 40:							
Liquid limit				40 max	41 min	40 max	41 min
Plasticity index	6 max	NP		10 max	10 max	11 min	11 min
Group index			4 max	8 max	12 max	16 max	20 max
General rating as subgrade	Excellent to good			Fair to poor			

(Subgroups)

General classification	Granular materials (35% or less passing No. 200)							Silt-clay materials (more than 35% passing No. 200)			
	A-1		A-3	A-2				A-4	A-5	A-6	A-7
Group classification	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5, A-7-6
Sieve analysis, per cent passing											
No. 10	50 max										
No. 40	30 max	50 max	51 min								
No. 200	15 max	25 max	10 max	35 max	35 max	35 max	35 max	36 min	36 min	36 min	36 min
Characteristics of fraction passing No. 40:											
Liquid limit				40 max	41 min	40 max	41 min	40 max	41 min	40 max	41 min
Plasticity index	6 max		NP	10 max	10 max	11 min	11 min	10 max	10 max	11 min	11 min
Group index	0		0	0		4 max		8 max	12 max	16 max	20 max
Usual types of significant constituent materials	Stone fragments, gravel, and sand		Fine sand	Silty or clayey gravel and sand				Silty soils		Clayey soils	
General rating as subgrade	Excellent to good							Fair to poor			

INSTRUCTIONAL TEXT

Excerpt from

Research Report No. 62-2
Texas Highway Department

COMPARISON OF CONCRETE PAVEMENT LOAD-STRESSES AT AASHO ROAD TEST WITH PREVIOUS WORK

By

W. R. Hudson

Prepared for Presentation at the
42nd Annual Meeting of the Highway
Research Board
1963

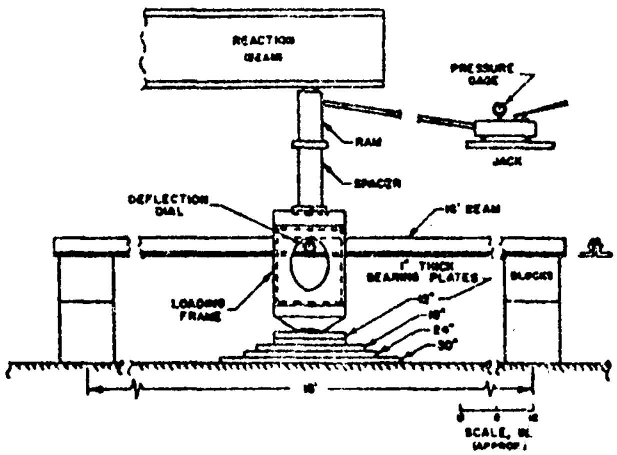


FIGURE B-1 - Apparatus for Plate Load Test

PLATE LOAD TESTS - DETERMINATION OF "K"

The following is a simple procedure for determining the modulus of subgrade reaction (k) which was used to determine k at the AASHO Road Test.

Equipment

Basic equipment of: (1) reaction trailer, (2) hydraulic ram and jack; (3) various sizes of steel spacers for use where needed at depths; (4) a 12 inch diameter cylindrical steel loading frame cut out on two sides to allow use of center deflection dial; (5) spherical bearing block; (6) 1 inch thick steel plates, 12, 18, 24 and 30 inches diameter; and (7) 16 foot long aluminum reference beam. A schematic diagram of the apparatus is given in Figure B-1.

The reaction trailer was of the flat-bed type, having no springs and four sets of dual wheels on the rear. For the tests on the AASHO Road Test a cantilever beam protruding from the rear of the trailer was used as a reaction. The distance load to rear wheels was eight feet. A maximum reaction of about 12,000 pounds could be obtained with a 17,000 pound loaded rear axle.

A standard hydraulic ram was used to apply the load. A calibration curve, which was checked periodically, was used to convert gage pressures to load in pounds.

The load was applied to the plates through the 12 inch diameter steel loading frame and the sperical bearing block. Deflection was measured with a dial gage as shown in Figure B-1.

The weight of the loading frame and plates was allowed to act as a seating load for which no correction was made.

Test Procedures

Tests were made in areas about 3 to 4 feet wide. The procedure provided for the application and release of 5, 10, and 15 psi loads on a 30 inch plate and for measurement of the downward and upward movement of the plate. The loads were applied slowly with no provision for the deformation to come to equilibrium.

Basic steps in the procedure were:

1. Test area was covered with fine silica sand and leveled by rotating the plate.

2. Equipment was set in place (Figure B-1.)

3. A seating pressure of 2 psi was applied and released. Dial gages were set to zero.

4. First increment of pressure was applied, held fifteen seconds and dial gage read.

5. Load was then released and dial gage read at end of fifteen second period.

6. Load was reapplied and released in the same manner three times and readings were taken each time.

7. Steps 4 through 6 were repeated for second and third increments of psi load.

8. Gross and elastic deflections were computed from dial gage readings.

k-Values were Computed as Follows:

a. Gross k-value, k_g = the unit load divided by the maximum gross deflection after three applications of the load. The reported k was an average of these computations.

b. Elastic k-value, k_e = the unit load divided by the elastic deformation at each application of each incremental load. The reported k_e was an average of all nine of these computations (3 loads x 3 applications each).

c. $k_e \simeq 1.77\ k_g$ describes the relationship between the two k values as developed through correlation from numerous tests on the AASHO Road Test.

LESSON OUTLINE
AASHTO DESIGN GUIDE FOR RIGID PAVEMENTS

Instructional Objectives

1. To provide the student with a basic knowledge and understanding of the AASHTO design guide for rigid pavements.

2. To illustrate the practical use of the AASHTO design guide for rigid pavements.

Performance Objectives

1. The student should be able to identify and explain each of the rigid pavement design inputs used in the AASHTO method.

2. The student should be able to design simple rigid pavements using the AASHTO method.

Abbreviated Summary	Time Allocation, min.
1. Introduction	5
2. Design Equation	5
3. Design Inputs	10
4. Joints and Load Transfer	10
5. Reinforcement Requirement	10
6. Design Example	10
	50 minutes

Reading Assignment

1. AASHTO Interim Guide, Chapter III, Appendix D.3, D.4.

2. NCHRP 128, pp 26-37, pp 90-99.

LESSON OUTLINE
AASHTO DESIGN GUIDE FOR RIGID PAVEMENTS

1.0 INTRODUCTION

1.1 Based on Results of AASHTO Road Test

1.1.1 Basic Equation for Road Test Conditions.

$$\log W_{t_{18}} = 7.35 \log (D + 1) - 0.06 + G_t/\beta$$

where

$$G_t = \log \left[0.333(4.5 - p_t) \right]$$

$W_{t_{18}}$ = 18 kip single axle loads at end of time t

p_t = serviceability at end of time t

D = thickness of slab, inches

1.1.2 Use Stress Calculated from Slab Theory to Design for Conditions Other Than Road Test.

It was necessary to modify the general road test equation using experience and theory. This was accomplished by comparing stresses calculated from strain measurements on the road test pavement slabs with stresses calculated using the theoretically based formulas.

1.2 Limitations

(a) Westergaard theory applies (as modified by Spangler).
(b) No regional factor.
(c) No specific consideration of internal drainage.
(d) Traffic analysis for design relationship based on AASHO Road Test.
(e) Adequacy of design based on information from soils and materials surveys and laboratory tests.
(f) Design strengths for subgrade and pavement structure must be achieved through proper construction techniques.

1.3 Nomenclature

1.3.1 Pavement.

The concrete, surface including the base (often called subbase) is referred to as the pavement.

1.3.2 Purpose of base or subbase course.

(a) Control pumping.
(b) Control frost action.
(c) Drainage.
(d) Alleviate effects of volume change of subgrade.
(e) Expedite construction.
(f) Increase modulus of subgrade reaction (k).
(g) Provide uniform, stable, and permanent support.

2.0 DESIGN EQUATION

The variables considered in the AASHO Road Test were load, slab thickness and number of axle applications.

Some variables, which were constant at the Road Test would very under normal design conditions, including the subgrade reaction (k). modulus of the concrete, strength of the concrete, and load transfer devices and effectiveness. Other considerations must be given to the environment, subbase thickness and quality and pavement age.

2.1 Theory (Based on Spangler) Equations

$$\sigma = \frac{JP}{D^2}\left[1 - \frac{a_1}{\ell}\right]$$

where

σ = concrete stress, psi

P = load, lbs

D = slab thickness, inches

*a_1 = center of load to corner, inches

* ℓ = radius of relative stiffness

J = load transfer factor

* The derivation of this equation and the more complete definition of these terms is covered in Lesson 13 - Westergaard solutions.

The load transfer factor "J" is taken to be:

3.2 for jointed reinforced concrete pavements (JRCP)
2.2 for continuously reinforced concrete pavements (CRCP)

This term is also called the "pavement continuity term." Pavement continuity is defined as the percentage of load transferred across a pavement discontinuity, such as a joint or crack. It is recommended that the above values be used until more data or experience is gained regarding pavement continuity. The term may be adjusted based on observations of deflections for the various pavement types, under varying degrees of support, and environmental conditions.

2.2 AASHO Road Test Rigid Equation

$$\log W_{t_{18}} = 7.35 \log (D + 1) - 0.06 + \frac{G_t}{1 + \frac{1.624 \times 10^7}{(D + 1)^{8.46}}}$$

$$+ (4.22 - 0.32p_t) \log \left[\left(\frac{S'_c}{2.15.63J'}\right)\left(\frac{D^{0.75} - 1.132}{D^{0.75} - \frac{18.42}{\left(\frac{E}{K}\right)^{0.25}}}\right)\right]$$

where

$W_{t_{18}}$ = total 18-kip load applications

D = slab thickness, inches

E = concrete elastic modulis, psi

k = Westergaard's modulus of subgrade reaction, pci

p_t = serviceability at end of time t

S'_c = 1/3 point flexural strength of concrete

J' = load transfer factor

G_t = ratio of loss of serviceability at time t to the potential loss taken to a point where $p_t = 1.5$

$$G_t = \log \left[0.333 (4.5 - p_t)\right]$$

This equation is solved by nomograph (Visual Aid 15.1)

In addition the 1981 edition of AASHTO guide also has an alternate design chart for rigid pavements which permits you to consider more variables such as load transfer condition (J factor) and several levels of p_t. The alternate design chart is illustrated in Visual Aid 15.2.

3.0 DESIGN INPUTS

3.1 Flexural Strength of Concrete (S'_c)

The modulus of rupture (S'_c) at 28 days is determined by the test procedure specified in AASHTO Designation T-97, using third-point loading.

3.1.1 Work Stress (f_t). The scale indicated in Visual Aid 15.1 is based on working stress in the concrete where

$$f_t = \frac{S'_c}{c}$$

where c is a safety factor.

"c" is commonly taken as 1.33. The higher the value, the higher the confidence in the adequate design. However, a "c" of 2.0 can add 1 to 2 inches of slab thickness.

3.2 Modulus of Elasticity of Concrete (E)

The modulus of elasticity is determined by ASTM Designation C649 (cylindrical compression test). The E value at the Road Test was 4,200,000 psi.

3.3. Modulus of Subgrade Reaction (k) (Visual 15.3)

The modulus of subgrade reaction at the Road Test was 60 pci. A method to estimate the composite k_c-value based on subbase thickness and stiffness is outlined in Appendix D of the AAHTO Interim Guide. The composite k_c-value is used to determine the pavement thickness. Although granular material was used in the AASHTO Road Test sections, stabilized subbases are generally used in most of the rigid pavement construction today.

4.0 JOINTS AND LOAD TRANSFER

4.1 Expansion Joints

The primary function of an expansion joint is to prevent the development of damaging compressive stresses due to volume changes in the slab. A 3/4 to 1 inch joint is suggested; however, consideration might be given to using suitable terminal anchorage devices in combination with expansion joints. (Not often used)

4.2 Contraction Joints

The purpose of contraction joints is to provide an orderly arrangement of cracking that occurs. These may be sawed or formed and their depth should be greater than 1/4 of the thickness of the pavement slab.

(a) Need mechanical load transfer.
(b) Usually use a 4 to 5-foot skew on a 24 foot width pavement.

4.3 Longitudinal Joints

Longitudinal joints are used to prevent the formation of irregular longitudinal cracks. A depth of greater than 1/4 the thickness of the slab is usd here also. Steel tie bars are used to prevent faulting and adjoining lane separation.

4.4 Load Transfer Devices

4.4.1 Desirable characteristics.

(a) Simple in design and practical to install
(b) Properly distribute load stresses
(c) No restraint to longitudinal movement
(d) Mechanically stable under wheel loads
(e) Resistant to corrosion

4.4.2 Minimum Design Requirements.

(Visual Aid 15.4) The minimum design requirements for round dowels is shown in Visual Aid 15.4.

4.5 Tie Bars

(a) Holding abutting slabs together.
(b) Designed to withstand maximum tensile forces induced by subgrade drag.
(c) Unit weight of concrete is assumed equal to 144 lb/ft^3 (Visual Aid 15.5).

5.0 REINFORCEMENT REQUIREMENTS

The primary purpose of reinforcement is not to prevent cracking but to hold tightly closed any cracks that may form.

5.1 Design Formula

The steel percentage in a jointed concrete pavement or as transverse steel regardless of the pavement type is

$$A_s = \frac{FLW}{2f_s}$$

where

A_s = cross-sectional area of steel per foot width of slab, square inches;

F = coefficient of resistance between slab and subgrade;

L = distance between free transverse joints or free longitudinal edges, ft;

W = weight of pavement slab, lb/sq. ft;

f_s = allowable working stress in the steel, psi.

5.2 Graphical Solution (Visual Aids 15.6 and 15.7)

This formula is used for both longitudinally and transverse steel, and is solved graphically as shown in Visual Aids 15.6 and 15.7.

5.3 CRCP Longitudinal Steel

$$P_s = (1.3 - 0.2F)\ \frac{S'_c}{f_s} \times 100$$

where

P_s = percentage of steel required

F = friction factor

S'_c = tensile strength of concrete

f_s = working steel stress

5.4 Bar Spacing (Visual Aid 15.8)

The steel percentage is used to determine the bar size and maximum spacing. Transverse steel can be reduced as you approach the free edges (Visual Aid 15.9).

6.0 DESIGN EXAMPLE

6.1 Input Data

Interstate highway (rural) - 9,000,000 equivalent 18-kip SAL's per 20 years

S_c = 650 psi (AASHTO T-97)

E = 4,200,000 psi (ASTM C469)

k = 200 psi (Westergaard analysis)

Assume p_t = 2.5

6.2 Calculate Working Stress

$$f_t = \frac{S_c}{C}$$

(assume C = 1.33 since on a rural interstate the capacity of a probable detour would suffice for short periods)

f_t = 650/133 = 490 psi

6.3 Determination of Slab Thickness

From Visual Aid 15.1 or 15.2

D = 9.6 in → 10 in

6.4 Design of Load Transfer Devices

From Visual Aid 15.4

dowel diameter = 1-1/4 inches

dowel length = 18 inches

dowel spacing = 12 inches

6.5 Design of Tie Bars

From Visual Aid 15.5

Assume 5/8-inch bars to be used

minimum overall length = 30 inches

minimum spacing = 48 inches

6.6 Design of Reinforcement

Assume slab length = 40 feet

slab width = 24 feet

f_s = 45,000 psi

F = 1.5

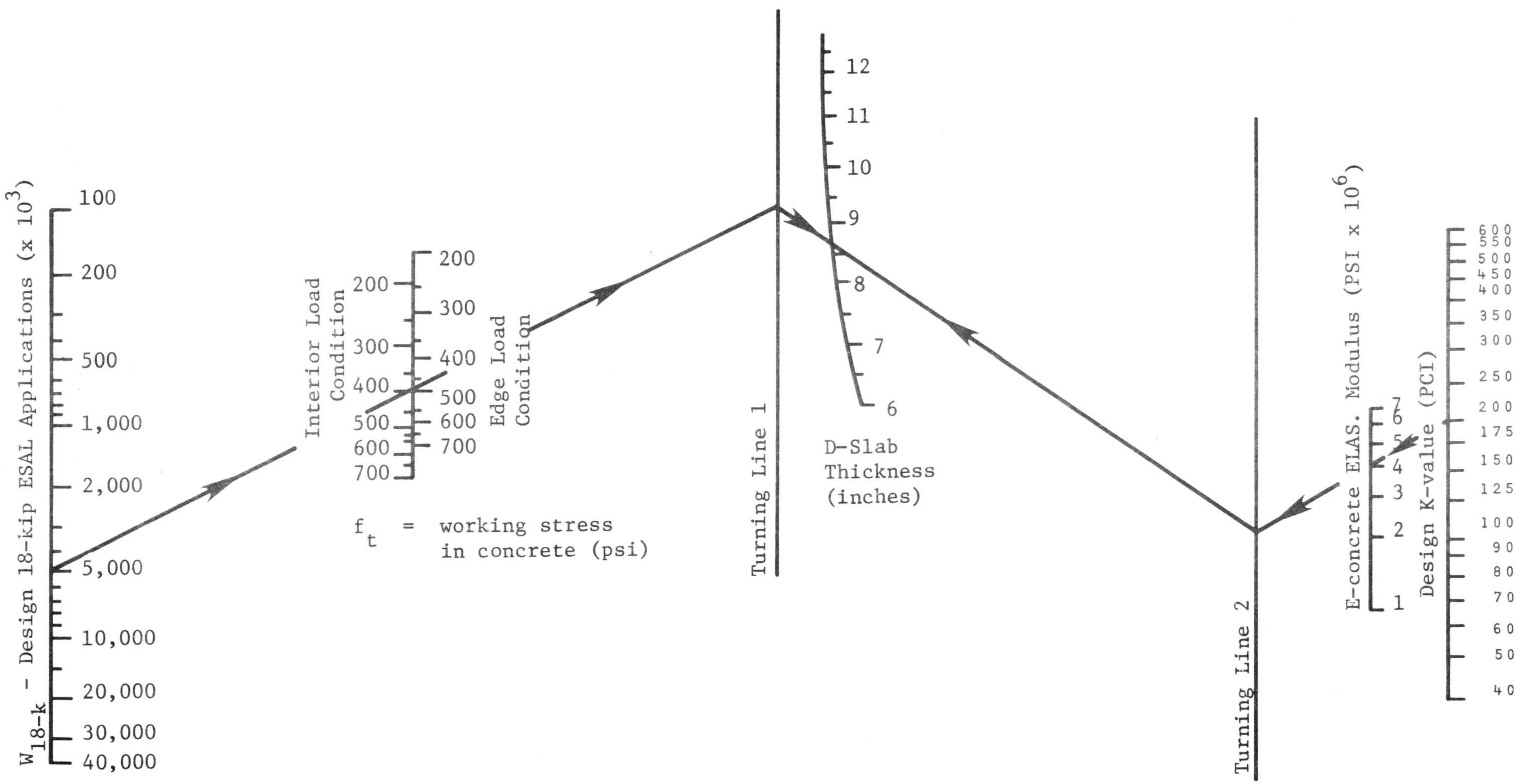
Using Visual Aid 15.6

A_s (longitudinal) = 0.09 sq. in.

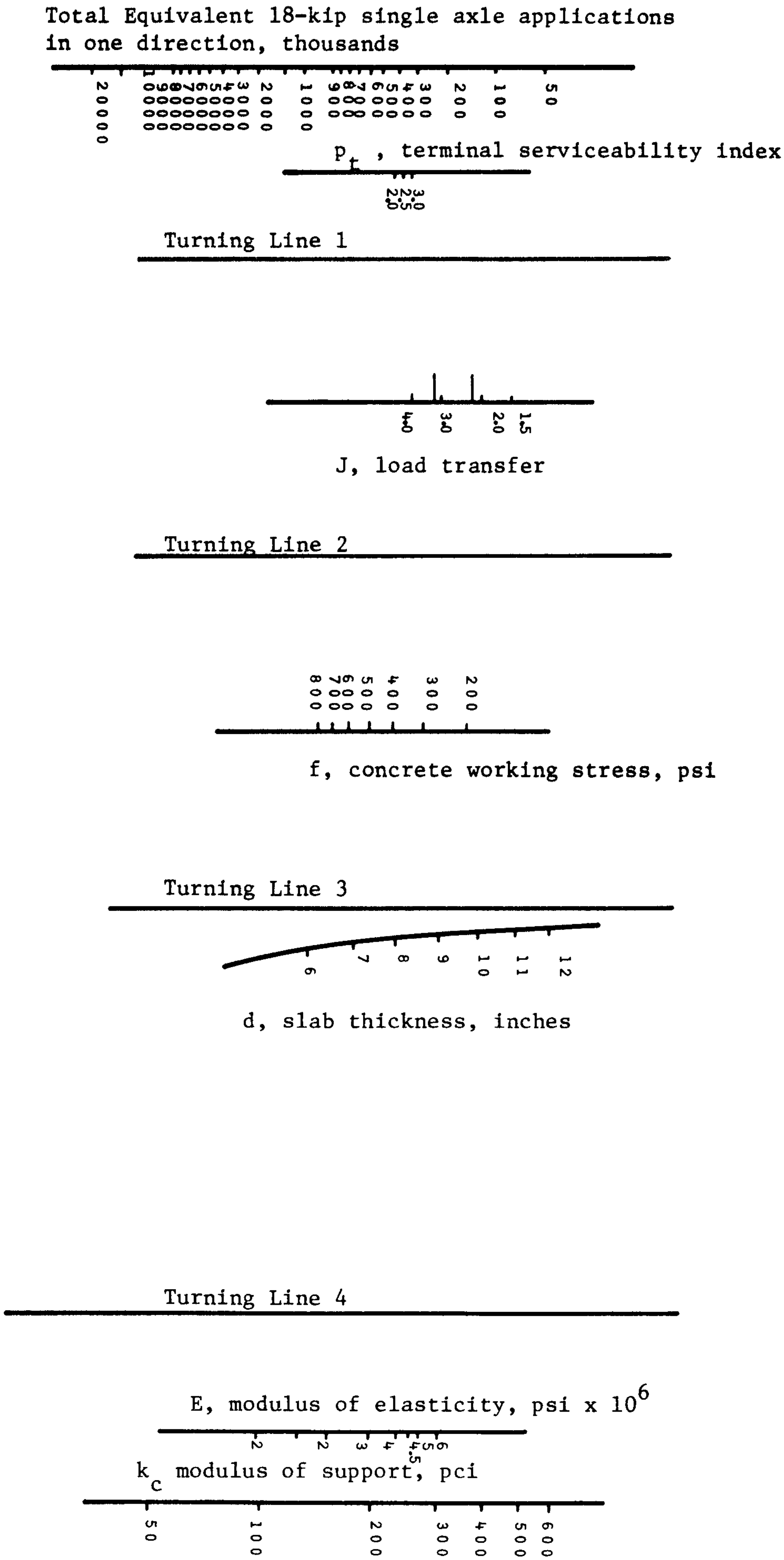
A_s (transverse) = 0.05 sq. in.

LESSON OUTLINE
AASHTO DESIGN GUIDE FOR RIGID PAVEMENTS

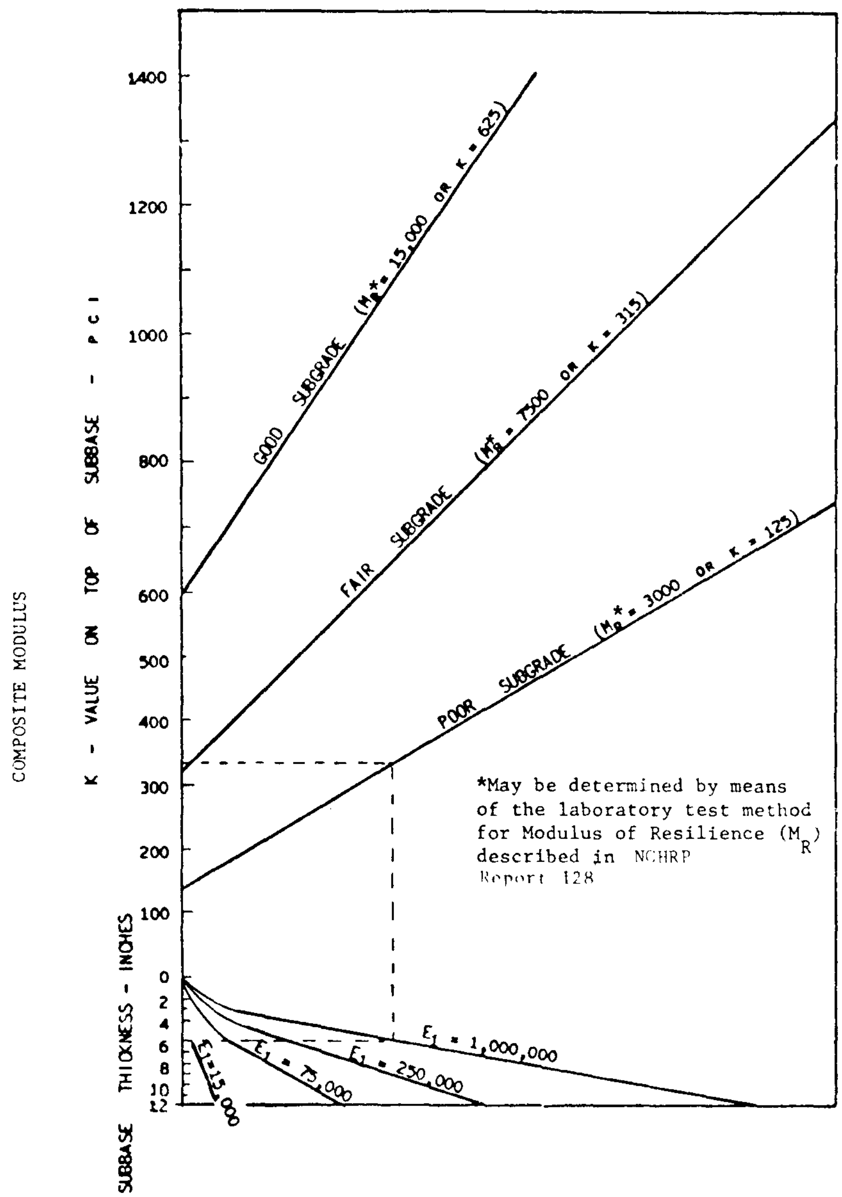
VISUAL AID	TITLE
Visual Aid 15.1.	Design chart for rigid pavement p_t = 2.5 (from AASHTO Interim Guide).
Visual Aid 15.2.	Design chart, alternate procedure for design of rigid pavements.
Visual Aid 15.3.	Chart for estimating composite k-values.
Visual Aid 15.4.	Design chart for load transfer devices.
Visual Aid 15.5.	Design chart for tie bars (after Table III-2, AASHTO Interim Guide).
Visual Aid 15.6.	Distributed steel percentage (after Fig D4.4 - AASHTO Interim Guide).
Visual Aid 15.7.	Nomograph for the design of steel reinforcement.
Visual Aid 15.8.	Reduction of transverse steel across pavement (after Fig D4-5 AASHTO Interim Guide).
Visual Aid 15.9.	Nomograph for determining the bar spacing design (after Fig D4-6 AASHTO Interim Guide).



Visual 15.1. Design Chart for Rigid Pavement p_t = 2.5
(from AASHTO Interim Guide)



Visual 15.2 Design chart, alternate procedure for design of rigid pavements.



Visual 15.3. Chart for Estmating Subbase k-value.

VISUAL 15.4. DESIGN CHART FOR LOAD TRANSFER DEVICE

PAVEMENT THICKNESS IN.	DOWEL DIAMETER IN.	DOWEL LENGTH IN.	DOWEL SPACING IN.
6	3/4	18	12
7	1	18	12
8	1	18	12
9	1 1/4	18	12
10	1 1/4	18	12
11	1 1/4	18	12
12	1 1/4	18	12

VISUAL 15.5. DESIGN CHART FOR TIE BARS

			1/2 in. Diameter Bars				5/8 in. Diameter Bars			
			Minimum	Maximum Spacing, in.*			Minimum	Maximum Spacing, in.		
Type and grade of steel	Working Stress, psi	Pavement Thickness, in.	overall length, in.**	Lane Width, 10 ft.	Lane Width, 11 ft.	Lane Width, 12 ft.	overall length, in.**	Lane Width, 10 ft.	Lane Width, 11 ft.	Lane Width 12 ft.
Grade of billet or axle steel	30,000	6	25	48	48	48	30	48	48	48
		7		48	48	45		48	48	48
		8		48	44	40		48	48	48
		9		43	39	35		48	48	48
		10		38	35	32		48	48	48
		11		35	32	29		48	48	45
		12		32	29	26		48	45	41

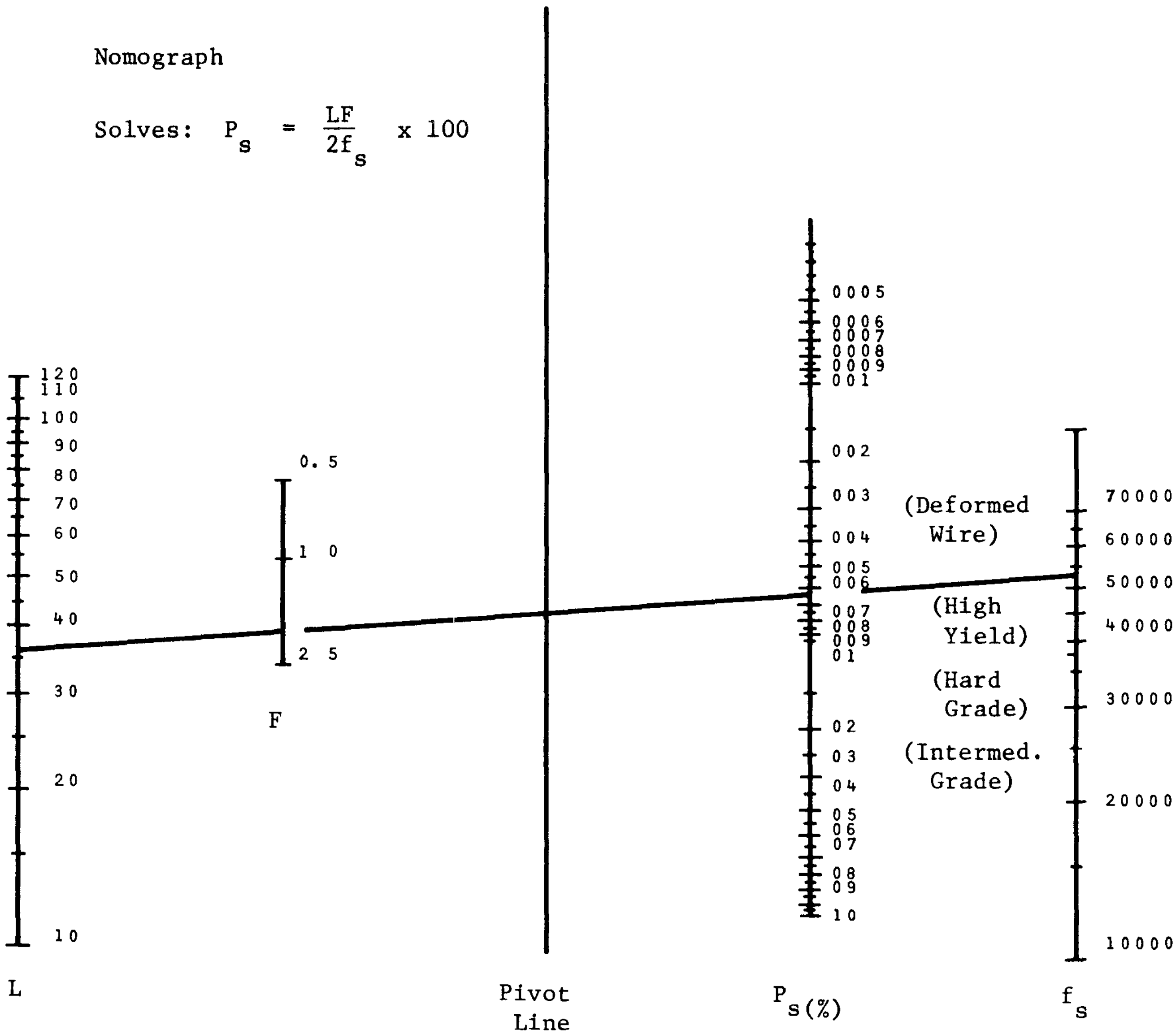
* It is recommended that spacing of tie bars should not exceed 48 inches.

**350 psi assumed for bond stress (u).

Length includes 3-inch allowance for centering.

Nomograph

Solves: $P_s = \frac{LF}{2f_s} \times 100$



Example Problem:

$L = 36$ ft
$F = 1.9$
$f_s = 52{,}500$ psi

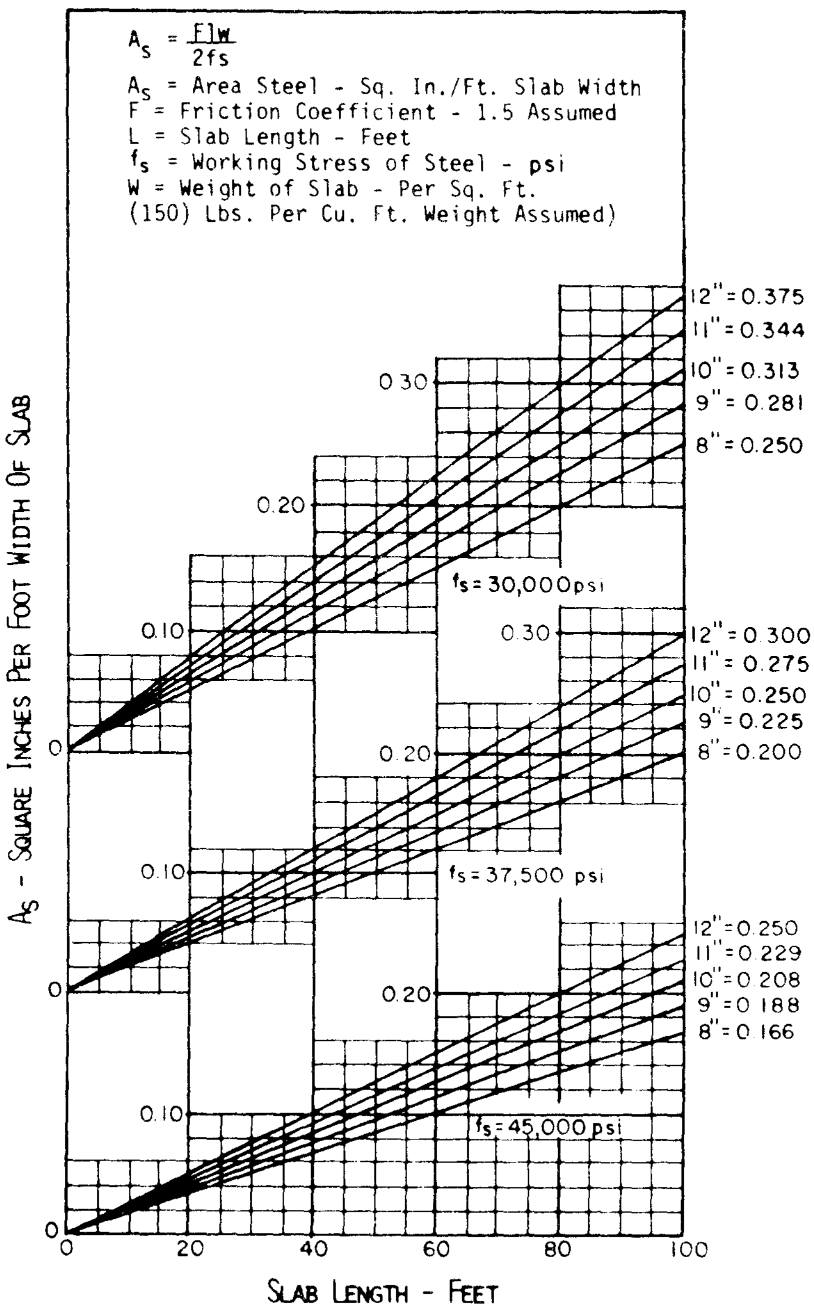
Answer:
$R_s = 0.067\%$

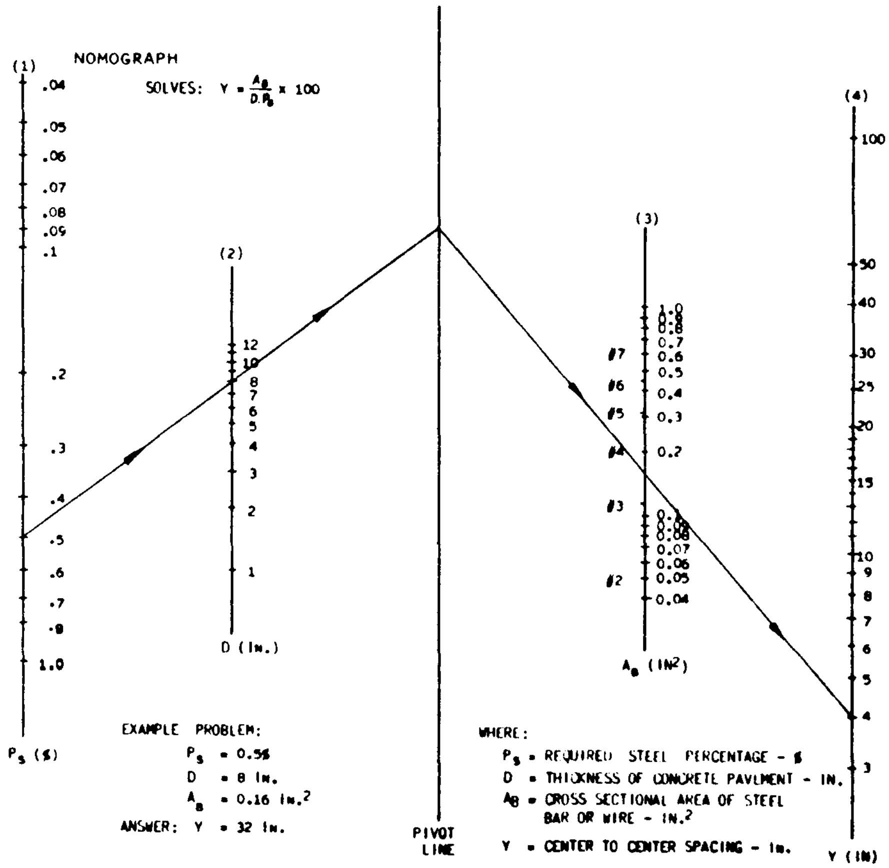
where:

P_s = required steel percentage - %
L = width of slab - feet
F = friction factor of subbase
f_s = allowable working stress in steel - psi (0.75 of yield strength recommended, the equivalent of safety factor of L33)

Visual 15.6 Chart for Determining Steel Percentage (After Fig D4.4 - AASHTO Interim Guide).

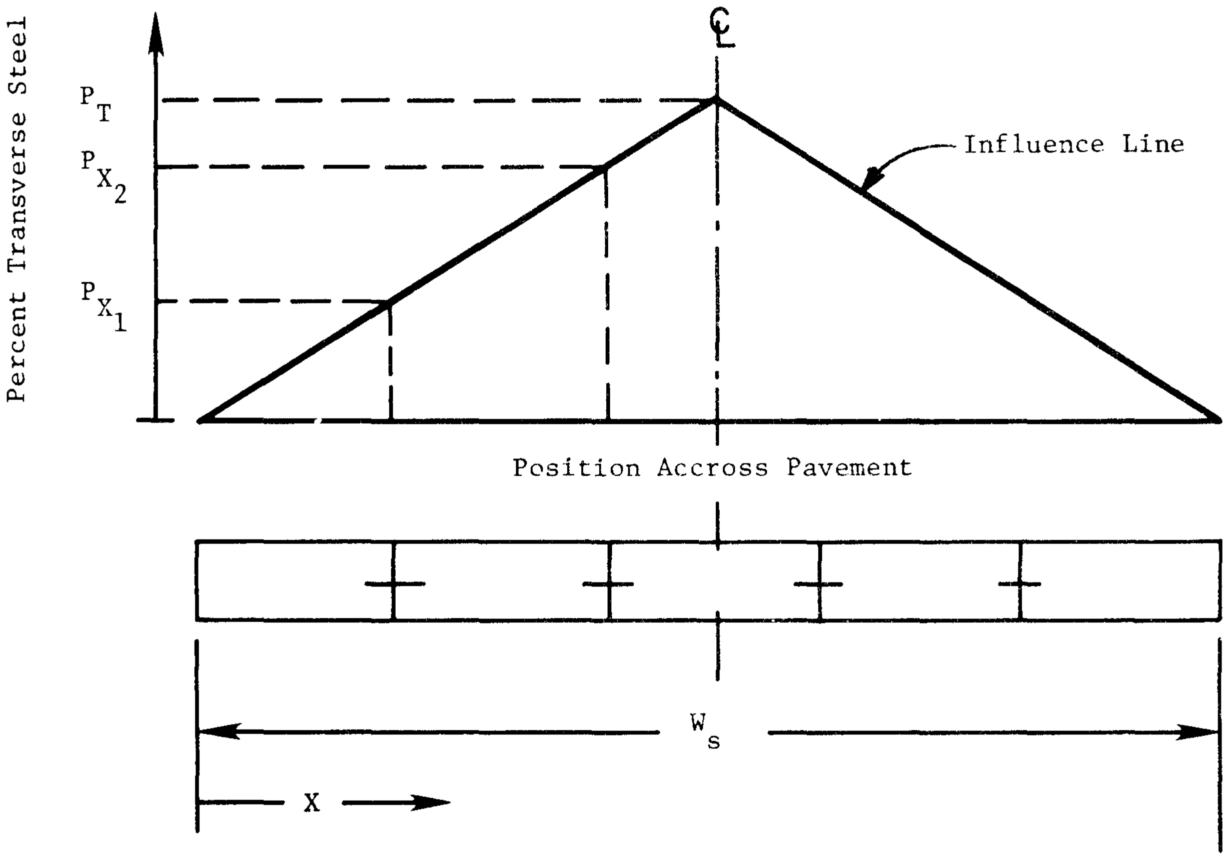
VISUAL 15.7. NOMOGRAPH FOR THE DESIGN OF STEEL REINFORCEMENT





Visual 15.8. Nomograph for Determining the Bar Spacing Design.

Visual 15.9. Reduction of Transverse Steel Across Pavement
(After Fig D4-5 ; AASHTO Interim Guide)



$$P_X = 2P_S \frac{X}{W_s}$$

where:

P_S = design percent steel in center of pavement

W_s = total width of pavement slab.

X = distance from a free edge to the most interior point of the area under consideration

P_X = reduced percent transverse steel at location X

LESSON 16

GUEST LECTURER

TO BE ANNOUNCED

LESSON OUTLINE
FLEXIBLE PAVEMENT/ELASTIC LAYERED THEORY

Instructional Objectives

1. To provide the student with a basic understanding of flexible pavement theory in terms of models available for mechanistic analysis of flexible pavements.

2. To explain the assumptions and limitations of the various structural models.

Performance Objectives

1. The student should be able to state all of the assumptions and limitations of the available structural models used in analysis of flexible pavements.

2. The student should be able to analyze the stresses and strains in a typical flexible pavement based on one of the presented models.

Abbreviated Summary	Time Allocation, mins.
1. Background	10
2. One - Layered Theory	30
3. Two - Layered Theory	30
4. Three - Layered Theory	30
	100

Reading Assignment

1. Yoder and Witczak - Chapter 2, pp 24-78

Additional Reading

1. Fosler, C. R., and R. G. Ahlvin, "Stresses and Deflections Induced by a Uniform Circular Load," Proceedings, Highway Research Board, 1954, pp 467-470.

LESSON OUTLINE
FLEXIBLE PAVEMENT/ELASTIC LAYERED THEORY

1.0 BACKGROUND

1.1 Flexible Pavement

Pavement structure composed of layers with decreasing moduli with depth; usually composed of an asphalt wearing course with layers of granular base or subbase for the purpose of reducing stress on the top of the subgrade.

1.2 Analysis Procedure (Visual Aid 17.1)

Layered theory is applied to predict load stresses.

1.3 Fundamental Assumptions of Layered Theory (Visual Aid 17.2)

A uniform circular load is assumed at the surface and information of stress and strain can be obtained at any point. Layered theory can not handle discontinuities. Pavement materials are characterized assuming linear elastic behavior.

2.0 ONE - LAYER THEORY

2.1 Background

Developed by Boussinesq (French mathematician/engineer) in early 1800's; Love solved differential equations in late 1800's.

2.2 General Assumptions (Visual Aid 17.3)

Bell - shaped distribution of stress on horizontal plane and maximum stress at shallow depth, i.e., near surface.

2.3 Formula for Point Load (Visual Aid 17.3)

No material properties are involved in the determination of stresses.

$$\sigma_z = K \frac{P}{Z^2}$$

$$K = \frac{3}{2\pi} \frac{1}{[1 + (r/z)^2]^{5/2}}$$

where

σ_z = vertical stress, in z direction

P = point load, lbs

r = radial distance from point load

z = depth

2.4 Formulas for Area Load (Visual Aid 17.4)

2.4.1 Stresses.

$$\sigma_z = P\left[1 - \frac{Z^3}{(a^2 + Z^2)^{3/2}}\right] \qquad \text{at} \quad r = 0$$

$$\sigma_r = \frac{P}{2}\left[1 + 2\mu - \frac{2(1 + \mu)Z}{(a^2 + Z^2)^{1/2}} + \frac{Z^3}{(a^2 + Z^2)^{3/2}}\right]$$

where

σ_r = radial stress

P = contact pressure

a = loaded area radius

μ = Poisson's ratio

2.4.2 Strains.

$$\varepsilon_z = \frac{1}{E}\left[\sigma_z - 2\mu\,\sigma_r\right] \quad \text{and}$$

$$\varepsilon_r = \frac{1}{E}\left[\sigma_r - 2\mu\,\sigma_z\right]$$

where

ε_z = vertical strain

ε_r = radial strain

E = Young's modulus of elasticity

σ_r and σ_z have been defined earlier.

2.4.3 Deflection. To calculate deflection, strain, ε_z is intergrated over some depth (from $z = z$ to $z = \infty$).

(a) Flexible plate - deflection equation (Visual Aid 17.5). In this case the loaded plate is assumed to have zero stiffness.

$$\Delta = \text{deflection} = \frac{3pa^2}{2E\ (a^2 + z^2)^{1/2}} \quad \text{for } \mu = 0.5$$

$$\Delta \text{ at surface} = \frac{1.5\ Pa}{E} \quad (\text{at } z = 0).$$

(b) Rigid plate deflection equation (Visual Aid 17.5) - (i.e., constant deflection over the area of the loaded plate)

$$\Delta \text{ at surface} = \frac{pa\ \pi(1 - \mu^2)}{2E}$$

$$\text{For } \mu = 0.5;\quad \Delta_{Rigid} = 1.15\ \frac{Pa}{E}$$

(c) Application of deflection formulas - If we measure deflection, we can predict Young's modulus.

2.6 Corps of Engineers' Work

They found CBR method for pavement design to be useful to design airstrips. They transformed the original design curves to correspond to the heavier plane loads by using Boussinesq theory computations.

2.6.1 Charts Developed by Foster and Ahlvin (Visual Aid 17.6).

(a) Solutions for various parameters expressed in terms of various functions A, B, C, ..., H. (Visual 17.7) Assuming zero deflection in pavement deflection (Visual 17.8).

(b) Influence charts - Stress computed in percent of contact pressure with depth and offset in terms of radii (Visual 17.9a-e).

2.7 Example Problem (Visual Aid 17.10)

(a) Stresses shown on elements A and B are to be solved by using the influence charts.

(b) As the material is assumed to be linear elastic; principle of superposition is valid and should be used to determine stresses on element B.

2.8 Problems with One-Layer Theory (Visual Aid 17.11)

One layer theory does not take into account the influence of different types of subgrade material. The effect of stiffer pavement layer is ignored which is unreasonable in the case of thick asphalt concrete surface layers.

3.0 TWO - LAYERED THEORY

3.1 Background

Developed by Burmister at Columbia University in the early 1940's; developed for airport pavement design; first solved problem for two layers in terms of deflection and conceptually established three-layer problem.

3.2 Assumptions

3.2.1 Constitutive Equations.

(a) Homogeneous
(b) Isotropic
(c) Linear elastic material

3.2.2 Governing Equations. The governing equations are related to the condition of static equilibrium of the element.

3.2.3 Boundary Conditions.

(a) Infinite lateral dimensions.
(b) Finite thickness of surface layer and bottom layer of infinite thickness.
(c) Upper layers weightless.
(d) Layers in continuous contact.
(e) Surface layer free of shearing.
(f) Full continuity at the interface (i.e., transfer of shear strain along the interface).

3.3 Burmister Stresses (Visual Aid 17.12)

The stress in Burmister's layered theory are dependent on; E_1/E_2 ratio.

3.4 Burmister Deflections

The equation for deflections are:

(a) Flexible plate equation.

$$\Delta = 1.5 \frac{pa}{E_2} F_2$$

(b) Rigid plate equation.

$$\Delta = 1.18 \frac{pa}{E_2} F_2 \quad \text{(Assuming Poisson's ratio = 0.5)}$$

where

p = unit load on circular plate

a = radius of plate

E_2 = modulus of elasticity of lower layer

F_2 = dimensionless factor (Visual Aid 17.13).

4.0 THREE-LAYER THEORY

Tabular solutions by Jones; also solved by Hank and Scrivner.

4.1 Solve for the following stresses (Visual Aid 17.14)

σ_{z1} - vertical stress at interface 1

σ_{z2} - vertical stress at interface 2

σ_{r1} - horizontal stress at bottom of layer 1

σ_{r2} - horizontal stress at bottom of layer 2

σ_{r3} - horizontal stress at top of layer 3

4.2 Stress solutions (For axisymmetric condition)

4.2.1 Expressed in terms of the following parameters.

k1 or K1 = E_1/E_2 ;

k2 or K2 = E_2/E_3 ;

a_1 or A = a/h_2 and

H = h_1/h_2 .

(Graphical solutions by Peatlic and tabular solutions by Jones).

4.2.2 Vertical stresses.

σ_{z1} = p (ZZ1)

σ_{z2} = p (ZZ2)

(Use graphs in Fig 2.9 and Table 2.3 presented in text of Reference 2, Yoder and Witzcak)

ZZ1 and ZZ2 are stress factors to be determined from the graphs.

4.2.3 Horizontal and Tangential Stresses. Solutions are for Poisson's ratio of 0.5.

$$\sigma_{z1} - \sigma_{r1} = p(ZZ1 - RR1)$$

$$\sigma_{z2} - \sigma_{r2} = p(ZZ2 - RR2)$$

$$\sigma_{z2} - \sigma_{r3} = p(ZZ2 - RR3)$$

(Use graphs in Fig 2.9 and Table 2.3 presented in text of Reference 2, Yoder and Witzcak)

4.2.4 Horizontal Strains. (Solutions for strains)

$$\varepsilon_{r1} = \frac{\sigma_{r1}}{E_1} - \mu_1 \frac{\sigma_{t1}}{E_1} - \mu_1 \frac{\sigma_{z1}}{E_1}$$

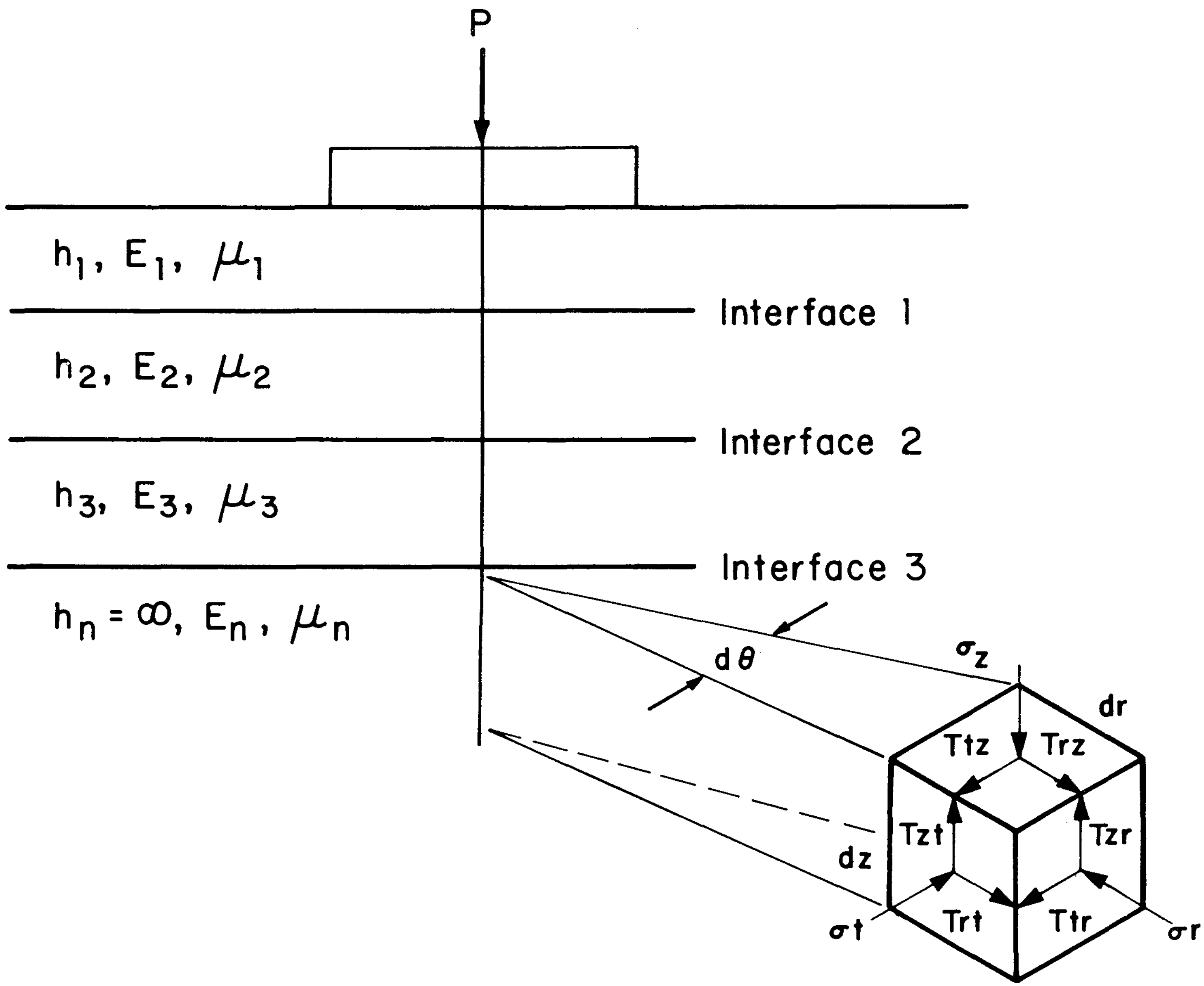
Due to symmetry $\sigma_{r1} = \sigma_{t1}$

Therefore, for $\mu = 0.5$;

$$\varepsilon_{r1} = \frac{1}{2E} (\sigma_{r1} - \sigma_{z1})$$

LESSON OUTLINE
FLEXIBLE PAVEMENT/ELASTIC LAYERED THEORY

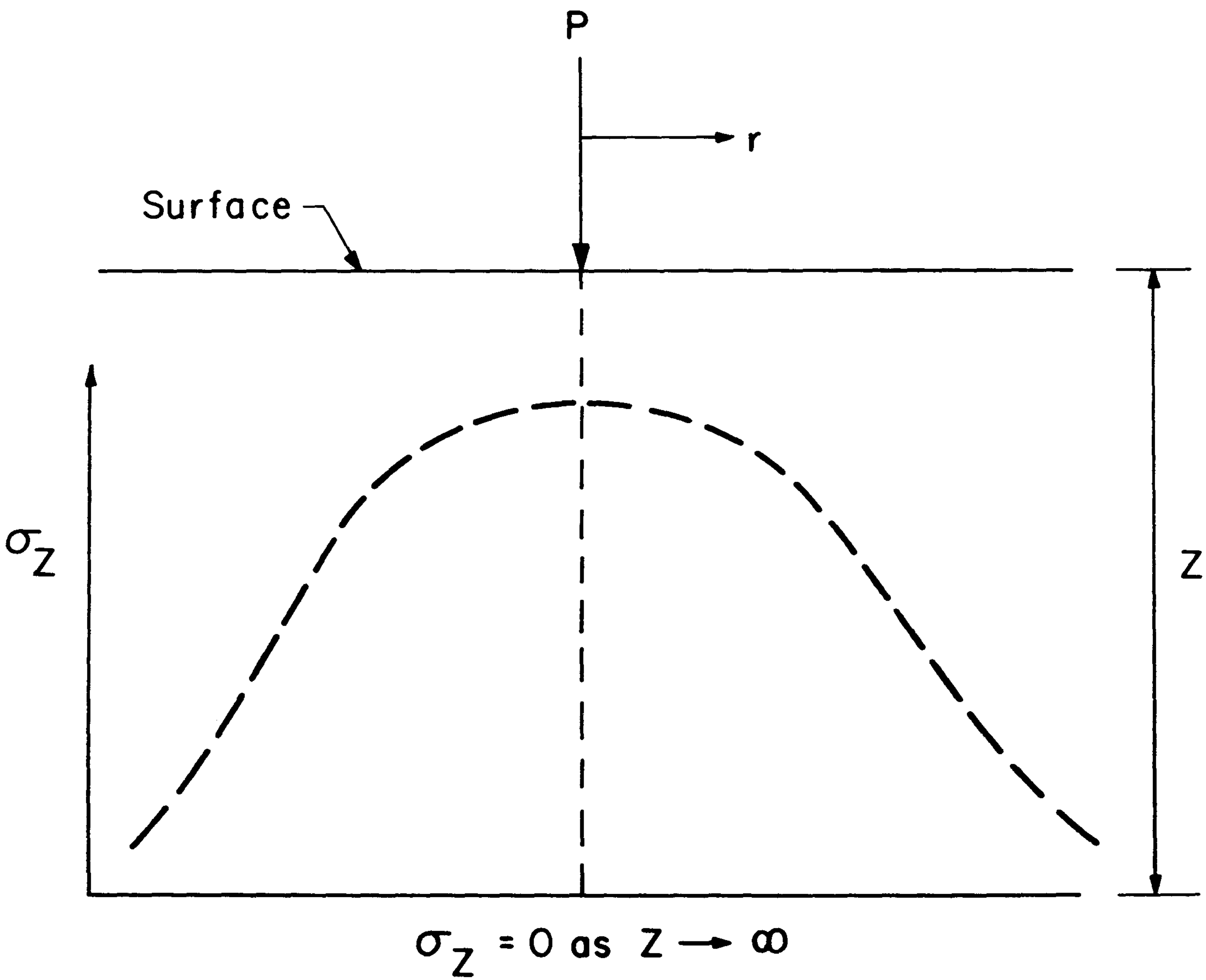
VISUAL AID	TITLE
Visual Aid 17.1.	Generalized multilayered elastic system.
Visual Aid 17.2.	Multilayered elastic system assumptions.
Visual Aid 17.3.	Point loading, Boussinesq One-Layer theory.
Visual Aid 17.4.	Area loading, Boussinesq One-Layer theory.
Visual Aid 17.5.	Deflection patterns for flexible and rigid plates.
Visual Aid 17.6.	Stresses in a One-Layered system.
Visual Aid 17.7.	One-Layer elastic equation.
Visual Aid 17.8.	Surface deflection assumption - One-Layer theory.
Visual Aid 17.9(a).	Influence chart for vertical stress, σ_z.
Visual Aid 17.9(b).	Influence chart for radial stress, σ_r.
Visual Aid 17.9(c).	Influence chart for horizontal stress, σ_t.
Visual Aid 17.9(d).	Influence chart for shear stress, τ_t.
Visual Aid 17.9(e).	Influence chart for vertical deflection, Δ.
Visual Aid 17.10.	Example problem; One-Layer solution.
Visual Aid 17.11.	Comparison of calculated and measured stress under 24-inches of aggregate base.
Visual Aid 17.12.	Burmister Two-Layer stress influence curves.
Visual Aid 17.13.	Burmister Two-Layer deflection influence curves.
Visual Aid 17.14.	Stress solutions in a Three-Layer system.



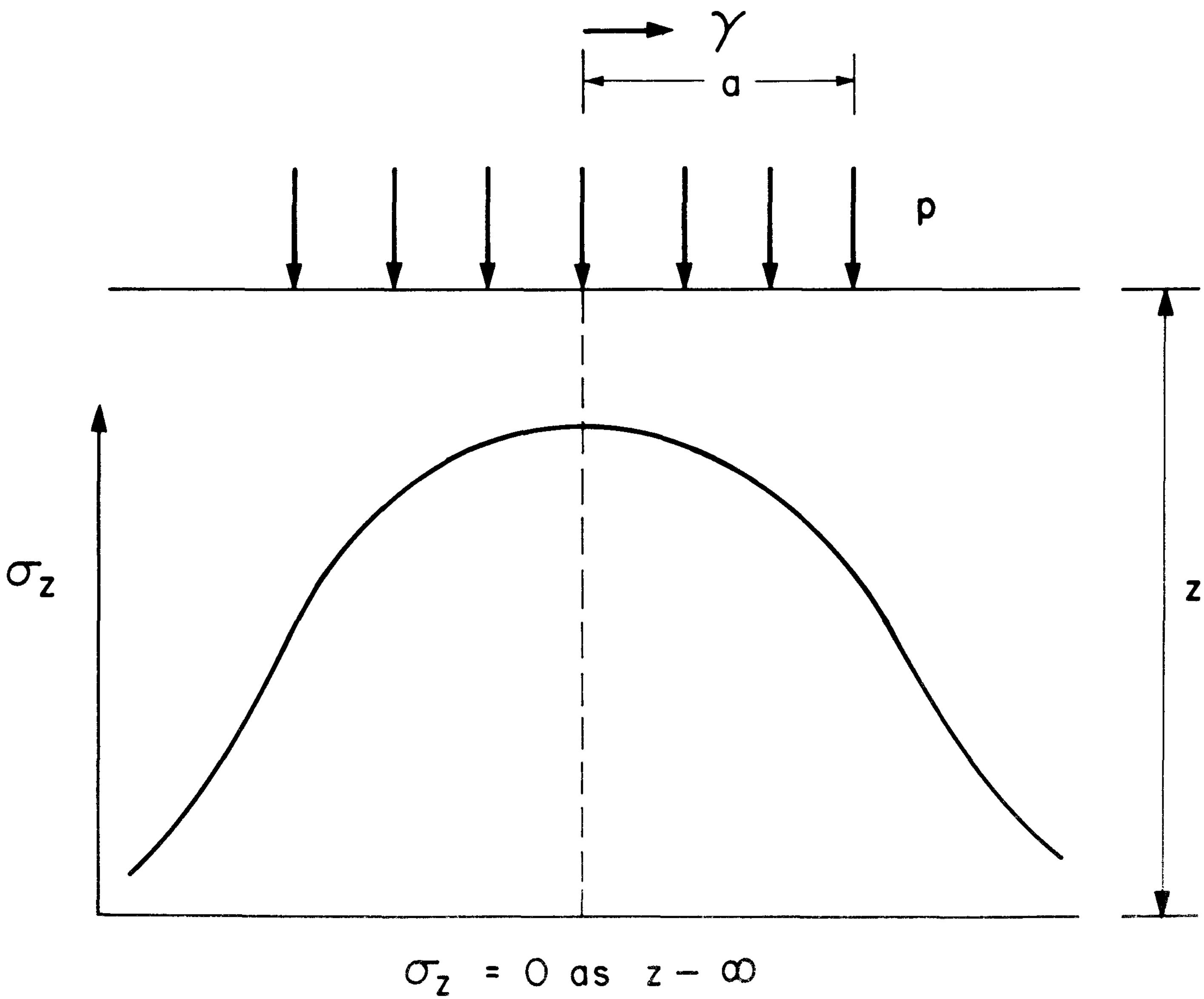
Visual Aid 17.1. Generalized multilayered elastic system.

Visual Aid 17.2. Multilayered elastic system assumptions.

1. HOMOGENEOUS PROPERTIES
2. FINITE THICKNESS OF LAYERS (EXCEPT BOTTOM LAYER)
3. INFINITE LATERAL DIMENSIONS
4. ISOTROPIC PROPERTIES
5. FULL FRICTION AT LAYER INTERFACES
6. NO SHEAR FORCES AT SURFACE
7. MATERIAL CHARACTERIZED BY POISSON'S RATIO (μ) AND ELASTIC MODULI (E)



Visual Aid 17.3. Point loading, Boussinesq One-Layer Theory.



Visual Aid 17.4. Area loading, Boussinesq One-layer theory.

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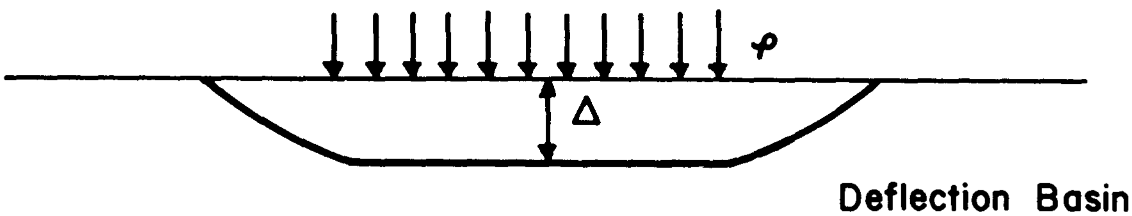
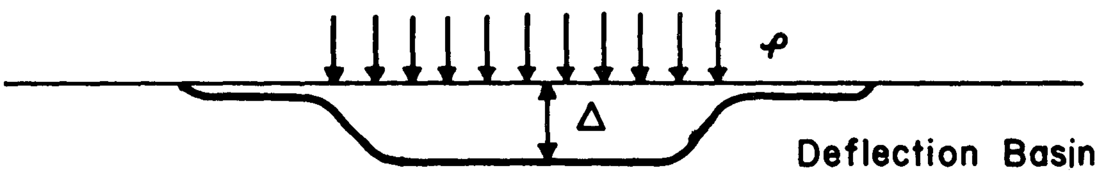
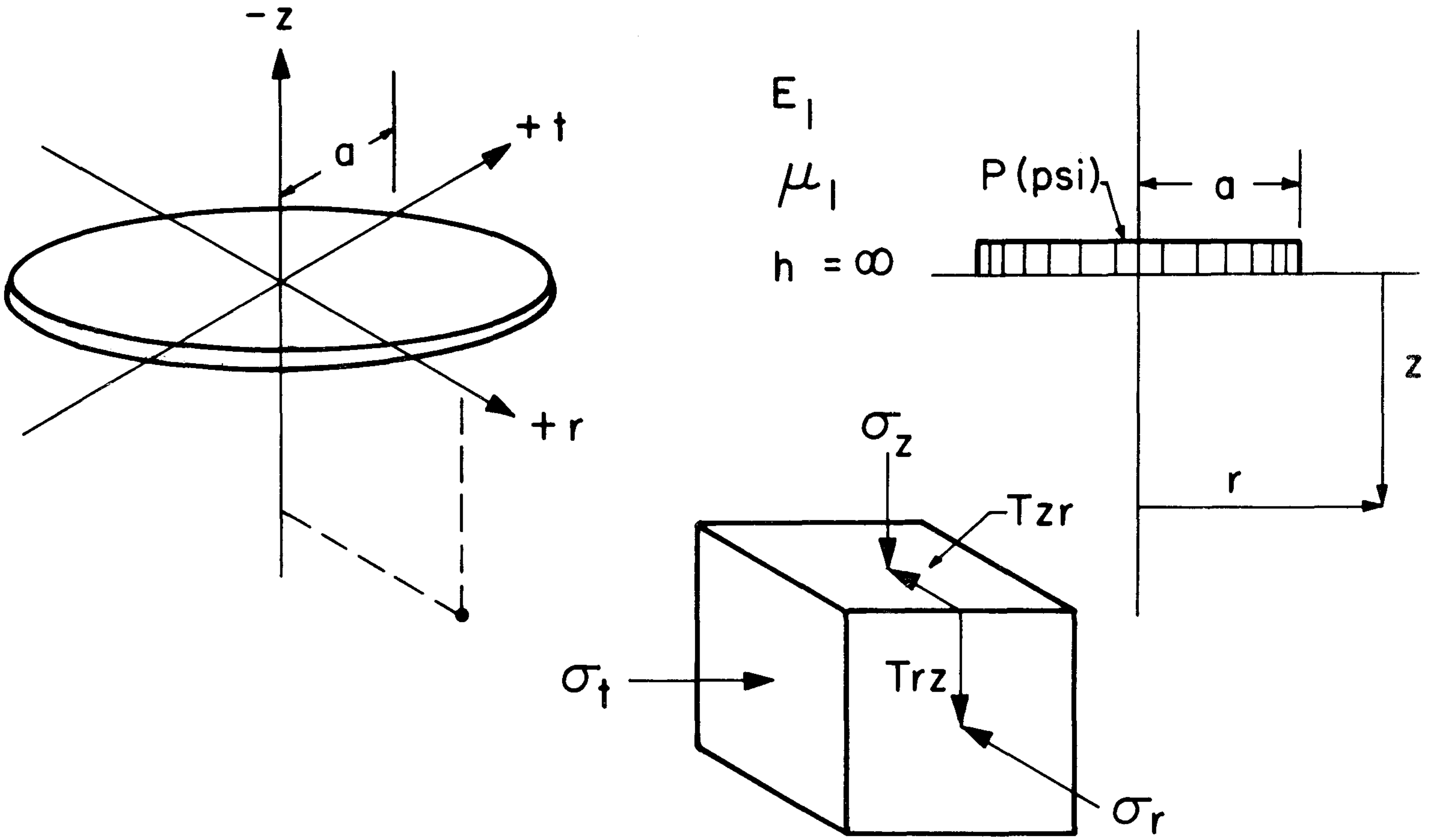
Visual Aid 17.5. Deflection patterns for flexible and rigid plates.



Flexible Plate Deflection



Rigid Plate Deflection

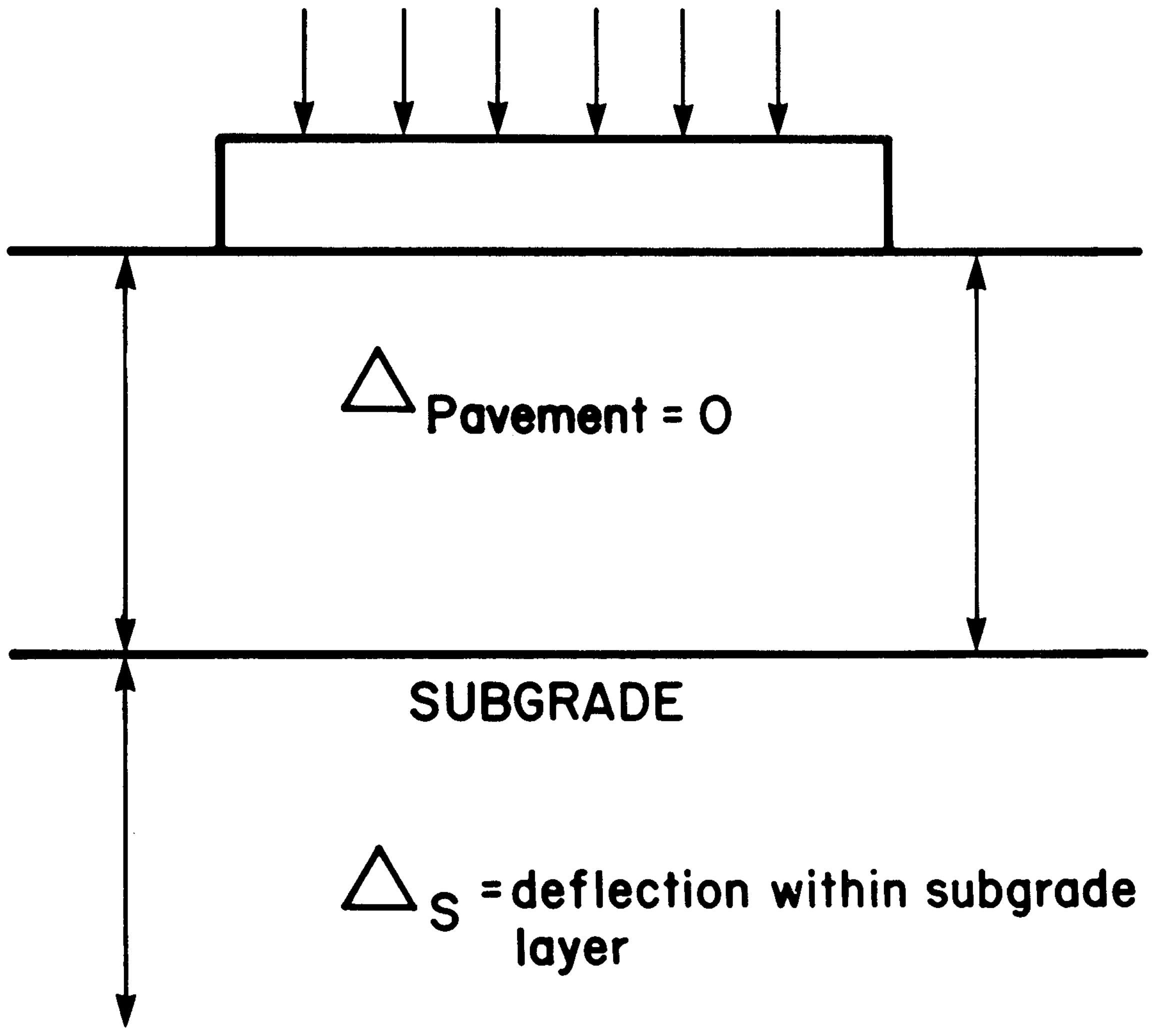


Visual Aid 17.6. Stresses in a One-Layer System.

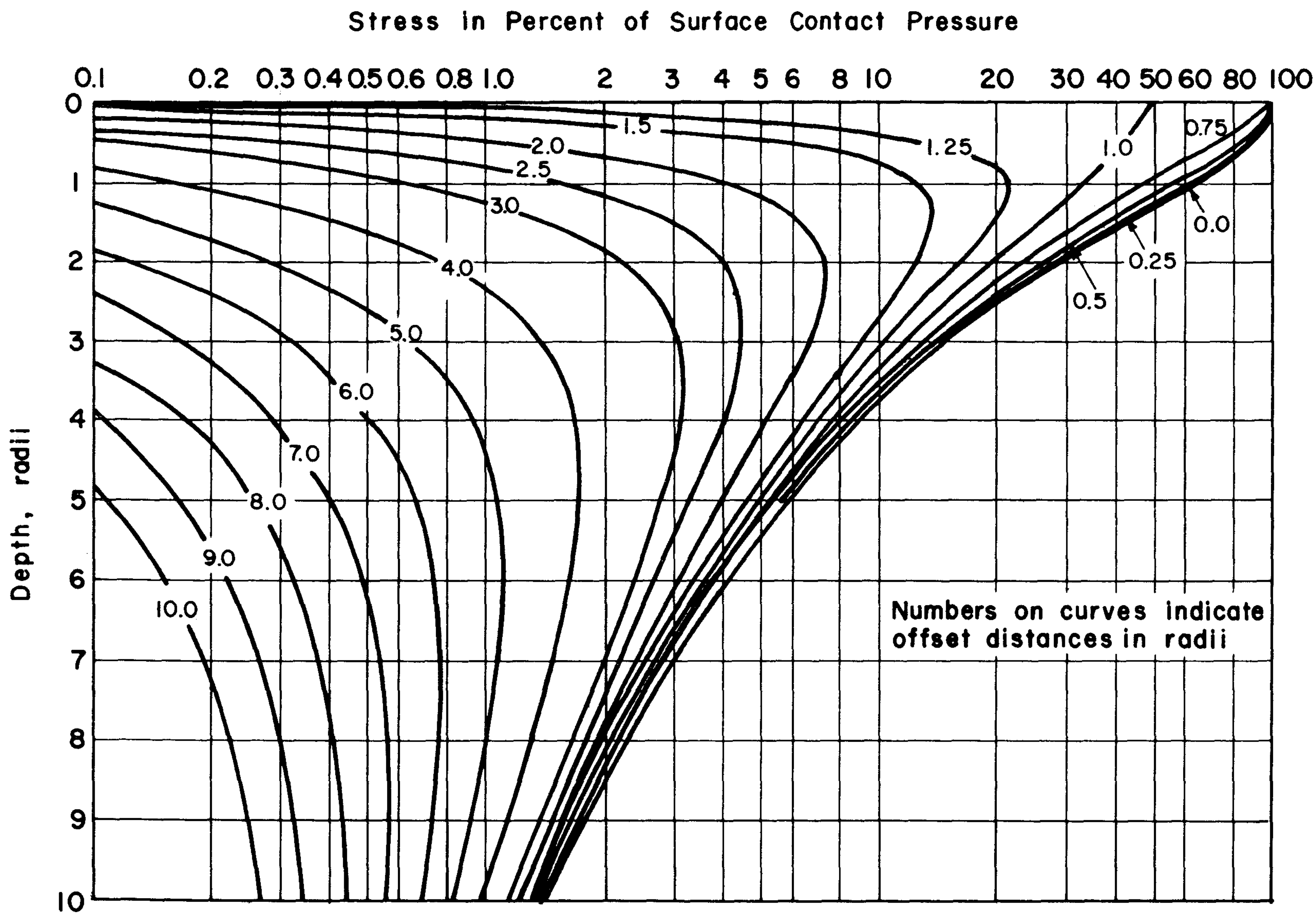
Parameter	General Case	Special Case ($\mu = 0.5$)
Vertical stress	$\sigma_s = p[A + B]$	(same)
Radial horizontal stress	$\sigma_r = p[2\mu\ A + C + (1 - 2\mu)\ E]$	$\sigma_r = p[\ A + C\ ]$
Tangential horizontal stress	$\sigma_t = p[2\mu\ A - D + (1 - 2\mu)\ E]$	$\sigma_t = p[\ A - D\ ]$
Vertical radial shear stress	$\tau_{rs} = \tau_{sr} = pG$	(same)
Vertical strain	$\varepsilon_s = \dfrac{p(1 + \mu)}{E_1}\ [(1 - 2\mu)\ A + B]$	$\varepsilon_s = \dfrac{1.5p}{E_1}\ B$
Radial horizontal strain	$\varepsilon_r = \dfrac{p(1 + \mu)}{E_1}\ [(1 - 2\mu)\ F + C]$	$\varepsilon_r = \dfrac{1.5p}{E_1}\ C$
Tangential horizontal strain	$\varepsilon_t = \dfrac{p(1 + \mu)}{E_1}\ [(1 - 2\mu)\ E - D]$	$\varepsilon_t = \dfrac{1.5p}{E_1}\ D$
Vertical deflection	$\Delta_s = \dfrac{p(1 + \mu)a}{E_1}\left[\dfrac{z}{a}\ A + (1 - \mu)\ H\right]$	$\Delta_s = \dfrac{1.5pa}{E_1}\left(\dfrac{z}{a}\ A + \dfrac{H}{2}\right)$
Bulk stress	$\theta = \sigma_s + \sigma_r + \sigma_t$	
Bulk strain	$\varepsilon_\theta = \varepsilon_s + \varepsilon_r + \varepsilon_t$	
Vertical tangential shear stress	$\tau_{st} = \tau_{ts} = 0$ [σ_t (ε_t) is principal stress (strain)]	
Principal stresses	$\sigma_{1,\ 2,\ 3} = \dfrac{(\sigma_s + \sigma_r) \pm \sqrt{(\sigma_s - \sigma_r)^2 + (2\tau_{rs})^3}}{2}$	
Maximum shear strain	$\tau_{max} = \dfrac{\sigma_1 - \sigma_3}{2}$	

Visual Aid 17.7. One-layer elastic equations (Ref 1).

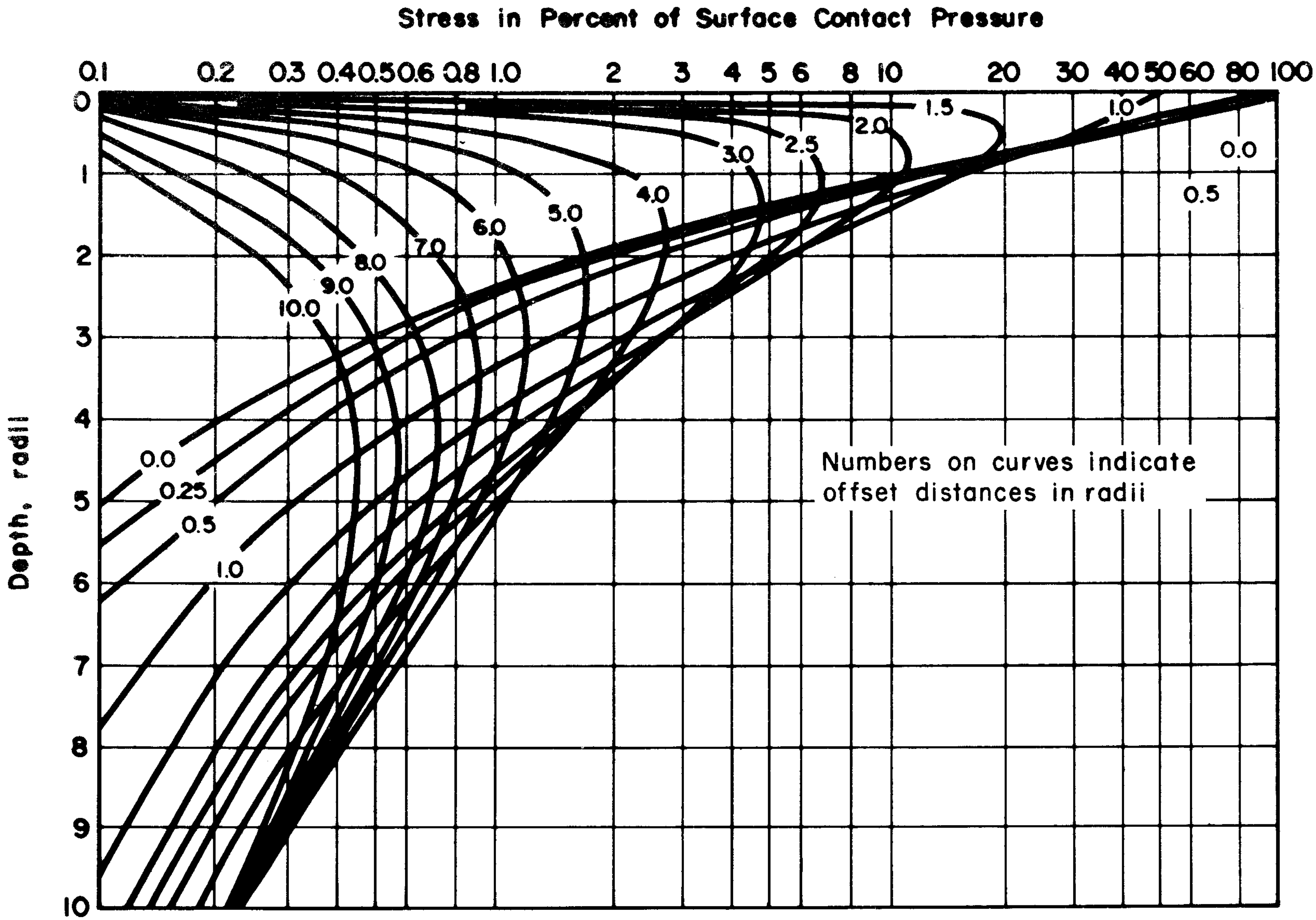
Visual Aid 17.8 Surface deflection assumption, One-layer theory.



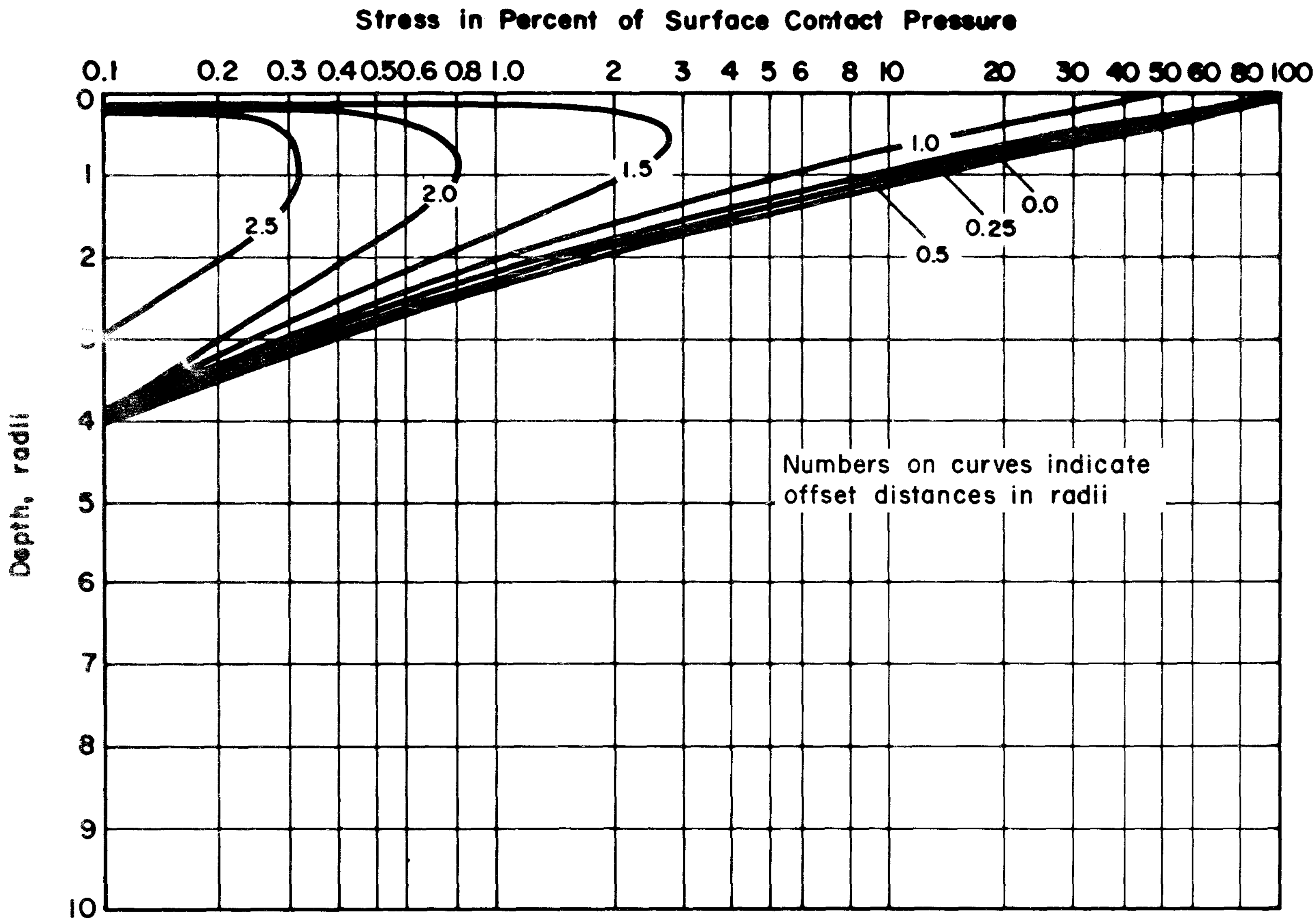
$$\Delta_T = \Delta_S + \cancel{\Delta_P}^{\,0} = \Delta_S$$



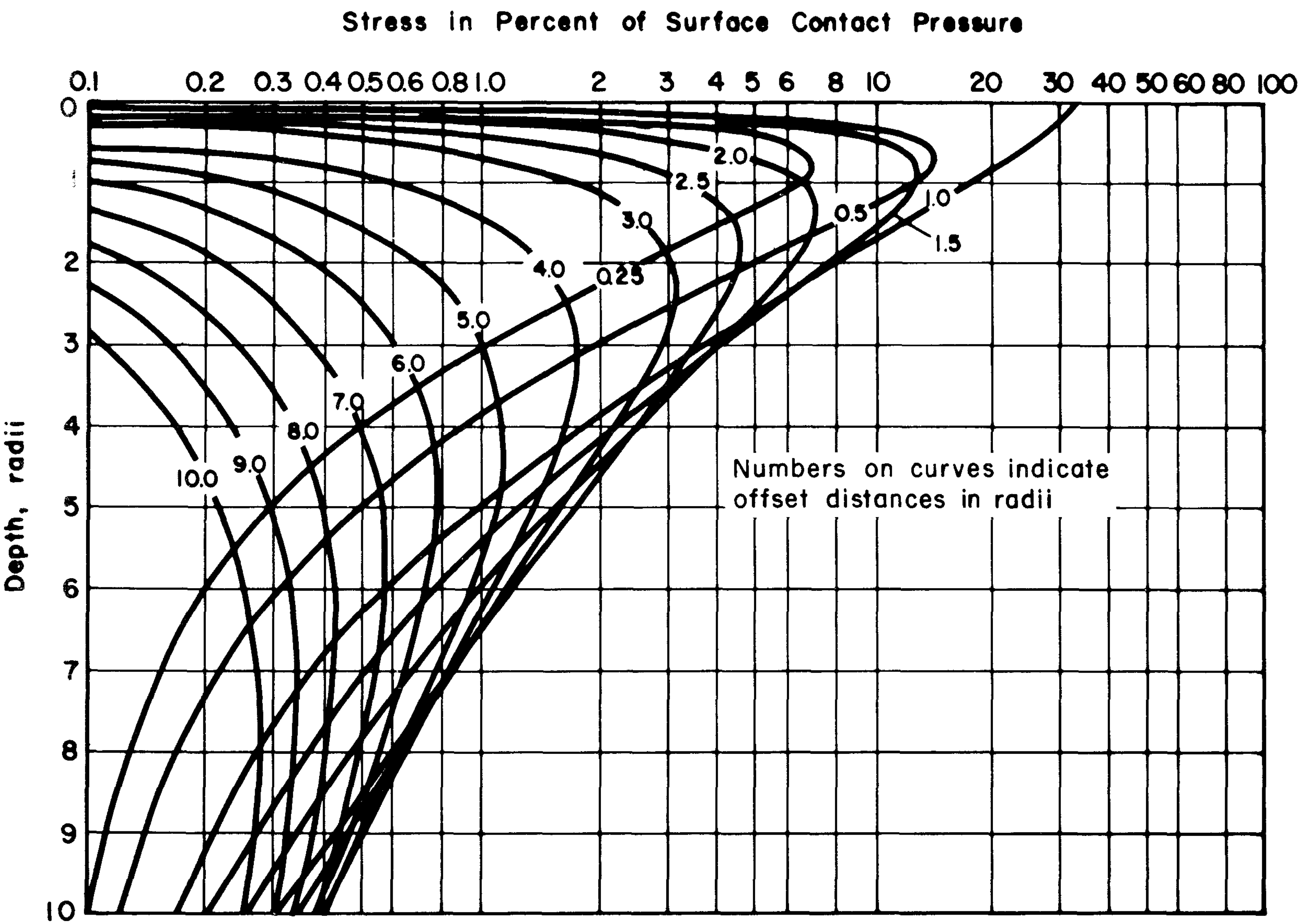
Visual Aid 17.9(a). Influence chart for vertical stress σ_z.



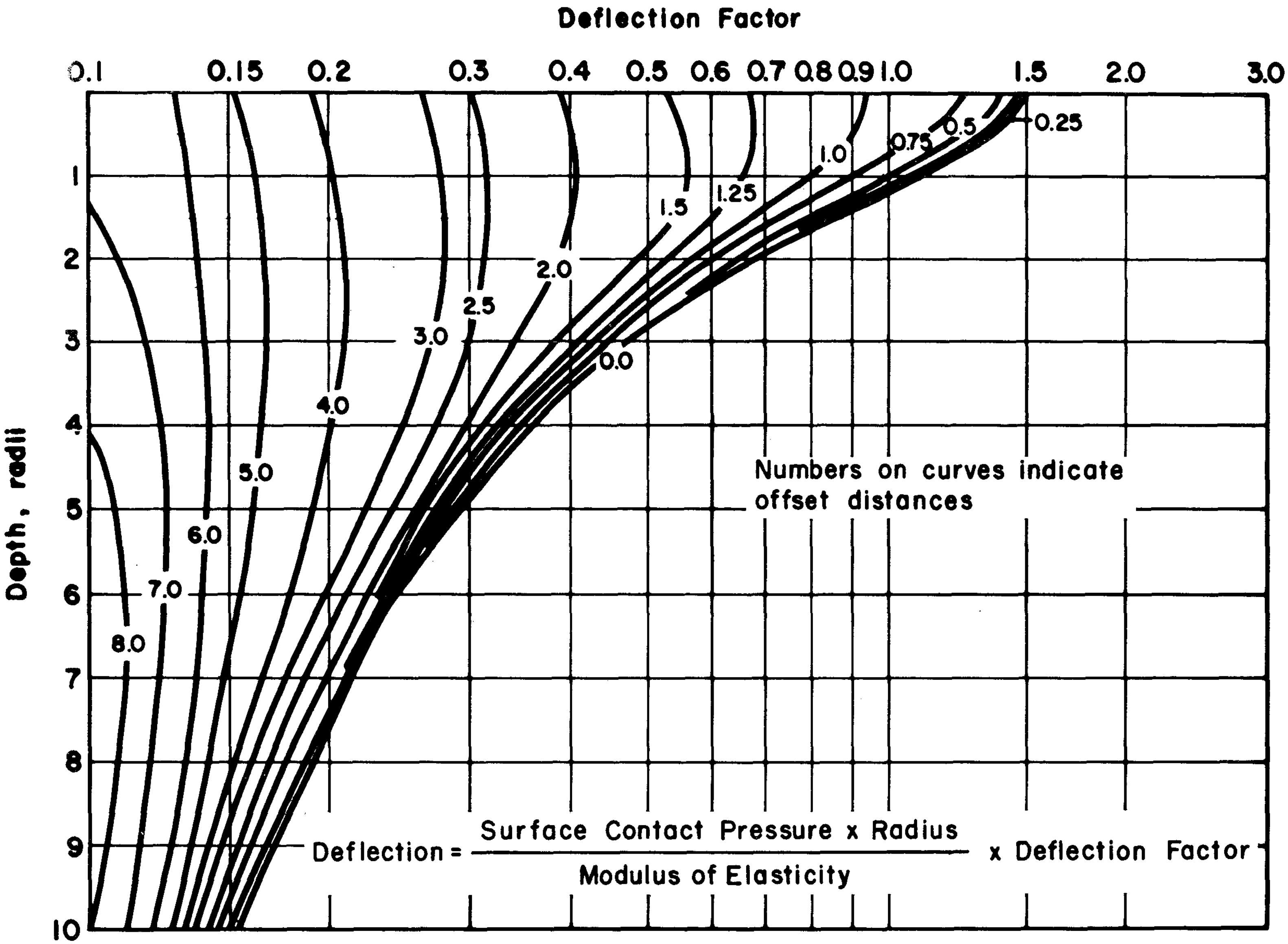
Visual Aid 17.9(b). Influence chart for radial stress σ_r.



Visual Aid 17.9(c). Influence chart for horizontal stress σ_t.

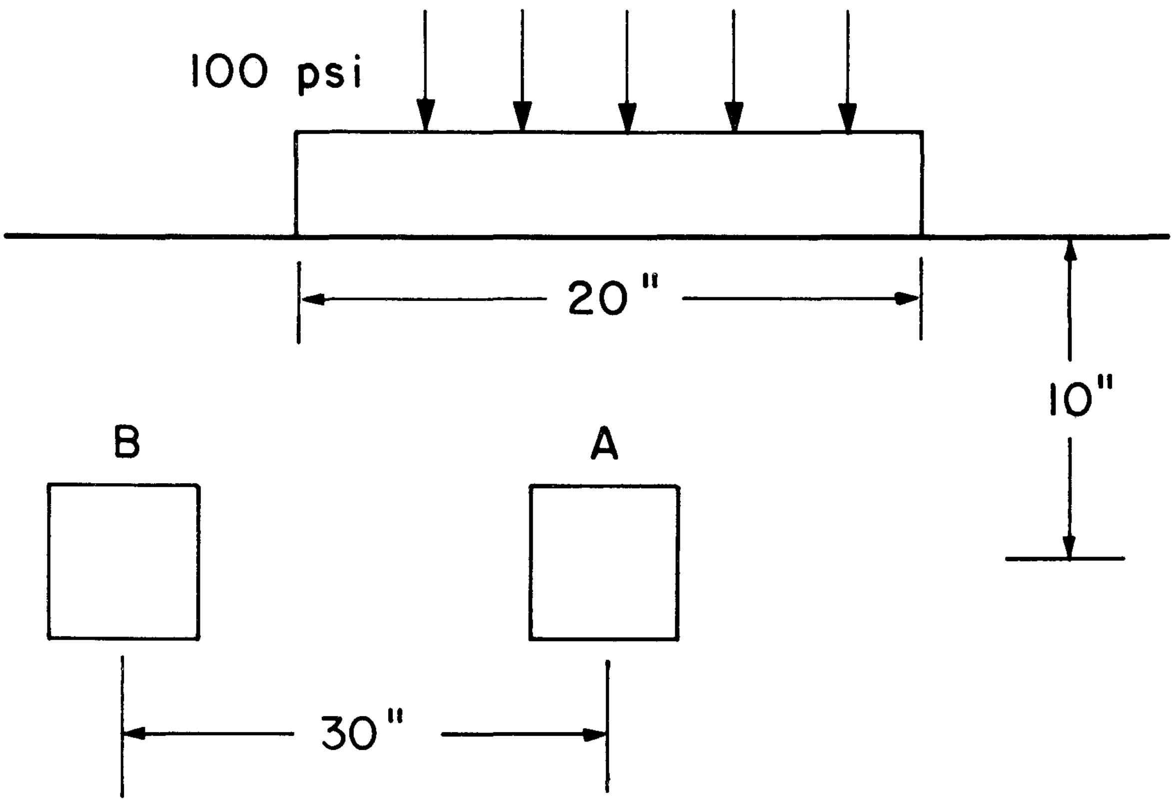


Visual Aid 17.9(d). Influence chart for shear stress, τ_t.



Visual Aid 17.9(e). Influence chart for vertical deflection, Δ.

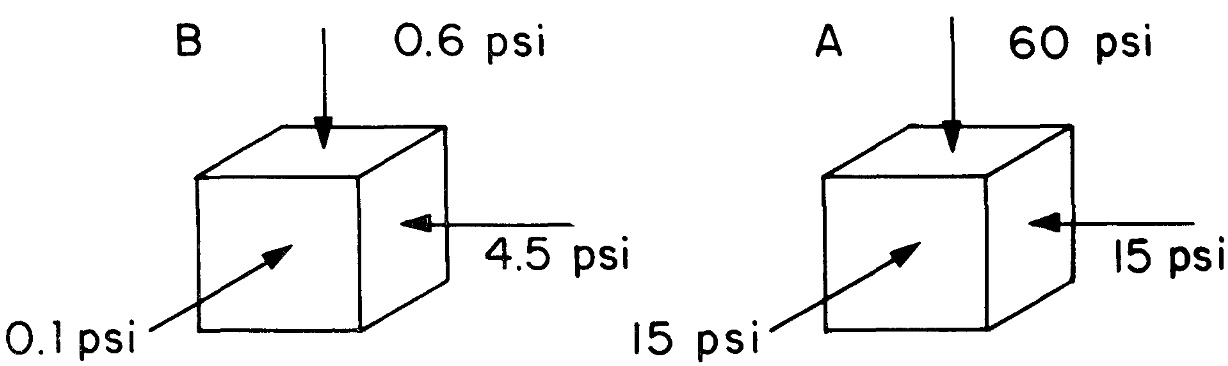
Visual Aid 17.10. Example Problem; One-layer stress condition.

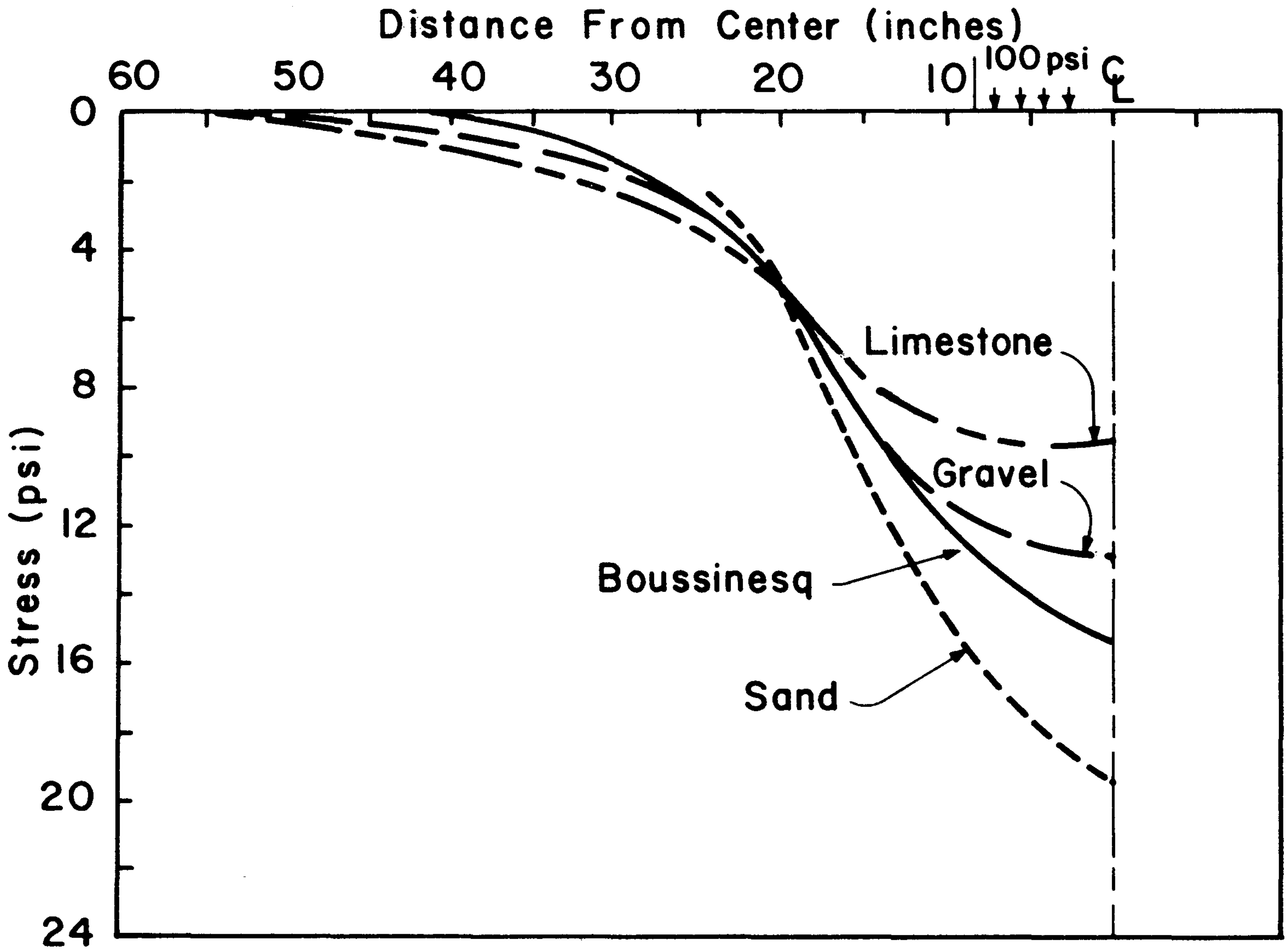


$z/a = 10/10 = 1$

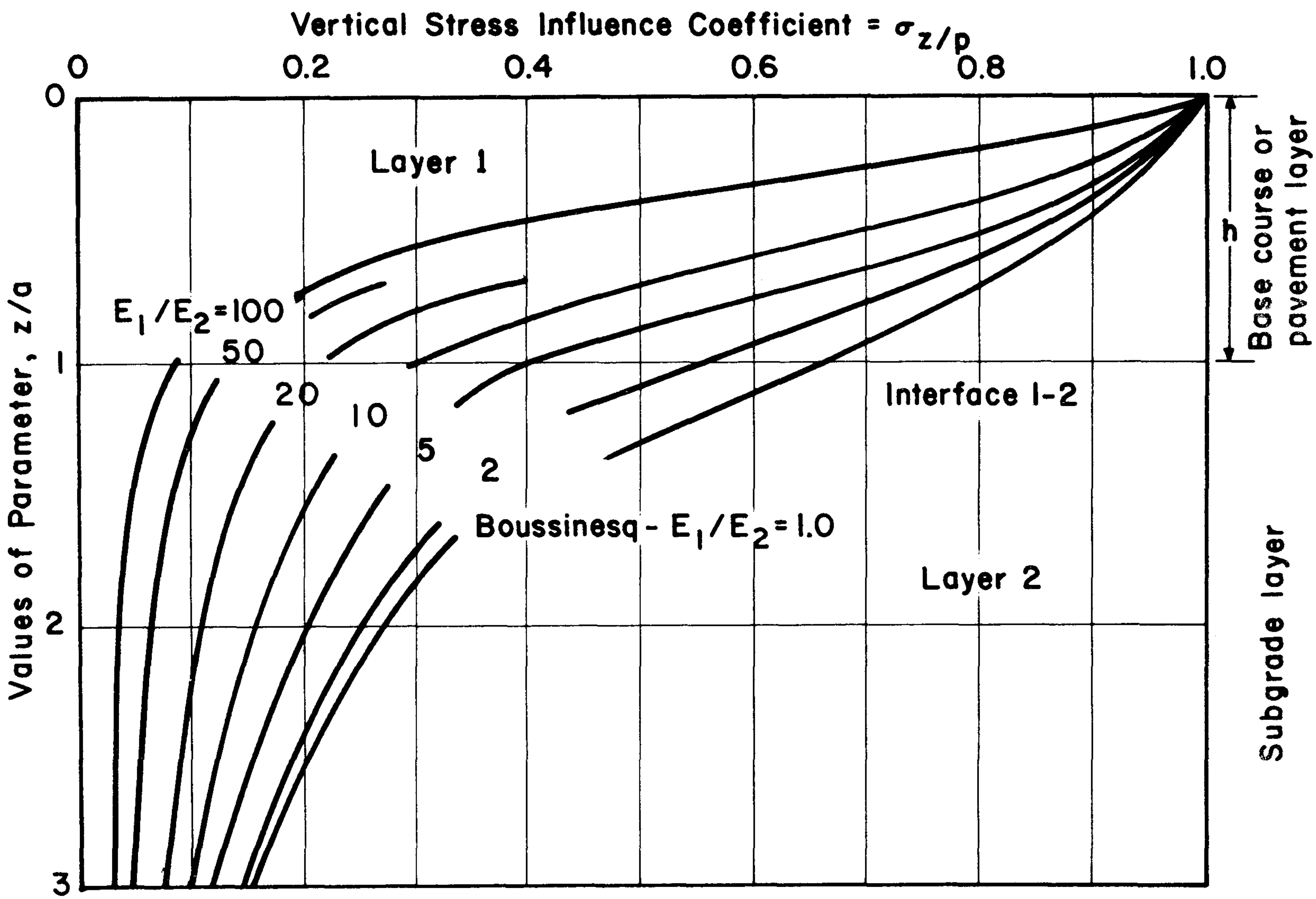
Case A $r/a = 0$

Case B $r/a = 30/10 = 3$

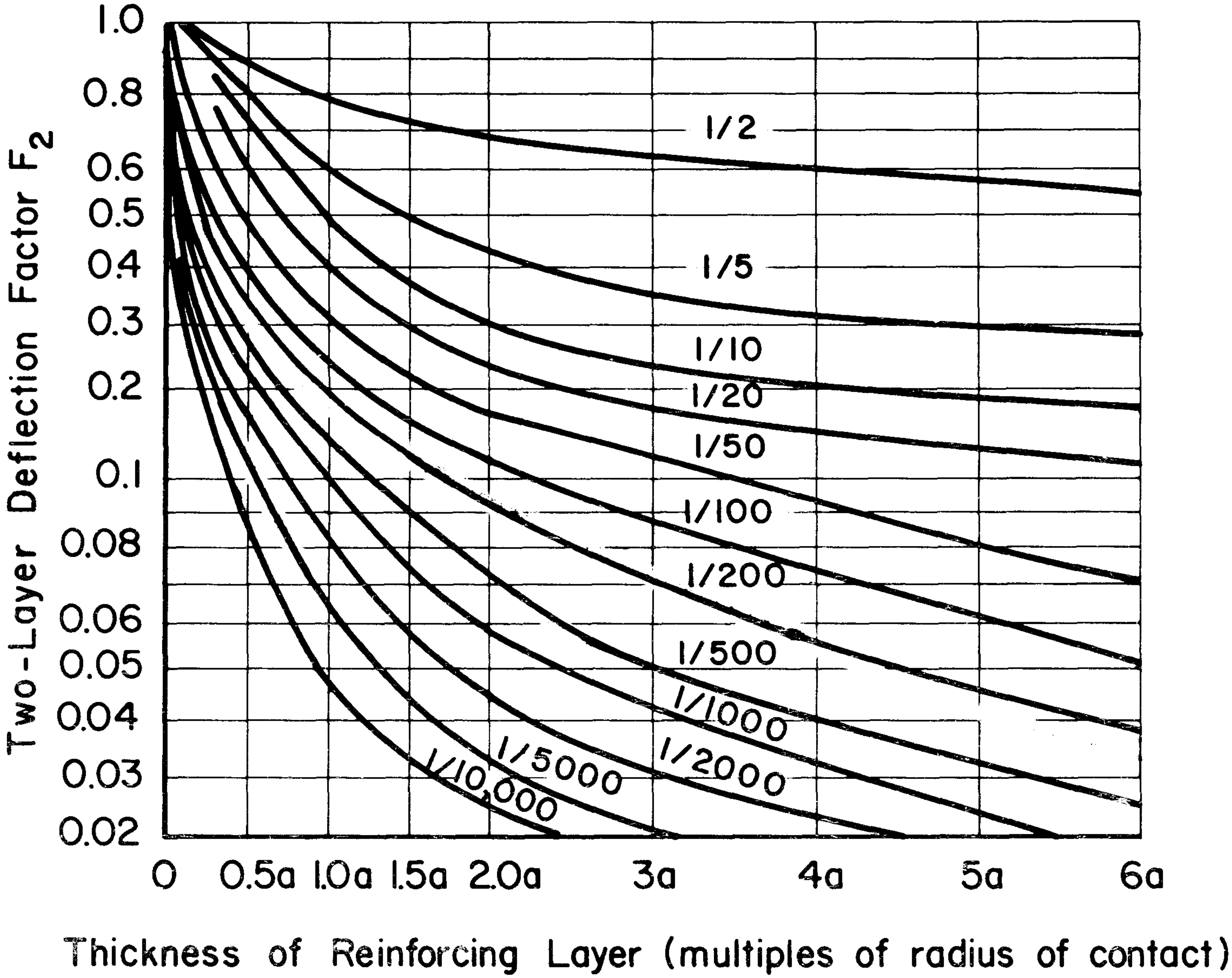




Visual Aid 17.11. Comparison of calculated and measured stress under 24-inches of aggregate base.



Visual Aid 17.12. Burmister Two-Layer stress influence curves.



Visual Aid 17.13. Burmister Two-Layer deflection influence curves.

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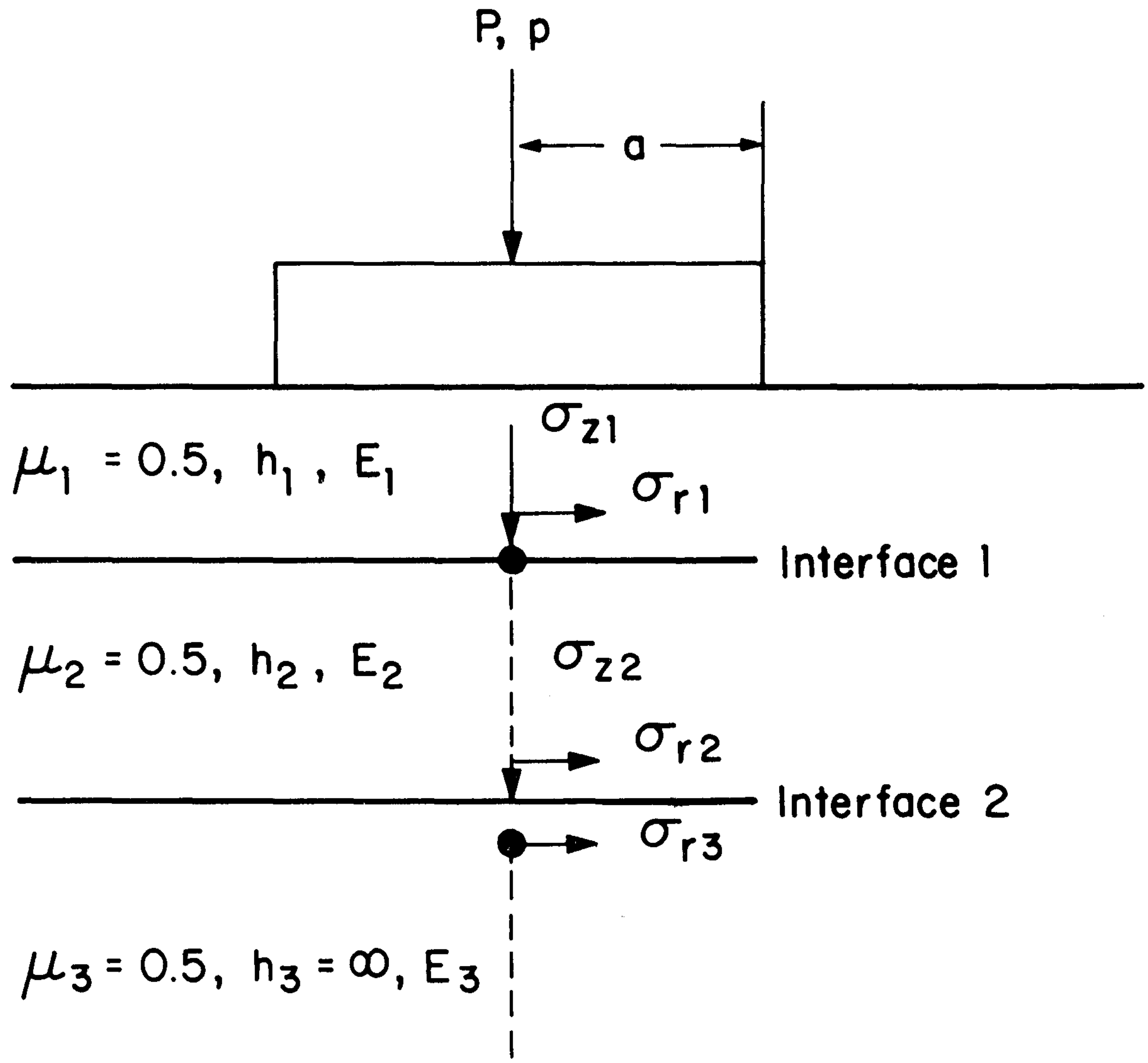
Visual Aid 17.14. Stress solutions in a Three-layer system.



LESSON OUTLINE
COMPUTER PROGRAMS FOR THE ANALYSIS OF ELASTIC LAYERED SYSTEMS

Instructional Objectives

1. To provide the student with a basic understanding of the most commonly used computer routines for the analysis of elastic layered systems.

Performance Objectives

1. The student should be able to explain the assumptions behind each of the computer routines.

2. The student should be able to explain the advantages and limitations of each of the computer routines.

3. The student should be able to analyze an elastic layered system using at least one of the routines discussed.

Abbreviated Summary	Time Allocations, min.
1. Background	10
2. Layer5	5
3. Layer15	5
4. Layit	5
5. ELSYM5	10
6. BISAR	10
7. Comparison	5
	50 minutes

Reading Assignment

1. Haas & Hudson - Chapter 13, pages 139-150

2. Instructional Text

LESSON OUTLINE
COMPUTER PROGRAMS FOR THE ANALYSIS OF ELASTIC LAYERED SYSTEMS

1.0 BACKGROUND

1.1 Basis for Structural Models

Most programs are based on Burmeister's work for the layers, extended to more than three layers.

1.1.1 Material Properties. The models assume that materials properties are characterized by linear elastic, homogenous and isotropic behavior.

1.1.2 Subgrade. Depending on the model, the subgrade is assumed to be semi-infinite.

1.2 Improvements (Extensions) on Burmister's Work

1.2.1 Five or More Layers Analyzed. Since the advent of high speed computers, the once laborious calculation required by Burmeister's layered theory can easily be extended to multiple layers.

1.2.2 Principle of Superposition Used for Multiple Loads. The use of this principle allows the influence of multiple loads to be examined.

1.2.3 Slippage Between Layers Also Considered in Some Programs. The effect of shear strength (or lack of) between layers can be studied with some select models.

1.2.4 Non-vertical Loading. Eccentric loading is a variable whose effect can now be analyzed.

1.3 Difference in Capabilities

All computer models use the same theory but vary in their capabilities for handling multiple layers, multiple loads, load orientation, etc.

1.4 Model Complexity

On most cases as the complexity of the model increases, the degree of difficulty in using the model increases.

1.5 Evaluation of Models

When deciding which model to use, the engineer should consider ease of data input, ease of output interpretation, computer time costs and the assumptions and limitations of the model.

2.0 LAYER5

LAYER5 was developed by Chevron Research Company and has the capability of analyzing stresses and displacements in a 5-layered elastic system under a uniformly distributed load on a circular loaded area.

2.1 Assumptions

(a) weightless layers
(b) material linearly elastic, homogenous, and isotropic obeying Hooke's law
(c) uniform thickness of layers with infinite lateral dimensions
(d) single vertical load uniform over a circular area
(e) governing equation is equilibrium
(f) boundary conditions

 (1) no slip at layer interfaces
 (2) surface free of stresses outside loaded area
 (3) stresses and strains zero at infinite depth

2.2 Required Input

(a) vertical, tangential, radial, shear, and bulk stresses
(b) vertical displacement
(c) radial and tangential shear strain

3.0 LAYER 15

LAYER 15 was also developed by Chevron Research Company in California. It is basically an extension of LAYER5 to 15 layers.

4.0 LAYIT

Layered elastic theory assumes that materials can be characterized by their elastic constants: modulus of elasticity and Poisson's ratio. However, most unbound materials used in pavements are stress sensitive. LAYIT takes stress sensitivity into account through an iterative procedure. This model is a nonlinear program developed by Chevron Research Company.

4.1 Assumptions

(a) Material characterized by elastic modulus and Poisson's ratio.
(b) Unbound Materials Stress Sensitive.

$$E_R = A(\theta)^B$$

where

E_R = resilient modulus

A = intercept of best-fit line

B = slope of best-fit line

θ = sum of principal stresses (lab)

(c) Iterative process (Visual Aid 18.1)
(d) Consideration of gravity stress (over burden pressure).
(e) Required input

(1) Wheel load and tire pressure
(2) Layer data

-- thickness,
-- Poisson's ratio,
-- initial estimate of elastic modulus,
-- A and B values for top layers, and
-- values for E_R versus deviator stress for subgrade (Visual Aid 18.2).

(f) Output

(1) vertical, tangential, radial, shear, and bulk stresses,
(2) vertical displacement, and
(3) radial and tangential shear strain.

5.0 ELSYM5

The ELSYM5 program was developed by the Institute of Transportation and Traffic Engineering, University of California. This program calculates the various component stresses, strains, and deflections, along with principle values in a three dimensional elastic layered system.

5.1 Modification of ELSYM5 (Improvements)

(a) Multiple loads by principle of superposition.
(b) Consideration of rigid base below subgrade.
(c) Cartesian coordinate system.

5.2 Assumptions

(a) Weightless layers.
(b) Linear elastic homogeneous, isotropic material that obeys Hooke's Law.
(c) Uniform thickness of layers with infinite laterial dimensions.
(d) Boundary conditions

    (1) full friction at layer interfaces,
    (2) option of zero friction between bottom layer and rigid base, and
    (3) surface free of shear.

(e) Vertically applied loads over circular area.

5.3 Required Input

(a) Any two of load magnitude, tire pressure, and load radius.
(b) Load positions.
(c) Layer data for up to 5 layers

    (1) thickness,
    (2) Young's moduli, and
    (3) Poisson's ratio.

(d) Location of responses to be determined.
(e) Friction at rigid base interface (full or none).

5.4 Output (Responses at Desired Locations)

(a) Principal stresses and strains.
(b) Normal Stresses and strains.
(c) Displacements.

6.0 BISAR (BISTRO)

The BISAR model was developed by Shell Research. This program computes stresses, strains and deflections in elastic multilayered systems subjected to one or more vertical or unidirectional tangential load.

6.1 Advantages Over Others

(a) Consideration of tangential loads, and
(b) Capability of variable friction at interface.

6.2 Uses Burmister's Theory

6.3 Assumptions

(a) Weightless layers.
(b) Material linearly elastic, homogeneous, isotropic obeying Hooke's Law.
(c) Uniform thickness of layers with infinite lateral dimensions.
(d) Boundary conditions

    (1) continuous vertical normal stresses,
    (2) continuous shear stress,
    (3) vertical contact maintained,
    (4) horizontal displacements proportional to shear stress, and
    (5) interface friction varies between full and none.

(e) Bottom layer semi-infinite.

6.4 Required Input

(a) Layer data, (up to 10 layers)

    (1) thickness,
    (2) elastic moduli, and
    (3) Poisson's ratio.

(b) Friction at each interface.
(c) Location, magnitude, and orientation of up to ten loads.
(d) Type and location of desired responses.

6.5 Output (Depends on What Was Specified)

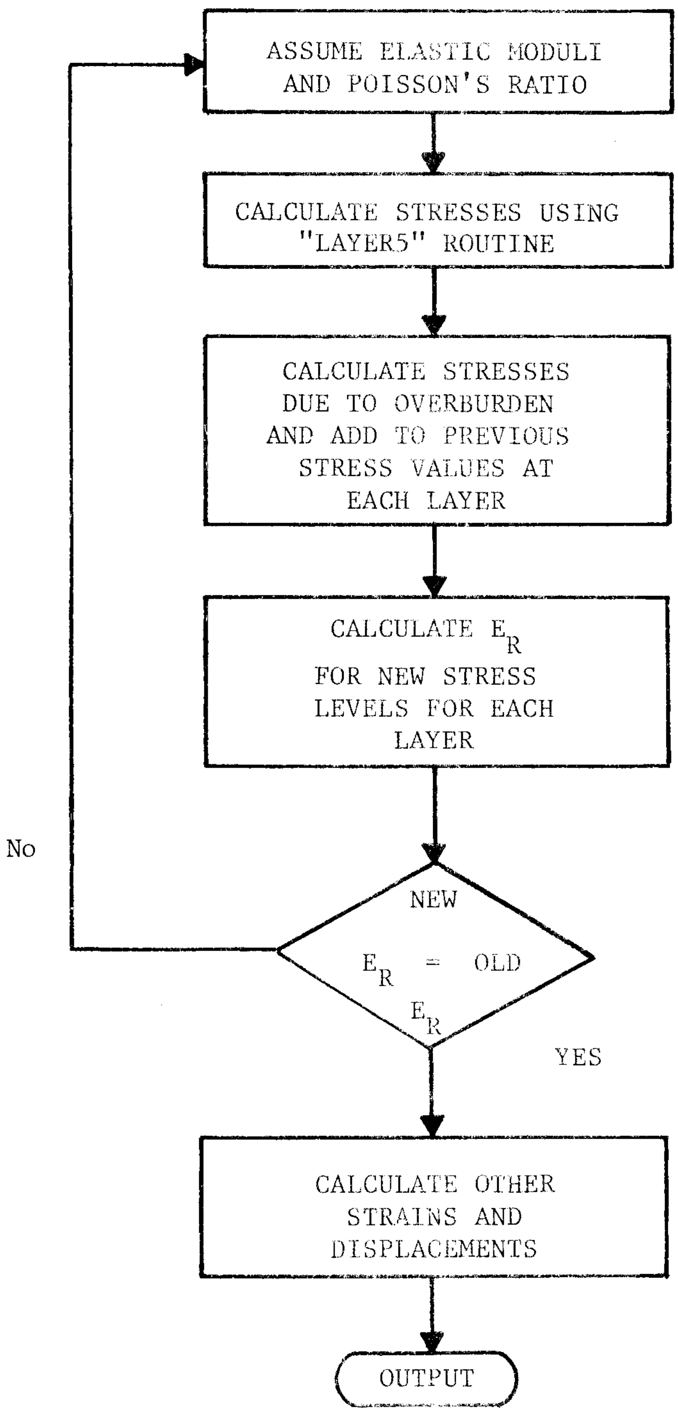
(a) Cylindrical components of stress, strain, and displacement for each load.
(b) Cartesian coordinates of stress, strain, and displacement.
(c) Principal stresses and strains.
(d) Maximum Shear Stresses and strains.
(e) Midpoints of Mohr stress circles.
(f) Strain energy of distortion.
(g) Total strain energy.

7.0 COMPARISON

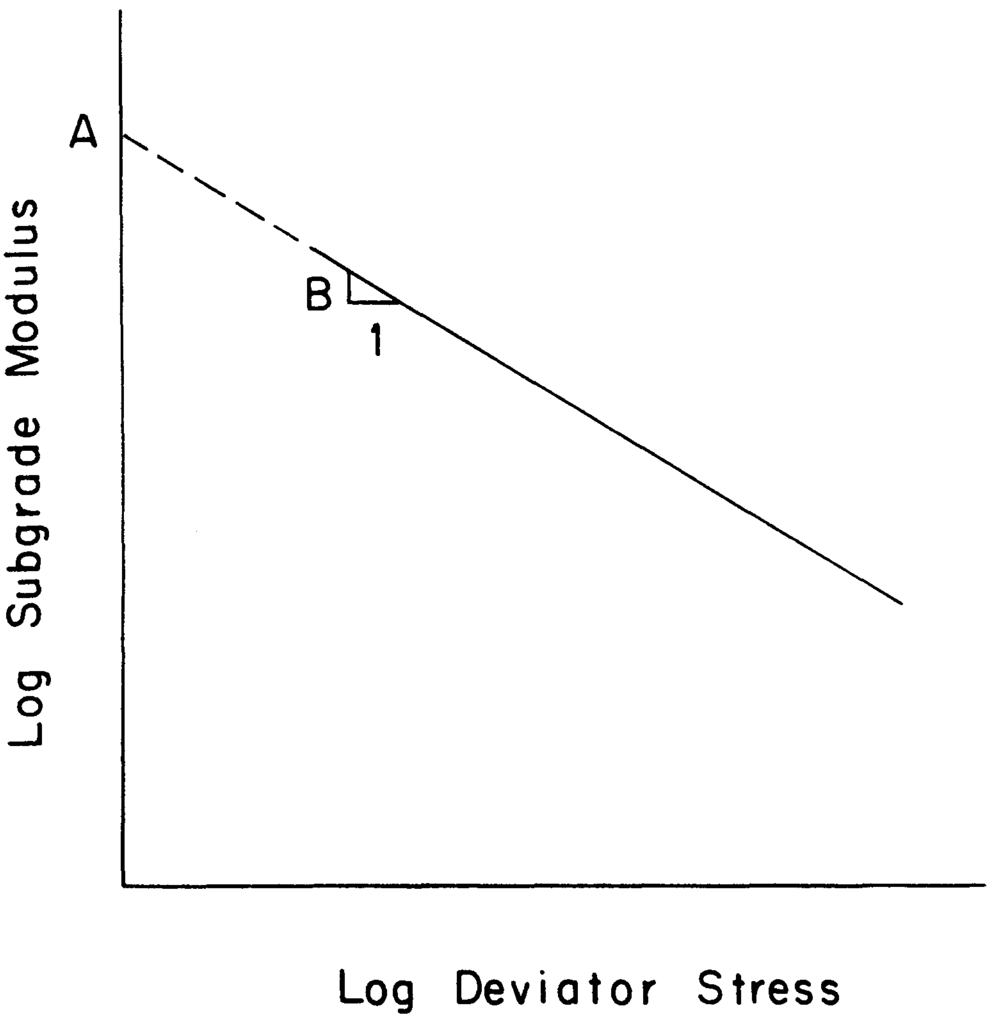
A comparison of the various layer programs is presented in tabular form in Visual Aid 18.3. Only items which bear comparison are entered on the table; thus, items which are common to all the programs are omitted.

LESSON OUTLINE
COMPUTER PROGRAMS FOR THE ANALYSIS OF ELASTIC LAYERED SYSTEMS

Visual Aid 18.1. Algorithm for LAYIT.



Visual Aid 18.2. Stress sensitivity of the subgrade modulus.



Comparison	LAYERS	LAYER15	LAYIT	ELSYM5	BISAR
1. Material property assumptions	Weightless, linear elastic	Weightless, linear elastic	Weightless, linear elastic	Weightless, linear elastic	Weightless, linear elastic
2. Maximum number of layers	5	1	1	10	10
3. Maximum number of loads	1	1	1	10	10
4. Horizontal loads possible	No	No	No	No	Yes
5. Rigid base possible	No	No	No	Yes	No
6. Material stress sensitivity considered	No	No	Yes	No	No
7. Variable friction at interfaces	No	No	No	Only at Rigid Base Interface	Yes
8. Principle stresses calculated	No	No	No	Yes	Yes
9. East of input	Easy	Easy	Easy	Easy	Difficult
10. Output interpretation	Easy	Easy	Easy	Easy	Difficult
11. Relative computation time	1	0.9-1.1	1.1-1.3	1.2-1.4	3.8-4.6

Visual Aid 18.3. Comparison of the various layer programs.

INSTRUCTIONAL TEXT

COMPARISON OF STRESSES, STRAINS AND DEFLECTIONS CALCULATED WITH VARIOUS LAYER PROGRAMS

By

Otto Schnitter
Graduate Research Assistant
University of Texas at Austin

1977

CONTENTS

CHAPTER I INTRODUCTION

Boundary value problems for layered theory have been solved through the use of both conventional numerical techniques and finite element techniques (Ref 1). The full developement of the solutions with conventional numerical techniques only become feasable with the advent of the computer.

Boussinesq developed a linear elastic layer theory for analysis of pavements, which may be considered as a one-layer problem. He assumed that the half space, on which a vertical load is applied, is perfectly elastic and homogeneous. Stresses and deflection can be obtained anywhere in the half space. This theory did not take the material properties into account and is not considered to be useful.

In the early 1940's Burmeister (Ref 1), laid the foundation for solution of elastic layers on a semi-infinite elastic subgrade. He first solved the boundary value problem for two layers, assuming continuous interface, and conceptually established the solutionsof three layered problems.

In the development of this theory, Burmeister assumed that each layer is homogeneous, isotropic and linear elastic and that each layer extends infinitely in the horizontal direction. Each layer is assumed to have a finite thickness, except for the bottom layer which is of infinite depth. The boundary and continuity conditions require the layers to be in continuous contact and that there is no shear nor normal stress on the surface outside the loaded area.

The load is assumed to be a uniformly distributed vertical pressure over a circular loaded area. Burmeister also assumed that the materials are weightless (Ref 1) and that the stress and deflection in the bottom layer must be equal to zero at infinite depth.

Since Burmeister's original development of layered theory (Ref 2), several important extensions of the theory have been developed and incorporated in computer programs. Most elastic layered theory programs are capable of analyzing at least five layers . Some programs use the principle of superposition in order to consider the effect of multiple loads. At least one computer program (BISAR) is capable of analyzing the effect of slippage between the layers and non vertical loading conditions. Iterative techniques have been developed to analyse non-linear elastic materials.

Objectives

The objective in this study is to compare the stresses, strains and deflections obtained from various Layer Programs, available at the University of Texas.

The discussion in this section will be limited to five programs, i.e. ELSYMS5, LAYER5, LAYER15, LAYIT and BISAR, since these are at present time the available operational layered programs at the University of Texas. BISTRO is also available, but not operational.

LAYER5

LAYER5 was developed by Chevron Research Company and has the capability of analyzing stresses and displacements in a 5-layered elastic system under a uniformly distribited load on a circular loaded area.

Input data to be provided are the wheel load and tire pressure, and for each layer, the layer number, modulus of elasticity, Poisson's ratio and thickness except for the bottom layer where a semi-infinite thickness is assumed.

The layered system consists of a maximum of five layers of different homogeneous, ideally elastic materials. Each layer is of uniform thickness and infinite dimensions in all horizontal directions. The bottom layer is assumed to be semi-infinite. Fig. 1 shows the details of the system. It will be noted that a cylindrical coordinate system is used.

The program computes the following numerically at any point in the layered system:

1. Vertical, tangential, radial, shear and bulk stress.
2. Vertical displacement.
3. Radial, tangential and shear strain.

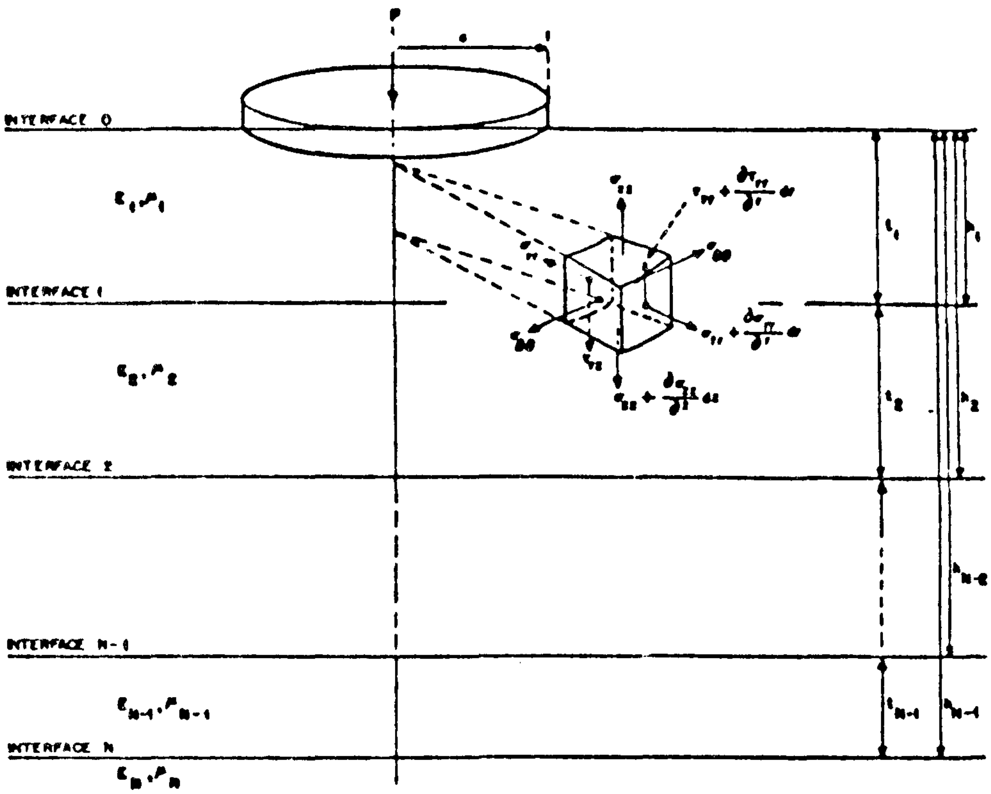


Figure 1. Stresses in a multi-layer system.

(After Ref. 5)

The governing equation in the mathematical model is that of equilibrium, all materials obey Hooke's law and the boundary conditions are as follows:

1. No slip occurs at interfaces.
2. The surface is free of stresses outside the loaded area.
3. Stresses, strains and displacements are finite at infinite depth.

LAYER15

LAYER15 was also developed by Chevron Research Company, California and is basically an extension of LAYER5 to 15 layers.

LAYIT

One of the basic assumptions using layered elastic theory is that materials can be characterized by their elastic constants: modulus of elasticity and Poisson's ratio. Most unbounded materials used in pavement structures are, however, stress sensitive, i.e. their modulus of elasticity depends on the stress level in the material. In this program the stress sensitivity of the material can be taken into account through an iterative process.

For a given material the relation between the resilient M_R and the sum of the principal stresses (θ), must be determined and expressed as follows:

$$M_R = A\,(\theta)^B$$

where A and B are constants

Mathematical Model of Iteration.

1. The modulus of elasticity and Poisson's ratio are assumed for each layer and using these values in the LAYER5 program, the stresses are estimated.

2. Stresses due to the overburden pressure are estimated at each depth and added to the stresses from the layered program.

3. The moduli of resilience at these value of stresses are calculated from the relations for each layer.

4. The moduli of resilience as determined in the previous step are compared with the assumed values. If they are close, the iteration will close else the procedure will be repeated.

5. The criteria for closing the iterarion is the Chi Square statistical test.

Input data to be provided are:

1. Wheel load and tire pressure.

2. Initially assumed elastic moduli, Poisson's ratios, thicknesses and values for coefficients A and B (as discussed above) for all layers except the bottom layer.

3. Initially assumed modulus of elasticity and values of resilient modulus versus deviator stress for the bottom layer.

The output is basically the same as for LAYER5.

ELSYM5

The ELSYM5 program was developed by the Institute of Transportation and Traffic Engineering, University of California.

This program calculates the various component stresses, strains and displacements, along with principal values in a three dimensional elastic layered system loaded weight one to ten identical uniform circular vertical loads. The system consists of one to five layers each of which is assumed to be homogeneous, ideally elastic, of uniform thickness and infinite in the horizontal direction. The bottom layer may be semi-infinite in thickness or may be given a finite thickness, in which case the program assumes the bottom elastic layer, is supported on a rigid base.

The top surface of the system is free of shear and all interfaces are continuous. With a rigid base the interface between the bottom elastic layer and the base has to be made either fully continuous or slippery.

A rectangular coordinate system (X,Y,Z) with the XY plane at Z = 0 (the top surface of the system) is used.

Input data to be provided are:

1. Any two of the load, tire pressure or radius of loaded area.

2. For each layer the number of the layer, modulus of elasticity, Poisson's ratio and thickness.

3. If the bottom elastic layer is resting on a rigid base, it should be indicated whether full friction or no friction is to be assumed for the rigid base interface.

4. Load positions in the coordinate system is tobe provided.

The output of the program gives for each depth all the results for each XY point. The results are the total effect of all the loads applied on the pavement system. This program calculates principal stresses and strains in addition to the normal stresses strains and displacements calculated by the other programs. The version of ELSYM5 used in this study can not calculate the vertical strain at the top of the subgrade directly, because when a z value is determined to be on an interface, the results are determined using the characteristics of the upper of the two layers. This problem can however be overcome by requesting results 0.01 inch below this interface.

BISAR

The BISAR (Ref 3) program was developed by Shell Research and is a extension of the earlier developed BISTRO program. This program (BISAR) computes stresses, strains and displacements in elastic multilayered systems subjected to one or more vertical or unidirectional tangential loads. Loads are assumed to be uniformly distributed over a circular loaded area.

The layers can be allowed to slip over each other under the following conditions at the interface:

i. Continuous vertical normal stress;

ii. continuous shear stress;

iii. vertical contact to be maintained;

iv. relative horizontal displacementis proportional to the shear stress;

v. the interface friction parameter can vary between zero (complete adhesion) and one (frictionless slip).

The basic theory used in this program is that of Burmeister which is based on full three-demensional linear elasticity (Ref 4).

Input data to be provided are as follows:

1. The number of layers in the system.

2. Modulus of elasticity, Poisson's ratio and thickness of each layer (except fot the thickness of the base layer).

3. The interface friction at the base layer.

4. The number of loads.

5. The vertical component of each load.

6. The horizontal component of each load and it's angle with the X-axis.

7. The positions of the loads.

8. The calculation requirements such as stress, strain and displacements components to be computed,and the number and positions of the points where these quantities have to be computed.

The following can be computed by the program:

1. For each load seperately the cylindrical component stress, strain and displacement.

2. For the combined action of all loads the following:

a. the cartesian components of stress, strain and displacement;

b. the principal values of stress and strain;

c. maximum shear stress and shear strain;

d. the mid points of the Mohr stress circles;

e. the strain energy of distortion;

f. the total strain energy.

This program only calculates those components which are requested. Table 1 compares the five programs discussed:

TABLE 1. COMPARISON OF LAYERED PROGRAMS.

	ELSYM 5	LAYER 5	LAYER 15	LAYIT	BISAR
1. PROGRAM ASSUMPTIONS:					
a. Material Properties	Elastic	Elastic	Elastic	Elastic	Elastic.
b. Boundary Conditions: (i) At Interfaces	Full friction allows slip if rigid base	Full friction	Full friction	Full friction	Full friction or varying amount of slip
(ii). At bottom layer	Infinite depth or finite	Infinite depth	Infinite depth	Infinite depth	Infinite depth
2. MAJOR FEATURES:					
Number of layers	5	5	15	5	10
Number of loads	10	1	1	1	10
Possibility of applying horizontal loads	No	No	No	No	Yes.
3. INPUT REQUIREMENTS					
Material Properties	Modulus (E) Poisson's Ratio (ν)	Modulus (E) Poisson's Ratio (ν)	Modulus (E) Poisson's Ratio (ν)	Initially assumed Modulus (E), Poisson's Ratio (ν) Modulus vs. Σ Principal Stresses	Modulus E Poisson's Ratio (ν)

TABLE 1 (CONTINUED)

	ELSYM 5	LAYER 5	LAYER 15	LAYIT	BISAR
Dimensions	Tickness (Bottom layer semi infinite or Finite)	Tickness (Bottom layer semi infinite)	Tickness (Bottom layer semi infinite	Tickness (Bottom layer semi Infinite	Tickness (Bottom layer semi infinite)
Load	any two of: Load, Area or Tire Pressure	Load Tire Pressure	Load Tire Pressure	Load Tire Pressure	Load or Stress and radius of loaded area
Coefficient of slipage	No slip except with rigid base	No slip	No slip	No slip	Coefficient of slipage to be specified
Unit weight of materials	weightless	Weightless	weightless	Required	weightless
4. MODEL OUTPUT.					
Displacement of top layer	Yes	Yes	Yes	Yes	Yes
Strain at surface away from load	Yes	Yes	Yes	Yes	Yes
Strain at Interfaces	Yes	Yes	Yes	Yes	Yes
Strain at top of subgrade	Indirectly	No	No	No	Yes
Stress throughout base & sub base	Yes	Yes	Yes	Yes	Yes
5. COMPUTER COST (compared to LAYER 5 - %)	125-140	100	— *	110-130	380-460

* With LAYER 15 each layer was devided into two, each with half the origional thickness - no cost comparison could be made

TABLE 1 (Continued)

	ELSYM 5	LAYER 5	LAYER 15	LAYIT	BISAR
6. EASE OF INPUT	Easy	Easy	Easy	Easy	Time Consuming Difficult Format
7. EASE OF INTERPRETING OUTPUT	Easy	Easy	Easy	Easy	Difficult to read.

CHAPTER 3. DESIGN OF THE EXPERIMENT

Presently the following layered programs are available at the University of Texas:

1. ELSYM5
2. LAYER5
3. LAYER15
4. LAYIT
5. BISAR
6. BISTRO

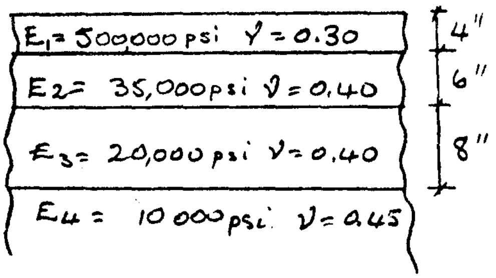
It was found that BISTRO is at present not operational and since BISAR is a extension of the BISTRO program, it was decided to eliminate BISTRO for the purpose of this study, so the first five programs as listed above have been compared.

In order to compare the programs thoroughly, a sensitivity analysis should be conducted on each program and the results of these studies compared with each other. Such a study would however be beyond the scope of a term project like this and therefore it was decided to limit this study to the analysis of a number of typical highway pavements.

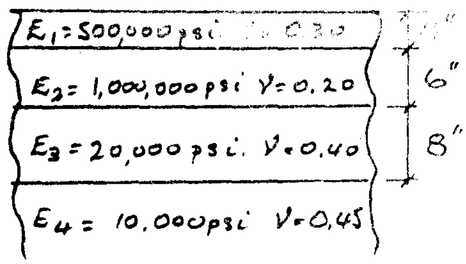
Pavements studied.

It was decided to analyse one multilayered flexible pavement, one multilayered flexible pavement with a stabilized base, a full depth asphalt pavement and a rigid pavement.

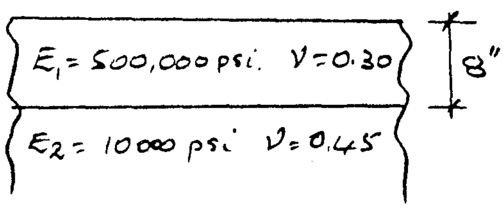
The dimensions, material properties and loading can be seen in figure 2 as follows:



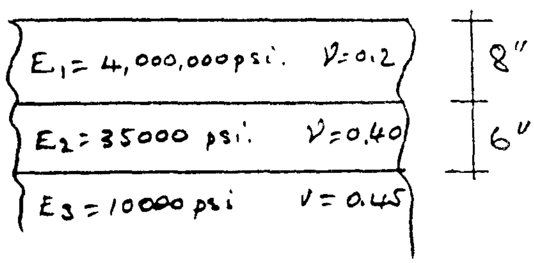
PAVEMENT NO. 1.
(FIG. 2a)



PAVEMENT NO. 2.
(FIG. 2b)



PAVEMENT NO. 3.
(FIG. 3c)



PAVEMENT NO. 4
(FIG. 3d)

PAVEMENT SYSTEMS ANALYSED.

FIGURE 2.

Pavement No. 1 (figure 2a):

Multilayered Flexible Pavement.

Pavement No. 2 (figure 2b):

Multilayered Flexible Pavement with the Stabilized Base.

Pavement No. 3 (figure 2c):

Full Depth Asphlt Concrete Pavement.

Pavement No. 4 (figure 2d):

Rigid Pavement.

Each of the four pavements have been analysed with each of the five computer programs mentioned earlier in this section. Stresses, strains and deflections have been compared as follows:

a. Vertical stresses (σ_Z) at variousdepths directly under the load.

b. Horizontal stresses (σ_t) at varios depths directly under the load.

c. Horizontal stresses (σ_t) at the bottom of the first layer at various radial distances from the load.

d. Surface deflections at various radial distances from the load.

d. Vertical strain at the top of the subgrade directly under the load.

CHAPTER 4. <u>RESULTS</u>.

Figures 3 to 18 are graphical comparisons of results obtained with the various computer programs; Tables 2 - 5 compares the vertical strains as compelled by the different programs for each pavement investigated.

As pointed out under the discussion of the programs and as also indicated in Table 1, it was not possible to find the vertical strain at the top of the subgrade directly with any of the programs, but BISAR. For the ELSYM5, LAYER5, and LAYER15 it was necessary to calculate this strain with the formula.

$$\epsilon_z = \frac{1}{E}\left[\sigma_x - \nu(\sigma_t + \sigma_r)\right]$$

Since ELSYM5 normally does not give results at the top of the subgrade, the strain at the top of the subgrade has not been determined with this program.

For the purpose of this report the following is a list nomenclature:

σ_z = Vertical stress

σ_t = Horizontal tangentical stress

σ_r = Horizontal radial stress

ϵ_z = Vertical strain

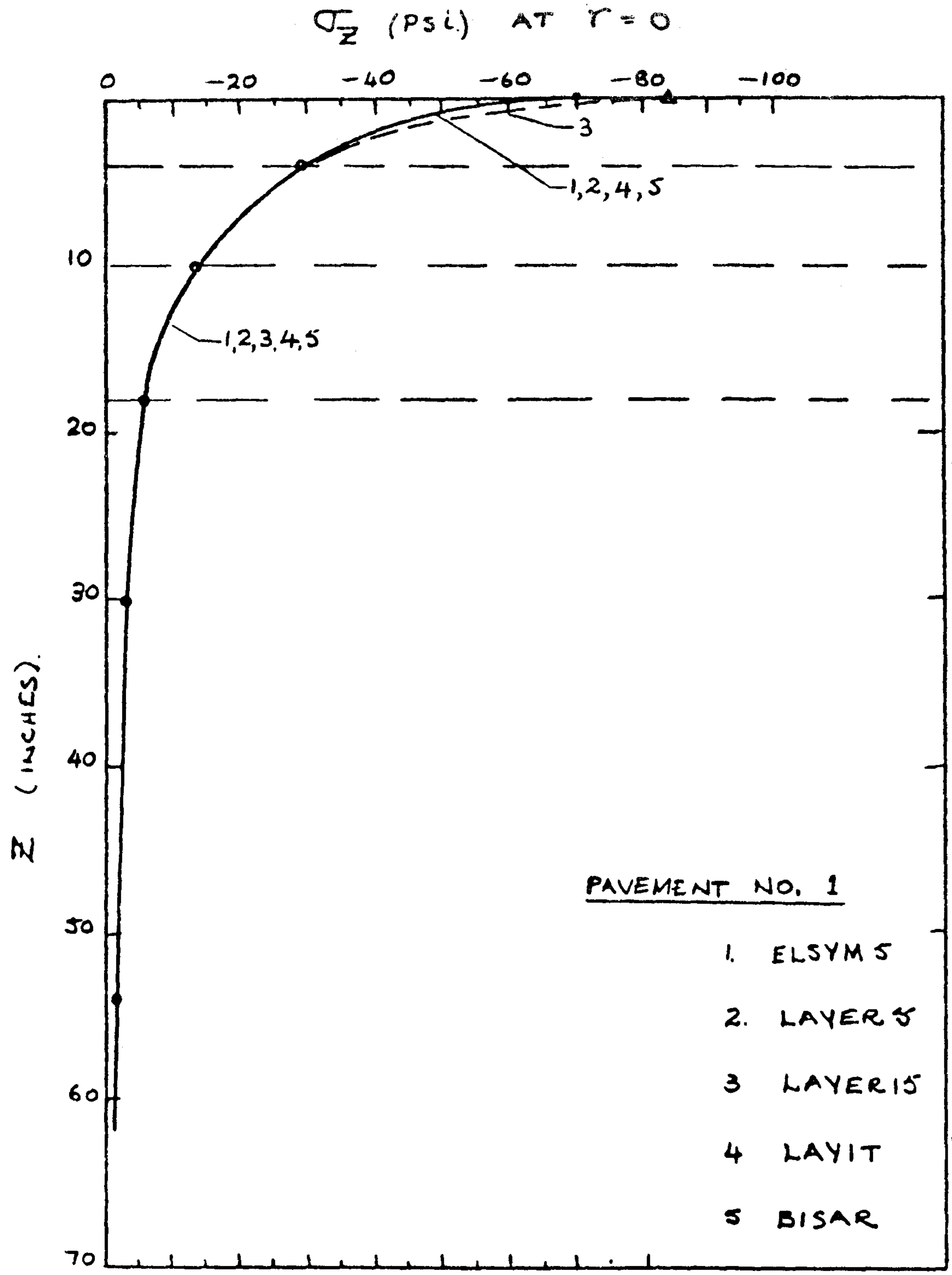
ϵ_t = Horizontal tangential strain

ϵ_r = Horizontal radial strain

W = Vertical deflection

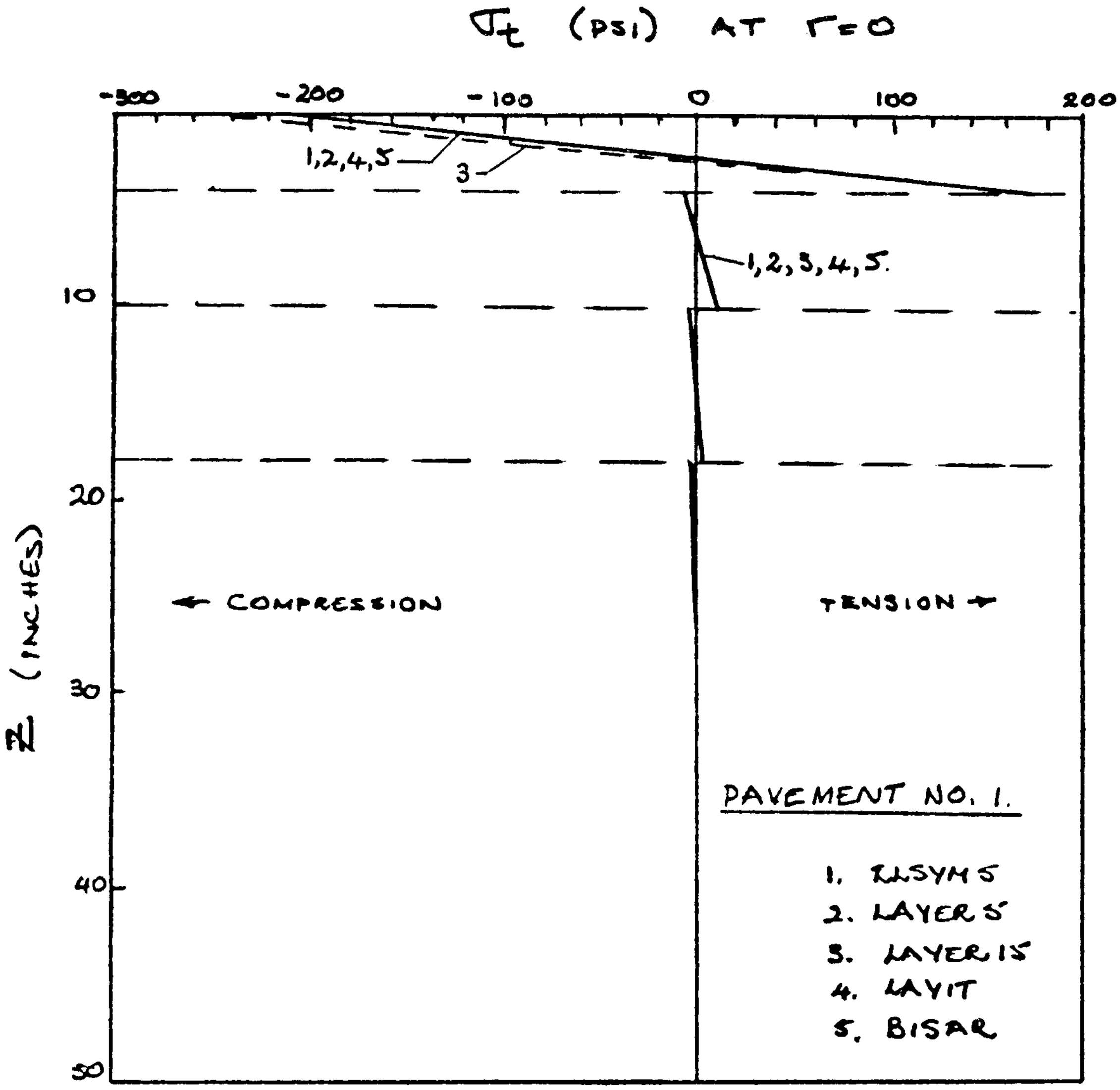
Z = Depth

r = Radial distance from the load



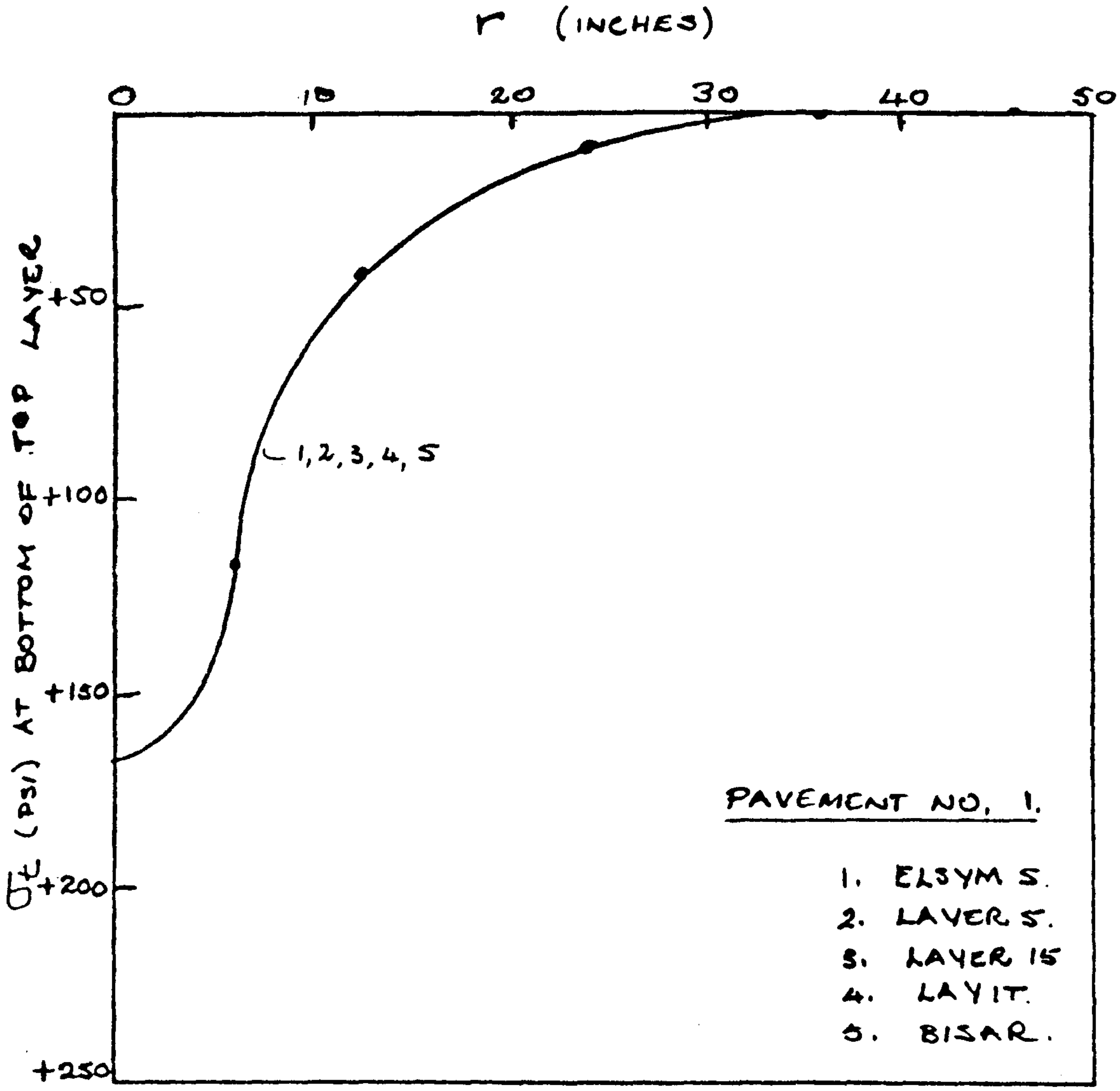
VERTICAL STRESS UNDER THE LOAD

FIGURE 3



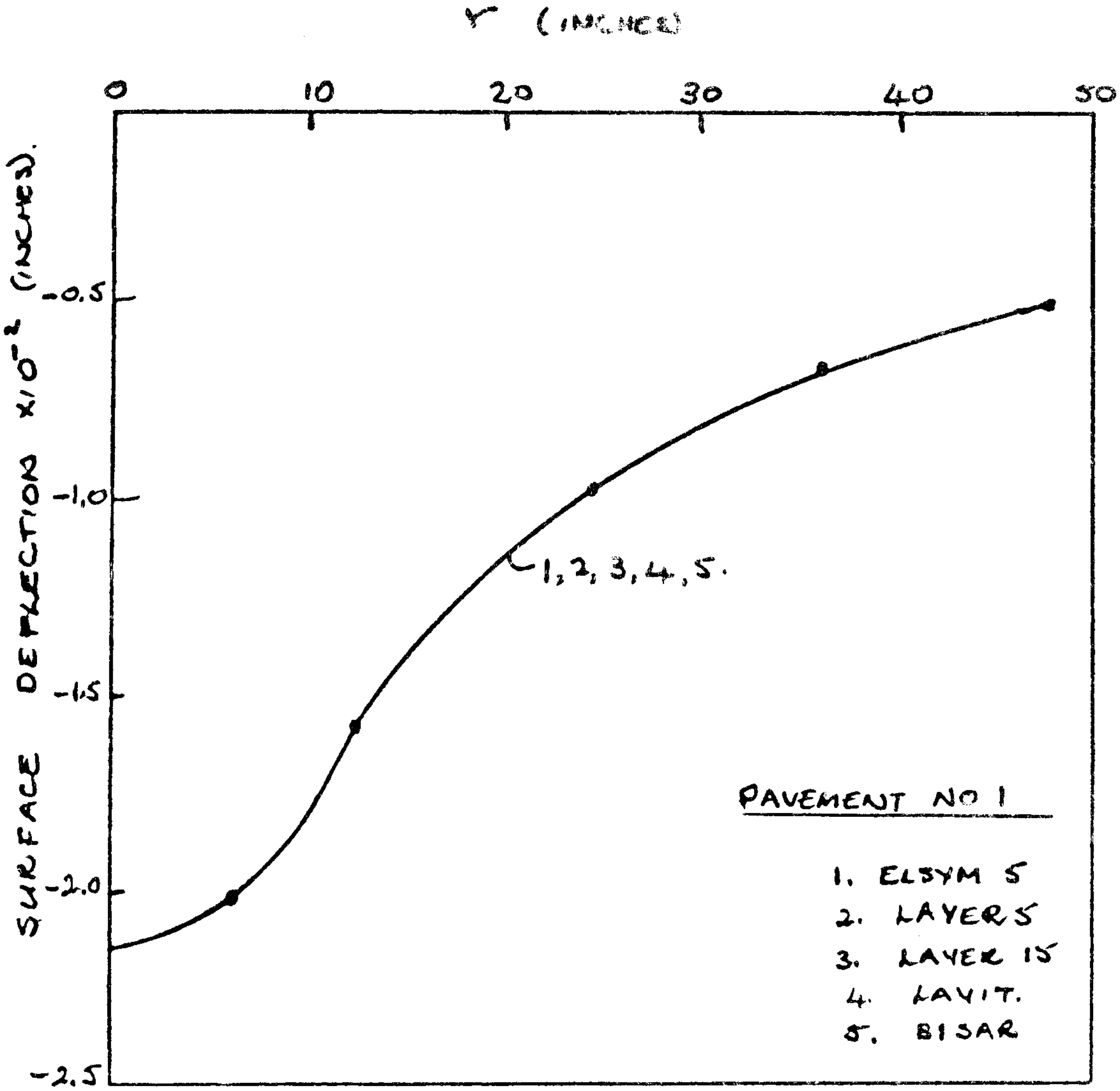
HORIZONTAL STRESS UNDER THE LOAD.

FIGURE 4.



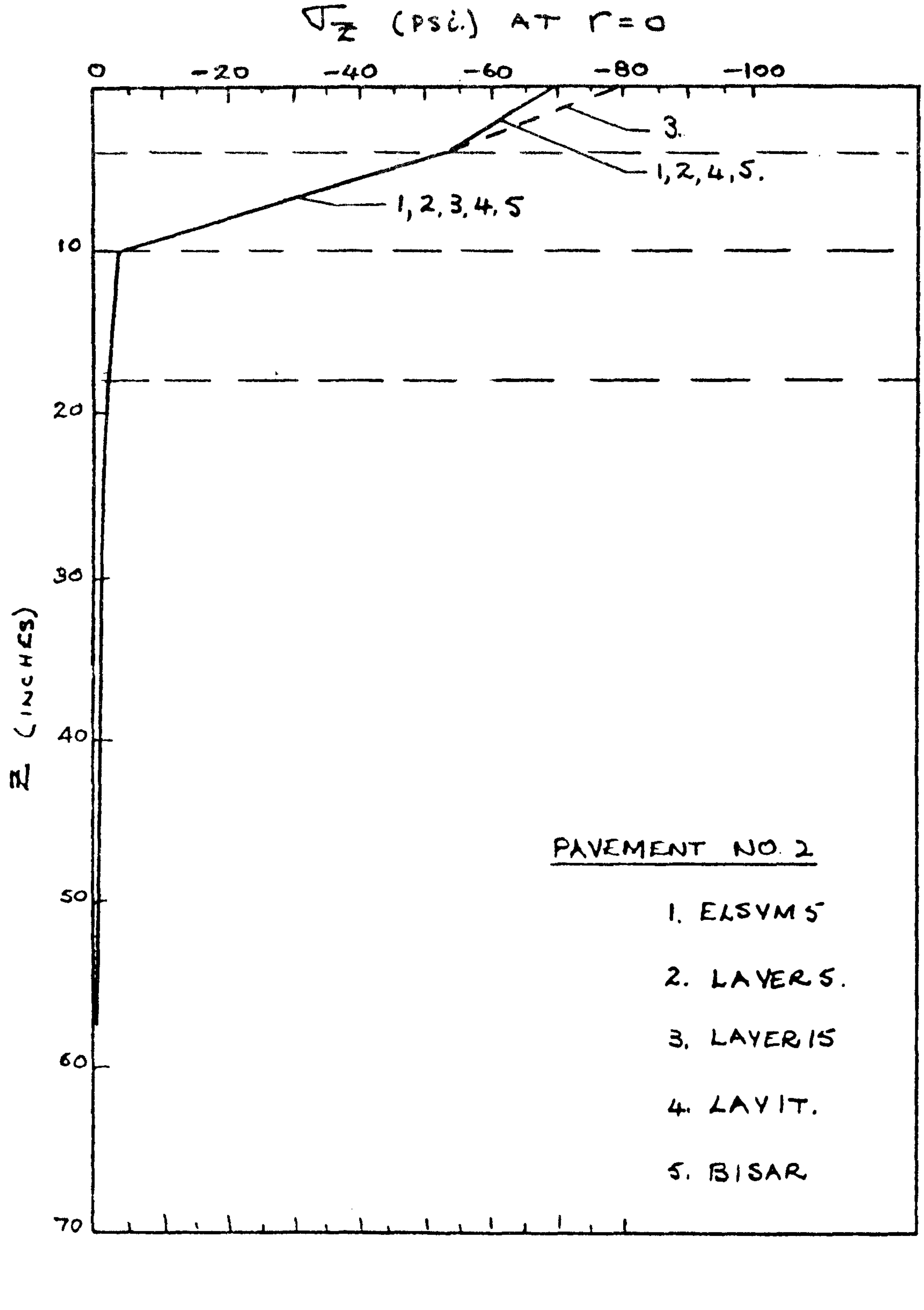
HORIZONTAL STRESS AT THE BOTTOM OF THE TOP LAYER

FIGURE 5.



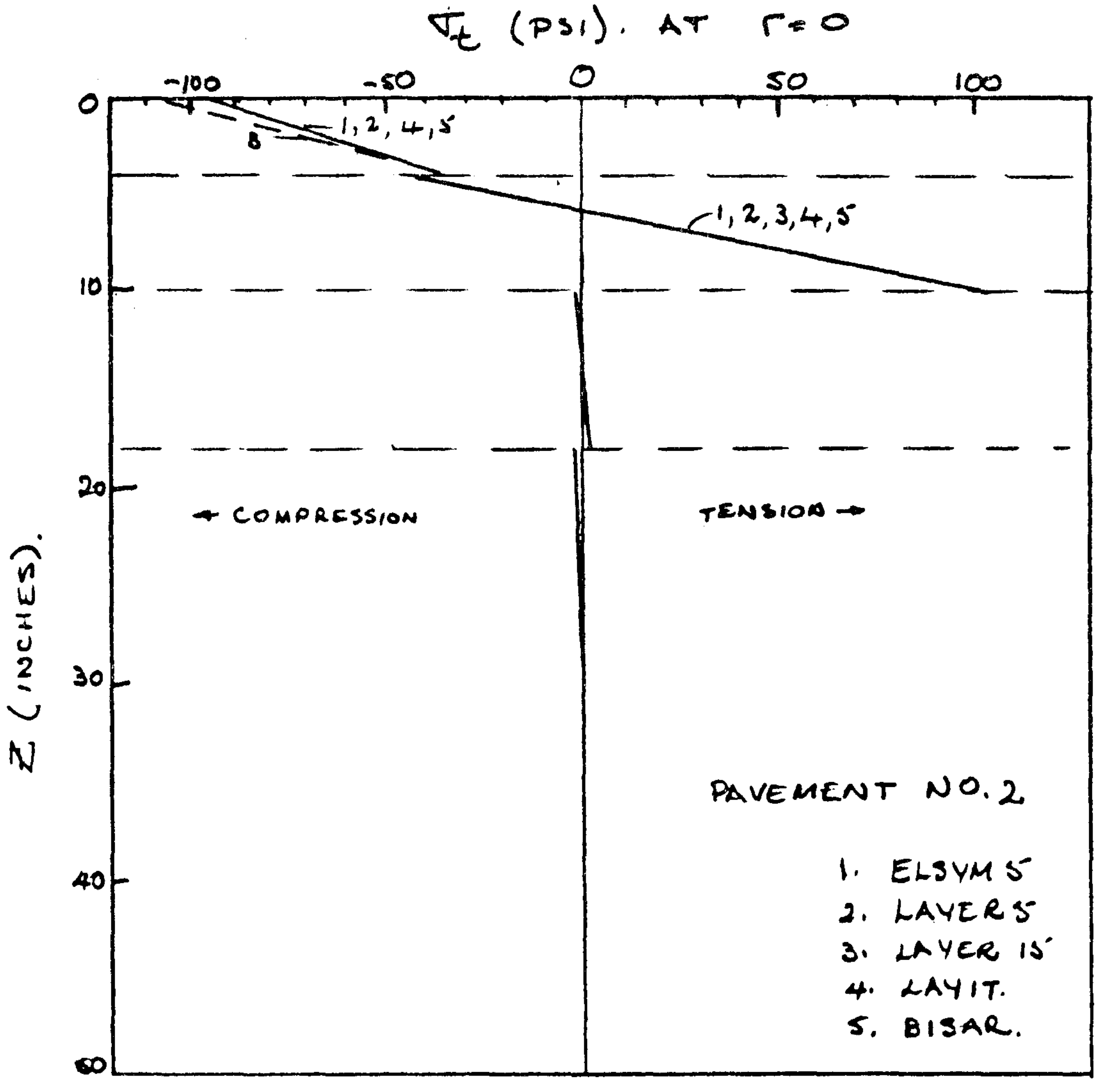
SURFACE DEFLECTIONS

FIGURE 6



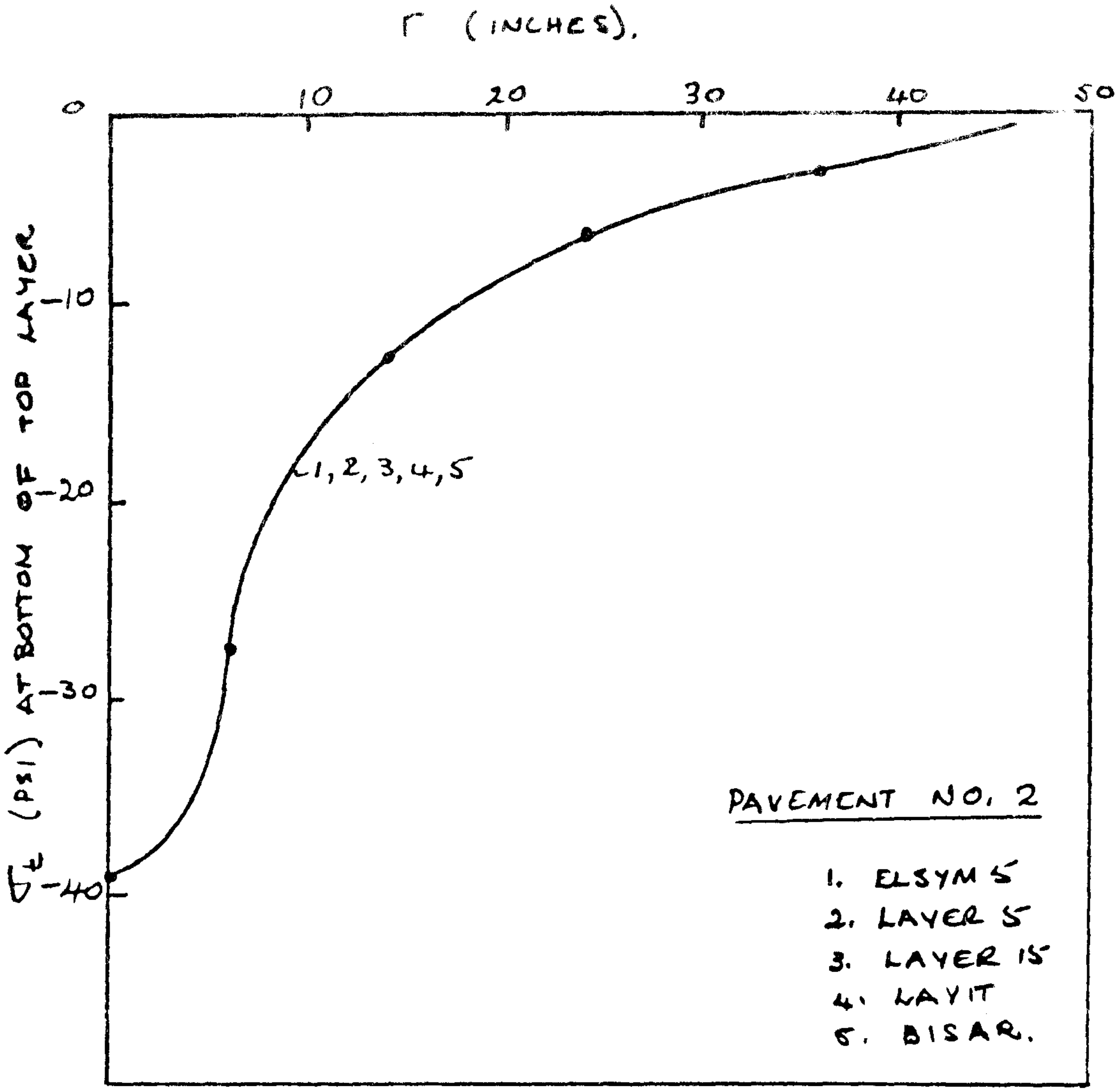
VERTICAL STRESS UNDER THE LOAD.

FIGURE 7



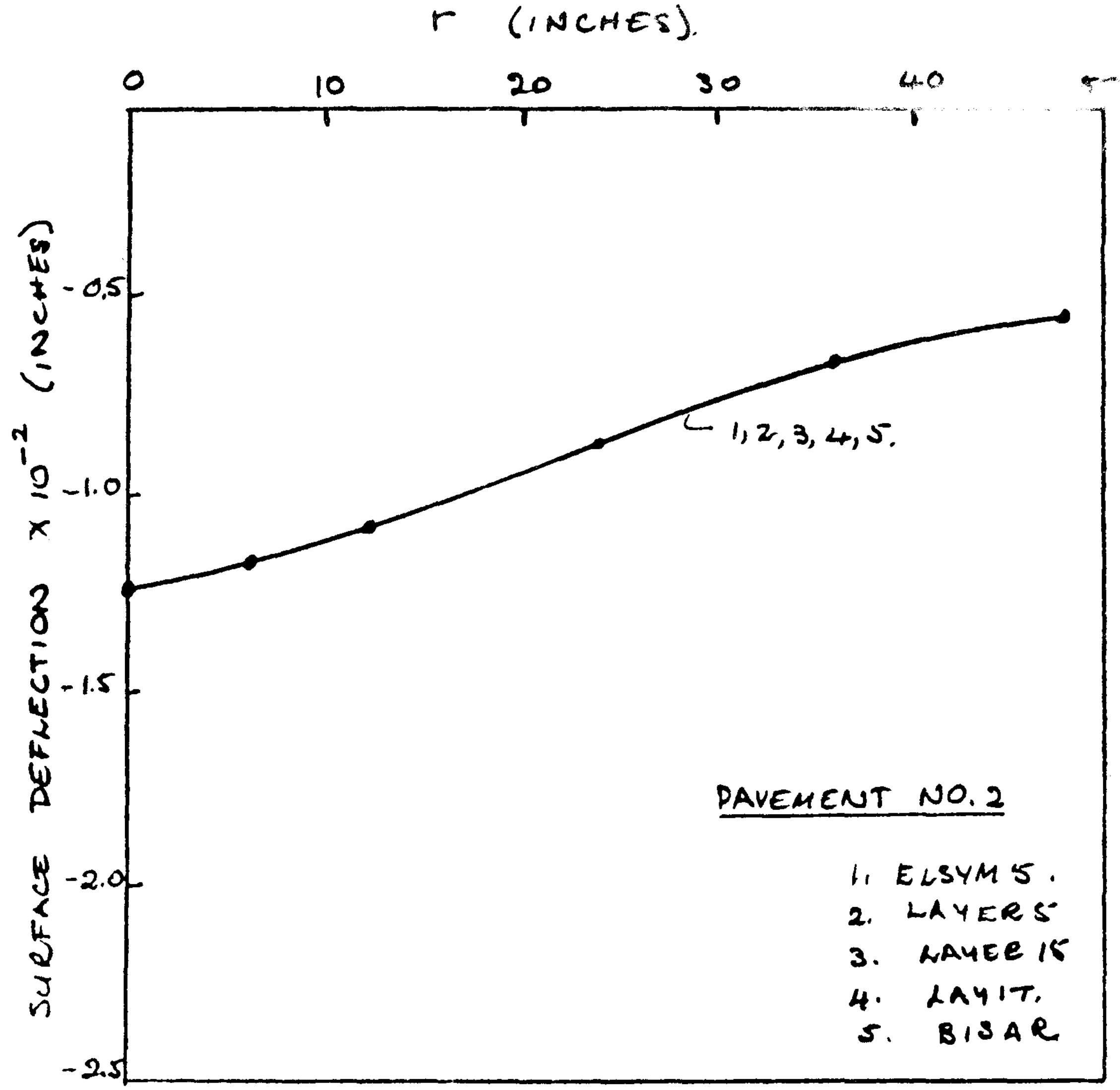
HORIZONTAL STRESS UNDER THE LOAD

FIGURE 8



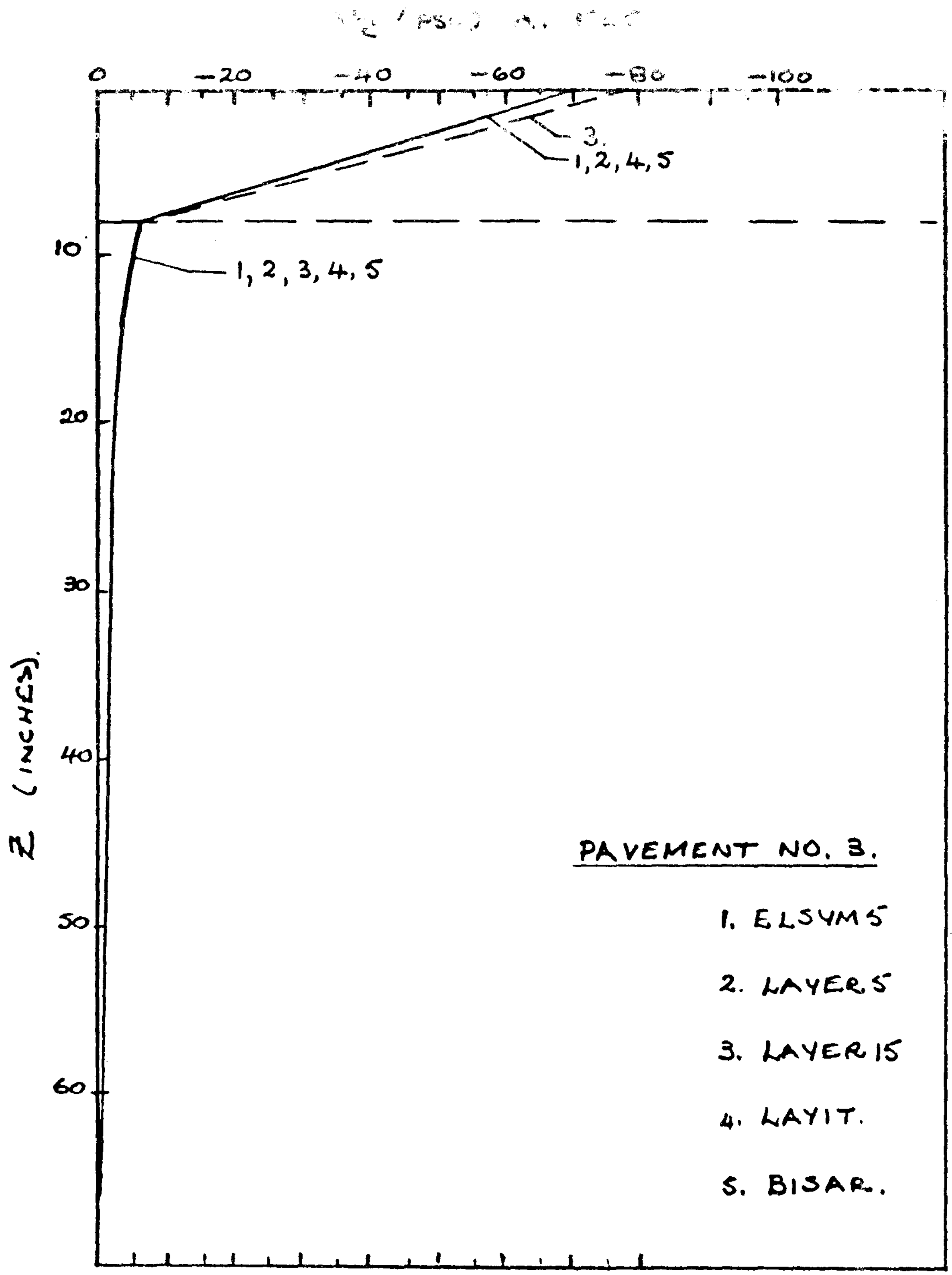
HORIZONTAL STRESS AT THE BOTTOM OF THE TOP LAYER

FIGURE 9



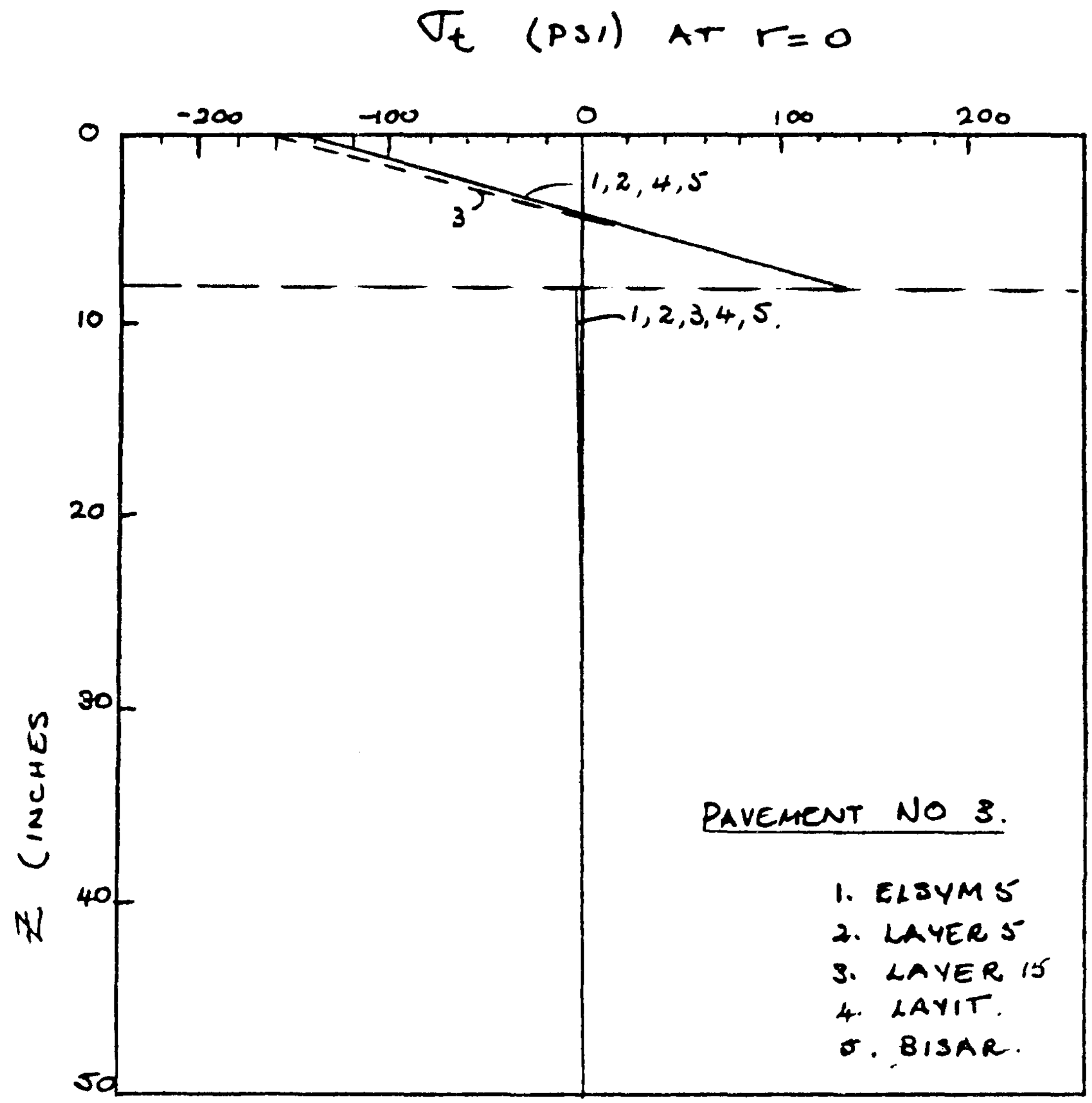
SURFACE DEFLECTIONS.

FIGURE 10.



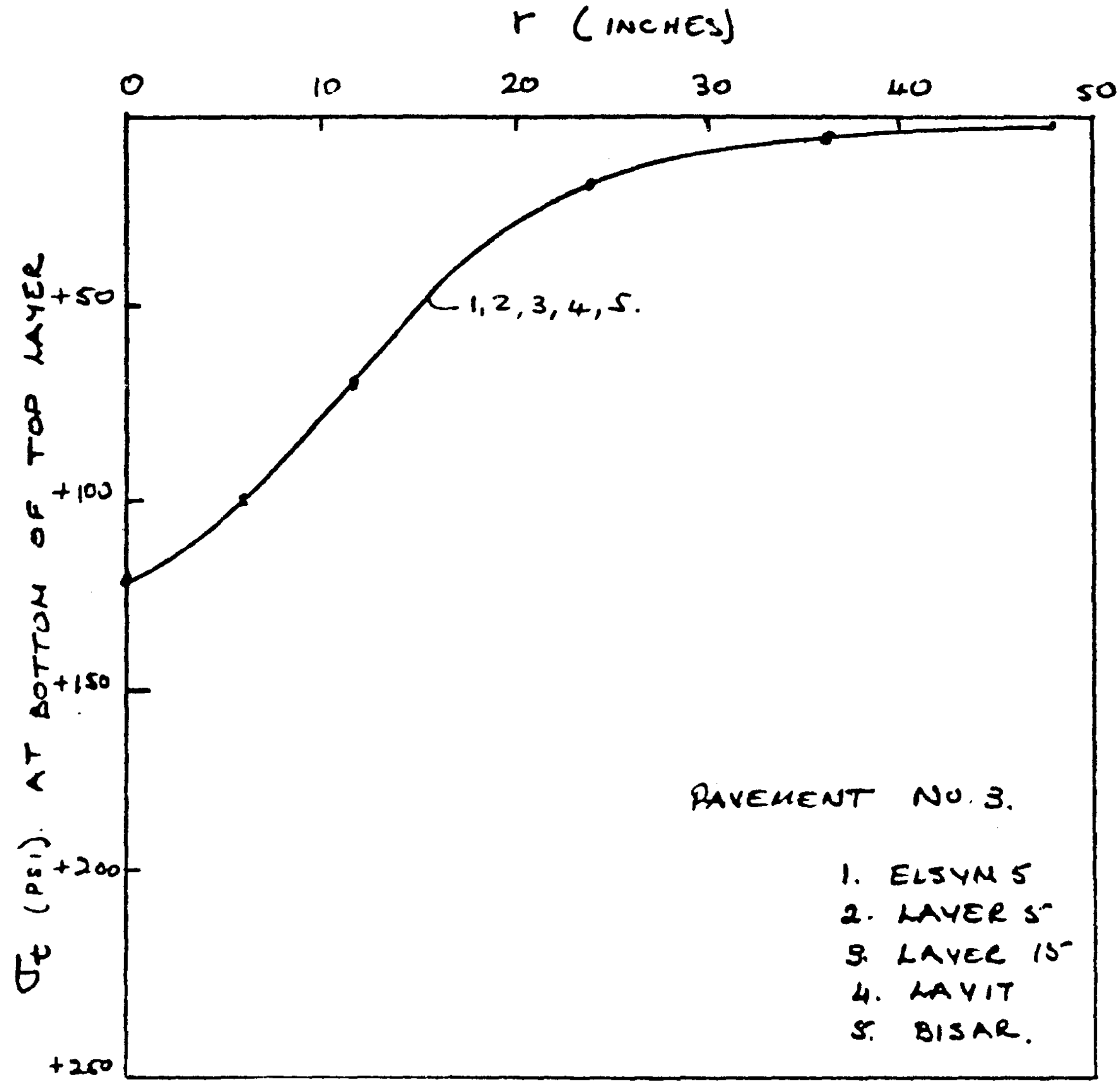
VERTICAL STRESS UNDER THE LOAD.

FIGURE 11



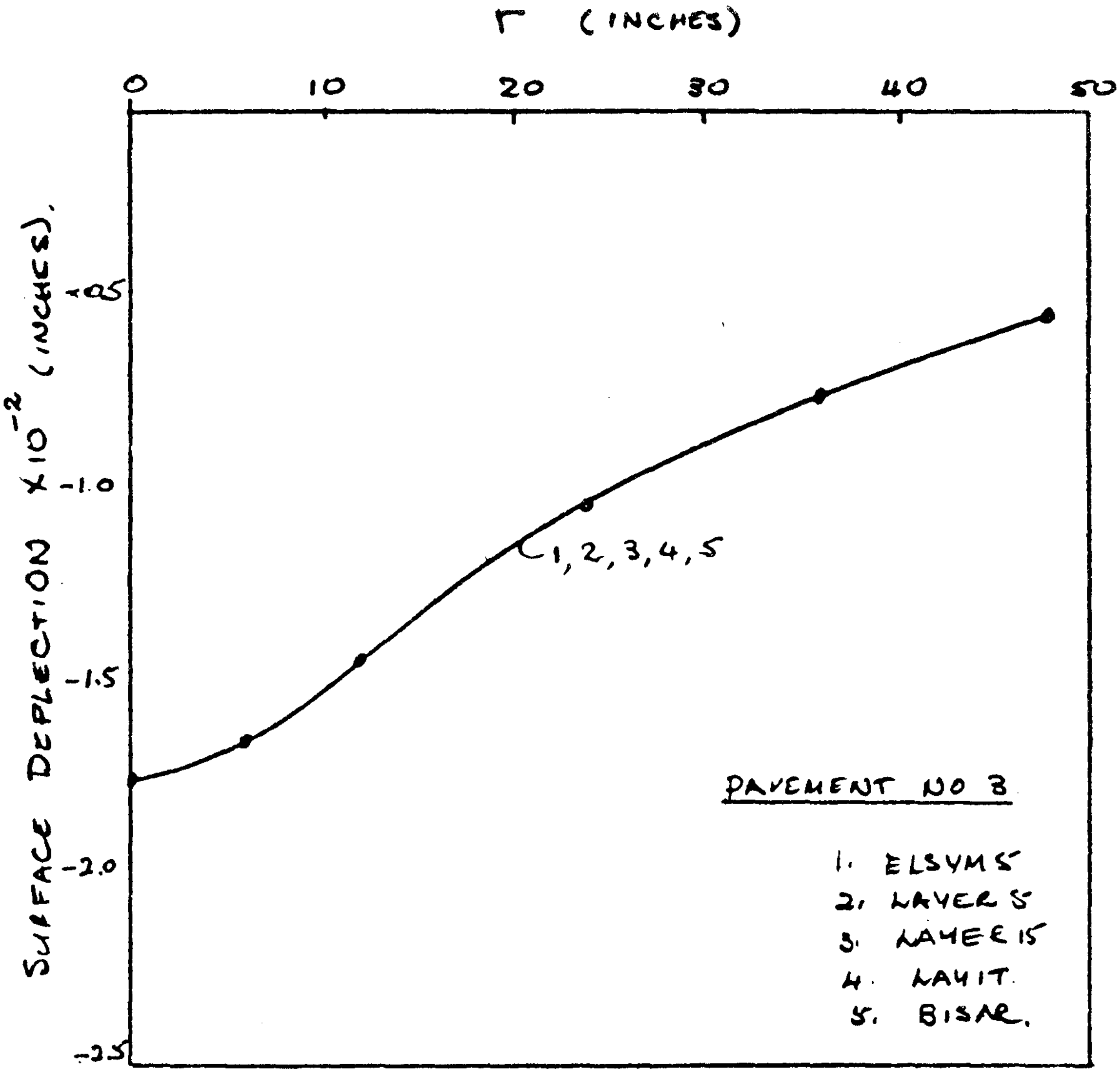
HORIZONTAL STRESS UNDER THE LOAD

FIGURE 12



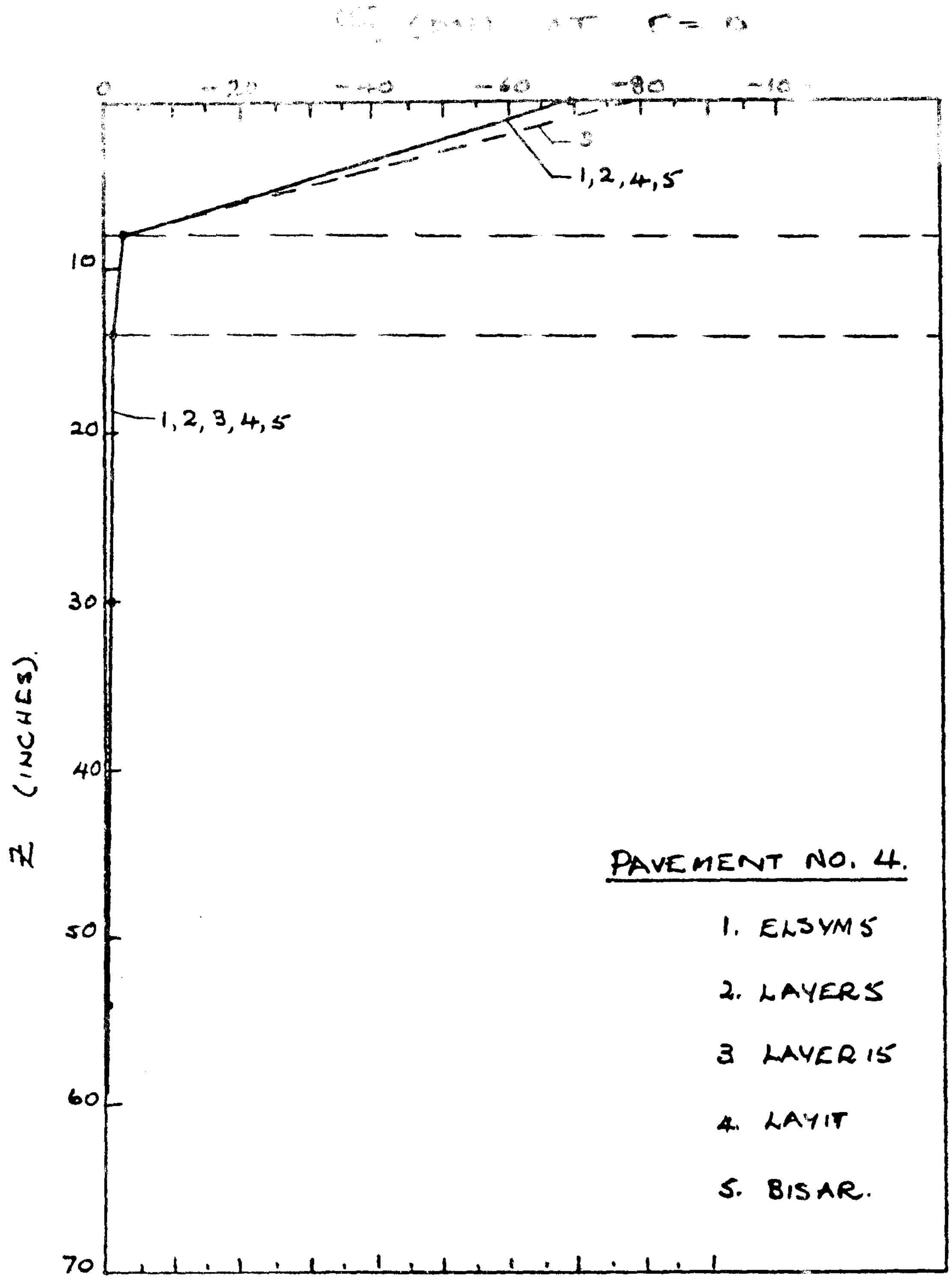
HORIZONTAL STRESS AT THE BOTTOM OF THE TOP LAYER

FIGURE 13



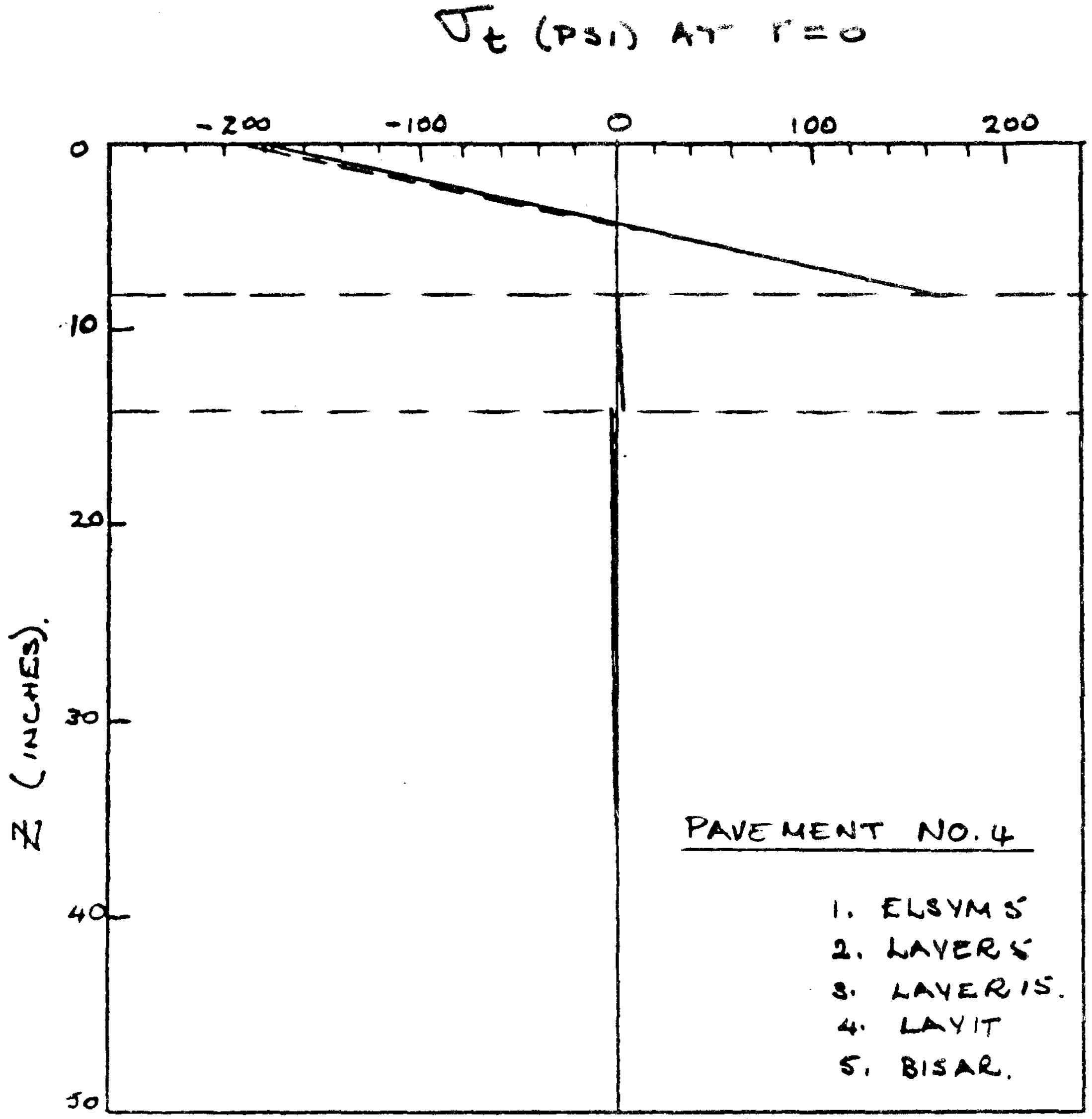
SURFACE DEFLECTIONS

FIGURE 14



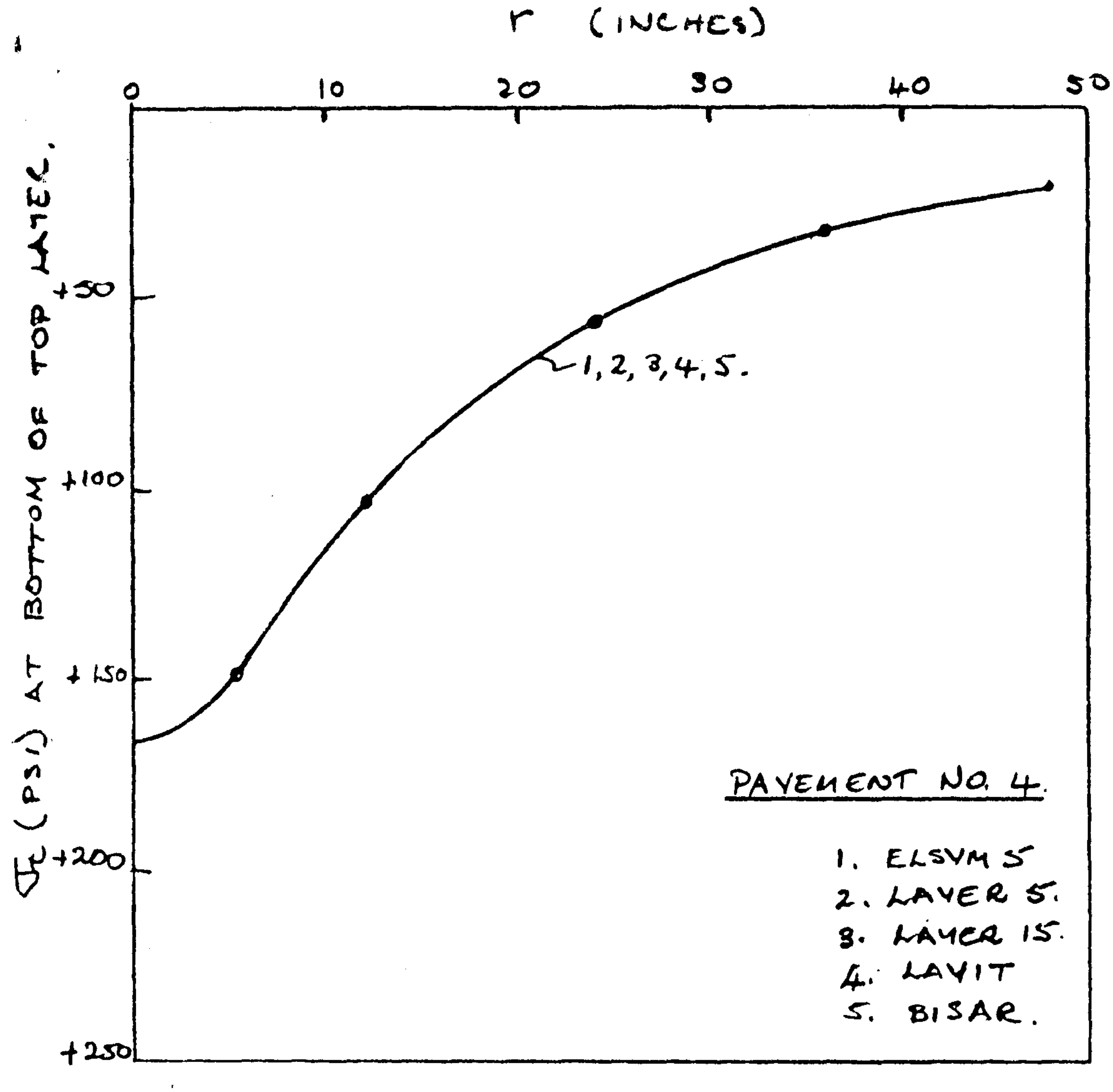
VERTICAL STRESS UNDER THE LOAD.

FIGURE 15



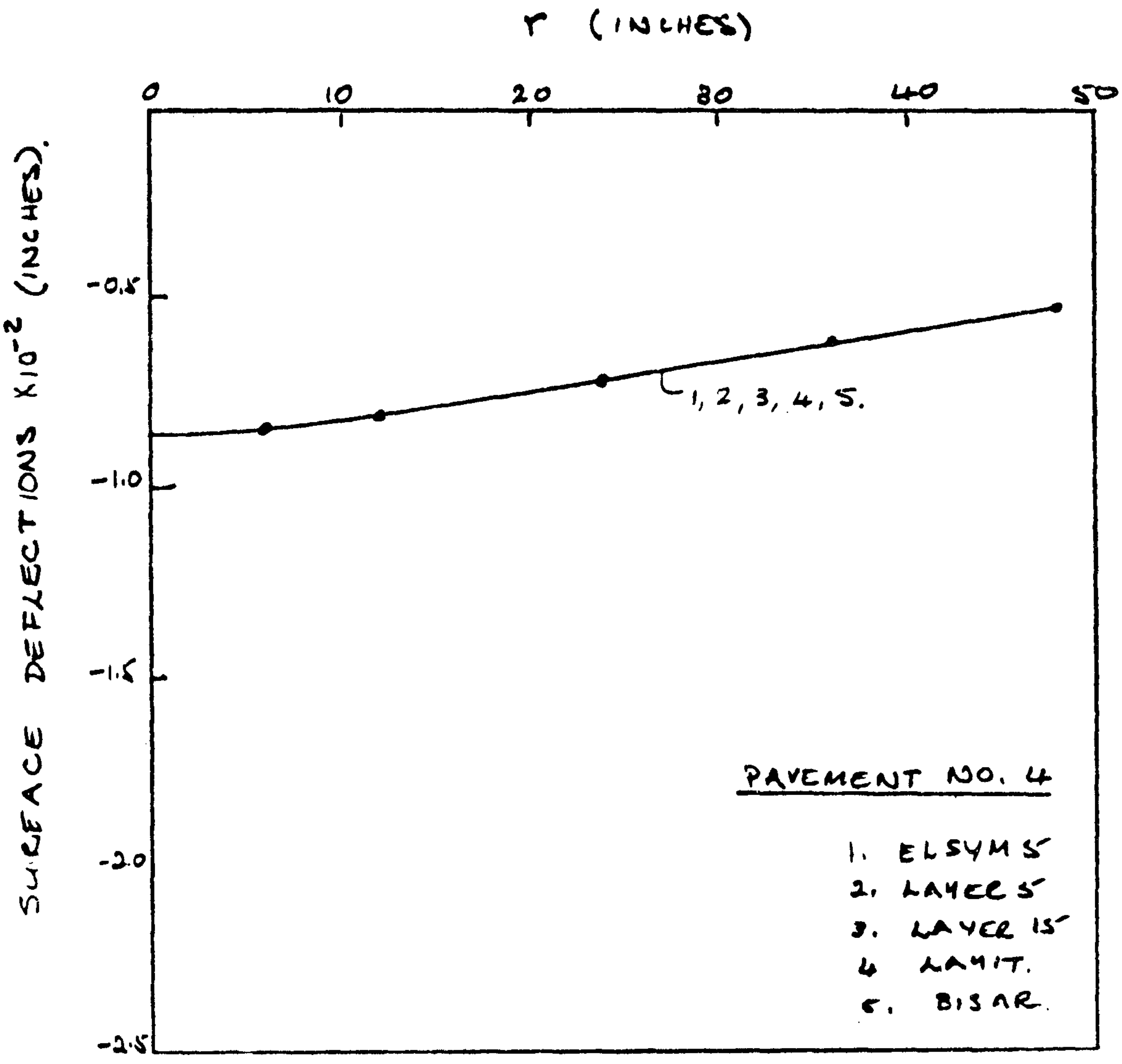
HORIZONTAL STRESS UNDER THE LOAD

FIGURE 16



HORIZONTAL STRESS AT THE BOTTOM OF THE TOP LAYER

FIGURE 17



SURFACE DEFLECTIONS

FIGURE 18

TABLE 2: VERTICAL STRAINS AT TOP OF SUBGRADE

(Pavement No. 1)

	E_Z at top of subgrade
BISAR	$-5.1928\ 10^{-4}$
ELSYM5	- *
LAYER5	$-5.1989\ 10^{-4}$ **
LAYER15	$-5.1952\ 10^{-4}$ **
LAYIT	$-5.1892\ 10^{-4}$ **

* Could not be obtained directly with ELSYM5.

** Obtained by hand calculation:

$$E_Z = \frac{1}{E}\left[\sigma_Z - \nu\,(\sigma_r + \sigma_t)\right]$$

TABLE 3: VERTICAL STRAINS AT TOP OF SUBGRADE

(Pavement No. 2)

	E_Z at top of subgrade
EISAR	$-2.0490 \quad 10^{-4}$
ELSYM5	- *
LAYER5	$-2.03809 \quad 10^{-4}$ **
LAYER15	$-2.036495 \quad 10^{-4}$ **
LAYIT	$-2.038235 \quad 10^{-4}$ **

*Could not be obtained directly with ELSYM5.

**Obtained by hand calculation:

$$\varepsilon_Z = \frac{1}{E}\left[\sigma_Z - \nu\,(\sigma_r + \sigma_t)\right]$$

TABLE 4: VERTICAL STRAINS AT TOP OF SUBGRADE

(Pavement No. 3)

	E_Z at top of subgrade
BISAR	$-1.1459 \quad 10^{-4}$
ELSYM5	- *
LAYER5	$-1.10746 \quad 10^{-4}$ **
LAYER15	$-1.106194 \quad 10^{-4}$ **
LAYIT	$-1.107124 \quad 10^{-4}$ **

*Could not be obtained directly with ELSYM5.

**Obtained by hand calculation:

$$E_Z = \frac{1}{E}\left[\sigma_Z - \nu\,(\sigma_r + \sigma_t)\right]$$

TABLE 5: VERTICAL STRAIN AT TOP OF SUBGRADE

(Pavement No. 4)

	E_Z at top of subgrade
BISAR	- *
ELSYM5	$-4.6889 \ 10^{-4}$ **
LAYER5	$-4.7016 \ 10^{-4}$ **
LAYER15	$-4.6991 \ 10^{-4}$ **
LAYIT	$-4.70218 \ 10^{-4}$ **

*Could not be obtained directly with ELSYM5.

**Obtained by hand calculation:

$$E_Z = \frac{1}{E}\left[\sigma_Z + \nu\,(\sigma_r + \sigma_t)\right]$$

E = Modulus of elasticity

ν = Poisson's ratio

MR = Resilient modulus

A comparison of computer costs have been made for the pavements analysed as can be seen in Table 6.

Tables A1 - A4 in the appendix, summerize the results obtained with the various programs for Pavement No. 1, Tables A5 - A8 for Pavement No. 2, Tables A9 - A12 for Pavement No. 3 and Tables A13 - A16 are the results for Pavement No. 4.

CHAPTER 5. DISCUSSION OF RESULTS

In studing the results obtained from the various programs for the pavements analysed, the following can be seen:

1. The values obtained for stresses, both horizontal and vertical, with all five computer programs, were for all practical purposes the same for each pavement system analysed, except for the stresses at the surface (z = 0), where in all cases LAYER15 predicted higher stresses (see figures 3, 4, 7, 8, 11, 15 and 16). This is however generally not a position of interest in normal pavement design.

2. The values obtained for the horizontal stresses at the bottom of the first layer were for all practical purposes identical with all programs for each pavement considered. This value is of importance for determing the fatique life of the surfacing material.

TABLE 6: COMPARISON OF COMPUTER COSTS

	COMPUTER COST $			
	Pavement 1	Pavement 2	Pavement 3	Pavement 4
ELSYM5	0.73	0.71	0.45	0.56
LAYER5	0.52	0.54	0.36	0.44
LAYER15 *	0.72	0.72	0.53	0.61
LAYIT	0.94	0.94	0.58	0.70
PISAR	2.40	2.44	1.36	1.86

* Each layer has been put in as two layers;
Each with half the thickness of the layer with LAYER15.

These results can be seen on figures 5, 9, 13 and 17.

3. Practically the same surface deflections were predicted by all programs for each pavement system analysed. This can be seen on figures 6, 10, 14 and 18.

4. In studying the values obtained for vertical strain at the top of the subgrade at r = 0 (Tables 2 - 5), it will be noted that slight variations occurred, but it is to be noted that the mximum difference among the results obtained was only 3% for Pavement No. 4 (Table 5) which is for practical design purposes within reasonable limits.

It can be said that in general the results obtained by the different programs are very simmilar, which indicates that probably anyone of these programs can be used with confidence.

Tables 1 and 6 shows the comparison of computer costs and it can be seen that, not taking LAYER15 into consideration, since double the amount of layers has been calculated for each problem in this case, LAYER5 proved to be the cheapest with LAYIT 10% - 30%, ELSYM5 25% - 40% and BISAR 280% - 360% more expensive than LAYER5.

It was also found that the input to BISAR is extremely tedious and numerious cards in E12.6 format have to be punched. The rest of the programs are fairly easy to use.

CHAPTER 6. CONCLUSIONS

It can be concluded that all five programs i.e. LAYER5, LAYER15, LAYIT, ELSYM5 and BISAR use basically the Burmeister theory and takes into account full three-dimentional linear elaticity.

The values of stresses, strains and deflections obtained for each pavement system analysed, were for all practical purposes the same with the different programs,therefor anyone of the programs can be used with confidence - as far as accuracy is concerned.

Certain programs have however other advantages such as being cheap and easy to use (LAYER5, ELSYM5 and LAYER15) or being able to take the stress dependancy of materials into account (LAYIT), or to allow slip at the interface between layers (BISAR) or to allow more than one load to be applied to the system (ELSYM5 and BISAR), or to allow horizontal load to be applied to the system (BISAR). Since practically the same values for stresses, strains and deflections can be obtained by anyone of these programs, it seems as if these other features, as mentioned above, might be the criteria in selecting a layered program to be used for a specific pavement system.

As pointed out earlier, this investigation was based on typical pavement systems. It might be that extreme values for some of the input variables might lead to other conclusions.

REFERENCES

1. McCullough B.F., "A Pavement Overlay Design System Considering Wheel Loads, Temperature Changes and Performance" a Dissertation submitted to the University of California.

2. Zaniewski J., "Procedures for Rehabilitation of Flexible Pavements ." A Dissertation submitted to U.T. 1977.

3. De Jong D.L., Pentz M.G.F. and Korswagen A.R., "Computer program, BISAR, Layered systems and normal and tangential surface loads". An External Report, Shell Research B.V.

4. Shell Research "BISAR Users Manual", Koninklijke / Shell-Laboratorium, Amsterdam.

5. Information obtained from Mr. Randy Wallin, Centre for Highway Research.

APPENDIX

TABLE A1. VERTICAL STRESS (σ_z) AT r=0

(PAVEMENT NO. 1)

Z	σ_z (PSI.)				
	ELSYMS	LAYERS	LAYERIS	LAYIT	BISAR
0	-70.00	-70.00	-83.36	-70.00	-70.00
-4	-29.68	-29.68	-29.67	-29.68	-29.68
+4		-29.68	-29.67	-29.68	-29.68
-10	-12.61	-12.61	-12.61	-12.61	-12.61
+10		-12.61	-12.61	-12.61	-12.61
-18	-5.33	-5.33	-5.32	-5.33	-5.33
+18		-5.33	-5.32	-5.33	-5.33
30	-2.64	-2.64	-2.64	-2.64	-2.65
54	-1.04	-1.04	-1.04	-1.04	-1.07
78	-0.56	-0.56	-0.56	-0.56	-0.57

TABLE A2 HORIZONTAL STRESS (σ_t) AT r=0

(PAVEMENT NO. 1.)

Z	σ_t (PSI)				
	ELSYMS	LAYERS	LAYERIS	LAYIT	BISAR
0	-235.2	-235.2	-245.8	-235.2	-235.1
-4	+169.7	+169.7	+169.7	+169.7	+169.7
+4		-4.8	-4.8	-4.8	-4.9
-10	+8.1	+8.1	+8.1	+8.1	+8.1
+10		+1.0	+1.0	+1.0	+1.0
-18	+4.2	+4.2	+4.2	+4.2	+4.2
+18		-0.1	-0.1	-0.1	-0.1
30	0	0	0	0	0
54	0	0	0	0	0
78	0	0	0	0	0

TABLE A 3 HORIZONTAL STRESS (σ_t) AT THE BOTTOM OF THE TOP LAYER

(PAVEMENT No 1)

r	σ_t (PSI)				
	ELSYM 5	LAYER 5	LAYER 15	LAYIT	BISAR
0	169.7	169.7	169.7	169.7	169.7
6"	116.6	116.6	116.6	116.6	116.6
12"	36.7	36.7	36.7	36.7	36.7
24"	4.5	4.5	4.5	4.5	4.5
36"	0.5	0.5	0.5	0.5	0.5
48"	-0.2	-0.2	-0.2	-0.2	-0.2

TABLE A 4 SURFACE DEFLECTION

(PAVEMENT No 1)

r	W ($\times 10^{-2}$) in.				
	ELSYM 5	LAYER 5	LAYER 15	LAYIT	BISAR
0	-2.33	-2.33	-2.33	-2.33	-2.33
6"	-2.06	-2.06	-2.06	-2.06	-2.06
12"	-1.58	-1.59	-1.58	-1.58	-1.58
24"	-1.00	-1.00	-1.00	-1.00	-1.00
36"	-0.69	-0.69	-0.69	-0.69	-0.69
48"	-0.51	-0.51	-0.51	-0.51	-0.51

TABLE A5. VERTICAL STRESS (σ_z) AT $r=0$

(PAVEMENT No 2)

Z	σ_z (psi)				
	ELSYM 5	LAYER 5	LAYER 15	LAYIT	BISAR
0	-70.00	-70.00	-79.97	-70.00	-70.00
-4	-53.90	-53.90	-52.90	-53.90	-53.90
4		-53.90	-53.90	-53.90	-53.90
-10	-4.46	-4.46	-4.46	-4.46	-4.48
10		-4.46	-4.46	-4.46	-4.48
-18	-2.43	-2.43	-2.42	-2.43	-2.46
18		-2.43	-2.42	-2.43	-2.46
30	-1.51	-1.51	-1.51	-1.51	-1.56
54	-0.77	-0.77	-0.77	-0.77	-0.81
78	-0.47	-0.47	-0.47	-0.47	-0.49

TABLE A6 HORIZONTAL STRESS (σ_t) AT $r=0$

(PAVEMENT No 2)

Z	σ_t (PSI)				
	ELSYM 5	LAYER5	LAYER15	LAYIT	BISAR
0	-98.89	-98.89	-106.81	-98.89	-98.73
-4	-39.50	-39.50	-39.48	-39.50	-39.46
4		-42.18	-42.14	-42.17	-42.10
-10	+103.26	+103.30	+103.20	+103.04	+103.01
10		-0.19	-0.19	-0.19	-0.22
-18	+1.23	+1.23	+1.23	+1.23	+1.21
18		-0.43	-0.43	-0.42	-0.46
30	-0.21	-0.21	-0.21	-0.21	-0.22
54	-0.08	-0.08	-0.09	-0.09	-0.06
78	-0.05	-0.05	-0.05	-0.05	-0.02

TABLE A7 HORIZONTAL STRESS (σ_t) AT THE BOTTOM OF THE TOP LAYER

(PAVEMENT No. 2)

r	σ_t psi				
	ELSYM 5	LAYERS	LAYER IS	LAYIT	BISAR.
0	-39.50	-39.50	-39.48	-39.50	-39.46
6"	-28.94	-26.89	-26.87	-26.89	-26.85
12"	-12.94	-12.93	-12.92	-12.94	-12.93
24"	-6.38	-6.38	-6.37	-6.38	-6.38
36"	-3.28	-3.28	-3.28	-3.27	-3.28
48"	-1.73	-1.73	-1.73	-1.73	-1.73

TABLE A8 SURFACE DEFLECTION

(PAVEMENT NO 2)

r	w ($\times 10^{-2}$) in				
	ELSYM 5	LAYERS	LAYERIS	LAYIT	BISAR
0	-1.24	-1.24	-1.24	-1.24	-1.26
6	-1.18	-1.18	-1.18	-1.18	-1.19
12	-1.07	-1.07	-1.07	-1.07	-1.07
24	-0.87	-0.87	-0.87	-0.87	-0.87
36	-0.69	-0.69	-0.69	-0.69	-0.69
48	-0.55	-0.55	-0.55	-0.55	-0.55

TABLE A9 VERTICAL STRESS (σ_z) AT r=0

(PAVEMENT No 3)

Z	σ_z (psi)				
	ELSYM5	LAYERS	LAYER15	LAYIT	BISAR
0	−70.00	−70.00	−77.83	−70.00	−70.00
−8	−6.59	−6.59	−6.58	−6.59	−6.57
8		−6.59	−6.58	−6.59	−6.57
14	−4.56	−4.57	−4.57	−4.57	−4.55
30	−2.24	−2.24	−2.24	−2.24	−2.24
48	−1.22	−1.22	−1.22	−1.22	−1.24
72	−0.64	−0.64	−0.64	−0.64	−0.67

TABLE A10 HORIZONTAL STRESS (σ_t) AT r=0.

(PAVEMENT No 3)

Z	σ_t (psi)				
	ELSYM5	LAYERS	LAYER15	LAYIT	BISAR.
0	−151.7	−151.7	−151.9	−151.7	−151.4
−8	+126.6	+126.6	+126.5	+126.6	+126.3
8	−2.1	−2.1	−2.1	−2.1	−2.1
14	−1.05	−1.05	−1.05	−1.05	−1.05
30	−0.3	−0.3	−0.3	−0.3	−0.3
48	−0.1	−0.1	−0.1	−0.1	−0.1
72	0	0	0	0	0

TABLE A 11 HORIZONTAL STRESS (σ_t) AT THE BOTTOM OF THE TOP LAYER.

(PAVEMENT No 3)

r	σ_t (psi)				
	ELSYM5	LAYERS	LAYER15	LAYIT	BISAR
0	126.6	126.6	126.5	126.6	126.3
6	99.7	99.7	99.7	99.7	99.4
12	57.1	57.1	57.1	57.1	57.1
24	19.9	19.9	19.9	19.9	19.9
36	7.1	7.1	7.1	7.1	7.1
48	2.5	2.5	2.5	2.5	2.5

TABLE A 12 SURFACE DEFLECTION

(PAVEMENT No 3.)

r	w ($\times 10^{-2}$) in				
	ELSYM5	LAYERS	LAYER15	LAYIT	BISAR
0	-1.77	-1.78	-1.77	-1.77	-1.78
6	-1.66	-1.66	-1.66	-1.65	-1.66
12	-1.43	-1.42	-1.42	-1.42	-1.42
24	-1.03	-1.03	-1.03	-1.03	-1.03
36	-0.74	-0.74	-0.74	-0.73	-0.74
48	-0.54	-0.54	-0.54	-0.54	-0.54

TABLE A 13 VERTICAL STRESS (σ_z) AT r=0.

(PAVEMENT NO 4)

Z	σ_z (PSI)				
	ELSYM5	LAYER5	LAYER15	LAY17	BISAR
0	-70.00	-70.00	-79.77	-70.00	-70.00
-8	-2.43	-2.43	-2.43	-2.43	-2.49
8		-2.43	-2.43	-2.43	-2.49
-14	-1.44	-1.44	-1.44	-1.44	-1.50
14		-1.44	-1.44	-1.44	-1.50
30	-0.88	-0.88	-0.88	-0.88	-0.94
48	-0.60	-0.60	-0.60	-0.60	-0.64
72	-0.41	-0.41	-0.41	-0.41	-0.42

TABLE A14 HORIZONTAL STRESS (σ_t) AT r=0

(PAVEMENT NO. 4)

Z	σ_t (PSI)				
	ELSYM5	LAYER5	LAYER15	LAY17	BISAR
0	-185.67	-185.70	-192.40	-185.67	-188.37
-8	167.10	167.10	166.97	167.09	169.67
8		0.34	0.34	0.33	0.33
-14	1.63	1.63	1.63	1.63	1.67
14		-0.37	-0.37	-0.37	-0.40
30	-0.19	-0.19	-0.19	-0.19	-0.19
48	-0.12	-0.12	-0.11	-0.12	-0.09
72	-0.07	-0.07	-0.07	-0.07	-0.03

TABLE A 15 HORIZONTAL STRESS (σ_t) AT THE BOTTOM OF THE TOP LAYER

(PAVEMENT NO 4)

r	σ_t (psi)				
	ELSYM5	LAYERS	LAYER15	LAYIT	BISAR
0	167.10	167.10	166.97	167.09	169.67
6	142.33	142.30	142.22	142.33	144.93
12	103.56	103.60	103.42	103.56	103.12
24	57.25	57.25	57.20	57.23	57.26
36	34.45	34.45	34.42	34.45	34.45
48	21.37	21.37	21.36	21.37	21.38

TABLE A16. SURFACE DEFLECTION

(PAVEMENT NO. 4)

r	W ($\times 10^{-2}$) in				
	ELSYM5	LAYERS	LAYER15	LAYIT	BISAR
0	-0.87	-0.87	-0.87	-0.87	-0.88
6	-0.85	-0.85	-0.85	-0.85	-0.86
12	-0.81	-0.81	-0.81	-0.81	-0.81
24	-0.71	-0.71	-0.71	-0.71	-0.71
36	-0.61	-0.61	-0.61	-0.61	-0.61
48	-0.52	-0.52	-0.52	-0.51	-0.52

THE INSTITUTE OF TRANSPORTATION AND TRAFFIC ENGINEERING, UNIVERSITY OF CALIFORNIA

May 22, 1972

ELSYM5 CDC 6400
3/72-3
Gale Ahlborn

ELASTIC LAYERED SYSTEM WITH NORMAL LOADS

Description

The Elastic Layered System computer program (ELSYM5) will determine the various component stresses, strains and displacements along with principal values in a three-dimensional ideal elastic layered system. The layered system being loaded with one or more identical uniform circular loads normal to the surface of the system.

The top surface of the system is free of shear. Each layer is of uniform thickness and extends infinitely in the horizontal direction. All elastic layer interfaces are continuous. The bottom elastic layer may be semiinfinite in thickness or may be given a finite thickness, in which case the program assumes the bottom elastic layer is supported by a rigid base. With a rigid base, the interface between the bottom elastic layer and the base has to be made either fully continuous or slippery.

All locations within the system are described by using the rectangular coordinate system (X, Y, Z) with the XY plane at Z = 0 being the top surface of the elastic system where the loads are applied. The positive Z axis extends vertically down from the surface into the system.

The applied loads are described by any two of the three following items; load in pounds, stress in pounds per square inch, radius of loaded area in inches. The program determines the missing value. Each layer of the system is described by modulus of elasticity, Poisson's ratio and thickness. Each layer is numbered with the top layer as one and numbering each layer consecutively downward.

Program Operating Notes

The program tests all input data. If any input data is out of range as specified under "Limitations," the problem is terminated for that system with an error message and the program goes on to the next system for operation.

The program uses the convention that compressive stresses are negative and tensile stresses are positive.

The output of the program gives for each depth (Z) all the results for all the XY points. The results for each point are the total results for that point obtained by summing the contribution by each load. When a Z value is determined to be on an interface, the results are determined using the characteristics of the upper of the two layers.

Limitations

The following are limitations of the program and/or method.

1. Number of different systems for solution; minimum of one, maximum of five.
2. Number of elastic layers in the system; minimum of one, maximum of five.
3. Number of identical uniform circular loads; minimum of one, maximum of ten.
4. Number of points in the system where results are desired; minimum of one (one XY and one Z), maximum of 100 (ten XY and ten Z).
5. Where there is a rigid base specified, the maximum Z value cannot exceed the depth to the rigid base.
6. All input values except XY positions must be positive.
7. Poisson's ratio must not have a value of one. Poisson's ratio for a bottom elastic layer on a rigid base must not be within the range of 0.748 to 0.752.
8. The program uses a truncated series for the integration process that leads to some approximation for the results at and near the surface and at points out at some distance from the load.

Input Cards

The notation CC refers to card columns, with the range of columns being inclusive. All "Real" values (REAL) are punched with a decimal point as a part of the value and all "integer" values (INTEGER) are to be punched without a decimal point and right justified in the data field.

1. CC 1 - 5 (INTEGER) number of systems to be run.

2. CC 1 - 3 (INTEGER) punch the number 999.

   CC 5 - 60 (ALPHA) any combination of alphameric characters may be used to identify the problem to be solved.

3. CC 1 - 5 (INTEGER) number of elastic layers in the system.

   CC 6 - 10 (INTEGER) number of uniform circular loads to be applied normal to the surface of the system

   CC 11 - 15 (INTEGER) number of XY locations where results are desired.

   CC 16 - 20 (INTEGER) number of Z locations where results are desired.

4. CC 1 - 5 (INTEGER) layer number
   CC 6 - 10 (REAL) thickness of layer in inches
   CC 11 - 15 (REAL) Poisson's ratio of layer
   CC 16 - 25 (REAL) modulus of elasticity for layer.

One card for each elastic layer in the system, leave thickness blank for bottom elastic layer when layer is to be semiinfinite in thickness. If bottom elastic layer is resting on a rigid base, insert the thickness of the bottom elastic layer and CC 30 - 31 (ALPHA) punch FF for full friction rigid base interface <u>or</u> CC 30 - 31 (ALPHA) punch NF for no friction rigid base interface. Cards have to be in sequence from top to bottom elastic layer.

5. CC 1 - 10 (REAL) load force in pounds

   CC 11 - 20 (REAL) load pressure in pounds per square inch

   CC 21 - 30 (REAL) load radius in inches. Any two of the above items can be input, program determines the third. Only one card required.

6. CC 1 - 10 (REAL) X position of a load

CC 11 - 20 (REAL) Y position of a load

One card per load

7. CC 1 - 10 (REAL) X position for evaluation

CC 11 - 20 (REAL) Y position for evaluation

One card for each XY position for evaluation

8. CC 1 - 5 (REAL) first Z value for evaluation

CC 6 - 10 (REAL) second Z value for evaluation

CC 11 - 15 (REAL) third Z value for evaluation, etc.

Only one card required, maximum of ten values on the card.

To evaluate a second system, follow card type 8 by card types 2 - 8 for the second system, etc.

LESSON OUTLINE

AASHTO INTERIM GUIDE FOR FLEXIBLE PAVEMENTS

Instructional Objectives

1. To provide the student with a basic knowledge of the AASHTO design guide for flexible pavements.

2. To illustrate the practical use of the AASHTO design guide for flexible pavements.

Performance Objectives

1. The student should be able to identify each of the design inputs used in the AASHTO method.

2. The student should be able to perform a simple thickness design using the AASHTO method.

Abbreviated Outline	Time Allocation, mins.
1. Introduction	10
2. Design Equation	10
3. Design Inputs	20
4. Design Example	10
	50

Reading Assignment

1. Yoder and Witczak - Chapter 15, pp 504-519

2. AASHTO Interim Guide - Chapter II.

LESSON OUTLINE
AASHTO INTERIM GUIDE FOR FLEXIBLE PAVEMENT

1.0 INTRODUCTION

1.1 Based on Results of AASHO Road Test (Slides 19.1 - 19.9)

The general background of the AASHO Road Test has been discussed in previous lectures.

1.2 Defines Failure Based on User Considerations

1.2.1 Serviceability. Ability of a pavement to serve the traffic for which it was designed.

1.2.2 Performance. Ability of a pavement to satisfactorily serve traffic over a period of time.

1.2.3 Rating Scale of Serviceability - PSR. (Visual Aid 19.1)

1.2.4 Correlation Between PSI and Pavement Properties.

$$PSI = 5.03 - 1.91 \text{ Log } (1 + SV) - 1.38\, \overline{RD}^2 - .01(C + P)^{.5} \qquad \text{(Eq 1)}$$

where

PSI = present serviceability index

SV = slope variance

$\overline{RD}$ = rut depth

C + P = cracking and patching indices

1.3 Basis for Design Equations (Slides 19.15, 19.16, and 19.17)

(a) Effect of component thickness and material type.
(b) Effect of magnitude and frequency of axle loads.
(c) Effect of performance of test sections.

2.0 PERFORMANCE EQUATION FOR FLEXIBLE PAVEMENTS (AASHO ROAD TEST)

2.1 The General AASHO Road Test Equation (Slides 19.18, 19.19, 19.20 and 19.21)

$$G_t = \beta(\text{Log } W_t - \text{Log } \rho)$$

where

G_t = a function (the logarithm) of the ratio of serviceability loss at time to the maximum loss to a serviceability index level of 1.5.

β = a function of design and load variables that influence the shape of the performance curve

$$\beta = 0.40 + \frac{1094}{(\overline{SN} + 1)^{5.19}} \qquad \text{(Eq 3)}$$

for the AASHO Road Test conditions, and for an 18,000 pound single axle load.

SN = structural number

W_t = axle load applications to time t

ρ = a function of design and load variables denoting the expected number of axle load applications to a serviceability index of 1.5.

$$\text{Log } \rho = 9.36 \text{ Log } (\overline{SN} + 1) - 0.20 \qquad \text{(Eq 4)}$$

for AASHO Road Test conditions, and for an 18,000 pound single axle load.

2.2 AASHTO Design Equation (Slides 19.22 - 19.26)

Combining and rewriting the number of axle loads carried can be expressed as

$$\text{Log } W_{t_{18}} = 9.36 \text{ Log } (SN + 1) - 0.20 + \frac{G_t}{0.40 + \dfrac{1094}{(\overline{SN} + 1)^{5.19}}} \qquad \text{(Eq 5)}$$

where

$W_{t_{18}}$ = number of 18,000 pound single axle loads to time t (if equivalent axle loads are used, this can be expressed as EAL_{18} to time t)

$$G_t = \text{Log}[(4.2 - p_t)/(4.2 - 1.5)] \qquad \text{(Eq 6)}$$

where p_t equals serviceability index at time t.

2.3. The Extention of Design Equation (Slides 19.27 - 19.35)

In order to extend Eq 5 to other subgrade types ar^d climates, it was necessary to develop the following:

2.3.1 Soil Support. Soil supp... value, S ith a scale ranging from 1.0 to 10.0, with the road test subgrade soil having a value of 3.0.

2.3.2 Regional Factor. Regional factor, R, with a potential range of 0.5 to 5.0.

2.4 Final Design Equation

The final design equation incorporating soil support and regional factor is

$$\text{Log } W_{t_{18}} = 9.36 \text{ Log } (SN + 1) - 0.20 + \frac{G_t}{0.40 + \frac{1094}{(\overline{SN} + 1)^{5.19}}}$$

$$- \text{ Log } R + 0.372 \ (S - 3.0) \qquad \text{(Eq 7)}$$

where

R = regional factor

S = soil support value for the particular site and conditions

$\overline{SN}$ = weighted structural number (for the soil support and regional factors used)

3.0 DESIGN INPUTS

3.1 Terminal Serviceability (p_t)

The lowest serviceability that can be tolerated on the road at the end of the traffic analysis period before further action is warranted

3.1.1 Usually taken as 2.0 or 2.5.

(a) High volume roads p_t = 2.5

(b) Low volume roads p_t = 2.0

3.1.2 Very low volume roads reduce traffic analysis time period.

3.2 Regional Factor (R) (Visual Aid 19.3)

A numerical factor used to adjust the structural number of a flexible pavement structure for climatic and environmental conditions, different from those at the AASHO Road Test.

(a) Not well documented.
(b) Many states have developed own charts. (Visual Aid 19.4)
(c) Usual range is from 0.5 to 4.0.

3.3 Structural Number (SN)

An index number derived from an analysis of traffic, roadbed soil conditions, and regional factor that may be converted to thickness of various flexible pavement layers through use of suitable layer coefficients related to the type of material being used in each layer of the pavement structure.

3.3.1 Assumed structure. (Visual Aid 19.5)

3.3.2 Structural number equation.

$$SN - a_1D_1 + a_2D_2 + a_3D_3$$

where

a_i = layer coefficient for i^{th} layer (Visual Aid 19.6)

D_i = layer thickness for i^{th} layer

(i = 1, 2, 3)

3.3.3 Layered concept design check. Thickness of pavement above any specific layer must be enough such that excessive stresses do not occur in that layer (Visual Aid 19.5).

3.4 Soil Support (S_i)

An index number that expresses the relative ability of a soil or aggregate mixture to support traffic loads through a flexible pavement structure.

3.4.1 Not determined by direct testing.

3.4.2 Correlations. (Visual Aid 19.7)

(a) CBR
(b) R-value
(c) Texas triaxial
(d) Group index
(e) Resilient Modulus
(f) Others (Pedology, Frost index, experience, etc.)

4.0 DESIGN EXAMPLE

4.1 Input Data

(a) Interstate Highway - 1,000 equivalent 18-kip SAL's per day
(b) Regional Factor = 1
(c) Subgrade CBR = 11 (sandy clay)
(d) Subbase CBR = 20 (sand-gravel)
(e) Base (CBR = 78 (crushed stone)
(f) Surface Modulus = 5×10^5 psi (asphalt concrete)
(g) ASSUME $p_t = 2.5$

4.2 Subgrade Support Values and Structural Numbers (Visual Aids 19.8 & 19.9)

(a) Top of subgrade $S = 5.0 \rightarrow SN = 4.15$
(b) Top of subbase $S = 5.2 \rightarrow SN = 3.60$
(c) Top of Base $S = 8.0 \rightarrow SN = 2.95$

Therefore

$$a_1D_1 + a_2D_2 + a_3D_3 \geq 4.15$$

4.3 Layer Coefficients (Visual Aid 19.6)

(a) surface $a_1 = 0.46$

(b) base $a_2 = 0.14$

(c) subbase $a_3 = 0.095$

Therefore

$$0.46D_1 + 0.14D_2 + 0.095D_3 \geq 4.15$$

4.4 Minimum Layer Thicknesses

$$D_1(\text{min}) = \frac{SN_2}{a_1} = \frac{2.95}{0.46} = 6.41 \text{ (6.5 in.)}$$

$$D_2(\text{min}) = \frac{SN_3 - SN_2}{a_2} = \frac{3.60 - (6.5)(.46)}{0.14} = 4.36 \text{ (4.5 in.)}$$

$$D_3(\text{min}) = \frac{SN_4 - (SN_2 + SN_3)}{a_3} = \frac{4.15 - \left[(6.5)(.46) + (4.5)(.14)\right]}{0.095}$$

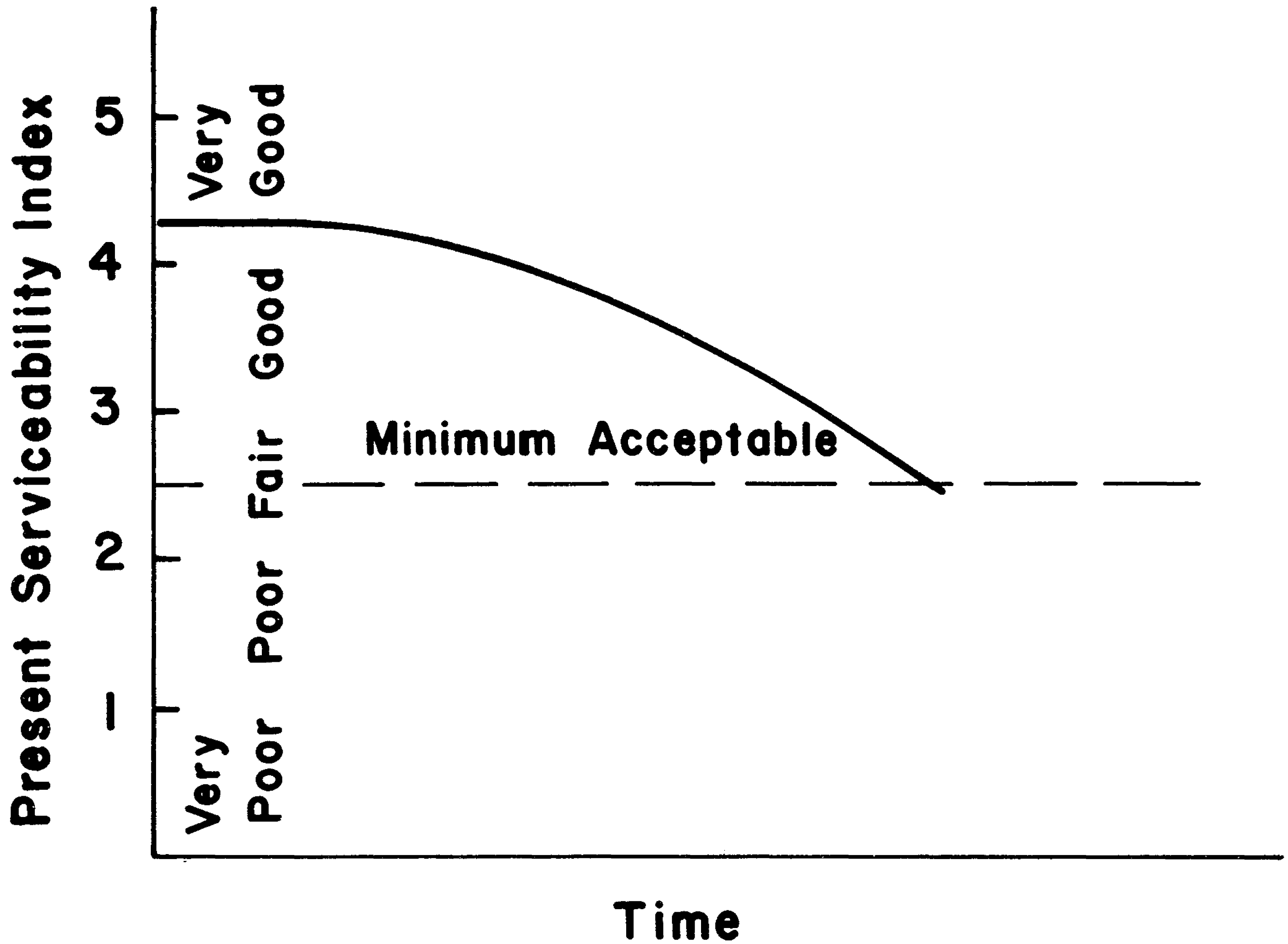
$$= 5.58 \text{ (6 in.)}$$

therefore

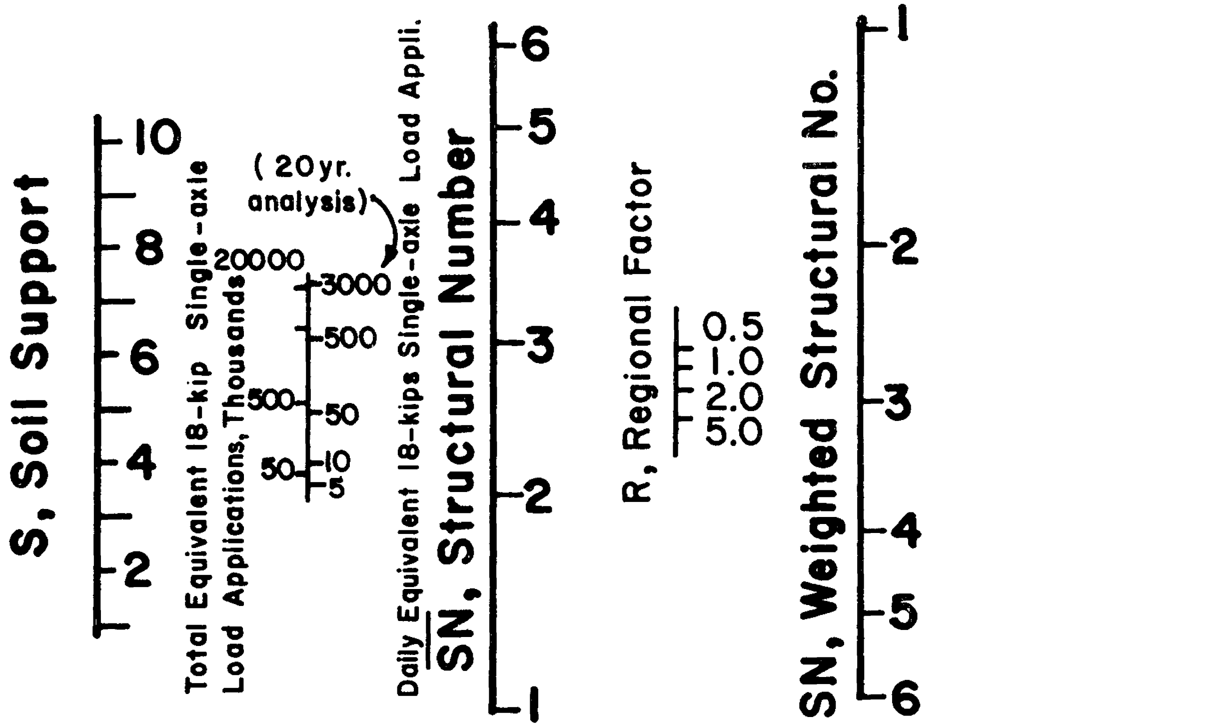
$$6.5(.46) + 4.5(.14) + 6(.095) \geq 4.15$$
$$4.19 \geq 4.15 \checkmark$$

LESSON OUTLINE
AASHTO INTERIM GUIDE FOR FLEXIBLE PAVEMENTS

VISUAL AID	TITLE
Visual Aid 19.1.	Serviceability concept.
Visual Aid 19.2(a).	Design nomographs for AASHTO procedure.
Visual Aid 19.2(b).	Design nomographs for AASHTO procedure.
Visual Aid 19.3.	Hypothetical regional factors for ASHTO design procedure.
Visual Aid 19.4.	Factors for climatic and environmental effects, Idaho.
Visual Aid 19.5.	Assumed pavement structure for AASHTO design procedure.
Visual Aid 19.6(a).	Nomographs for layer coefficient determination.
Visual Aid 19.6(b).	Nomographs for layer coefficient determination.
Visual Aid 19.6(c).	Nomographs for layer coefficient determination.
Visual Aid 19.6(d).	Nomographs for layer coefficient determination.
Visual Aid 19.6(e).	Nomographs for layer coefficient determination.
Visual Aid 19.7(a).	Soil support correlation after AASHTO.
Visual Aid 19.7(b).	Soil support correlation after Vantil, et al.
Visual Aid 19.8.	Determination of soil support values for design example.
Visual Aid 19.9(a).	Determination of structural numbers for design example.
Visual Aid 19.9(b).	Determination of structural numbers for design example.

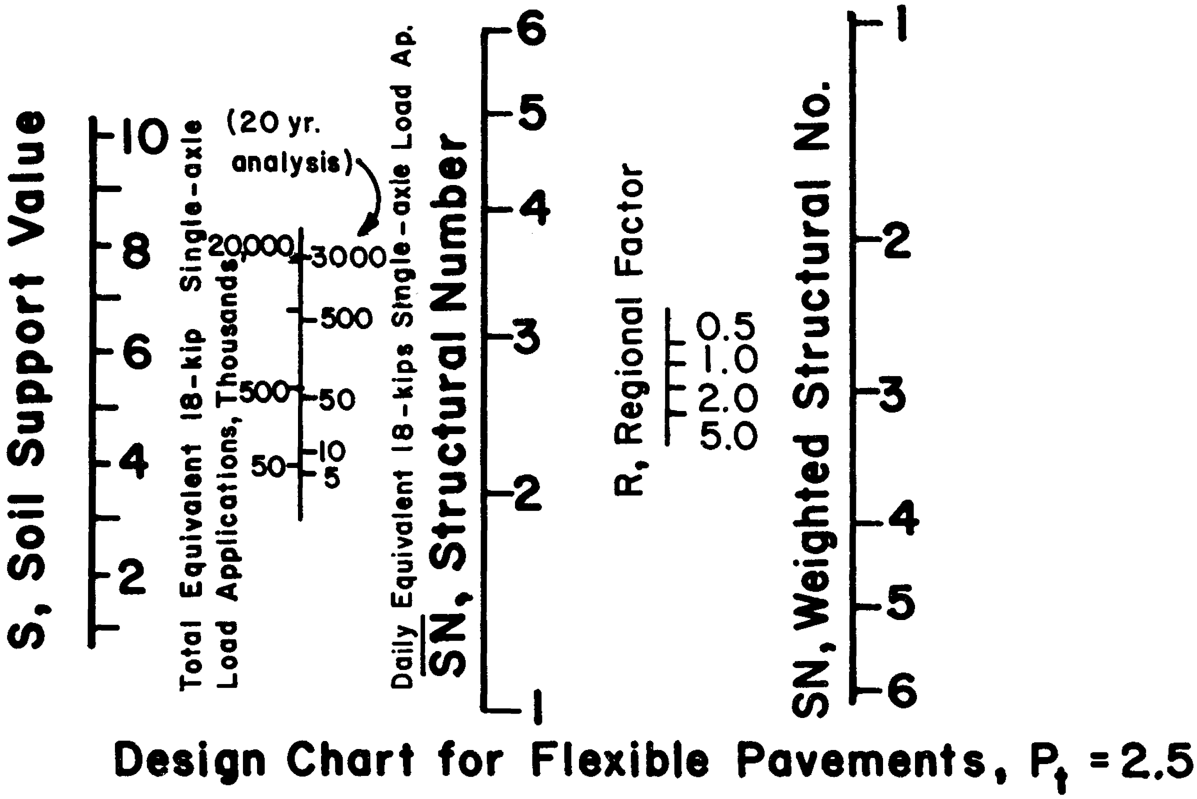


Visual 19.1. Serviceability Concept

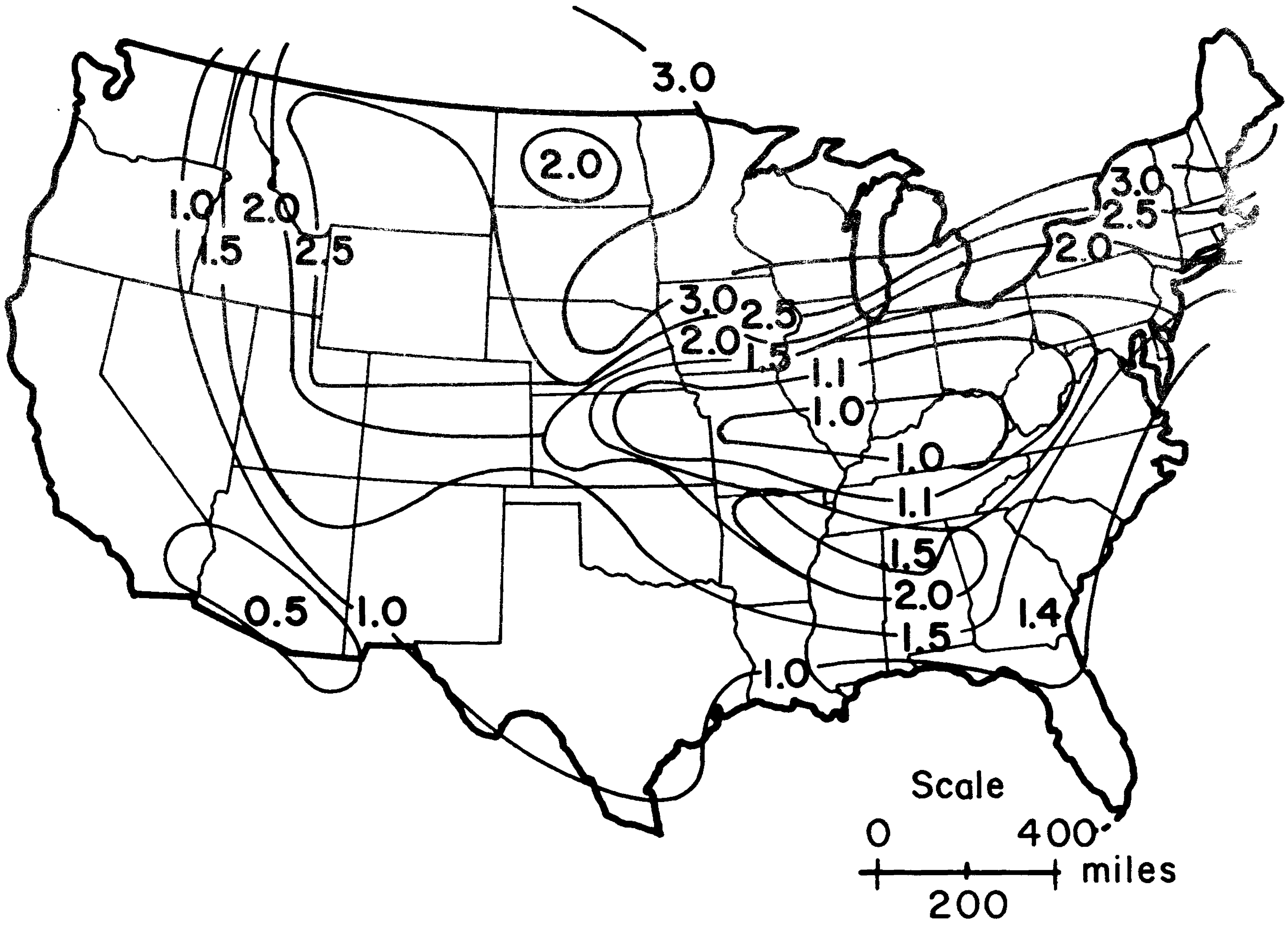


Design Chart for Flexible Payments, $P_t = 2.0$

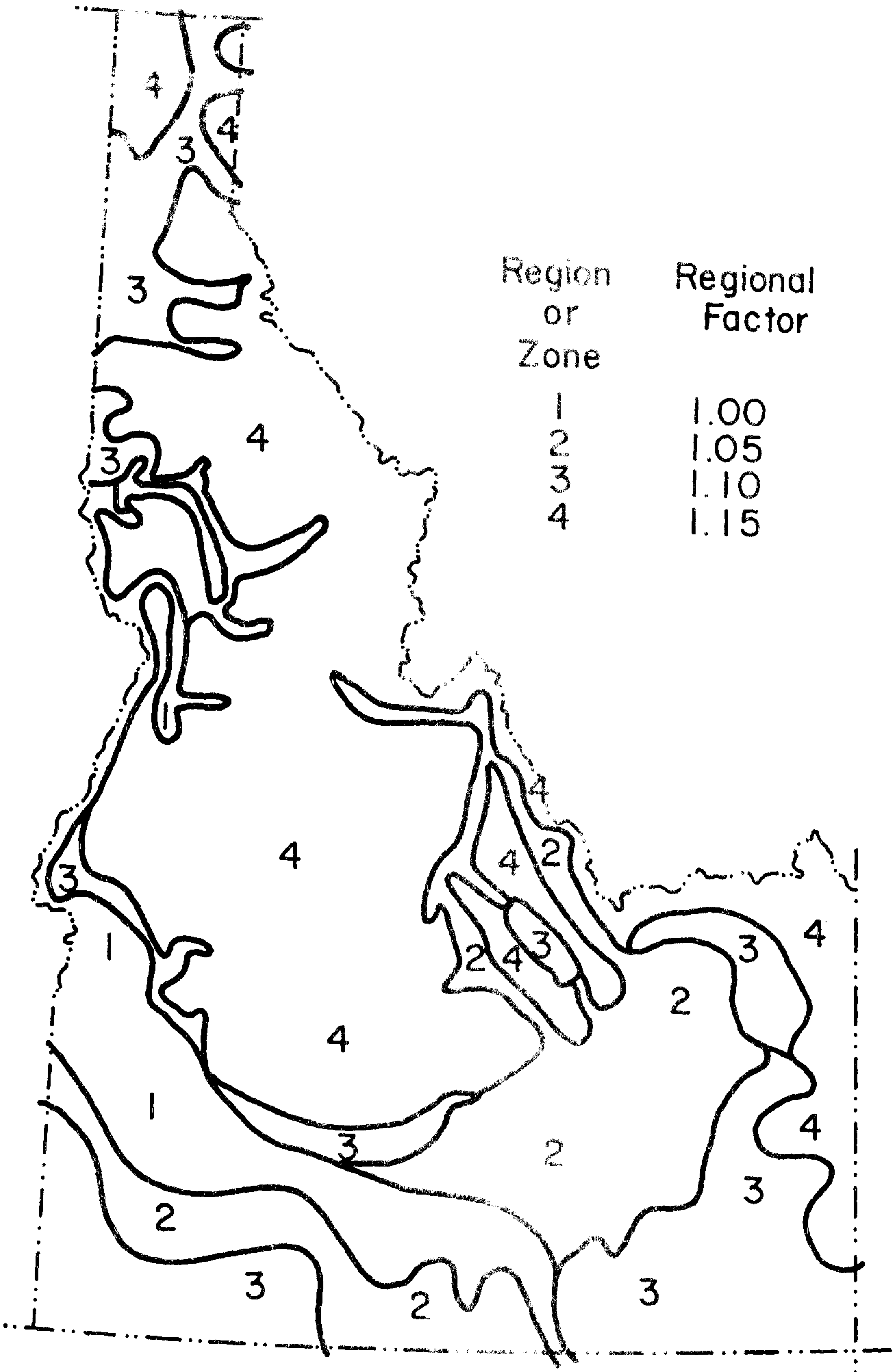
Visual 19.2(a) Design nomographs for AASHTO procedure



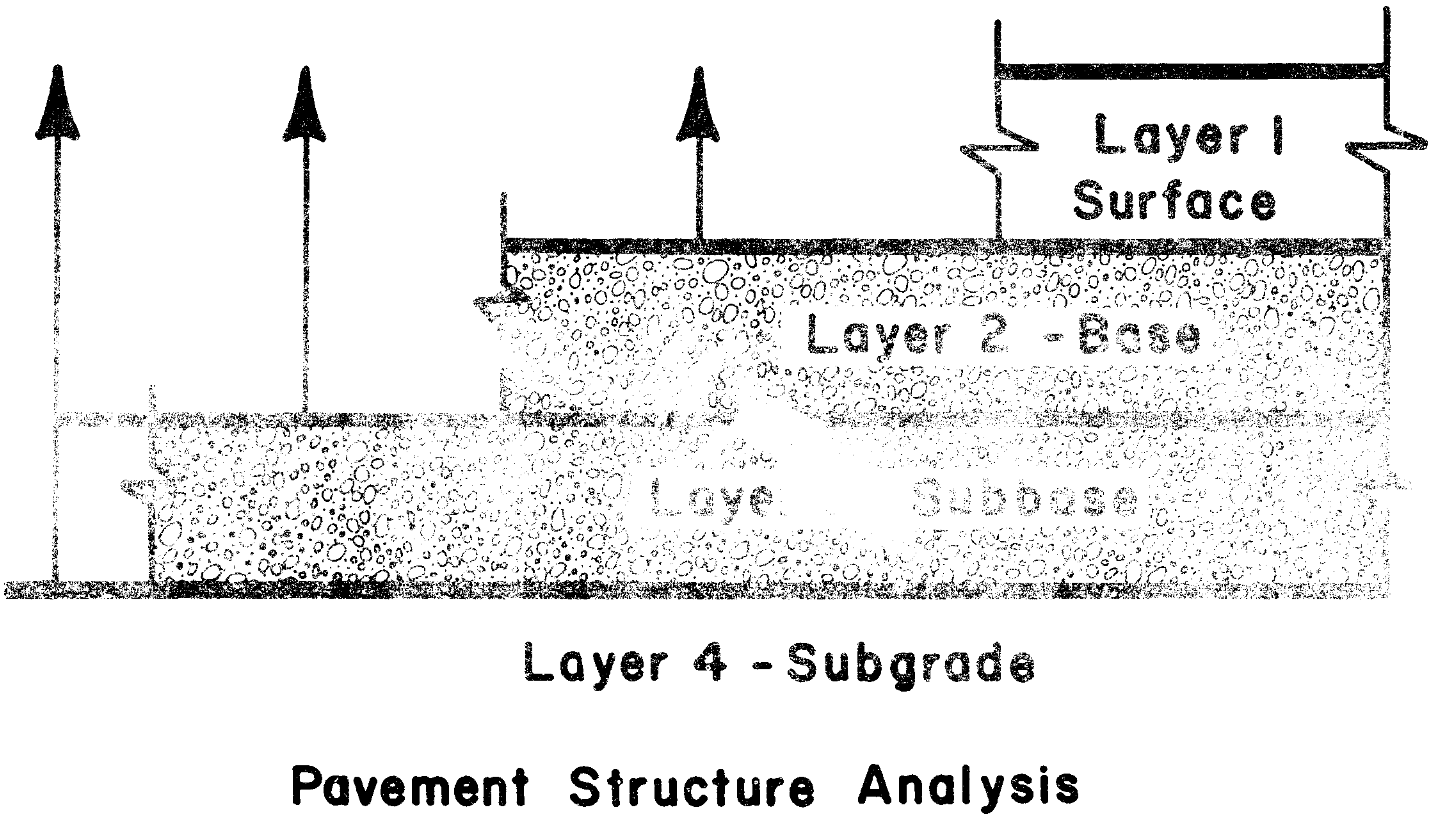
Visual 19.2(b) Design nomographs for AASHTO procedure



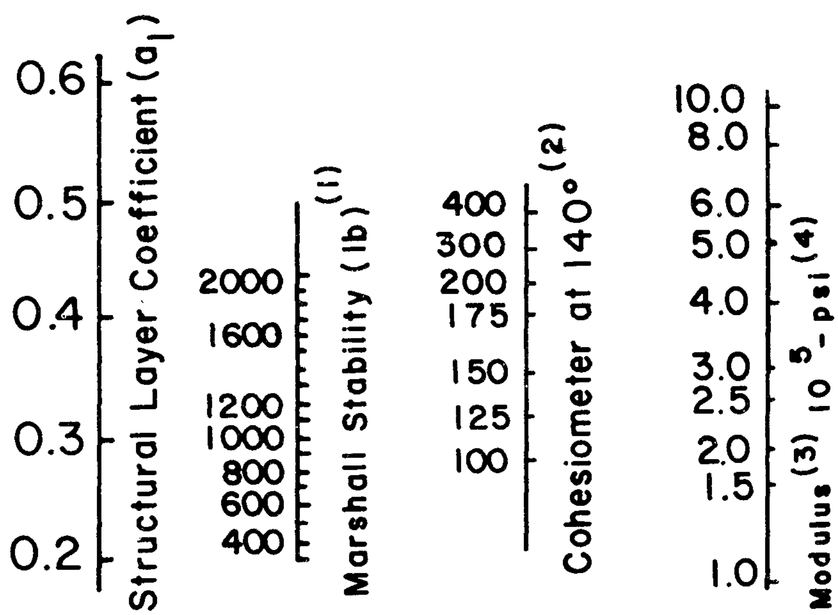
Visual 19.3. Hypothetical regional factors for AASHTO design procedure.



Visual 19.4. Factors for climatic and environmental effects, Idaho.



Visual 19.5. Assumed pavement structure for AASHTO design procedure.



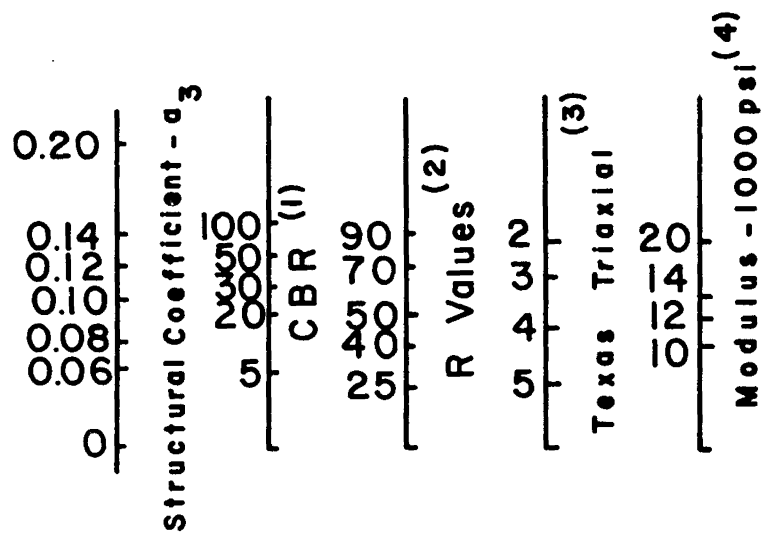
(1) Scale derived by averaging correlation obtained from the Asphalt Insitute, Illinois, Louisiana, New Mexico, and Wyoming.

(2) Scale derived by averaging correlations obtained from California and Texas.

(3) Scale derived on this project.

(4) Modulus at 68° F

Visual 19.6(a). Nomographs for layer coefficient determination.



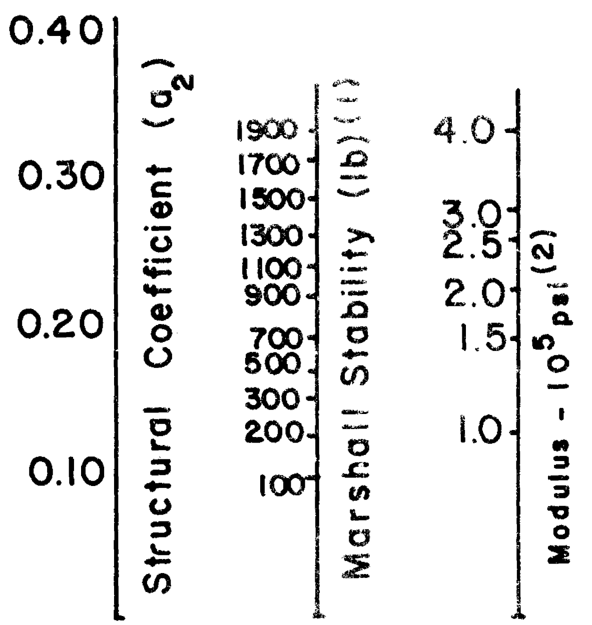
(1) Scale derived from correlations from Illinois.

(2) Scale derived from correlations obtained from the Asphalt Institute, California, New Mexico, and Wyoming.

(3) Scale derived from correlations obtained from Texas.

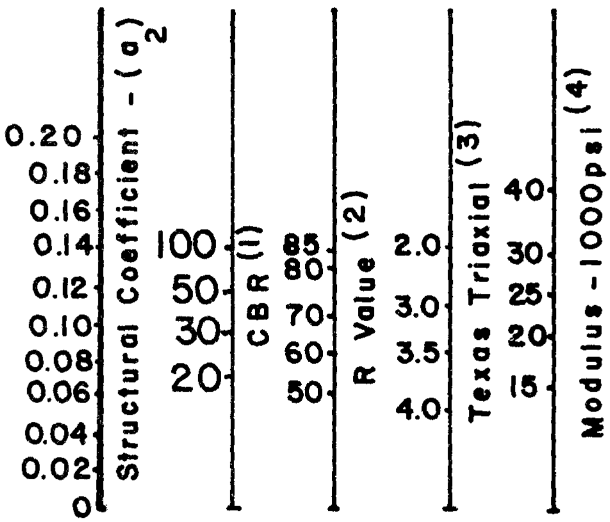
(4) Scale derived on this project.

Visual 19.6(b). Nomographs for layer coefficient determination.



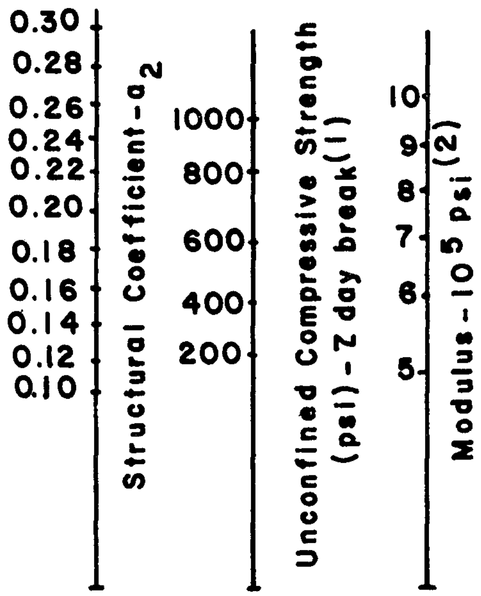
(1) Scale derived by correlation obtained from Illinois(III).
(2) Scale derived on this project.
(3) Modulus at 68° F

Visual 19.6(c). Nomographs for [illegible] coefficient determination.



(1) Scale derived by averaging correlations obtained from Ill.
(2) Scale derived by averaging correlations obtained from Cali., New Mexico and Wyoming.
(3) Scale derived by averaging correlations obtained from Texas.
(4) Scale derived on project.

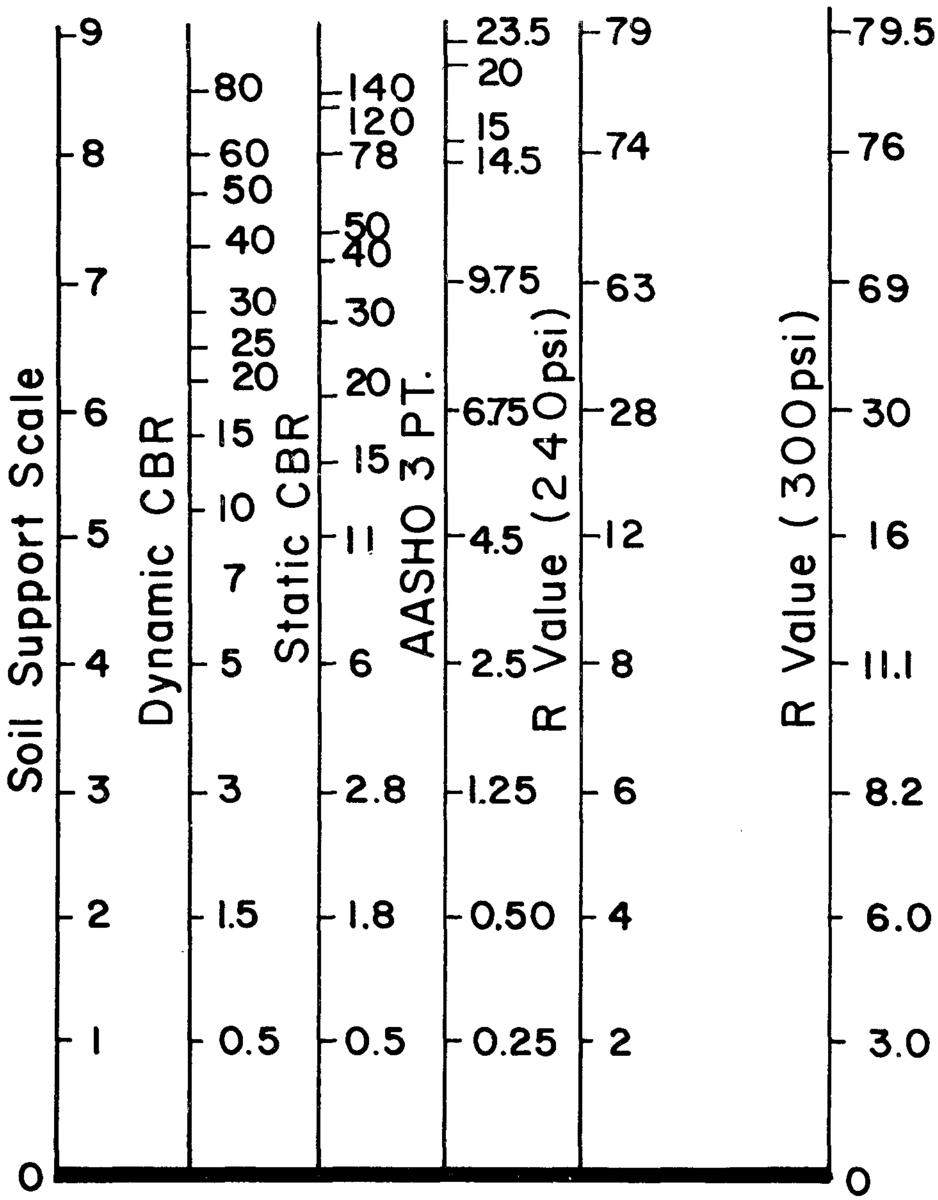
Visual 19.6(d). Nomographs for layered coefficient determination.



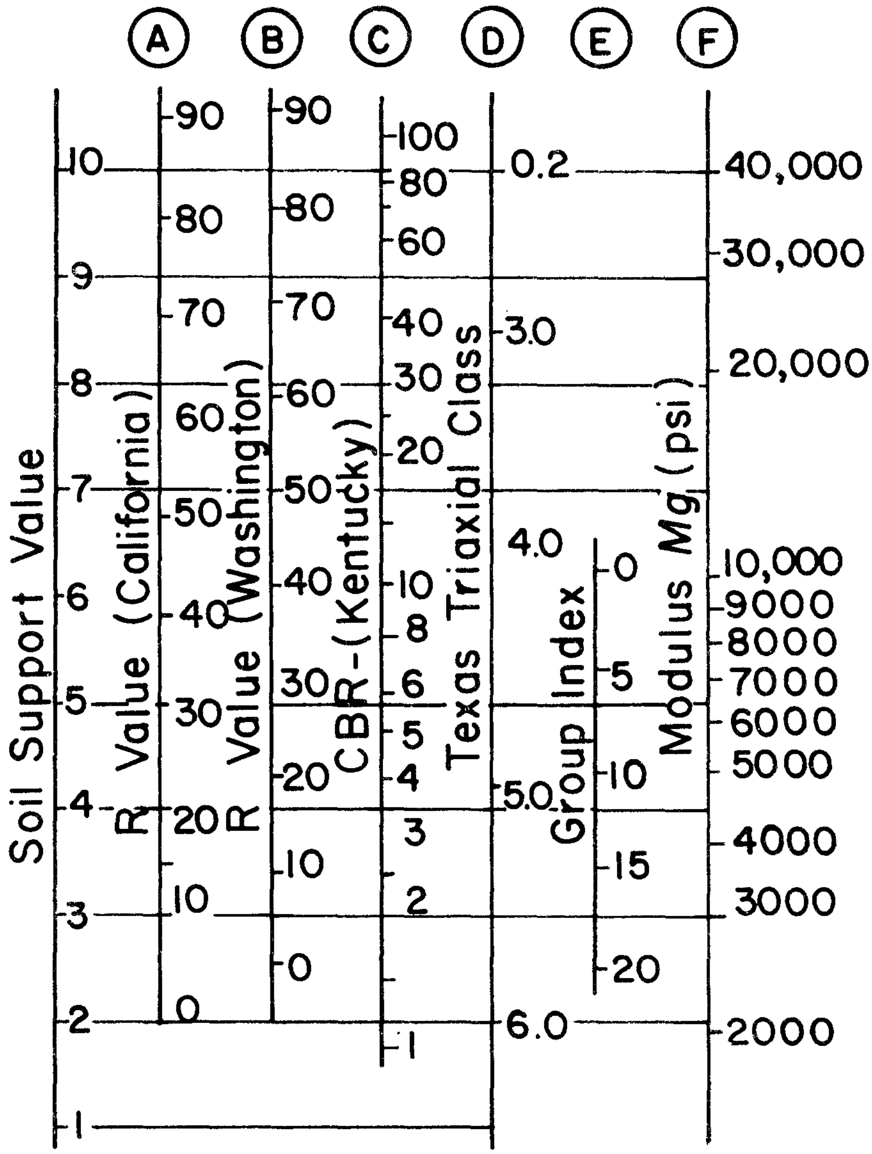
(1) Scale derived by averaging correlations from Ill., La., and Texas.

(2) Scale derived on this project.

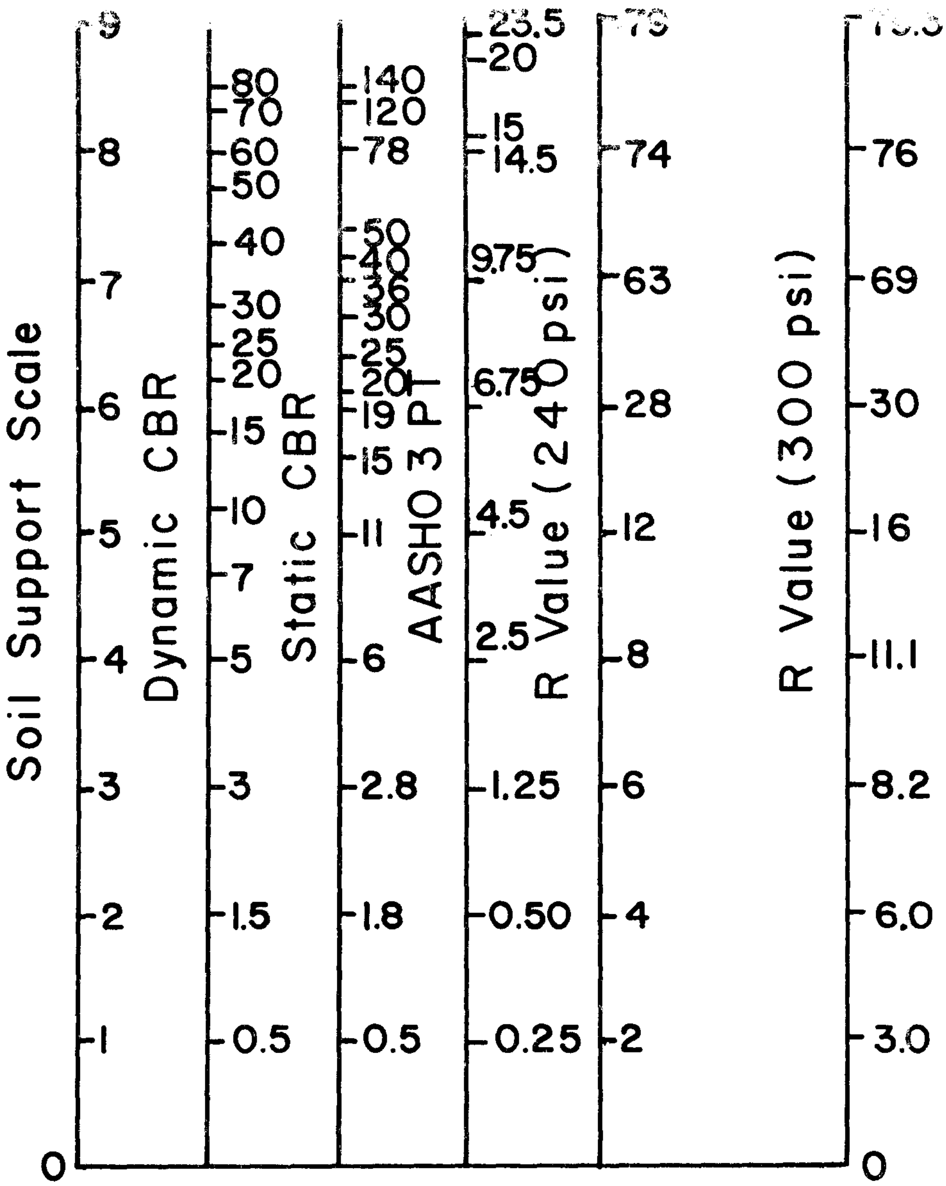
Visual 19.6(e). Nomographs for layered coefficient determination.



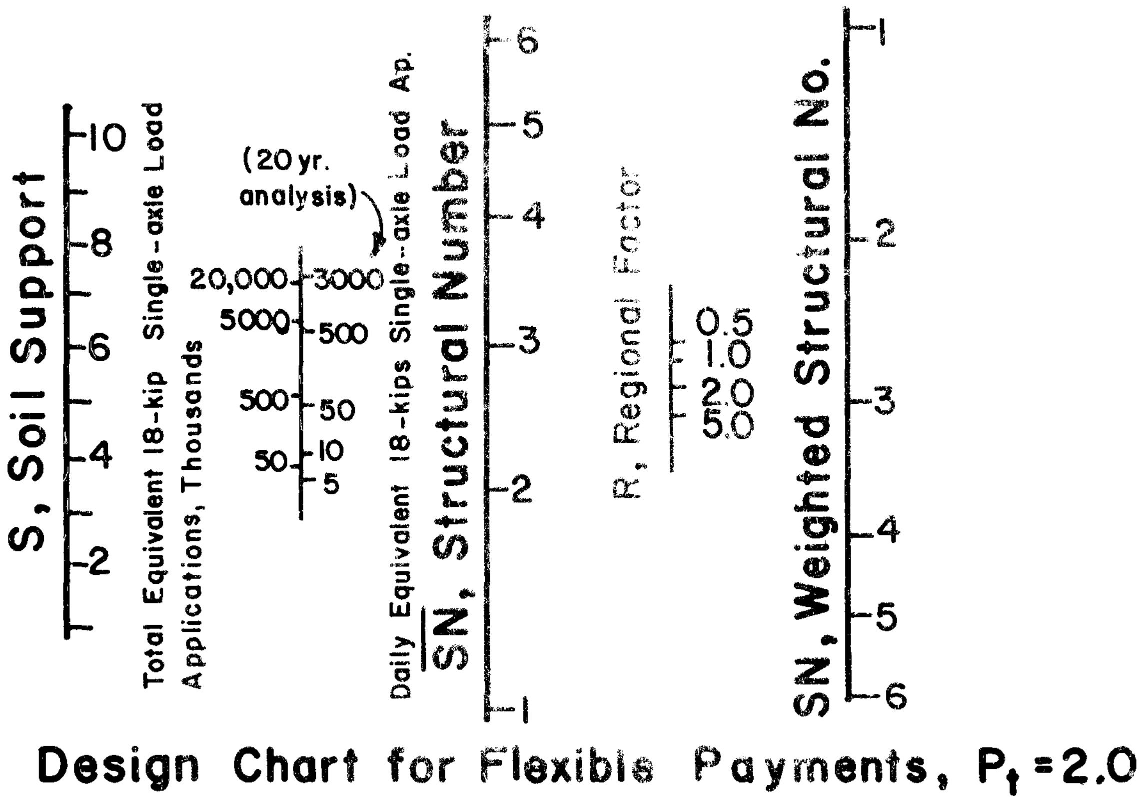
Visual 19.7(a). Soil support correlation after AASHTO.



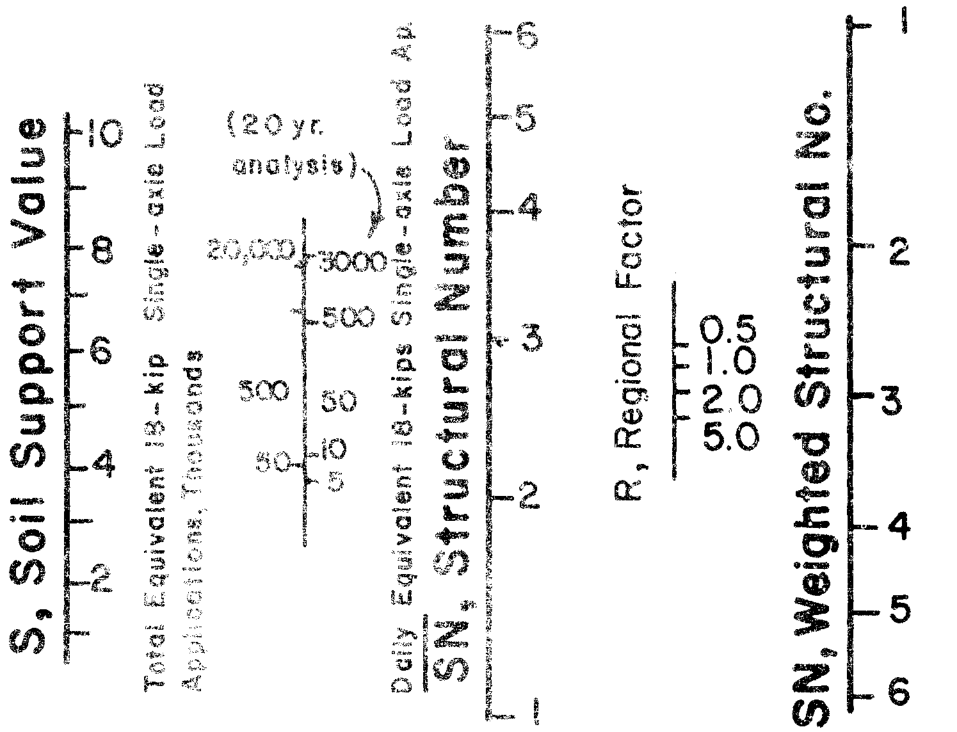
Visual 19.7(b). Soil support correlations after Vantil, et al.



Visual 19.8. Determination of soil support values for design example.

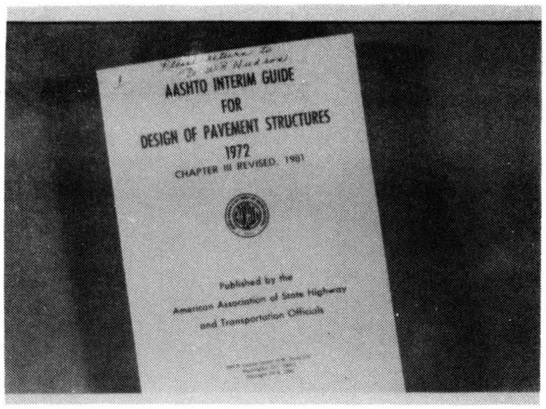


Visual 19.9(a). Determination of structural numbers for design example.

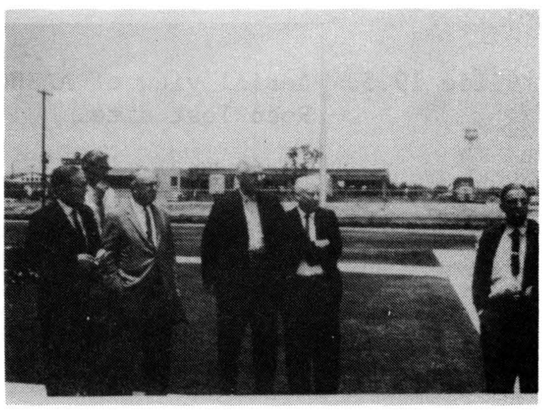


Design Chart for Flexible Pavements, $P_t = 2.5$

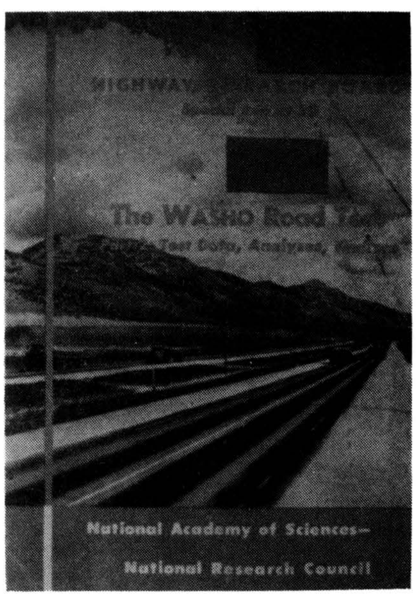
Visual 19.9(b). Determination of structural numbers for design example.



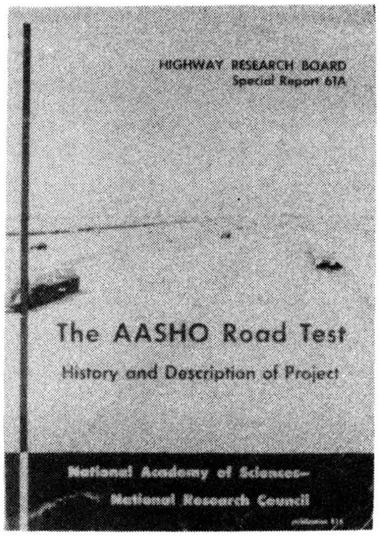
Slide 19.1. AASHTO Design Guide.



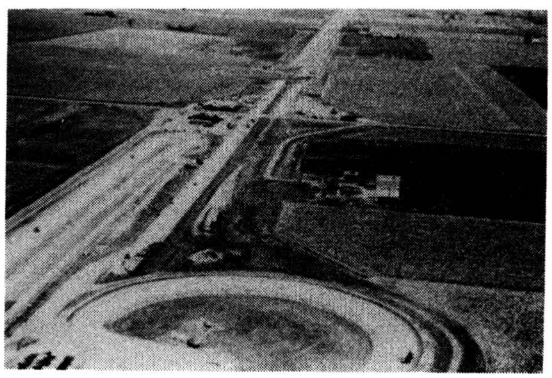
Slide 19.2. Some staff members of the AASHO Road Test.



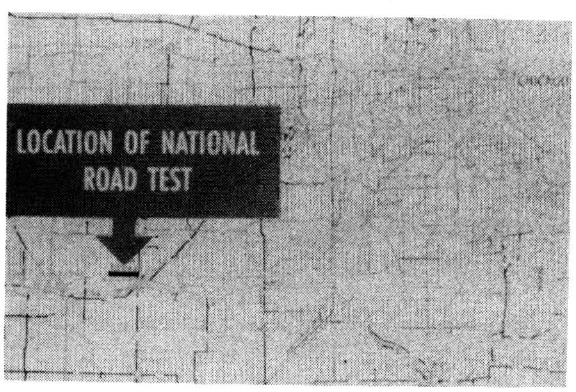
Slide 19.3. WASHO Road Test.



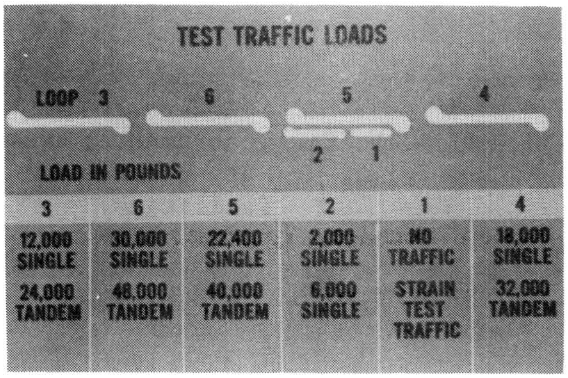
Slide 19.4. The AASHO Road Test - project description.



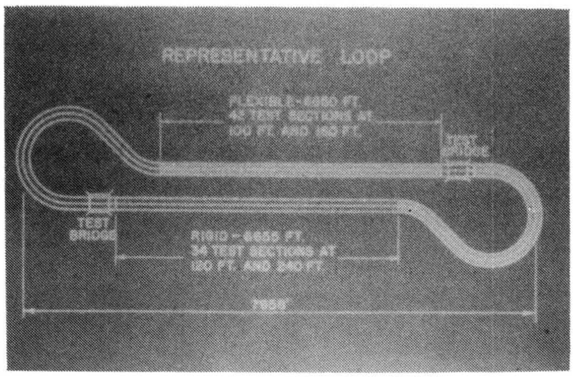
Slide 19.5. Aerial view of AASHO Road Test site.



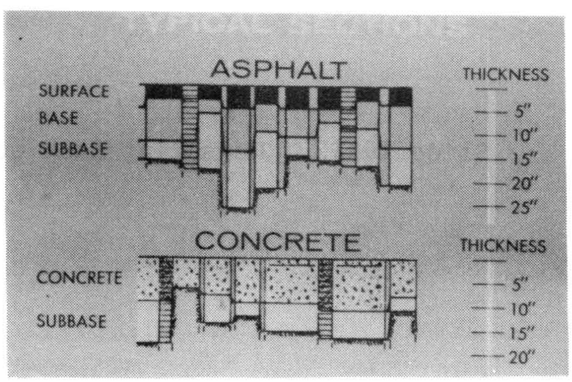
Slide 19.6. Map showing the location of AASHO Road Test site.



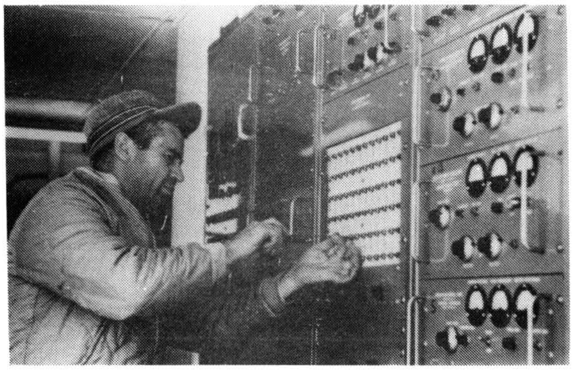
Slide 19.7. Test traffic loads.



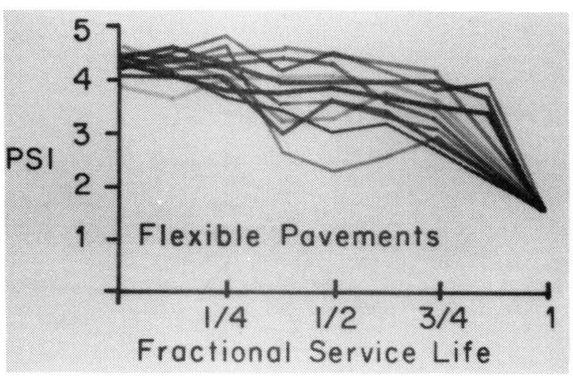
Slide 19.8. Typical layout of a test loop.



Slide 19.9. Layer thicknesses of asphalt and concrete pavements constructed at AASHO Road Test site.

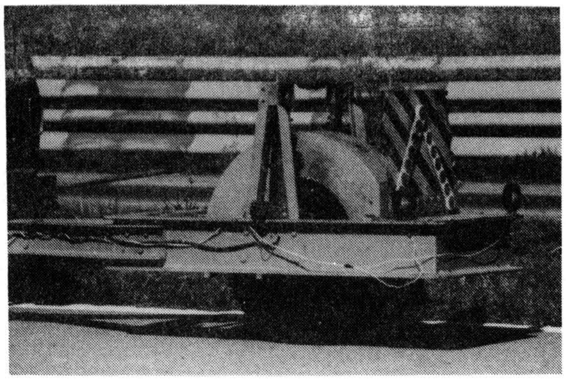


Slide 19.10. Instrumentation and data processing facilities.



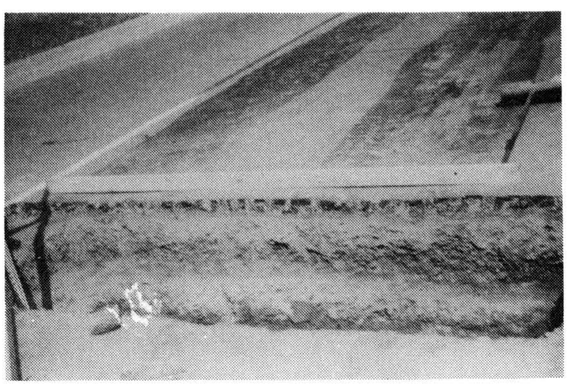
Slide 19.11. Serviceability history of flexible pavements.

Slide 19.12. Measuring road roughness.

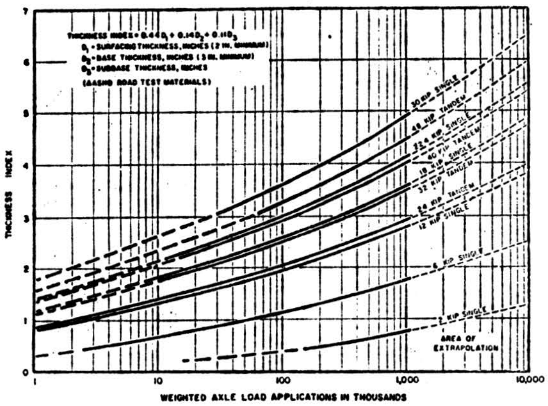


Slide 19.13. Road roughness device.

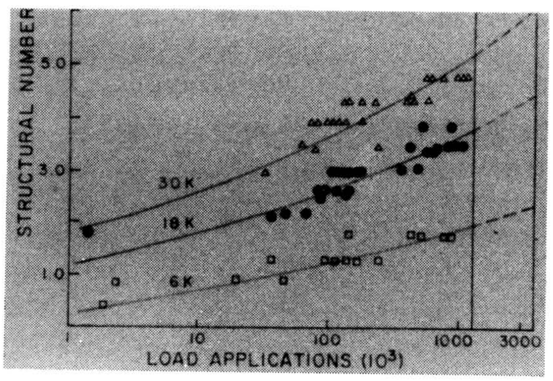
Slide 19.14. Example of preuse control on AASHO Road Test.



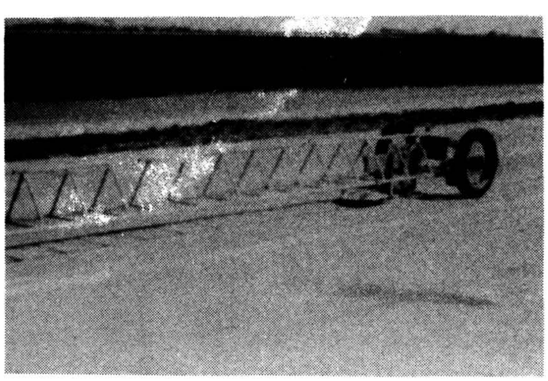
Slide 19.15. Examination of sub-layers by excavating a ditch (destructive testing).



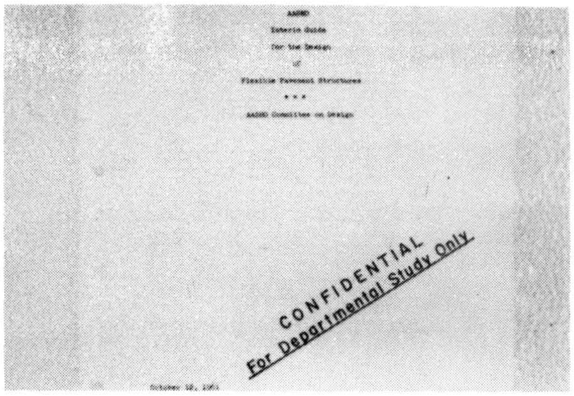
Slide 19.16. Development of design equations.



Slide 19.17. Development of design equations based on road test findings.



Slide 19.18. Deflection measuring device.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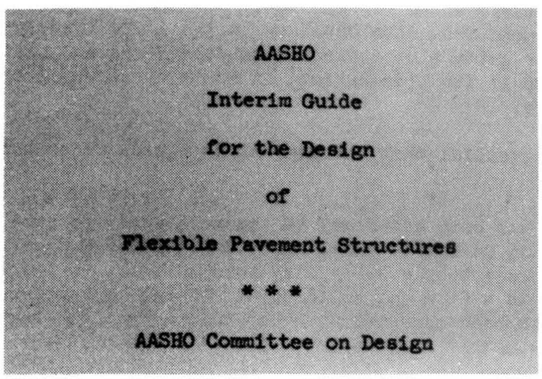
Slide 19.19. AASHO Interim Guide.


AASHO

Interim Guide

for the Design

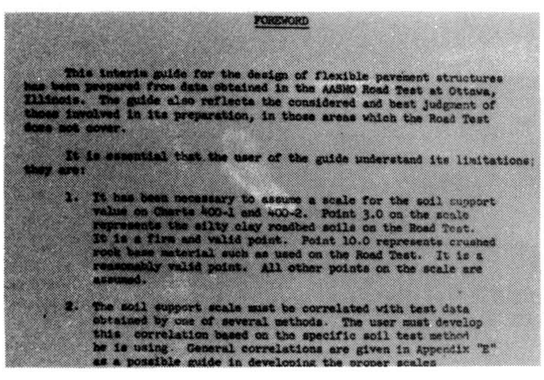
of

Flexible Pavement Structures

* * *

AASHO Committee on Design

Slide 19.20. Guide for the Design of Flexible Pavements.


FOREWORD

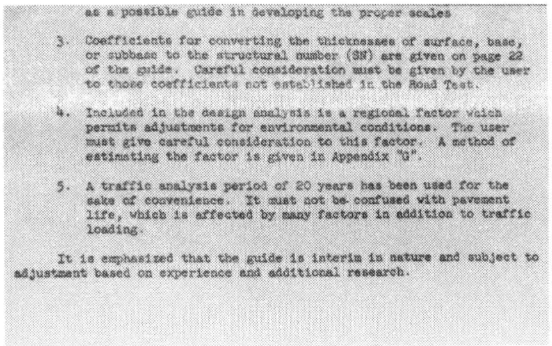
This interim guide for the design of flexible pavement structures has been prepared from data obtained in the AASHO Road Test at Ottawa, Illinois. The guide also reflects the considered and best judgment of those involved in its preparation, in those areas which the Road Test does not cover.

It is essential that the user of the guide understand its limitations; they are:

1. It has been necessary to assume a scale for the soil support value on Charts 400-1 and 400-2. Point 3.0 on the scale represents the silty clay roadbed soils on the Road Test. It is a firm and valid point. Point 10.0 represents crushed rock base material such as used on the Road Test. It is a reasonably valid point. All other points on the scale are assumed.

2. The soil support scale must be correlated with test data obtained by one of several methods. The user must develop this correlation based on the specific soil test method he is using. General correlations are given in Appendix "E" as a possible guide in developing the proper scales

Slide 19.21. Scope and limitations of AASHO Guide.



as a possible guide in developing the proper scales

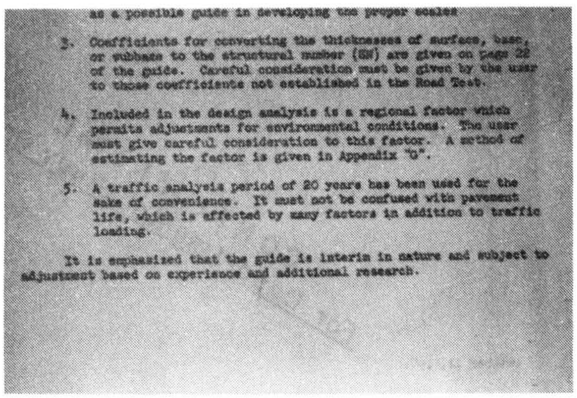
3. Coefficients for converting the thicknesses of surface, base, or subbase to the structural number (SN) are given on page 22 of the guide. Careful consideration must be given by the user to those coefficients not established in the Road Test.

4. Included in the design analysis is a regional factor which permits adjustments for environmental conditions. The user must give careful consideration to this factor. A method of estimating the factor is given in Appendix "G".

5. A traffic analysis period of 20 years has been used for the sake of convenience. It must not be confused with pavement life, which is affected by many factors in addition to traffic loading.

It is emphasized that the guide is interim in nature and subject to adjustment based on experience and additional research.

Slide 19.22. Scope and limitations of AASHO Guide (continued).



as a possible guide in developing the proper scales

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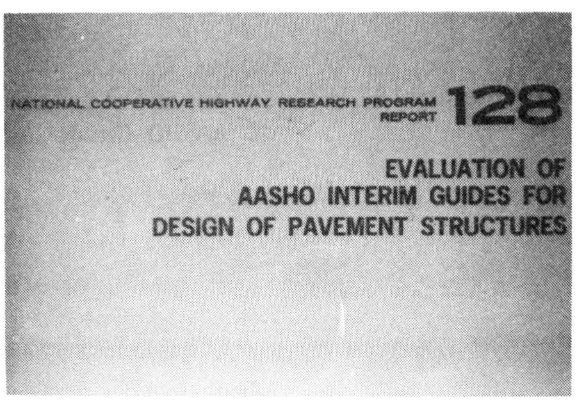
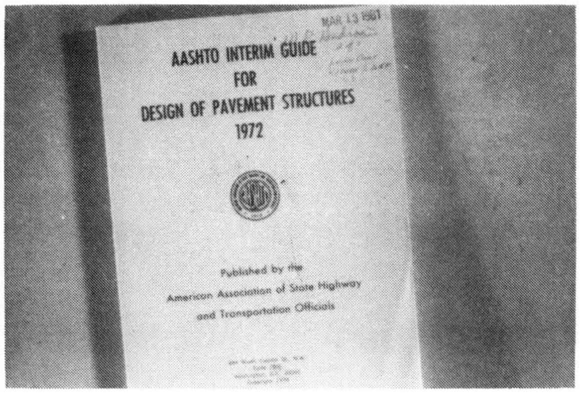
It is emphasized that the guide is interim in nature and subject to adjustment based on experience and additional research.

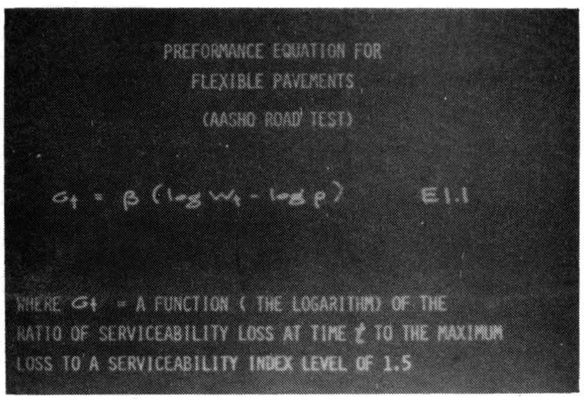
Slide 19.23. Scope and limitations of AASHO Guide (continued).



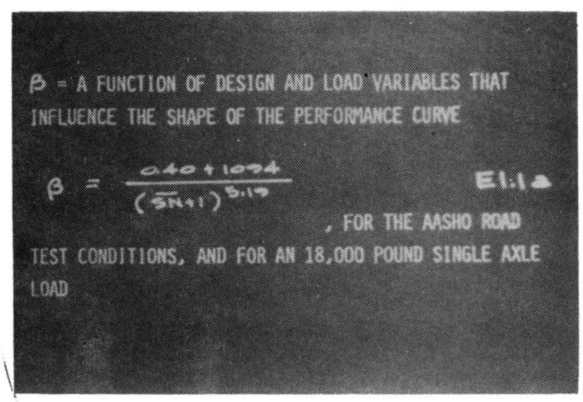
Slide 19.24. NCHRP Report 128 Evaluation of AASHO Interim Guide.



Slide 19.25. AASHTO Interim Guide, 1972.



Slide 19.26. Performance equation for flexible pavements.



Slide 19.27. Definitions of terms in the flexible pavement performance equation.

$\overline{SN}$ = STRUCTURAL NUMBER

W_t = AXLE LOAD APPLICATIONS TO TIME, t

ρ = A FUNCTION OF DESIGN AND LOAD VARIABLES DENOTING THE EXPECTED NUMBER OF AXLE LOAD APPLICATIONS TO A SERVICEABILITY INDEX OF 1.5

$$\text{LOG } \rho = 9.36 \text{ LOG } (\overline{SN} + 1) - 0.20 \qquad \text{E1.1b}$$

FOR AASHO ROAD TEST CONDITIONS, AND FOR AN 18,000 POUND SINGLE AXLE LOAD

Slide 19.28. Definitions of terms in the flexible pavement performance equation (continued).

COMBINING AND REWRITING THE NUMBER OF AXLE LOADS CARRIED CAN BE EXPRESSED AS

$$\text{LOG } W_{t_{18}} = 9.36 \text{ LOG } (\overline{SN} + 1) - 0.20 + \frac{G_t}{0.40 + \frac{1094}{(\overline{SN} + 1)^{5.19}}} \qquad \text{E1.2}$$

WHERE $W_{t_{18}}$ = NUMBER OF 18,000 POUND SINGLE AXLE LOADS TO TIME (IF EQUIVALENT AXLE LOADS ARE USED, THIS CAN BE EXPRESSED AS EAL_{18} TO TIME t)

$$G_t = \text{LOG } \left[(4.2 - P_t) / (4.2 - 1.5) \right] \qquad \text{E1.2a}$$

P_t = SERVICEABILITY INDEX AT TIME

Slide 19.29. Development of design equations.

TO EXTEND E1.2 TO OTHER SUBGRADE TYPES AND CLIMATES, IT WAS NECESSARY TO DEVELOP THE FOLLOWING:

1. SOIL SUPPORT VALUE, S, WITH A SCALE RANGING FROM 1.0 TO 10.0, WITH THE ROAD TEST SUBGRADE SOIL HAVING A VALUE OF 3.0, AND
2. REGIONAL FACTOR, R, WITH A POTENTIAL RANGE OF 0.5 TO 5.0.

Slide 19.30. Development of soil support value and regional factor.

Slide 19.31. The final design equation.

THE FINAL DESIGN EQUATION INCORPORATING SOIL SUPPORT AND REGIONAL FACTOR IS

$$\text{LOG } W_{t_{18}} = 9.36 \text{ LOG } (\overline{SN} + 1) - 0.20 + \frac{G_t}{0.40 + \frac{1094}{(\overline{SN} + 1)^{5.19}}}$$

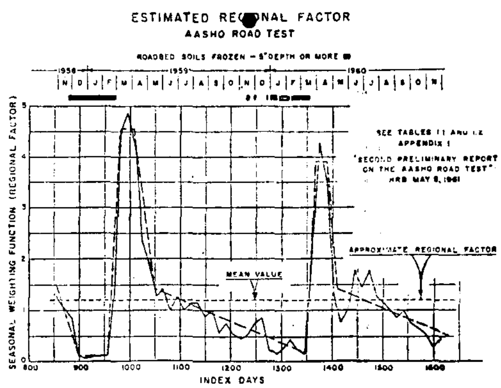
$$- \text{ LOG } R + 0.372 \; (S - 3.0) \qquad \text{E1.3}$$

WHERE R = REGIONAL FACTOR

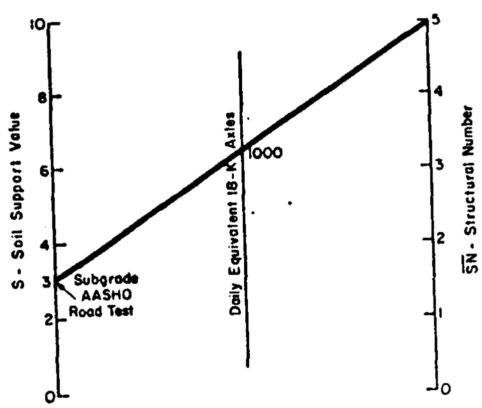
S = SOIL SUPPORT VALUE FOR THE PARTICULAR SITE AND CONDITIONS

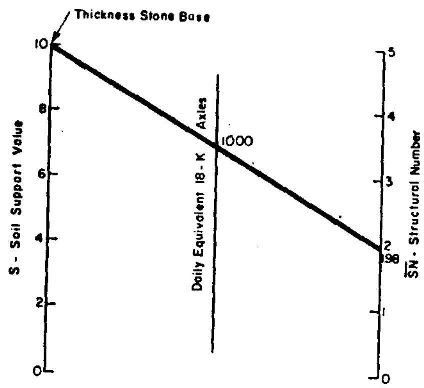
$\overline{SN}$ = WEIGHTED STRUCTURAL NUMBER (FOR THE SOIL SUPPORT AND REGIONAL FACTORS USED)

Slide 19.32. Estimated regional factor (AASHO Road Test).

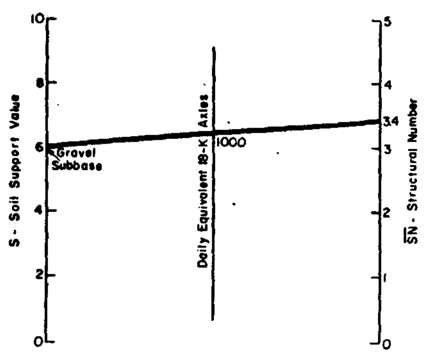


Slide 19.33. AASHO flexible pavement design nomographs (illustration of AASHO Road Test subgrade).





Slide 19.34. Use of nomograph: SN as a function of soil S-value and daily ESAL for thick stone base.



Slide 19.35. Use of nomograph: SN as a function of soil S-value and daily ESAL for gravel subbase.

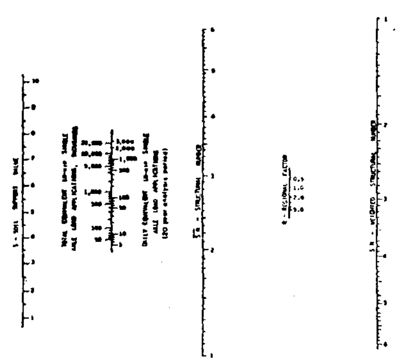
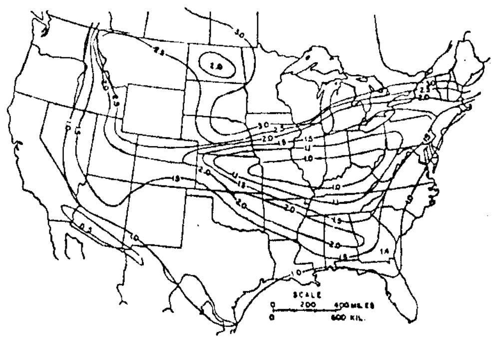
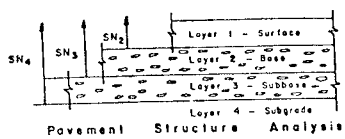


Figure II-1 Design Chart for Flexible Pavements, $p_t = 2.5$

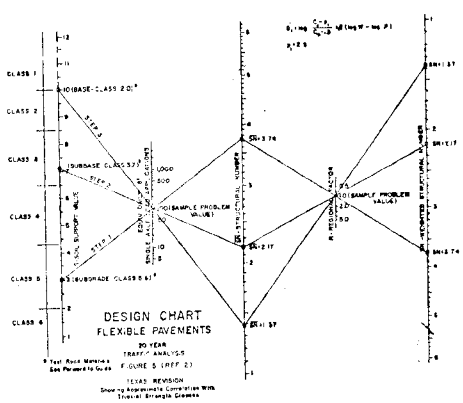
Slide 19.36. Complete AASHO Design nomograph.



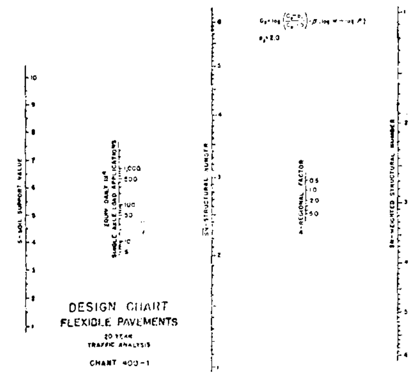
Slide 19.37. Regional factors for various regions (illustration only - not a design aid).



Slide 19.38. Application of AASHO Interim Guide.



Slide 19.39. Application of AASHO Design chart.



Slide 19.40. Design chart of flexible pavements.

LESSON OUTLINE
AASHO LAYER COEFFICIENTS

Instructional Objectives

1. To provide the student with the basic definition and use of AASHO layer coefficients and to explain the procedure used in the development of these coefficients from AASHO Road Test data.

2. To familiarize the students with the procedures used in the development of structural layer coefficients for pavement materials other than the AASHO Road Test materials. To provide the students with the charts for structural layer coefficients of asphaltic concrete surface course, base and subbase courses and layer equivalency factors.

3. To explain to students various limitations inherent in the use of layer coefficients for flexible pavement design.

Performance Objectives

1. The student should be able to understand the use of AASHO layer coefficients in the procedure for design of flexible pavement. The student should also be able to explain the concepts and procedure involved in the development of AASHO layer coefficients.

2. The student should be able to understand how structural layer coefficients and layer equivalency factors are developed for materials other than AASHO Road Test materials.

3. The student should be able to recognize the inherent limitations and errors involved in the use of layer coefficients for the design of flexible pavements.

Abbreviated Outline	Time Allocated, min.
1. Background	20
2. Layer Coefficients for Materials other than AASHO Road Test Materials	20
3. Limitations in the Use of AASHO Layer Coefficients	10
	50 minutes

Reading Assignment

1. AASHTO Interim Guide - Appendix C.

2. NCHRP 128 - Chapter 1 and 2

LESSON OUTLINE
AASHO LAYER COEFFICIENTS

1.0 BACKGROUND

1.1 Use of AASHO Layer Coefficients

AASHO Layer Coefficients are used in the design of flexible pavements following the procedure outlined in AASHTO Interim Guide for Design of pavement structures.

1.1.1 Structural Number. The design equation of AASHO Interim Guide (for flexible pavements) is solved for Structural Number, SN by using nomographs presented in the guide (Visual Aid 20.1).

1.1.2 SN Equation. The design SN value obtained from the preceding step is then used to obtain the required thickness of each layer using the relationship expressed as SN equation (Visual Aid 20.2). The layer coefficients indicate relative strength of different pavement layers.

1.2 Development of AASHO Layer Coefficients

1.2.1 AASHO Road Test Materials. The layer coefficients of AASHO Road Test materials are average values (Visual Aid 20.3).

1.2.2 Layer Coefficients of AASHO Road Test Loops. The analyses of AASHO Road Test data showed that the linear expression (Visual Aid 20.2) gave the best relationship for the thickness index (structural number). The layer coefficients a_1, a_2, a_3 are therefore the best estimates of regression coefficients of explanatory variables D_1, D_2, and D_3. (Visual Aid 20.4).

1.2.3 Relationship Between Axle Loads and Design. (Visual Aid 20.5) The average values of the regression coefficients (as shown in Visual Aid 20.4) are used as the representative layer coefficients in the relationships developed between design and axle load applications.

2.0 LAYER COEFFICIENTS FOR MATERIALS OTHER THAN AASHO ROAD TEST MATERIALS

The layer coefficients obtained from the AASHO Road Test data are representative of the relative strength of the materials used for the flexible pavement research.

2.1 Approaches

Different approaches were used by various agencies investigating the structural layer coefficients with respect to material types, properties and position in the pavement structure.

2.1.1 Theoretical Studies. Layered elastic theory was used to establish structural layer coefficient using such limiting criteria as surface deflection, tensile strain in asphaltic concrete layer and vertical compressive strain on the subgrade layer.

2.1.2 Materials Tests. Such as Marshall stability test, cohessiometer test, resilient modulus, CBR, Texas Triaxial test, etc.

2.1.3 Field Investigations and Engineering Judgement.

2.2 Structural Layer Coefficients Recommended by AASHTO Interim Guide (Visual Aid 20.6)

2.3 Structural Layer Coefficients (NCHRP 128)

Based on the experience of different agencies and highway departments, charts are presented in NCHRP report 128.

2.3.1 Layer coefficient for Asphaltic Concrete Surfacing (a_1). The nomograph was developed by using the properties of AASHO road test materials: for example the average value of Marshall stability on the Road Test, 2,000 lbs was as base for increasing or decreasing a_1. (Visual Aid 20.7).

2.3.2 Layer Coefficient for Base Material (a_2).

(a) granular base (Visual Aid 20.8),

(b) cement treated base (Visual Aid 20.9), and

(c) Bituminous treated base (Visual Aid 20.10).

2.3.3 Layer Coefficient for Subbase Material (a_3). This chart was developed for granular material (Visual Aid 20.11).

2.4 Layer Equivalency Factors

Some agencies have developed equivalent factors to convert thickness of 1 inch asphalt concrete surfacing to equivalent thickness of crushed stone base, etc. (Visual Aid 20.12).

3.0 LIMITATIONS ON THE USE OF AASHO LAYER COEFFICIENTS

3.1 AASHO Road Test Condition

The AASHO layer coefficients were developed based on AASHO Road Test data, materials and environmental conditions. AASHTO Interim Guide says" . . Careful consideration must be given by user agencies in selecting applicable coefficients . ." The layer coefficients developed by other agencies are generally based on experience, correlation studies and engineering judgment.

3.2 AASHO Road Test Pavements

The AASHO layer coefficients (Visual Aid 20.3) correspond to four layer structure of AASHO Road Test flexible pavements. This factor should be considered while applying AASHO design procedures for a 2 or 3 layer structure.

3.3 Derivation of AASHO Layer Coefficients

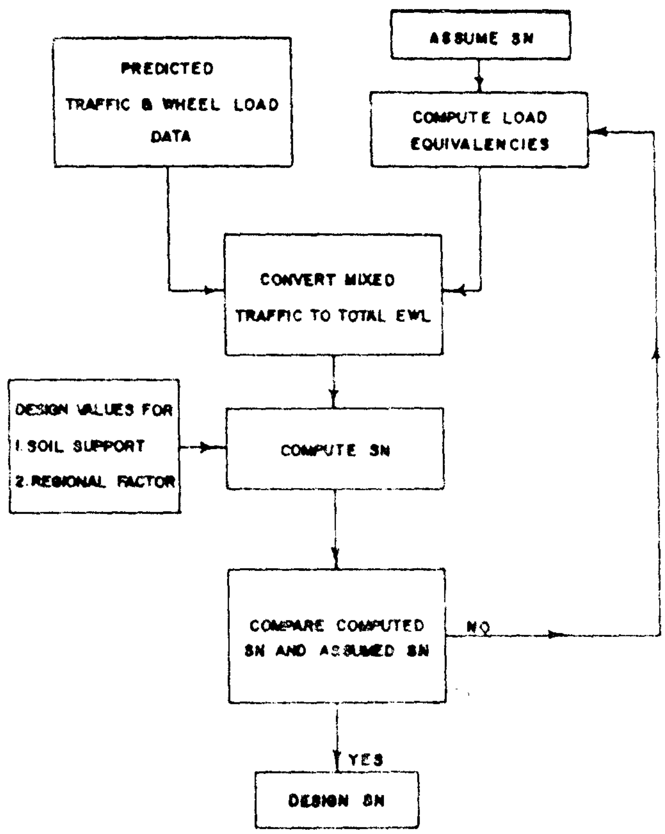
3.3.1 AASHO Layer Coefficients are "Average". As discussed earlier, AASHO layer coefficients are "average" values of the regression coefficients analyzed for all test loops. (Visual Aid 20.13).

3.3.2 The AASHO layer coefficients currently in use for AASHTO Interim Guides design procedure were derived to solve the design equation using weighted applications of axle load through the use of seasonal weighting function. If the unweighted applications are used then the corresponding "average" layer coefficients are 0.37, 0.14 and 0.10 for a_1, a_2 and a_3 respectively.

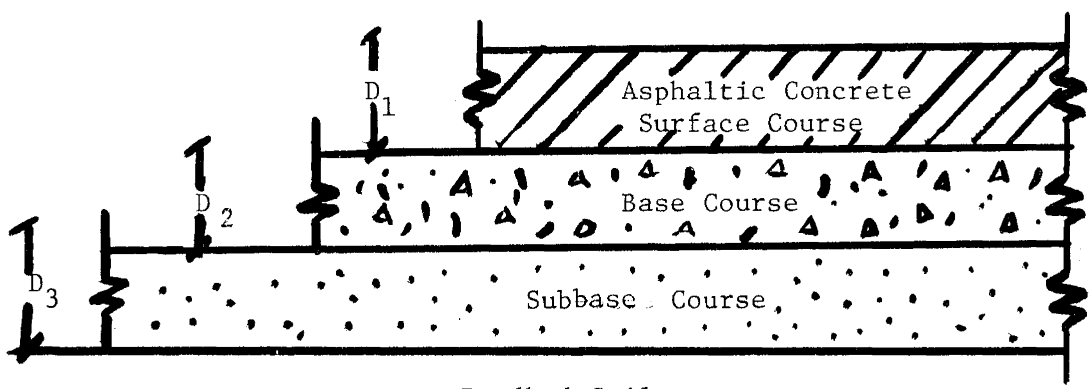
LESSON OUTLINE
AASHO LAYER COEFFICIENTS

VISUAL AID	TITLE
Visual Aid 20.1.	Iterative procedure to compute SN.
Visual Aid 20.2.	SN equation.
Visual Aid 20.3.	Layer coefficients, AASHO Road Test materials (AASHO Interim Guide 1972).
Visual Aid 20.4.	AASHO Road Test loops layer coefficients (HRB SR 61E - Report 5).
Visual Aid 20.5.	Relationship between axle load and design - AASHO Road Test data (HRB SR 61E - Report 5).
Visual Aid 20.6.	Structural layer coefficients (AASHO Interim Guide, 1972).
Visual Aid 20.7.	Variation of AC surface course coefficient (a_1) (NCHRP Report 128).
Visual Aid 20.8.	Layer coefficient (a_2) for granular base (NCHRP Report 128).
Visual Aid 20.9.	Layer coefficient (a_2) for cement - treated base (NCHRP Report 128).
Visual Aid 20.10.	Layer coefficient (a_2) for bituminous - treated base (NCHRP Report 128).
Visual Aid 20.11.	Layer coefficient (a_3) for subbase material (NCHRP Report 128).
Visual Aid 20.12.	Layer equivalency factors.
Visual Aid 20.13.	Effect of using actual range of AASHO layer coefficients.

Visual Aid 20.1. Iterative procedure to compute SN.



Visual Aid 20.2. SN equation.



$$SN = a_1D_1 + a_2D_2 + a_3D_3$$

Where: a_1, a_2, a_3 = Layer coefficient for surface, base and subbase course materials respectively.

and D_1, D_2, D_3 = Thickness of surface, base and subbase courses, respectively in inches.

Visual Aid 20.[illegible] Layer coefficients, AASHO road test materials (AASHO Interim Guide 1972).

a_1 = 0.44 asphaltic concrete surface course

a_2 = 0.14 crushed stone base course

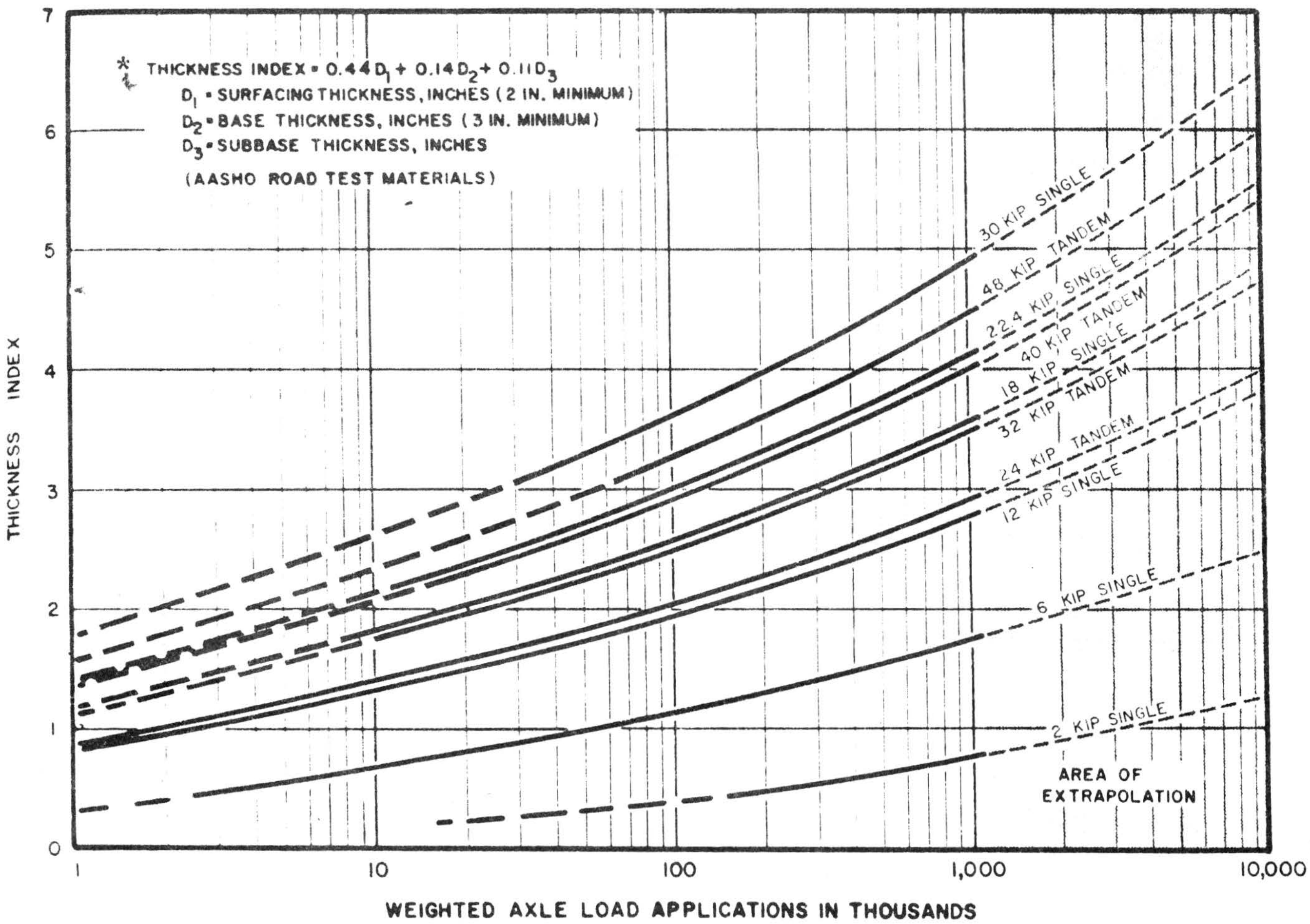
a_3 = 0.11 sandy gravel subbase course

Visual Aid 20.4. AASHO road test loops layer coefficients (HRB SR 61E - Report 5).

LAYER COEFFICIENTS

	LOOP 2	LOOP 3	LOOP 4	LOOP 5	LOOP 6	WEIGHT. AVG.
ASPHALT (a_1)	$\boxed{.83}$	.44	.44	.47	$\boxed{.33}$	.44
BASE (a_2)	.25	.16	.14	.14	.11	.14
SUBBASE (a_3)	.09	.11	.11	.11	.11	.11

Visual Aid 20.5. Relationship between axle load and design - AASHO road test data (HRB SR 61E - Report 5).



Main factorial experiment, relationship between design and axle load applications at $p = 1.5$ (from Road Test equations).

*Thickness index = SN

Visual Aid 20.6. Structural layer coefficients (HRB SR 61E - Report 5)

Pavement Component	Coefficient[3]
Surface Course	
Roadmix (low stability)	0.20
Plantmix (high stability)	0.44*
Sand Asphalt	0.40
Base Course	
Sandy Gravel	0.07[2]
Crushed Stone	0.14*
Cement-Treated (no soil-cement)	
Compressive strength @ 7 days	
650 psi or more[1] (4.48MPa)	0.23[2]
400 to 650 psi (2.76 to 4.48MPa)	0.20
400 psi or less (2.76MPa)	0.15
Bituminous-Treated	
Coarse-Graded	0.34[2]
Sand Asphalt	0.30
Lime-Treated	0.15-0.30
Subbase Course	
Sandy Gravel	0.11*
Sand or Sandy-Clay	0.05-0.10

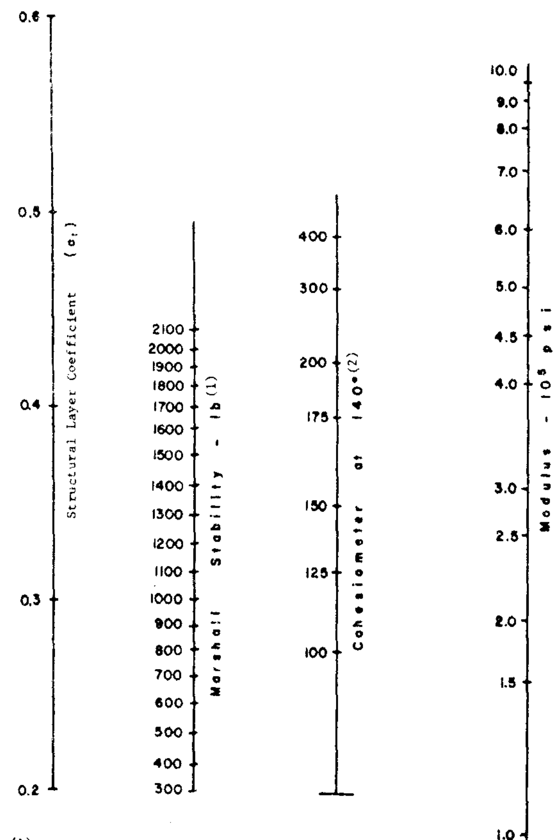
* Established from AASHO Road Test Data

[1] Compressive strength at 7 days.

[2] This value has been estimated from AASHO Road Test data, but not to the accuracy of those factors marked with an asterisk.

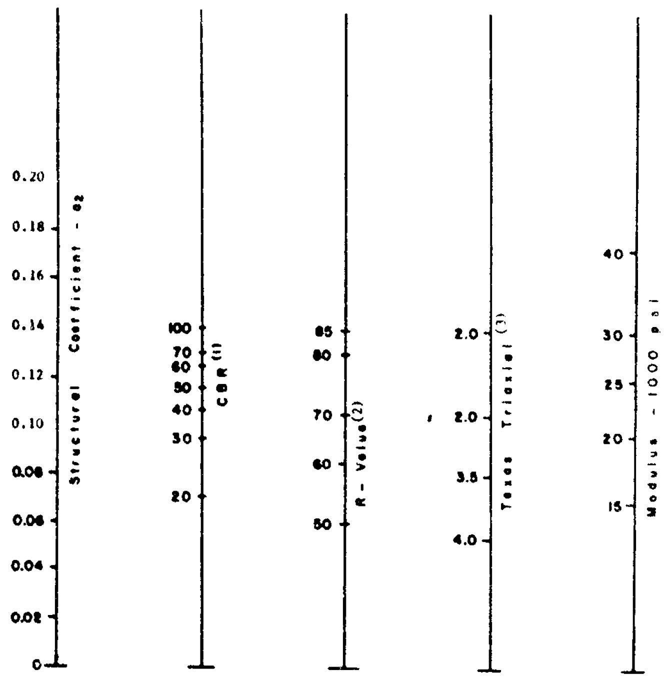
[3] It is expected that each state will study these coefficients and make such changes as experience indicates necessary.

Visual Aid 20.7. Variation of A.C. surface course coefficient (a_1) (NCHRP Report 128).



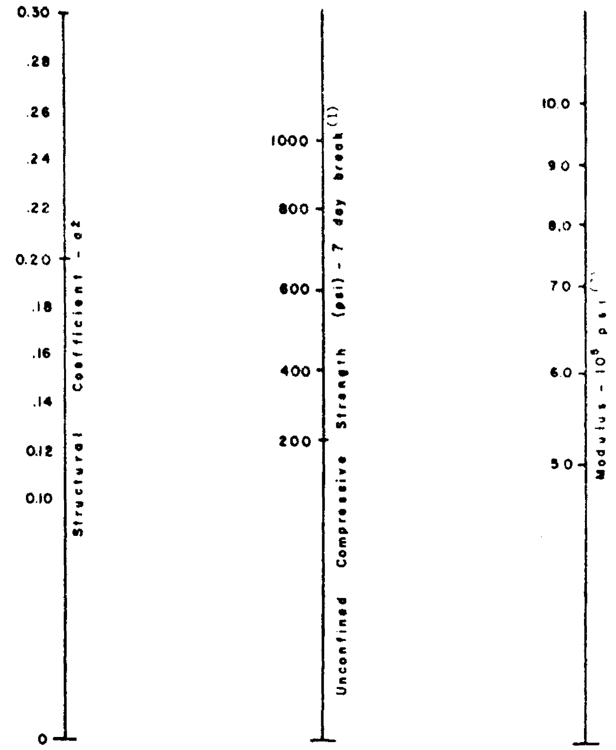
(1) Scale derived by averaging correlations obtained from The Asphalt Institute, Illinois, Louisiana, New Mexico, and Wyoming.
(2) Scale derived by averaging correlations obtained from California and Texas.

Visual Aid 20.8. Layer coefficient (a_2) for granular base (NCHRP Report 128).



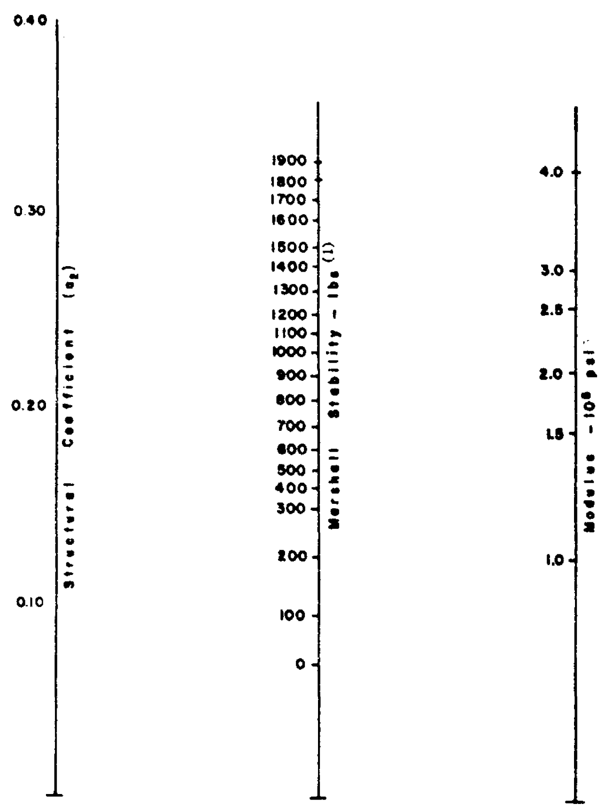
(1) Scale derived by averaging correlations obtained from Illinois.
(2) Scale derived by averaging correlations obtained from California, New Mexico, and Wyoming.
(3) Scale derived by averaging correlations obtained from Texas.

Visual Aid 20.9. Layer coefficient (9_2) for cement - treated base (NCHRP Report 128).



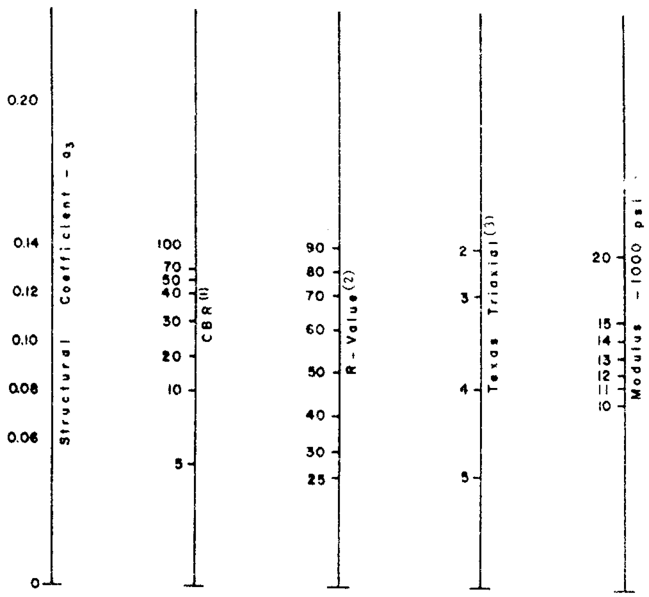
(1) Scale derived by averaging correlations from Illinois, Louisiana, and Texas.

Visual Aid 20.10. Layer coefficient (a_2) for Bituminous - treated base (NCHRP Report 128).



(1) Scale derived by correlation obtained from Illinois.

Visual Aid 20.11. Layer coefficient (a_3) for subbase material (NCHRP Report 128).



(1) Scale derived from correlations from Illinois.
(2) Scale derived from correlations obtained from The Asphalt Institute, California, New Mexico, and Wyoming.
(3) Scale derived from correlations obtained from Texas.

Visual Aid 20.12. Layer equivalency factors.

A. Asphalt Institute (Manual Series MS-1)

Base Quality	Thickness of Untreated Base Layer for each 1.0 inch of AC layer
i) High Quality (Minimum CBR = 100%)	2.0
ii) Low Quality (Minimum CBR - 20%)	2.7

B. California Division of Highways (NCHRP Report 128)

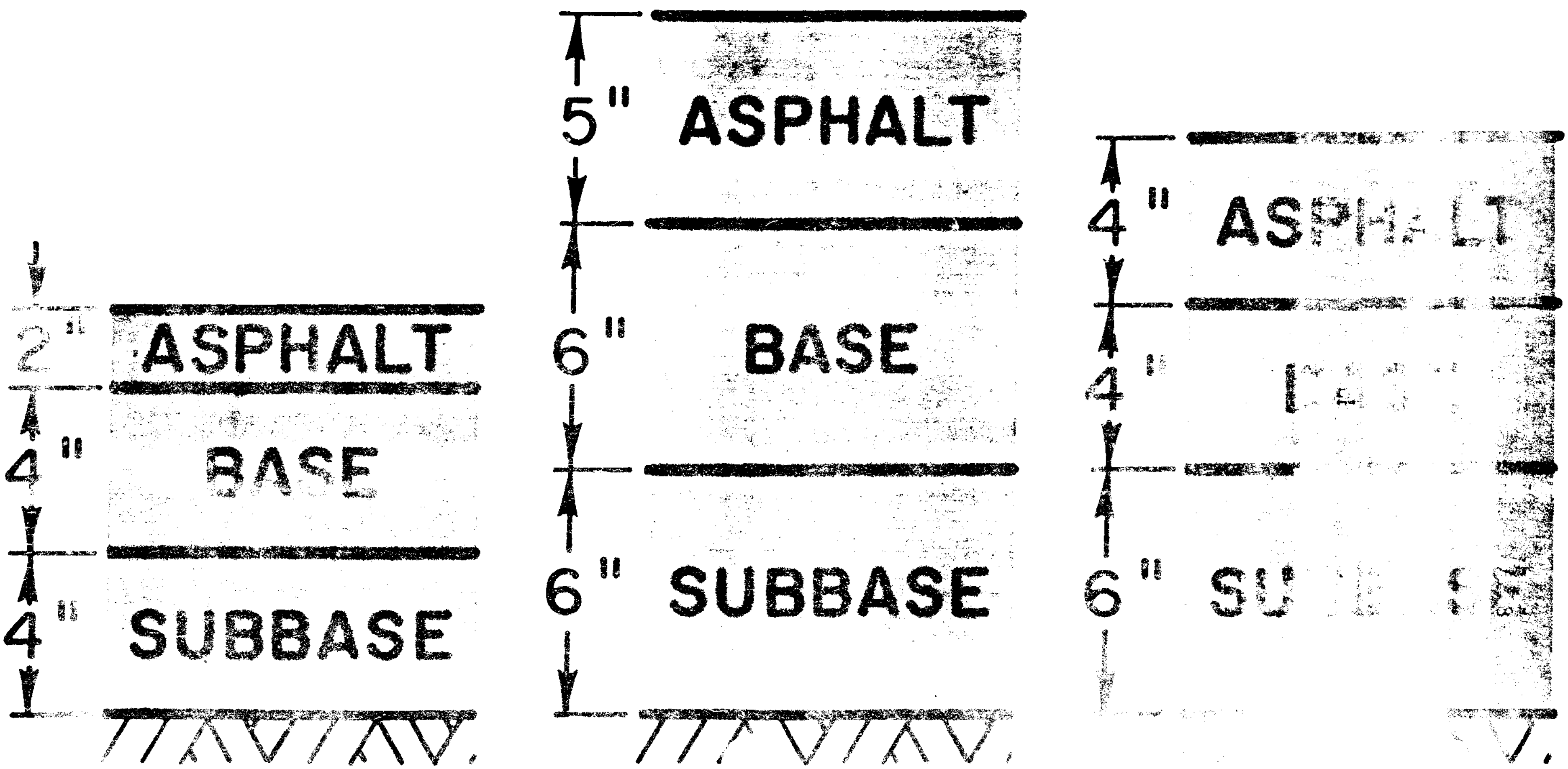
PROPOSED EQUIVALENCIES FOR BITUMINOUS MATERIALS (Thickness of Gravel Layer Required to Equal 1 In. of Asphaltic Concrete)

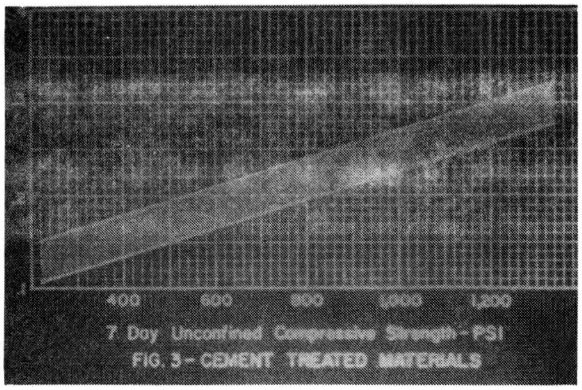
	GRAVEL EQUIVALENCY (IN.)		
ROAD CLASS	TRAFFIC INDEX RANGE	AASHO MATERIAL	CALIF. MATERIAL
Heavy industrial	12	2.0	1.6
	11	2.1	1.7
Heavy truck traffic	10	2.2	1.8
	9	2.3	1.9
Medium truck traffic	8	2.4	2.0
	7	2.6	2.1
Light truck traffic	6	2.8	2.3
Residential streets	5	3.0	2.5
	4	3.0	2.5

Visual Aid 20.13. Effect of Using Actual Range of AASHO Layer Coefficients.

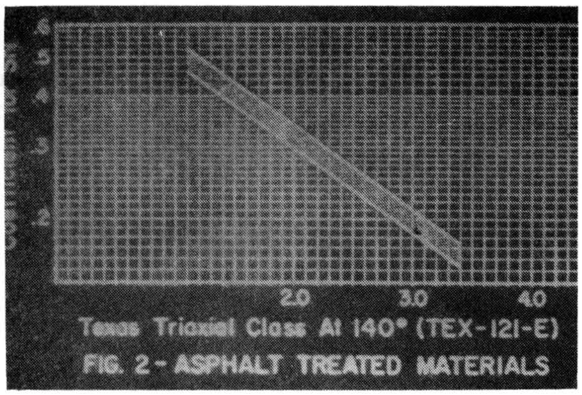
EXAMPLE: REQUIRED SN = 3.0

LOOP 2	LOOP 6	AVERAGE
$a_1 = .83$	$a_1 = .33$	$a_1 = .44$
$a_2 = .25$	$a_2 = .11$	$a_2 = .14$
$a_3 = .09$	$a_3 = .11$	$a_3 = .11$

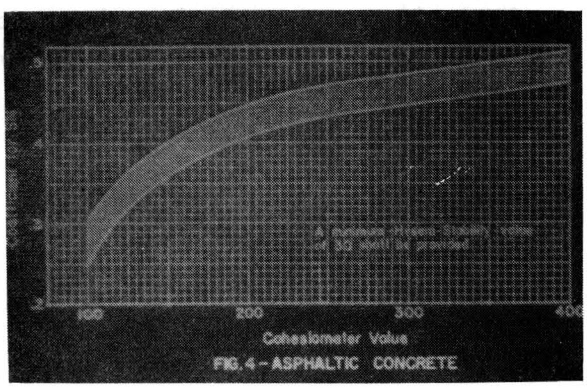




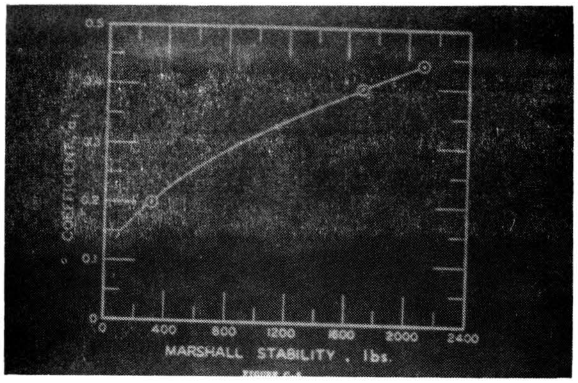
Slide 20.1. Development of layer coefficients for cement treated materials.



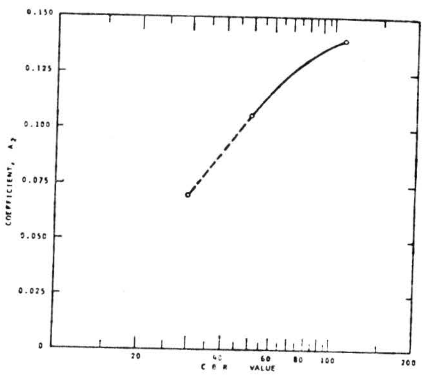
Slide 20.2. Development of layer coefficients for asphalt treated material.



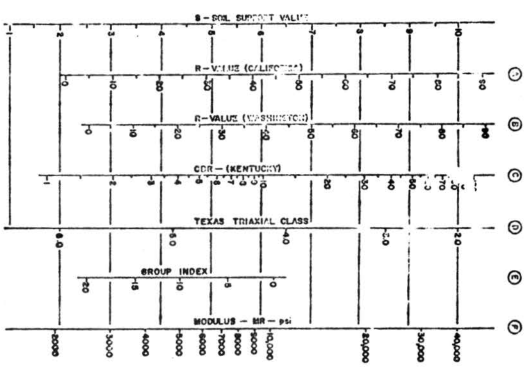
Slide 20.3. Development of layer coefficients for asphaltic concrete based on cohesiometer values.



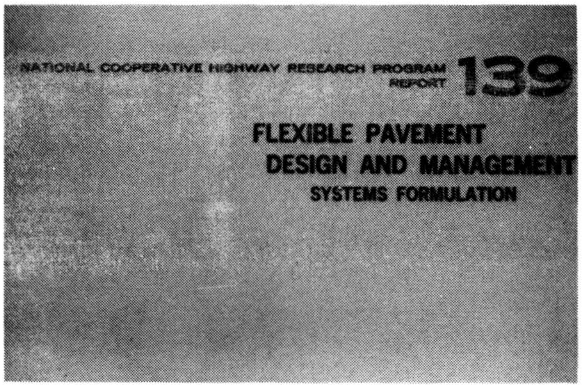
Slide 20.4. Development of layer coefficients for asphaltic concrete based on Marshall stability.



Slide 20.5. Development of layer coefficients based on CBR values.



Slide 20.6. Correlation chart for soil support values.



Slide 20.7. NCHRP Report 139 flexible pavement design and management.